Attachment A
Applicant's Project Description and Clarifications

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APPLICANT'S PROJECT DESCRIPTION AND CLARIFICATIONS INDEX

A.1. *Chehalis River Basin Flood Damage Reduction Project Description.* Chehalis River Basin Flood Control Zone District. September 2018.

- *Chehalis Basin Strategy: Combined Dam and Fish Passage Design Conceptual Report.* June 2017.
- *Chehalis Basin Strategy: Fish Passage CHTR Preliminary Design Report*. February 2018.

A.2. *Chehalis River Basin Flood Control Combined Dam and Fish Passage Supplemental Design Report, FRE Dam Alternative.* September 2018.

A.3. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility - Project Purpose and Need Clarification. November 30, 2018.

A.4. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility - Project Purpose and Need Clarification. January 11, 2019.

A.5. Letter from Betsy Dillin (Chehalis River Basin Flood Control Zone District) to Diane Butorac (Washington Department of Ecology) and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Flood Control Zone District Project Description Clarification. January 14, 2019.

A.6. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility - Project Alternatives History and Alternative Selection. February 12, 2019.

A.7. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility - Project Alternatives History and Alternative Selection. March 1, 2019.

A.8. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility - Project Alternatives History and Alternative Selection. March 7, 2019.

A.9. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility - Project Alternatives History and Alternative Selection. March 15, 2019.

A.10. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility - Project Alternatives History and Alternative Selection. March 19, 2019.

A.11. Email from Betsy Dillin to Diane Butorac (Washington Department of Ecology) and Janelle Leeson (U.S. Army Corps of Engineers). Regarding: Additional Construction information for the Dam and Levee. April 16, 2019.

A.12. Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Bob Thomas and Evan Carnes (U.S. Army Corps of Engineers). Regarding: Chehalis River Basin Water Retention Facility and Levee Improvements - Project Need, Purpose, and Description. May 7, 2019.

A.13 Email from Betsy Dillin to Diane Butorac (Washington Department of Ecology) and James R. Thomas (U.S. Army Corps of Engineers). Regarding: Access Road Clarification. June 24, 2019.

A.14 Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Diane Butorac (Washington Department of Ecology) and Bob Thomas and Brandon Clinton (U.S. Army Corps of Engineers). Regarding: Construction Schedule Supplemental Information. September 18, 2019.

A.15 Email from Betsy Dillin to Diane Butorac (Washington Department of Ecology). Regarding: Levee Trail and Pe Ell Roads. October 29, 2019.

A.16 Letter from Erik Martin, PE (Chehalis River Basin Flood Control Zone District) to Diane Butorac (Washington Department of Ecology) and Brandon Clinton (U.S. Army Corps of Engineers). Regarding: Airport Levee Design Update. November 22, 2019.

A.17 Email from Betsy Dillin to Diane Butorac (Washington Department of Ecology). Regarding: Minor clarifications of Project Description in SEPA EIS. January 27, 2020.

A.1. Chehalis River Basin Flood Damage Reduction Project Description. Chehalis River Basin Flood Control Zone District. September 2018.

Chehalis River Basin Flood Control Zone District

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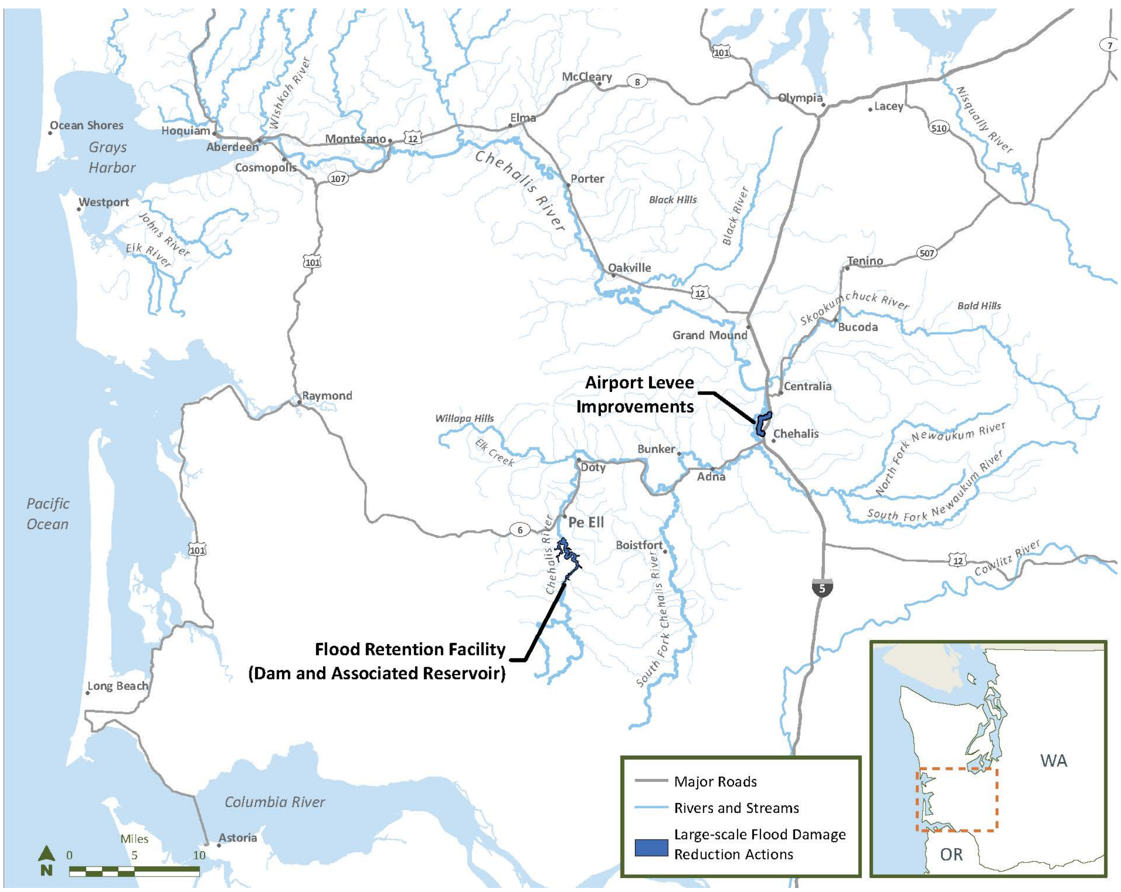
Chehalis River Basin Flood Damage Reduction Project

Project Description

Occasional catastrophic flood damage from the Chehalis River devastates homes, farms, businesses, churches, and schools. It also freezes transportation in much of Southwest Washington when I-5 and Highways 6 and 12 are closed.

The Chehalis River Basin Flood Control Zone District (FCZD) is proposing to construct a flood retention facility, or dam, near Pe Ell, Washington, and levee improvements around the Centralia-Chehalis Airport in Chehalis, Washington, to reduce flood damage during a major or catastrophic flood (see Figure 1, Vicinity Map below). This document provides a description of FCZD's proposal to inform the public scoping process for the separate federal and state Environmental Impact Statements.

Figure 1
Vicinity Map



The proposed facilities are intended to substantially reduce damages during a major or catastrophic flood. The amount of flood risk reduction will vary throughout the basin. Previous studies and research have predicted that the project will:

1. Reduce of the closure due to overtopping of Interstate 5 freeway to 24 hours or less during a 100-year flood event.
2. Reduce damage from major flooding along the Chehalis River main stem. Hydraulic analysis shows that 100-year flood peak levels will be lowered by 10 feet or more at the Doty gauge, and by 1 foot or more at the Mellen Street gauge in Centralia. This level of reduction in flood levels translates to a substantial decrease in the severity of flooding on more than 4,000 acres as well as substantial relief from the more than $900 million of economic impacts estimated to occur during a major flood event.
3. Provide future leaders in the Chehalis Basin the flexibility to address additional increases in peak flood levels and decreases in stream flow during summer months through an adaptable design approach.

The project would have significant reductions to flood risk, however it will not protect communities from all flooding, nor is it designed to stop regular, small-scale annual flooding from the Chehalis River. Flood protection provided by the facilities would not result in immediate changes to Federal Emergency Management Agency (FEMA) flood hazard mapping; however, FEMA mapping updates would continue to occur. This large scale flood project is also not intended to supplant the need for smaller local flooding projects, such as flood proofing or farm pads. Projects constructed within the existing floodplain will continue to follow floodplain development regulations.

The temporary reservoir associated with the dam would be present only during major flooding. The water in the reservoir would be released as soon as it is safe to do so after the flood event, and is therefore considered to be temporary. At all other times, the river will flow through the dam's low level outlet works at its normal rate of flow and volume and allow fish passage both upstream and downstream. This system will achieve dual goals of flood damage reduction benefit while having minimal, if any, impacts on normal streamflow in the Chehalis River.

A "major flood" along the Chehalis River is the level at which flooding in Lewis County results in road closures and floodwaters encroach on some homes and businesses. In addition, major flooding in Thurston County results in the inundation of farmlands and roads, including U.S. 12. The threshold for a major flood is defined as 38,800 cubic feet per second (cfs) at the Grand Mound gage located along the Chehalis River in Thurston County. This flood has about a 15% probability of occurrence in any year (or a 7-year recurrence interval). Major floods include events greater than 38,800 cfs with a lower frequency of occurrence such as 10-year, 100-year, and 500-year floods (10%, 1%, and 0.02% probability of occurrence in any year).

The type of dam that has been selected for EIS analysis is known as a Flood Retention Expandable (FRE) facility, which consists of a dam with a temporary reservoir. The FRE dam would temporarily retain water in the event a major flood as previously described. The river would flow normally during regular conditions or in smaller floods. The dam would only transition to flood retention operations during a major flood. Specific flow release operations would depend on inflow and the need to hold water to relieve downstream flooding as flood waters recede.

The FRE dam is considered to be expandable because it is proposed to be built with a foundation and hydraulic structure capable of supporting future construction of a larger dam with up to 130,000 acre feet of storage. This project, which may or may not occur, would be subject to a separate NEPA and SEPA process and permitting if pursued in the future.

Conceptual FRE plan views and cross-section views are provided in Figures 2 through 4.

Figure 2
FRE Site Plan View

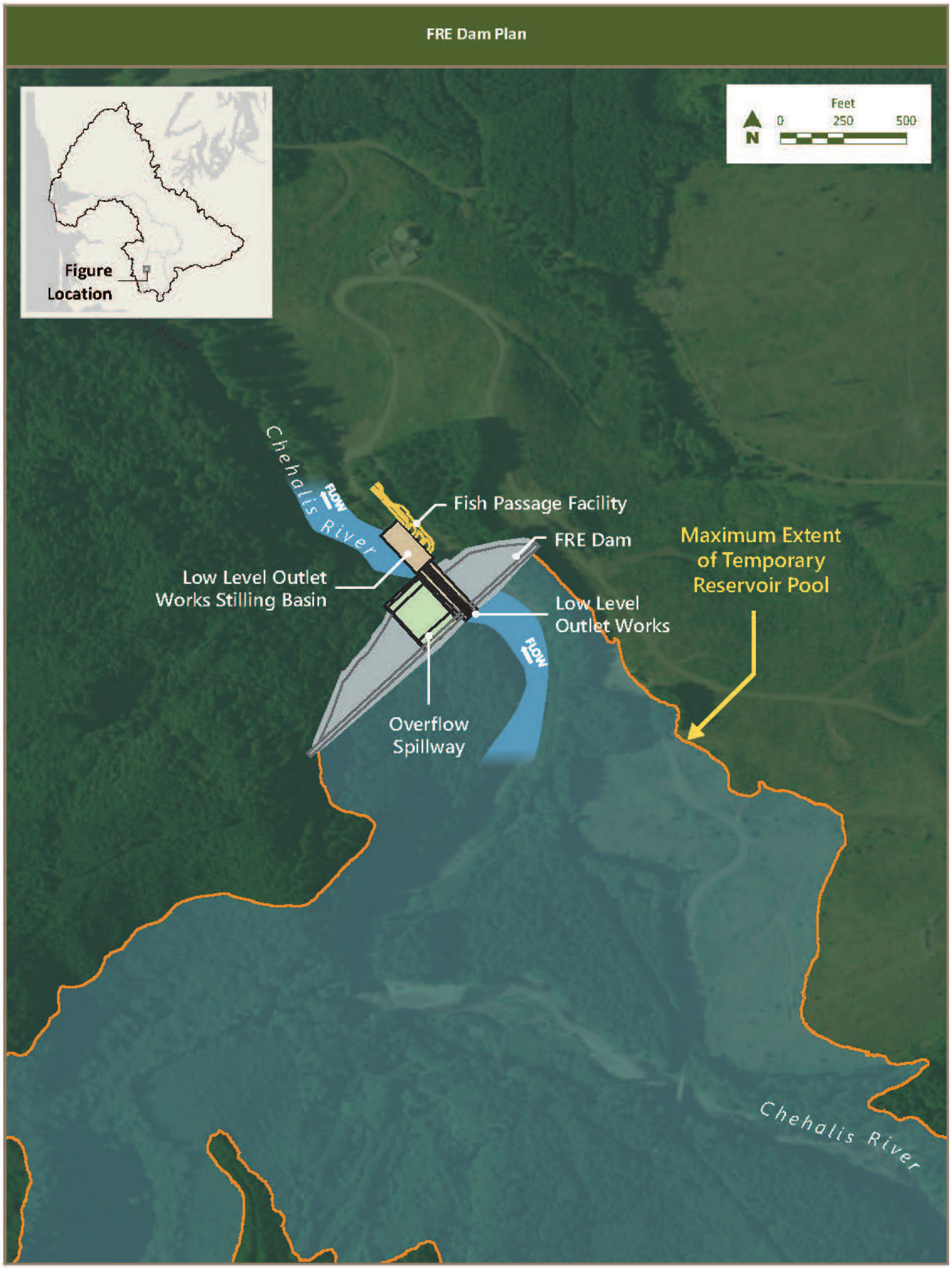


Figure 3

FRE Dam Plan View

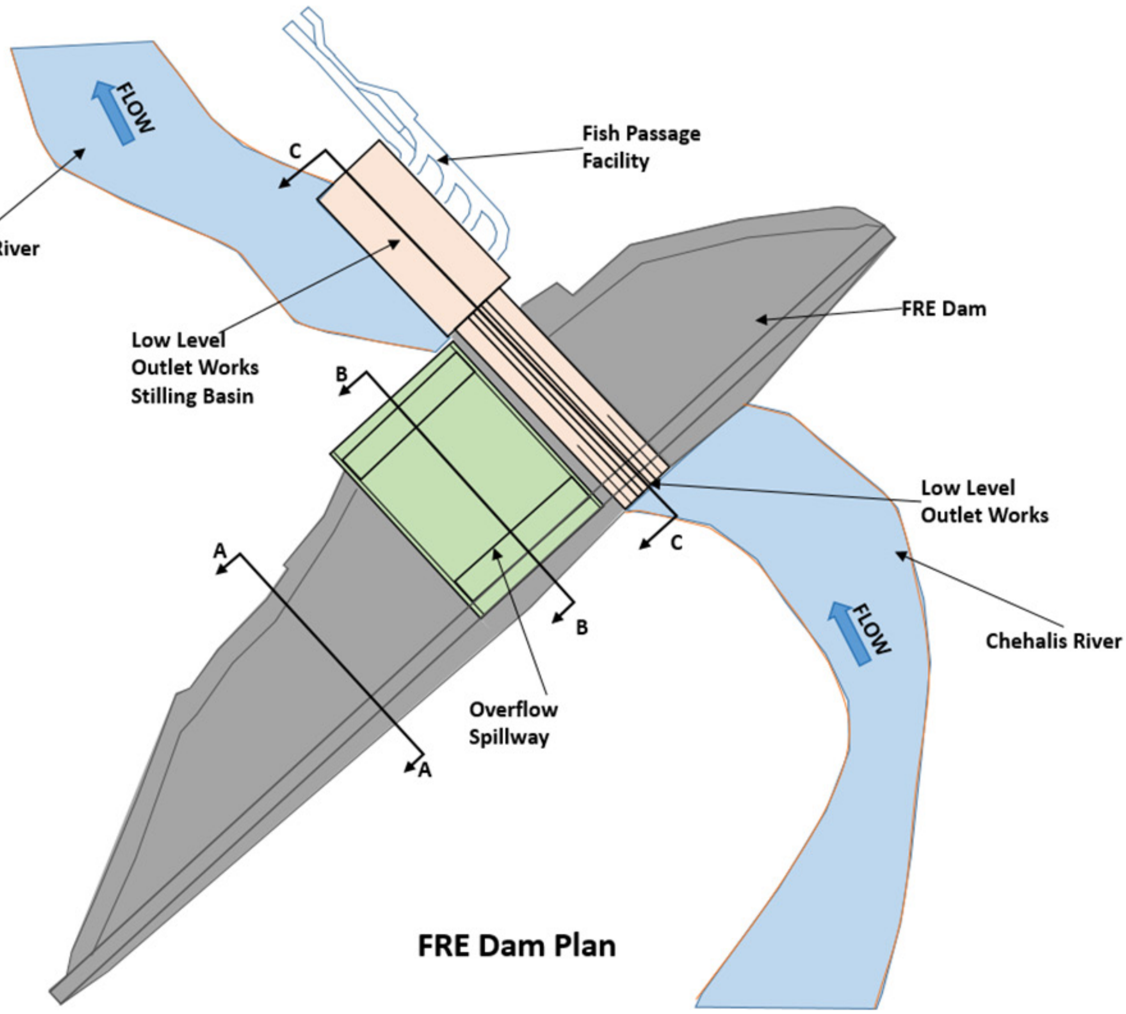
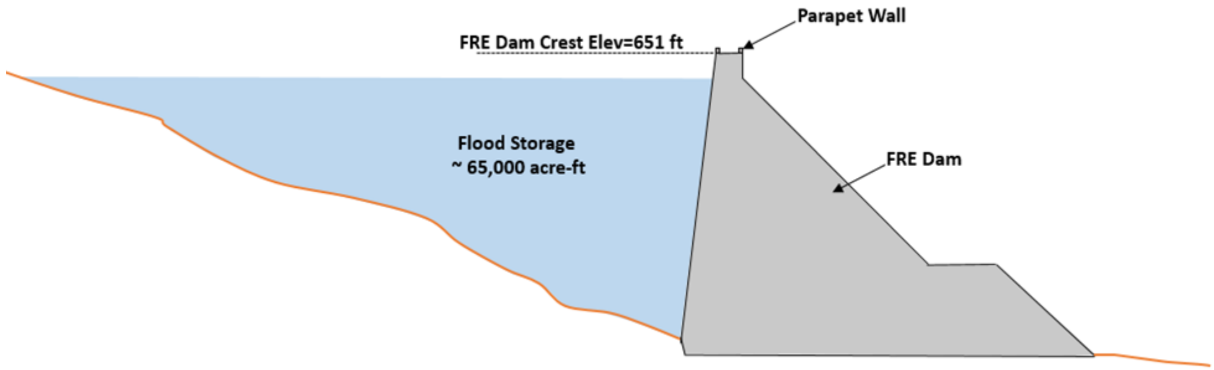
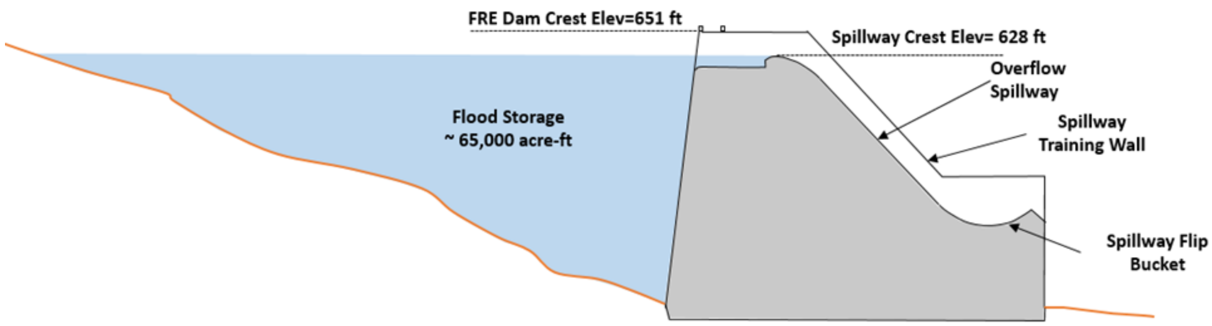


Figure 4

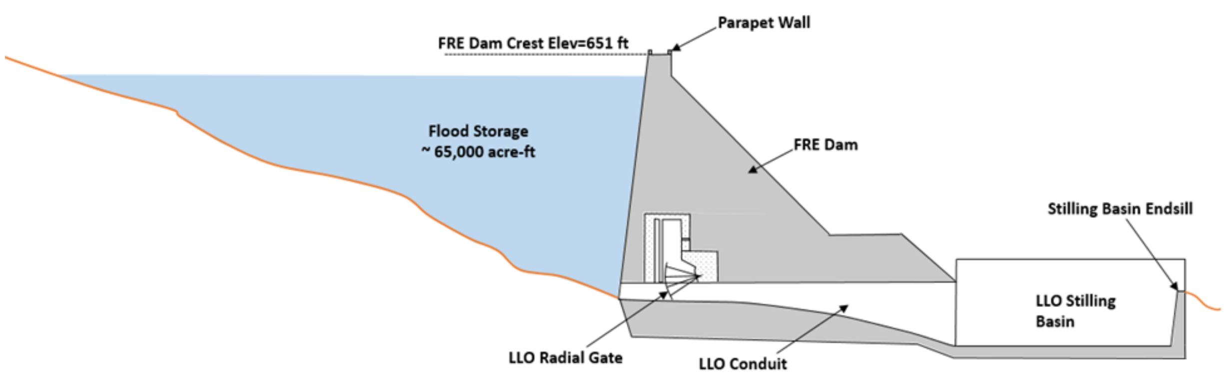
FRE Section Views



Section A-A
FRE Dam Non-Overflow Section



Section B-B
FRE Dam Overflow Spillway



Section C-C
FRE Dam Low Level Outlet Works (LLO)

The dam is not intended to result in any residential or community development at or around the reservoir. Creation of a temporary reservoir is not intended to encourage development because it would be contrary to the Chehalis Basin Strategy objectives of reducing flood damage to properties, minimizing threats to human safety from floods, and protecting and restoring aquatic species habitat.

Construction Considerations

A summary of construction and operational considerations for the FRE is described below. For more information, refer to the Combined Dam and Fish Passage Design Conceptual Report (July 2017) and Supplemental Design Report for the FRE Dam Alternative (September 2018) available at http://chehalisbasinstrategy.com/publications/.

Infrastructure

Construction activities would necessitate a detour or bypass road for Forest Road (FR) 1000, which is a main access road for Weyerhaeuser forestry operations. The FR 1000 bypass or detour would also be needed during flood conditions when the dam is in operation and FR 1000 is inundated. Up to 6 miles of FR 1000 would be inundated and unavailable during major peak flood retention, at which time a detour could be used consisting of FR A-line, FR F line, and FR 2000 to rejoin FR 1000 upstream of the reservoir.

Construction of the FRE dam would include development of a quarry site, material storage, material processing, and areas for construction offices and equipment storage. Concrete aggregate could be mined within the FRE facility site or nearby, depending on aggregate availability and a concrete batch plant would be located nearby to produce concrete. Three potential quarry sites have been identified; the most promising is within the reservoir inundation area approximately 2 miles from the potential dam location and accessed from Forest Road (FR) 1000. Material from the quarry site would be crushed and processed for use in the dam and other structures. A concrete production facility would include both roller compacted concrete (RCC) and conventional concrete production. It would be located above and northeast of the dam. The location was chosen based on access for transport of materials to the site and to allow efficient transport of the RCC to the dam. The site would include the following:

- RCC batch plant
- Conventional concrete batch plant
- Aggregate crushing and screening
- Aggregate storage
- Fly ash storage

The dam would be constructed with roller-compacted concrete, which is more cost-effective than other types of construction methods, and would be designed to retain a flow volume similar to the 2007 flood. A new power line would be needed for the construction and operation of the dam to power pumps, gates, instruments, and other controls. The alignment for new power lines would be selected to avoid and minimize impacts, including using existing local transmission lines and locating the line along areas cleared for dam construction.

Temporary Construction Flow Diversion and Fish Passage

A 20-foot modified horseshoe-shaped tunnel would carry water past the construction site. An upstream cofferdam would direct upstream water into the diversion tunnel. A much smaller downstream cofferdam would be constructed to protect the construction area for the stilling basin and fish collection channel.

The temporary diversion tunnel would accommodate fish passage during construction of the dam, and permanent fish passage facilities would be constructed and operated with the dam. Fish passage facility designs are currently conceptual in nature. The most conservative guidance for fish passage and protection was followed, and the following documents provided the engineering design guidelines used during conceptual design:

- *Anadromous Salmonid Passage Facility Design* (NMFS 2011)
- *Best Management Practices to Minimize Adverse Effects to Pacific Lamprey* (Entosphenus tridentatus; USFWS 2010)
- *Draft Fish Protection Screen Guidelines for Washington State* (Nordlund and Bates 2000)
- *Draft Fishway Guidelines for Washington State* (WDFW 2000)
- *Water Crossing Design Guidelines* (Barnard 2013)

Vegetation Management

In addition to removal of vegetation for the dam structure, tree clearing and vegetation removal would occur within the reservoir area; details have been provided in a Pre-construction Vegetation Management Plan (Appendix J to the Programmatic EIS). Goals of the Pre-construction Vegetation Management Plan include reducing the extent of tree clearing and vegetation removal in the reservoir footprint and reducing the amount of woody material that would accumulate in the reservoir during a flood. The FCZD is working to improve the Vegetation Management Plan to address the long-term vegetation management as part of the maintenance and operation of the dam. It is expected that a very conservative approach will be studied as part of the Project EIS, and the final Plan will have fewer impacts than what is being currently assumed.

Permanent Structure

The top of the dam structure would be 1,220 feet long. The maximum structural height of the dam is estimated to be up to 254 feet, including 3 to 5 feet of freeboard as a factor of safety. The dam includes a 210-foot-wide emergency spillway, which would discharge into a 70-foot-wide and 230-foot-long stilling basin. The stilling basin would allow for containment and control of all flows over the emergency spillway. The spillway crest elevation (628 feet) would be above the maximum estimated reservoir flood pool elevation for a 100-year flood. The spillway is expected to be used very rarely, and for events of very short duration. A flip bucket would be constructed to launch the spillway flow a safe distance downstream of the dam and to dissipate the energy in the river channel. Upstream of the dam, an anchored log boom would help contain large woody material (LWM). At the dam, steel bar racks would protect the river opening entrances from LWM that could not pass through the low-level outlet works downstream.

Table 1 provides a summary of the potential changes to surface water quantity at and above the dam, and the inundation extent is provided on Figure 5.

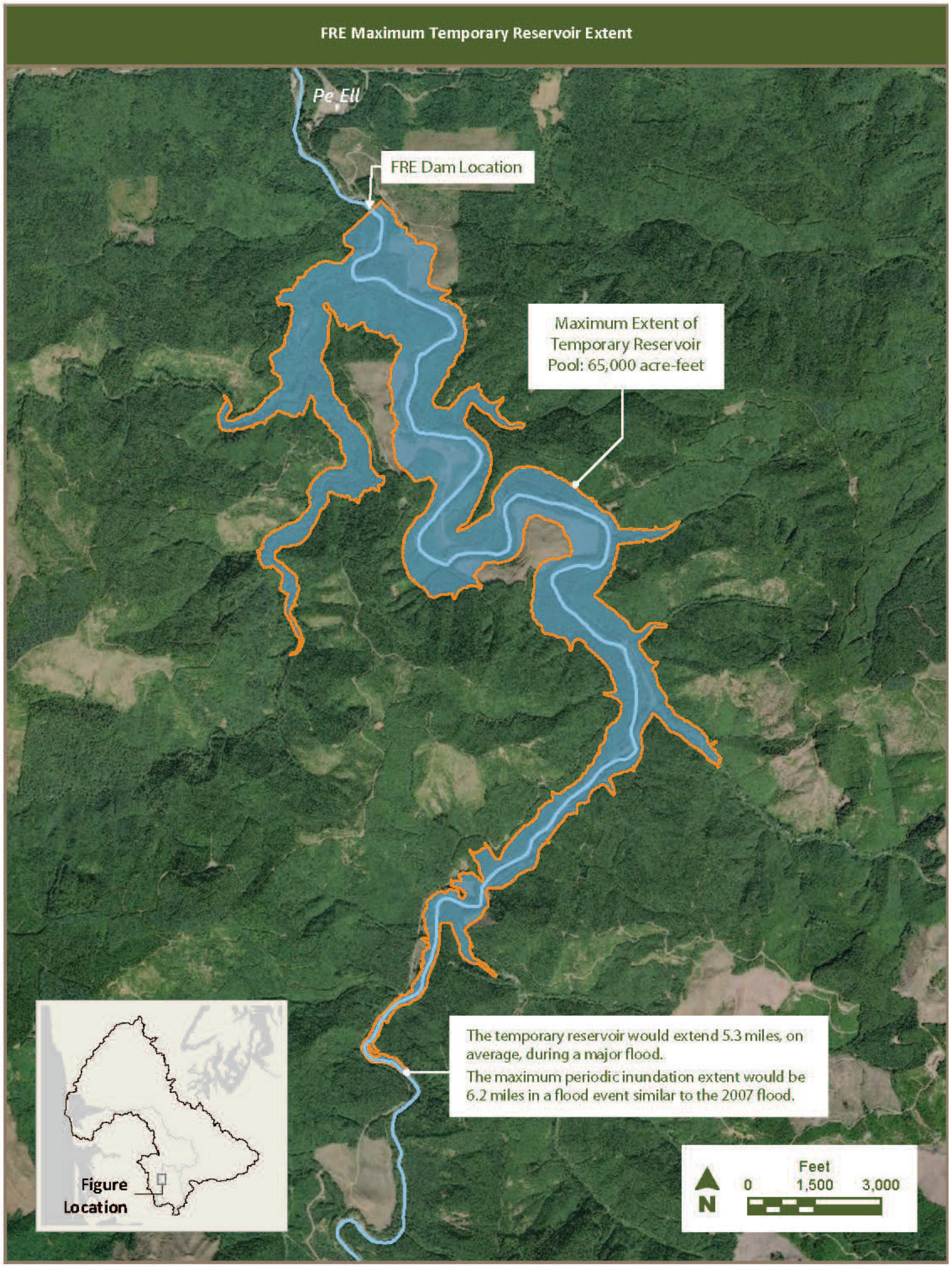
Table 1

FRE Temporary Reservoir Conditions for Surface Water Quantity

CONDITIONS	DURING FLOOD OPERATIONS [1]	MAXIMUM PERIODIC OPERATIONS [2]	COMMENT
Reservoir permanency	Reservoir inundation upstream of the FRE dam would be temporary (up to 32 days)	Up to 32 days	
Inundation extent	Temporary reservoir would extend 5.3 miles, on average	6.2 miles	
Inundated area	188 acres (median)	778 acres	
Reservoir elevation	513 feet (median)	620 feet	Elevation of the river bed at the proposed dam site is 420 feet
Maximum design reservoir elevation	628 feet		Design reservoir elevation is defined as the invert of the spillway
Reservoir depth	88 feet	195 feet	
Maximum design reservoir depth	203 feet		
Capacity	65,000 acre-feet		Capacity is defined as from the base of the dam to the invert of the spillway

1. Major is any time flood retention occurs. That threshold is measured at Grand Mound and corresponds to emergency management declarations. “Major Flood” has flow of 38,800 cfs at Grand Mound
2. To account for a flood similar to the 2007 flood

Figure 5
FRE Reservoir Extent



Permanent Fish Passage

Fish passage would be provided primarily through five openings installed along the river bottom at the base of the dam. During construction, a river bypass tunnel would be constructed for use until the dam openings are operational. The dam outlet openings would be 230 feet in length. They are anticipated to replicate the stream discharge and velocity rating curves exhibited by the natural channel at the dam site (through which fish currently pass without the dam), up through river discharges of 4,000 cfs.

The primary means of upstream and downstream fish passage at the dam is via the low-level outlet openings. However, when water is impounded behind the dam during high-flow events, the low-level outlet would be closed. Fish passage would be provided via a collection, handling, transport, and release (CHTR) facility during the high-flow, short-term periods of time when the dam outlets are closed. The CHTR facility is also commonly referred to as a trap-and-haul facility. The CHTR would be operated as needed, which is anticipated to be approximately 30 days after a major flood event, while the reservoir is being drawn down. The CHTR would consist of a short fish ladder, a fish lift, holding galleries, sorting stations, and transportation via trucks to release sites upstream of the reservoir. A detailed description of the need, research, methods, and physical design can be found in the CHTR Preliminary Design Report, available at http://chehalisbasinstrategy.com/publications/.

Location

The proposed dam would be located on Weyerhaeuser and Panesko Tree Farm property, south of State Route (SR) 6 in Lewis County, on the main-stem Chehalis River at approximate River Mile 108, about 1 mile south of (upstream of) the Town of Pe Ell. The legal description of the property is: Section 03 Township 12N Range 05W Gov Lot 13 Pt Gov Lot 14 W2 SW & SE SW EX RD, and the parcel number is 016392004000.

Property acquisition within the dam and reservoir footprint would be required, and the land would no longer be managed as commercial forestland.

The watershed area upstream of the dam is 68.9 square miles.

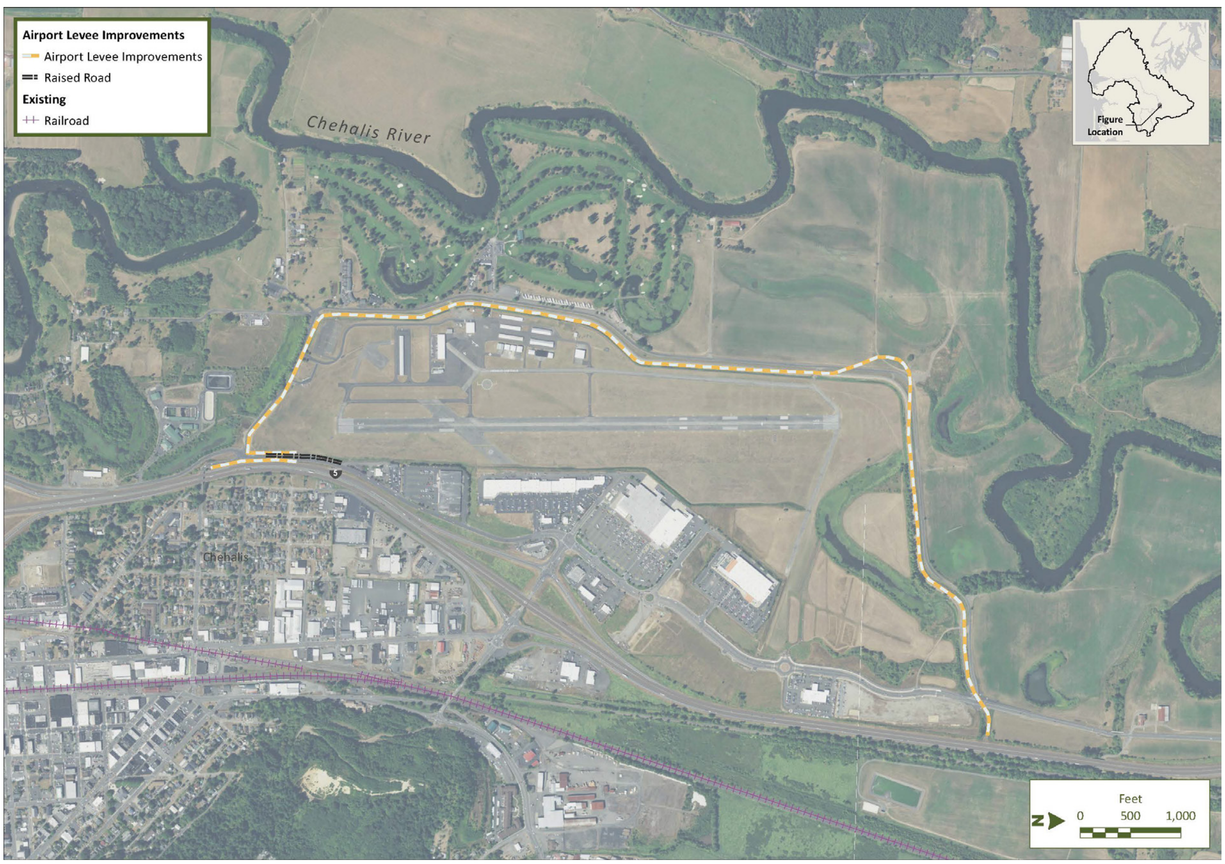
Project Description – Airport Levee Improvements

The Airport Levee Improvements include raising the existing levee around the Centralia-Chehalis Airport as well as a portion of Airport Road, to provide protection from 100-year flood levels for the Chehalis-Centralia Airport, local businesses, and a portion of I-5 (see Figure 6). This would elevate the height of the existing 9,511-foot-long levee by 4 to 7 feet. The existing levee would be raised by adding earthen materials or floodwalls on top. There is no proposed change to the extent or location of the levee unless it is raised by 7 feet. This would affect the northwest corner of the levee and could require "bumping" the levee out to avoid interference with the flight path of the airport runway.

In addition to the levee, 1,700 feet of Airport Road would be raised to meet the airport levee height along the southern extent of the airport. All utility infrastructure would be replaced, and the West Street over-cross approach would be terminated. Overall, these improvements result in up to 11,211 lineal feet of protective levee.

The legal description of the property is: Section 30 Township 14N Range 02W -- PT SEC 19 & 30 BTWN HWY, ST HELENS AVE. The parcel number is 005605080001.

Figure 6
Airport Levee



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A.1.a. *Chehalis Basin Strategy: Combined Dam and Fish Passage Design Conceptual Report*. June 2017.

Chehalis Basin Strategy

Combined Dam and Fish Passage

Conceptual Design Report

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

Prepared for: State of Washington Recreation and Conservation Office and Chehalis Basin Work Group

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ACRONYMS AND ABBREVIATIONS LIST

AACE	Association for the Advancement of Cost Engineering
ACI	American Concrete Institute
AEP	annual exceedance probability
AF	acre-feet
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASR	alkali-silica reactivity
AWS	auxiliary water supply/American Welding Society
CBFS	Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project
CFD	Computational fluid dynamics
cfs	cubic feet per second
CHTR	collect, handle, transfer, and release
cm/sec	centimeters per second
CMCE	Controlling Maximum Credible Earthquake
CM	Construction Management
CMS	conditional mean spectrum/spectra
CSZ	Cascadia Subduction Zone
D/S	Downstream
DNR	Department of Natural Resources
DSHA	deterministic seismic hazard analysis
DSO	Dam Safety Office
EM	engineering manual
ER	engineering report
FERC	Federal Energy Regulatory Commission
FMPC	fixed multi-port collector
fps	feet per second
FOS	factor(s) of safety
FRFA	Flood Retention Flow Augmentation

FRO	Flood Retention Only
FSC	floating surface collector
FTE	full-time equivalent
ft	foot/feet
ft/sec	foot/feet per second
G	gravity
GERCC	grout-enriched roller compacted concrete
HDC	Hydraulic Design Criteria
HEC	Hydrologic Engineering Center
HEC-RAS	Hydrologic Engineering Center River Analysis System
HEC-ResSim	Hydrologic Engineering Center Reservoir System Simulation
HMR	Hydrometeorological Report
I-5	Interstate 5
ICOLD	International Commission on Large Dams
IDF	Inflow design flood
IEEE	Institute of Electrical and Electronics Engineers
JS	joint set
km	kilometer(s)
MCE	Maximum Credible Earthquake
MCRCC	medium cementitious roller compacted concrete
min	minimum
mm	millimeter(s)
msa	maximum-size aggregate
msl	mean sea level
Mw	moment magnitude
NEC	National Electrical Code
NEMA	National Electrical Manufacturers' Association
NESC	National Electrical Safety Code
NFPA	National Fire Protection Association
NMFS	National Marine Fisheries Service
NOHRSC	National Operational Hydrologic Remote Sensing Center
NRCS	Natural Resources Conservation Service

NTS	net transition structure
NWMLS	Northwest Multiple Listing Service
NWS	National Weather Service
O&M	operations and maintenance
OBE	operating basis earthquake
OFM	Office of Financial Management
OPCC	opinion of probable construction cost
OSHA	Occupational Safety and Health Organization
pcf	pounds per cubic foot
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
Qa	(modern) Quarternary alluvium
Qao	(older) Quarternary alluvium
RCC	roller compacted concrete
RCW	Revised Code of Washington
RMR	Rock Mass Rating
ROW	river outlet works
RQD	Rock Quality Designation
SPF	standard project flood
SR	State Route
SSD	saturated surface dry
Tcb	pillow basalt
Tcs	Crescent Formation siltstone/claystone
TM	Technical Memorandum
U/S	upstream
UHS	uniform hazard spectrum
UL	Underwriters Laboratories, Inc.
USACE	United States Army Corps of Engineers
USBUREC	United States Bureau of Reclamation
USFWS	United States Fish and Wildlife Service

USGS	United States Geological Survey
WA	Washington
WDFW	Washington Department of Fish and Wildlife
Work Group	Chehalis Basin Work Group
WSDOT	Washington State Department of Transportation
WSE	Watershed Science & Engineering
WSEL	water surface elevation

EXECUTIVE SUMMARY

This report has been prepared in support of the Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project (CBFS Project). Through the course of this project, alternative water retention structure types and locations, options for protecting Interstate 5 (I-5) with or without dams, and other small flood reduction projects throughout the Chehalis Basin have been evaluated.

This report is a continuation of this previous work and builds upon the previously completed Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR, 2014). The 2014 memorandum included discussion of the following items:

- Feasibility of four roller compacted concrete (RCC) and rockfill dam configurations, as shown in Table ES-1 below
- Feasibility of alternative fish passage systems
- Potential climate change impacts on dam design and project cost
- Planning-level evaluation of hydropower viability
- An opinion of probable construction cost (OPCC) for alternative dam configurations
- Schedule required for completion of planning, permitting, design, and construction

Table ES-1
Summary of Initial Dam and Fish Passage Configuration Alternatives

DAM TYPE	UPSTREAM FISH PASSAGE	DOWNSTREAM FISH PASSAGE
Flood Retention RCC Dam	Run of the river tunnels at the base of the dam with collection, handle, transfer, and release (CHTR) facility	Run of the river tunnels at the base of the dam
Multipurpose RCC Dam	CHTR facility	Combination of tributary and head-of-reservoir collection facilities
Multipurpose RCC Dam	Conventional fishway with an experimental fishway exit structure	Floating surface collector
Multipurpose Rockfill Dam	Conventional fishway and exit structure	Floating surface collector

The memorandum (HDR, 2014) identified no fatal flaws in the project and provided recommendations for additional evaluations required to advance the project to a conceptual level of design. It should be noted that the Flood Retention RCC Dam is now referred to as the Flood Retention Only (FRO) dam, and the Multipurpose RCC Dam is now referred to as the Flood Retention Flow Augmentation (FRFA) dam.

As a result of the 2014 technical memorandum, the Chehalis River Basin Flood Authority (Flood Authority) made the decision that the RCC dam is the preferred configuration for conceptual design. Furthermore, the option of including hydropower as part of the project configuration was eliminated from consideration. Subsequent to those decisions, the Anchor QEA team was asked to perform site investigations in order to update concept-level engineering analyses, design layouts, and cost estimates of the FRO and FRFA RCC dam design and fish passage configurations. In addition, the team was requested to verify alternative sources of aggregate that could be used for both RCC and conventional concrete construction and provide additional information on long term operations and maintenance costs for the dam.

An initial site investigation program was completed in early 2015, and a Phase 1 Site Characterization Technical Memorandum (TM) (HDR and Shannon & Wilson Inc. [S&W], 2015) presenting the results was submitted in August of 2015. Additional site characterization (Phase 2) and engineering evaluations were completed from 2015-2016. Additional hydrologic and hydraulic evaluations have been performed by WSE.

This report documents the additional site characterization work and engineering evaluations that have been performed on the FRO and FRFA alternative dams and fish passage configurations since the 2015 study was completed.

As documented within this report, the FRO and a FRFA RCC dam configurations with alternative fishways, fish collector, and experimental exit structures identified during the 2014 study are still viable options for achieving CBFS Project objectives. The Phase 2 site characterization work and updated geotechnical evaluations have reduced the uncertainty associated with foundation excavation requirements, appropriateness of seepage mitigation strategies, and need for landslide mitigation. Supplemental hydraulic evaluations have resulted in refined spillway and fish passage design. In-depth structural evaluations have confirmed the geometry of the preliminary cross-section design of the dam developed in earlier phases of study and the location of the profile axis of the dam.

The design team anticipates that the main RCC structure could last for centuries. Fish passage facilities would likely have a more limited life span than the main RCC dam structure. The following are the estimated life spans for various components of the fish passage facilities:

- Collect, handle, transfer, and release (CHTR) facility: 25 years
- Conventional fishway: 50 years
- Floating surface collector (FSC): 30 years
- Fixed multi-port collector (FMPC): 50 years

An updated estimate of total project direct costs including the opinion of probable construction cost (OPCC) is provided within this report and is summarized in Table ES-2 below. These costs are presented on a 2016 cost basis. An update of these costs to a 2017 cost basis, and including some additional refinements to the hydraulic structures configuration that have been completed as part of the development of an expandable Flood Control dam Option (FRX), are presented in a separate supplemental report titled Flood Retention Expandable (FRX) Dam Option (HDR, 2017).

Table ES-2
Estimated Total Direct Project Costs (including OPCC, 2016 Dollars)

FISH PASSAGE OPTION	LOWER BOUND COST ($ MILLION)	WEIGHTED/MIDDLE COST ($ MILLION)	UPPER BOUND COST ($ MILLION)
FRO			
FRO RCC Dam	$209	**$250**	$306
Upstream Fish Passage: CHTR Facility	$11	**$14**	$18
Downstream Fish Passage	Integral to dam construction		
Total	$220	**$264**	$324
FRFA			
FRFA RCC Dam	$315	**$371**	$450
Upstream Fish Passage: CHTR Facility	$11	**$14**	$18
Downstream Fish Passage: Floating Surface Collector	$69	**$86**	$112
Total	$395	**$471**	$580

The updated OPCC is consistent with the estimate made during the 2014 study. Updated drawings and opinion of probable construction cost are provided with this report, and recommendations are made for further evaluation. The completion of this report is intended to allow selection of the preferred alternative and advancement of the project to preliminary design. Based on the design team's experience with other large dam and fish passage facilities, it is anticipated that the time required to complete final design and construction would likely be 6 to 11 years from the publication of this report.

Operation and maintenance costs for the two alternatives were developed with consideration of the requirements for replacement of dam components that are subject to wear and trash and sediment removal, as well as staffing and equipment needed for the dam and fish passage facilities. The annual cost (2016 dollars) is as follows:

- FRO: $628,000 per year
- FRFA: $2,178,000 per year

1 INTRODUCTION

1.1 Project Background

The Chehalis Basin (basin) has historically been prone to flooding. The economic damages of the 2007 flood alone were estimated at more than $900 million, with one-third of that damage coming from disruption and damage to the transportation system, including Interstate 5 (I-5), other state highways, and rail lines. Many different flood hazard mitigation projects and approaches have been proposed and studied in response to the major floods in the Chehalis Basin. After the 2007 flood, the Chehalis River Basin Flood Authority (Flood Authority) was created to focus on developing flood hazard mitigation measures throughout the basin and to identify and implement flood damage reduction projects. The Flood Authority has been studying water retention in the upper Chehalis River Basin along with smaller flood hazard mitigation projects in the lower portion of the basin.

In 2011, the Washington State Legislature required the Office of Financial Management (OFM) to prepare a report on alternative flood damage reduction projects and – in coordination with tribal governments, local governments, and state and federal agencies – to recommend priority flood hazard mitigation projects for continued feasibility assessment and design work. In response to the legislative direction, the Ruckelshaus Center published the Chehalis Basin Flood Hazard Mitigation Alternatives Report in December 2012. That report compiled existing information on the potential flood hazard mitigation projects that seemed of most interest to basin leaders and decision makers at that time. Potential flood hazard mitigation benefits, adverse impacts, costs, and implementation issues were summarized for each project to the degree that such information was available. Along with that effort, the Chehalis Basin Work Group (Work Group), composed of Chehalis Basin leaders, recommended to then Washington Governor Christine Gregoire a series of actions that, taken together, would represent a significant investment to reduce flood damage, enhance natural floodplain function and fisheries, and put basin leaders on firm footing to make critical decisions about large-scale projects. The Work Group recognized that habitat loss in the basin has contributed to a reduction in native fish populations and set the goal to develop a basin wide strategy to integrate flood damage reduction and environmental enhancement.

The Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project (CBFS Project) is evaluating the feasibility of mitigating flood hazards within the basin while exploring opportunities to enhance ecological conditions, aquatic habitat, and the abundance of fish in the basin. The scope of the Project has included studying alternative water retention structures (dams), options for protecting I-5 and other floodplain at-risk facilities and structures with or without a dam, and other small flood reduction projects throughout the basin. As this project has proceeded, viable options to accomplish project objectives have narrowed as a result of analysis and evaluation conducted by the project team. Along with several other options, considered either independently or in combination, the floodwater retention concept consisting of a dam on the upper Chehalis River has been advanced through the conceptual design phase.

The initial layout, technical feasibility, and opinion of probable construction cost (OPCC) of multiple alternate dam and fish passage configurations at the proposed dam site were documented in HDR's 2014 Combined Dam and Fish Passage Alternatives Technical Memorandum. The memorandum also provided recommendations on additional site characterization and engineering evaluations that would be required to reduce design uncertainty, refine estimated project costs, and support selection of a preferred alternative. An initial site investigation program was completed in early 2015, and a Phase 1 Site Characterization Technical Memorandum (TM) (HDR and Shannon & Wilson Inc. [S&W], 2015) presenting the results was submitted in August of 2015. Additional site characterization (Phase 2) and engineering evaluations were completed from 2015-2016. Additional hydrologic and hydraulic evaluations have been performed by WSE.

1.2 Proposed Dam Location and Size

The proposed dam site was selected from several alternative locations identified and evaluated in previous studies (S&W, 2009a; 2009b). The design storage volumes and corresponding estimated water storage elevations for the Flood Retention Only (FRO) and Flood Retention Flow Augmentation (FRFA) configurations presented in this document are summarized in Table 1-1. The storage volumes and corresponding dam heights and inundation areas are subject to change as climate change and operation studies advance through the planning process. Further evaluation of alternate dam sites is not included in the current study.

The proposed dam site is located south of State Route (SR)-6 in Lewis County, Washington, on the main stem of the Chehalis River, about 1 mile south of Pe Ell (the southwest corner of Section 3, Township 12N, Range 5W). Figure 1-1 shows the dam site location. Figure 1-2 shows an example of the approximate flood storage inundation area (65,000 acre feet of flood storage at the spillway crest) for the FRO configuration. Figure 1-3 shows the maximum inundation limits of the flow augmentation pool (65,000 acre feet of water storage) as well as combined flow augmentation and flood storage pool at the spillway crest (65,000 and 130,000 acre feet, respectively) for the FRFA project configuration.

Table 1-1
Summary of Dam Storage Volumes and Maximum Water Surface Elevations

CONFIGURATION	WATER STORAGE VOLUME (ACRE FEET)	FLOOD STORAGE VOLUME (ACRE FEET)	MAXIMUM WATER STORAGE ELEVATION (FEET)	MAXIMUM FLOOD STORAGE ELEVATION (FEET)
FRO	0	65,000	-	628
FRFA	65,000	65,000	628	687

Notes: 1. Maximum flood storage volumes and elevations are to spillway crest and do not include flood routing capacity between the design flood (100-year event) and the Probable Maximum Flood (PMF).

Figure 1-1
Dam Site Location

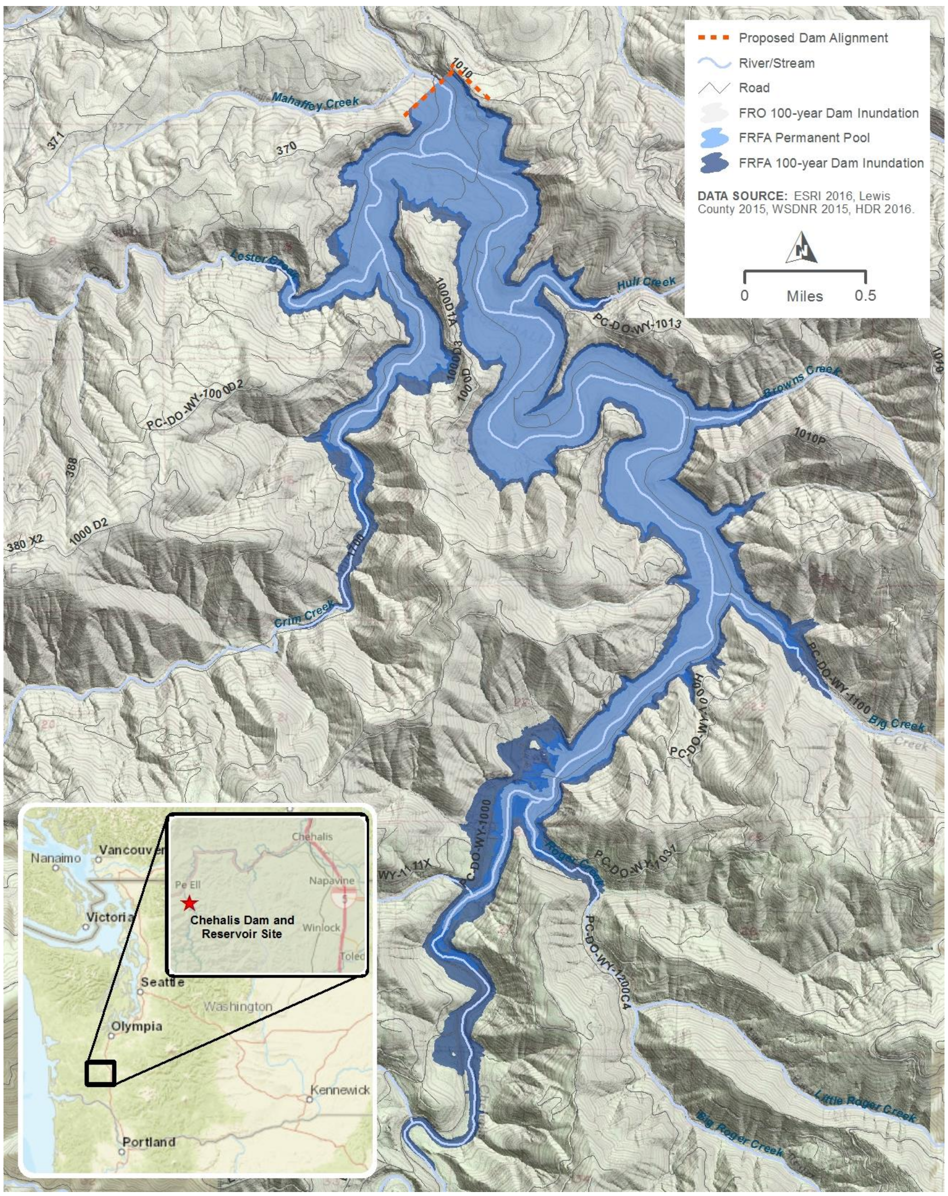


Figure 1-2
FRO Flood Storage Inundation Area (65,000 Acre-Feet of Flood Storage)

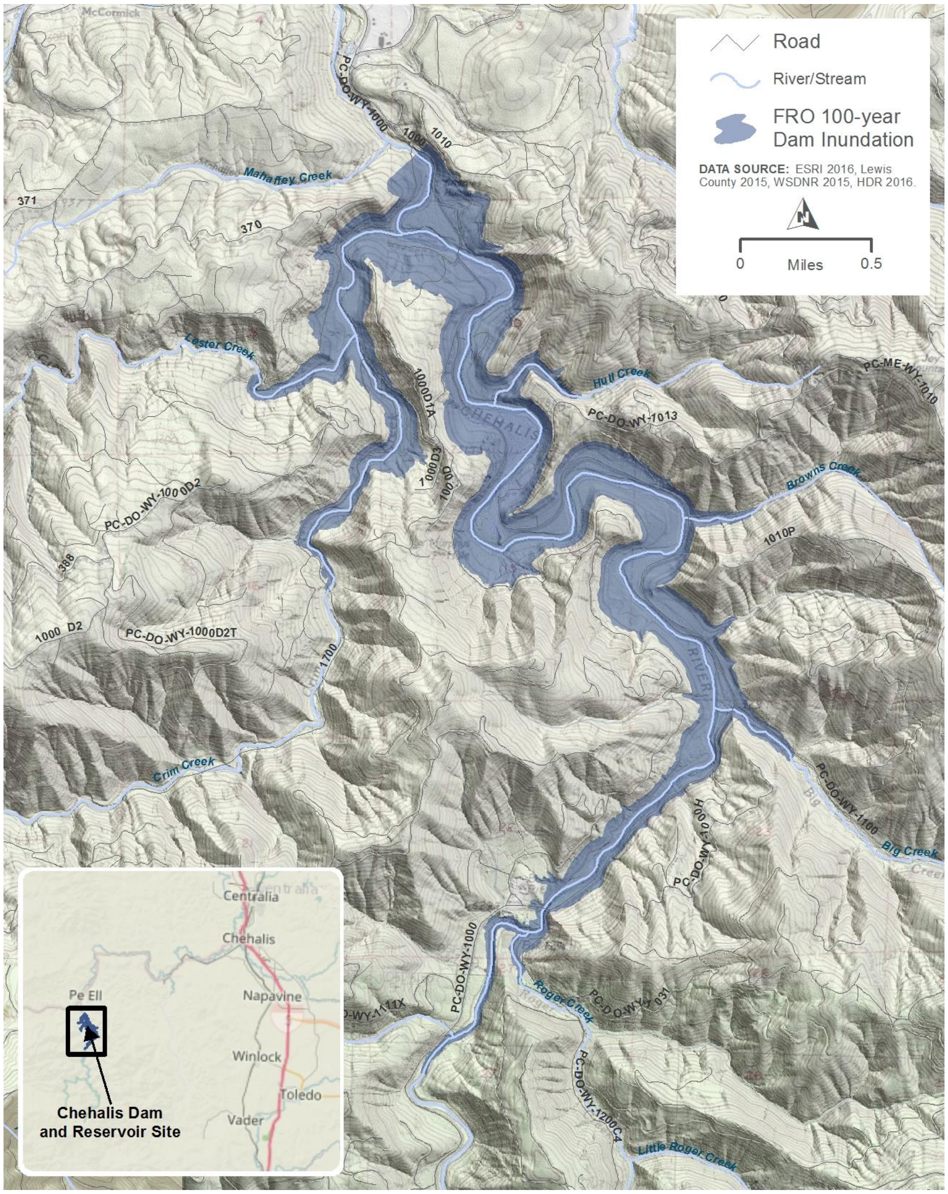
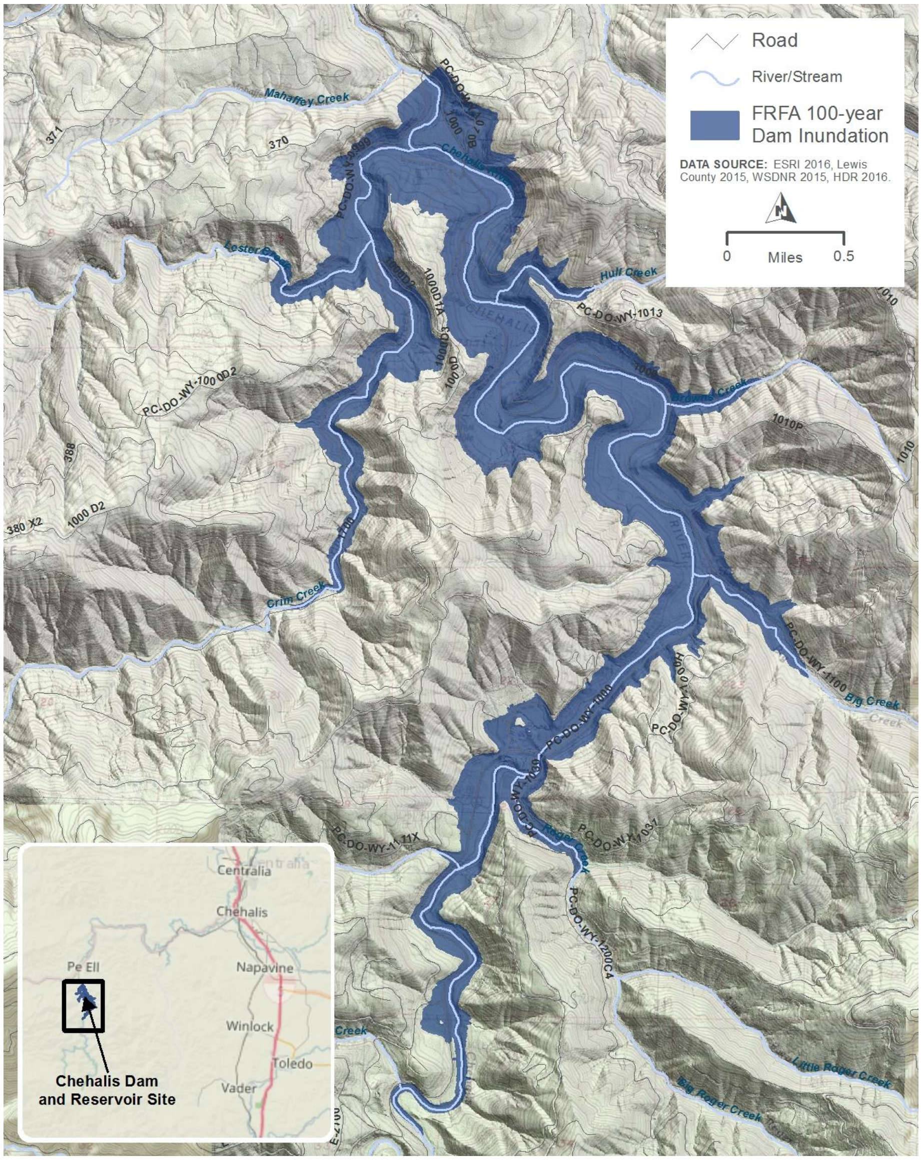


Figure 1-3
FRFA Maximum Reservoir Inundation Area (65,000 Acre-Feet of Flood Storage)



1.3 Previous Reports

The previously completed documents listed below form the basis for the updated conceptual dam design options presented in this technical memorandum:

- Interim Fish Passage Design Criteria Technical Memorandum, October 2013 – The Interim Fish Passage Design Criteria Technical Memorandum evaluated biological and technical aspects of fish passage facility alternative development.

- Draft Dam Design Technical Memorandum, March 2014 – The objectives of the Dam Design Technical Memorandum (TM) were to identify any fatal flaws that would limit or preclude construction of a water retention structure at the proposed location on the main stem of the Chehalis River, and to develop technically feasible options for a flood retention or a multipurpose dam at that site.
- Fish Passage Design Technical Memorandum, May 2014 – The Fish Passage Design TM evaluated potential fish passage technologies, established design criteria, and developed options for upstream and downstream passage of adult and juvenile fish that could be integrated with feasible water retention (dam) structures.
- Combined Dam and Fish Passage Alternatives Technical Memorandum, September 2014 – The Combined Dam and Fish Passage Alternatives TM built upon the findings of the Dam Design TM and the Fish Passage Design TM to combine selected dam design options with selected fish passage options to describe four integrated alternatives that can be compared in terms of function, constructability, and capital and operations and maintenance costs.
- Interim Dam Design Criteria Technical Memorandum, draft version December 2015 – The Interim Dam Design Criteria Technical Memorandum combined dam and fish passage criteria to facilitate discussion. The fish passage criteria continued to develop in collaboration with representatives from various resources agencies.

1.4 Purpose and Objectives

This report documents the additional evaluations completed during 2015-16 and provides information needed by the Work Group and stakeholders in the region to support their decision whether to advance the project to the next phase of project design and permitting. Recommendations on supplemental analysis required to advance the design are also provided. Detailed evaluations of specific topics are attached to this memorandum as appendices. These memoranda include:

- Appendix A – Maps and Drawings
- Appendix B – Hydraulic Design of Spillway and Reservoir Outlet Works
- Appendix C – Dam Foundation Design
- Appendix D – Seismic Evaluation
- Appendix E – RCC Materials Sourcing
- Appendix F – Opinion of Probable Dam Construction Cost, Schedule and Constructability
- Appendix G – Fish Passage Alternative Concept Design

This memorandum and the attached documentation are presented for consideration and review by the Water Retention Technical Committee and other technical committees working on the project.

1.5 Scope of Services

The work performed to develop this report included the following tasks:

- Development/update of design criteria for dam and fish passage facility design
- Acquisition of field and laboratory test data including soil/rock borings and surface and borehole geophysical surveys
- Site observation and review of LiDAR data/aerial photographs
- Development of a detailed site characterization supported by newly acquired field data
- Creation of a 3D geologic model in support of design analysis
- Evaluation of foundation excavation and treatment requirements for the dam, hydraulic structures, and collection, handle, transfer, and release (CHTR) and fish ladder facilities
- Hydraulic analyses to support updated design of fish passage, flood control, construction diversion, and water quality outlet works
- Hydraulic analyses to support updated design of the dam's emergency spillway
- Assessment of landslide potential, mitigation strategies, and costs
- Identification and evaluation of alternative quarries near the dam site that would be used to generate aggregate materials for roller compacted concrete (RCC) and conventional concrete materials required for dam, appurtenant structure, and fish passage facilities
- Evaluation of debris hazards and debris management strategies to be included in the updated conceptual design
- Seismic response structural analysis to confirm the cross-section requirements for the dam
- Research on precedent at similar facilities and development of operations and maintenance strategies and costs for flood retention and multipurpose flood retention facilities
- Development of preliminary design-level estimates of probable construction costs for the FRO and FRFA project alternatives
- Development of recommendations for next steps in project development
- Preparation of documentation (this report) summarizing the above information

1.6 Project Team

The following personnel were involved in the various evaluations required to complete the updated conceptual designs:

Project Manager:	Beth Peterson, P.E.
Technical Manager and Lead Civil Eng:	Keith Moen, P.E.
Lead Dam Engineer:	Keith A. Ferguson, P.E.
Lead Geotechnical Engineer:	Dan Osmun, P.E.
Geological Engineers and Geologists:	Andrew Little, E.I.T.

	John Charlton, P.Geologist
Lead Hydraulic Engineer:	Ed Zapel, P.E.
Lead Fish Passage Designer:	Michael Garello, P.E.
Constructability and Cost Estimating:	Jeffrey Allen, P.E.
Project Support:	Gokhan Inci, Ph.D., P.E. Geotechnical Eng.
	Mathew Prociv, P.E., Fish Passage Design
	Shaun Bevan, P.E., Fish Passage Design
	John Ferguson, Ph.D., Fish Passage Biology (Anchor QEA)
	John Hess, P.E., Materials Engineering
	Ali Reza Firoozfar, Ph.D., E.I.T., Civil/Hydraulic Engineer
	Carl Mannheim, P.E., Senior Civil/Hydraulic Engineer
	Anna Mallonee, Engineering Intern

Additional staff support provided for drawings, document production, and quality control.

2 CHEHALIS DAM STRUCTURE OPTIONS

2.1 FRO and FRFA Operational Approach

The FRO and FRFA would both be designed to provide downstream flood protection benefits, but would have different dam hydraulic heights, operational approach, and potential storage volumes. The smaller of the two, the FRO dam alternative, would be designed to only store flood flows as needed to control downstream river flows to the desired Grand Mound gage control flow. Most of the time, the dam outlet works in the FRO alternative would remain fully open and unregulated. The larger FRFA dam alternative would be designed to provide a permanent storage pool to allow augmentation of downstream river flows during low flow periods for fish and aquatic habitat enhancement, while also providing storage volume above the permanent pool for floodwater storage to accommodate extreme precipitation events. The permanent pool elevation (and resulting storage volume) for the FRFA dam alternative would be varied depending on annual hydrology and water management objectives.

2.2 FRO Design

The FRO dam and reservoir would be comprised of a concrete (RCC) gravity dam structure with a right abutment construction diversion tunnel, low-level fish passage and flood control outlet works, an ungated spillway, and supplemental fish passage facilities. The FRO alternative would be designed to temporarily store floodwater only when the downstream gage at Grand Mound is forecasted to rise above 38,000 cubic feet per second (cfs) within 48 hours. Such temporary storage events are estimated to have only a one in seven-year recurrence interval. After flood regulation operations are commenced and the outlet works begin regulating outflows, fish passage through the outlet works would no longer be available. Debris management operational plans and potential operational modifications associated with climate change scenarios have necessitated consideration of redundant fish passage facilities that would be operated during periods of flood retention and subsequent debris removal. At all other times, the project is expected to retain no water and to allow all river flows to pass, with only minor restriction of river flow and pool accumulation at the upstream face of the dam.

The FRO's primary components are the following:

- A dam sized for flood storage with estimated maximum dam structural height of 254 to 270 feet depending on final foundation elevation
- An RCC dam crest length of approximately 1,225 feet
- A dam section that is contained entirely within the river valley and would not require a saddle dam along the right abutment ridge line to provide closure

- An ungated overflow spillway, designed to pass the PMF without dam crest overtopping, including a spillway chute, flip bucket, and plunge pool
- Construction diversion tunnel through right abutment
- Low-level outlet sluices for sediment and fish passage, as well as flood control operations
- Energy dissipation Type 2 hydraulic jump stilling basin with floor baffles downstream of sluices
- Fish passage facilities designed for free passage upstream and downstream prior to and after flood, and trap and haul during flood regulation periods
- Target flood detention storage capacity of 65,000 acre feet
- Additional conceptual-design drawings of this alternative dam and appurtenant structures configuration are included in Appendix A.

2.3 FRFA Design

The FRFA dam and reservoir would consist of a concrete (RCC) gravity dam structure with low-level flood control outlets and multilevel water quality outlets, fish passage facilities, and an ungated spillway through the crest of the dam. The FRFA alternative would maintain a permanent pool behind the dam and be designed to provide water storage and releases for flow augmentation from the permanent pool to enhance certain aquatic species habitat, and a flood management pool between the designated permanent pool level and the spillway crest for flood operations. The FRFA alternative would also consist of a concrete gravity dam structure with a right abutment construction diversion tunnel, low-level flood control outlet(s), three to five water quality release outlets, an emergency spillway, and separate fish passage facilities.

The FRFA's primary components include the following:

- A dam sized for the combined flood storage and water quality with estimated dam structural height of 313 to 330 feet depending on final foundation elevation
- An RCC dam crest length of approximately 1,600 feet
- A central core rockfill section approximately 700 feet long along the right abutment ridge that is perpendicular to the main RCC dam section
- An ungated overflow spillway designed to pass the PMF without dam crest overtopping, including a spillway chute, flip bucket, and plunge pool
- Multiple outlet works including water quality inlets/outlets that draw water from multiple levels within the reservoir and low-level flood control outlet sluices
- Energy dissipation Type 2 hydraulic jump stilling basin with floor baffles downstream of sluices
- A recommended upstream fish passage by trap and haul or fishway; a recommended downstream fish passage by trap and haul
- A permanent reservoir pool of up to 65,000 acre feet to be used for flow augmentation in late summer and fall prior to the winter rainy season to enhance fish habitat

- Up to 65,000 acre feet of flood storage volume to be activated in flood events larger than the estimated 5-year recurrence interval event.
- Additional conceptual-design drawings of this alternative dam and appurtenant structures configuration are included in Appendix A.

In previous studies it was assumed that the RCC dam alignment would turn and follow the right abutment ridge to a point where the section became small, then transition to a low earthfill embankment dam. During Phase 2 of the site characterization, additional information on the right abutment was obtained from three borings and geophysics profiles. The borings identified a change in the rock quality on the right abutment from that assumed in the previous studies. The required excavation to achieve an acceptable RCC dam foundation was estimated to be very deep, requiring the RCC dam section to transition to an embankment section at a point closer to the main dam. In addition, as the excavation requirements were updated, it became apparent that a central earth core rockfill dam would be more appropriate for the turned portion of the alignment than the original earthfill design concept. Performance and material balance benefits associated with this change are as follows:

- The rockfill dam can be constructed with steeper upstream and downstream slopes because compacted rockfill is stronger than compacted earthfill.
- The overall footprint of the right abutment dam is smaller than what would be required for an earthfill section.
- A significant quantity of rockfill material will be available from required excavations for the RCC dam and from rock in the upper portions of the RCC aggregate quarry resulting in an economical source of materials.

A rockfill dam can generally be designed to make efficient use of on-site materials and still provide a safe structure with good seepage and stability performance. Additional analyses will be necessary to evaluate the need for a deeper seepage cutoff below the dam in this area.

3 GUIDELINES AND CRITERIA

3.1 Hydrologic and Hydraulic Design Requirements

The hydrologic study performed by WSE (WSE, 2016), coupled with the hydrologic modeling of flood storage attenuation by Anchor QEA (Anchor QEA, 2014), forms the basis for hydraulic design for the FRO and FRFA design alternatives. The hydrologic design criteria that apply to both configurations are as follows:

- Project inflow design flood (IDF) is the PMF, which is estimated to be 69,800 cfs.
- The spillway capacity will be equal to the PMF, with freeboard to the dam crest.
- Flood storage equal to 65,000 acre feet, approximately equal to the flood volume of the 2007 flood of record.

The FRFA and FRO alternatives will vary as follows:

FRO:

- PMF maximum reservoir elevation is 650 ft msl
- Spillway crest elevation is 628 ft msl
- Dam crest elevation is 651 ft msl
- Minimum flood storage reservoir elevation = natural riverbed elevation
- Maximum flood storage elevation with no spillway flow is 628 ft msl
- Minimum low-level flood regulation sluice capacity is at least 15,000 cfs at reservoir elevation 550 ft msl, but not limited to that for higher reservoir elevations

FRFA:

- PMF maximum reservoir elevation is 709 feet (ft) mean sea level (msl)
- Spillway crest elevation is 687 ft msl
- Dam crest elevation is 710 ft msl
- Minimum flood storage reservoir elevation is 628 ft msl
- Maximum flood storage elevation with no spillway flow is 687 ft msl
- Minimum low-level flood regulation sluice capacity is at least 15,000 cfs at reservoir elevation 550 ft msl, but not limited to that for higher reservoir elevations
- Maximum flow augmentation reservoir elevation is 628 ft msl, to provide the gross storage volume of 65,000 acre ft between elevation 425 and 628 ft msl

- Typical minimum flow augmentation reservoir elevation is 588 ft msl (585 ft msl with climate change scenario), though full volume is achieved between elevation 425 and 628 ft msl (Anchor, 2016)

3.2 Dam Design Guidelines and Requirements

The current project is being funded by Washington State, but final design and construction may include federal funding and/or federal reviews. The strategy for identification of design criteria outlined in this document is to develop a dam design that will satisfy both state and federal requirements.

3.2.1 State

The Washington State Department of Ecology Dam Safety Office (DSO) uses a risk-informed decision-making framework that incorporates consequence-dependent design levels such that increasingly stringent criteria are applied as the potential for life loss or property damage increases. The procedure can also be tied to a downstream hazard classification (consistent with federal guidelines) of the proposed dam.

Establishing the design/performance goal for the dam under the DSO guidelines is a two-step process. First, a numerical rating of the consequences of dam failure is estimated using guidance provided in Technical Note 2 of DSO's dam safety guidelines (Washington State, 1993). 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. Numerical values are assigned to each parameter, and values are totaled to estimate the consequence rating points.

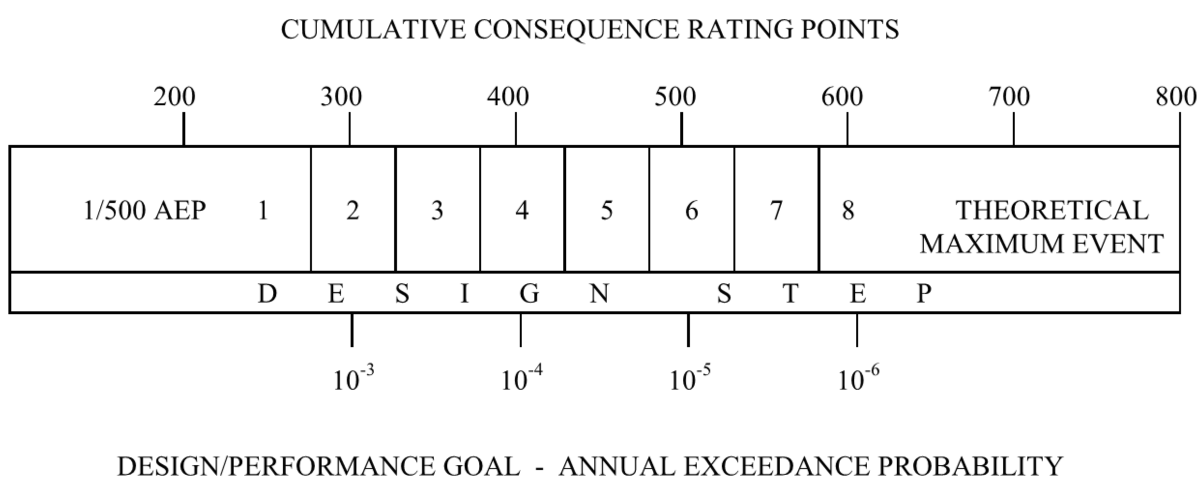
Both dam configurations present unusual considerations in assigning an appropriate consequence rating because of the amount of the reservoir storage pool that is dedicated to flood storage only and the very limited amount of time that the flood storage is utilized. The consequence rating for the FRO configuration would be very low when the reservoir is not storing water, which would be most of the time. Similarly, the consequence rating for the FRFA configuration would be lower when the water storage level is at normal operational pool elevation, compared with when it is at flood pool capacity. Therefore, it is reasonable to present a range of consequence rating values that represent the range of operations for each configuration. Table 3-1 below presents a summary of the estimated consequence ratings for the proposed Chehalis Dam project FRO and FRFA configurations using this approach.

Table 3-1
Summary of Preliminary Consequence Rating for Chehalis FRO and FRFA Dam Configurations

CONSEQUENCE CATEGORY	INDICATOR PARAMETER	RATING – FRO	RATING – FRFA
Capital value of project	Dam height	140	150
Capital value of project	Project benefits	10	40
Potential for loss of life	Catastrophic index	0-60	60-70
Potential for loss of life	Population at risk	0-140	140-200
Potential for loss of life	Adequacy of warning	0	50-100
Potential for property damage	Items damaged or services disrupted	0-140	170-180
Base points		150	150
Total rating		**300-640**	**760-890**

The consequence rating points from Table 3-1 have been used to inform the hydrologic, structural, and geotechnical design/performance goals for the dams. Specifically, the design step and the annual exceedance probability (AEP) were identified using the information shown on Figure 3-1 below from the DSO guidelines:

Figure 3-1
DSO Design Step and Estimated Loading Condition Recurrence Interval Based on Consequence Rating



The following are the design steps and corresponding AEP guidelines for the conceptual/preliminary design of the alternative dam configurations:

FRO	Design Step: 2 to 8	AEP: $1x10^{-3}$ to $1x10^{-6}$
FRFA	Design Step: 8	AEP: $1x10^{-6}$

The DSO consequence-dependent design process results in a Design Step 8 requirement for both dam configurations under flood-loading conditions. This design step corresponds to the very low AEP values shown. Design Step 8 is intended to correspond with the theoretical maximum design events and loading conditions. Because there is great uncertainty in extrapolating flood and earthquake AEPs to such very small values, the use of maximum deterministic values such as the Maximum Credible Earthquake or Probable Maximum Flood loadings is appropriate for design. The PMF has been used as the design flood for the dam and spillway. However, because of the unique seismic hazard associated with the Cascadia Subduction Zone and the very large earthquakes this hazard can generate, a project-specific risk-informed approach has been developed and used for the cross-section design of the dam based on the AEP estimated discussed above, and the risk-tolerance guidelines used by the USBUREC and USACE. The information provided below outlines the basis for development of the risk-informed structural design approach.

Based on our current knowledge of the population at risk in the downstream corridor, as well as our experience with the federal hazard classification system for dams, the proposed FRO and FRFA dam configurations classify as “high” hazard potential structures. The DSO-related design step and AEP requirements outlined above are consistent with a “high” hazard potential and downstream hazard classification of 1C or 1B, as shown in Table 2 in the DSO guidelines.

It should be noted that the DSO design step and AEP requirements outlined above will continue to be reviewed throughout the CBFS project until such time as final designs are underway and will be subject to change based on input from the state. The DSO guidelines are largely focused on analysis of existing low embankment structures, which constitute the majority of the dams under state jurisdiction. Gravity and RCC dams and barriers are not discussed in detail. With regard to large concrete dam structures, the dam safety guidelines include the following statement:

> *“Concrete structures present a number of unique design problems. Generally, only specialty firms, well versed in the peculiarities posed by such structures, are qualified to formulate a suitable design. It would be misleading to imply that these guidelines could somehow substitute for the requisite experience and judgment necessary to design a suitable concrete impounding structure.”*

A unique attribute of concrete dams is that they provide a basis for adaptation of more "risk-informed" design criteria. In high seismic hazard areas, earthquake loading typically governs the cross-sectional requirements of the structure. A risk-informed design involves setting the cross-section of the dam for a loading condition that is something less than the AEP with the expectation that there should be no damage to the structure during such an event. An allowance for minimal to tolerable damage is identified and adopted for severe loadings in order to balance safety and cost objectives.

Using a target cross-section configuration for the dam, potential loading up to and including the AEP condition are then analyzed and the expected structure performance is evaluated. Tolerable deformations and post-earthquake stability with a suitable margin of safety must then be demonstrated. If deformations are small, and the post-earthquake stability indicates a very low potential for a catastrophic failure of the dam, the cross-section is adopted. If deformations are large, or if the post-earthquake stability of the structure is not acceptable, the base design criteria are increased, and the cross-section of the dam is modified until acceptable performance is obtained.

3.2.2 Federal

Federal agencies have well established guidelines for evaluating the safety of existing concrete dams such as the RCC configuration proposed for the Chehalis project. The federal agencies that have established design criteria and guidelines include the U.S. Army Corps of Engineers (USACE), U.S. Bureau of Reclamation (USBUREC), the Federal Energy Regulatory Commission (FERC), and the Natural Resources Conservation Service (NRCS). Although there are some differences in the details of the federal agencies' guidelines, the general approach is relatively consistent and the agencies often refer to the guidelines developed by the other agencies. As noted above, the intent of the designs under the current scope of work will be to satisfy design criteria of all federal agencies where it is reasonable to do so; however, when in question, the USACE guidance will be given precedent.

3.2.2.1 USACE

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

- EM 1110-2-2200, Gravity Dam Design, June 1995
- EM 1110-2-2006, Roller-Compacted Concrete, January 2000
- EM 1110-2-2100, Stability Analysis of Concrete Structures, December 2005
- ER 1110-2-1156, Safety of Dams – Policy and Procedures, March 2014

From ER 1110-2-1156:

> *"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 for USACE criteria require written concurrence from the USACE DSO."*

The USACE is currently in the process of updating a number of its guidance documents, including several of those listed above, to incorporate more risk-informed criteria and methodology.

3.2.2.2 USBUREC

In addition to publishing numerous dam design books and guidelines, the USBUREC has been a leader in developing and incorporating risk-informed dam safety and design methods and guidelines. Applicable USBUREC design guidance is as follows:

- Design Standard No. 2 – Concrete Dams
- Design Standard No. 3 – Water Conveyance and Fish Facilities
- Design of Small Dams, 1987
- Design of Gravity Dams, 1976
- Roller-Compacted Concrete, Design and Construction Considerations for Hydraulic Structures, 2005
- Interim Dam Safety Public Protection Guidelines, A Risk Framework to Support Dam Safety Decision-Making, August 2011
- Reclamation Consequence Estimating Methodology, Interim, Guidelines for Estimating Life Loss for Dam Safety Risk Analysis, February 2014
- USBUREC and USACE, Dam Safety Risk Analysis Best Practices Training Manual, 2012

3.2.2.3 FERC

FERC regulates 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 highly evolved methodology and guidance on the design and safety evaluation of concrete structures:

- FERC Guidelines, Chapter 3, Gravity Dams, revised October 2002

3.2.3 International

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 Chehalis project dam alternatives.

- B145 – Physical Properties of Hardened Concrete in Dams, January 2009
- B165 – Materials for Concrete Dams, November 2013
- B126 – Roller Compacted Concrete Dams, 2003

The ICOLD concrete dam committee is in the process of updating Bulletin 126, and this document should be generally available by the end of 2016. When complete, this bulletin will represent the most comprehensive guidance on design of RCC dams based on the emerging best practices from major projects around the world.

3.3 Structural Design Guidelines and Requirements

A concept-design level risk-informed approach was established to evaluate and confirm the cross-section requirements of the RCC dam. The maximum non-overflow and overflow sections of the FRFA configuration were used in the analyses because these sections represent the highest structural height, and 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 adopted for the updated conceptual design includes four separate but related criteria, as follows:

1. Elastic response of the dam for earthquake loads between a 500- and 2,500-year recurrence interval event.
2. A limited-damage, nonlinear response of the dam for loads up to a 5,000- to 10,000-year event. For this load partition, cracking of the dam would be allowed to occur, but deformations would be limited to less than 1 foot.
3. A nonlinear response of the dam to loads ranging from a 10,000- to 50,000-year event. Cracking of the dam would be allowed to occur, but deformations would be limited to less than 5 feet.
4. The post-earthquake stability of the structure would have a factor of safety greater than 1.1 so that catastrophic failure of the dam would not occur under any loading condition including events greater than a 50,000-year loading.

Other import structural components will be required for either project configuration including spillway training walls up to 50 feet in height, a large intake tower for the water quality outlet works, and debris management racks for all outlet works. Detailed design of these structural elements has not been performed during the updated conceptual design. Instead, configurations of the structural elements have been based on experience with analyses, design, and construction of similar systems on projects having similar hydrologic, hydraulic, and seismic design criteria. Future design of these components of the FRFA and FRO configurations will be based on state and federal design guidance (USACE and USBUREC) as well as the following:

- American Institute of Steel Construction (AISC) Specification for Design Fabrication and Erection of Structural Steel for Buildings
- American Welding Society (AWS) Standard D.1.1, Structural Welding Code

- American Concrete Institute (ACI) 318, Building Code Requirements for Reinforced Concrete
- AISC Allowable Stress Design – 13th Edition
- Occupational Safety and Health Administration (OSHA)

3.4 Mechanical Design Guidelines and Requirements

Mechanical Design (Gate hoist):

- Construction Management Association of America Specification No. 70, Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes

Gate Roller and Roller Path Design:

- American Society of Civil Engineers Paper No. 3000, Fixed Wheel Gates for Penstock Intakes, by Skinner

3.5 Electrical Supply and Controls Design Guidelines and Requirements

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

- American National Standards Institute (ANSI)
- Applicable local codes and standards
- Institute of Electrical and Electronics Engineers (IEEE)
- National Electrical Code (NEC), ANSI/ National Fire Protection Association (NFPA) 70 latest edition
- National Electrical Manufacturers Association (NEMA), Power Switching Equipment, Publication SG-6
- National Electrical Safety Code (NESC), ANSI C2 latest edition
- National Fire Protection Association
- Occupational Safety and Health Administration (OSHA)
- Underwriters Laboratories Inc. (UL)

3.6 Geotechnical Design Guidelines and Requirements

Geotechnical design criteria have been used to establish the following elements of the dam design:

1. Excavation Objective
   A. Multi-attribute model based on Rock Mass Rating (RMR), Rock Quality Designation (RQD), and compression wave velocity and weathering descriptions. Between borings, excavation

objective based on results of seismic refraction surveys with compression wave velocity ≥ about 9,000 feet per second (ft/sec)

B. Foundation excavation is planned to expose acceptable quality foundation rock, to facilitate appropriate treatment of poor foundation anomalies, and to shape the foundation to lessen abrupt cross-sectional changes and stress concentrations. Supplemental objectives include seeking high confidence that good foundation is exposed at the hydraulic structure's planned foundation lines and grades. Additionally, excavation design seeks to favorably support construction access with, for example, curtain grouting.

Dam foundation excavation and subsequent cleaning, mapping, and treatment, including foundation curtain and consolidation grouting operations, are nearly always critical path work activities. Chehalis will not be an exception. Consequently, thoughtful preconstruction exploration and investigation, as well as anticipatory anomaly treatment design, are necessary risk management tools. For example, if drilling is unable to reveal much about probable conditions beneath the active riverbed, are the foundation design, construction cost, and schedule reasonably tolerant of an objectionable shear zone or subsurface chemical weathering? If foundation rock is not found where needed for the river outlet works conduit, is the design reasonably adaptable? If not, perhaps additional design, excavation details, treatment plans, and/or cost contingency may be warranted.

The concept-level design does not anticipate consolidation grouting in addition to a foundation grout curtain. During preliminary design, seepage design, anticipated rock conditions, and dam stability analysis can inform the need for including a consolidation or blanket grout program across the dam's foundation. A consolidation grout program, if found necessary for the FRFA, may not be necessary for the FRO design.

2. Foundation stability
   A. Any joint sets that show a kinematic potential for sliding under the dam will be designed for the following factors of safety (FOS):
      i. Static loading: ≥ 1.5
      ii. Flood loading: ≥ 1.2
      iii. Post-earthquake stability: FOS ≥ 1.1
   B. Landslides at critical locations:
      i. Adjacent to construction excavations: Minimum (Min) FOS ≥ 1.2 to 1.3
      ii. Reservoir or downstream near critical structures: Min FOS ≥ 1.5 to 1.75
3. Excavation stability
   A. Temporary excavation slope: Min FOS >= 1.2
   B. Permanent excavation slopes: Min FOS >= 1.5

4. Foundation seepage control
   A. Multi-line grout curtain designed to a closure criteria of 1 by 10^{-5} centimeters per second (cm/sec)
   B. Foundation drain holes downstream of the grout curtain designed to reduce uplift pressures beneath the dam to maximum 1/3 of hydraulic height at the line of drain holes. Holes to be oriented to maximize pickup potential of the identified joint sets J1 through J5. Well screens with filter pack to be installed in all drain holes that penetrate foundation siltstone/ claystone lenses

4 HYDROLOGY

4.1 Basin Hydrology

The hydrologic analysis supporting the development of the Chehalis dam alternatives was conducted by Watershed Science & Engineering (WSE). The following discussion summarizes their analysis and relevant inputs to the dam design. This information was provided in two cited sources (WSE, 2014, and WSE, 2016).

The Chehalis River Basin comprises approximately 2,200 square miles within the southwest portion of the State of Washington. The highest elevations of the basin lie within the Willapa Hills at less than 3,500 feet above sea level. In general, the basin is dominated hydrologically by a temperate climate with abundant fall, winter, and spring rainfall, with some occasional snowfall but relatively little consistent snowpack development, and drier, cool summers. Its hydrology is typical of low- to moderate-elevation lowland coastal basins of the Pacific Northwest, with generous winter base flow and periodic high flows resulting from Pacific storms carrying heavy rainfall from the southwest. Orographic rainfall is significant in the higher elevations of the basin, with lesser amounts in the lowland areas. Average annual rainfall varies from about 45 inches per year in the vicinity of Chehalis and Centralia up to as much as 120 inches per year at the higher elevations in the Willapa Hills. The bedrock characteristics of the basin result in relatively high rainfall runoff and low groundwater contribution. As a result, the drier summer periods generate characteristically low to very low flows throughout the basin. Tributaries to the Chehalis River include the South Fork Chehalis, Newaukum, Skookumchuck, Black, Satsop, and Wynoochee Rivers.

4.2 Flood Event Hydrology

Flood events on the Chehalis are generally the result of warm frontal systems moving across the basin from southwest to northeast. These systems typically drive warm, moisture-laden atmospheric “rivers” from the equatorial regions of the Pacific Ocean to the Pacific Northwest. Rainfall in excess of 12 inches per day can result from these storms. Historic flood events such as the flood of record, which occurred in December 2007, are almost always the result of such atmospheric events. The 2007 storm generated rainfall intensities in excess of 1 inch per hour in some areas of the basin, and record amounts for 12- and 24- hour duration. Peak flows exceeded all previously recorded historic levels at a number of gaging stations throughout the basin. In particular, the USGS estimated that the peak discharge at the Doty gage reached 63,100 cfs, which was more than twice the previous record flow. Farther downstream, however, the peak flows were lower, with the peak flow at Grand Mound exceeding the previous highest flow by only 6 percent. The December 2007 event has been estimated to be greater than a 100-year recurrence interval at the Doty gage.

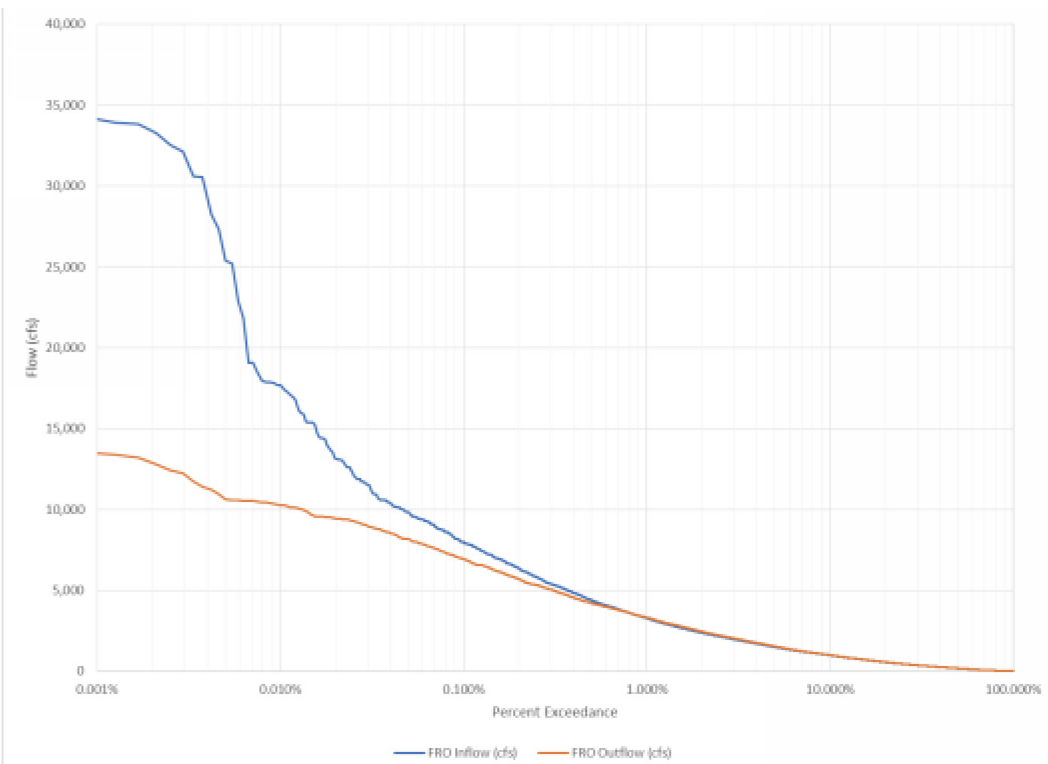
4.3 Site Hydrology

The period of record for the historical data begins in October 1988 and extends into 2015. Chehalis River flow at the proposed dam site (inflow) was estimated by Anchor QEA and WSE using the USGS gage flow data at Doty and scaling these to the dam site (Anchor QEA, 2016). Figure 4-1 presents the following flow exceedance curve and Table 4-1 presents the estimated peak flow magnitude for various return intervals from that same reference report.

4.3.1 Probable Maximum Flood

The Probable Maximum Flood (PMF) at the dam site is estimated to be 69,800 cfs (WSE, 2016), based on a HEC-HMS rainfall-runoff model of the tributary basin using values of probable maximum precipitation, including snow-melt, derived from the National Weather Service (NWS) Hydrometeorological Report No. 57 (HMR 57).

Figure 4-1
Flow Duration Curve at the Dam Site



Source: Anchor QEA 2016

Table 4-1
Estimated Peak Flow Magnitudes and Frequencies

RETURN PERIOD (YEARS)	PEAK FLOW (CFS)
2	6,920
10	13,061
20	16,053
100	24,223
500	35,688

Source: Table 4.2, Anchor QEA 2016

5 GEOLOGIC SETTING

5.1 Regional Geology

The Chehalis dam site is located on the northern edge of the Willapa Hills, a very large upwarped anticline, extending from the Columbia River on the south to the eastern reach of the Chehalis River Valley between Doty and Chehalis, Washington. The axis of the anticline is in the heart of the Willapa Hills near such topographic high points as Boistfort Peak and Little Onion (S&W, 2009a). Because the dam site is on the northern limb of the Willapa Hills anticline, the large gentle fold causes the volcanic and sedimentary rocks in this area to generally dip to the north at about 10 to 30 degrees. The core of the Willapa Hills consists of early Eocene-age, 49- to 56-million-year-old intrusive and extrusive mafic volcanic rocks that are typically moderately to very strong and form steep slopes. The mafic volcanic rocks consist of the Crescent Formation and gabbro that has intruded into the older volcanic and sedimentary rock. Overlying the Crescent Formation are siltstone and claystone of the McIntosh Formation, which is relatively weak, forming relatively gentle slopes (S&W, 2009a). The McIntosh Formation was deposited contemporaneously with the late stages of Crescent Formation deposition, resulting in siltstone lenses interbedded with pillow basalt flows of the Crescent Formation (Moothart, 1992).

Younger (last 2 million years) Quaternary surficial deposits of sediments such as stream alluvium, colluvium, and landslide deposits are found along the valley floor, and many slopes mask the underlying bedrock, except where overburden has been stripped by mass wasting or where slopes are too steep to develop soil cover. In addition to sedimentary deposits, most of the underlying bedrock is overlain by residual overburden soil that supports heavy vegetative cover. These Quaternary deposits range in thickness from 0 feet at outcrops and river channels to about 110 feet at the toe of the largest landslide in the area, and are typically about 30 to 40 feet thick.

No evidence of active faults has been found within the immediate vicinity of the potential dam site. A 100-foot-wide fault zone was noted by Shannon & Wilson, Inc. (2009a) to be about 800 feet upstream of the current potential dam alignment. This fault zone is described as bounded by low-angle (25 to 40 degrees), west-northwest trending faults and consisting of tightly folded beds of claystone/siltstone interlayered with gabbro and a 45-foot-wide zone of breccia. The fault zone was inferred to likely be contemporaneous with middle-Eocene (34- to 50-million-years-ago) intrusion of volcanic rocks. The geologic map by Wells and Sawlan (2014) indicates three high-angle faults near the dam site to the east, south, and west. These faults have not been directly observed in the field and are likely Tertiary Period faults that have not experienced movement since the middle Miocene age (10 to 20 million years ago). The closest active fault is the Doty fault, which is an east-west trending zone of fault strands 8 miles north of the dam site. For a more detailed discussion of faults, refer to the Phase 1 Site Characterization Technical Memorandum (HDR and S&W Inc., 2015).

5.2 Investigation Program

The geologic conditions at the potential Chehalis dam site have been evaluated to date through a two-phase site characterization process. The work performed as part of the Phase 1 program included the following:

1. Boring and geophysical investigation of foundation conditions within the potential dam footprint, including left and right abutments and valley bottom
2. Initial evaluation of the dam foundation investigation data to be used in updating the conceptual-level design configuration of the dam, outlet works, and spillway structures
3. Field evaluation of landslides in the left abutment area of the potential dam and around the perimeter of the reservoir
4. Seismic hazard assessment to identify earthquake hazards and related design criteria
5. Initial investigations of potential aggregate sources that could be used to construct an RCC dam

As is typical for geologic exploration programs, completion of the first phase of site characterization informed the design of the potential dam, but it also identified additional data needs. A Phase 2 site characterization was begun in early 2016 to address the data needs identified in the Phase 1 study and to update the conceptual design of the dam. The work performed for the Phase 2 site characterization included the following:

1. Additional boring and geophysical investigations of foundation conditions within the dam footprint and along potential dam structures such as outlet works and the construction diversion tunnel
2. Advanced evaluation of the dam foundation investigation data for updating the conceptual-level design configuration of the dam, outlet works, and spillway structures
3. Additional borings and geophysical investigations of landslides in the dam abutment areas and within the reservoir
4. Additional borings, geophysical investigations, and laboratory testing of materials from three potential aggregate sources (quarries) that could be developed and used to construct an RCC dam. Borings, seismic refraction tomography survey lines, and laboratory testing were completed at two potential quarry sites. The third potential quarry site is an inactive quarry where representative samples were available and obtained for lab testing
5. Instrumentation was also installed during the data collection process to facilitate monitoring of groundwater levels and to measure any active slope movement at inclinometer locations in key landslide areas. The readings from this instrumentation will be useful in subsequent phases of design and potentially through construction and during operation of the completed project if

the project is advanced. Phase 2 also identified more data collection needs to be conducted in future site characterization work

The work performed for the combined Phase 1 and 2 site characterization program has expanded the understanding of foundation conditions for the FRO or FRFA dam and reservoir configurations. This information has been utilized to update a 3D geologic model of the site and to support geotechnical and structural evaluations and analyses described later in this report. Concept-level designs have been updated based on these evaluations and analyses along with an assessment of probable construction cost. For further detail refer to the memorandum discussing Phase 2 site characterization report (HDR, 2017b). Overall findings from the combined site characterization program include the following:

- The site is complex from the standpoint of the design of a large and high-hazard RCC dam and the associated hydraulic structures (spillway, fish passage and flood control outlet works, water supply outlet works, and construction diversion abutment tunnel).
- Although the site is complex, no geologic characteristics were identified that would preclude the use of either the FRO or FRFA dam configurations.
- Although use of a rockfill dam design is still considered feasible, there are technical challenges that would be difficult to mitigate, and project costs would be higher than for the RCC design. The RCC dam type remains preferred for either the FRO or FRFA configurations.
- Three alternative quarry sites suitable to generate RCC and conventional concrete aggregate have been confirmed in reasonably close proximity to the site.
- The saddle dam design has been updated to a composite earthfill/rockfill configuration to address issues related to foundation bedrock materials obtained as part of the Phase 2 program.
- The rock structure information gathered during the combined Phase 1 and 2 site characterization work indicates that jointing and fractures orientations in the bedrock indicate a low risk of potential for sliding wedges and block surfaces beneath the dam and along temporary and permanent excavated slopes.
- Highly fractured zones have been identified in the bedrock beneath the dam. Foundation grouting to seal these zones and reduce seepage will be an important component of the dam design.
- Refined design, quantity takeoffs, and unit cost evaluations have resulted in an updated opinion of probable construction costs that is consistent with the estimates developed in support of the Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR, 2014) and presented in an interim cost estimate update provided in July 2016.

5.3 Dam Site Geology

5.3.1 Soil

Several types of Quaternary deposits overlay bedrock at the potential dam site. Landslides exist in the left and right abutment areas both upstream and downstream of the potential dam alignment and on the hill slope to the south (upstream) of the potential construction diversion tunnel and dam outlet works. The dam site landslide deposits consist of highly variable, mostly unsorted and unstratified debris that is often characterized by hummocky terrain. The soils range from sandy silt (MH) to clayey, silty sand (SC/SM) to gravel with silt and sand (GP-GM), and they can contain clasts ranging from gravel to boulders up to several feet in diameter. The four landslides at the dam site range from about 70 to 110 feet thick at the toe and will require either complete removal or adequate stabilization to mitigate failure risks that could cause damage to, or affect the operation of, the dam or appurtenant structures. A comprehensive study of both the dam site and reservoir landslides is provided in the Phase 2 Site Characterization Report (HDR September 2016)

Colluvium is present extensively along the hillslope that makes up the right abutment and less extensively mixed with residual soil on the left abutment. It consists of poorly sorted, loose to dense, sandy to gravelly clay or silt deposited on or at the base of hillslopes, primarily through gravity-driven transport of weathered rock and soil. These deposits may contain high percentages of subangular boulders consisting of basalt and gabbro ranging widely in size, and could be more than 2 feet in maximum dimension. Seismic refraction surveys suggest that these materials range from 0 to 25 feet vertical thickness along the upper slope of the right abutment.

Stream alluvium is located along the valley floors of the Chehalis River and its tributary streams and consists primarily of very loose to loose, stratified slightly 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. Qao tends to be denser than Qa, contains gravel, and is prominent in the dam site on the west bank of the Chehalis River beneath the center of the main dam alignment.

5.3.2 Bedrock

The foundation of the potential Chehalis Dam consists almost entirely of Crescent Formation basalt with Crescent Formation siltstone/claystone forming irregularly sized and shaped lenses within the basalt. Crescent basalts are often in the form of pillow basalt flows but can also be locally intrusive as gabbro. Several sequences of volcanism occurred during the deposition of Crescent 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 that were lithified to siltstone/claystone

(Tcs) and occasionally claystone breccias consisting of basalt clasts in a claystone matrix by subsequent events of pillow basalt flow deposition.

The Tcb ranged from weak to very strong. The strength of these materials increases 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, and the basalt is locally slightly vesicular. The Tcs encountered ranges from locally 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. Brecciated siltstone (and claystone) was observed within the dam vicinity. These materials were likely created as angular clasts of basalt that were eroded from the host rock between volcanic events and accumulated in depressions that were overlain with silt and/or clay.

The McIntosh Formation (Tml), deposited contemporaneously and subsequent to the Crescent Formation, represents a thick sequence of locally tuffaceous marine siltstone and claystone with interbedded arkosic sandstone and basaltic sandstone. Tml was observed outside of the dam footprint near the right abutment and is mapped extensively to the north of the site (Wells and Sawlan, 2014). In light of the close similarities between Tml and Tcs, it is possible that the Tcs found within the ridge below the FRFA saddle dam could actually be McIntosh Formation; however, the Tml and the Tcs have identical engineering material properties.

5.3.3 Structure

During the Phase 1 site characterization, four outcrop locations were found for mapping of joint sets. All of the outcrops consisted of Crescent Formation basalt. Joint strike measurements were analyzed utilizing an equal-area stereonet to evaluate joint sets. Three prominent joint sets (JS) labeled JS-1, JS-2, and JS-3 were identified in the surface outcrop areas. JS-1 was observed to have joint spacing of about 4 feet, and JS-2 was observed to coincide with the slope of the outcrop. During both Phase 1 and 2 site characterization, a downhole televiewer was used in most borings. The televiewer joint data were also analyzed on an equal-area stereonet, and two additional joint sets were identified, JS-4 and JS-5. A summary of all five primary joint sets is shown in Table 5-1.

Table 5-1
Summary of Identified Joint Sets from Outcrop Mapping and Televiewer Data

JOINT SET	STRIKE	DIP DIRECTION	DIP	DATA SOURCE
JS-1	75	165	70	Outcrop Mapping
JS-2	348	78	81	
JS-3	271	361	18	
JS-4	338	68	16	Televiewer
JS-5	125	215	53	

The bedrock structure data from both Phases 1 and 2 were used to perform a kinematic slope and foundation stability analysis using the methods described by Goodman (1980) to evaluate the potential for development of unstable blocks during excavation of the foundation, or beneath the dam during normal and flood operations. The information gathered so far at the dam site indicates that there is a fairly low chance of block failure development along excavation slopes or in the foundation of the dam; however, further data collection and analysis is needed during future design phases to more fully assess these risks. The full results of the kinematic analysis are presented in the Phase 2 Site Characterization TM (HDR 2017b).

6 SEISMIC DESIGN

The most important safety concern of concrete dams subjected to earthquakes is excessive cracking, which can lead to significant damage or potential instability from sliding or overturning. Sliding could occur on an existing plane of weakness in the dam foundation, at the foundation-dam interface, or along RCC lift surfaces within the dam. Although some major concrete dams have experienced strong ground motion with some damage, it is of note that there has been only one major concrete dam failure in recent times as a result of earthquake-induced ground motions. This failure was in Taiwan, where the dam was constructed directly on an active fault that experienced about 30 feet of vertical displacement during the earthquake. In general, instability of gravity dams caused by excessive cracking of the concrete is most likely to occur in the upper half of the dam.

Development of a dam cross-section that includes appropriate defensive design measures is required to address safety and earthquake performance concerns. Defensive measures for concrete dams might include the following:

- Adequate dam and foundation seepage control and drainage is typically the first line of defense against dam and/or foundation instability, in part because it is the most economical.
- Typical upstream and downstream slopes may be modified as needed to meet tensile stress and sliding stability design criteria. Use of a chimney section with a curved transition to the downstream slope of the dam can reduce tensile stresses in downstream dam face in the upper portion of the dam. The upstream face of the dam can be sloped to reduce hydrodynamic forces on the dam and tensile stresses that can develop at the upstream heel.
- RCC mixes, design provisions, and construction procedures may be implemented to achieve uniformity of the concrete materials and adequate direct tensile and shear strength without causing excessive thermal cracking problems.
- An appropriate excavation objective can be identified such that when combined with the other defensive measures, it results in an economical configuration. Similarly, the excavation profile along the axis of the dam should have minimum geometric irregularities and gradual variations in structural stiffness. Over-excavation and use of backfill concrete and shaping blocks in foundation defect areas can be effective measures to improve seismic performance in critical areas.
- Effective quality control during construction will achieve desired foundation preparation, strength of the concrete, and bonding of the dam to the foundation. Appropriate cleaning, preparation, and treatment of lift surfaces provides good bond for both seepage control and seismic performance within the body of the dam.

- Control joints and crack inducers can be incorporated into the design to control cracking and accommodate small differential displacements in a well-designed dam profile.

6.1 Project Seismic Hazard Setting

Project setting and hazards are described in more detail in the Phase 1 Site Characterization Technical Memorandum (HDR and S&W, 2015). The northern Cascadia forearc is shown on Figure 6-1 and is positioned within two tectonic convergence regimes that deform western Washington: east-west contraction across the Cascadia Subduction Zone (CSZ) and north-south shortening from the northward migration of forearc blocks. The combined effect of the two produces complex and diverse deformation within the northern edge of the Cascadia forearc that can trigger large, damaging earthquakes from multiple seismogenic sources in the western Washington region (see Figure 6-2).

Figure 6-1
Plate Boundaries

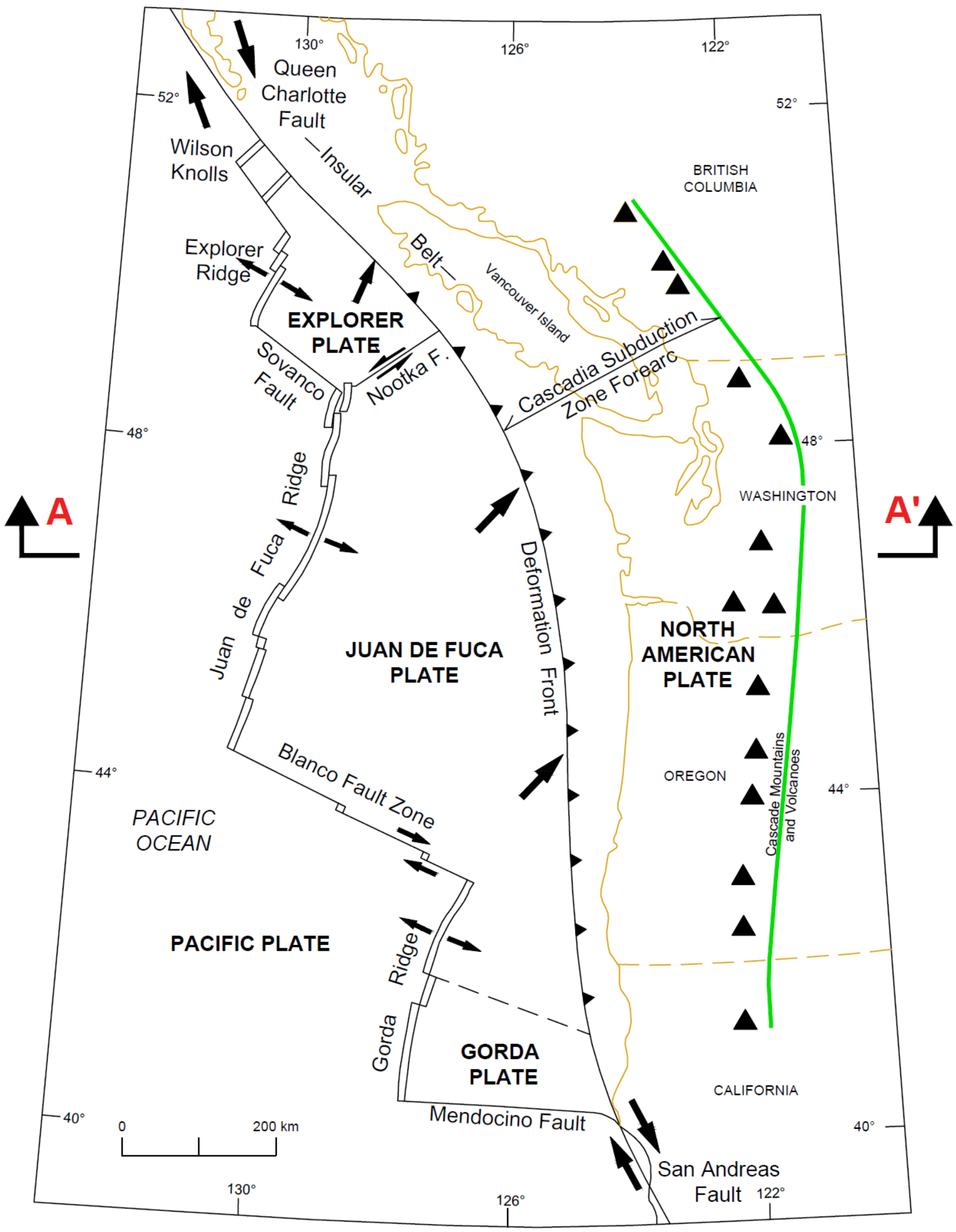
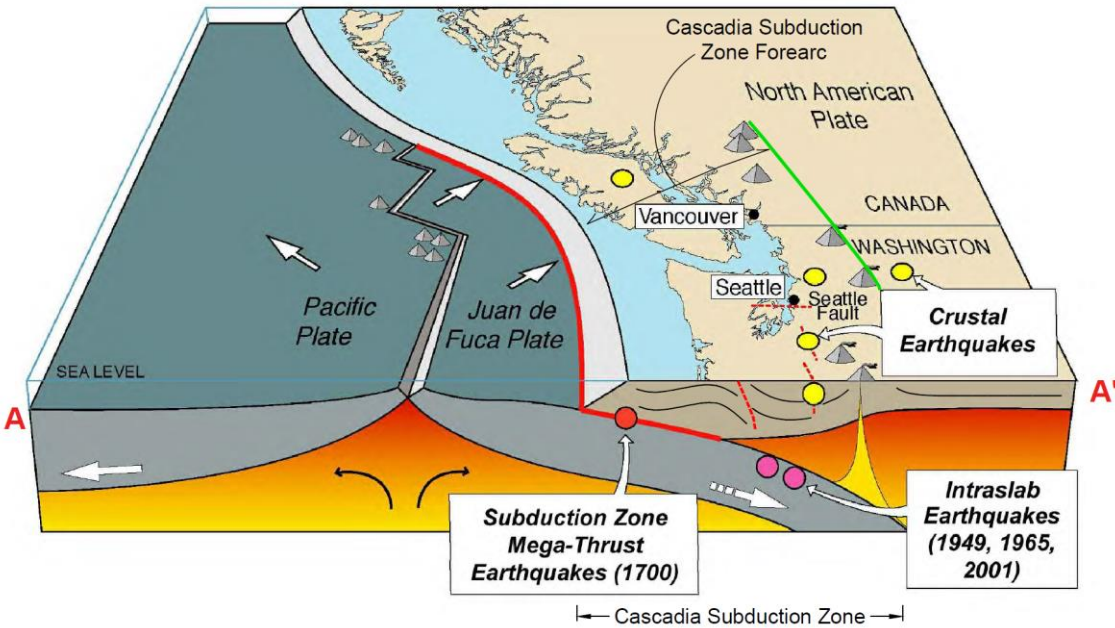


Figure 6-2
Typical Geologic Cross-section



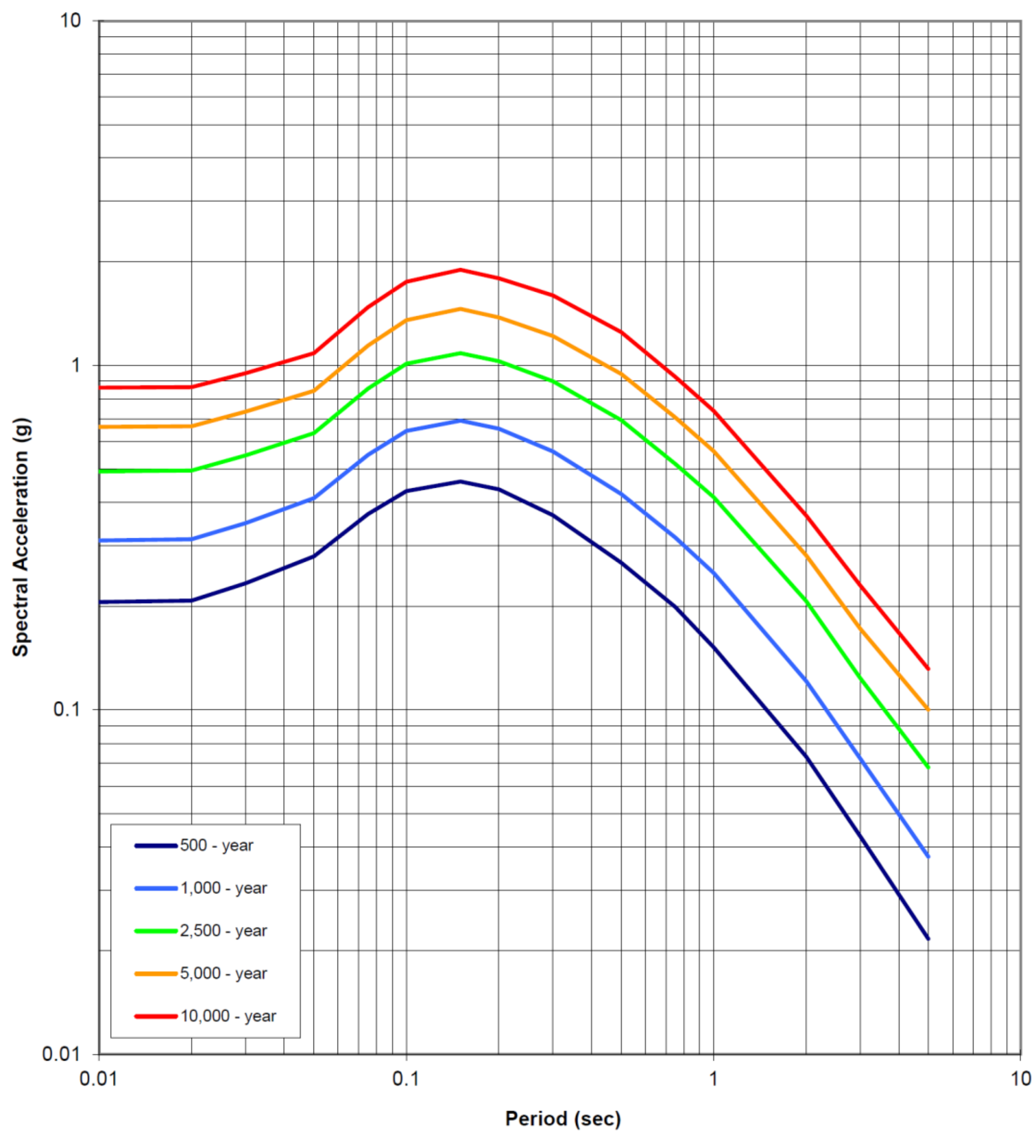
6.1.1 Seismic Hazards

The horizontal uniform hazard spectra (UHS) from the probabilistic seismic hazard analysis (PSHA) for estimated 500-, 1,000-, 2,500-, 5,000-, and 10,000-year return periods were developed and plotted, and represent the sum of the hazards from various regional seismic sources included in the seismic source characterization model for the Chehalis dam site (Shannon & Wilson, Preliminary Design Earthquake Time Histories, 2015). The estimated hazard curves from the PSHA for horizontal ground motion versus mean annual rate of exceedance or return periods are shown on Figure 6-3 and indicate, based on current knowledge, that the CSZ interface is the dominant contributor to the ground motion hazard at the site for return periods of 500 to 10,000 years.

A deterministic seismic hazard analysis (DSHA) was completed to estimate the ground motions from a specific seismogenic source at the site regardless of the rate at which earthquakes are generated from the source. In a DSHA, the various seismic parameters (e.g., fault type, rupture dimensions, and maximum magnitude) for each potential earthquake source are evaluated, and a Maximum Credible Earthquake (MCE) is determined for that source. Using the distance between the site and the source, the ground motions at the site for a given MCE were estimated, and the MCE source that produces the largest (strongest) ground motions at the site produces the Controlling Maximum Credible Earthquake (CMCE).

Figure 6-3
Horizontal Uniform Hazard Spectrums

CRD_UHS.xlsm 9/30/2015



NOTE

1. Uniform Hazard Spectra (UHS) values were determined for a soil condition with shear wave velocity of 1,500 meter per second for upper 30 meter (Vs30).

Preliminary Design Earthquake Time Histories
Chehalis Basin Flood Mitigation
Pe Ell, Washington

5% DAMPING
MEAN HORIZONTAL UHS

September 2015 21-1-21897-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

The results of the PSHA were used to identify and characterize earthquake sources that produce the MCE, locations of potential fault rupture, and the source mechanisms to estimate deterministic ground motions (i.e., spectra). Uncertainties in seismic source characterization are reflected in logic tree weights of the PSHA. However, in the deterministic approach, the MCE can be identified by selecting the most likely or "best estimate" for each source parameter (i.e., fault type, location, geometry, maximum magnitude, and source-to-site distance). The source parameters that are given the highest weight in PSHA are considered the most likely in defining the MCE for the deterministic analysis.

The potential MCEs evaluated for the current deterministic study are as follows:

- Moment magnitude (Mw) 8.9 CSZ interface earthquakes at source-to-site distances of 71 kilometers (km) and mean-plus-one standard deviation ground motions because of the relatively short recurrence interval (i.e., about 500 years) for these events
- Mw 7.5 CSZ intraslab earthquakes directly beneath the site at a distance of 43 km
- Mw 7.1 Olympia Fault events at a distance of 48 km from the site
- Mw 6.9 Doty fault events at a distance of 13 km from the site

Thirteen time history sets were developed (39 individual time histories) for use in conceptual design updates, and for preliminary design. Each time history set is composed of three time histories, two orthogonal horizontal components, and one vertical component.

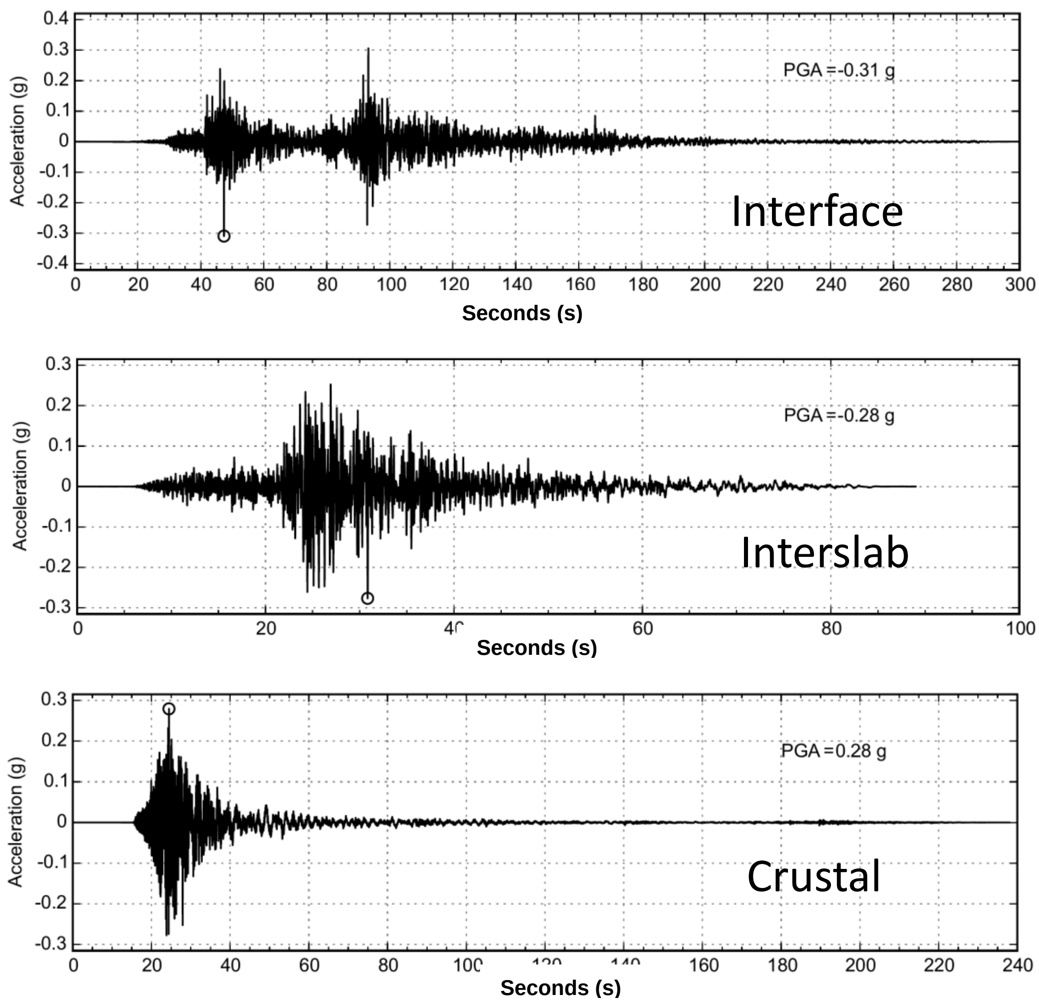
Specifically, time history sets were developed for the following:

- Ten sets corresponding to probabilistic ground motions with an estimated 2,500-year return period
  - Five sets matched to conditional mean spectra (CMS) conditioned at a 0.2-second period
  - Five sets matched to CMS conditioned at a 1.0-second period
- Scaling factors for the 10 probabilistic 2,500-year ground motions to scale them to other return periods ranging from 500 to 10,000 years
- Three sets corresponding to deterministic MCE ground motions for the following sources:
  - CSZ Mw 8.9 interface event (one set)
  - CSZ Mw 7.5 intraslab event (one set)
  - Shallow crustal Mw 6.9 Doty fault event (one set)

Examples of the interface, interslab, and crustal event time histories of horizontal ground motions are shown in Figure 6-4. It should be noted that conceptual design updates and preliminary design studies will not involve individual analyses using every ground motion in the data set. Instead, a number of the motions will be evaluated to identify which have the greatest influence on structural response. After these are identified, various structural analyses will be completed with those targeted motions.

Another important item must be noted relative to the earthquake hazards at the Chehalis dam site. As can be seen on the example ground motions on Figure 6-4, the duration of strong shaking is substantially different for the different hazards. A period of strong shaking of 140 to 180 seconds may occur for a CSZ event, while the corresponding duration of strong shaking may be 50 to 60 seconds for an interslab event and 20 to 30 seconds for a crustal event. The duration of shaking is directly tied to the length of fault rupture and the corresponding magnitude of the event.

Figure 6-4
Time Histories



6.1.2 Doty Fault Length Effect on Seismic Hazard

During the assessment of earthquake sources and the corresponding site ground motion hazard, the deterministic and probabilistic ground motions were performed using a Doty fault length of approximately 50 km. The Doty fault structure is visible on existing aeromagnetic and gravity anomaly data, and based on these data sets, researchers at the USGS and Washington State Department of Natural Resources (DNR) have questioned whether the fault might extend farther west than currently mapped, to near the Washington coast.

The USGS has recently acquired a new high-resolution aeromagnetic data set of the area and will use that, along with newly acquired LiDAR data, to closely analyze the fault length and geometry of the Doty fault in the future. Additionally, researchers at the USGS and DNR are planning field campaigns to identify active structures and potential recurrence rates along the Doty fault zone. These studies are forthcoming, and no results are available yet.

If the Doty fault extends farther to the west, it would be capable of producing larger-magnitude earthquakes. As described in our ground motion hazard study, the maximum magnitude of Mw 6.9 was estimated for a 50 km long Doty fault. If the fault length increased to 100 km, the corresponding maximum magnitude would increase to Mw 7.2. Extending the fault farther to the west does not change the nearest site-to-source distance; for either a 50 km long or 100 km long fault, the site-to-source distance remains 13 km.

A preliminary assessment has been made of the potential impact to the site ground motion hazard should the future USGS/DNR studies support the hypothesis that the Doty fault is longer and capable of generating larger earthquakes. For the deterministic motions, the increased Doty fault length/magnitude ground motions are still lower than the CSZ interface, and the CSZ interface would remain as the controlling deterministic ground motion hazard source. Probabilistic motions could increase by a few percent if the future USGS/DNR studies support a longer, active Doty fault. The following provides a description for the basis of these assessments.

- Deterministic Ground Motion Assessment: For the deterministic hazard assessment, the deterministic response spectrum for a longer 100 km fault with a larger magnitude Mw 7.2 event was estimated. The deterministic response spectrum for the hypothesized 100 km long Doty fault is about 10 to 30 percent larger than that of the shorter, 50 km long fault. However, the controlling (i.e., larger) deterministic spectrum is still the CSZ interface, which is about 50 to 175 percent larger than the spectrum for the hypothesized 100 km long Doty fault.
- Probabilistic Ground Motion Assessment: For the probabilistic hazard assessment, we have not rerun the PSHA for a long Doty fault length. A review of the results of the PSHA suggests that for the range of ground motion return periods from 500 years to 10,000 years, the percent of the total ground motion hazard contributed from all the modeled shallow crustal faults (i.e., Doty, Olympia, Saddle Mountain., Grays Harbor, Willapa Bay, and Gales Creek) ranges from less than 1

percent to no more than 3.5 percent. The largest contributor to the total ground motion hazard by far is the CSZ interface, at about 50 to 98 percent of the hazard. Because of the lack of evidence of Holocene rupture, the Doty fault was assigned an activity weight of 0.3 (i.e., a 30 percent probability that it is active) in the probabilistic seismic hazard analyses performed in 2015. If the USGS/DNR studies determined that the fault has experienced Holocene rupture, the activity weighting would change from 0.3 to 1.0 (i.e., 100 percent probability that it is active). Increasing the activity weighting factor would tend to increase the hazard contribution from the Doty fault. However, given the relatively low contribution from the modeled shallow crustal faults to the total ground motion hazard (i.e., less than 1 to 3.5 percent), the expected increase in the probabilistic ground motion hazard associated with a 100 km Doty fault length would likely be less than 10 percent, probably on the order of only a few percent of the total hazard.

7 DAM STRUCTURAL DESIGN

7.1 RCC Dam Design Load Cases

Design of concrete dams typically involves evaluation of a range of loading conditions. For example, the load cases outlined by USACE EM 1110-2-2200, Gravity Dam Design, June 1995, include the following:

1. Load Condition No. 1 – unusual loading condition – construction
   A. Dam structure completed
   B. No headwater or tailwater
2. Load Condition No. 2 – usual loading condition – normal operating
   A. Pool elevation at top of closed spillway gates where spillway is gated, and at spillway crest where spillway is ungated
   B. Minimum tailwater
   C. Uplift
   D. Ice and silt pressure, if applicable
3. Load Condition No. 3 – unusual loading condition – flood discharge
   A. Pool at standard project flood (SPF)
   B. Gates at appropriate flood-control openings and tailwater at flood elevation
   C. Tailwater pressure
   D. Uplift
   E. Silt, if applicable
   F. No ice pressure
4. Load Condition No. 4 – extreme loading condition – construction with operating basis earthquake
   A. Operating basis earthquake (OBE)
   B. Horizontal earthquake acceleration in upstream direction
   C. No water in reservoir
   D. No headwater or tailwater
5. Load Condition No. 5 – unusual loading condition – normal operating with operating basis earthquake
   A. Operating basis earthquake
   B. Horizontal earthquake acceleration in downstream direction
   C. Usual pool elevation
   D. Minimum tailwater
   E. Uplift at pre-earthquake level
   F. Silt pressure, if applicable
   G. No ice pressure

6. Load Condition No. 6 – extreme loading condition – normal operating pool levels with maximum credible earthquake
   A. Maximum credible earthquake
   B. Horizontal earthquake acceleration in downstream direction
   C. Usual pool elevation
   D. Minimum tailwater
   E. Uplift at pre-earthquake level
   F. Silt pressure, if applicable
   G. No ice pressure
7. Load Condition No. 7 – extreme loading condition – probable maximum flood
   A. Pool at probable maximum flood
   B. All gates open and tailwater at flood elevation
   C. Uplift
   D. Tailwater pressure
   E. Silt, if applicable
   F. No ice pressure

Based on review, the cross-section requirements for the FRO and FRFA configurations will be controlled by loading conditions with the highest water levels and strongest earthquake (Load Conditions 6 and 7). A risk-informed design process has been used to confirm the cross-section requirements of the dam. This process included development of a 2D finite element model 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 was used to estimate the potential for, and magnitude of, sliding along the base of the dam.

Additional load cases from the suite of study cases outlined by USACE will be performed during preliminary design of the preferred project configuration.

7.2 Stability and Structural Analyses

The post-earthquake stability analysis performed in support of design development for the FRO and FRFA dam cross-sections verified that sufficient sliding resistance was available to meet the minimum required post-earthquake sliding stability FOS. 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. The dam cross-section configurations for both the overflow (spillway) and non-overflow sections of the dam shown on the plans provided in Appendix A meet all established geotechnical/structural design criteria and would have a very low probability of failure following a major earthquake event. Additional refinement of the material properties used in the stability analysis, as well as updated stability calculations, will be required as the design proceeds through subsequent phases of design work. Detailed documentation of the stability analysis performed for the FRO and FRFA configurations and design criteria is provided in Appendix D.

7.2.1 Basic Gravity Analysis

A basic gravity analysis was performed to evaluate FRFA dam stability for the scenario of water standing at normal pool elevation (628 feet) and for the scenario of the PMF flowing through the spillway (687 feet). The FRO dam was not evaluated because the FRFA dam configuration would be in the critical condition (the FRO top-of-dam elevation is lower than that of the FRFA, resulting in reduced uplift and horizontal loading on the dam). The conclusion of the gravity analysis was that for the normal pool and PMF water elevations, the non-overflow and overflow sections of the dam remain in compression along the base. The FRFA dam as shown in the drawings in Appendix A meets foundation compression criteria under normal pool and flood loading conditions.

7.2.2 Earthquake Analysis

Response spectra analysis was performed for 500-, 2,500-, 5,000-, and 10,000-year return periods with water at the assumed normal pool elevation (628 feet). Based on an assumed 45-degree friction angle, the response spectra analyses indicated the potential for sliding in both upstream and downstream direction during earthquakes with instantaneous minimum FOS of about 0.5 for the 10,000-year return period event. Consequently, the analyses suggest that base cracking at both upstream and downstream toes would occur.

Foundation drains are proposed along the axis of the dam to reduce foundation pressure and thereby reduce uplift on the dam. Following sliding, it is possible that the drains will be ineffective and that a residual shear strength condition may exist along the foundation/rock interface. It is also possible that a crack could extend across the entire base of the dam. Because of the potential impact of earthquakes on the effectiveness of the drain system and resulting uplift pressures, “no drain” and “with drain” conditions were evaluated for the dam site. An extreme uplift scenario was also evaluated, which assumes full upstream water pressure below the dam extending to the downstream toe of the dam. The response spectra analysis for these three uplift scenarios yielded estimated post-earthquake FOS for sliding under normal pool loading conditions. With an assumed friction angle of 45 degrees, post-earthquake FOS are 1.8 for the “no drain” condition and 2.1 for the “with drain” condition. For the extreme uplift scenario and a friction angle of 30 degrees, the post-earthquake FOS is 1.1. A friction angle of approximately 40 degrees would be required to ensure a post-earthquake FOS exceeding 1.5.

The FRFA dam would have acceptable factors of safety for project earthquake stability criteria.

7.2.3 Earthquake Sliding Analysis

A non-linear time-history analysis was performed to evaluate the effects of earthquakes over time assuming an interface friction angle between the foundation and the base rock of 45 degrees. Assuming a rigid base, this analysis shows sliding of about 0.43 feet for the 2,500-year return period, 1.07 feet for the 5,000-year return period, and 4.91 feet for the 10,000-year return period. Assuming a flexible base, this analysis shows sliding of less than 0.05 feet for the 2,500-year return period, 0.08 feet for the 5,000-year return period, and 0.53 feet for the 10,000-year return period. The actual friction angle between the foundation and base rock could be higher or lower than the assumed value of 45 degrees and should be confirmed using direct shear testing with representative concrete and bedrock materials.

The FRFA dam meets project criteria for sliding.

7.2.4 Post-Earthquake Uplift Analysis

A post-earthquake factor of safety was calculated for three uplift conditions (Normal, Full, and Extreme) and for friction angles that ranged from 30 to 55 degrees in 5 degree increments. The “normal” uplift condition was calculated based on USACE EM 1110-2-2200 Gravity Dam Design, 1995. “Full” uplift was calculated assuming a linear distribution from full head at the upstream heel to the downstream tailwater condition. “Extreme” uplift was calculated assuming full upstream head exists all the way to the downstream toe. For all but the 30 and 35 degree friction angles for the extreme uplift condition, the factor of safety was above 1.5. For the lower friction angles and the extreme uplift condition the factor of safety remained above 1.0.

8 HYDRAULIC DESIGN

8.1 Introduction

This section summarizes the hydraulic design criteria, reservoir storage and flow capacities, and the descriptions and hydraulic characterizations of the outlet structures: spillway and the spillway chute; flip bucket and plunge pool; outlet works; stilling basin. More detailed information on the hydraulic design is included in Appendix B.

8.2 Design Criteria

Table 8-1 below summarizes the design criteria used for the hydraulic design of the FRO and FRFA dam alternatives.

Table 8-1
Hydraulic Design Criteria

PARAMETER	DESIGN CRITERION	COMMENT/REFERENCE
Probable Maximum Flood (PMF) also the Project Inflow Design Flood (IDF)	69,800 cfs	PMF, as required by Washington State Dam Safety Guidelines (WSE, 2016)
Flood Regulation Storage	65,000 AF	The equivalent flood volume of the December 2007 flood event of record (Anchor QEA, 2014)
Flow Augmentation Storage	65,000 AF	FRFA only (Anchor QEA, 2014)
Low Level Flood Regulation Outlet Works Flood Routing Flow Capacity	15,000 cfs at reservoir EL 550; total for all conduits combined	Minimum flow capacity of low level flood control outlets needed to release the full equivalent flood storage volume of the 2007 flood of record hydrograph back into the river within one week
Low Level Flood Regulation Outlet Works Maximum Regulated Flow Capacity	15,000 cfs at reservoir EL 628 (FRO), or EL 687 (FRFA)	Maximum flow capacity required at full flood pool under gate-controlled conditions
Maximum Fish Passage Flow	2,200 cfs	5 % exceedance flow; unrestricted fish passage for all flows up to 2,200 cfs
Minimum Fish Passage Flow	30 cfs	95 % exceedence flow
Minimum flood flow release	250 cfs	Minimum flow release during flood regulation operations
Minimum water quality outlet works flow	500 cfs	Each outlet is capable to discharge 500 cfs with a minimum of 35 feet of submergence.
Stilling basin design flow	15,000 cfs	Flow at reservoir flood elevation (FRO = 628 feet; FRFA = 687 feet)

8.3 FRO

8.3.1 Design Flows and Storage

The design flow capacity for the FRO alternative spillway is identical to that for the FRFA alternative. Both dams must be able to pass the estimated PMF of 69,800 cfs (WSE, 2016), as required under the Washington State Dam Safety Office guidelines without overtopping the dam crest. The spillway is designed to pass at least 69,800 cfs and up to a total of 75,000 cfs with no dam crest overtopping. It should be noted that WSE revised their estimate of the PMF event downward to 69,800 cfs from 75,000 cfs after preliminary design had been accomplished (WSE, 2016). The low-level outlet works for the FRO alternative are also designed to release the 2007 flood event equivalent storage volume in the reservoir back into the river at a rate that would restore full capacity within one week.

Unlike the FRFA alternative, however, the FRO typically would allow water from all minor high-flow events up to about 6,000 cfs to be passed through the dam with the sluice gates fully open, unless flood regulation operation is commenced in response to larger flooding concerns downstream. All sediment and most small debris would pass through the dam unimpeded. The sluices have been designed to provide sufficient capacity at these smaller flow events to prevent developing backwater upstream of the sluices for flows up to and above the high fish passage flow (2,000 cfs).

For the FRO alternative the reservoir is normally empty; that is, there is no reservoir, and the river flow passes unimpeded through the dam sluices at all times, until and unless a flood regulation operation is initiated. Flood regulation operation would be initiated whenever the downstream river flow at the Grand Mound gage site is forecast to increase above 38,000 cfs within 48 hours. Flood regulation operations commence by gradually closing the low-level sluice outlet gates to reduce the outflow to the minimum value of 250 cfs over the specified downstream ramping rate period. The sluices would remain closed until the Grand Mound gage flow is predicted to fall below the control value of 38,000 cfs within the travel time from the proposed dam to the Grand Mound gage. The flood storage capacity of the reservoir is expected to store all inflows until the flood regulation operation ends, following which the stored volume would be released over a period of up to a week or more.

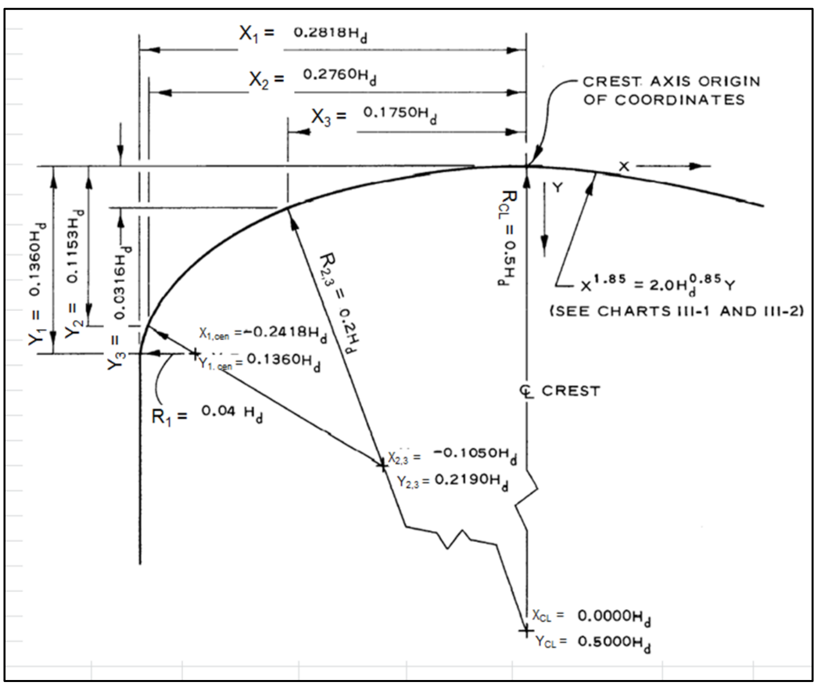
The total available flood storage reservoir volume between the existing river water surface elevation and elevation 628 ft msl for the FRO alternative is 65,000 AF. This volume is equal to the estimated volume of the December 2007 flood event, which was estimated as a 300-1000 year event. Spill flow would not be expected to occur for flood events of lesser magnitude than the 2007 event.

8.3.2 Spillway and Spillway Chute

The FRO spillway is an uncontrolled ogee crest, discharging to a smooth-faced conventional concrete chute cast over the top of the RCC mass dam section, identical to that of the FRFA alternative. The crest is set at elevation 628 ft msl with a width of 200 feet, and is designed to pass up to 69,800 cfs with a minimum of 4 feet of freeboard to the top of the upstream dam crest parapet wall. The equivalent unit discharge at 69,800 cfs design capacity is 350 cfs per linear foot, and 375 cfs/ft at 75,000 cfs. As with the FRFA alternative, the design discharge capacity has been conservatively estimated using a slightly lower discharge coefficient (C_d = 3.84) than is typically found for smooth ogee designs, to ensure adequate capacity without risk of overtopping. Design guidance utilized in the design of the crest shape included USACE EM 1110-2-1603, Hydraulic Design of Spillways; the USACE Hydraulic Design Criteria (HDC); and the USBUREC Design of Small Dams.

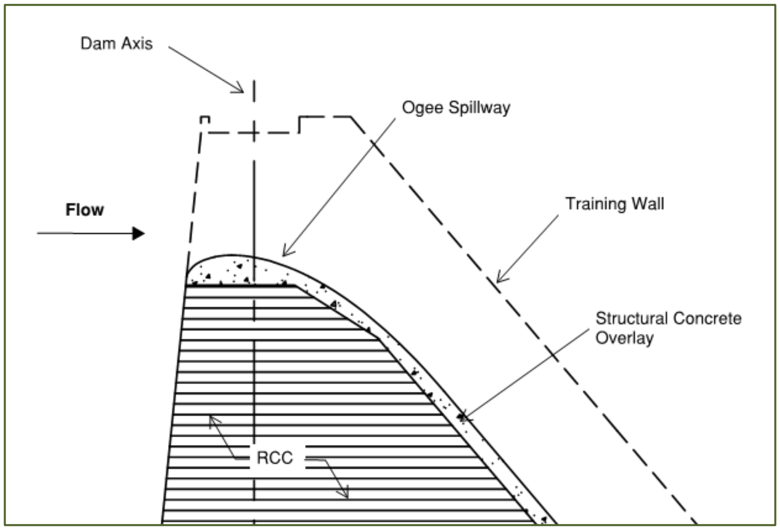
The FRO crest shape has been designed for driving head (H_d) of 30 feet, though the maximum anticipated effective head (H_e) achieved by the spillway under the PMF event is only about 22 feet. This "overdesign" permits the ogee shape to be cast on top of the underlying RCC dam structural outline and reach tangency with the overall downstream dam structure slope with approximately 3 feet of concrete overlay. The crest shape shown in Figure 8-1, below, is used for both the FRO and FRFA designs. Likewise, the same structural concrete overlay construction shown in Figure 8-2 is assumed for both the FRO and FRFA alternatives.

Figure 8-1
USACE Hydraulic Design Criteria 111-2/1 Design of Ogee Crest Shape



It is assumed that the RCC construction will proceed in lifts of approximately 1 foot, which would leave a finished concrete face with 1-foot steps at the design face slope of 0.85H:1V. The chute design assumes a structural overlay of concrete on the ogee crest and the face of the chute. Appropriate doweling and structural reinforcement would be required to securely anchor the structural concrete overlay to the RCC dam structure (Figure 8-2 below).

Figure 8-2
Spillway Crest and Chute Design



8.3.3 Flip Bucket and Plunge Pool

The FRO spillway is expected to be used very rarely, and for events of short duration. Therefore, no spillway stilling basin is provided. Rather, a flip bucket and preformed impact plunge pool will be constructed to dissipate the energy of spillway flows. As is the case for the FRFA alternative, the reservoir modeling conducted to date indicates that spill events are likely to occur with recurrence of 100 to 500 years. Small spill discharges would be expected to cascade from the lip of the flip bucket and pass across the rockfill material to the river below, contained on one side by the low wall adjacent to the stilling basin.

The flip bucket design is based on a minimum unit discharge of 350 cfs/foot of width and a maximum of 375 cfs/ft at maximum spillway flow with the bucket invert at elevation 475 ft msl and the lip at elevation 490 ft msl. The flip bucket design followed the same USACE guidance as the FRFA alternative. The flow profile down the spillway chute was evaluated using the same methods, with the result that at maximum discharge the toe velocity is about 100 feet per second and depth of about 4 feet, yielding a minimum bucket radius of 43.6 ft. However, we have used the same 50-foot radius for both the FRFA and FRO alternative flip bucket designs. Trajectory calculations determined an approximate river impact zone about 375 feet downstream of the lip for the FRO dam alternative. The rockfill design below the flip bucket would be developed during the next phase of the study.

8.3.4 Outlet Works

The FRO design has three low-level sluice outlets, consisting of a single large 12 ft wide by 20 ft high sluice at invert elevation 408 ft msl and a pair of 10 ft wide by 16 ft high sluices at invert elevation 411 ft msl. A large, full height trashrack extending from the riverbed to the dam crest will exclude most large trees from the sluice conduits and provide excess open area under all reservoir elevations to pass the desired project outflows. The larger gate will be used to pass the majority of sediment bedload in the river, as well as most small debris. Some sediment is expected to pass through the higher sluice conduits as well, but the lower sluice will intentionally receive the most wear from sediment passage over time. Repairs to the sluice floor are expected every few years to bring the sacrificial concrete floor surface back to original grade.

With all three FRO low-level flood regulation sluices open, up to approximately 8,600 cfs can be passed through the sluices with reservoir elevation at 427 ft msl without transitioning to orifice or pressure flow in any of the sluice opening. The 15,000 cfs maximum regulated project outflow can be passed at reservoir elevations greater than about 450 ft msl with wide open gates. The maximum 15,000 cfs project outflow can be passed entirely through the pair of 10 ft by 16 ft sluices at reservoir elevation of about 550 ft msl, with both gates controlling flow at 75% open setting. Typical flood regulation operation would initiate closure of the larger sluice at any time the pool levels exceed about elevation 500 ft (msl), to prevent excessive wear on the large sluice floor due to bed sediments entrained in high velocity flow. The higher gates in the pair of 10 ft by 16 ft sluices are expected to entrain considerably less bed load sediment, though the specific elevation details to confirm this and establish the final higher sluice gate seat elevation would have to be evaluated using a physical scale model. Mud Mountain Dam on the White River in western Washington (owned by USACE) is designed similarly, and its three outlet sluices operate much like that proposed for the FRO design alternative.

The FRO’s pair of 10 ft by 16 ft sluice gates pass flow into parallel conduits separated by a center dividing wall terminating about 100 feet downstream of the gate seats. Downstream of the divider wall, the outflows from both gates would combine into a 22-foot-wide by 16-foot-high single conduit. A parabolic drop of about 30 feet in the floor of the sluice conduit carries the discharge into the downstream stilling basin floor at an elevation of 381.2 ft msl.

The large lower radial gate has a width of 12 ft, a height of 20 ft, and a radius of about 44 ft. The two smaller radial gates have a width of 10 ft each, a height of 16 ft, and a radius of about 35 ft. Hydraulic cylinder operators for each gate would ensure positive closure under all flow conditions. Side seals and top seal would either be inflatable using reservoir static water pressure, or else the gate trunnion would be provided with an eccentric rotator to compress the top seal. Both sealing types have been used with success in high head applications such as this, as indicated above. Radial gates were selected for the FRO design as it is important that there are no side gate slots in which sediment might accumulate and jeopardize the closure ability of the primary control gates for the dam.

The two smaller sluice gates are designed to pass up to 5,000 cfs each with 61 feet of static head on each gate opening at the 75 percent open setting (12 feet), while the larger gate can pass the same 5,000 cfs with 29 feet of static head on the gate opening at the 75 percent open setting (15 feet). This ensures that the full 15,000 cfs desired sluice discharge capacity is available at reservoir elevations as low as 462 ft msl in a fully controlled manner with all gates at 75 percent open setting, which is about 166 feet below the spillway crest.

At full flood storage reservoir elevation of 628 ft msl, each of the two smaller sluice gates at 75 percent open can pass up to about 9,500 cfs, and the larger gate can pass up to about 14,200 cfs alone at 75 percent open gate condition. As with the FRFA design, the paired design of the two smaller gates was selected to ensure that finely controlled flood regulation would be available with a single gate as needed, given that the larger gate will likely be closed and one of the two smaller gates might also be closed. Adjustment of a single 10 ft wide gate in 6-inch typical lift increments gives about 380 cfs per increment at the maximum flood regulation reservoir elevation of 628 ft msl. The ability to closely control downstream flows is important for both the FRO and FRFA alternatives, so that required ramping rates can be achieved. Flood regulation operation would include operation of the sluices at reservoir elevations up to the spillway crest of 628 ft msl. At reservoir elevation above the spillway crest, sluice operation may be curtailed to avoid adverse flow conditions within the stilling basin.

As in the FRFA design, each sluice conduit is provided with an emergency bulkhead gate a few feet upstream of the radial gate, as well as dewatering bulkheads at the inlet to the sluice and the outlet to the sluice. The emergency bulkhead gate would be a vertical panel, likely a roller gate, with hydraulic operator, and is designed to close under full flow at maximum reservoir elevation. The upstream and downstream dewatering bulkheads are simple, vertically hung panels that are designed to close under no flow. They are provided to isolate and dewater each sluice conduit so that inspections and repairs can be accomplished under safe working conditions.

8.3.5 Stilling Basin

The stilling basin for the FRO design receives all sluice flows at all times. For design purposes, the stilling basin has been designed for the full flood pool (elevation 628 ft msl) regulated outflows of up to 15,000 cfs maximum design discharge from the two 10 ft by 16 ft sluice gates. Discharge from the large 12 ft x 20 ft sluice would be curtailed at reservoir elevations above about 500 ft (msl), hence the basin design for the higher heads passed by the two smaller sluices would readily accommodate the lower energy flow passed by the large sluice. The design flow velocity entering the basin is approximately 125 feet per second, with a Froude number of about 10.8 at entry. Since the flow jet would expand quickly following entry into the wider basin, the design energy would be less than the initial entry value. Following USACE guidance, a baffled stilling basin length of approximately 230 ft is obtained, assuming a 70-foot width overall. The end sill elevation was selected to be commensurate with the natural bedrock-controlled stream bed elevation of about 417 ft msl, and the width of 70 feet provides a water surface elevation of about 433.5 ft at the full sluice outlet discharge of 15,000 cfs. The downstream conjugate depth at 15,000 cfs with the expanded jet is approximately 52 ft, yielding a basin floor elevation of about 381.2 ft, which provides adequate energy dissipation within the basin. HEC-RAS modeling of the natural downstream channel (described in more detail in Appendix B) indicates that the natural water surface at the end sill location is about 422 ft msl at the maximum stilling basin capacity of 15,000 cfs, ensuring hydraulic control by the end sill, since submergence of the elevation 417 ft msl end sill is just 5 feet against a driving head of 16.5 ft.

8.4 FRFA

8.4.1 Design Flows and Storage

The design flow for the FRFA alternative spillway is identical to that for the FRO alternative. Both dams must be able to pass the estimated PMF of 69,800 cfs (WSE, 2016), as required under the Washington State Dam Safety Office guidelines without overtopping the dam crest. The spillway is designed to pass at least 69,800 cfs and up to a total of 75,000 cfs with no dam crest overtopping. It should be noted that WSE revised their estimate of the PMF event downward to 69,800 cfs from 75,000 cfs after preliminary design had been accomplished (WSE, 2016).

The low-level outlet works for the FRFA alternative are also designed to release the 2007 flood event equivalent storage volume in the reservoir back into the river at a rate that would restore full capacity within one week.

The multiport water quality outlet works are designed to pass up to 500 cfs from any reservoir level within the flow augmentation pool, which typically varies from elevation 588 to 628 ft msl (585 to 628 ft msl with climate change considered). Each 48-inch-diameter conduit will pass more than 500 cfs with a minimum of 35 feet of submergence below the reservoir surface. They are designed to accommodate multiple withdrawal levels within the flow augmentation reservoir pool as needed to manage downstream release water temperatures. One large, low-level port is 84 inches in diameter and is designed to pass more than 800 cfs from the lowest level of the flow augmentation reservoir pool, in case additional quantities of cool stored water are required to meet downstream water temperature needs.

The FRFA alternative is designed for a water supply storage capacity of 65,000 acre feet between reservoir elevation 425 ft msl and 628 ft msl, and a temporary flood storage capacity of 65,000 acre feet above reservoir elevation 628 ft msl up to the spillway crest elevation of 687 ft msl. The flood storage capacity is the equivalent runoff from the December 2007 flood event, which has been estimated at greater than 500-year recurrence interval at the Doty gage site. Flood regulation operation would be initiated whenever the downstream river flow at the Grand Mound gage site is forecast to increase above 38,000 cfs within 48 hours. Flood regulation operations commence by gradually closing the low-level sluice outlet gates to reduce the outflow to the minimum value of 250 cfs over the specified downstream ramping rate period. The sluices would remain closed until the Grand Mound gage flow is predicted to fall below the control value of 38,000 cfs within the travel time from the proposed FRO Chehalis dam to the Grand Mound gage. The flood storage capacity of the reservoir is expected to store inflows until the flood regulation operation ends. Spill events would not be expected to occur until the flood storage volume of 65,000 acre feet is exhausted and the reservoir rises to the spillway crest elevation of 687 ft msl.

8.4.2 Spillway and Spillway Chute

Like that of the FRO, the FRFA spillway is designed as an uncontrolled ogee crest, discharging to a smooth-faced conventional concrete chute cast over the top of the RCC mass dam section. The crest is set at elevation 687 feet msl with a width of 200 feet, and is designed to pass up to 69,800 cfs with a minimum of 4 feet of freeboard to the top of the upstream crest parapet wall. The equivalent minimum unit discharge at 69,800 cfs design capacity is 350 cfs per linear foot, and 375 cfs/ft at 75,000 cfs. The design discharge capacity has been conservatively estimated using a slightly lower discharge coefficient (C_d = 3.84) than is typically found for smooth ogee designs, to ensure adequate capacity without risk of overtopping. Design guidance utilized in the design of the crest shape included the USACE EM 1110-2-1603, Hydraulic Design of Spillways; the U.S. Army Corps of Engineers' HDC; and the USBUREC Design of Small Dams (see Figure 8-1 above).

For the FRFA design, the crest shape has been designed for a maximum head of 30 feet, though the maximum anticipated effective head achieved by the spillway under the PMF event is only about 22 feet. This "overdesign" permits the ogee shape to be cast on top of the underlying RCC dam structural outline and reach tangency with the overall downstream dam structure slope with approximately 3 feet of concrete overlay (see Figure 8-2 above).

8.4.3 Flip Bucket and Plunge Pool

As is the case for the FRO spillway, the FRFA spillway is expected to be used very rarely, and for events of short duration. Therefore, the extra expense of constructing a conventional concrete energy dissipation basin for spillway flows at the toe of the dam is unnecessary, and instead a flip bucket design with preformed plunge pool in the bedrock structure downstream of the toe of the dam is preferable. Based on the geology of the site, the downstream rock formation appears adequate to provide occasional spillway flow dissipation. Based on the reservoir modeling conducted to date, spill events are likely to happen for 300-year to 1,000-year recurrence interval events. Small spill discharges would be expected to cascade from the lip of the flip bucket and fall onto the rockfill material at the spillway toe adjacent to the sluice outlet stilling basin structure. Additional design refinement in the next phase of the project may include a more detailed evaluation of erosion protection for the rockfill adjacent to the sluice stilling basin. At this stage, a low containment wall about 5 feet high directs these minor spillway flows across the rockfill material adjacent to the stilling basin and to the river channel below.

The flip bucket design is based on a maximum unit discharge of 375 cfs/foot of width at maximum spillway flow, with the bucket invert at elevation 475 ft msl and the lip at elevation 489.6 ft msl. The flip bucket was designed according to guidance provided in USACE EM 1110-2-1603, Hydraulic Design of Spillways, as shown in Figure 8-3 below. The approach depth of the nappe at the flip bucket toe was calculated by two methods, the first using a standard step water surface profile down the chute, and the second using simply the potential energy of the available hydraulic head from the reservoir level to the flip bucket toe. The methods developed comparable results, with a maximum nappe depth at the bucket toe at the larger design discharge of 375 cfs/ft of about 3.4 feet, a flow velocity of about 117 feet per second, resulting in a minimum design bucket radius of 51.6 feet. A bucket radius of 50 feet was selected for simplicity. Simple trajectory calculations were made based on the USACE guidance to determine an impact location approximately 400 to 500 feet downstream of the lip. For unit discharges between about 50 cfs/ft and the design unit discharge of 375 cfs/ft, the impact zone would be confined to this area. For unit discharges less than 50 cfs/ft and nappe depths of less than about 1 foot, the jet would impact nearer to the dam toe as a result of the much greater proportion of energy loss accumulated down the chute. The rockfill design would accommodate unit discharges of up to about 50 cfs per foot without entrainment or plucking of stones. The specific gradation of the stone surface material has not been calculated in this conceptual design analysis but will be included as a refinement during the next design phase.

Figure 8-3
Spillway Flip Bucket Design (USACE EM 1110-2-1603, Hydraulic Design of Spillways)

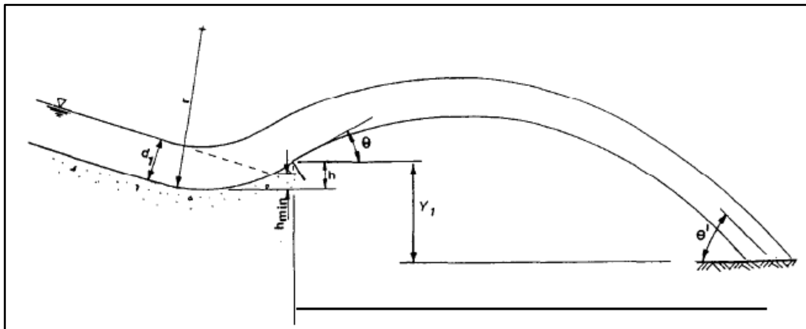


Figure 7-2. Parameters used in the design of a flip bucket

8.4.4 Outlet Works

The FRFA design alternative low-level outlets consist of a pair of 10 ft wide by 16 ft high sluices at invert elevation 420 at the base of the dam. A large, full height trashrack protects these sluice gates from entraining large debris that cannot be passed through the dam. As described above, the low-level flood regulation sluices are designed for a maximum controlled discharge of 15,000 cfs at any reservoir elevation within the full operating range of the project (reservoir elevation 588 ft msl to 687 ft msl). The two sluice gates would be separated by a center divider wall terminating about 100 feet downstream of the gate seats. Downstream of the divider wall, the outflows from both gates would combine into a 22 ft wide by 16 ft high single conduit. A parabolic drop of about 43 feet in the floor of the sluice conduit transitions the flow into the downstream stilling basin floor at elevation 377 ft (msl).

The radial gates have a width of 10 feet, a height of 16 feet, and a radius of about 35 feet. Hydraulic cylinder operators would ensure positive closure under all flow conditions. Side seals and top seal would either be inflatable using reservoir static water pressure, or else the gate trunnion would be provided with an eccentric rotator to compress the top seal. Both sealing types have been used with success in high head applications such as this (up to 285 feet). The bottom seal is compressed by the weight of the gate and the operator down force. Radial gates were selected for several reasons, listed below:

- They reduce the gate operator load by transmitting the hydrostatic forces to the trunnion bearing instead of as friction on a slide gate seating surface
- They eliminate gate slots, which in a sediment- and debris-rich environment can cause problems in fully seating the gate
- They are more reliably and positively controlled than cable-hung vertical gates at these heads
- They do not suffer from pressure regime shifts at small gate openings due to the lip geometry as do vertical gates

The sluice gates are designed to pass up to 7,500 cfs each at the 75 percent open setting at reservoir elevation 559 ft (msl) and as little as 133 feet of static head on each gate. This ensures that the full 15,000 cfs desired sluice discharge capacity is available at reservoir elevations as low as 559 ft msl in a fully controlled manner, which is about 29 feet lower than the typical normal low reservoir pool elevation. At full flood storage reservoir elevation of 687 ft msl, each gate at 75 percent open can pass up to 10,500 cfs. The paired design was selected to ensure that fairly precise flood regulation was available with a single gate as needed. Adjustment of a single 10 ft wide gate in 6-inch typical lift increments gives just 330 cfs per increment at the minimum typical reservoir elevation of 588 ft msl. This is important in controlling downstream releases during the flood regulation operation so that required ramping rates can be achieved. Flood regulation operation would include operation of the sluices at reservoir elevations up to the spillway crest of 687 ft msl. At reservoir elevation above the spillway crest, sluice operation may be curtailed to avoid adverse flow conditions within the stilling basin. Each sluice conduit is provided with an emergency bulkhead gate a few feet upstream of the radial gate, and dewatering bulkheads at the inlet to the sluice and the outlet to the sluice. The emergency bulkhead gate would be a vertical panel, likely a roller gate, with hydraulic operator, and is designed to close under full flow at maximum reservoir elevation. The upstream and downstream dewatering bulkheads are simple vertically hung panels that are designed to close under no flow. They are provided to isolate and dewater each sluice conduit so that inspections and repairs can be accomplished in safe working conditions.

8.4.5 Stilling Basin

The stilling basin for the FRFA design receives flood regulation outflows from the two 10 ft by 16 ft sluice gates, up to a design discharge of 15,000 cfs at maximum reservoir elevation at the spillway crest elevation of 687 ft msl. The design flow velocity entering the basin is approximately 140.4 feet per second, with a Froude number of about 12.6 at entry. Since the flow jet would expand quickly following entry into the wider basin, the design entry energy would be less than the initial entry value. Following USACE guidance, a baffled stilling basin length of approximately 230 ft is obtained, assuming a 70-foot width overall. The end sill elevation was selected to be commensurate with the natural bedrock-controlled stream bed elevation of about 417 ft msl, and the width of 70 feet provides a water surface elevation of about 433.5 ft at the full sluice outlet discharge of 15,000 cfs. The downstream conjugate depth at 15,000 cfs with the expanded jet is approximately 56 ft, yielding a basin floor elevation of 377 ft, which provides adequate energy dissipation within the basin. HEC-RAS modeling of the natural downstream channel (described in more detail in Appendix B) indicates that the natural water surface at the end sill location is about 433.5 ft msl at the maximum stilling basin capacity of 15,000 cfs, ensuring hydraulic control by the elevation 417 ft msl end sill, since submergence of the end sill is just 5 feet against a driving head of 16.5 ft.

The multiport low-flow outlet conduits would discharge through individual valves into the stilling basin from a valve setting above the maximum expected regulating flow stilling basin water surface elevation of 433.5 ft msl. We anticipate these valves would likely be of the hollow cone type, such as a Howell-Bunger design, or perhaps fixed-cone or plunger type valves. The design of the discharge valves for the multiport outlets will be refined in the next phase of designs. For cost estimation purposes, we have assumed Howell-Bunger valves will be selected.

9 MATERIALS

9.1 Aggregate Sourcing

The suitability of three quarry sites was investigated (see Figure 9-1 for locations). Material from all three sites was tested. Although Quarry sites 1 and 3 are preferable from a transportation standpoint, Quarry 3, known as the Rock Creek quarry, is the only one with material that tested favorably in terms of strength, durability, specific gravity, and absorption. Further quarry investigations and a comprehensive mix design program are required to confirm the acceptability of either Quarry 1, 2, or 3. Table 9-1 summarizes the quality testing that has been completed.

Figure 9-1
Quarry and Aggregate Sources

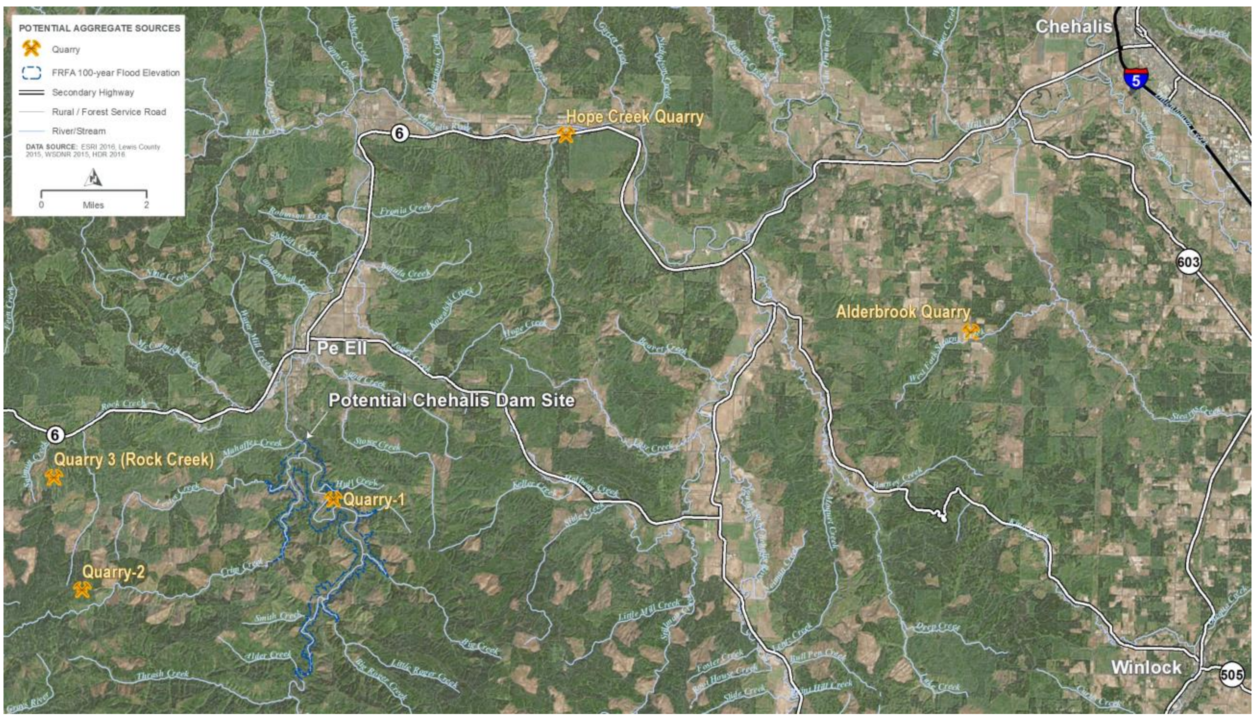


Table 9-1
Quarry Material Testing Results

QUARRY SITE	NO. OF SAMPLES	AVERAGE RESULTS			
		COMPRESSIVE STRENGTH (PSI)	ABSORPTION %	LA ABRASION LOSS %	ALKALI SILICA REACTIVITY 16-DAY AVG LENGTH CHANGE %
Quarry 1	3	4,278	6.47	27.1	0.093
Quarry 2	2	11,433	3.88	24.45	0.035
Rock Creek	1	NA	1.37	18.9	0.035

If supplemental investigation and mix design program results are favorable, Quarry 1 or other nearby basalt may be used for RCC aggregate. Alternatively, nearby basalt quarries and potentially commercial basalt quarries could be utilized. Regarding site-developed aggregate sourcing, it may be important to understand how the quarries might be developed and how processing would be approached.

The basalts may contain surface and subsurface weathering. Potentially, the rock quality within a deposit might also exhibit varying quality – in terms of abrasion resistance, specific gravity, absorption, etc. Consequently, the mining and processing is likely to include:

- Stripping of overburden and highly weathered basalt
- Benched quarry development, for example 10 vertical to 1 horizontal or completely vertical walls. Benches would be installed periodically (8-12 feet wide) at a vertical spacing of every 20-40 feet.
- Unacceptable formation rock would be disposed of
- Primary jaw crushing would perform the initial material processing
- Smaller material would be eliminated (1 – 1½ inch minus materials) after passing through the jaw
- Secondary cone crushing
- Potential tertiary crushing HIS or VSI (horizontal or vertical shaft impact) for better particle shape and to crush more fine sands
- Predominantly dry screening would be used to sort materials by sizes favorable for recombination into the materials needed for the project (RCC, conventional concrete, etc.)

The Site Characterization Report (HDR 2016) evaluates the various quarry options. A preliminary look at developing Quarry 1 for the FRFA alternative led to an assumed development of 20 acres, removal of about 40 feet and 1,300,000 cubic yards (cy) of overburden, a nominal 80-foot benched basalt cut depth, and production of about 2,800,000 tons of aggregate in 22 months to construct 1,550,000 cy of RCC in 12 months.

Site-developed quarry operations for the FRO will be similar in terms of quarry development, with the potential that lower quality aggregate may potentially be used (because of the lower dam height and lower stresses in the concrete) if verified in a mix design program. Although quantities for the FRO alternative will be significantly less, the efficiency of Quarry 1 might be reduced because the lower rock demand still requires removing a thick overburden layer (40 ft). The RCC plant and crushing plant could be reduced with the lower RCC quantity (750,000 cy for the FRO versus 1.3 million cy for the FRFA).

9.2 Mix Design

The RCC mix will be preliminarily designed as a medium-cementitious RCC (MCRCC) mix, anticipating 200-250 pounds per cubic yard (#/cy) cementitious content with 50 percent fly ash replacement; utilizing local or regional 1- to 1½-inch maximum-size aggregate (msa) basalt; targeting 2,100 to 3,600 pounds per square inch (psi) unconfined compressive strength and a unit weight over 142 pounds per cubic foot (pcf). Refer to Appendix E, which contains more detail on the aggregate, mix design, and mix design program. A well-graded aggregate with low to moderate fine content and set retarding admixtures will be used to create a consistency favorable for grout-enriched RCC (GERCC) faces and to facilitate good workability and lift joint cohesion. The GERCC is a component of the upstream seepage control strategy and will help to produce a durable and aesthetically acceptable downstream face.

Basalt aggregates from quarries within or near the reservoir perimeter, and preferably close to the dam, are hoped to be used for the RCC. Preliminary investigations suggest sufficient deposits exist to amply meet the aggregate needs. Early quality testing shows acceptable durability, but high absorptions, variable and low strengths, and marginal to low specific gravities suggest a preliminary classification of Quarries 1 and 2 to be poor to marginal. Conceptual-level costs should conservatively consider higher-quality aggregates coming from perhaps Quarry 3 and/or the commercial sources, Alderbrook and Hope Creek, which all exhibit acceptable durability as well as good specific gravities and low absorptions. Alternatively, more cementitious material might be needed to get acceptable design strength with the marginal basalt. At concept-level, it seems highly likely that the site or nearby basalts would be economical and could satisfy modified design requirements. Consequently, it seems important to consider further investigation, a well-developed mix design program, and identification of potential areas that can be included in environmental surveys and considerations. Figure 9-1 identifies the potential aggregate sources.

Cement, Class F fly ash, and slag supply are currently available in the region with regional and national manufacturers and distributors including CalPortland, Lafarge, Ash Grove, and Lehigh. Regional fly ash may come from TransAlta's Centralia, Washington, coal-fired power plant (which has a contract with Lafarge for use of fly ash generated from their facility), but often supply is eliminated or greatly reduced during high regional hydropower production. Consequently, Canadian fly ash is also available, through CalPortland's Portland rail terminal, supplied by a facility in Genesee, Alberta. Further coal-fired plant generation reductions are anticipated, which will make fly ash an even tighter commodity. In addition to regional cement mills, CalPortland supplies Asian cement from its parent, Tokyo-based Taiheiyo Cement Corp. Currently, high-quantity cement and fly ash are similarly priced, roughly $110/ton at distribution terminals in Portland and about $130 at the Chehalis site. Facing potentially tight and diminishing fly ash supply in the years ahead, preliminary design and the Mix Design Program should consider evaluating thermal and seepage control design alternatives. For example, consideration should be given to whether tighter joint spacing, more aggressive batched RCC temperature control, or external upstream face membranes are acceptable alternatives to including fly ash, and if so, exploring the pricing sensitivity for when these options become appealing.

A strong mix design program is anticipated to verify whether site aggregates can meet design objectives and to evaluate a range of cementitious content and determine the sensitivity of fly ash reduction mitigation strategies. Adiabatic heat-rise or other thermal-specific testing may be evaluated if strong consideration is given to lessening or eliminating pozzolan from select mixes. Additionally, the mix design program will provide a confident basis for cost estimating and specification development, both serving to help manage procurement and construction risk.

Comparing the FRO with the FRFA highlights a few RCC design considerations. For the FRO, long-term reservoir storage is not an operational design requirement, and lower head and shorter load exposures could allow some RCC design adjustments. The RCC mix might allow for lower unconfined compressive strengths, which may in turn allow for lower quality aggregates and/or lower cementitious quantity. The need for seepage control might be limited to that required for dam and foundation stability without concern for water loss and lift joint cohesion, and even RCC impermeability may be less important. Together, these might allow wider joint spacing and fewer seepage control features, as well as a reduced need for admixtures. Understanding of the potential flexibility of design options increases confidence in the assumption of basing conceptual-level costs on Quarry 1. Finally, RCC placement is simplified to the degree that the dam height is reduced and to the degree that the integration with river outlet and fish passage systems is simplified. For design planning, the mix design program could be less rigorous for the FRO.

9.3 Facing Elements

GERCC forms the design basis for both the upstream and downstream dam faces; the upstream for seepage control, and the downstream for form control, durability, and aesthetics. Although GERCC is an established facing alternative worldwide, it is constructed with less precise quality control measures than conventionally cast-in-place facing. Quality challenges include placing and aligning face drains and waterstops, and ensuring consistent grout application and internal consolidation to foster appropriate wet and cured properties tied to water/cement ratio, slump, density, and strength. Quality issues can contribute to unplanned seepage, less effective capture of face seepage, and localized uncontrolled cracking around the dam vertical joints. Conventional concrete facing elements placed concurrently with the RCC are less risky for seepage control, and they warrant consideration in developing upper boundaries to the cost estimate to protect the cost estimate if GERCC is not ultimately selected during design.

Conventional reinforced concrete is contemplated for the downstream sloping spillway and can be either cast in place concurrently as an RCC facing element, or placed subsequently as a second-stage concrete after the dam has been completed. If it is placed concurrently, reinforcement and anchors must be cast into the RCC placement, and tight surface tolerances are hard to achieve and maintain. Numerical and physical modeling may be required to assess the configuration and required surface characteristics of the spillway. If conventional reinforced concrete is placed subsequently, the RCC surface must be prepared by removing loose and lightly consolidated RCC from the sloping face and drilling and grouting anchors if anchors are not cast into the RCC during placement.

Conceptual-level planning considers GERCC upstream face, GERCC downstream nonoverflow faces, and a second-stage cast-in-place conventional spillway surface. For the FRO, we have assumed the GERCC facing systems as well.

9.4 Contraction and Lift Joints

The conceptual design anticipates RCC construction, full-width, from abutment to abutment vertical progression, placing RCC in multiple shifts, 5-7 days per week, effecting nearly continual placement. Dam contraction joints are anticipated to be on 70-foot centers, composed of formed and caulked vertical face joints, backed by bond breaker to a formed face drain 12 to 18 inches downstream of the face. Downstream of the formed drain, a bond-breaking plate will be installed the full width of the RCC (upstream to downstream) in every other lift, creating weakened planes to guide the dam cracks in controlled locations from upstream to downstream. If diversion or other construction sequencing dictates a monolithic approach – building multiple segments of the dam in vertical blocks rather than in a full-lift, abutment-to-abutment configuration – the foundation will be evaluated for potential adverse differential deformation.

The near continuous 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 and free of debris, delaying the initial set of the RCC, and maintaining a moist (saturated surface dry [SSD] to slightly wetter) surface before subsequent RCC is spread and compacted is critical to achieving good bond between lifts, creating cohesion, and allowing for tensile stress across joints. Consequently, the MCRCC 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 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 good bond between lifts. Cold joint treatments also will be incorporated when weekly placement or other scheduled interference causes delay, or after breakdown, weather events, or other unplanned delays or lift damage has occurred. Concept-level design and cost estimates have assumed that approximately 10 percent of the lift surfaces will require a level of joint preparation.

Depending upon stability analyses and to what degree joints serve as water retention seepage management, the joint design may potentially be less robust for the FRO than for the FRFA.

9.5 Other RCC Design Considerations

Although finer design details may not be critical at the concept design level, these considerations can still be relevant and useful at this stage of project development. The concept-level spillway design is a smooth, overtopping spillway chute with an energy dissipating flip bucket. The non-overflow downstream slope can either be constructed in a step configuration or a sloping configuration. Steps are much easier to construct and therefore faster and more economical. If a step configuration is used, step height should be evaluated aesthetically and for public safety. For example, if 2-foot lifts are used on a 0.85:1 downstream slope, the dam face bears a significant fall hazard. The face is still accessible if 6-foot steps are utilized, but the fall hazard reduces to a single step or two, and a 6-foot step's visual scale may be more in line with Chehalis' size. Though not anticipated for the concept design, seismic stability analysis might lead to a radius downstream slope to vertical chimney transition.

Structure interference and transitions are important design considerations that should be evaluated on a case-by-case basis. For example, the outlet pipe encasement should be designed with a batter on free walls and, where possible, located against the foundation rock on one side. If an encasement must cross the RCC independent of an abutment, sufficient room for equipment placement and compaction should be allowed between the abutment-side wall and the foundation. Gallery adits create similar zones where an RCC lift could easily be split from access, creating small placement and compaction areas that add time and cost and make it difficult to ensure quality construction. Structures, as with foundation anomalies, should be evaluated as potential stress concentrations, often necessitating adding or relocating a dam joint to best control the dam cracking location.

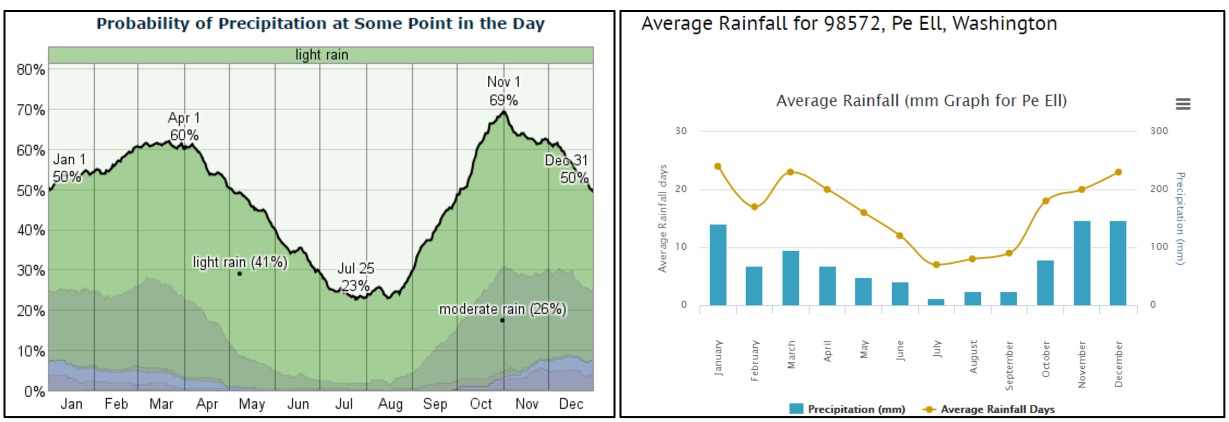
The intake access tower, which is anchored to the sloping upstream face, must be constructed after the RCC. Access during construction, particularly for cranes, is important and could affect sequencing as well as cost. Finally, diversion sequencing is an important consideration in the RCC design. The concept-level approach calls for nominal and high-frequency flood flows to bypass the work area through the diversion tunnel, providing enough time to construct the RCC dam as a whole. If the construction risk is evaluated to be too great, consideration needs to be given to how the dam and RCC design would change if the diversion was designed to stage the river flows through the work area, routed first around the permanent river outlet works, and then, after that portion of the dam was complete, through the river outlet works until the remainder of the dam is completed.

10 CONSTRUCTION CONSIDERATIONS

10.1 Construction Phase Flood Risks

Rainfall and the resulting runoff will likely affect construction operations, schedule, and cost. Figures 10-1-and 10-2 provide rainfall data for Chehalis and Pe Ell, Washington. As shown in these figures, the 6-month period of October through March have the highest probability of rainfall. More than 20 percent of all days from mid-October through March have moderate rainfall. Three of the months within this 6-month period (November, December, and January) average more than 10 inches of rainfall; four months have more than 20 rainy days per month. For seven months of the year (October through April) more than 50 percent of days have rain. Although light rainfall (less than 0.10 inches/hour) might not present problems for RCC, even brief periods of more moderate rainfall could prompt stoppages, which often involve costly delay and cleanup. Freezing weather and snow is rare in the area and is not likely to be a significant design or construction issue.

Figure 10-1
Rainfall Data for Chehalis (Left) and Pe Ell, Washington (Right)



Sources: https://weatherspark.com/averages/29924/Chehalis-Washington-United-States, http://us.worldweatheronline.com/v2/weather-averages.aspx?q=98572

Hydrologic data were used to evaluate peak flow rates that would be likely to occur during the construction period. It is assumed at this conceptual design phase that diversion of 2 to 5-year recurrence interval flows would be appropriate. The 2 year event results in ta flow of about 6,900 cfs, while the 5-year event results in a flow of 9,800 cfs at the dam site.

10.2 Temporary Flow Diversion

Construction diversion is arguably the highest risk construction component of the project, in terms of both cost and schedule. Constructing the diversion is critical-path work, as is much of the work that relies on that diversion. Foundation excavation and preparation, curtain and consolidation grout programs, lower-level intake and river outlet structures, stilling basin, lower-level fish passage structures, and the RCC through at least elevation 480 ft are all concentrated in the dam footprint, and each is highly dependent on the others and on remaining free from flooding.

It is assumed that a 20 ft modified horseshoe-shaped tunnel (developed during previous design evaluations) would carry water past the construction site. This tunnel dimension was determined to be a practical size that could be cost effectively advanced using drill and blast techniques and conventional mining equipment. An upstream cofferdam would direct upstream water into the diversion tunnel causing flows to bypass the construction site. The use of this diversion would minimize construction delays due to runoff, improve construction site safety, minimize damage potential to the foundation area under construction, and reduce construction cost increases due to damages and delays. A range of upstream diversion cofferdam elevations and compositions were evaluated.

Varying the upstream cofferdam elevation provides different degrees of protection from flooding during construction. Table 10-1 indicates the flows that can be accommodated through the diversion tunnel (with 3 feet of freeboard on the upstream cofferdam) by varying the upstream elevation. The upstream cofferdam crest elevation of 465 ft msl was selected for conceptual design to provide up to about 7,000 cfs diversion capacity through a partially to fully lined construction diversion tunnel. This corresponds to approximately a 2.8-year recurrence interval flow event.

Table 10-1
Flow Recurrence Interval Table for Diversion Tunnel Capacity at Upstream Cofferdam

RIVER DISCHARGE (CFS)	RECURRENCE INTERVAL (YRS)	RIVER WS ELEVATION AT U/S COFFERDAM (FT, MSL)	U/S COFFERDAM ELEVATION (FT, MSL)
13,061	10	506	509
9,870	5	478	481
6,920	2	457	460
4,606	1.25	446	449

Note: 3 feet of freeboard assumed

A much smaller downstream cofferdam would similarly be constructed to protect the construction area for the stilling basin and fish collection channel. However, because the downstream cofferdam is well downstream of these features, and the receiving river reach is fairly steep, the downstream backwater conditions are expected to be mitigated considerably For example, a similar 2-year-recurrence interval level of protection with 3 feet of freeboard requires a cofferdam elevation of just 416.35 ft msl, only 6 feet above the riverbed elevation at the tunnel portal. Table 10-2 below shows approximate downstream cofferdam elevations and flow recurrence interval. The downstream cofferdam crest has been set at 435 ft msl, more than high enough to prevent backwater from the construction diversion tunnel discharge from entering the dam foundation construction area, but also high enough that it can be used as an access road across the river channel to reach the left bank construction area.

Table 10-2
Flow Recurrence Interval Table for Downstream Cofferdam

RIVER DISCHARGE (CFS)	RECURRENCE INTERVAL (YRS)	RIVER WS ELEVATION AT D/S COFFERDAM (FT, MSL)	D/S COFFERDAM ELEVATION (FT, MSL)
12,502	10	417.16	420.16
9,870	5	415.65	418.65
6,580	2	413.35	416.35
4,606	1.25	412.32	415.32

Note: 3 feet of freeboard assumed

RCC was identified as the material that would most appropriately balance cost and durability considerations for the upstream cofferdam. An upstream cofferdam built from RCC would be resistant to erosion in the event that flows during construction exceeded the capacity of the diversion tunnel. The RCC option would also provide better access into the excavated footprint. The cofferdam and diversion tunnel capacity will meet the risk-based design approach used by the USBUREC as well as the requirements of the DSO. Overtopping of the downstream cofferdam presents less of a risk since inundation of the berm toe would not result in cofferdam failure. Because the downstream cofferdam would be much smaller, rockfill could be more cost effective than RCC.

10.3 Diversion Sequence

Diversion of the river would be accomplished by first driving a 20-foot-diameter tunnel through the right abutment bedrock formation, beginning at a point about 500 feet upstream of the future dam axis and terminating at a point about 850 feet downstream of the future dam axis. It will be constructed by conventional drill and blast methods. After the tunnel is completed, including lining of the invert, a shallow sump about 5 feet deep would be excavated in the upstream river channel roughly at the head of the steep chute drop section about 500 feet upstream of the dam axis to same elevation as the tunnel portal invert at about elevation 427 ft msl.

An RCC cofferdam would be raised immediately downstream of this shallow sump up to an elevation of about 465 ft msl. This elevation should provide protection against overtopping for flows up to about 7,000 cfs (approximate 2.8-year return period). However, a flow of approximately 4,750 cfs is expected to have a water surface elevation coincident with the crown of the diversion tunnel, which is the limit of tunnel capacity before it transitions to orifice flow. Higher flows will submerge the tunnel entrance, but the cofferdam has sufficient freeboard to pass considerably more flow, as noted above. The cofferdam design should consider means to easily raise the cofferdam from 465 ft msl through 480 ft msl, depending upon final design, contract approach, and risk allocation.

Both FRO and FRFA dam options share the same diversion scheme, in that a single diversion tunnel would be advanced through the right abutment from a diversion point several hundred feet upstream of the future dam axis to an outlet point downstream of the future fish CHTR facility. The invert of the tunnel would be lined with concrete (shotcrete) up to the spring line to protect against rock erosion. Contrary to the previous concept for the diversion tunnel, the flood control gates would be located within the dam section and would be paired with dedicated outlet sluices leading to a stilling basin downstream of the dam toe, rather than permanently operated from within the diversion tunnel. Following construction, the tunnel would be plugged and a small-diameter outlet valve would be constructed in the lower third of the plug to provide for a future low reservoir outlet drain or emergency pool lowering outlet in case of damage to the flood control gates or equipment. An adit tunnel leading to this plug section and the low-level valve operation system would be driven from the downstream toe of the dam as shown in the drawings. The adit would provide maintenance vehicle and foot access to the valve and tunnel downstream of the plug.

A summary of the entire diversion sequence, further developed in Section 15 – Construction Schedules, is as follows:

- Construct the tunnel portals
- Drive the tunnel from the downstream
- Divert the river and construct the upstream and downstream cofferdams
- Excavate and prepare the dam foundation, emphasizing the right for outlet construction
- Construct the outlet works foundation and lower levels, including necessary foundation grouting
- Perform curtain grouting and consolidation grouting, if required
- Place and complete the RCC
- Complete outlet works intake and gate construction
- Complete stilling basin and downstream fish passage construction
- Place tunnel bulkheads, and route river through outlet works
- Construct tunnel plug

Contract breakdown, risk tolerance, and contract risk allocations each will be considered in determining the final design parameters for the diversion cofferdam. Although an initial cofferdam height might be set at 465 ft, design and contract provisions would be well served to facilitate a higher cofferdam and assign the dam contractor responsibility for flows less than at least a 10-year recurrence interval.

10.4 Alternative Diversion Approach

Consideration has been given to an alternative diversion approach that routes the river through a bypass excavated through the lower-left abutment, allowing potentially greater flow capacity. Work would be completed on the full right abutment, outlet works, and the lower right half of the dam while the river is routed through an open-cut left abutment channel. When the right-side work was complete and the outlet works was ready for flow, the left abutment excavation, foundation work, and dam would be constructed while flows were passed through the outlet works. Advantages to this approach would include potentially greater diversion flow during the right-side construction, avoidance of underground construction, and lessening of the upstream and downstream right-bank portal interference with other features. Some disadvantages would be a reduced routing capacity during the left-side construction; greater potential to encounter unexpected foundation conditions near the area where right and left work meet; and split RCC and dam construction, potentially increasing both time and expense. Nevertheless, alternative diversion approaches might be worth designing or allowing, depending on the contract delivery approach and planned risk allocation.

10.5 Construction Access

Construction access from left to right banks for the RCC dam construction is proposed across both the upstream and downstream cofferdam structures. The existing right bank (facing downstream) upstream access roadway is at about elevation 465 ft msl, and the tunnel upstream portal crown elevation is about elevation 447 ft msl. Therefore, it would be logical to construct the upstream cofferdam to elevation 465 ft msl so that access is available across the cofferdam without approach grades, which would provide approximately 2.8-year recurrence interval protection for the upstream portion of the work area. The downstream right-bank access roadway and the downstream left upper-bank bench are at about elevation 466 to 470 ft msl, which is well above the 5-year cofferdam protection level of about 419 feet. Therefore, the downstream access roadway could drive the cofferdam elevation much more than the level of protection against overtopping. An elevation of 435 ft msl has been selected at this phase of design. A concept-level approach to construction site access and staging local to the dam itself is included in the drawings in Appendix A.

Upstream and downstream diversion cofferdams connect access roads to the left abutment and reservoir and downstream river banks. Primary project staging above the right abutment takes advantage of more favorable topography. Although contractors may evaluate other options, staging concrete and RCC operations above the left abutment works well with Quarry 1, Quarry 3, and commercial aggregate deliveries. Access road grades should be designed with reasonably constant grades, generally limited to 10 percent except where construction equipment and off-road haul units can safely navigate steeper slopes. For example, it might be necessary to construct steeper access out of the dam footprint where the upstream cofferdam and the downstream features limit space. Although more room upstream of the footprint might be more accommodating to construction activities, the upstream cofferdam has been located where outcropping basalt provides strong confidence that a good upstream portal can be constructed with minimal excavation and disruption to existing roads. Excavated tunnel muck, dam footprint, and hydraulic structure excavation will be wasted near the site, preferentially on the upstream side within the reservoir limits. Acceptable locations downstream should be considered, however, if they can be constructed with minimal impact to existing side-hill drainage topography and the river. Potentially, excavated spoils can be used to buttress landslide areas adjacent to the dam.

It is important to note that the RCC spillway break creates split left and right RCC construction as the dam nears its finished height. Upper left abutment access, although not essential, greatly simplifies the upper RCC construction on the left side. Generally, the preliminary look at site staging and access affirms the contractor will be able to design and construct reasonable access to all the salient project features. Consequently, access and staging approaches should not adversely affect the schedule or cost.

For the FRO alternative, the upper right abutment staging will be even higher in comparison with the dam crest, perhaps driving a need for contractors to secure more area lower and nearer the work. Spoil quantities would be less, and impacted areas would probably be reduced. Access to each area remains important, and having several sites where contractors can stage equipment, forms, pipe, gates, plants, maintenance, offices, etc., remains very similar and important.

A concept-level RCC production and placement plan is contained in Appendix F.

FRO RCC dam construction differences stem from the lack of a right abutment wing, the approximately 70-foot lower height, and several design aspects that could be eliminated or less involved than in the FRFA. For example, the dam joint spacing might be much wider, without formed drains, and potentially without bond-breaking plate; and the upstream face could be wholly or partly formed RCC and not GERCC, since water storage is not intended. RCC placement will be simpler, with less potential for peripheral impacts. Consequently and considering lower quantities, contractors may elect smaller plants and less sophisticated delivery systems, making the full impact to RCC operations and costs less intuitive.

11 FISH PASSAGE OPTIONS

11.1 Introduction

The fish passage design study team carried forward five potential fish passage alternatives to accommodate upstream and downstream fish passage with the FRO and FRFA structural flood damage reduction alternatives. The fish passage study includes refinement of design criteria, further concept-level design development, performance assessment, and evaluation of costs for potential fish passage facilities that could accommodate passage of upstream and downstream migrating fish species. These activities were performed in collaboration with members of the Flood Damage Reduction Technical Committee and in concert with numerous other physical and biological studies being performed by others to evaluate potential flood damage reduction and aquatic species enhancement strategies.

This section summarizes the refined design criteria, updated fish facility concepts, and anticipated fish passage performance values for the FRO and FRFA structural alternatives described in previous sections of this document. More detailed information on fish passage alternative development can be found in Appendix G, Fish Passage Alternative Concept Design.

11.2 Purpose and Intent

The integration of fish passage systems is a necessary component of flood damage structure design as required by the State of Washington's regulatory authority defined in Revised Code of Washington (RCW) 77.57.030, *Fishways required in dams, obstructions – penalties, remedies for failure,* which requires that dam owners provide safe and timely fish passage for all fish species and fish life stages present in an affected area. Fish passage facility conceptual design has occurred simultaneously with dam design efforts throughout the CBFS Project. The purpose of this section is to summarize the results and conclusions of fish passage facility concept development performed in 2015-16, which built on the preceding design development activities presented in HDR, 2014. This information is intended to be used by the Work Group to inform decisions regarding the integration and performance of potential fish passage technologies with flood damage reduction structural alternatives being developed by the dam design study team.

11.3 Design Criteria

The following paragraphs describe the biological and technical design criteria used to inform the development of fish passage alternatives for this study.

11.3.1 Collaboration with Technical Committees

The fish passage design team and members of the Chehalis Basin Strategy – Flood Damage Reduction Technical Committee coordinated and carried out several fish passage subcommittee meetings throughout development of this study. These meetings became forums for information transfer, detailed discussion, and decision making relative to biological and technical aspects of fish passage facility alternative development. Of primary importance were the discussion, interpretation, and formulation of design criteria that could be carried forward throughout fish passage facility alternative design. Participants attending these meetings included representatives from the following organizations:

- Washington Department of Fish and Wildlife (WDFW)
- U.S. Fish and Wildlife Service (USFWS)
- National Marine Fisheries Service (NMFS)
- Washington State Department of Ecology
- Quinault Indian Tribe
- State of Washington Consultant Study Team

11.3.2 Biological Design Criteria

As part of the CBFS Project, WDFW has led an extensive field sampling program to collect data and better understand the phenology, abundance, habitat requirements, distribution, and migration patterns of fish present within the Chehalis River and more specifically in the potentially impacted areas of the dam structure and inundation limits of the reservoir. Using new and historically available data, WDFW has assisted the fish passage design team with biological fish criteria development in collaboration with other participating technical committee stakeholders. The three primary types of biological design criteria that have the most influence on facility type, size, and configuration relate to the following:

- Fish occurrence and distribution: Informs the selection of species and life stages targeted for fish passage design
- Fish migration timing: Informs the seasonality, anticipated hydrologic conditions, and duration of periods where target fish species may be expected to migrate upstream and/or downstream of the dam location
- Fish abundance: Informs the annual number of fish that require passage as well as the peak daily rate of migration that influences facility size and operation requirements

11.3.2.1 Fish Occurrence and Distribution

The selection of fish species and life stages for fish passage design was derived from field-specific data obtained by WDFW in 2015 and 2016 in addition to readily available historical documentation developed for the Chehalis Basin. In general, the State of Washington interprets its regulatory authority

(Revised Code of Washington [RCW] 77.57.030, *Fishways required in dams, obstructions – penalties, remedies for failure*) to require provision for passage of all fish and fish life stages believed to be present in the system. For the purposes of fish passage alternative development, anadromous and fluvial 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 both upstream and downstream passage. These primary species and their known swimming and leaping abilities were used to influence fish passage technology selection and development of specific technical design criteria. Species known to occur downstream of the dam project area were selected for consideration but did not directly influence the development of specific technical design criteria.

The life histories and specific life stages of each target species were also considered relative to their known occurrence, distribution, and movement through the project area. Life stages of specific species were selected if they have been observed moving – or are believed to move – through the project area (either upstream or downstream).

Target fish species and their respective life stages that were selected for the purposes of design development in this study are presented in Table 11-1.

Table 11-1
Target Fish Species and Life Stages Selected for Design

SPECIES	UPSTREAM	DOWNSTREAM
Spring Chinook	Adult, Juvenile	Juvenile
Fall Chinook	Adult, Juvenile	Juvenile
Coho	Adult, Juvenile	Juvenile
Winter Steelhead	Adult, Juvenile	Adult, Juvenile
Coastal Cutthroat	Adult, Juvenile	Adult, Juvenile
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	Not applicable

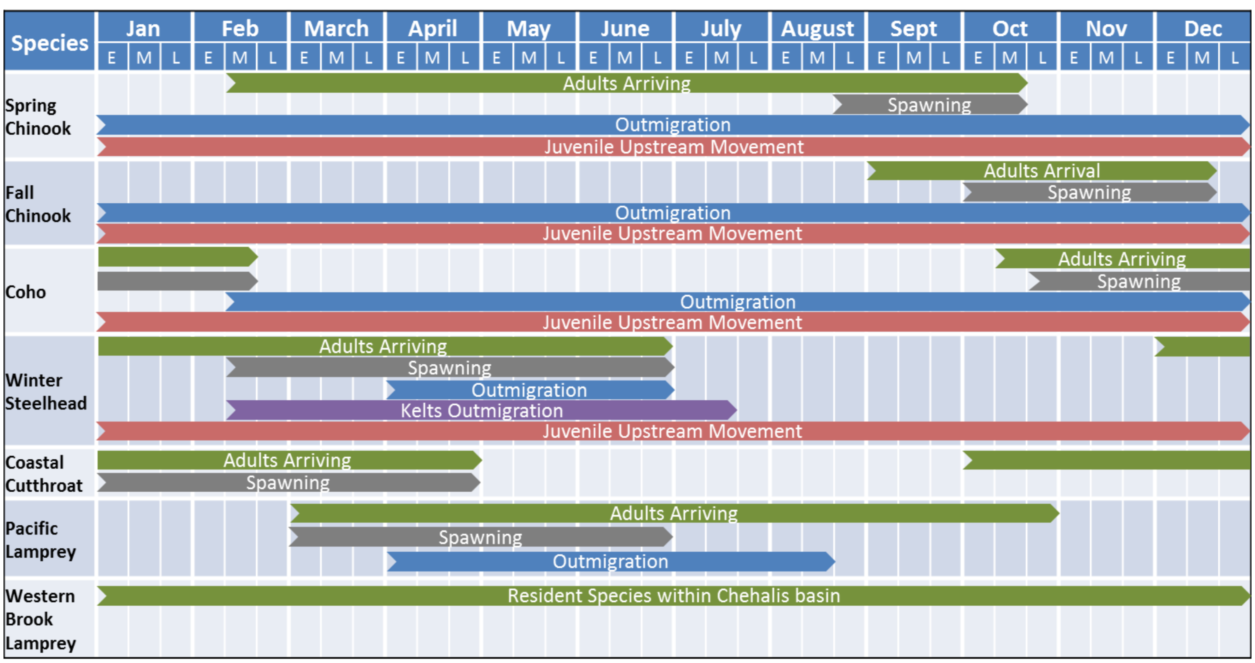
Bull trout are believed to occur downstream of the proposed dam location so they were removed by the Fish Passage Technical Subcommittee as a target species but remained a species of consideration throughout alternative development and concept design. Of the species and life stages targeted for upstream passage, juvenile salmonids, resident fishes, and lamprey exhibit the most variable life history, are the weakest swimmers, and represent the most challenging species and life stages requiring passage. Therefore, technical design criteria used to target the passage requirements of these species and life stages were believed to also accommodate the requirements of bull trout.

Passage technologies for lamprey are relatively new, and few facilities exist in the western United States that target lamprey for passage or collection and transport above dams. Where applicable, readily available best practices, lessons learned from experimental facilities on the Columbia River, and interviews with researchers who specialize in the understanding of lamprey behavior and navigational capabilities were used to inform lamprey passage facility requirements and anticipated performance. In addition to salmonids and the anadromous Pacific lamprey, multiple resident fish species and two species of resident lamprey (western brook and river) are believed to inhabit and transit the proposed dam area.

11.3.2.2 Fish Migration Timing

Each target species is known to have unique migration behavior and is believed to pass upstream or downstream through the project area at specific times of the year for specific durations. The migration timing and duration influence the design and operation of proposed fish passage facilities by defining physical, operational, and environmental conditions expected to occur while passage facilitation is required. The migration timing and duration for each selected fish species and life stage was discussed at fish passage subcommittee meetings as new information was collected in the field. The resulting conclusions, which were used in fish passage alternative design development, are shown in Figure 11-1.

Figure 11-1
Anticipated Migration Periods of the Targeted Species and Life Stages (Periodicity)



Notes: E=Early, M=Middle, and L=Late

The selected values provide a conservative summary of upstream migration, spawning, and outmigration periods suitable to inform robust fish passage designs. Aquatic target species' actual migration and spawning periods are far more complicated and nuanced. For the purposes of alternative development and preparation of conceptual designs, these nuances are not anticipated to be controlling factors in the design and comparison of the fish passage options.

11.3.2.3 Fish Abundance

Fish abundance was evaluated by WDFW and discussed during fish passage subcommittee meetings. Abundance was described in terms of peak annual and peak daily rates of migration. The peak daily rate of migration for both upstream and downstream migrating fish influences the size of many components to fish passage alternatives. The following paragraphs summarize the conclusions from two references developed by WDFW (WDFW, 2016a and 2016b). These results were consulted for the purposes of design development during this study.

11.3.2.4 Upstream Migration

Upstream migration rates were estimated based upon two factors: 1) historic data relative to adult spawner survey results and escapement records, and 2) proposed annual peak goals after project implementation and potential habitat restoration. The peak rate of annual migration for adult salmonids moving upstream was as follows:

- Coho: 12,900
- Fall Chinook: 3,900
- Spring Chinook: 1,350
- Steelhead: 5,630

Numbers for adult upstream migrating Pacific lamprey, cutthroat trout, resident fish, and juvenile salmonids have not been estimated. Although they are an important influence on the overall design of each fish passage alternative, their peak rate of migration is currently unknown and is not anticipated to significantly influence facility size to the extent of adult salmonids.

Given the total number of anticipated adult upstream migrants, WDFW proposes the use of NMFS guidance to derive peak daily values from peak annual estimates. Given the information presented in the literature, peak daily count can be estimated as 10 percent of the maximum annual run (Bates, 1992). To be conservative, NMFS suggests an estimate of 20 percent of the peak daily count based on Bell (1991). If 20 percent of the peak daily count is used, and the peak day is calculated as being 10 percent of the annual run, then the peak hourly count is approximately 2 percent of the annual run.

11.3.2.5 Downstream Migration

Table 11-2 summarizes the total abundance numbers recommended for reference in designing downstream passage alternatives for juvenile salmon and steelhead. These values represent the total number of fish expected to be produced upstream of the location selected for the dam. These numbers do not reflect habitat degradation resulting from the inundation pool, which will restrict spawning and rearing habitat available for all species, but especially Chinook salmon (Ashcraft et al., 2016).

Table 11-2
Predicted Abundance of Juvenile Salmon and Steelhead that will Migrate Downstream from Freshwater Habitat above River Mile 108 of the Chehalis River

SPECIES	LIFE STAGE	MIGRATION PERIOD	MAXIMUM ABUNDANCE
Coho Salmon	Fall Parr	September – December	340,000
	Spring Smolt	March – June	17,000
Steelhead Trout	Fall Parr	September – December	97,000
	Spring Smolt	March – June	14,500
Chinook Salmon	Subyearling (Fry)	January – April	229,000
	Subyearling (Parr/Smolt)	May – August	114,500
	Yearling	March – June	11,000

For spring smolts, freshwater capacity and migration timing were used to predict total daily arrivals between January and August using two example migration curves originating from other river systems. Timing curve 1 represented a free flowing river (Coweeman River), whereas timing curve 2 represented a dammed river where smolts rear in cooler stream temperatures and navigate a reservoir during their downstream migration (Cowlitz River). The expected daily numbers (mean and maximum values) of downstream migrants were similar between the two migration timing curves when all species were included. However, when only coho salmon and steelhead trout were included, mean and maximum values were higher under timing curve 1 than timing curve 2. This difference is relevant because of the uncertainties associated with continued production of Chinook salmon above river mile 108 were a dam to be constructed. The difference between the two scenarios results from the smolts of coho salmon and steelhead trout having a more protracted migration timing under timing curve 2 than timing curve 1.

For fall migrants, timing curves were not available, and daily numbers were approximated based on available information. Estimates of daily numbers of fall migrants were the summary statistics (mean, maximum daily values) for spring smolts of coho salmon and steelhead trout increased by a multiplier of 17.0.

Table 11-3
Predicted Daily Numbers of Juvenile Salmon and Steelhead Originating from Freshwater Habitat Upstream of River Mile 108 in the Chehalis River

	SPRING SMOLTS (JANUARY – AUGUST)		SPRING SMOLTS (JANUARY – AUGUST) COHO AND STEELHEAD ONLY		FALL MIGRANTS (SEPTEMBER – DECEMBER) COHO AND STEELHEAD ONLY	
Daily Metric	Daily Abundance (Timing 1)	Daily Abundance (Timing 2)	Daily Abundance (Timing 1)	Daily Abundance (Timing 2)	Daily Abundance (Timing 1)	Daily Abundance (Timing 2)
Mean	1,919	1,882	203	82	3,451	1,394
Maximum	11,013	10,935	3,265	668	55,505	11,356

11.3.3 Technical Design Criteria

This section identifies specific design criteria and references specific sources of design criteria and guidance relevant to the development of fish passage designs. Technical fisheries design criteria are typically broken into two categories – criteria and guidelines. Criteria are specific standards for fish passage design that require an approved variance from the governing state or federal agency in order to deviate from the established criteria. Guidelines provide a range of values or, in some instances, specific values that the designer should seek to achieve but that can be adjusted in light of project-specific conditions. Site-specific biological and physical rationale for deviating from an agency-established criterion is required; values different from established guidelines should support better performance or solve site-specific issues and may be requested by governing agencies during development of the design.

The following documents provide the guidelines that are used during concept design. If two or more agencies provide differing guidance on a design criterion, the most conservative guidance for fish passage and protection will be followed.

- National Marine Fisheries Service (NMFS). 2011. Anadromous Salmonid Passage Facility Design.
- United States Fish and Wildlife Service (USFWS). 2010. Best Management Practices to Minimize Adverse Effects to Pacific Lamprey.
- Washington Department of Fish and Wildlife (WDFW). 2000. Draft Fish Protection Screen Guidelines for Washington State.
- Washington Department of Fish and Wildlife (WDFW). 2000. Draft Fishway Guidelines for Washington State.
- Washington Department of Fish and Wildlife (WDFW). 2013. Water Crossing Design Guidelines.

A more detailed account of how each specific criterion or guideline was applied to each fish passage facility alternative is provided in Appendix G – Fish Passage Alternative Concept Design.

11.3.3.1 Selection of Fish Passage Design Flows

Fish passage design flow criteria influences several factors associated with fish passage facility size and complexity. Guidelines presented by NMFS and WDFW are based on exceedence calculations of mean daily flows but can be modified to suit site-specific requirements. The exceedence flows statistically represent the flow equaled or exceeded during certain percentages of the time when migrating fish may be present or collected at a facility. The established guidelines are used to set instream flow depths, flow velocities, debris and bedload conditions, fish attraction requirements, tailwater fluctuations, and numerous other factors that a facility might experience while target fish species are migrating.

NMFS (2011) requires 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. WDFW (2000b) suggests a 10 percent exceedence flow be used as a high design flow. NMFS (2011) requires a low fish passage design flow equal to the mean daily stream flow that is exceeded 95 percent of the time during periods when migrating fish are typically present. WDFW recommends that a low flow be established based upon site-specific conditions.

Mean daily flows for water years 1940 through 2012 from USGS gage 12020000 near Doty were reduced using basin area and mean annual precipitation to estimate flows at the proposed dam site. Five percent and 95 percent exceedence flows at the dam site were also calculated for each adult species using their respective upstream migration timing. The lowest 95 percent exceedence flow and the largest 5 percent exceedence determined the fish passage design flow for which both FRO and FRFA upstream fish passage facilities will be designed. The lowest 95 percent exceedence flow is the 95 percent exceedence flow of 18 cfs, which occurs during the Spring Chinook migration period. The highest 5 percent exceedence flow is 2,197 cfs, which occurs during the coho migration period. Therefore, fish passage facilities will be designed to operate from a low fish passage flow of 16 cfs to 2,200 cfs. Although future dam operations for the FRFA dam configuration might limit the low flows to a much higher value, this lower, more conservative value was used for conceptual design purposes in both dam configurations.

11.3.3.2 Selection of Target Range of Reservoir Fluctuation

Anticipated reservoir pool fluctuation for the FRFA dam configuration is a significant factor in determining the type, size, and complexity of upstream and downstream fish passage facilities. Upstream fish passage technologies may require safe release or exit of fish to the reservoir pool. Downstream fish passage technologies occurring in the reservoir either float or include multiple inlets to maintain a hydraulic connection with the reservoir surface. Each type of fish passage technology must accommodate some form of continuous hydraulic connection throughout the anticipated range of pool elevations. As the pool fluctuations become larger, the facilities become larger and more complex. In many cases, certain fish passage technologies can be dismissed because they are unable to accommodate large pool fluctuations.

Historic river flows from October 1, 1988, through January 1, 2016, were input into a reservoir operations simulation model to estimate the FRFA reservoir elevations. The first water year (1989) was omitted from the results summarized below because it included filling of the reservoir and therefore did not represent a typical operating scenario.

The conservation pool of the FRFA reservoir is expected to normally fluctuate between WSEL 588 to 628 (40 feet) over the course of a year and is regulated to seasonally enhance water quality and instream flow downstream of the FRFA dam structure. Flood events may bring the reservoir pool higher than WSEL 628; potentially as high as WSEL 709 in a PMF event. Extreme drought or reservoir drawdown conditions may bring the reservoir pool as low as WSEL 520. Therefore, the normal operational range of the reservoir is anticipated to be 40 feet, while periodic extremes could cause the reservoir to fluctuate up to 189 feet.

11.4 Fish Passage Alternatives for Flood Retention Only Dam

One fish passage alternative was carried forward in the design process to accommodate both upstream and downstream fish passage for the FRO dam alternative. Fish passage is provided primarily through the integration of three multi-use conduits occurring through the base of the dam. The conduits are to be used in combination with flood control operations and, therefore, when flood events occur, fish passage is no longer accommodated through the conduits and a separate collect, handle, transport, and release facility (CHTR) is provided until flood operations cease. A description of the proposed fish passage alternatives for the FRO dam alternative is provided in the following paragraphs.

11.4.1 Conduits

The FRO conduits are a fish passage system intended to provide a route for adult salmon and steelhead, resident fish, and lamprey to volitionally pass upstream and downstream of the FRO dam. The FRO conduits consist of two 10-foot-wide by 16-foot-high and one 12-foot-wide by 20-foot-high rectangular tunnels through the base of the FRO dam alternative. All three conduits are designed to mimic the hydraulic and sediment conveyance characteristics of the natural river channel at the dam location. During normal flow, the river is conveyed in an open-channel hydraulic condition through the conduits. A bed of natural substrate forms on the floor of the conduits under these conditions. During high flow periods, water is impounded behind (upstream of) the dam, the conduit gates are closed, and fish passage through the conduits is stopped. When the conduits are closed, upstream fish passage is provided via the CHTR facility (see Section 11.4.2).

11.4.1.1 Design Elements

The FRO conduits consist of two primary design elements:

- The 10’x16’ Conduits – Two 10-foot-wide by 16-foot-high by 100-foot-long concrete conduits are cast in place at the base of the FRO dam. Each conduit contains a radial gate capable of shutting off flow through the conduits. The conduits are open to the river at the upstream end and combine into a single 22-foot-wide by 130-foot-long conduit at the downstream end. The larger conduit is open to the stilling basin at the downstream end.

- The 12’x20’ Conduit – One 12-foot-wide by 20-foot-high by 230-foot-long concrete conduit is cast in place at the base of the FRO dam. The conduit contains a radial gate capable of shutting off flow through the conduits. The conduit is open to the river at the upstream end and to the stilling basin at the downstream end.

11.4.1.2 Theory of Operation

The FRO 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 flow conditions up through the high fish passage design flow as required by NMFS criteria. The high fish passage design flow is 2,000 cfs. Hydraulic modeling results indicate the conduits replicate the natural stream discharge and velocity rating curves exhibited by the natural channel well above the high fish passage design flow through river discharges of 4,000 cfs.

The Chehalis River channel at the proposed location of the FRO dam is a natural rectangular channel incised in hard rock up to about a 4,000 cfs river flow. Above 4,000 cfs the river rises above the incised rock channel and begins to widen across an existing terrace. The narrow and deep rectangular rock channel creates natural river velocities that are well in excess of the 2-feet-per-second fish passage velocity suggested by NMFS for fish passage design. However, it was agreed among the stakeholders that mimicking the natural hydraulic conditions was the most appropriate approach for the design of the conduits, in part because the incised rock channel remains upstream and downstream of the dam after the dam is constructed.

The FRO conduits effectively meet the needs of the CBFS Project while still providing flow velocities that mimic naturally occurring conditions upstream and downstream of the proposed dam structure. Hydraulic analysis of the conduits and the reaches of natural stream channel above and below the proposed dam shows a typical water depth of 6 feet in the two 10-foot-wide by 16-foot-high conduits and a water depth of 9 feet in the 12-foot-wide by 20-foot-high conduit. The conduits are anticipated to replicate the natural stream flow and velocity exhibited by the natural channel through which fish will pass whether the dam is in place or not up through river discharges of 4,000 cfs. Sediment transport analysis of the conduits shows that the invert of the conduits will be bedded with natural sediment during normal operations. During higher flow events above the anticipated fish migration flows and during flood retention scenarios, the sediment is anticipated to flush out of the conduits. Material will naturally begin to mobilize out of the conduits as discharges rise above 4,000 cfs or by higher flow velocity occurring under the radial flood control gates as the gates are closed.

11.4.1.3 Anticipated Fish Passage Performance and Survival

The following is the anticipated fish passage performance and survival for the conduits.

Table 11-4
FRO Conduits Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	95%	99%	**94%**
Fall Chinook	95%	99%	**94%**
Coho	95%	99%	**94%**
Winter Steelhead	97%	99%	**96%**
Coastal Cutthroat	93%	99%	**92%**
Pacific Lamprey	97%	99%	**96%**
Western Brook Lamprey	97%	99%	**96%**
JUVENILE UPSTREAM			
Spring Chinook	65%	99%	**64%**
Fall Chinook	65%	99%	**64%**
Coho	65%	99%	**64%**
Winter Steelhead	80%	99%	**79%**
Coastal Cutthroat	65%	99%	**64%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
ADULT DOWNSTREAM			
Spring Chinook	-	-	**-**
Fall Chinook	-	-	**-**
Coho	-	-	**-**
Winter Steelhead	98%	75%	**74%**
Coastal Cutthroat	98%	80%	**78%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
JUVENILE DOWNSTREAM			
Spring Chinook	100%	85%	**85%**
Fall Chinook	100%	85%	**85%**
Coho	100%	85%	**85%**
Winter Steelhead	100%	95%	**95%**
Coastal Cutthroat	100%	85%	**85%**
Pacific Lamprey	100%	95%	**95%**
Western Brook Lamprey	100%	95%	**95%**

There are no known existing conduits similar to the proposed FRO conduits that could assist in predicting fish passage performance and survival in the FRO conduits. The likely surrogate for a technology of this nature is fish passage through culverts, which has been studied extensively. In addition, some studies at Mud Mountain Dam provide information on the success of out-migrant fish through similar conduits. Performance and survival through the FRO conduits is based the success of out-migrants at Mud Mountain Dam and the performance of a similar technology – culverts – and is adjusted based on conditions that are unique to the FRO dam on the Chehalis River. The following is a summary the information collected regarding the potential performance of conduits similar to the FRO conduits:

- In general, there are few examples of conduits through dams that are configured for the purpose of fish passage. No known conduits of this nature have been identified in a similar situation for the purposes of upstream passage. The likely surrogate for a technology of this nature would be fish passage through culverts, which has been studied in detail over the past several decades. Culvert fish passage information exists with regard to design rationale, guidelines, and velocity targets. Available information suggests that passage through long conduits of this nature can be successful when velocity and depth criteria are met.
- Design guidelines are readily available for adult salmonid upstream passage. Guidelines and swim capabilities for juvenile upstream passage can be derived from the literature, but formal design guidelines are not available.
- Mud Mountain Dam is an example of successful routing out-migrant fish downstream through a similar type of conduit. Available information suggests that the performance levels and survival for outmigrating juveniles is high as long as velocity criteria are met and the conduit is kept clear of debris and free from sharp edges protruding into the water column. Mud Mountain Dam is on the White River in Washington State.
- A review of literature characterizing the results of barotrauma studies suggests that out-migrants can be passed through partially open radial gates or open valves with a high level of survival when fish are being passed downstream at 1 atmosphere or approximately 34 feet of static water depth. Survival was documented to incrementally decrease as depth and/or pressure increased or as valve openings decreased.

A number of factors specific to the FRO dam, this location, and the Chehalis River adjust the expected performance and survival for this alternative from the information gathered for similar conduits. Some of these factors include:

- It is assumed that the performance and survival values are provided for periods of time when the fish passage conduits are open. The CHTR facility would be operated when flood retention gates are closed. Performance and survival would then default to those values provided for CHTR during periods of flood retention.
- Passage performance is largely a function of the engineering design and capability to provide adequate depths and velocities.

- Roughness elements are planned in the larger center conduit, which would provide a corridor of water velocity suitable for juvenile upstream migration more often.
- Larger adult salmonids were given a higher level of performance and survival for upstream migration than juveniles, given that hydraulic criteria for juvenile fish would be met less often.
- Juvenile salmonids were also given a high level of performance for outmigration. Survival for outmigrating juveniles was lowered slightly in light of the potential interaction with the upstream trashrack. It was assumed that if debris loading occurs, juvenile fish would be more susceptible to being swept into a debris-laden trashrack, which may cause more injury or mortality.
- Inlet and outlet conditions are anticipated to impact juvenile survival during downstream migration through hydraulics, predation, and other factors. Therefore, weaker swimming fish such as cutthroat trout, Chinook, and coho have slightly lower survival rates than those of steelhead.
  - Outmigrating post-spawn adult steelhead are less energetic and possibly more susceptible to injury during downstream migration and are also given a slightly decreased survival.
  - Juvenile winter steelhead are less dependent upon the hydraulic fringe. They generally exhibit a larger size and better swimming ability, which makes them more capable of ascending the conduit.
  - Juvenile steelhead are more capable of handling the varied hydraulic conditions in the conduit as well as other factors at the inlet and outlet. Predation is less of a factor for them than for other species.

11.4.2 Collect, Handle, Transport, and Release Facility

The primary means of upstream and downstream passage at the FRO dam is via the conduits. When water is impounded behind the FRO dam during high flow events, the conduits are closed, and fish passage is provided via a collect, handle, transfer, and release (CHTR) facility. Resident and juvenile fish and lamprey have varied life histories that can accommodate infrequent interruptions in upstream and downstream passage of moderate duration. The CHTR is a fish passage alternative intended to collect migrating adult salmon and steelhead moving upstream and safely transport them upstream of the FRO dam. Although the facility is not designed specifically for upstream migrating juvenile salmon and steelhead, its design does not exclude them. Upstream migration of juvenile species through trap and transport facilities has been documented and is expected. If juvenile species do enter the CHTR facility, means of holding and transport are provided.

The CHTR facility is not anticipated to operate for most of the year when implemented in conjunction with the FRO dam configuration. When flood control scenarios require its operation, it is a staffed facility that is operated 24 hours a day until flood operations cease and passage through the conduits resumes. The CHTR for the FRO dam is similar to the CHTR facility for the FRFA dam, as described in more detail in Section 11.5.2, Collect, Handle, Transport, and Release Facility , with a few exceptions.

The following subsections focus on describing the differences between the FRO CHTR and the FRFA CHTR facilities.

11.4.2.1 Design Elements

The CHTR associated with the FRO dam is similar to that proposed for the FRFA dam. However, differences in the operation, conditions, and fish passage alternatives associated with the FRO dam result in some differences in the design elements described for the FRFA dam CHTR:

- The FRO dam CHTR has a longer fish ladder entrance pool than the FRFO configuration. The fish ladder entrance is configured optimally in consideration of access, fish attraction, and constructability constraints. In general, the ladder entrance is in the same location for both the FRO and FRFA dam CHTR alternatives. However, the shorter height of the FRO dam sets the stilling basin further upstream and the fish ladder entrance pool is elongated to conform to these stilling pool dimensions.
- Lamprey passage is not required in the FRO CHTR alternative, so facilities for lamprey passage are not included. Lamprey passage is accommodated through the FRO conduits during normal operational scenarios when no pool is present upstream of the dam structure. Lamprey provisions are provided for the CHTR facility in conjunction with the FRFA dam configuration.
- The FRFA dam configuration includes technical provisions for the collection and transport or bypass of downstream migrating fish from the reservoir, whereas migrating fish passing the FRO structure are passed downstream through open conduits. Therefore, provisions for receiving, evaluating, and passing downstream migrating fish are unique only to the FRFA CHTR facility and are not provided in the FRO CHTR facility.
- Numerous components of the FRO CHTR facility (e.g. sorting, holding, workup, and transfer facilities, as well as the electrical/mechanical and storage buildings) will be vacant and unmonitored for long durations. As such, the FRO CHTR facilities are provided with additional security measures befitting their mostly vacant status.

11.4.2.2 Theory of Operation

The CHTR associated with the FRO dam operates the same way as the CHTR proposed for the FRFA dam. However, differences in the operation, conditions, and fish passage alternatives associated with the FRO dam result in some differences in the theory of operation described for the FRFA dam CHTR:

- Upstream and downstream fish passage is provided through the conduits for the majority of the year. When water is impounded behind the FRO dam during high flow events, the conduits are closed, and fish passage for adult salmonids is provided via the CHTR facility throughout flood scenario operations. While in operation, the FRO CHTR facility is staffed daily to perform upstream transport activities until passage through the conduits resumes. As flood operations cease and the flood pool recedes, upstream and downstream passage of all species and life-stages resumes through the conduits and the FRO CHTR facility performs shut-down procedures.

- Upstream and downstream lamprey passage is provided via the conduits when they are open. The infrequent interruptions in lamprey passage are expected to have minimal adverse impact on the populations. Therefore, facilities for lamprey passage are not provided as they are in the FRFA CHTR alternative.
- Downstream passage of juvenile salmon and steelhead is provided via the conduits when they are open. The infrequent interruptions in juvenile salmon and steelhead downstream passage are expected to have minimal adverse impact on the populations. Therefore, facilities for juvenile salmon and steelhead downstream passage are not required or provided as they are in the Floating Surface Collector and Fixed Multi-Port Collector alternatives.
- Upstream passage of juvenile salmon and steelhead is provided via the conduits when they are open. The infrequent interruptions in juvenile passage are expected to have minimal adverse impact on the populations. Therefore, upstream passage of juvenile fish passage is not required as it is in the FRFA CHTR alternative. However, upstream migration of juvenile species through trap and transport facilities has been documented and is expected to occur at some level during FRO CHTR operations. Although the CHTR is not specifically designed for upstream passage of juveniles, juveniles may pass through the facility. With the exception of facilitating the offloading of juvenile transport tanks from the floating surface collector (FSC), the same holding, sorting, and transport facilities for juveniles that included in FRFA CHTR are included in the FRO CHTR.

11.4.2.3 Anticipated Fish Passage Performance and Survival

The anticipated fish passage performance and survival for the FRO CHTR fish passage alternative during its periodic operational periods is shown in Table 11-5. The FRO CHTR facility is anticipated to perform similarly to the FRFA CHTR facility for adult upstream migration of adult steelhead and salmon. Given that provisions for lamprey and juvenile upstream migrating fish are not provided for these short durations, passage performance is reduced from that shown for the FRFA CHTR facility.

Table 11-5
Collect, Handle, Transport, and Release Alternative, Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	93%	98%	**91%**
Fall Chinook	93%	98%	**91%**
Coho	93%	98%	**91%**
Winter Steelhead	93%	98%	**91%**
Coastal Cutthroat	55%	98%	**54%**
Lamprey	0%	90%	**0%**
JUVENILE UPSTREAM			
Spring Chinook	55%	90%	**49.5%**
Fall Chinook	55%	90%	**49.5%**
Coho	55%	90%	**49.5%**
Winter Steelhead	60%	90%	**54%**
Coastal Cutthroat	50%	90%	**45%**

The anticipated fish passage performance and survival is based on the performance of other CHTR facilities and is adjusted based on conditions that are unique to the alternative proposed for the FRO dam on the Chehalis River. There are numerous examples of trap and transport facilities in the Pacific Northwest that collect and transport adult anadromous salmonids with high levels of performance and with very low levels of injury or direct mortality.

A number of factors specific to the FRO dam, this location, and the Chehalis River adjust the expected performance and survival for this alternative compared with other similar facilities. Some of these factors include:

- Modern adult collection facilities are typically designed for the collection of adult upstream migrating salmonids. As such, the FRO CHTR adheres to the design guidelines for adult salmonid passage. Provisions for juvenile, resident, and lamprey passage are not provided for the FRO CHTR alternative. Therefore, it is anticipated that juvenile and resident passage will be reduced and lamprey passage will cease until the FRO flood operations cease and the conduits reopen.
- There is a higher level of confidence that a CHTR facility will perform well for upstream migrating adult Chinook, coho, and steelhead. Like facilities had performance and survival values around 90 percent and 98 percent, respectively.
- Reduced performance and survival values were provided for cutthroat trout compared with adult Chinook, coho, and steelhead. The reduced performance value is most substantially attributed to the general focus of other facilities on adult salmonids and the lack of data on cutthroat trout collection.

Anticipated fish passage performance and survival for the FRFA dam is discussed in Section 11.5 below.

11.5 Fish Passage Alternatives for the Flood Retention, Flood Augmentation Dam

Two downstream and two upstream fish passage alternatives were carried forward in the design process to provide fish passage for the FRFA dam alternative. The FRFA fish passage alternatives were refined during this phase of work based upon the specific physical configuration and operational parameters anticipated for the FRFA dam alternative. The FRFA fish passage alternatives included the following:

- Upstream Fish Passage – Conventional Fish Ladder
- Upstream Fish Passage – CHTR Facility
- Downstream Fish Passage – Floating Surface Collector
- Downstream Fish Passage – Fixed Multi-Port Collector

A brief summary of the primary alternative intent, components, and performance characteristics is provided in the following subsections. Information related to operations and maintenance requirements and cost are provided in Sections 12 and 0, respectively.

11.5.1 Conventional Fish Ladder

The Conventional Fish Ladder alternative is intended to provide a route for adult salmon and steelhead to volitionally pass upstream of the FRFA dam to the reservoir. The conventional fish ladder consists of a fish ladder entrance at the stilling basin, a 2,900-foot-long half-ice harbor fish ladder, and a 41-gate fish ladder exit into the reservoir. Water is supplied to the fish ladder through one of the fish ladder exit gates. Additional attraction water is provided to the fish ladder entrance via a pipeline connected to the water quality outlet pipes. The headwater and tailwater can vary throughout the range of fish passage flows from water surface elevation (WSEL) 588 to 628 and WSEL 419.5 to 422.8, respectively.

11.5.1.1 Design Elements

The conventional fish ladder consists of three primary design elements:

- Fish Ladder Entrance – The fish ladder entrance is a concrete structure to the right of the stilling basin that consists of four fish ladder entrance gates, one low-velocity entrance gate, a lamprey ramp, a lamprey collection and transport facility, an entrance pool, and an auxiliary water upwell and diffusion chamber. Water is supplied to the auxiliary water upwell and diffuser chamber via a pipe from the water quality outlet works. The fish ladder entrance is cut into hard rock on the right river bank. An access road is also cut into the hillside to the right of the fish ladder entrance.
- Half-Ice Harbor Fish Ladder – The half-ice harbor fish ladder is over 2,200 feet long, extending from the fish ladder entrance to the fish ladder exit. It consists primarily of 8-foot-wide by 10-foot-long pools with double-length resting pools located approximately every 10 pools. There is

1 foot of vertical drop between pools. It is a slab-on-grade structure benched into the hillside with an access road along the full length. A transport channel penetrates the dam at an invert elevation of 583, connecting to the fish ladder exit in the reservoir.

- Fish Ladder Exit – The fish ladder exit consists of 41 8-foot-wide by 14-foot-long half-ice harbor type pools with 1-foot vertical drops per pool. It is 10 feet high and has a concrete ceiling. There are automated gates with motorized actuators opening to the reservoir at each pool. It is a slab-on-grade structure benched into the hillside with an access road along the full length.

Additional information regarding design elements of the conventional fish ladder is provided in Appendix G.

11.5.1.2 Theory of Operation

The conventional fish ladder is intended to provide year-round, safe, volitional upstream passage for migrating adult salmon and steelhead for the full range of reservoir pool elevations during normal operation. Although adult salmon and steelhead will only pass upstream during certain periods of the year, operation of the conventional fish ladder year-round is intended to accommodate resident fish that currently traverse this reach of the Chehalis and may wish to move upstream at any time. Juvenile salmon and steelhead are not precluded from moving upstream through the fish ladder, though the fish ladder is not designed specifically to accommodate them.

The conventional fish ladder meets the NMFS design criteria for passing adult salmonids, including 1-foot drops across the fish ladder baffles, attraction flows at the entrance greater than 10 percent of the 5 percent exceedence flow (250 cfs), and a hydraulic drop at the primary entrance gate of 1 to 1.5 feet. In addition to meeting NMFS criteria at the fish ladder entrance, the low-velocity entrance has been incorporated to accommodate juvenile fish and lamprey. Juvenile fish pass over two weirs at the entrance, each with 6-inch hydraulic drops across them as suggested by the literature, to enter the fish ladder. Similarly, lamprey entering the low-velocity entrance are provided with flat, smooth surfaces to attach to as they make their way up a separate flume to a collection tank. The collection tank is regularly transported upstream in a trap-and-transport-type operation.

Auxiliary water is provided to the fish ladder entrance from the reservoir water quality control works through a large-diameter conduit. An in-line energy dissipation valve reduces much of the pressure in the auxiliary water supply pipe. The remaining energy is dissipated in the deep upwell at the fish ladder entrance. The energy dissipation valve is an automated valve that will adjust to meet the desired flow by burning more or less energy based on the reservoir elevation. Diffuser baffles for the auxiliary water supply will be hydraulically tested at startup and set once. No additional adjustment of the diffuser baffles should be necessary.

Turning pools and resting pools are located throughout the fish ladder to provide fish places to rest on their 210-foot climb to the reservoir.

Approximately 30 cfs is supplied to the fish ladder via the fish ladder exit gates. Of the 41 fish ladder exit gates, only one gate operates at a time to maintain a continuous hydraulic connection with the reservoir at the reservoir surface. The gates operate automatically based on the reservoir elevation. Each gate accommodates 1 foot of reservoir operation. As the reservoir fluctuates during its normal operation from WSEL 628 to 588, the fish ladder exit gates open and close to maintain 30 cfs in the fish ladder. Since each gate can only accommodate about 1 foot of water surface fluctuation, the gates automatically track the reservoir and switch on and off as necessary. When operation transitions from one gate to another, the high gate will slowly close while the lower gate slowly opens.

A concrete ceiling runs the full length of the fish ladder exit to minimize the opportunity for debris to enter the fish ladder when portions of or all of the fish ladder exit is submerged. The ceiling also limits the height of the structure, minimizing expensive additional structural support features. Air management systems are included in the ceiling to prevent entrapment of air and floatation forces when the facility is submerged.

When the reservoir water elevation rises above the normal operating maximum, the emergency shutoff gate at the dam penetration closes automatically. The emergency shutoff gate stops flow down the fish ladder. During such events, the fish ladder will drain through the orifices back to the tailwater. The fish ladder will remain dry until the emergency shutoff gate is reopened. Closing and opening of the shutoff gate will occur slowly to allow fish in the ladder to escape downstream and to prevent surging in the fish ladder, respectively. However, in the event of a catastrophic event such as an earthquake or runaway (uncontrolled discharge) in the fish ladder downstream, the emergency shutoff gate will close quickly to minimize the risk to the dam.

11.5.1.3 Anticipated Fish Passage Performance and Survival

The following is the anticipated fish passage performance and survival for the Conventional Fish Ladder alternative.

Table 11-6
FRFA Conventional Fish Ladder Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	80%	99%	**79%**
Fall Chinook	80%	99%	**79%**
Coho	80%	99%	**79%**
Winter Steelhead	80%	99%	**79%**
Coastal Cutthroat	70%	99%	**69%**
Lamprey*	60%	90%	**54%**
JUVENILE UPSTREAM			
Spring Chinook	0%	50%	**0%**
Fall Chinook	0%	50%	**0%**
Coho	0%	50%	**0%**
Winter Steelhead	0%	50%	**0%**
Coastal Cutthroat	0%	50%	**0%**

*Note that lamprey are passed through a lamprey ramp, collection hopper, and transport to the head of reservoir.

The anticipated fish passage performance and survival is based on the performance of other volitional fish ladders and adjusted based on conditions that are unique to the alternative proposed for the FRFA dam on the Chehalis River. There are limited examples of volitional passage fish ladders in the Pacific Northwest designed for hydraulic heads greater than 150 feet, or volitional passage fish ladders that accommodate reservoir fluctuations at the ladder exit of 30-40 feet. The following is a summary of the performance of similar fish ladders:

- Mid-height ladders up to 150 ft tall on the Columbia River show a high level of passage performance for adult anadromous salmonids and bull trout.
- The North Fork Fish Ladder on the Clackamas River in Oregon is an example still in operation that extends over 2 miles and ascends a height of 240 feet. The North Fork ladder passes from 95 to 100 percent of adult Chinook and steelhead. Bull trout have also shown a high level of success but are not the current focus of monitoring efforts, so less data is available. The 2-mile-long fish ladder at North Fork performs at levels higher than other examples of its kind for the following reasons:
  - The ladder entrance accommodates up to 280 cfs of attraction flow through multiple entrances using an auxiliary water supply (AWS) originating from the Faraday Diversion Dam
  - The water temperature and water quality composition originating from the AWS are identical to those of the river that are triggering fish movement
  - The reach between North Fork Dam and Faraday Diversion Dam is not subject to high levels of thermal gain

  - North Fork Reservoir exhibits limited thermal stratification as a result of its narrow configuration and hydropower operations; therefore, the water entering the ladder exit is similar to that of the river to begin with
  - Cool water is injected into the ladder at two additional downstream locations, which further improves fish attraction during portions of the year

A number of factors specific to the FRFA dam, this location, and the Chehalis River adjust the expected performance and survival for this alternative from that of similar facilities. Some of these factors include:

- The ladder performance and survival values include fish entering the ladder entrance, release into the reservoir, and passage through the reservoir. The performance and survival documented at similar facilities did not always include these pathways.
- Reservoir transit introduces some level of uncertainty and will require site-specific verification.
- Loss of migration cues at the reservoir through flow modification may influence the timing of upstream migrants.
- The ladder is anticipated to perform at high performance values at or above 80 percent, with survival values of 99 percent for adult Chinook, steelhead, and coho. The value of 80 percent was selected to accommodate uncertainty of reservoir transit.
- Slightly lower performance is anticipated for cutthroat trout, given potential issues with attraction into the ladder entrance as well as the expenditure of energy and motivation required by smaller fish. However, tagged bull trout have been documented to ascend the North Fork Ladder in as few as 10 to 12 hours. (Cutthroat trout are expected to perform similarly to bull trout.) The lower performance value of 70 percent was selected to accommodate uncertainty of reservoir transit.
- Little is known about the motivation of juvenile fish to ascend a high ladder such as this. Given this uncertainty and the likelihood that the ladder would be optimized for adult salmonids, low performance and survival values of 0 percent and 50 percent, respectively, were selected.
- It is assumed that provisions to improve lamprey passage would be incorporated into the ladder entrance, baffles, floor, and walls, and a separate trap and transport facility for lamprey would be provided. However, given the lack of data available relative to lamprey passage in high dam facilities, low performance (60 percent) and high survival (90 percent) were selected.

11.5.2 Collect, Handle, Transport, and Release Facility

The CHTR for the FRFA dam is a fish passage alternative intended to collect migrating adult salmon and steelhead, juvenile salmon and steelhead, resident fish, and lamprey moving upstream and safely transport them upstream of the FRFA dam. The CHTR consists of a short fish ladder, a fish lift, holding galleries, sorting stations, and transportation. Fish enter the CHTR through the stilling basin, where they are attracted to the flow from the fish ladder entrance. Flow from the fish ladder guides them through the entrance pool, up the fish ladder, and into a fish trap and holding area. Inside the holding area they are crowded into a hopper, which is lifted to a flume. After they are released in the flume, the fish are sorted by size through bar grating and diverted to holding galleries. When released from the holding galleries, the fish move to sorting stations where they can be manually sorted and examined or simply passed through to transport tanks. Fish are transferred from the transport tanks via water-to-water transfer directly to transport trucks. The trucks are driven above the dam to predetermined release sites in the reservoir or on Chehalis River or one of its tributaries. The FRFA CHTR is a staffed facility that is operated year-round.

11.5.2.1 Design Elements

The CHTR consists of six design elements:

- Fish Ladder – The fish ladder consists of a fish ladder entrance at the stilling basin and a 600-foot-long half-ice harbor fish ladder to the fish lift. The fish ladder entrance is identical to that described in Section 11.5.1.1. The half-ice harbor fish ladder is about 510 feet long, extending from the fish ladder entrance to the fish lift, and is similar in layout and construction to the half-ice harbor fish ladder described in Section 11.5.1.1. However, each pool is designed with a vertical hydraulic drop of 0.7 feet, which meets the NMFS criteria for juvenile salmonids and is also comparable to the swimming capabilities of resident fish like cutthroat and bull trout.
- Fish Lift – The fish lift consists of a trapping mechanism, hopper, and lift system at the upstream end of the fish ladder. A trapping mechanism directs fish into the hopper and prevents them from escaping. The lift system raises the hopper, a rectangular metal tank containing water and fish, over 80 feet to a flume that leads to the handling and transport facility. A diffuser system in the walls around the hopper provides 30 cfs of attraction water to the hopper sump and supplies the fish ladder.
- Holding Galleries – Three holding galleries are provided for adult fish, juvenile fish, and lamprey. The adult and juvenile holding galleries are located at the handling and transport facility. The lamprey holding gallery is located adjacent to the first fish ladder turning pool. The flume from the top of the fish lift carries adult and juvenile fish to their respective holding galleries.
- Sorting Stations – From the holding galleries, adult and juvenile fish are crowded into flumes that carry them to visual inspection tanks, anesthesia tanks, or work-up stations. They are then sent to one of three circular holding tanks to await transport.
- Transportation – The three circular holding tanks are supported on an elevated platform above the transport truck loading area. Vertical collars on the bottom of the circular tanks can be connected to the transport tanks on trucks to facilitate water-to-water transfer. Transport tanks are equipped with life support systems to ensure fish safety and health during their transport to release points upstream and downstream.
- Mechanical/Electrical and Storage Buildings – Prefabricated, concrete masonry unit (CMU), or buildings of similar construction are located adjacent to the handling and sorting facility to

house mechanical and electrical equipment and provide storage for equipment and materials associated with the CHTR. The buildings are secured facilities with outdoor lighting to reduce the risk of vandalism or theft.

Additional information regarding design elements of the CHTR is provided in Appendix G.

11.5.2.2 Theory of Operation

While adult salmon and steelhead will only pass upstream during certain periods of the year, operation of the CHTR year-round is intended to accommodate resident fish, lamprey, and juvenile salmon and steelhead that currently traverse this reach of the Chehalis River and may wish to move upstream at any time.

The proposed fish ladder for the FRFA meets the NMFS design criteria for passing juvenile salmonids, including 0.7-foot drops across the fish ladder baffles and attraction flows at the entrance greater than 10 percent of the 5 percent exceedence flow (250 cfs). Provisions for attraction and collection of lamprey, juvenile salmonids, and resident fish are provided at the ladder entrance similar to the FRFA conventional fish ladder entrance described in Section 11.5.1, Conventional Fish Ladder.

A resting pool and two turning pools are evenly spaced throughout the fish ladder to provide fish areas to recover on their 40-foot climb to the fish lift.

About 30 cfs is supplied to the fish ladder via the fish lift. Water to the fish lift comes from the reservoir via a pipeline that branches off the auxiliary water supply pipeline. Prior to entering the fish lift, a pipe branches off the fish lift supply pipe to provide water to the handling and sorting facilities. The remaining water passes into a stilling chamber around the fish lift sump, where the water is stilled and diffused through wall screens into the hopper area and travels down the fish ladder.

Fish and lamprey pass up the fish ladder through the fish ladder entrance gates and low-velocity entrance. Lamprey are attracted to the low velocity and smooth surfaces of the lamprey flume, which they follow to a holding tank adjacent to the first turn in the fish ladder. The holding tank is removed and manually transported upstream, where lamprey are safely released above the dam. Adult and juvenile fish continue up the fish ladder to a hopper where they are trapped and held. The hopper trap capacity is determined by the maximum daily fish return and by the number of fish expected to be trapped before the trap catch is transported. The poundage of fish is determined by the weight of an average fish targeted for trapping, multiplied by the maximum number of fish.

The motorized lift mechanism is triggered by a user set frequency and raises the fish over 80 feet to an elevated flume at the upstream end of the handling and transport facility. A mechanical trip automatically opens a door on the hopper and safely releases fish into the wetted flume. Inside the flume, fish travel by gravity to a section of smooth bar grating in the floor where adults are separated from juveniles. The adults and juveniles travel in dedicated flumes to separate holding galleries.

From the adult holding gallery, adult fish are crowded and pass over the false weir. They are then directed to one of two visual inspection tanks by means of an automated diverter gate. The operator determines where the fish should be held and directs the fish back into the adult primary transport flume. Fish are then sorted by actuated panel gates, within the adult primary transport flume, to any of three holding tanks, bypass, or adult anesthesia tank and work-up station.

From the juvenile holding gallery, fish are brailed into the juvenile primary transport flume. The juvenile fish are directed to the anesthesia tank and juvenile work-up station. From the juvenile work-up station the operator directs the fish back into the adult primary transport flume. Fish are then sorted by actuated panel gates, within the primary transport flume, to any of three holding tanks.

Anesthetized adult and juvenile fish are routed to separate circular holding tanks to allow monitoring of fish to ensure full recovery from the anesthetic effect prior to transport.

Transport trucks carrying tanks specially made to transport fish drive to a dedicated loading area beneath the circular tanks. Fisheries personnel attach a vertical collar that extends from the bottom of the circular tank to the tank on the transport truck. Tanks on the transport trucks are filled full with water prior to connection to the collar. When connections are made, the collar is also filled with water to facilitate water-to-water transfer of the fish. A horizontal gate in the floor of the tank is opened hydraulically, connecting the circular tank to the tank on the transport truck. Water in the transport truck tank is slowly drained through juvenile criteria screens in the tank wall. This water is drained directly onto the pavement of the loading area where it is carried via a floor drain back to the river. As water is drained in the truck tank, the water elevation in the circular tank slowly recedes. The slope in the floor of the circular tank ensures that all the fish exit the circular tank through the floor gate and into the truck tank. After the water level is drawn down to the top of the transport tank, a gate in the floor of the circular tank is closed, the empty vertical collar is detached from the truck tank, and the door to the truck tank is sealed.

11.5.2.3 Anticipated Fish Passage Performance and Survival

The following is the anticipated fish passage performance and survival for the FRFA CHTR fish passage alternative.

Table 11-7
Collect, Handle, Transport, and Release Alternative, Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	93%	98%	**91%**
Fall Chinook	93%	98%	**91%**
Coho	93%	98%	**91%**
Winter Steelhead	93%	98%	**91%**
Coastal Cutthroat	88%	98%	**86%**
Lamprey	60%	90%	**54%**
JUVENILE UPSTREAM			
Spring Chinook	60%	90%	**54%**
Fall Chinook	60%	90%	**54%**
Coho	60%	90%	**54%**
Winter Steelhead	65%	90%	**58.5%**
Coastal Cutthroat	60%	90%	**54%**

The anticipated fish passage performance and survival is based on the performance of other CHTR facilities and is adjusted based on conditions that are unique to the alternative proposed for the FRFA dam on the Chehalis River. There are numerous examples of trap and transport facilities in the Pacific Northwest that collect and transport adult anadromous salmonids with high levels of performance and with very low levels of injury or direct mortality. Following are a few examples that were used as a basis of comparison:

- Merwin Dam Adult Collection Facility – Lewis River, Washington State
- North Fork Adult Sorting Facility – North Fork Clackamas River, Oregon State
- Lower Baker Adult Collection Facility – Baker River, Washington State
- Cougar Dam Adult Collection Facility – South Fork McKenzie River, Oregon State
- Cowlitz Adult Collection Facility – Cowlitz River, Washington State
- White River Diversion Dam Adult Collection Facility – White River, Washington State
- Minto Adult Collection Facility – North Santiam River, Oregon State
- Foster Fish Collection Facility – South Santiam River, Oregon State

A number of factors specific to the FRFA dam, this location, and the Chehalis River adjust the expected performance and survival for this alternative compared with other similar facilities. Some of these factors include:

- Modern adult collection facilities are typically designed for the collection of adult upstream migrating salmonids. Juvenile collection during upstream migration has historically been incidental, and, therefore, limited data exists.

- There is a higher level of confidence that a CHTR facility will perform well for upstream migrating adult Chinook, coho, and steelhead. Like facilities had performance and survival values around 90 percent and 98 percent, respectively.
- Reduced performance and survival values were provided for cutthroat trout compared with adult Chinook, coho, and steelhead. The reduced performance value is most substantially attributed to the general focus of other facilities on adult salmonids and the lack of data on cutthroat trout collection.
- Upstream migrating juvenile fish were given lower performance and survival values than those for adults. These values reflect the uncertainty of attracting fish into a ladder entrance, predation, and motivation to ascend the ladder into a holding gallery. Additional provisions could be engineered into a facility of this nature to improve juvenile fish collection and safe transfer. These include multiple low-head entrances, lower head differential between pools, and segregation zones in holding galleries to decrease predation. Such provisions should be explored by the fish passage subcommittee during preliminary design. However, changes made to improve juvenile fish collection might adversely affect adult fish collection and should be evaluated accordingly.
- The reduced size, motivation, and swimming capability for juvenile cutthroat trout resulted in lower juvenile upstream passage performance and survival values compared with those associated with adult upstream passage.
- Since lamprey passage through such facilities has not historically been a focus, limited data is available. Additional research is needed. One source of data may be ongoing efforts on the Mid-Columbia and Yakima Rivers, but this has not been investigated. This information can be used to inform revisions to the table as data becomes available. Given the high level of uncertainty associated with collection of lamprey in a trap and transport situation, lower performance and survival values were provided.

11.5.3 Floating Surface Collector

The Floating Surface Collector alternative is intended to collect and safely pass outmigrating adult steelhead, juvenile salmonids, and juvenile and adult resident fish from the FRFA reservoir to a designated release point downstream of the FRFA dam structure. Attraction flows of 500 to 1,000 cfs are generated at the entrance to a large floating collection barge, which creates a positive flow-net that reaches out into the reservoir. Fish entering this “zone of influence” are triggered to move downstream toward the artificial reservoir outlet. Fish are guided from the reservoir into the entrance of the collection barge with the assistance of guidance and lead net systems. Fish entering the floating collection barge are moved via transport vehicle downstream of the dam and released into the Chehalis River. The floating surface collector system would operate year-round when reservoir elevations are within the anticipated normal operational range of WSEL 588 to 628.

11.5.3.1 Design Elements

In general, the Floating Surface Collector alternative consists of six primary design elements:

- Floating Collection Barge – The floating collection barge is the most complex operational component of the FSC. It consists of floatation, attraction flow pumps, dewatering screens, and fish sorting and handling facilities. Ballast tanks on the bottom of the barge keep the facility afloat while attraction pumps draw water into the bow of the structure, through screens meeting NMFS juvenile criteria, and over a weir leading to fish sorting and holding facilities at the stern. The substantial power requirements of the barge are supplied from electrical facilities on shore via high capacity overhead or submarine cables. Access is provided from the dam via collapsing walkways and stairs that automatically adjust to the changing reservoir level.
- Net Transition Structure – The net transition structure (NTS) is composed of a system of large, fabricated, impermeable panels attached to a steel frame. The panels are positioned at the upstream face of the collection barge and are configured to create a gradual and uniform increase in flow velocity from the reservoir to the inlet of the primary dewatering screens. The net transition structure transitions over its 75 foot length from 75 feet wide and roughly 65 feet tall upstream to 16 feet wide and roughly 15 feet tall downstream. Floatation tanks on the top of the NTS on the port and starboard sides keep the structure afloat. Guide nets extend outward from the upstream edges of the NTS. A lead net extends outward into the reservoir from the middle of the upstream edge of the NTS.
- Mooring and Anchorage Systems – The collection barge is moored in a fixed horizontal position using a system of three vertical guide rails. Two guide rails located at the rear of the collection barge are structurally anchored to the upstream face of the FRFA dam, while one guide rail is located near the port bow of the collection barge and is independently supported by a system of piles. The guide rails allow the collection barge to rise and fall with the reservoir. The NTS is anchored to the upstream face of the collection barge. The NTS rises and falls with the reservoir in unison with the collection barge.
- Guidance and Lead Nets – Guidance nets extend upstream from the corners of the NTS horizontally to the shore and vertically to the reservoir bottom. A lead net extends from the center of the NTS at its upstream end horizontally upstream several hundred feet and vertically to the bottom of the reservoir. The tops of the nets are attached to floats at the reservoir surface. As the reservoir fluctuates, the nets fold and unfold over themselves to maintain a physical barrier to fish for the full depth of the reservoir. The nets consist of solid panels near the surface and netting below of a mesh that excludes juvenile fish. The nets are fully anchored.
- Fish Transport System – Hoppers holding fish and water, located at the aft end of the collection barge, are lifted off the barge by a crane mounted to the top of the dam. Transport tanks with life support systems are mounted to trucks and driven to the top of the dam to receive the

hopper contents. Transfer of fish is water-to-water. Transport tanks are driven to sorting and holding facilities for evaluation and return to the river downstream of the dam.

- Log Boom – The log boom is reconfigured from what is normally required for other fish passage alternatives and is solely for operation of the dam in order to accommodate the most biologically effective arrangement of the FSC, NTS, and nets.

Additional information regarding design elements of the Floating Surface Collector is provided in Appendix G.

11.5.3.2 Theory of Operation

The floating surface collector is intended to provide safe passage for downstream migrating juvenile salmonids, adult steelhead, and resident fish year-round, within the normal anticipated operational range of reservoir pool elevations. During reservoir flood operations outside of typical reservoir elevation levels, the floating surface collector must safely accommodate rising reservoir elevations without damage. Juvenile salmonids and adult steelhead are expected to pass downstream during certain periods of the year, but resident fish are known to migrate both upstream and downstream periodically throughout the year. The floating surface collector is therefore also intended to provide resident fish opportunities for downstream migration throughout the year.

The floating surface collector will operate normally within a 40-foot range of reservoir elevations with the capability of safely accommodating a potential fluctuation of up to 189 feet during episodic flood or drought operations. The conservation pool of the FRFA reservoir is expected to fluctuate normally between WSEL 588 to 628 (40 feet) over the course of a year and is regulated to seasonally enhance water quality and instream flow downstream of the FRFA dam structure. Flood events may bring the reservoir pool higher than EL 628; potentially as high as EL 709 in a PMF event. Extreme drought or reservoir drawdown conditions may bring the reservoir pool as low as EL 520. During flood or extreme drought events, the floating surface collector will be placed in a nonoperational state but will rise or lower vertically to the high and low elevation limits safely on the guide rails and without damage to its components. It is expected that significant debris removal and inspection of the facility will be required following flood events prior to startup.

The fish screens located on the collection barge are designed to be in conformance with NMFS juvenile screening criteria up to an anticipated attraction flow of 500 cfs (inclusive of a factor of safety). The onboard low-head attraction pumps have capability to increase the attraction flow up to a maximum of 1,000 cfs. Approach velocity increases proportionally at the higher attraction flow. Other like facilities are known to operate at these higher attraction flows, outside of the standardized approach velocities, for several months out of the year to improve collection efficiency during the peak rate of outmigration. Typically, these periods of higher attraction flows correspond with migration periods when larger smolts with greater swimming capability are anticipated to occur in the reservoir. For the purposes of estimating costs and level of effort for the facility, it is assumed that the facility is operated for

10 months a year with an attraction flow of 500 cfs and two months of each year with an attraction flow of up to 1,000 cfs. This is consistent with how other facilities are operated in the Pacific Northwest, such as Upper and Lower Baker.

Full-depth guidance nets are placed upstream of the net transition structure and expand outward to points on the left and right banks of the reservoir. The guidance nets are attached to the upstream end of the net transition structure at its port and starboard corners. The tops of the nets are designed to rise and fall with the reservoir to ensure fish guidance is maintained for the targeted normal reservoir fluctuation. At extreme flood stages, the linear array of floats that support the top of the nets are allowed to fill with water and the tops of the nets submerge, allowing for debris passage over the spillway. Additional systems of floats and weights attached to the nets lower in the water column allow the guidance nets to fold at designated locations to maintain a full exclusion barrier throughout the storage pool range without binding or other damage to the nets. However, because of the proximity of the net systems to the spillway, it is likely the nets will be laden with debris and possibly damaged during extreme events. As with the collection barge itself, the nets will require debris removal and careful inspection by divers prior to startup after large flood events.

The attraction flow pumps draw in flow to the inlet of the net transition structure, creating a positive flow net toward the collection barge and along the edges of the fish guidance nets. Outmigrating fish sense the positive movement of water and are behaviorally triggered to move toward the simulated outlet of the reservoir. As the fish move toward the inlet to the collection barge, the gradually varying changes in geometry of the net transition cause fish to experience a gradual and uniform increase in flow velocity. As water is drawn into the collection barge, dewatering screens begin to uniformly remove the water that passes down the gradually narrowing collection channel. This process continues until the majority of the water has passed through the primary dewatering screens and approximately 105 cfs is moving at a targeted capture velocity of 8 ft/s. Any fish present within the water column are considered captured at this point and continue downstream through a short deceleration flume where the flow is reduced to 6 cfs and a velocity of approximately 6 fps. Here, fish pass through a grader flume that separates them into three specific size classes and conveys them into separate holding pools at the rear of the collection barge (for example: fry less than 80 millimeters (mm); smolt, juveniles, and residents up to 160 mm; and fish larger than 160 mm, which may be post-spawn adult steelhead or larger cutthroat trout or bull trout). From the holding pools, technicians operate mechanical crowders that encourage the two larger size classes of fish into one of two hoppers. The hoppers are raised out of the collection barge using an overhead gantry crane and placed on the bed of a transfer vehicle. Fry are hand carried up in a separate fry box and placed in the same transport vehicle. All fish are then transported downstream to monitoring and evaluation facilities, and are eventually released back into the Chehalis River downstream of the FRFA dam structure, where they can continue their migration downstream.

11.5.3.3 Anticipated Fish Passage Performance and Survival

The following table summarizes the anticipated fish passage performance and survival for the Floating Surface Collector alternative. Discussion related to the rationale behind performance and survival value selection is provided in Attachment G of this document.

Table 11-8
Floating Surface Collector Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT DOWNSTREAM			
Spring Chinook	-	-	**-**
Fall Chinook	-	-	**-**
Coho	-	-	**-**
Winter Steelhead	50%	80%	**40%**
Coastal Cutthroat	65%	80%	**52%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
JUVENILE DOWNSTREAM			
Spring Chinook	65%	98%	**64%**
Fall Chinook	65%	98%	**64%**
Coho	65%	98%	**64%**
Winter Steelhead	65%	98%	**64%**
Coastal Cutthroat	65%	98%	**64%**
Pacific Lamprey	3%	10%	**0.3%**
Western Brook Lamprey	3%	10%	**0.3%**

Anticipated fish passage performance and survival is based on the performance of other floating fish collection facilities currently in operation at other locations and adjusted based on assumptions related to the conditions that are unique to a proposed FRFA dam on the Chehalis River. Existing facilities used to inform selection of performance and survival values are summarized in Table 11-8.

Table 11-9
Summary of FSCs Currently in Operation Used to Inform Design and Performance

FACILITY	OWNER – LOCATION	VERTICAL FLUCTUATION	ATTRACTION FLOW	FISH TRANSPORT	1ST YEAR OF OPERATION
Upper Baker	PSE – Baker River, WA	30 to 60	500/1,000	Trap and transport	2008
Lower Baker	PSE – Baker River, WA	30 to 60	500/1,000	Trap and transport	2013
Swift	PacifiCorp – Lewis River, WA	100	500/750	Trap and transport	2012
North Fork	PGE – Clackamas River, WA	2 to 10	600/1,000	7-mile long bypass conduit	2015
Cushman	Tacoma Power – Skokomish River, WA	20	250	Trap and transport	2015

The following are assumptions made regarding the performance and survival of fish passage through a floating surface collector at the proposed FRFA dam on the Chehalis River:

- The selected performance value is intended to accommodate fish passage from the head of the reservoir to the point of release downstream of the dam.
- It is assumed that collection performance of downstream migrating juvenile salmonids that have entered the zone of influence can be achieved in excess of 90 percent with the implementation of adequate guidance nets, lead nets, and attraction flow (See Upper Baker, Lower Baker, and North Fork FSCs).
- Swift and Cushman facilities listed above have reported combined reservoir transit and collection values on the order of 19 to 26 percent as a result of reservoir temperature, stratification, circulation, and fish transit issues. Many of these concerns are addressed through site-specific solutions integrated into the facility after the first several years of operation.
- The success of reservoir transit at this location is speculative, and site-specific studies are likely to be required after implementation to determine actual performance. All values were reduced from the expected facility performance to account for this uncertainty.
- Like facilities of this nature typically experience survival rates meeting or exceeding 98 percent for juvenile salmonids and resident fish.
- Juvenile lamprey passage performance and survival values are anticipated to be low given the uncertainty and lack of data available for lamprey and use of this type of fish passage technology. It is speculated that juvenile lamprey (ammocoetes and macropthalmia) will move to the head of reservoir. Although values may be higher than 30 percent in practice, the selected value was kept low to be conservative and represent an appropriate level of uncertainty.

11.5.4 Fixed Multi-Port Fish Collector with Fish Bypass Conduit

The fixed multi-port fish collector with fish bypass conduit (Multi-Port Collector) alternative is intended to collect downstream juvenile migrants, resident fish, and post-spawn adult steelhead and pass them safely downstream of the FRFA dam. The multi-port collector consists of four sets of vee-type dewatering screens, each staggered 10 feet vertically to provide downstream collection throughout the full range of normal reservoir fluctuation, from WSEL 628 to WSEL 588. Low-head, high-flow pumps pull water through the fish screens into a pump plenum common to all four sets of screens. Water drawn into the screens creates attraction for aquatic species moving downstream. A small amount of water remains in the channel at the end of the screens and carries species over a weir and into a bypass conduit. The bypass conduit carries species under pressurized flow downstream and through the dam at velocities faster than they can escape. On the downstream side of the dam, the bypass conduit transitions to open channel flow and enters a consolidation structure where the individual bypass pipes from each fish screen transition to a single pressurized conduit that carries the species downstream. The downstream moving species are returned safely to the Chehalis River downstream of the dam via an outfall structure. The outfall structure discharges into a deep, swift-moving pool in the river channel, where the aquatic species can continue their migration or volitional movement.

11.5.4.1 Design Elements

The multi-port collector consists of several design elements, including:

- Fixed inlets with dewatering screens – The multi-port collector is a single concrete structure that contains four fixed inlets with dewatering screens that are vertically staggered and discharge to a common pump plenum. Dewatered flow from the screens is discharged out the left side of the plenum via 18 low-head submersible pumps. Water and fish at the end of each screen channel flow down a traveling ramp into a hopper. Pressurized bypass pipes attached to each hopper carry fish and flow downstream. The multi-port collector structure is benched into the hillside of the reservoir such that roughly half of it is founded directly on rock, with the remainder founded on piles.
- A bypass penetration through the dam – The bypass pipes from each screen structure are routed downstream to penetrate the dam at a common invert elevation and as close to each other as safe and practical. Downstream of the dam, each of the pipes continues to a consolidation structure. Each of the four bypass pipes between the multi-port collector and the consolidation structure is a different size to accommodate the physical and hydraulic situations of each screen structure.
- A bypass consolidation structure – The consolidation structure is a 20-foot-wide by 40-foot-long by about 10-foot-tall open concrete structure. It is benched into the hillside and founded partially on rock and partially on piles. Four bypass pipes enter the structure on the upstream end. The interior is sloped to safely channel fish and flow to a single pressurized bypass pipe at the downstream end.

- A bypass conduit – A pressurized, 18-inch-diameter bypass conduit slopes from the consolidation structure downstream to the bypass outfall at the Chehalis River. The 2,900-foot-long bypass pipe is buried in the hillside at a slope necessary to maintain the appropriate hydraulic pressure.
- A bypass outfall – The bypass outfall structure is located on the banks of the Chehalis River downstream of the stilling basin. The concrete outfall structure transitions the fish and flow discharged from the bypass conduit to an open-channel rectangular flume. At the end of the outfall structure, flow plunges into a deep, swift portion of the Chehalis River.

Additional information regarding design elements of the floating surface collector is provided in Appendix G – Fish Passage Alternative Concept Design.

11.5.4.2 Theory of Operation

The multi-port collector is intended to provide safe passage for downstream migrating juvenile salmonids, adult steelhead, and resident fish year-round, through the full range of reservoir storage pool elevations. Although juvenile salmonids and adult steelhead will only pass downstream during certain periods of the year, resident fish currently traverse this reach of the Chehalis River regularly. Resident fish may move upstream and downstream multiple times throughout the year. The multi-port collector is intended to provide resident fish as close to the same opportunities to migrate downstream as possible. Other alternatives address upstream migration of resident fish.

The multi-port collector is intended to provide downstream passage during normal reservoir operation only. The reservoir fluctuates between WSEL 588 to 628 normally over the course of a year to enhance water quality downstream. Flood events may bring the reservoir pool higher than EL 628, potentially as high as EL 709 in a PMF event. During flood events, the multi-port collector may be completely submerged. Concrete lids over each screen structure are intended to reduce the amount of debris that could clog or damage facility components. Goosenecks, small ports in the ceiling and/or walls, or other devices are included to prevent entrapment of air and floatation forces when the facility is submerged. It is expected that debris removal and inspection of the facility will be required following flood events.

A full-depth barrier net is placed upstream of, and outside the hydraulic zone of influence of, the water quality outlet works and flood regulation sluice intakes. The barrier net is attached to the left, upstream corner of the multi-port collector and extends to the left bank of the reservoir. The design and operation of the net are similar to those described in the FSC alternative. The top of the net will rise and fall with the reservoir to ensure fish guidance is maintained for the targeted normal reservoir fluctuation. At extreme flood stages, the net will be submerged and will require debris removal and careful inspection by divers prior to startup after flood events. A thin wall above the lower collection structures excludes fish from passing downstream of the multi-port collector within the reservoir.

The fish screens are designed to NMFS juvenile criteria. Approach velocity criteria are met, with a factor of safety, when the facility is operated at its normal intended attraction flow of 500 cfs. The low-head submersible pumps have capability to increase the attraction flow to 1,000 cfs. Approach velocity increases proportionally at the higher attraction flow.

The attraction flow pumps are capable of drawing water from any of the four screen structures within the multi-port collector via the pump plenum. Only one screen structure operates at a time. Each screen structure operates over a 10-foot reservoir range. The highest screen structure operates from reservoir WSEL 628 to 618 while the bottom screen structure operates from reservoir WSEL 598 to 588. Each screen structure is capable of overlapping the next-higher screen structure's operation by 2 feet to ensure that downstream passage is continuous when transitioning from the operation of one screen structure to another. For example, the bottom screen structure may operate up to reservoir WSEL 600 while the next-higher screen structure is brought online for operation at the low end of its operating range (reservoir WSEL 598 to 608). Screen structures are never operated in a submerged condition. The water surface in the reservoir is always well below the access walkway above the fish screens.

When the multi-port collector begins operation, the attraction water pumps are off. The dewatering gates on the downstream side of the screen structure being started are fully opened. Then, the attraction water pumps are slowly started, bringing the attraction flow up to the desired amount. Transitioning from operation of one screen structure to another is similar, except the attraction water pumps continue running while one set of dewatering screens is slowly opened and the other set is slowly closed. Attraction flow is drawn in through the inlet of the screen structure being operated. As flow passes down the screen channel, water is drawn out through the fish screens and flow in the channel accelerates within NMFS criteria until approximately 25 cfs remaining in the channel passes over the end of the traveling ramp and into the bypass hopper.

Inside the hopper, the water surface is kept high to reduce the plunge depth off the end of the traveling ramp. The hopper is designed such that once flow falls into the hopper it is carried immediately and smoothly straight down into the vertical pipe at the bottom of the hopper and becomes pressurized flow. The pipe attached to the hopper slides within a larger pipe as the hopper elevation varies with the reservoir. A series of gaskets between the sliding pipe attached to the hopper and the larger stationary pipe seal the gap between the pipes and keep it mostly water tight. After the sliding pipe, the water continues down vertically, entering a 15-foot-radius vertical sweep to bring the pipe back up to the elevation needed to penetrate the dam. Pressurized flow in the pipe continues until the water is brought back to open channel flow at the consolidation structure on the downstream side of the dam. The bypass pipes from each of the four screen structures will operate at roughly the same flow rate, but each of the pipes is of a different diameter in order to achieve the same open-channel water surface elevation in the consolidation structure. The uniform water surface elevation in the consolidation structure allows the bypass flow for any normal operating reservoir elevation to be channeled to a single 18-inch-diameter bypass pipe for the remainder of the return to the river.

At the downstream end of the consolidation structure, the flow from any of the four screen structures is returned to pressurized pipe flow. Flow in this pipe may range from 15 to 25 fps. The bypass pipe leaves the consolidation structure and is immediately buried in the hillside. It continues for about 2,900 feet at a constant slope down to the river. The pipe length and slope are determined by the hydraulics to ensure the bypass pipe remains pressurized but does not experience a pressure more than 1 atmosphere above ambient air pressure.

The bypass outfall is located at the edge of the river, at the downstream end of the bypass pipe. The outfall structure transitions the bypass from a pipe to a rectangular shape and the water changes from pressurized flow to open-channel flow. Once achieving open-channel flow, the water flows a short distance before penetrating the construction diversion outlet headwall and plunging into a deep, swift portion of the Chehalis River. Plunge velocities will be no greater than 25 fps, the maximum allowed in the NMFS criteria. Above about a 2-year flood event, the bypass outfall channel will be submerged.

11.5.4.3 Anticipated Fish Passage Performance and Survival

The following is the anticipated fish passage performance and survival for the Multi-Port Collector alternative.

Table 11-10
Fixed Multi-Port Collector Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT DOWNSTREAM			
Spring Chinook	-	-	**-**
Fall Chinook	-	-	**-**
Coho	-	-	**-**
Winter Steelhead	50%	90%	**45%**
Coastal Cutthroat	65%	90%	**59%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
JUVENILE DOWNSTREAM			
Spring Chinook	65%	99%	**64%**
Fall Chinook	65%	99%	**64%**
Coho	65%	99%	**64%**
Winter Steelhead	65%	99%	**64%**
Coastal Cutthroat	65%	99%	**64%**
Pacific Lamprey	3%	20%	**0.6%**
Western Brook Lamprey	3%	20%	**0.6%**

The anticipated fish passage performance and survival is based on the performance of other fixed fish collection facilities and adjusted based on assumptions related to the conditions that are unique to a proposed FRFA dam on the Chehalis River. There are a number of fixed collector bypass facilities that can be used to inform selection of performance and survival values but are not identical to the proposed alternative. Examples are:

- River Mill Fixed Collector and Bypass
- Pelton-Round Butte Fixed Collector
- Soda Springs Bypass Facility
- Cowlitz Falls Fixed Collector and Bypass

The Cle Elum Dam Multi-Port Fixed Collection Facility with helical bypass is currently under construction. Physical modeling results are available through the U.S. Bureau of Reclamation but only provide information on the design of the helical bypass and do not provide insight into the performance of the collection ports.

The following are assumptions made regarding the performance and survival through Multi-Port Collector at the proposed FRFA dam on the Chehalis River:

- The selected performance value is intended to accommodate fish passage from the head of the reservoir to the point of release downstream of the dam.
- It is assumed that with adequate configuration in the dam face and sufficient attraction flow, each individual collection port should perform as well as the fixed and floating surface collectors currently in operation with performance values in excess of 0.9, similar to the River Mill and Soda Springs facilities.
- The success of reservoir transit is speculative, and site-specific studies are likely to be required after implementation has occurred to determine performance. All values were therefore reduced from the expected facility performance to account for this uncertainty in the reservoir.
- Survival values were slightly increased from those given for floating surface collectors to account for use of a passive bypass system.
- Lamprey passage performance and survival values are expected to be low, given the uncertainty and lack of data available for this type of technology.

12 OPERATIONS AND MAINTENANCE

12.1 Hydrologic Modeling of Operations

Modeling of the reservoir operations for the FRO and FRFA alternatives was conducted using HEC-ResSim by Anchor QEA with detailed descriptions of the operations and results presented in the Draft Operations Plan for Flood Retention Facilities (Anchor QEA, 2016). A summary of the flood operations for each alternative from that report are presented below.

12.1.1 FRO Alternative

- Flood operations are defined to begin when the flow at the Grand Mound gaging station is expected to exceed 38,800 cfs within 48 hours. Non-flood operations are defined to include all other scenarios.
- Flood operations include reducing project flows at a maximum rate of 200 cfs/hr to a discharge of 300 cfs by slowly closing the outlet gates.
- The 300 cfs outflow would continue until the peak of the flood passes Grand Mound, which is typically 48 to 72 hours.
- After the flood event, the pool will be emptied by opening the gates to increase project outflow at a maximum rate of 1,000 cfs/hr, which is estimated to limit the rate of pool water level drawdown to no greater than 10 feet per day to maintain reservoir sideslope stability.
- Debris management operations after a major flood would start when pool level is approximately El 528, at which time the drawdown rate would be limited to 2 feet per day to allow boats to safely remove logs and debris.
- Outflows would not be regulated for non-flood operations.

12.1.2 FRFA Alternative

- Flood operations are defined to begin when the flow at the Grand Mound gaging station is expected to exceed 38,800 cfs within 48 hours. Non-flood operations are defined to include all other scenarios.
- Flood operations include reducing project flows at a maximum rate of 200 cfs/hr to a discharge of 300 cfs by slowly closing the outlet gates.
- The 300 cfs outflow would continue until the peak of the flood passes Grand Mound, which is typically 48 to 72 hours.
- After a flood event, the pool will be reduced to the Conservation pool level (El 628 ft) by opening the gates to increase project outflow at a maximum rate of 1,000 cfs/hr, which is

estimated to limit the rate of pool water level drawdown to no greater than 10 feet per day to maintain reservoir sideslope stability.

- Specific debris management operations would not be necessary after a flood event, since there would be a permanent Conservation pool level that facilitates debris removal at all times other than during flood operations.

12.2 Dam Facilities Operations and Maintenance

12.2.1 Sediment Management

The FRO dam alternative sedimentation management costs are considered included in the annual debris management cost, since no additional staff would be required to conduct sediment management activities. These activities would take place during periods when there is no storage in the reservoir.

The FRFA has no sediment management costs expected, because the reservoir life span assumes that reservoir dead storage volume is adequate to store the bedload generated during the expected life span of the project.

12.2.2 Reservoir Maintenance (Debris Management)

Reservoir debris management operations have been assumed to be similar between the FRO and FRFA alternatives, and are expected to be conducted by the typically available staff. This is commensurate with other projects similar to the FRO and FRFA alternatives (Mud Mountain Dam and Howard Hanson Dam, respectively). These activities are expected to include:

- Vegetation management, harvesting potentially submerged trees
- Reservoir debris-handling labor for moving material to land-based handling area
- Debris handling/disposal trucks for hauling to final disposal site

12.2.3 Facilities Requirements

Required maintenance and periodic rehabilitation and possibly replacement of mechanical and electrical features such as gates, valves, hoists, and other similar features are expected over the life of the project. Some dam features and equipment will experience wear and tear due to typical use, such as sediment passage wear on the concrete sluice surfaces and the stilling basin floor. Weathering-related issues are addressed through operations and maintenance practices that extend the useful life of the dam.

- FRO
  - Operations and maintenance (O&M) costs for structural and mechanical equipment will be influenced by sediment effects as sediment is passed through the hydraulic structures causing additional wear and tear on sluices, gates, and stilling basin, with repairs expected at least every 5 years.

   - Major mechanical equipment such as radial gates and bulkheads are estimated to require rehabilitation every 20 years.
- FRFA
   - Sediment will not typically be passed through the outlet works, and significant wear of concrete surfaces is not expected.
   - Major mechanical equipment such as radial gates and bulkheads will require rehabilitation every 50 years.

12.3 Fish Passage Facility Operations Summary

The operational requirements of each fish passage alternative varies based upon the need to meet both upstream and downstream fish passage objectives for the range of target fish species and life stages identified in Section 11.3.2. Fish passage facility operation is therefore dictated by the anticipated upstream and downstream migration timing of fish migrating through the proposed dam reach. Facilities are anticipated to be operating to facilitate safe and timely passage while fish are present. A detailed description of the operational theory is presented for each potential fish passage facility in Sections 11.4 and 11.5 and describes how each specific design element works together to facilitate fish passage upstream or downstream. In addition, fish passage technologies are to be operated as a system of one upstream and one downstream facility type. In many cases, there are economic efficiencies associated with the same staff operating and maintaining both facilities. Therefore, for the purposes of assessing the overall personnel requirements and level of effort, systems of fish passage facilities were evaluated. Table 12-1, below, summarizes the overall operational requirements and durations for the purpose of estimating the level of effort and subsequent cost associated with annual operation of fish passage alternative options.

Table 12-1
Summary of Operational Requirements for Potential Fish Passage Facilities

ALTERNATIVE	AVERAGE ANNUAL DURATION	PERSONNEL REQUIREMENTS (FTE = FULL TIME EQUIVALENTS)	SPECIAL CONSIDERATIONS
FRO – Fish Passage Conduits and FRO CHTR Facility	FRO Conduits – 12 months FRO CHTR – 1 month	Operations Staff - 0.01 FTE Biological Staff - 0.03 FTE Maintenance Staff - 0.03 FTE Total = 0.07 FTE	• Operation of CHTR facility daily during operational periods. • Operation of transport vehicle required daily during CHTR operation.
FRFA – Upstream Conventional Fishway with Downstream Floating Surface Collector	12 months	Operations Staff – 0.75 FTE Biological Staff – 2.58 FTE Maintenance Staff – 0.15 FTE Total = 3.48 FTE	• Operation of facility daily during operation. • Daily inspections required to maintain optimum hydraulic settings and perform debris removal. • Operation of transport vehicle required daily during operation. • Creates substantial power demand of up to 1-megawatt during operation.
FRFA – Upstream Conventional Fishway with Downstream Fixed Multi-Port Collector	12 months	Operations Staff – 0.5 FTE Biological Staff – 1.75 FTE Maintenance Staff – 0.12 FTE Total = 2.37 FTE	• Operation of facility daily during operation. • Daily inspections required to maintain optimum hydraulic settings and perform debris removal. • Creates substantial power demand of up to 1-megawatte during operation.
FRFA – Upstream CHTR Facility with Downstream Floating Surface Collector	12 months	Operations Staff – 1.25 FTE Biological Staff – 3.75 FTE Maintenance Staff – 0.23 FTE Total = 5.23 FTE	• Operation of each facility daily during operation. • Daily inspections required to maintain optimum hydraulic settings and perform debris removal. • Operation of transport vehicle required daily during operation. • Creates substantial power demand of up to 1-megawatt during operation.
FRFA – Upstream CHTR Facility with Downstream Fixed Multi-Port Collector	12 months	Operations Staff – 1.0 FTE Biological Staff – 2.92 FTE Maintenance Staff – 0.2 FTE Total = 3.94 FTE	• Operation of each facility daily during operation. • Daily inspections required to maintain optimum hydraulic settings and perform debris removal. • Operation of transport vehicle required daily during operation.

ALTERNATIVE	AVERAGE ANNUAL DURATION	PERSONNEL REQUIREMENTS (FTE = FULL TIME EQUIVALENTS)	SPECIAL CONSIDERATIONS
			• Creates substantial power demand of up to 1-megawatt during operation.

12.3.1 Fish Passage Facility Maintenance

12.3.1.1 FRO – Fish Passage Conduits

Maintenance of the conduits is required regularly following impoundment events, annually, and infrequently as needed for the repair and replacement of major features. Operation of the conduits is a function included in the overall operation of the dam; therefore, maintenance of the conduits is addressed accordingly.

12.3.1.2 FRO and FRFA – CHTR Facility

Frequent and periodic maintenance of the CHTR facility will be required. A summary of the regular, annual, and periodic maintenance requirements is provided in the following paragraphs.

- Regular maintenance occurs throughout operation within the fish passage design flow range. It includes:
  - Debris removal within the fish ladder and fish lift areas. Supply water is screened so debris is expected to come primarily from falling debris, such as leaves.
  - Cleaning of tanks, holding galleries, work-up tables, flumes, and other holding and sorting facilities to ensure fish health.

Many features of the CHTR do not require maintenance on a regular basis but do require annual inspection and/or maintenance to keep them in good working order and prolong their functional life. Annual maintenance tasks may include:

- Service manual and motorized gate operators.
- Service trapping and lift systems, including repainting of steel parts and service motors.
- Inspect, service, and replace, if necessary, all gates, crowders, and other fish handling and sorting equipment pumps.
- Inspect fish screens and trashracks for damage.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Repaint buildings and steel pipes and structures.
- Replace trash rakes.

- Replace gate operators, trapping system components, lift system components, crowder motors, and other fish handling and sorting equipment.
- Replace bypass pipe(s) between bypass hoppers and the dam penetration as necessary as the result of damage from exposure, debris, or flood events.
- Replace in-line energy dissipation valve and other shutoff and control valves as necessary.

Unlike the CHTR facility for the FRFA dam alternative, the CHTR facility for the FRO dam alternative will have special maintenance requirements because the facility is expected to remain unused for a large portion of the year or even for numerous years at a time. Therefore, special attention to exercising equipment and maintaining systems in good working order will still be required even though the facility might not be operating. Regular maintenance anticipated for the FRO CHTR facility is summarized below:

- Startup
  - Inspect and operate motors, gates, gate actuators, and other equipment. Clean and lubricate as necessary. Ensure equipment is in good working order before beginning fish passage operations.
  - Clean all tanks, holding galleries, work-up tables, flumes, and other holding and sorting facilities.
  - Inspect electrical equipment.
  - Test all lighting and replace bulbs as necessary.
  - Inspect all doors and fence gates. Lubricate as necessary to ensure they are in good working order.
  - Clean debris from access roads and walkways.
- During operation
  - Same as described for FRFA dam alternative previously
- Shutdown
  - Service manual and motorized gate operators.
  - Inspect, service, and replace, if necessary, all gates, crowders, and other fish handling and sorting equipment pumps.
  - Inspect fish screens and trashracks for damage.
  - Shut down and depower all equipment not requiring power during dormant periods.
  - Move all loose equipment, parts, tools, and other items necessary for operation into the storage building.
  - Set lighting and security systems as required during vacancy.
  - Secure storage building, electrical/mechanical building, and the site as required during vacancy.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Service trapping and lift systems, including repainting steel parts and service motors.
- Repaint buildings and steel pipes and structures.
- Replace trash rakes.
- Replace gate operators, trapping system components, lift system components, crowder motors, and other fish handling and sorting equipment.
- Replace bypass pipe(s) between bypass hoppers and the dam penetration as necessary as a result of damage from exposure, debris, or flood events.
- Replace in-line energy dissipation valve and other shutoff and control valves as necessary.

12.3.1.3 FRFA – Conventional Fish Ladder

Regular maintenance of the conventional fish ladder will be required throughout operation. Maintenance activities can be broken into regular, annual, and infrequent activities.

Regular maintenance occurs throughout the normal operation of the reservoir. It includes:

- Inspection and debris removal inside the fish ladder, especially at the fish ladder exit following submergence, both at flood events and for the lower pools during normal operation. Material will be small enough to fit through the trashracks at the exit gates. Minimal debris removal is expected.
- Debris removal from trashracks at the exit gates.
- Inspection and debris removal from the auxiliary water diffuser screens.
- Inspection of mechanical and electrical systems following submergence, both at flood events and for the lower pools normal operation

Many features of the conventional fish ladder do not require maintenance on a regular basis but do require annual inspection and/or maintenance to keep them in good working order and prolong their functional life. Beyond the regular needs for annual inspection, regular cycles of submergence and exposure throughout the normal operating period put additional wear on all the equipment and metal work in the fish ladder exit. It is expected that maintenance tasks that may ordinarily be undertaken every few years on the half-ice harbor fish ladder and the fish ladder entrance will need to occur annually on the fish ladder exit. Annual maintenance tasks may include:

- Repaint steel structures at the fish ladder exit.
- Inspect, operate, and service the emergency shutoff gate at the dam penetration and the fish ladder entrance gates.
- Service manual and motorized gate operators.

- Inspect and service the in-line energy dissipation valve and fish ladder exit gates.
- Drain fish ladder and inspect all fish ladder pools, weirs, and orifices. Remove debris. Repair damaged concrete.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Repaint steel structures at the half-ice harbor fish ladder and the fish ladder entrance.
- Inspect and repair access grating and supports as necessary.
- Replace gate operators as necessary.
- Repair or replace fish ladder entrance and exit gates that may have been damaged during flood events.
- Maintain gravel bench at the fish ladder exit.
- Maintain access road.

12.3.1.4 FRFA – Floating Surface Collector

The floating surface collector is composed of numerous complex mechanical, structural, and floatation systems that must all work in tandem to produce adequate performance. Maintenance activities can be broken into regular, annual, and infrequent activities.

Regular maintenance is expected to occur throughout the normal operation of the facility. It can be expected to include:

- Weekly and sometimes daily removal of debris inside the screen structure and fish handling systems. Debris material will consist of buoyant and semi-buoyant material small enough to fit through the primary trashracks, which reside in front of the net transition structure.
- Weekly and sometimes daily removal of debris captured at the primary trashracks.
- Inspection and adjustment of mechanical screen-cleaning equipment.
- Inspection and adjustment of ballast, floatation, and hydraulic control systems.
- Typical vehicle maintenance activities for hopper transfer trucks and facility support boats.
- Removal of debris from guidance and lead nets following flood events.

It is expected that several maintenance tasks will be required on an annual basis on the floating surface collector. Annual maintenance tasks may include:

- Repaint steel structures including gantry equipment, access tower, guide rails, and mooring towers.
- Inspect, service, and replace if necessary all pumping equipment associated with attraction flow, ballasting systems, and service water.

- Service screen-cleaning system, including repainting of steel parts and service motors.
- Inspect and service ramp gates and hoppers, including repainting steel parts and service motors.
- Inspect fish screens and trashracks for damage.
- Inspect, modify, repair, and/or adjust porosity control systems behind the primary and secondary dewatering screens.
- Inspect and repair or replace guidance and lead nets.

Infrequent maintenance primarily refers to maintenance that is conducted on an as-needed basis. It may be several years between such maintenance activities or as components unexpectedly fail while in use. Infrequent maintenance activities may include:

- Replace or repair trash rakes.
- Replace gate operators.
- Replace instrumentation and control equipment associated with all ballast, hydraulic, and mechanical monitoring systems.
- Replace attraction water pumps as necessary as a result of damage or normal wear.
- Repair or provide replacement parts for hopper transfer trucks and facility support boats.
- Replace mooring buoys, floats, weights, and rope “frames” for net panels associated with the guidance and lead nets.

12.3.1.5 FRFA – Fixed Multi-Port Collector

Maintenance of the fixed multi-port collector will be required throughout its operation and can be broken into regular, annual, and infrequent activities.

Regular maintenance occurs throughout the normal operation of the reservoir. It includes:

- Debris removal inside screen structures following submergence, both following flood events and for the lower screen structures during normal operation. Material will be small enough to fit through the trashracks.
- Debris removal from trashrack of screen structure in operation.
- Inspection of mechanical and electrical systems following submergence, both in flood events and for the lower screen structures during normal operation.
- Removal of debris from guidance net following flood events.

Regular cycles of submergence and exposure throughout the normal operating period put additional wear on all the equipment and metal work in the multi-port collector. It is expected that maintenance tasks that might ordinarily be undertaken every few years on an exposed structure will need to occur annually on the multi-port collector. Annual maintenance tasks may include:

- Repaint steel structures.
- Repaint bypass pipes between bypass hoppers and the dam penetration, depending on material selected.
- Service manual and motorized gate operators.
- Service screen-cleaning system, including repainting steel parts and service motors.
- Inspect, service, and replace, if necessary, all attraction water pumps.
- Inspect and service ramp gates and hoppers, including repainting steel parts and service motors.
- Inspect fish screens and trashracks for damage.
- Inspect and repair or replace guidance and lead nets.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Replace attraction water pumps.
- Replace trash rakes.
- Replace gate operators.
- Repair or replace bypass outfall damaged by flood event.
- Replace bypass pipe(s) between bypass hoppers and the dam penetration as necessary as the result of damage from exposure, debris, or flood events.
- Repaint bypass pipes between bypass hoppers and the dam penetration as necessary and depending on pipe material selected.
- Replace attraction water pumps as necessary as the result of damage or normal wear.
- Replace mooring buoys, floats, weights, and rope "frames" for net panels associated with the guidance and lead nets.

12.4 Other O&M Considerations

Other O&M considerations include:

- Fish/environmental monitoring and reporting
- Operations, dam tender, security
- Administrative management, reporting and legal/insurance

These items are beyond the scope of conceptual design and will need to be evaluated during design.

13 OPINION OF PROBABLE COSTS

13.1 Cost Summary

Appendix F provides summarized OPCC for both the FRO and FRFA dam options. Estimated quantities and unit costs are provided for a reasonably thorough work breakdown structure for a concept-level estimate. Additionally, a low-cost range and a high-cost range are established by considering potential quantity variations and unit price variations important for a concept-level cost estimate. Each estimate generates a total for a contractor bid, to which the following factors are added: design contingency (20%), post-award construction contingency (10%), and a noncontract cost factor (25%). The noncontract costs are costs apart from the construction contract that would include design and site characterization, permits, construction management (CM), etc. Table 13-1 summarizes the opinion of probable construction costs for the FRO and FRFA options:

Table 13-1
Concept-Level Opinion of Probable Construction Costs: Summary of Key Information

	FRO	FRFA
Total Weighted Project Cost	250,000,000	371,000,000
Low End Project Cost	209,000,000	315,000,000
High End Project Cost	306,000,000	450,000,000
Project Cost Range from Total Weighted	84% - 122%	85% - 121%
RCC Unit Bid - Likely	$ 80.00	$ 83.00
RCC Unit Bid Range	$ 66.00 - $ 97.00	$ 69.00 - $ 100.00
RCC - as % of Contractor Bid	38%	47%
Main Dam (including outlet works) - as % of Contractor Bid	69%	74%
Phase 1 - Site Prep - Diversion - % of Contractor Bid	21%	17%
Diversion - as % of Contractor Bid	9%	6%

13.1.1 Estimated Life of Project (Years)

13.1.1.1 Dam

Large dams have a history of solid performance for well over 100 years. Understanding of the materials and construction techniques has improved, construction methods have become more efficient, and the quality of construction has increased significantly over that time. These important factors all contribute to the expected significant longevity of the dam and major hydraulic structures beyond decades, to perhaps centuries.

Key structural factors that typically limit longevity of the dam, such as wear and tear on the dam and associated machinery due to typical use, sediment passage wear on the concrete sluice surfaces and the stilling basin floor, and weathering-related issues, have been addressed through operations and maintenance practices that extend the useful life of the dam. Sediment transport processes are expected to wear the sluice conduit concrete and exposed steel features over time, with repairs expected at least every 5 years. Since sediment will not be typically passed through the outlet works of the FRFA alternative, significant wear of concrete surfaces is not expected. Therefore, all major concrete features are considered to have a typical expected life span of 100 years. Required maintenance and periodic rehabilitation and possibly replacement of mechanical and electrical features such as gates, valves, hoists, and similar features are expected over the life of the project. The expected life span and operational regime have been considered in the determination of the annualized O&M costs of structural and mechanical equipment subject to wear or fatigue. Because sediment will be passed through the FRO alternative, wear and tear on sluices, gates, and stilling basin will cause greater damage than in the FRFA option. As a result, we expect that major mechanical equipment such as radial gates and bulkheads will require rehabilitation every 20 years in the FRO alternative and every 50 years in the FRFA alternative.

The FRFA alternative is currently configured with 65,000 acre feet of active storage and 65,000 acre feet of dead storage that would be infilled with sediment over time. The FRO alternative has flood storage of 65,000 acre feet and no dead storage; however, the reservoir is dry when not retaining floodwater. Sedimentation studies indicated that bedload and 86 to 93 percent (2014) of the suspended load would be trapped in the reservoir for the FRFA alternative, with an estimated average load of 42 acre feet per year, which is a relatively small infill rate compared with the available 65,000 acre feet of dead storage for sediment storage. A simple calculation shows that, on average, this volume of bedload sediment will pass into the reservoir over about 1,500 years.

Approximately 25 to 50 percent of the bedload supply would be trapped in the FRO reservoir, the equivalent of 4.3 to 8.7 acre feet per year on average. Though minor removal activities might be initiated at intervals over the life of the project, we expect the reservoir to not be materially affected by the sediment inflow.

During the extreme 2007 flood, an estimated 2,050 to 3,100 acre feet of coarse sediment was delivered to streams in the watershed upstream of the proposed dam site. Some portion of this material was deposited in channels upstream of the dam site, but much of it would have been transported into the proposed reservoir area, which would have been full at the time under either scenario. The estimate of 3,100 acre feet of coarse sediment is less than 5 percent of the FRFA reservoir dead storage capacity.

FRO operations can accommodate annual sediment removal since there is no reservoir storage during normal flow conditions. The FRFA can be drawn down to allow sediment excavation without a reservoir if there is an operational advantage to do so.

13.1.1.2 Fish Facilities

Some fish passage facilities have records of solid performance for over 50 years while others are fairly new technologies that have been shown to be effective but are still being closely monitored by fisheries agencies. Continued maintenance and periodic rehabilitation are considered part of the estimated facility life span. For the purposes of this estimate, a fish passage facility is considered at the end of its useful life when the rehabilitation cost is estimated to be greater than the replacement cost. The estimated useful life of the fish passage facilities being considered is based on professional judgment and consideration of existing similar facilities.

For this project, we assume the following life spans for various components of the Fish Passage Facilities:

- Collect, handle, transfer, and release (CHTR) Facility: 25 years
- Conventional Fishway: 50 years
- Floating Surface Collector (FSC): 30 years
- Fixed Multi-Port Collector (FMPC): 50 years

13.2 Methodology, Contingencies, and Assumptions

The cost estimate will use multiple methods to arrive at the opinion of probable cost.

- The estimate will largely use unit price estimates developed from experience on other projects.
- For the RCC unit price, which is one of the most influential quantities on the project, a composite unit price will be generated from component costs.
- Comparison will be made to proxy projects of similar scale and size.

The potential for reservoir landslides could affect future costs from a capital-first cost perspective, as well as a long-term operation and maintenance cost perspective. This document does not explore these costs in detail. The landslide potential could lead to consideration of possible approaches to stabilization or mitigation, or landslides could simply be acknowledged as an ongoing reservoir operations and maintenance cost if no treatment is required.

13.3 Total Direct Project Costs

Total direct project construction costs for both the FRO and FRFA alternatives are summarized in Appendix F. At this point in our conceptual design, the costs for several new project features were included that were not specifically listed in previous cost estimates, and the design contingency has been reduced to reflect advancement of the design. As the design advances, it is anticipated that more individual line item costs may be identified, and design contingency costs would be reduced. Therefore, increases or decreases in the cost estimate compared with previous cost estimates are not necessarily reflective of changes in project costs; HDR's opinion of probable cost is reflected in the ranges provided. Discussion of the cost development and particular considerations for each are discussed below.

13.3.1 Roads

Direct costs for road construction not associated with dam construction and reservoir are not applicable for the following reasons:

1. Construction roads will be required to support construction activities within the dam footprint, quarry, diversion staging, and potential laydown and disposal areas; however, they are included in the current overall dam construction costs.
2. Access to lands affected by the reservoir will remain via alternate routes to forest harvest and management areas via existing forest roads. However, access to some parcels will be somewhat restricted without construction of new roads or extensions to existing road networks, which will increase the cost for timber maintenance and harvest. These costs have been included in the direct dam construction costs as an allowance for a nominal amount per mile of affected access roadways.

13.3.2 Land and Land Rights

Land and land rights were not included as line items in the previous biennium OPCC, and were assumed to be covered under the contingency amount. In this Phase 2 effort, we have developed estimates based on the areas of inundation for the FRFA and FRO options. All lands inundated below the elevation of the 100-year flood anticipated reservoir pool level are assumed to require fee title purchase for the project, while lands between the maximum reservoir pool expected for the design flood event (roughly equivalent to the 2007 event) and the 100-year inundation limit are assumed to require a flooding easement. These costs are now included in the direct dam construction costs.

The valuation that was used as an estimate for the land cost-per-acre for this project is based on research done using the county assessor's website and the Northwest Multiple Listing Service (NWMLS). After the assessed value of the impacted parcels and their uses were ascertained, the NWMLS was used to look up sales of comparable land in the area of the project, that have sold within 6 months of the current date. Three such sales were found, and their value was used for the basis. This value was doubled to $4,356/acre to account for the cost of timber loss and the possible abandonment of access roads that will need to be negotiated with the landowners. Although these actual sales make for a solid valuation at this point, other similar properties currently listed are in a higher range, and could possibly sell for higher prices, thus driving up the cost of the acquisition. By doubling the price range established by the comparable sales, we are hoping to lessen the impact of these variables, should the price increase when acquisition takes place for this project.

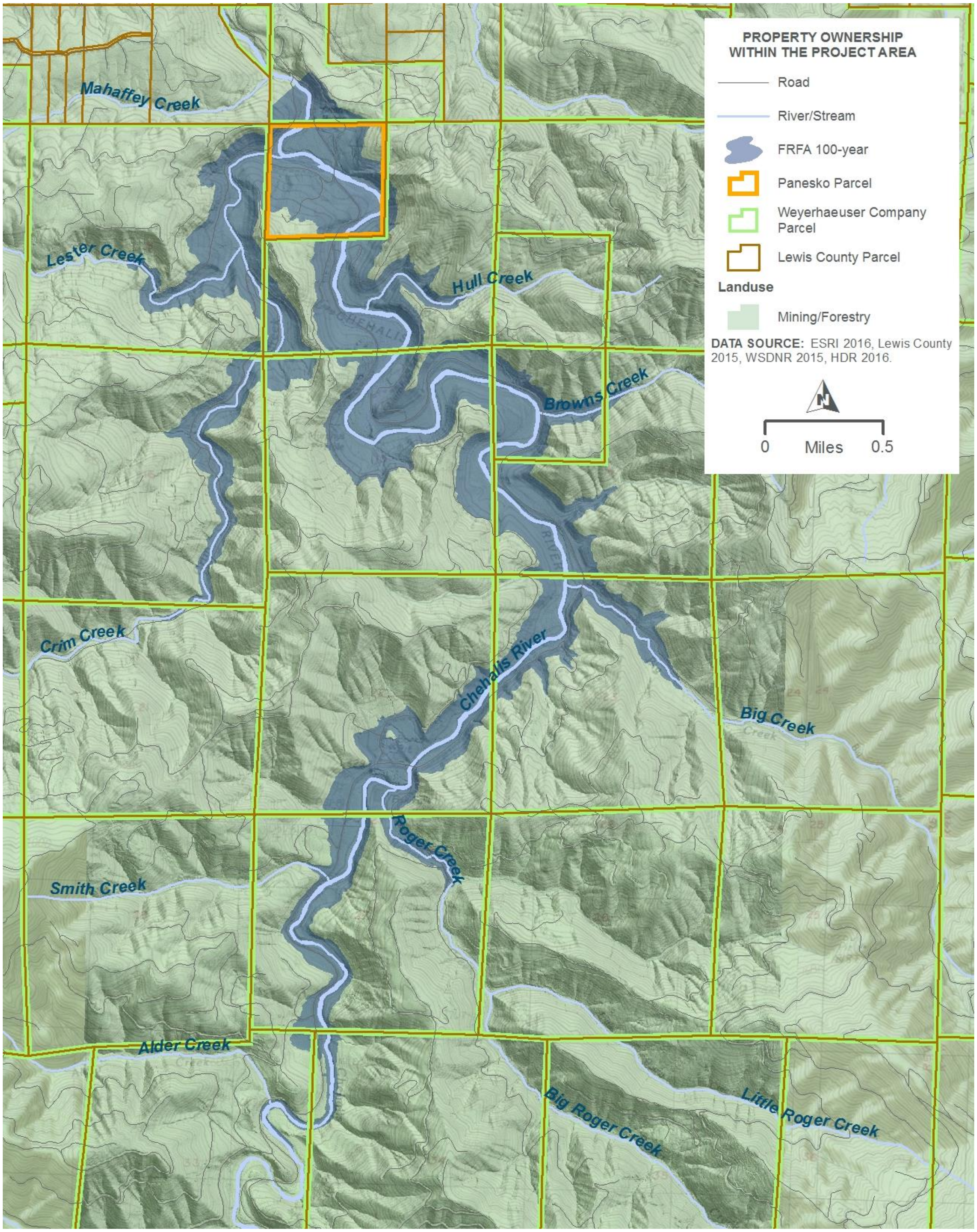
The anticipated number of landowners in the affected area is low (just three were identified), and the anticipated effort to work with them to acquire land for the project is not expected to be high. Although working with Weyerhaeuser will require negotiations, and the process may be lengthy, costs will be reduced by the fact that a single company owns much of the property needed. Title and appraisal costs are expected to be fairly standard and will yield the best valuation to use for the acquisition of these properties.

The FRO reservoir will be largely empty except during times of flooding. The approach adopted is conservative, looking at the flood inundation levels for determination of the land acquisition requirements. All lands inundated below the 100-year flood reservoir level would require outright purchase at full price, while inundated areas above that up to the maximum design reservoir elevations would require a flood easement equivalent to full price. However, depending on the landowner negotiations, the easement value might decrease after more details are developed.

The FRFA option will have a permanent pool. In addition to the area of land directly inundated, access to some of the forest lands will be limited or unavailable unless and until the existing forest road network is extended to those orphaned parcels. As with the FRO alternative, all lands inundated below the 100-year expected flood reservoir level will require outright purchase, and those above the 100-year up to the maximum design reservoir level will require flooding easement at full purchase price. Again, these easement costs might decrease as discussions with the landowners progress.

Figure 13-1 below illustrates the affected real estate within the reservoir limits.

Figure 13-1
Land and Land Rights Map of Project Area



13.3.3 Dam Structure and Related Equipment

13.3.3.1 Dam Foundation Excavation

Surficial materials and weathered rock will need to be excavated to provide an adequate foundation for the RCC structure. Preliminary excavation quantities have been calculated by estimating two critical excavation depths: 1) depth to top of rock, and 2) depth to limit of rippable rock and 3) depth to hydraulic structure foundations. All of the subsurface information collected to date, including primarily borings and geophysical lines, was considered when estimating these depths. Excavation from the ground surface to the top of rock is considered general or common excavation. The unit price for this excavation was estimated to range from $5.50 to $8.00 per cubic yard. The top of rock surface is readily observable from the borings; however, geophysics and other subjective judgment was applied to estimate the depth to top of rock at other locations. Excavation of weathered rock involves more effort, different equipment, and more time compared with common excavation. Some areas and types of weathered rock will be excavated more readily than other areas. The estimated depth to rippable rock was equated to the foundation excavation objective for the purpose of this quantity and cost estimate. Rippable rock is often an equipment-performance-based quality indicator, and is correlated to the geophysical p-wave velocity. Rock is considered rippable when a dozer (or track excavator) of a certain size (D10, for example), equipped with a single tooth ripper, can remove rock in a production mode. For this site, the limit of rippable rock was assumed to equate to a p-wave velocity of approximately 9,000 ft/sec. The depth was selected based on consideration of all the subsurface data, particularly the rock description, rock weathering, RQD, fracture spacing, downhole testing, and other indications of rock quality. The unit price for this excavation was estimated to range from $25 to $36 per cubic yard. Subsequent cost estimate development will itemize dental excavation and related foundation preparation and treatment that are not differentiated in the current estimate detail.

Recent information from the Phase 2 field exploration generally indicates the RCC foundation excavation objective is slightly lower than what was assumed after the Phase 1 field exploration. In some cases, the depth to rock of adequate quality was in excess of 50 feet. The limits of the dam foundation will continue to be refined as the design evolves and specific consideration is given to stability and seepage control measures.

13.3.3.2 RCC Dam

Unlike the dam foundation quantities, which depend on the amount of rock excavation across the site, the quantity of RCC is less variable because the cross-section geometry is fixed above the foundation contact. The slight increase in excavation depth mentioned above results in a slight increase in RCC quantities. The unit price of RCC is a more significant variable than the quantity, and it is primarily dependent on factors such as the facing and seepage control design choices, type and location of aggregate, the cost of cement, the cost of pozzolans, the size and production rate of the RCC plant, and the overall duration and speed of RCC placement. For the purpose of this cost estimate, the unit price of the RCC is likely to be about $80 per cubic yard, with a high-to-low range of $65 to $97 per cubic yard.

This unit price is based on judgment-level unit cost ranges for the composite components of aggregate, cement, and pozzolan, mixing, delivery, placement, dam joints, dam facing, and dam drainage features. Table 13-2 provides the FRFA's RCC unit cost development, while Appendix F contains the RCC unit price buildup for both the FRO and FRFA options.

Table 13-2
FRFA Roller Compacted Concrete (RCC) Dam Annual O&M Costs

COST AREA	COST CATEGORY	COST ITEM	COST BASIS	VALUE[1]	UNIT $	ANNUAL $[2]
DAM AND RELATED FACILITIES						
Reservoir	Vegetation Management	Part Time Labor	FTE	1	$65,000	$65,000
Reservoir	Debris Handling	Part Time Labor	FTE	1.5	$65,000	$97,500
Reservoir	Debris Handling	Loaders/Trucks/Operators	LS	1.5	$50,000	$75,000
Reservoir	Fish/ Environmental	Monitoring/Reporting	LS	1	$40,000	$40,000
Dam	Operations	Dam Tender/Security	FTE	1.5	$85,000	$127,500
Dam	Administrative	Management	FTE	0.5	$120,000	$60,000
Dam	Administrative	Reporting	FTE	0.3	$100,000	$30,000
Dam	Administrative	Legal/Insurance	LS	1	$150,000	$150,000
Dam	Maintenance	Part Time Labor	FTE	1	$80,000	$80,000
Dam	Inspections	Safety Inspections	LS	1	$10,000	$10,000
Dam	Mechanical	Repair/Replace Fund	% Cap	0.8%	$12,767,588	$102,000
Dam	Structural	Repair Fund	% Cap	0.2%	$60,432,000	$121,000
		Subtotal				**$958,000**
FISH PASSAGE RECOMMENDED ALTERNATIVE – CHTR & FLOATING SURFACE COLLECTOR						
Fish Passage	Operations	Operator/Monitor	FTE	1.25	$122,550	$154,000
Fish Passage	Biological	Monitoring/Reporting	FTE	2	$129,000	$258,000
Fish Passage	Biological	Part Time Labor	FTE	2	$44,400	$89,000
Fish Passage	Maintenance	Part Time Labor	FTE	0.23	$129,000	$30,000
Fish Passage	Structural/ Mechanical	Repair/Replace Fund	LS	1	$15,000	$15,000
Fish Passage	Trap and Haul	Loaders/Trucks/ Maintenance	LS	1	$13,533	$14,000
Fish Passage	Electricity	General Service Loads/Pumping	kWh	5,773,0005	$0.09	$520,000
Fish Passage	Biological	Monitoring & Evaluation	FTE	0.53	$129,000	$68,000
Fish Passage	Biological	Monitoring & Evaluation PT	FTE	1.05	$44,400	$47,000
Fish Passage	Science Costs	Lab Tests, etc.	EA	2	$18,000	$36,000
		Subtotal				**$1,231,000**
		Total Annual Cost				**$2,189,000**

Notes:
1. Values rounded to nearest whole number (LS), 0.1 percent, 0.01 FTE, or 1,000 kilowatt hours.

2. Values rounded to nearest $1,000 US dollars for simplicity. Details provided elsewhere may vary.

13.3.4 Fish Facilities

Fish passage facility alternatives vary widely with respect to scale, complexity, and constructability. Major considerations influencing the development of fish passage facility construction costs are summarized below. The following section summarized cost factors considered for each fish passage alternative, as well as the current opinion of probable construction cost concurrent with the conceptual level of design. Further discussion of cost and factors that influence fish facility cost are discussed in Appendix G – Fish Passage Alternative Concept Design.

- FRO Fish Passage Conduits – The fish passage conduits in the FRO dam are integral to the FRO dam itself. Therefore, the estimated construction cost of the fish passage conduits is included in the FRO dam cost estimate.
- FRO Upstream Fish Passage CHTR Facility – The CHTR has the lowest construction cost of all the upstream fish passage alternatives. The short height and length of the fish ladder, the minimal work on steep slopes, as well as the lack of necessity for physical structures for upstream release, more than offset the cost of sorting, holding, and transport facilities compared with a conventional fish ladder. The CHTR requires significantly more personnel and man-hours to operate and maintain. When the operation and maintenance cost is amortized, the total life-cycle cost of the CTHR is still substantially less expensive than a conventional fish ladder.
- FRFA Upstream Fish Passage: CHTR Facility – Construction cost factors for the FRO and FRFA CHTR facilities are expected to be similar because the functionality, complexity, and function of each alternative concept is identical. See FRO CHTR, above.
- FRFA Upstream Fish Passage: Conventional Fish Ladder – The height and length of this fish ladder, as well as the large reservoir fluctuation, make the conventional fish ladder more expensive than conventional fish ladders on other rivers. The foundation costs associated this alternative are also much greater than many other conventional fish ladders. Excavation on steep slopes and slope stabilization add substantially to the overall cost. In addition, most other conventional fish ladders are designed to operate with reservoir fluctuations of 10 feet or less, with a few in the 15- to 20-foot fluctuation range. The much larger fluctuation in this reservoir requires over twice as many exit gates as other fish ladders.
- FRFA Downstream Fish Passage: Fixed Multi-Port Collector – The multi-port collector has the largest capital cost of all the fish passage alternatives. Construction of the collector in its required location makes the foundation costs associated with this alternative cost much higher than the other alternatives currently considered. Excavation on steep slopes, pile foundations, and slope stabilization add substantially to the overall cost. In addition, it is physically impractical to employ a single, tall set of fish screens to collect fish over the large fluctuation in reservoir elevation during normal operation. Most other downstream V-screen type fish collection systems that have been constructed address fluctuations of 10 feet or less, with only one in the 20- to 30-foot fluctuation range (Pelton Round-Butte, Deschutes River, OR). The large

fluctuation in this reservoir would require multiple fixed V-screen dewatering bays to remain within this practical operational range. As much of the construction cost is related to the equipment, metal work, and concrete associated with a screen structure, the necessity for multiple screen structures also multiplies the total construction cost.

- FRFA Downstream Fish Passage: Floating Surface Collector – There are five full-scale facilities similar to the floating surface collector conceptualized for the Chehalis River. Construction costs for existing facilities are reported to range from $24 million (Cushman) to $60 million (Swift). Although these costs may capture the pure field costs associated with facility fabrication and initial deployment, it is important to recognize that they may not adequately represent total implementation costs, which are reportedly in excess of $100 million for facilities like Upper Baker, Lower Baker, and Swift. Costs for a floating surface collector at the Chehalis River are expected to be similar to the costs of other facilities currently in operation. The collection barge, guide nets, lead net, mechanical components, fish transport systems, and operational parameters are similar to those of other facilities built within the last decade. Information from the construction and implementation of these existing facilities enhances the level of confidence associated with anticipated costs for the majority of proposed facility elements. Conversely, the mooring and anchoring system for this facility is unique and must also accommodate approximately 189 feet of reservoir fluctuation – only comparable to Swift, at 100 feet. Therefore, some additional variances and contingencies must be accommodated for this individual design element to adequately capture anticipated costs.

The alternatives considered for fish passage in this study were largely in a state of development throughout much of the conceptual design phase of work. As specific biological design criteria elements were refined, so were the primary configurations of each fish passage element. In response, potential construction costs have changed since the release of the previously published Combined Dam and Fish Passage Design documentation (HDR, 2014). Key changes from the previous cost estimates are as follows:

1. The FRFA – Upstream Fish Passage Experimental Fishway and Exit Tower alternative was eliminated from further consideration. Updated flood control operations resulted in a lower seasonal range if anticipated reservoir fluctuations and, therefore, a more conventional linear type of ladder exit was selected for further development which has; much less complexity and lower cost.
2. The FRFA – Downstream Fish Passage Combination Collection Facilities alternative was eliminated from further consideration. The Fish Passage Technical Subcommittee agreed that the complex combination of site-specific reservoir characteristics and lack of proven track record led to a high level of uncertainty with regard to fish passage performance. Further discussion of this alternative led to the introduction of the Fixed Multi-Port Collector alternative in its place.

3. The FRFA – Downstream Fish Passage Fixed Multi-Port Collector was added as a downstream fish passage option. This option developed only in this most recent phase of work. Although such a collector has never been constructed as proposed, the cost estimate for this option is largely based on the fixed multi-port collection system and helical bypass currently under construction by the USBUREC at Cle Elum Dam, Washington. The estimated cost was assumed to be about 30 percent less than that of the Cle Elum outlet, based upon the relative size and complexity of construction.
4. Upon refinement of the Conventional Fishway alternative and elimination of the Experimental Exit Tower option (described above), the fish ladder exit for this alternative must be more complex than previously assumed to accommodate the range of anticipated reservoir fluctuations. The cost estimate has been updated to reflect this complexity.
5. The Fish Passage Subcommittee made several decisions relating to the Floating Surface Collector alternative that changed the original assumptions regarding the operation of the facility. The changes in operation changed the size of the facility. These decisions have also provided additional direction, allowing the design to be developed further. These changes resulted in an increase in the estimated construction and operation and maintenance costs of the floating surface collector. The specific changes include:

   - The attraction flow of the floating surface collector facility has been nearly doubled, up to a potential attraction flow of 1,000 cfs.
   - The fish guidance and lead nets are about 2.5 times longer than previously considered to accommodate a different collection barge position and guidance approach angle.
   - Several supporting structures and facilities have been added to make the fish passage and handling facilities reflect the latest practice and experience.
   - The collection barge mooring and anchorage components were expanded to accommodate a normal operational range of 40 vertical feet and an emergency range of 189 feet for flood control events.

The anticipated construction costs for the potential fish passage facilities concept alternatives are presented in Table 13-3. Middle, upper, and lower cost ranges are provided to represent 80 percent, 100 percent, and 130 percent of the base opinion of probable construction costs developed at this level of concept design. A more detailed basis of cost and summary of cost items for each alternative is provided in Appendix G – Fish Passage Alternative Concept Design.

Table 13-3
Fish Facilities Construction Costs

FISH PASSAGE OPTION	LOWER BOUND COST ($ MILLION)	MIDDLE COST ($ MILLION)	UPPER BOUND COST ($ MILLION)
FRO – Fish Passage Conduits	Integral to dam construction.		
FRO and FRFA – Upstream Fish Passage: CHTR Facility	$13.8	$18.4	$27.6
FRFA – Upstream Fish Passage: Technical Fishway	$49.1	$65.4	$98.1
FRFA – Downstream Fish Passage: Fixed Multi-Port Collector	$80.0	$106.6	$159.9
FRFA – Downstream Fish Passage: Floating Surface Collector	$62.0	$82.6	$123.9

13.3.5 Transmission Lines Substation Equipment (if applicable)

Transmission lines are not required; however, electrical distribution lines are required. Costs for electrical distribution service line extension to the dam site is assumed to be covered under the design contingency applied to the direct dam costs.

13.3.6 Sales Tax

Sales tax has not been itemized in the cost estimate. The cost estimate reflects contractor bid unit prices, which are presumed to include a degree of sales tax. As design progresses and the estimate classification deepens, sales will be added to resource- and productivity-based estimates. The anticipated tax breakdown would be as follows:

Washington State Sales Tax	6.500%
Lewis County Sales Tax	1.300%
Pe Ell Sales Tax	0.000%
Total Sales Tax	7.800%

13.3.7 Contingencies

Contingency is applied to the entire direct dam construction costs. An expected design contingency of 20% has been applied. Although we see the design contingency as conservative, as the design progresses and the work breakdown expands, the design contingency should be reduced. A post-award construction contingency of 10% has been applied and should be maintained throughout the design development. Through the development of a risk analysis and risk register, the construction contingency could be re-evaluated and adjusted to reflect better understanding.

13.3.8 Engineering and Construction Management Assistance

For the FRO and FRFA alternatives, we applied an estimated engineering and construction management factor to the total project construction costs after applying the design and construction contingency. We estimated a design and site characterization contingency factor of 7% to 9%, and an engineering and construction management assistance contingency factor of 9% to 12%. Together with the permitting costs, these factors have been applied through a “non-contract” costs factor of 25%.

13.3.9 Permitting Costs

We assumed a proportional factor of between 3% and 6% would be applied to cover the anticipated cost of permitting coordination prior to and during construction. This range has been applied through the “noncontract” costs factor of 25%.

13.3.10 Operation and Maintenance Costs

Table 13-4 estimates operation and maintenance costs for both the dam options.

Table 13-4
FRO Roller Compacted Concrete (RCC) Dam Annual O&M Costs

COST AREA	COST CATEGORY	COST ITEM	COST BASIS	VALUE [1]	UNIT $	ANNUAL $[2]
Reservoir	Vegetation Management	Part Time Labor	FTE	1.5	$65,000	$97,500
Reservoir	Debris Handling	Part Time Labor	FTE	1	$65,000	$65,000
Reservoir	Debris Handling/ Disposal	Loaders/Trucks/ Operators	LS	1	$50,000	$50,000
Reservoir	Fish/ Environmental	Monitoring/ Reporting	LS	1	$30,000	$30,000
Dam	Operations	Dam Tender/Security	FTE	0.7	$85,000	$59,500
Dam	Administrative	Management	FTE	0.3	$120,000	$36,000
Dam	Administrative	Reporting	FTE	0.3	$90,000	$27,000
Dam	Administrative	Legal/Insurance	LS	1	$50,000	$50,000
Dam	Maintenance/ Repairs	Part Time Labor	FTE	0.5	$80,000	$40,000
Dam	Inspections	Safety Inspections	LS	1	$9,000	$9,000
Dam	Mechanical	Repair/Replace Fund	% Cap	0.4%	$21,952,000	$88,000
Dam	Structural	Repair Fund	% Cap	0.1%	$50,850,000	$51,000
		Rounded Subtotal				**$603,000**
Fish Passage	Operations	Operator/Monitor	FTE	0.01	$122,550	$2,000
Fish Passage	Biological	Monitoring/ Reporting	FTE	0.01	$129,000	$2,000
Fish Passage	Biological	Part Time Labor	FTE	0.02	$44,400	$1,000
Fish Passage	Maintenance	Part Time Labor	FTE	0.03	$129,000	$5,000
Fish Passage	Structural/ Mechanical	Repair/Replace Fund	LS	1	$3,000	$3,000
Fish Passage	Trap and Haul	Loaders/Trucks/ Maintenance	LS	1	$3,038	$4,000
Fish Passage	Electricity	General Service Loads/Pumping	kWh	38,600	$0.09	$4,000
Fish Passage	Biological	Monitoring & Evaluation	FTE	0.01	$129,000	$2,000
Fish Passage	Biological	Monitoring & Evaluation PT	FTE	0.01	$44,400	$1,000
Fish Passage	Science Costs	Lab Tests, etc.	LS	0	$37,000	$1,000
		Rounded Subtotal				**$25,000**

COST AREA	COST CATEGORY	COST ITEM	COST BASIS	VALUE [1]	UNIT $	ANNUAL $[2]
Rounded Total Annual Cost						**$628,000**

Notes:
1. Values rounded to nearest whole number (LS), 0.1 percent, 0.01 FTE, or 100 kilowatt hours.
2. Values rounded to nearest $1,000 US dollars for simplicity. Details provided elsewhere may vary.

13.3.11 Property Tax, Insurance, etc. (if applicable)

Property taxes and insurance costs were not considered in this analysis, as it is expected that the real estate required for the project would be held by the State of Washington and as such would not be subject to taxation.

14 CONSTRUCTION SCHEDULE

14.1 Construction Sequence

A conceptual sequence for construction is described in Table 14-1. As illustrated by the sequence, construction schedule could be significantly influenced by flow conditions, and there are numerous schedule dependencies that would influence the completion date for the project.

Table 14-1
Preliminary Construction Sequence

WORK BREAKDOWN	SEQUENCE CONSIDERATIONS
TUNNEL CONSTRUCTION & INITIAL DIVERSION	
• Isolate portal areas from river flow	Low-flow limitation
• Slowly dewater and "fish" portal areas	Manually and safely remove fish from the areas to be dewatered and return them to the active river system
• Construct tunnel portals	Low-flow limitation
• Build the diversion tunnel by advancing from the downstream side portal to the upstream side portal	Low-flow limitation during completion of tunnel at upstream end.
• Build temporary berm to divert low flows through tunnel	River diverted – low flow
CONSTRUCT COFFERDAMS	
• Prepare flow diversion cofferdam foundation	Low-flow limitation
• Construct cofferdams, RCC upstream	Diversion ready for capacity flows; probable low-flow limitation; start flood risk
• Slowly dewater and "fish" areas behind cofferdams	Manually and safely remove fish from the areas to be dewatered and return them to the active river system
FOUNDATION PREPARATION	
• Excavate abutments & bottom	Emphasize right side to allow structure starts
• Prepare river outlet foundation	Start primary flood risk. Consider grout curtain continuity, avoid undercut
• Perform curtain grout	Include consolidation grouting, if required
• Foundation treatments & dental concrete	Prudent to allow some schedule contingency
LOWER-LEVEL HYDRAULIC STRUCTURES	
• Construct lower-level river outlet works (ROW)	Precedes RCC, through encasement. Includes flood control conduit/passage, ROW piping
• Construct initial energy dissipation, stilling basin	
• Construct lower-level fish passage	
RCC AND DAM	
• Place RCC – bottom to top of ROW encasement	Preceded by quarry development, initial aggregate processing, plant and delivery setup, trial section

WORK BREAKDOWN	SEQUENCE CONSIDERATIONS
• Place RCC – top of ROW to spillway break, include flip bucket mass block	RCC concurrent activities; aggregate processing, instrumentation, gallery construction, face and drain system construction, abutment preparation, ongoing ROW, stilling basin, fish passage construction. Primary flood risk complete
• Place RCC – right wing	
• Place RCC – right side to crest	
• Place RCC – left side to crest	
RIVER OUTLET WORKS	
• Complete intake, gate & service shaft, control structures	Gate & service shaft likely to precede RCC, but formed void could be considered
• Install flood regulating gate	
• Install river outlet gates	
• Install river outlet trash racks & metals	
• Complete ROW & flood regulating mechanical & controls	
• Complete control structure – electrical, mechanical & building trades	ROW ready for re-divert & tunnel plug
SPILLWAY	
• Complete stilling basin & energy dissipation structures	
• Complete plunge pool preparation	Spillway ready for re-divert & tunnel plug
• Construct spillway training walls	
• Construct spillway chute and flip	
• Construct spillway ogee	
• Construct spillway piers & bridge	
FISH PASSAGE	
• Complete downstream fish passage and mechanical	Fish passage ready for re-divert and tunnel plug
• Complete upstream fish passage	
• Complete fish passage conveyance	
• Complete fish passage mechanical & controls, building trades	
COMMISSIONING & RESTORATION	
• Complete electrical & mechanical commissioning	
• Final ROW access & grading construction	
• Breech cofferdams & re-divert river	Final flood risk complete
• Construct plug tunnel	
• Dam backfill, downstream and abutment grading	
• Complete quarry, access, & staging restoration	Preceded by reservoir clearing
• Project schedule contingency	

14.2 Contract Approach and Duration

The project duration, and ultimately the completed project delivery date, will depend upon permitting, design schedule, contract delivery approach, and contract execution. Concept-level planning should consider at least 8 years for engineering and construction. If final design were to begin in the beginning of 2017, this would lead to completion at the end of 2024. A shorter time frame could be realized if an accelerated project delivery became a driving project goal.

As discussed in the previous study (HDR, 2014), there could be advantages in building the project using two separate construction contracts. One contract could be used for construction of the river diversion, while the other contract could be used for construction of the main dam and associated fisheries facilities. The creation of two separate contracts would influence assignment of responsibility for construction and schedule risk, and could influence project cost.

Advantages of establishing separate contracts could include:

- By creating separate contracts, the owner could potentially maintain greater control over schedule risks by having greater control over diversion capacity and work sequence.
- Separate contracts could allow an earlier start to construction, which could allow early completion of diversion work and a reduced project duration.
- Separate contracts could make it possible to advance the project without having funding in place for the entire project.
- RCC construction and tunneling are specialized work. Separation of work into multiple contracts would allow specialists to competitively bid on separate portions of work, potentially resulting in lower bid costs.
- Depending on the timing of the separate contracts, use of a separate contract for tunnel diversion work could help isolate the construction of the dam from the schedule risk associated with the diversion contract.

Advantages of having a single contract could include:

- One contract assigns all responsibility of risk to one contractor, which could reduce the complexity of determining liability in the event there are requests for change orders or schedule modifications during construction.
- One contract provides the most flexibility to the dam contractor in determining how to conduct and schedule work. A contractor’s tolerance for risk and approaches for mitigating those risks could lead to lower project costs.
- One contract could encourage innovation.
- A single contract would likely result in a dam contractor assuming liability and costs associated with managing the work of tunneling subcontractor. Under the dual contract configuration, the

State would assume the costs of managing two contracts rather than one. Likewise, the State would assume the risk of overlapping contracts or contract interference.

- Under a single contract, the quality of the diversion work is the responsibility of the same party that is at risk in the event of the failure of the diversion, which could result in higher quality work and lower failure risk.

The selection of contracting approach would influence the project schedule. Table 14-2 provides an estimate of project timelines for each contracting approach for each dam option. As shown in the table, it is anticipated that the duration for the contract would be reduced by using separate contracts. A thorough review of contract structure will need to be considered during design. No consideration is shown for starting the first of a two-contract approach before final design is complete.

Table 14-2
Comparison of Schedule for Alternatives

	FRFA		FRO	
	1 CONTRACT	**2 CONTRACTS**	**1 CONTRACT**	**2 CONTRACTS**
Final Design – (after completing preliminary design and site characterization)	2 - 2.5 yr	2 - 2.5 yr	1.5 - 2 yr	1.5 - 2 yr
Additional permitting allowance	1.5 yr	1.5 yr	1 yr	1 yr
Procurement – Phase 1 bid/award	6 months	6 months	4 - 6 months	4 - 6 months
Phase 1 Construction		1 - 1.5 yr		1 - 1.5 yr
Procurement – Phase 2 bid/award		6 months[1]		4 - 6 months[1]
Phase 2 Construction		3 - 3.5 yr		2 -3 yr
Single Contract Construction	3 - 4 yr		2.5 - 3.5 yr	
Total	**7 - 8.5 yr**	**8 - 10 yr**	**5.5 - 7 yr**	**6.5 - 8 yr**

Note:
1. Concurrent with Phase 1 construction

15 ALTERNATIVES COMPARISON AND RECOMMENDATIONS

15.1 Alternatives Comparison

The evaluation performed in support of this report did not identify any fatal flaws associated with either the RFO or FRFA dam configurations. A summary of the main features of the FRO and FRFA dam configurations is provided in Table 15-1. The selection of the preferred alternative will need to be based on considerations within this report, selected fisheries objectives, and identified environmental objectives and permitting constraints. Although not developed as part of this evaluation, a hybrid alternative of FRFA dam height with FRO hydraulic structures and operations could be developed to allow FRO operation with the option of changing to FRFA operation in the future.

Table 15-1
Summary Comparison of Alternatives

COMBINED ALTERNATIVE	FRO	FRFA
Purpose	Flood Retention Only	Flood Retention and Flow Augmentation
Dam Type	Gravity - RCC	Gravity - RCC
Dam Structural Height (feet)	254	313
Water Storage Elevation (Spillway Crest Elevation, feet)	628	687
Emergency Spillway Type	Dam Crest	Dam crest
Reservoir Storage Volume (1,000 AF)	65	130
Recommended Upstream Fish Passage	Flow through channels and CHTR facility	CHTR
Recommended Downstream Fish Passage	Flow through channels	Floating Surface Collector
Construction Period (years)	2.5 – 3.5	3 – 4
Estimated Dam and Fish Passage Project Costs (2016 $Million)	$264,000,000	$471,000,000
Estimated Annual O&M Costs ($2016 $1,000)	$628,000	$2,178,000

Notes: AF = acre-foot, CHTR = collect, handle, transfer, and release, RCC = roller compacted concrete, NA = Not applicable O&M = operations and maintenance

15.2 Recommendations for Further Study

Upon selection of the preferred dam configuration and a decision by the State of Washington to advance the project, the project would be advanced to preliminary design. The preliminary design would be used for completion of environmental evaluation, documentation, and permitting. As documented below, the design team recommends the completion of additional evaluation in support of preliminary design (during the third biennium study period of July 2017 through June 2019).

15.2.1 Phase 3 Site Characterization

Site characterization studies are phased such that each subsequent phase builds on the information obtained from the previous phases. The Phase 1 and 2 explorations (completed during 2014-15 and 2015-16) improved the understanding of the geologic conditions at the proposed dam site to allow advancement of conceptual engineering design. The Phase 1 and 2 explorations also identified areas of additional geologic uncertainty and identified the need for further study in select areas. In addition, as the design of fish facilities and hydraulic structures has advanced, changes to the layout and designs justify the need for other explorations associated with updated layouts and configurations of the dam, spillway, outlet works, and fish passage facilities.

A third phase of site characterization should be conducted that would allow:

- Refinement of the configuration of the dam and associated hydraulic and fish passage structures
- Refinement of foundation excavation and treatment approach
- Development of a detailed landslide mitigation plan
- Reduced uncertainty on the availability on suitable material within the potential RCC aggregate quarry sites
- Development of a Class 3 cost estimate as defined by the Association for the Advancement of Cost Engineering (AACE No. 69R-12). This estimate would be suitable for budget authorization or control.

Phase 3 site characterization would include the following site investigation coordination activities:

- Develop scope, budget, and schedule.
- Obtain permits and clearances from government agencies and landowners.
- Prepare access roads for explorations.
- Coordinate project team meetings to include weekly conference calls, site visits, and one data review meeting.
- Coordinate meetings with DNR/Ecology for the Phase 3 investigations, as well as future investigations.
- Obtain final quotes for work by subcontractors.

A Phase 3 site investigation program would target specific areas and obtain specific information as follows:

- Additional drill holes along the RCC dam alignment to provide better interpretation of subsurface conditions for foundation engineering
- Additional drill holes along the saddle dam/embankment dam axis for the FRFA alternative to better define the transition between the RCC dam and the embankment dam and the potential need for a seepage cutoff wall – both of which are basically controlled by the geology and foundation conditions
- Additional drill holes along potential fish ladder alignments to define anticipated foundation conditions and excavation requirements
- Additional exploration of landslides in the area of the reservoir to evaluate the need and cost for stabilization and mitigation
- Further characterization of the extent and layout of the potential quarries to allow better definition of material sources for RCC aggregate that will be used to refine construction cost estimates. Material quality, quantity, and location have a significant impact on construction costs and overall project costs. Materials from these additional borings will be used to generate a sufficient quantity of RCC aggregate in order to complete a preliminary-design-level laboratory RCC mix design study
- Definition of geotechnical and foundation conditions to support fish facility design

The Phase 3 site investigation would include necessary coordination and execution of the following:

- Assistance with subcontracting and overseeing subcontractors including laboratory testing and pioneer road building
  - Assistance with the preparation of subcontract agreements including the development of scope of work requirements and specifications
  - Flagging of all Phase 3 boring locations and survey of the future investigations boring locations
  - Flagging and then survey (by others) of location of seismic refraction lines
- Coordination and oversight of site access improvements
- Field oversight of subcontractor operations and field logging and data collection
- Field oversight and logging for eight dam foundation borings ranging from 200-400 feet deep to:
  - Obtain soil and rock core samples for field logging and laboratory testing
  - Coordinate downhole geophysical testing (optical and acoustic televiewer and downhole P-S wave velocities) to assess discontinuity orientation and spacing and shear wave velocity of rock
  - Provide oversight and data recording for downhole water pressure tests to assess rock mass hydraulic conductivity

  - Oversee and document installation of vibrating wire piezometers in all dam foundation borings
  - Perform geologic interpretation of dam site area
- Field oversight and logging for 11 landslide evaluation borings to depths of 50 to 150 feet
  - Obtain soil samples for field logging and laboratory testing
  - Core into bedrock a sufficient depth to verify rock type
  - Oversee and document installation of vibrating wire piezometers and inclinometers in all landslide evaluation borings
  - Perform initial inclinometer survey and evaluation of survey data
  - Perform stability analyses for purposes of formulating conceptual landslide repairs
- Field oversight of drilling subcontractor and logging of five quarry investigation borings to depths of 100 to 200 feet
  - Obtain soil and rock core samples for field logging and laboratory testing to assess suitability and potential supply of rock for RCC aggregate sources
  - Oversee and document installation of vibrating wire piezometers in the five quarry investigation borings
- Field coordination of subcontractor for tomography (seismic refraction) lines to assess top of rock, compression and shear wave velocity, and rippability
  - Three seismic refraction lines along portions of the proposed dam alignment
  - Five seismic refraction lines across landslides identified during previous investigations
  - Four seismic refraction lines across potential RCC aggregate quarry locations
- Laboratory testing to include:
  - Soil index testing (Atterberg, moisture/density, gradation)
  - Dam foundation and tunnel rock evaluation (unconfined compressive strength, point load testing, direct shear)
  - Quarry rock quality testing (alkali-silica reactivity [ASR], LA abrasion, specific gravity, absorption)
  - Slake durability
  - Petrographic analysis
- Perform quality assurance, quality control of boring logs, hydraulic conductivity tests, geophysical explorations, and laboratory analyses.

Deliverables for Phase 3 Site Characterization would include:

- Technical memorandum including narrative discussing field activities, issues identified, and modifications to the program
- Detailed boring logs
- Summary tables of downhole water testing
- Hydraulic conductivity test data

- Inclinometer installation details
- Vibrating wire installation details
- Presentation of laboratory data
- Landslide characterization
- Conceptual recommendations for landslide remediation

15.2.2 Dam Safety Hydraulic Modeling

Additional computational fluid dynamics (CFD) modeling and physical modeling is recommended to advance the design. CFD modeling can evaluate potentially complex hydraulic interactions between river flows and the proposed structures. This modeling would assist in optimizing the performance and costs of the proposed structures. Physical modeling would help confirm the accuracy of modeling assumptions, allow relatively low-cost iterations to the design, and provide greater certainty on the performance of completed structures.

The design of outlet works and spillway facilities will vary depending on anticipated operational constraints or selected flood control objectives. In the first and second phases of the design, the anticipated frequency and pattern of operation of the project for flood damage reduction was assumed. The initial assumptions will require additional evaluation to determine viability and practicality of the desired operation. Although the flood control objectives should be similar for both the FRO and FRFA alternative concepts, the objective of flow augmentation for the FRFA concept will require additional evaluation. Adjustments may be required to determine hydraulic discharge capacity and the configuration necessary to accommodate the various dam operation objectives. The operational modeling needed to evaluate and define these features and refine the hydraulic capacity and operational capabilities of the outlet works would be accomplished in a collaborative manner, with the hydraulic design integrated into the operational modeling conducted to support the evaluation.

Specific study elements that will be pursued to help define the operational and dam safety capabilities of the proposed dam configuration include the following tasks:

- Refine operational rule curve and anticipated flood control operational model for the FRO alternative and assess implications for hydraulic capacity of dam outlet works and spillway to address the full range of potential flood event management.
- Refine operational rule curve and anticipate flood control operational model for the FRFA alternative and assess implications for hydraulic capacity of dam outlet works and spillway to address the full range of potential flood event management. This activity will also determine adjustments or refinements to outlet works design to accommodate the combined operation of the facility over a typical water year.

Physical scale hydraulic modeling will be necessary to refine all hydraulic features of the dam configuration of both the FRO and FRFA dam options. The physical model will permit the team to evaluate and test the capacity and hydraulic design of the dam outlet works and spillway. This will be necessary to evaluate and address energy dissipation, hydraulic capacity, sediment transport, and fish passage, and to determine specific hydraulic design data that will inform the final design of all hydraulic features. The physical scale model will address dam safety issues associated with flood operations and maximum hydraulic capacity available with the proposed dam and associated hydraulic facilities. The physical model will be used to finalize assumptions relative to sedimentation and hydraulic performance of the lower outlet conduits and stilling basin. These results will also be important to establish final estimates of fish passage performance. Specific items to be addressed in the physical scale model are as follows:

- Hydraulic capacity of spillway, crest, and chute, for both the FRO and the FRFA configurations
- Sediment transport and hydraulics of upstream, downstream, and flood control/fish passage sluice conduits for the Flood Regulation Only dam configuration
- Overall hydraulic reservoir approach conditions to help define flood regulation outlet intake configuration and ensure efficiency, determine the optimal location of the floating surface collector, and determine the configuration of the guidance and lead nets for the floating surface collector or multi-port collector alternatives by optimizing reservoir circulation patterns
- Debris impacts on trashrack design and associated impacts to flood control outlet design
- Spillway passage of debris and approach conditions
- Energy dissipation of spillway discharge structures and flood regulation outlet sluice structures, including downstream conditions.
- Coordinate the design of the stilling basin and fish ladder entrance to optimize the performance of each structure
- Fish passage facility design to ensure adequate approach conditions and estimate collection efficiency of fish ladders, barriers, and entrances downstream of the dam

15.2.3 Additional Considerations

The following additional items would need to be evaluated in support of preliminary design:

- Construction Approach
  - Development of a construction schedule
  - Contract delivery options
  - Construction risk assessment
  - Development of risk mitigation strategies
- Design Criteria

  - Potential effect of climate change on the size and configuration of the FRO and FRFA alternatives
- Dam Design
  - Refinement of the use of “risk-based” design criteria for the RCC dam that consider the seismic hazards of the region and construction costs
- Fish Passage
  - Depending on the fish passage alternative selected, design/modeling of facilities to optimize existing conditions and hydraulic characteristics both upstream and downstream of the dam
- Cost Estimates
  - Material availability for dam construction
  - Refinement of funding-level opinions of cost for dam and fish passage costs including appropriate final design and construction contingencies
- Operations and Maintenance
  - Further development of design and operation requirements for floods (climate change), debris management, and landslide prevention/mitigation
  - Considering the results of operational and water quality studies, refinement of storage use to best improve water quality downstream of the dam for fish and the effect of flow augmentation downstream

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Appendix A
Maps and Drawings

Chehalis Basin Strategy

Appendix A

Maps and Drawings

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

Prepared for: State of Washington Recreation and Conservation Office and Chehalis Basin Work Group

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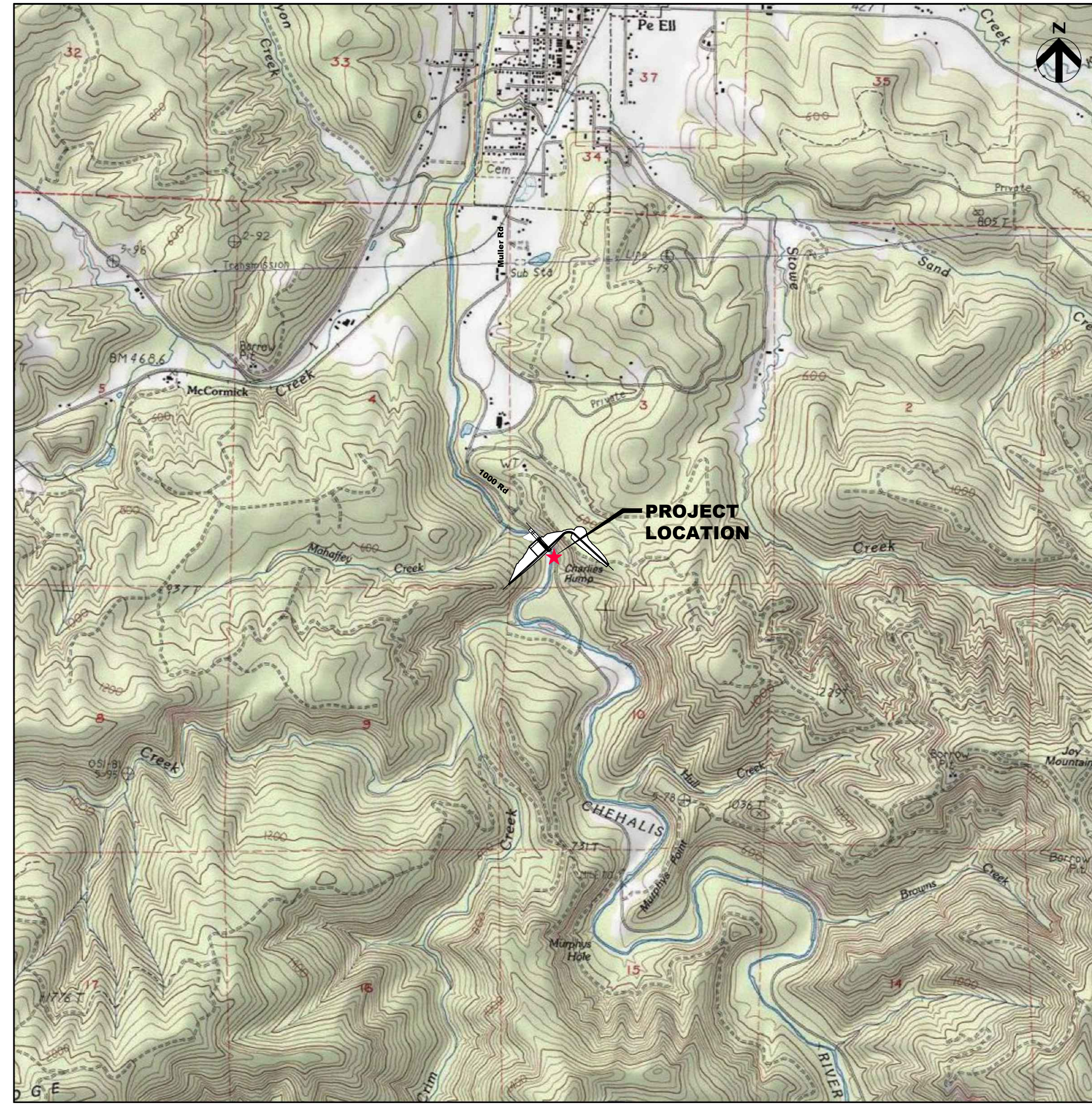
CHEHALIS BASIN STRATEGY
COMBINED DAM AND FISH PASSAGE
CONCEPTUAL DESIGN

WASHINGTON STATE
SCALE: 1" = 100 mi

PROJECT VICINITY
SCALE: 1" = 8 mi



PROJECT LOCATION
SCALE: 1" = 0.5 mi

HDR	VICINITY AND LOCATION MAPS	DATE JULY 2017
	CHEHALIS BASIN DAM	FIGURE G-1

INDEX OF DRAWINGS

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FLOOD RETENTION ONLY (FRO)

CIVIL

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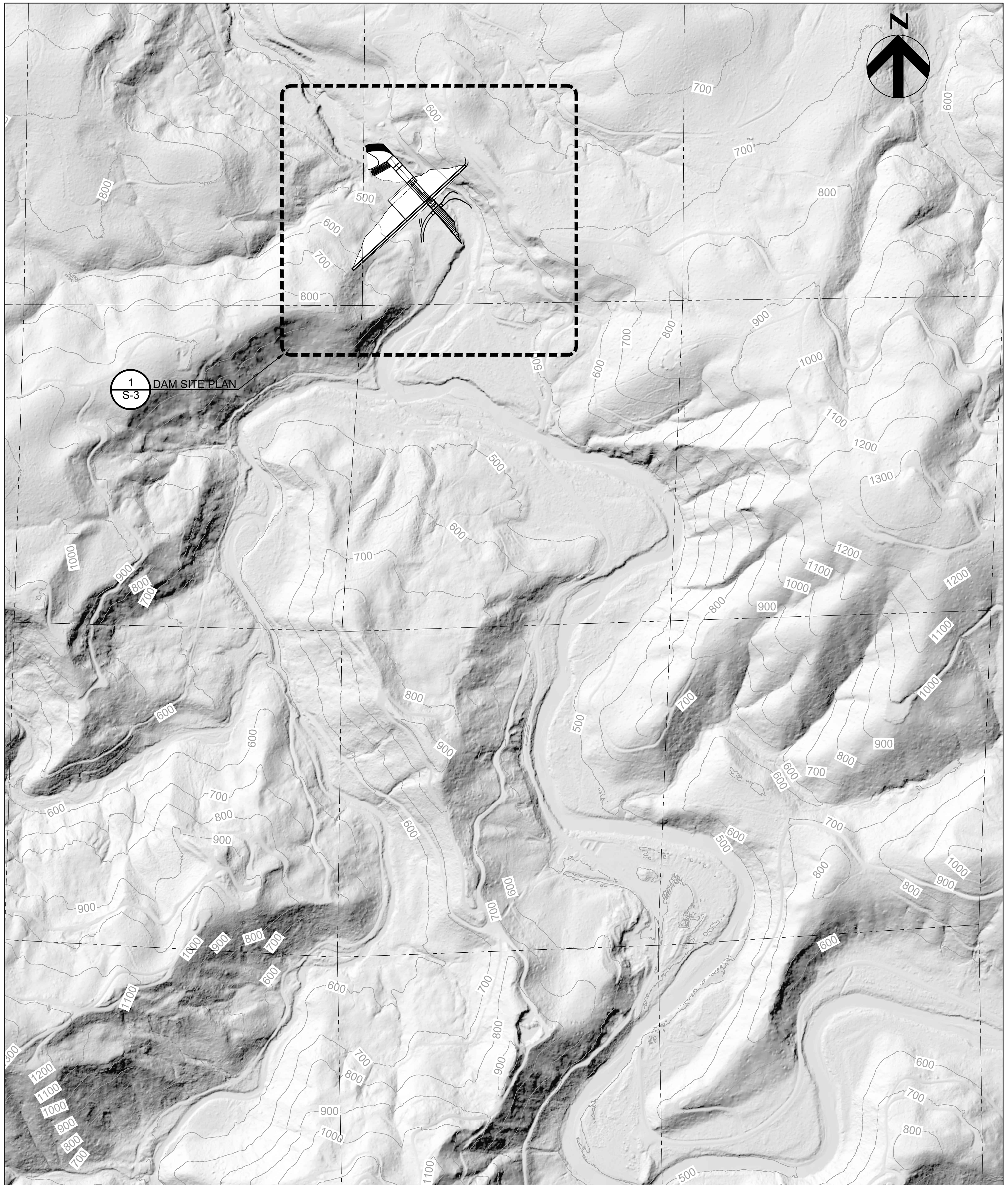


PLAN

(FRO ALTERNATIVE SHOWN)

0 250 500 1000

SCALE IN FEET

General Notes:

A. Coordinate System: NAD 1983 State Plane (Washington South) Feet
Geographic Coordinate System: GCS North American 1983
Projection: Lambert Conformal Conic
Vertical Datum: NGVD 88

B. Hillshade and contours derived from LiDAR data

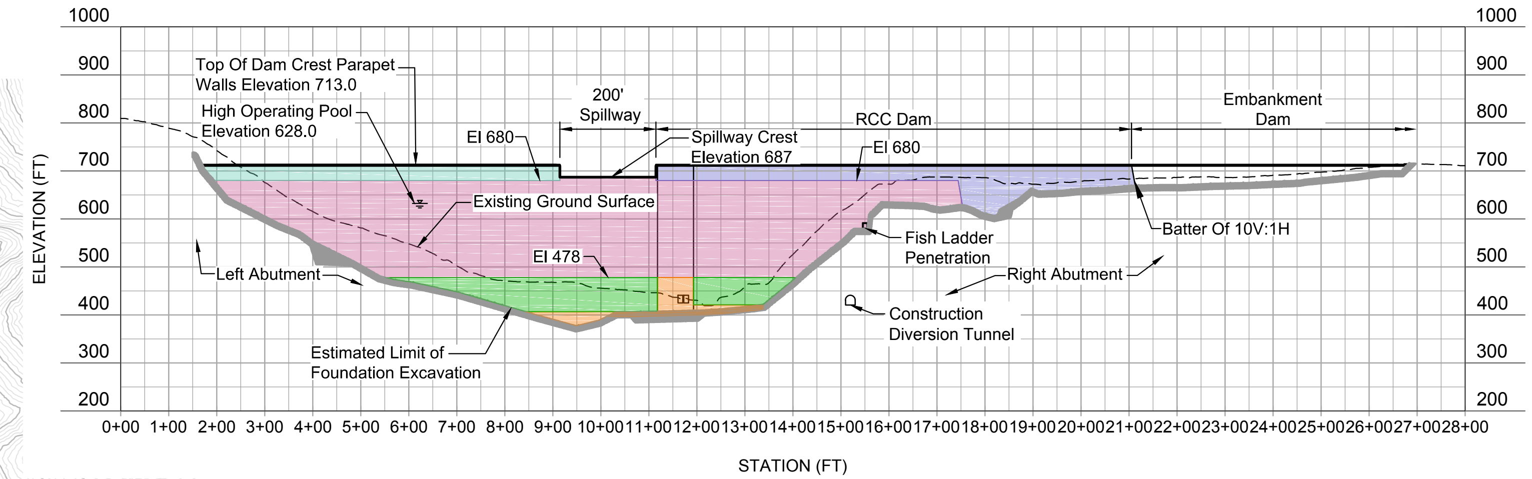
HDR

EXISTING SITE PLAN AND GENERAL NOTES

CHEHALIS BASIN DAM

DATE: JULY 2017

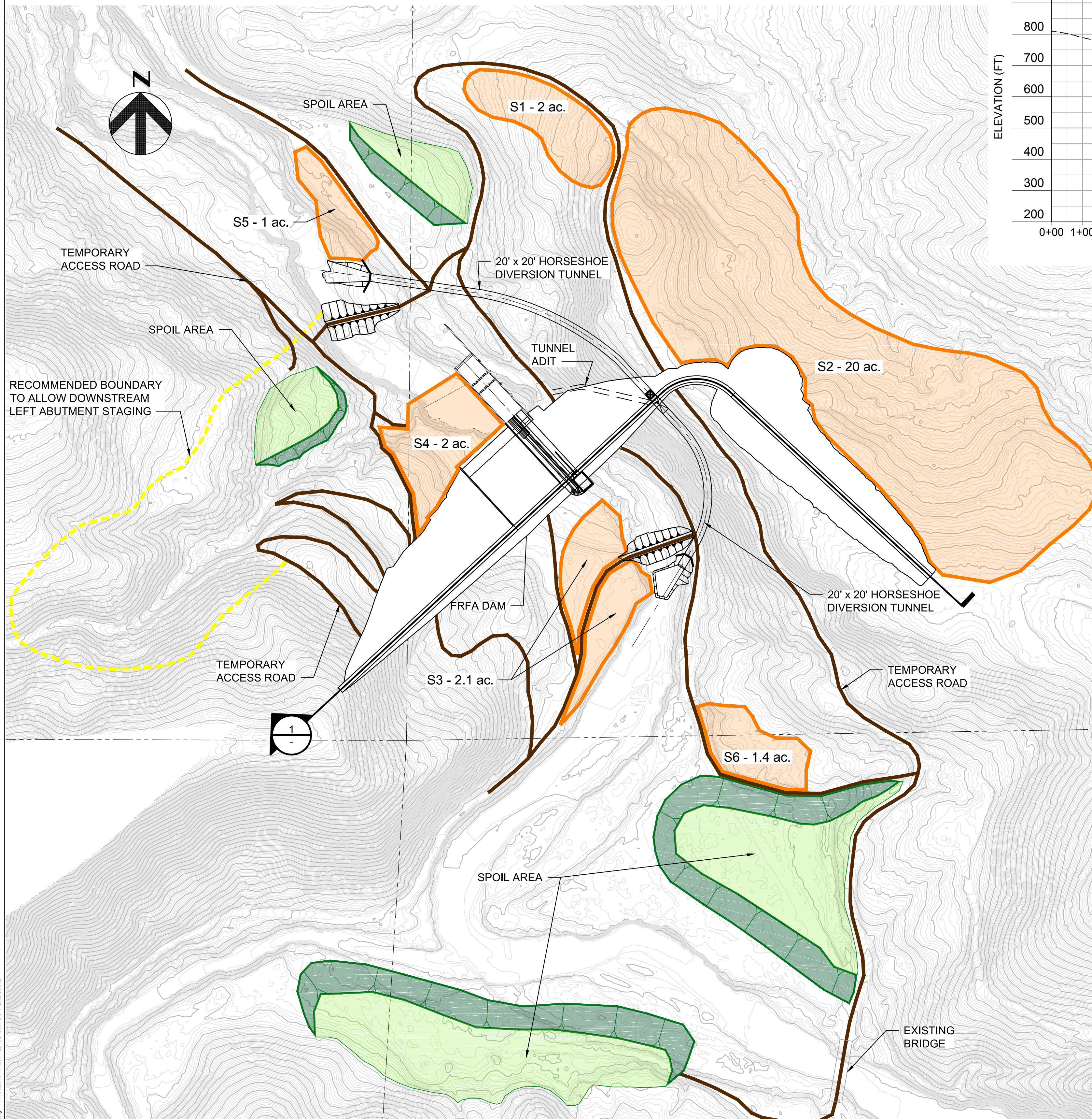
FIGURE: G-3



PROFILE LEGEND:

- Phase 1 : Complete foundation leveling concrete, intake and river outlet works encasement that passes through the dam.
- Phase 2 : Place RCC to the top of outlet works encasement .
- Phase 3 : Place RCC to the spillway crest.
- Phase 4 : Place RCC to the dam crest, right of spillway and filling the wing.
- Phase 5 : Place RCC to the crest, left of spillway.

POSSIBLE PHASING PROFILE 1/-



PLAN
(FRFA ALTERNATIVE SHOWN)

0 100 200 400
SCALE IN FEET

PLAN LEGEND:

- S1 - OFFICES, RECEIVING, SECURITY.
- S2 - 1) RCC PLANT, CONVENTIONAL CONCRETE PLANT, COOLING SYSTEMS, CEMENT & FLY ASH STORAGE, SUPPLEMENTARY AGGREGATE STOCKPILES.
  2) FABRICATION & MAINTENANCE.
  3) GATES & MECHANICAL STAGING.
- S3 - STRUCTURE & DAM STAGING
- S4 - (S2 - S3) SUPPLEMENTAL STRUCTURE & DAM STAGING.
- S5 - DOWNSTREAM PORTAL STAGING.
- S6 - UPSTREAM PORTAL STAGING.
- SPOIL AREA
- TEMPORARY ACCESS ROAD

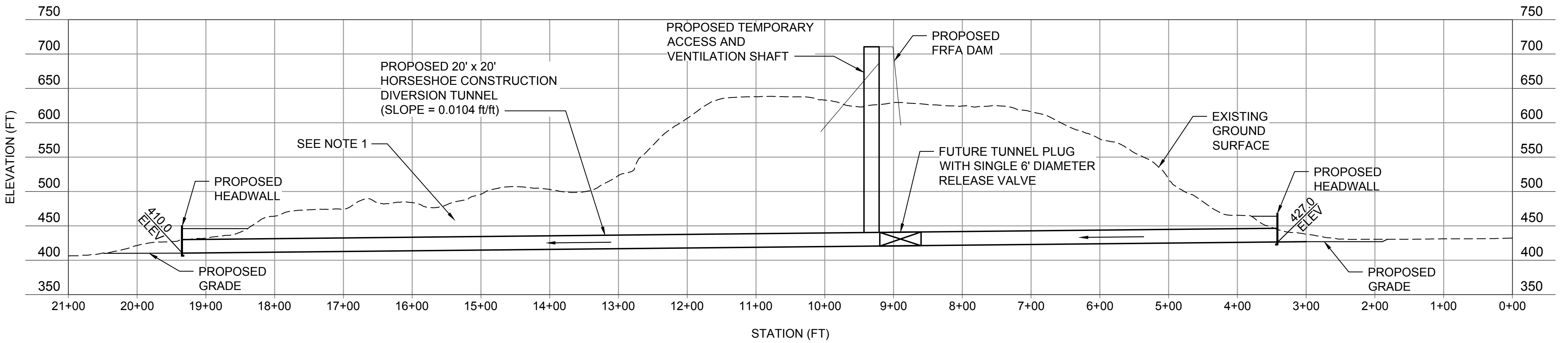
HDR

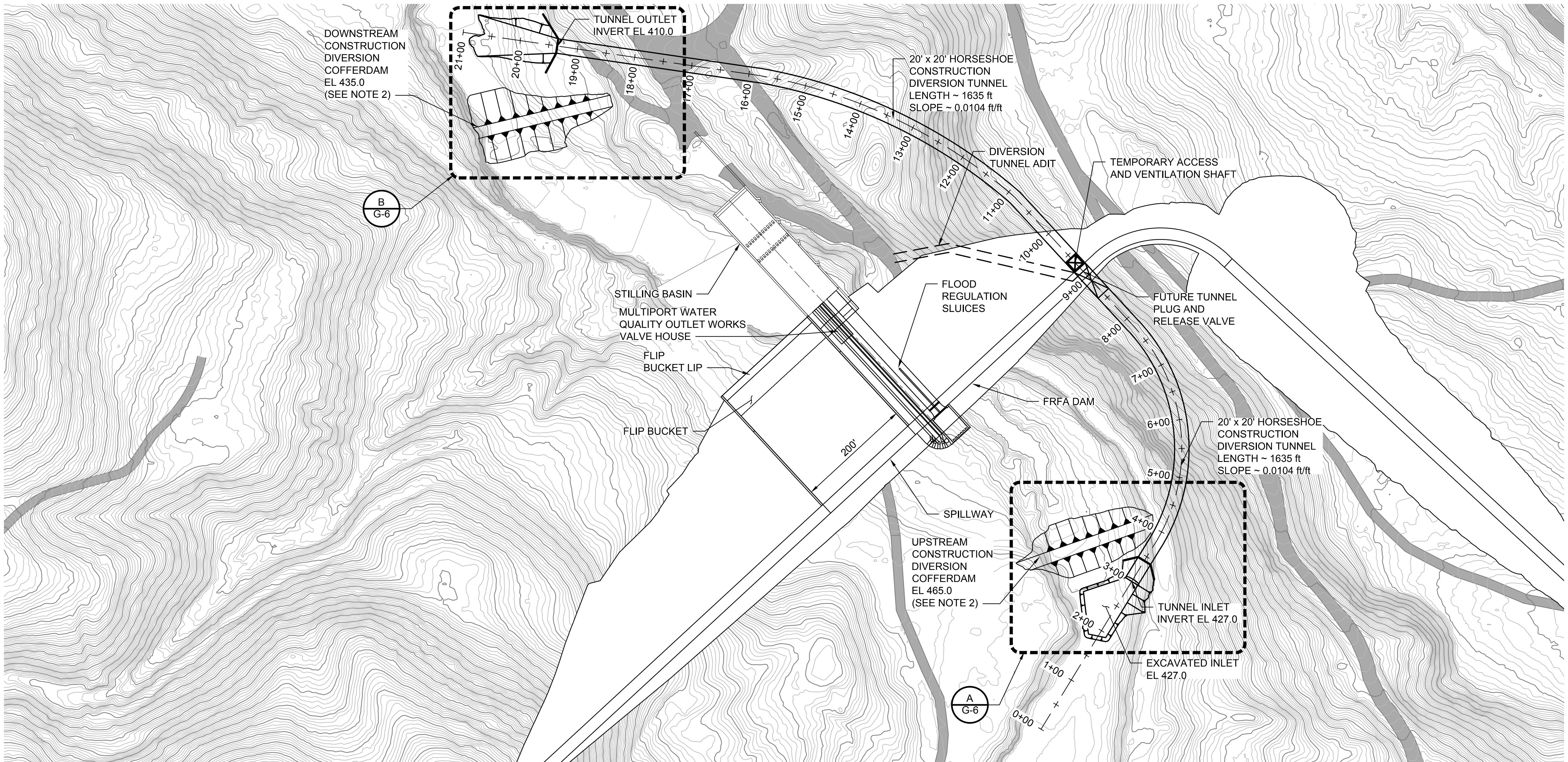
SITE PLAN, CONSTRUCTION PHASING AND WORK AREAS

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
G-4





NOTES:

1. DEPENDING ON THE GEOLOGY OF THE AREA, LAND SLIDE TREATMENT AND ADDITIONAL STRUCTURAL PROTECTION ACTIVITIES MAY BE REQUIRED WHICH WILL BE EVALUATED DURING THE PRELIMINARY DESIGN PHASE.

2. COFFERDAM CONFIGURATION IS FOR ROCKFILL CONFIGURATIION. ALTERNATE RCC CONFIGURATION TO BE EVALUATED DURING THE PRELIMINARY DESIGN PHASE.

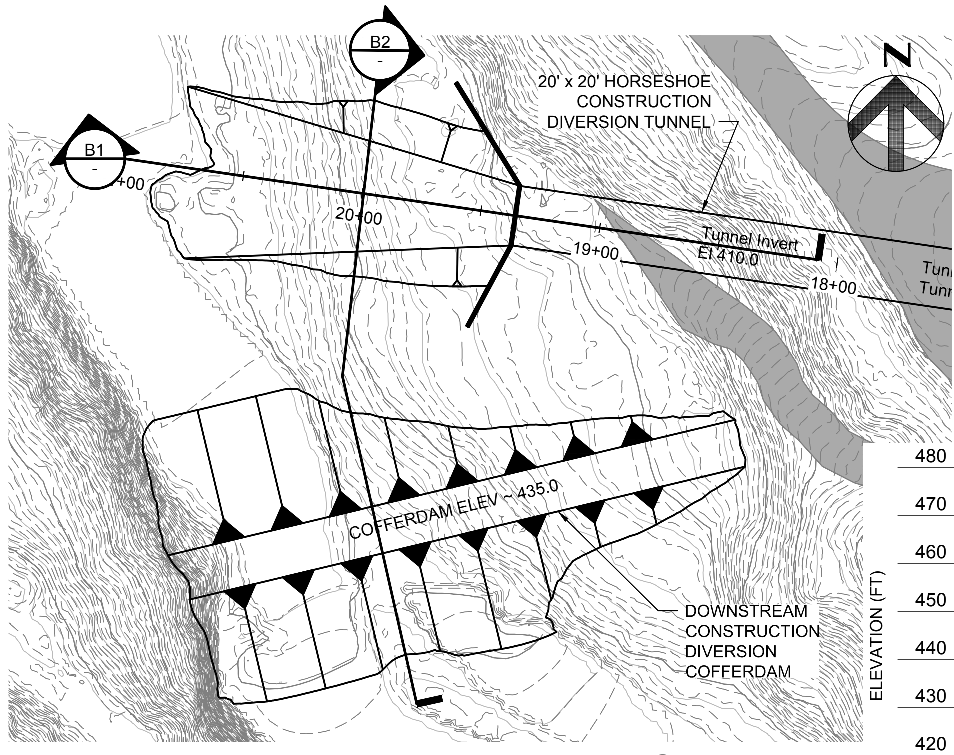
0 50 100 200
SCALE IN FEET

HDR

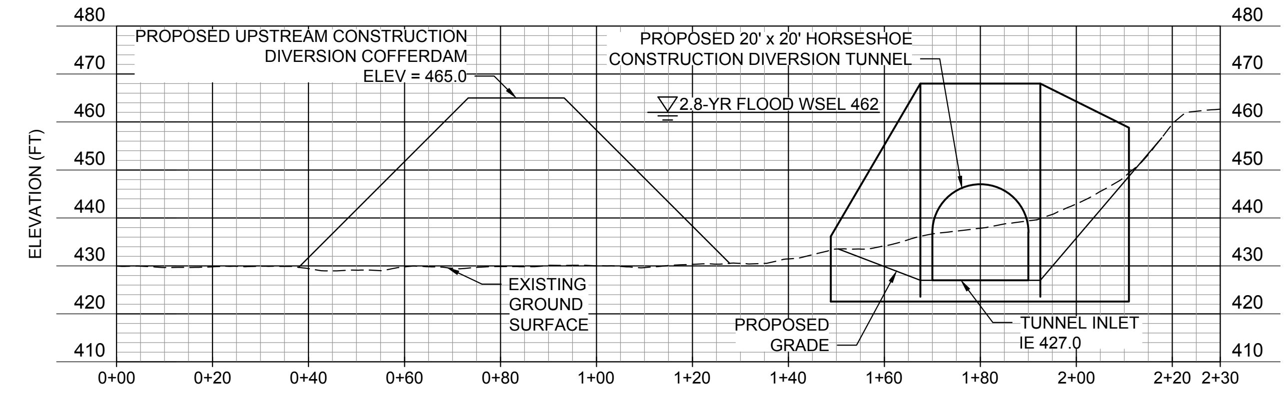
CONSTRUCTION DIVERSION PLAN

CHEHALIS BASIN DAM

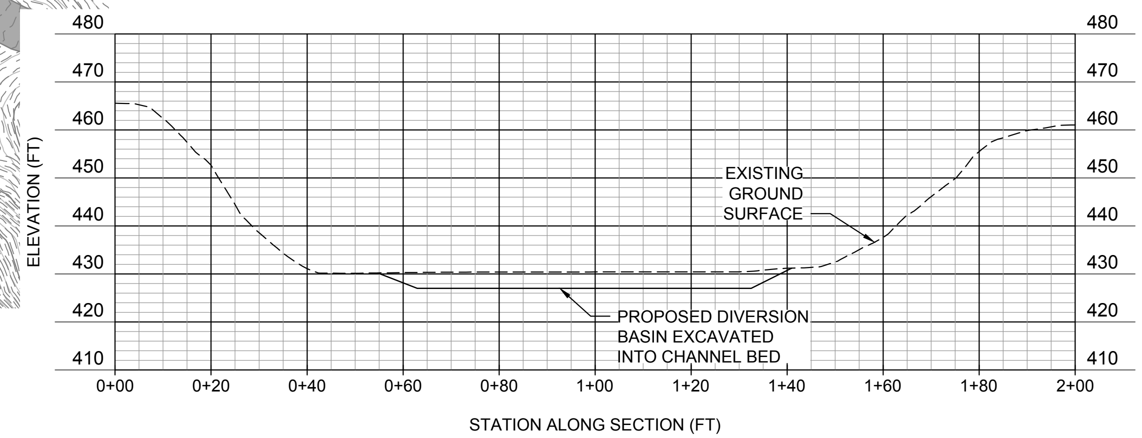
DATE	JULY 2017
FIGURE	G-5



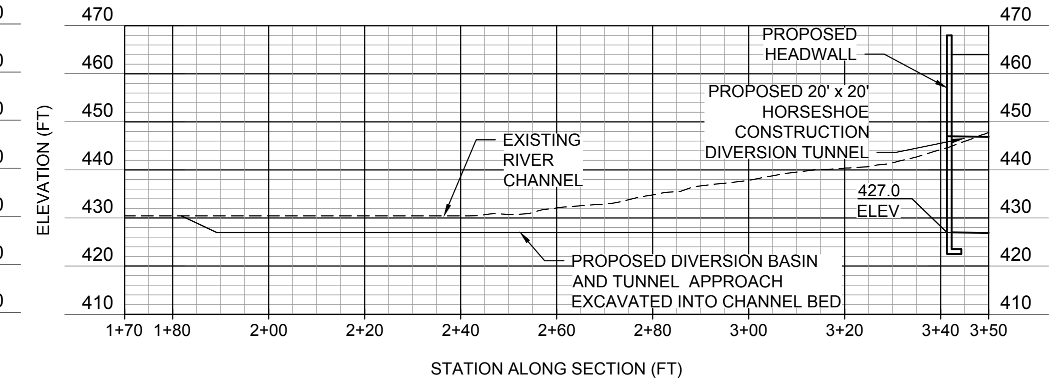
OUTLET PLAN DETAIL B G-5



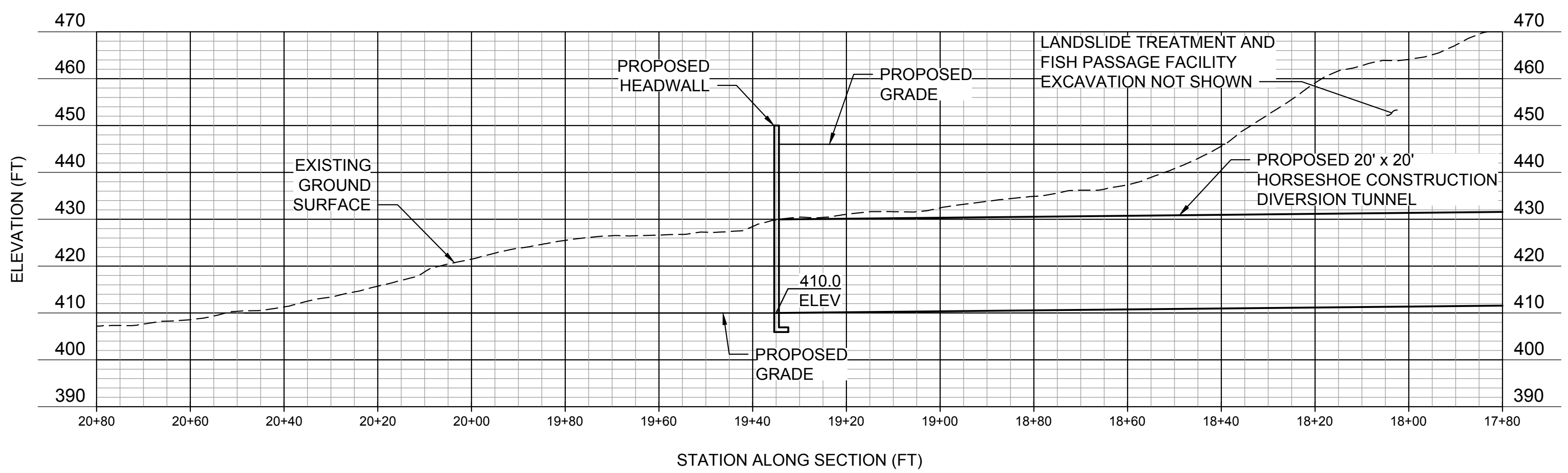
SECTION A-2



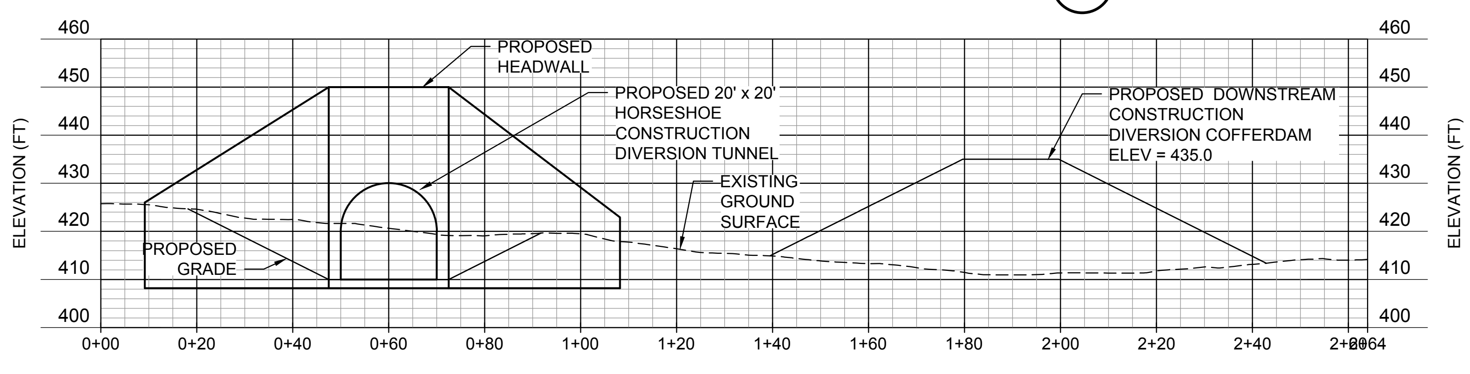
SECTION A-3



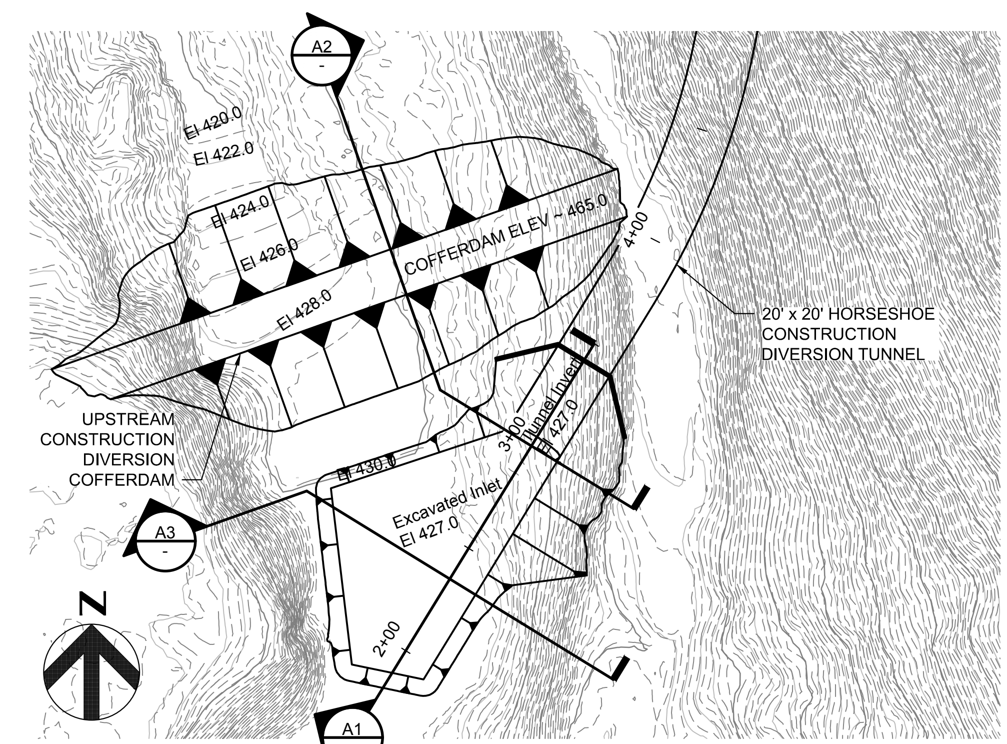
SECTION A-1



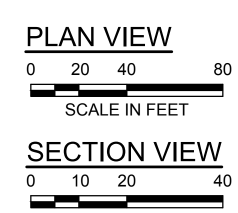
SECTION B-1



SECTION B-2



INLET PLAN DETAIL A G-5



HDR

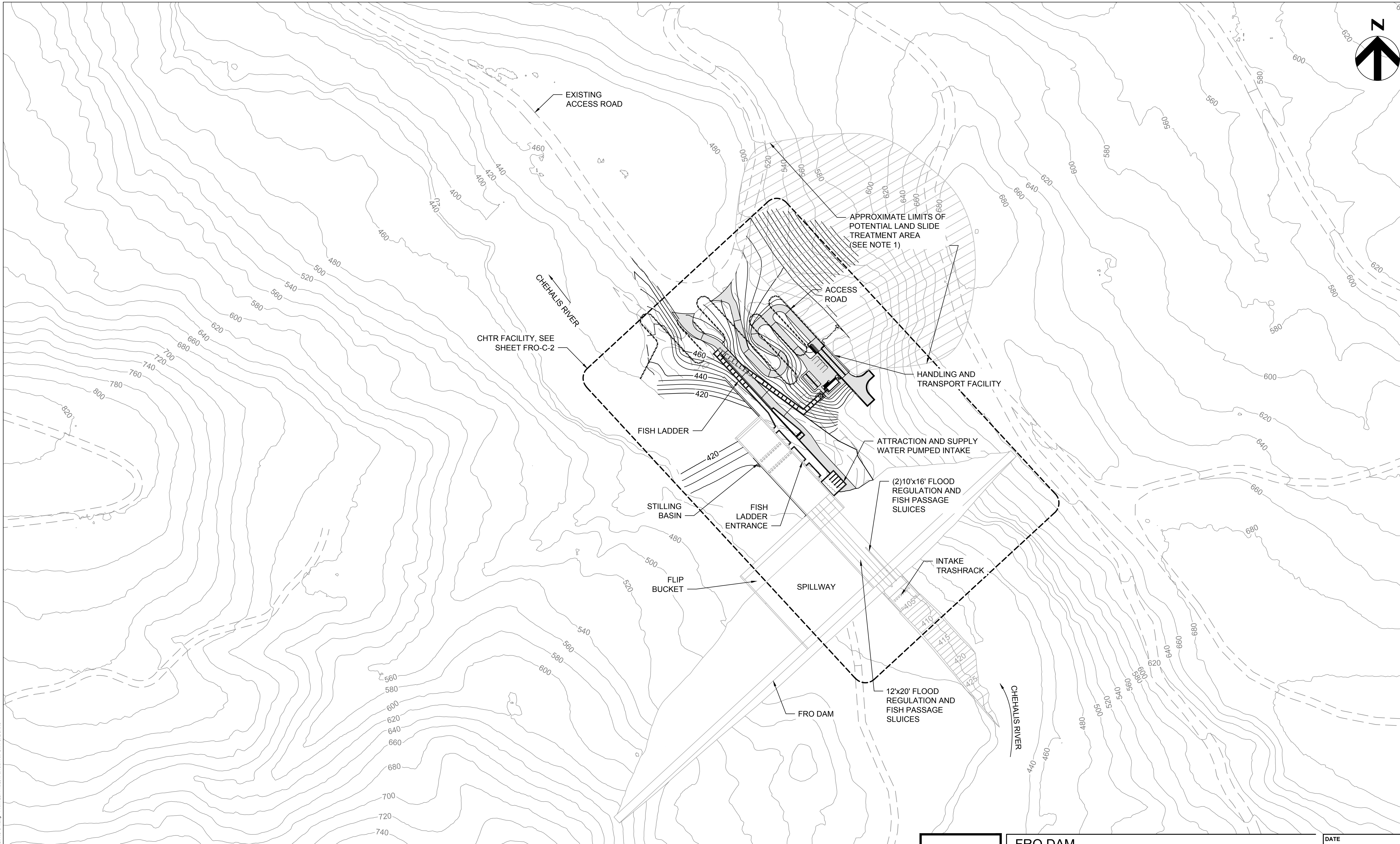
CONSTRUCTION DIVERSION DETAILS

CHEHALIS BASIN DAM

DATE JULY 2017

FIGURE G-6

HDR C:\pwworking\east\d1810462\G-6.dwg PRINTED: 7/10/2017 8:58:10 AM BY: PDXBORRO



NOTES:

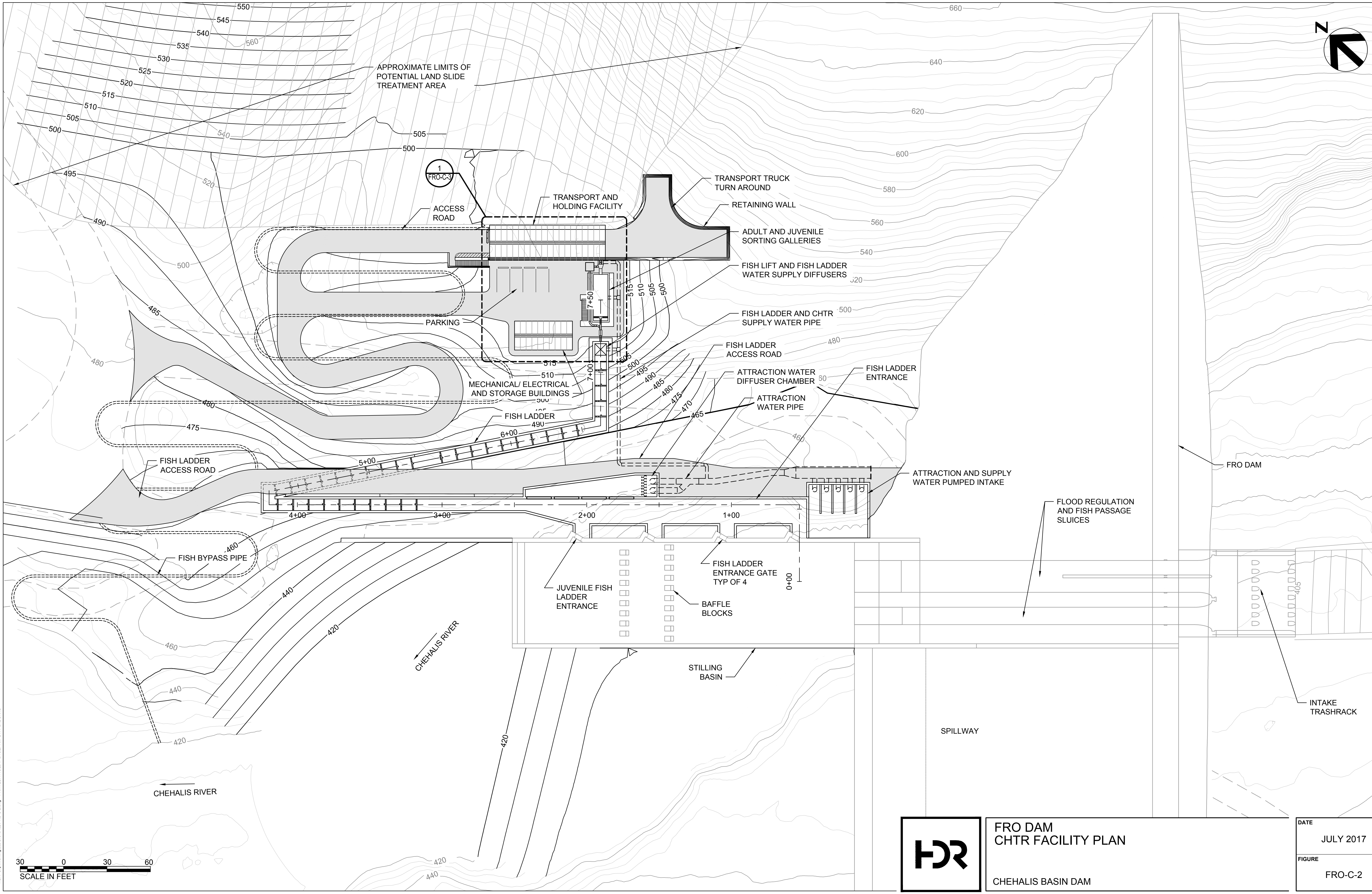
1. DEPENDING ON THE GEOLOGY OF THE AREA, SLIDE TREATMENT AND ADDITIONAL STRUCTURE PROTECTION ACTIVITIES MAY BE REQUIRED WHICH WILL BE EVALUATED DURING THE PRELIMINARY DESIGN PHASE.

HDR

FRO DAM
FISH PASSAGE FACILITIES LOCATION MAP

CHEHALIS BASIN DAM

DATE	JULY 2017
FIGURE	FRO-C-1

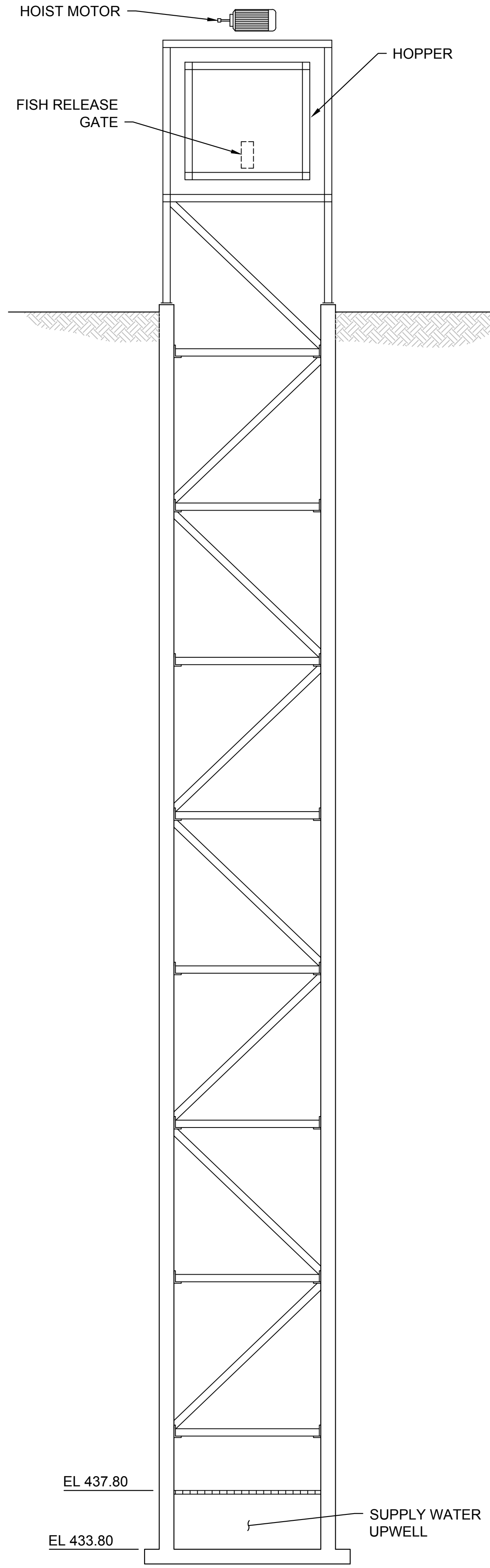


FRO DAM
CHTR FACILITY PLAN

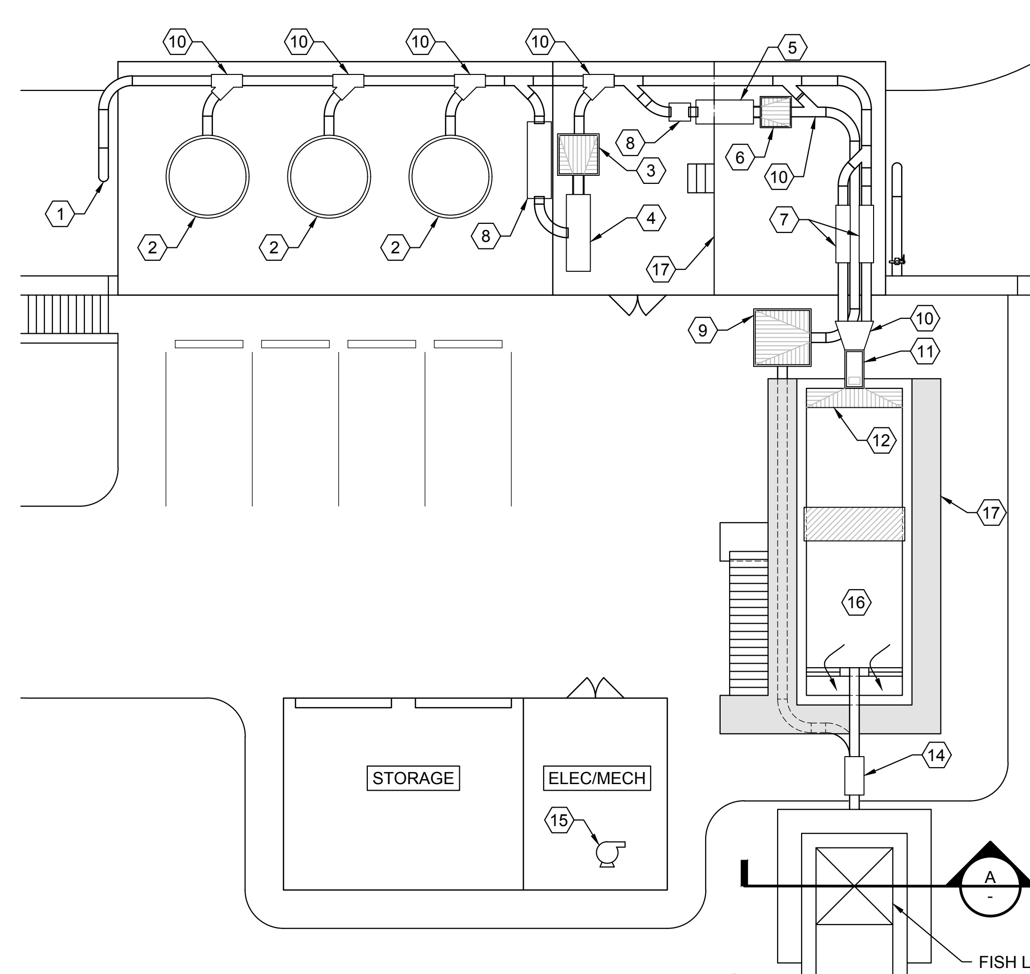
CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRO-C-2



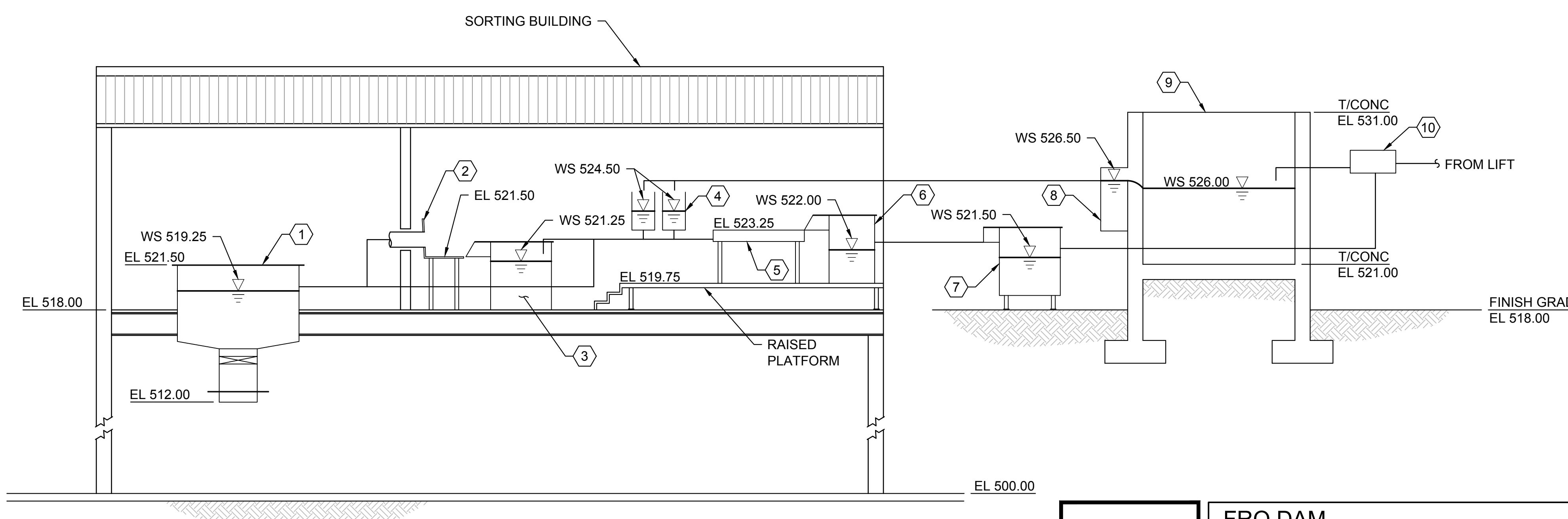
FISH LIFT A
SCALE: NTS



SORTING FACILITY PLAN 1 FRO-C-2
SCALE: NTS

KEYNOTES:

1. BYPASS PIPE
2. 8' DIA x 5' DEEP CIRCULAR HOLDING TANK
3. ADULT ANESTHESIA TANK WITH BRAIL
4. ADULT WORK-UP TABLE
5. JUVENILE WORK-UP TABLE
6. JUVENILE ANESTHESIA TANK WITH BRAIL
7. VISUAL INSPECTION TANKS
8. RECOVERY TANK
9. JUVENILE HOLDING GALLERY
10. DIVERTER GATE
11. FALSE WEIR
12. BRAIL
13. CROWDER
14. ADULT/JUVENILE GRADER
15. JW BOOSTER PUMP
16. ADULT HOLDING GALLERY
17. ELEVATED PLATFORM



KEYNOTES:

1. CIRCULAR HOLDING TANK
2. ADULT WORK-UP TABLE
3. ADULT ANESTHESIA TANK WITH BRAIL
4. VISUAL INSPECTION TANKS
5. JUVENILE WORK-UP TABLE
6. JUVENILE ANESTHESIA TANK WITH BRAIL
7. JUVENILE HOLDING GALLERY WITH BRAIL
8. FALSE WEIR
9. ADULT HOLDING GALLERY
10. GRADER

SORTING FACILITY HYDRAULIC PROFILE 2
SCALE: NTS

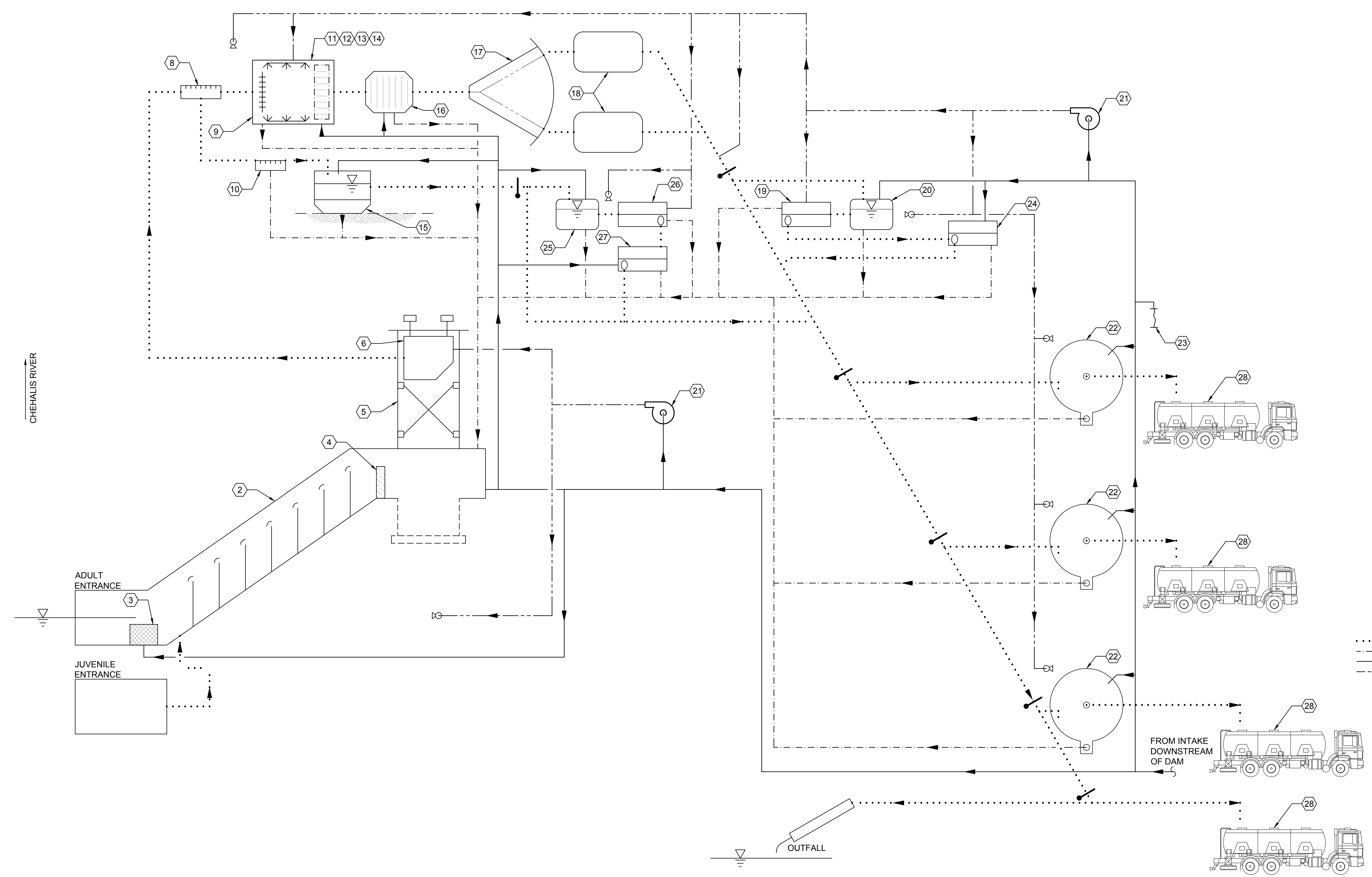
FDR

FRO DAM
CHTR FACILITY PLAN AND ELEVATION

CHEHALIS BASIN DAM

DATE: JULY 2017

FIGURE: FRO-C-3



KEYNOTES:

1. ATTRACTION WATER SUPPLY
2. FISHWAY
3. AUXILIARY WATER SUPPLY
4. TRAP GATE
5. HOPPER LIFT TOWER
6. HOPPER BUCKET
7. LAMPREY HOLDING HOPPER
8. GRADER
9. ADULT HOLDING GALLERY
10. DEWATERING SCREEN
11. SPRINKLER SYSTEM
12. CROWDER
13. HOLDING GALLERY UPWELL
14. HOLDING GALLERY BRAIL
15. JUVENILE HOLDING GALLERY
16. FALSE WEIR
17. AUTOMATIC DIVERTER GATE
18. VISUAL INSPECTION TANKS
19. ADULT WORKUP TABLE
20. ADULT HOLDING TANK
21. JOB WATER BOOSTER PUMP
22. CIRCULAR HOLDING TANK
23. TRUCK FILL
24. ADULT RECOVERY TANK
25. JUVENILE HOLDING TANK
26. JUVENILE WORKUP TABLE
27. JUVENILE RECOVERY TABLE
28. ADULT/ JUVENILE TRANSPORT TRUCK

LEGEND:

- FISH FLUME (FF)
- DRAIN (DR)
- WATER SUPPLY (WS)
- JOB WATER (JW)
- GATE
- HOSE BIB

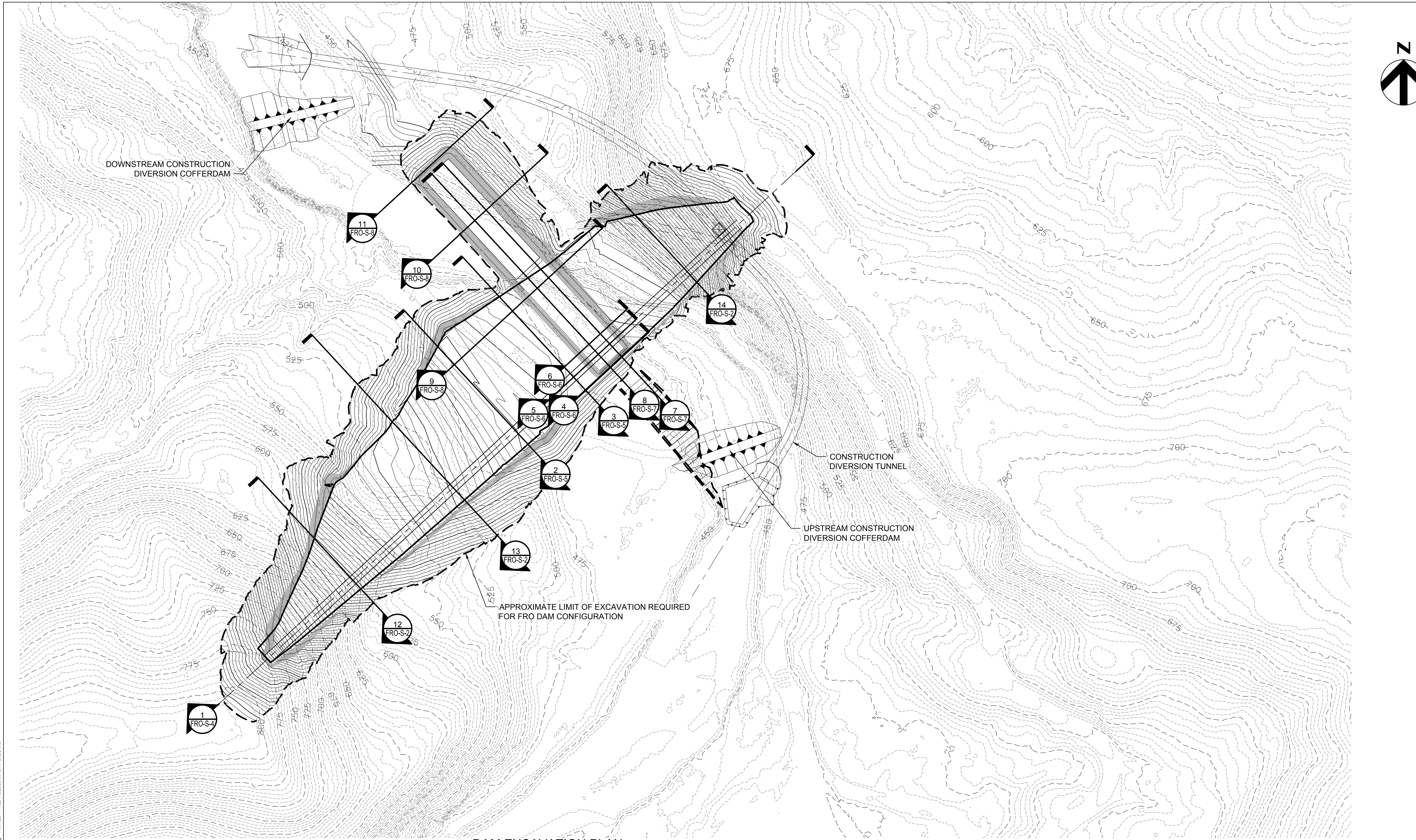
HDR

FRO DAM
CHTR FACILITY PROCESS DIAGRAM

CHEHALIS BASIN DAM

DATE: JULY 2017

FIGURE: FRO-C-4



DAM EXCAVATION PLAN

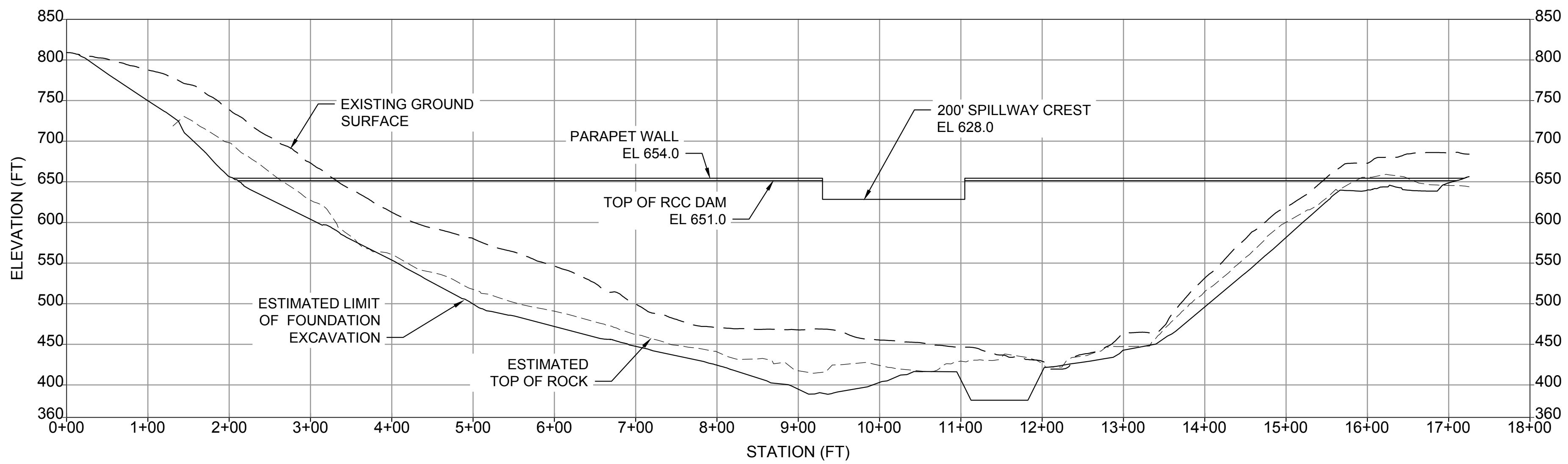
0 50 100 200
SCALE IN FEET

FRO DAM
DAM EXCAVATION AND
FOUNDATION TREATMENT PLAN

CHEHALIS BASIN DAM

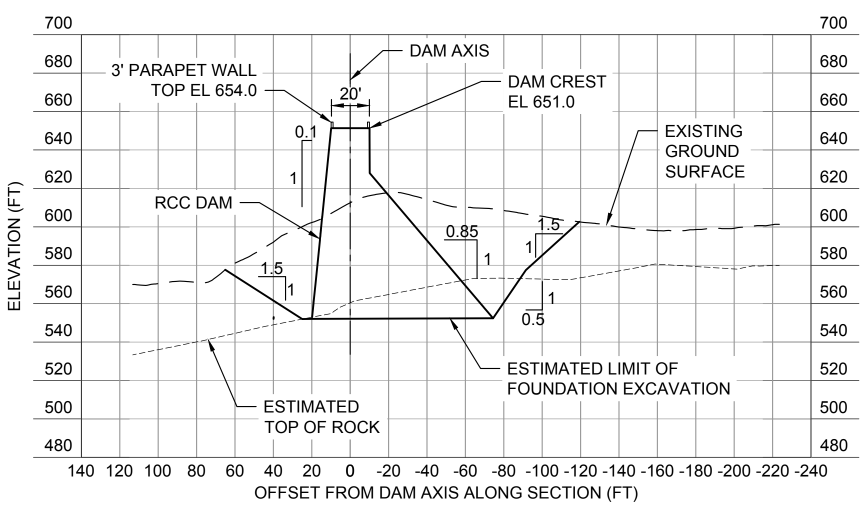
DATE
JULY 2017

FIGURE
FRO-S-1



DAM EXCAVATION PROFILE

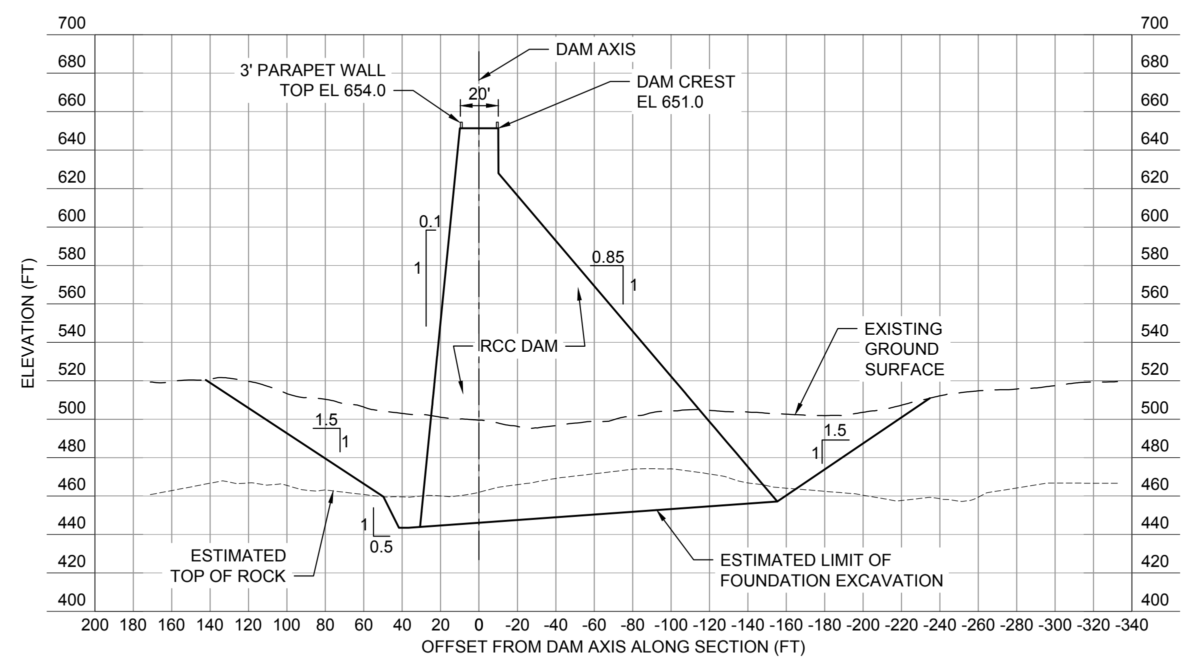
0 50 100 200
SCALE IN FEET



DAM EXCAVATION TYPICAL SECTION

12 FRO-S-1

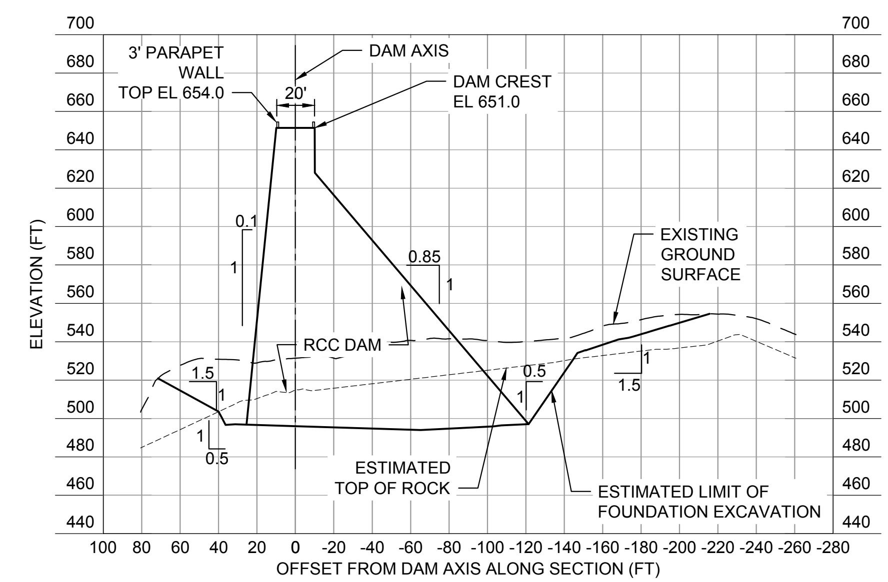
0 25 50 100
SCALE IN FEET



DAM EXCAVATION TYPICAL SECTION

13 FRO-S-1

0 25 50 100
SCALE IN FEET



DAM EXCAVATION TYPICAL SECTION

14 FRO-S-1

0 25 50 100
SCALE IN FEET

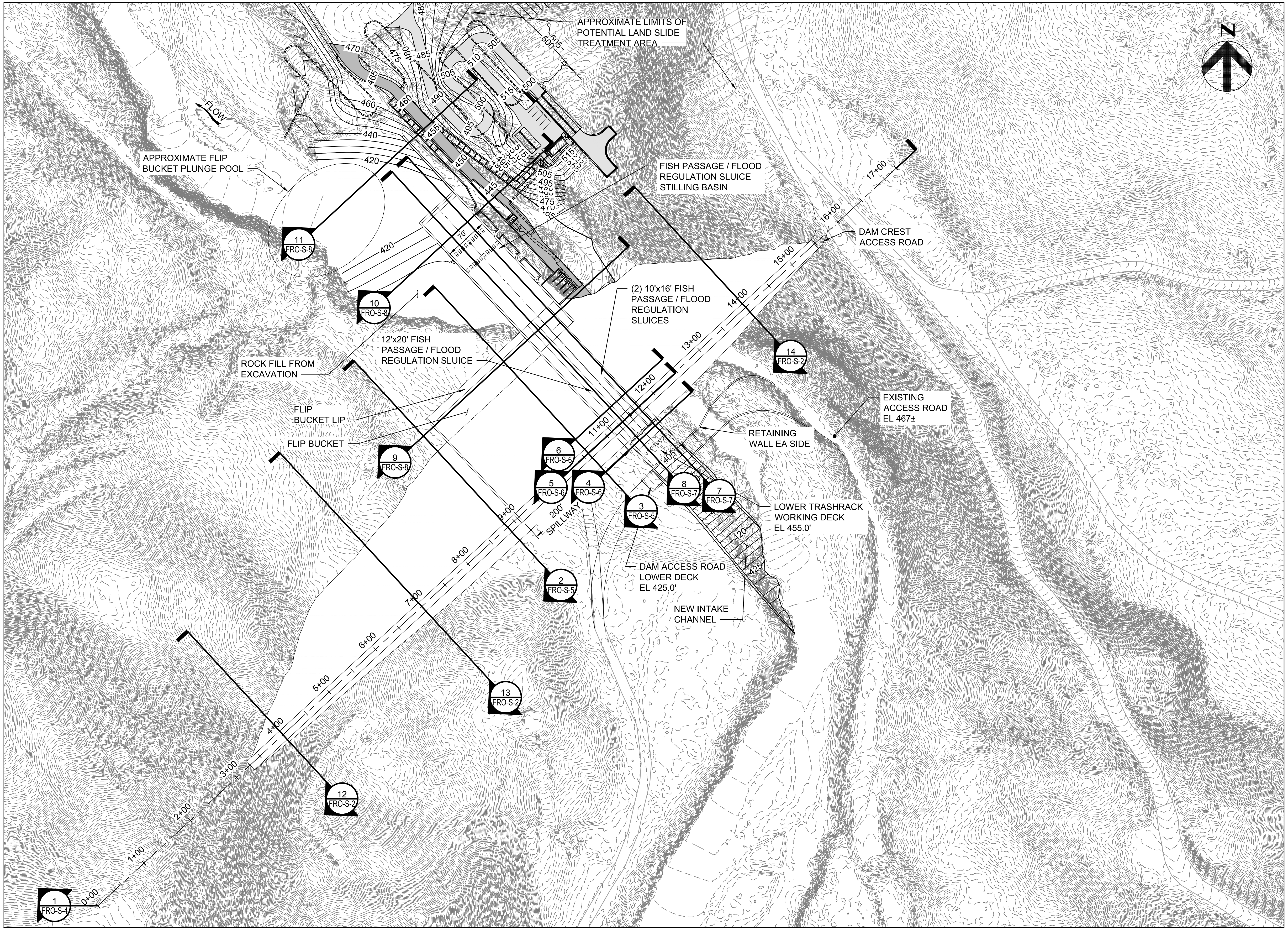
HDR

FRO DAM
DAM EXCAVATION PROFILE AND TYPICAL SECTIONS 12, 13 AND 14

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRO-S-2



PLAN

0 40 80 160
SCALE IN FEET

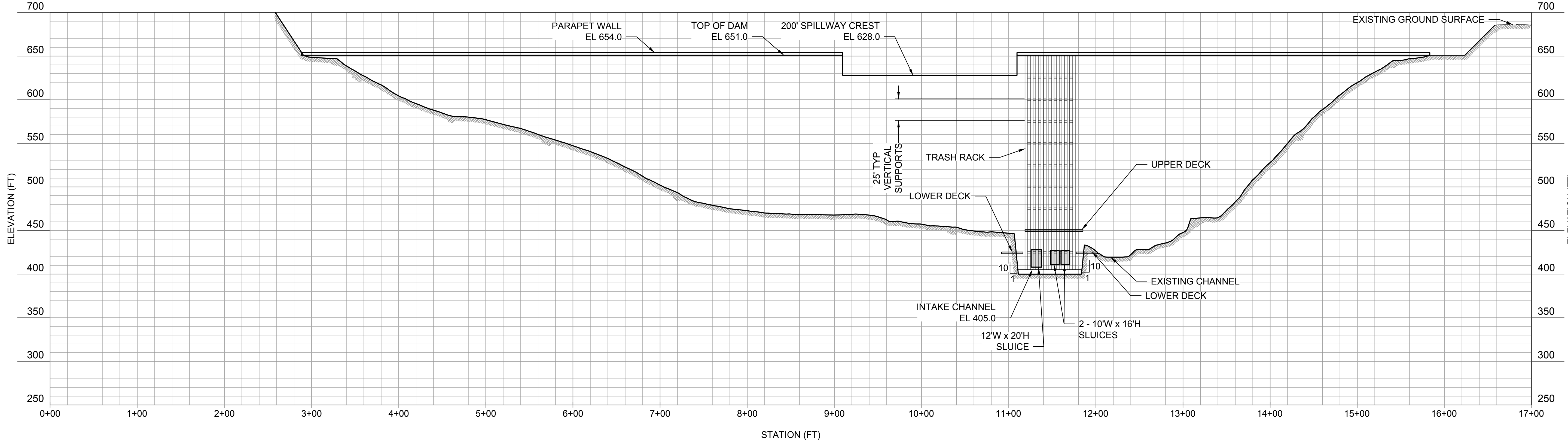
HDR

FRO DAM
DAM PLAN

CHEHALIS BASIN DAM

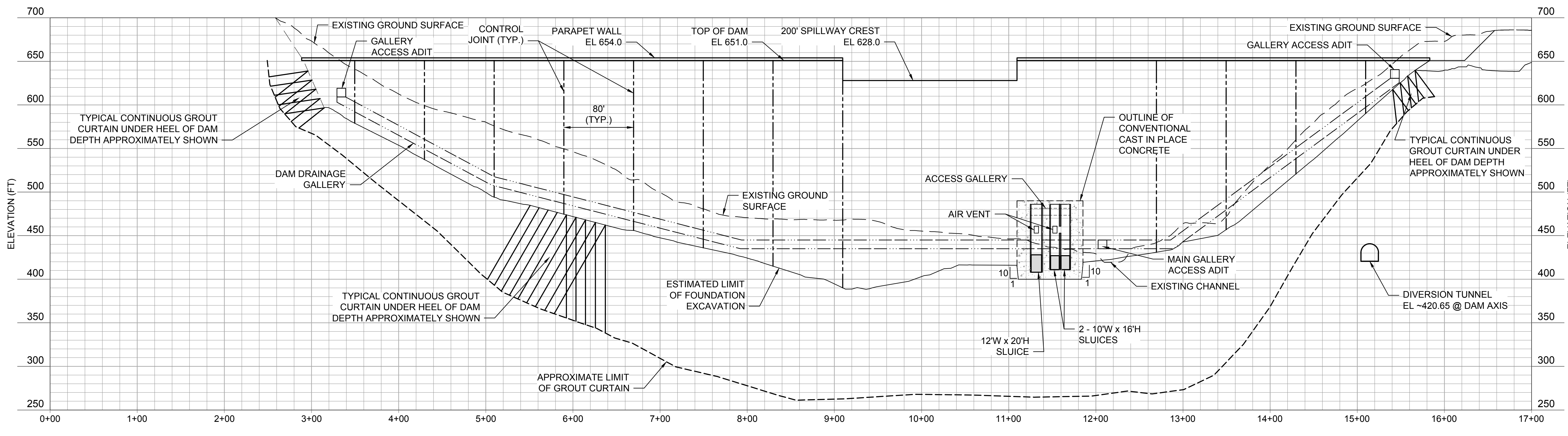
DATE
JULY 2017

FIGURE
FRO-S-3



DAM ELEVATION VIEW

0 30 60 120
SCALE IN FEET



SECTION ALONG DAM AXIS 1 FRO-S-3

0 30 60 120
SCALE IN FEET

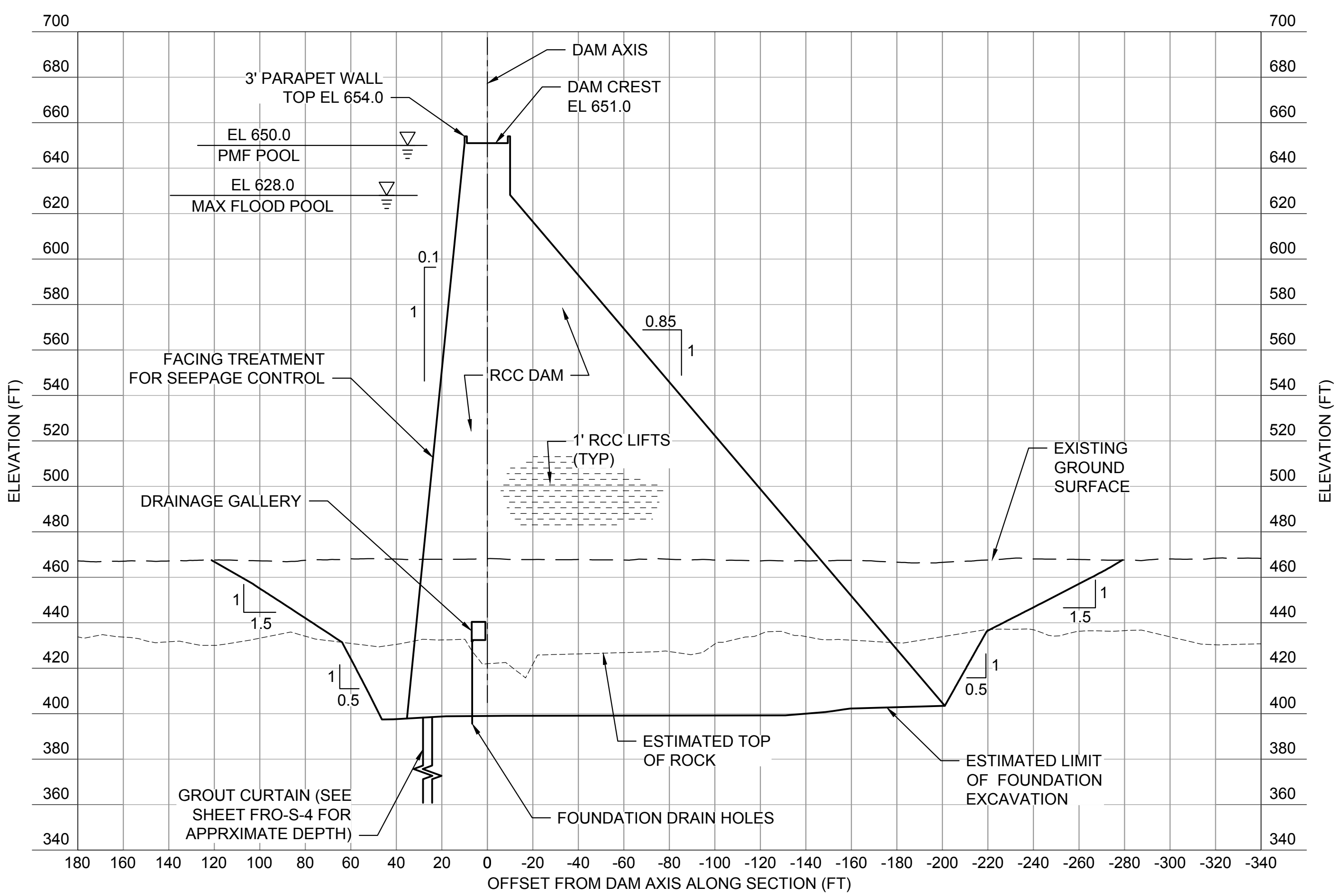
HDR

FRO DAM
DAM ELEVATION VIEW
AND AXIS SECTION 1

CHEHALIS BASIN DAM

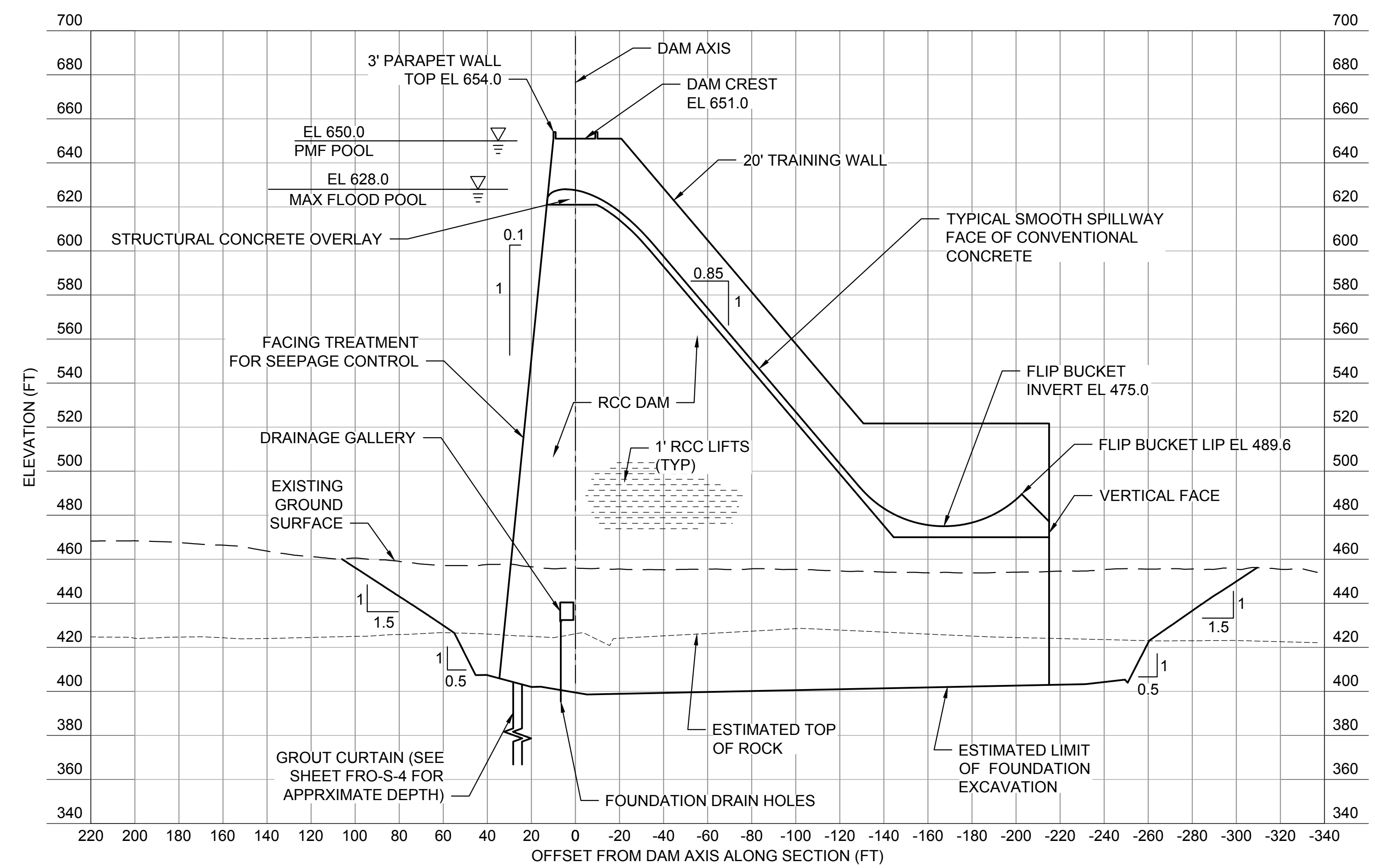
DATE: JULY 2017

FIGURE: FRO-S-4



SECTION 2 FRO-S-3

0 20 40 80
SCALE IN FEET



SECTION 3 FRO-S-3

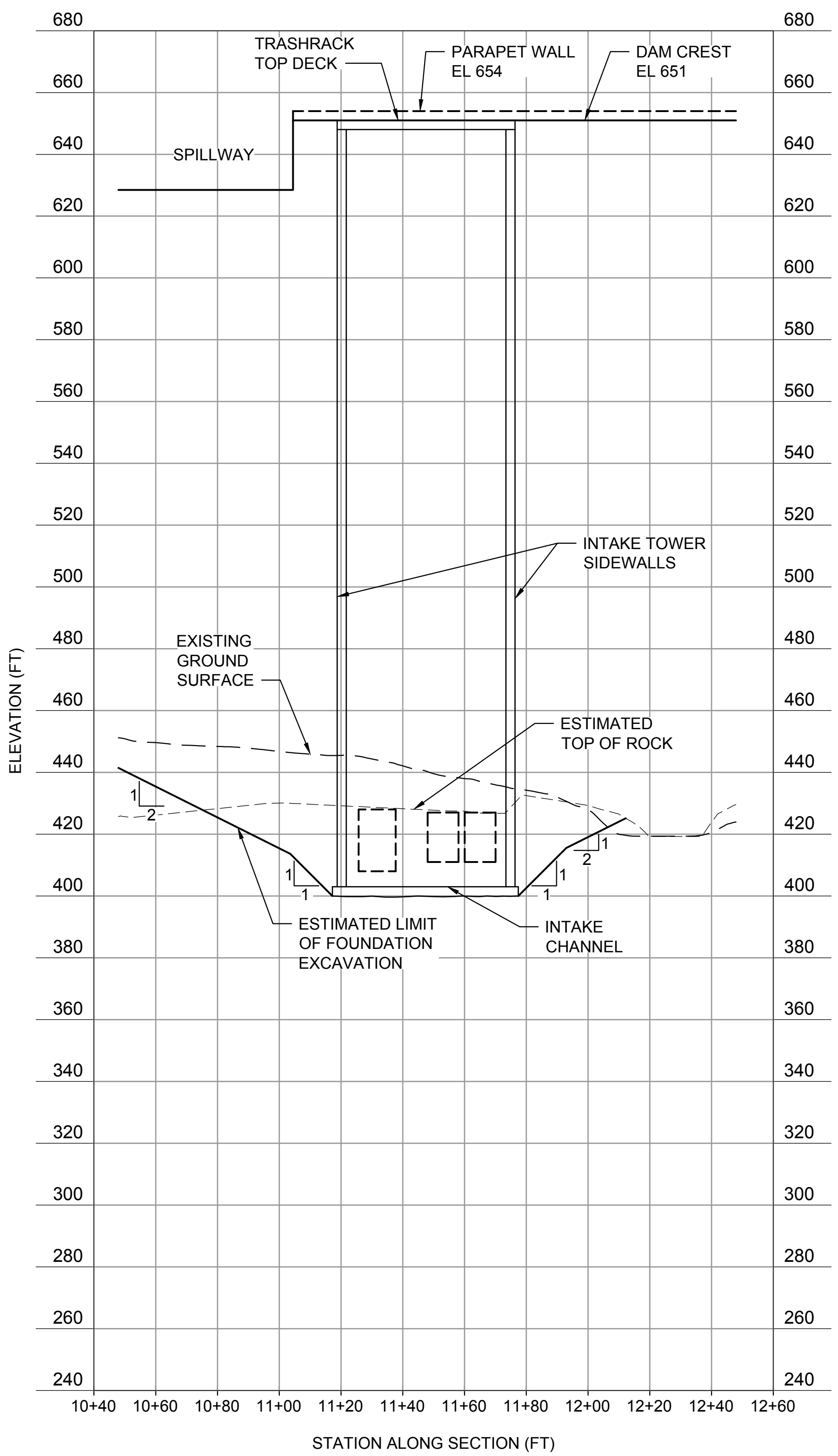
0 20 40 80
SCALE IN FEET

FRO DAM
TYPICAL DAM AND SPILLWAY
SECTIONS 2 AND 3

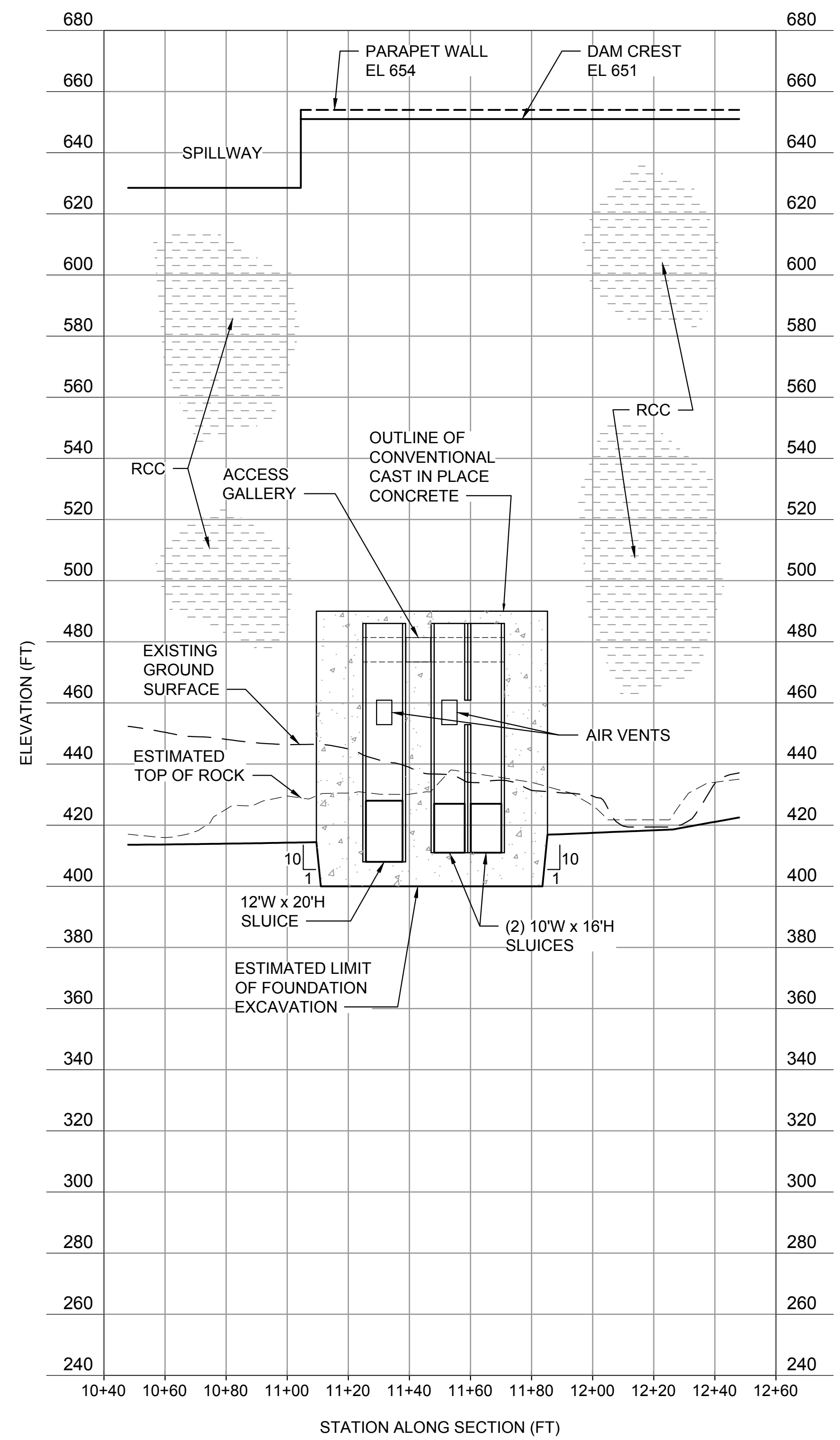
CHEHALIS BASIN DAM

DATE
JULY 2017

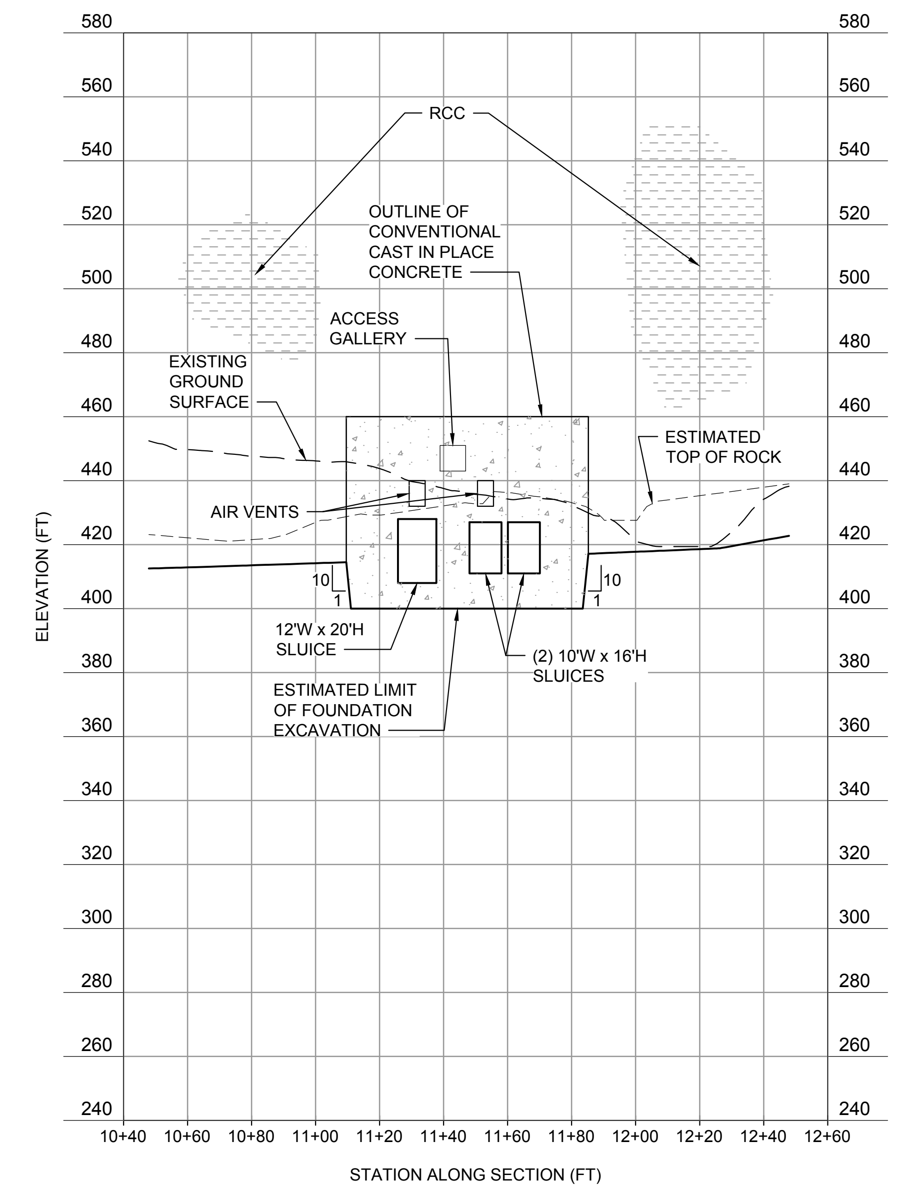
FIGURE
FRO-S-5



SECTION 4 FRO-S-3



SECTION 5 FRO-S-3



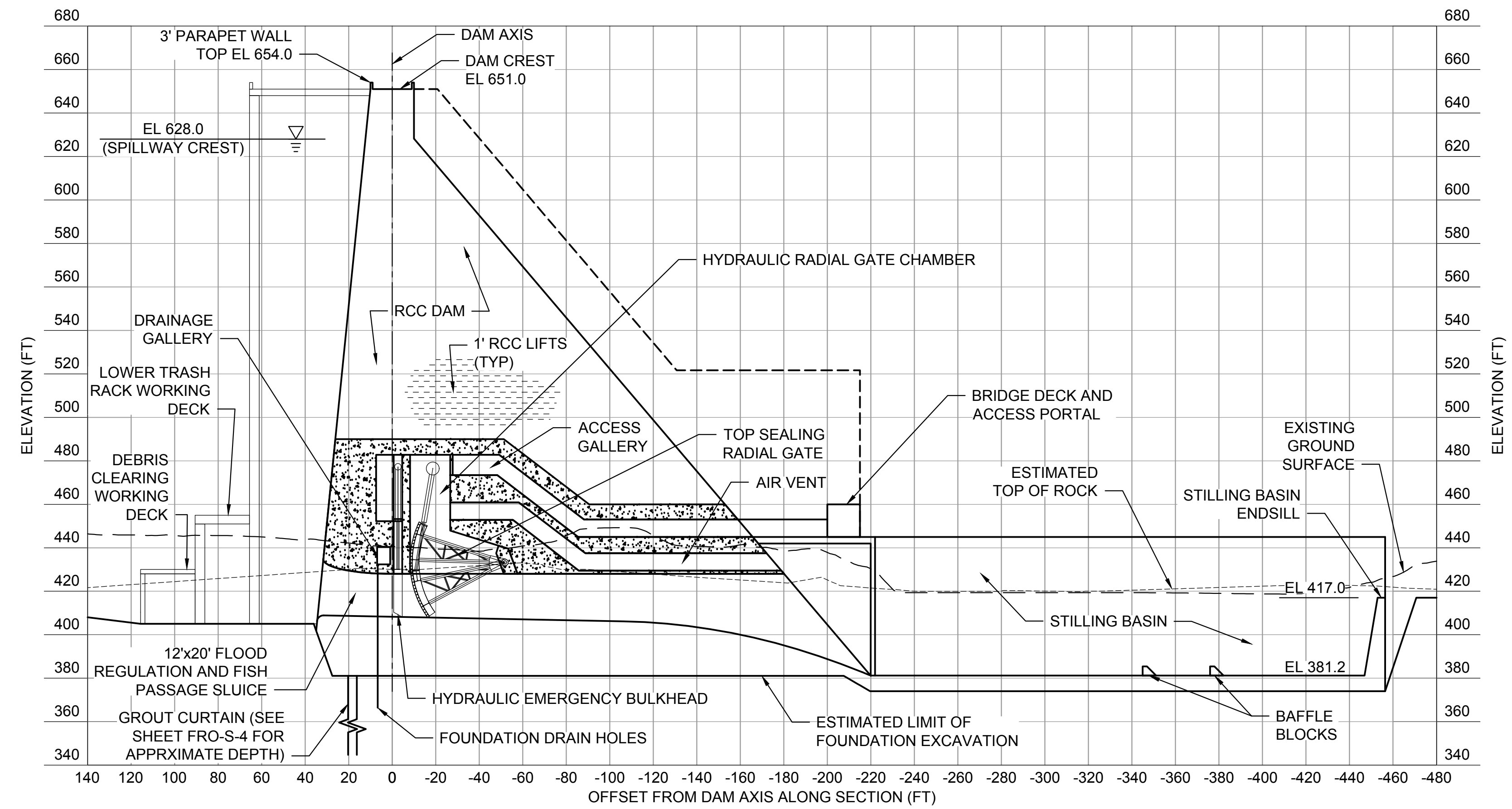
SECTION 6 FRO-S-3

0 15 30 60
SCALE IN FEET

FRO DAM
INTAKE TOWER AND SLUICES
SECTIONS 4, 5, AND 6

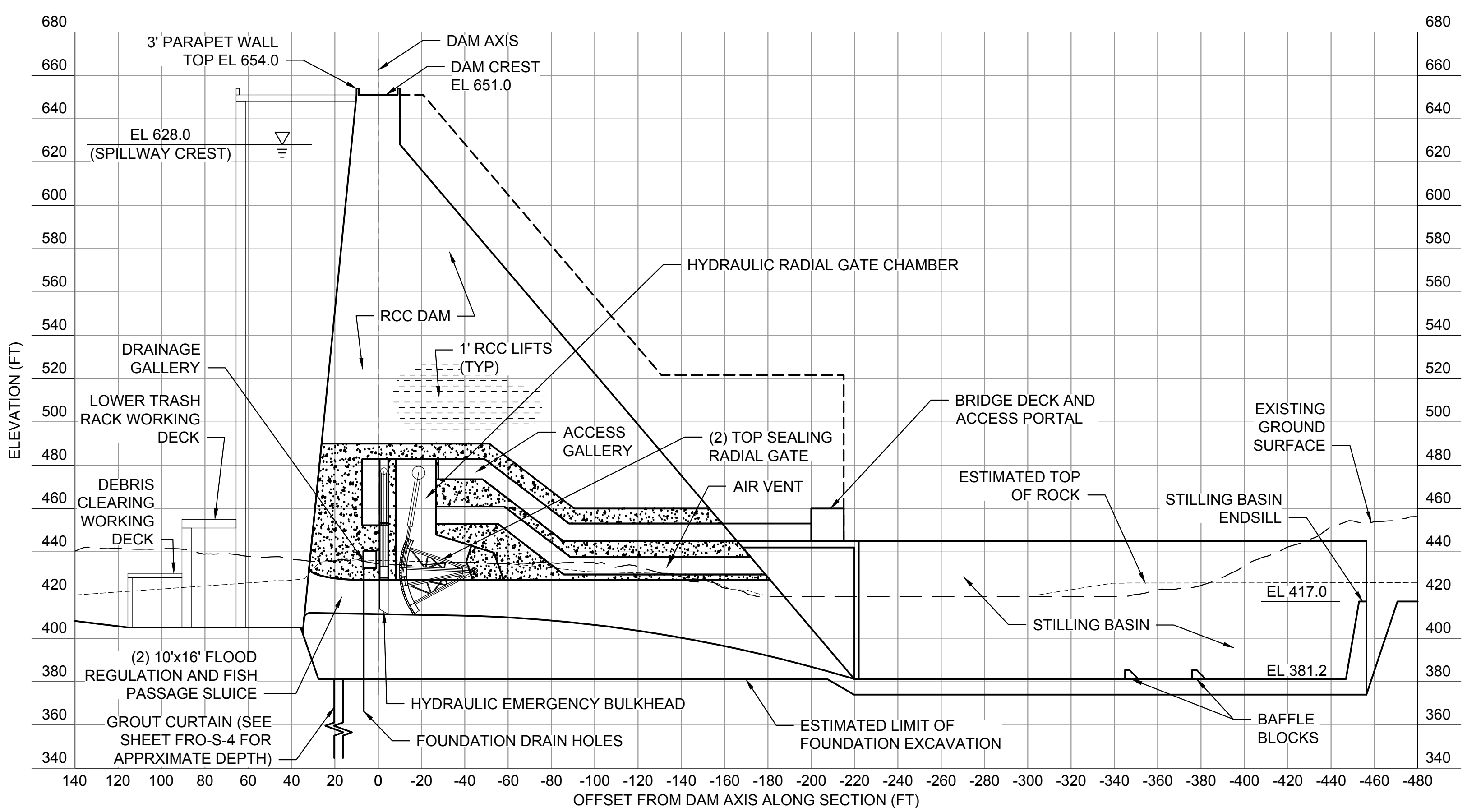
CHEHALIS BASIN DAM

DATE	JULY 2017
FIGURE	FRO-S-6



SECTION 8 FRO-S-3

0 20 40 80
SCALE IN FEET



SECTION 7 FRO-S-3

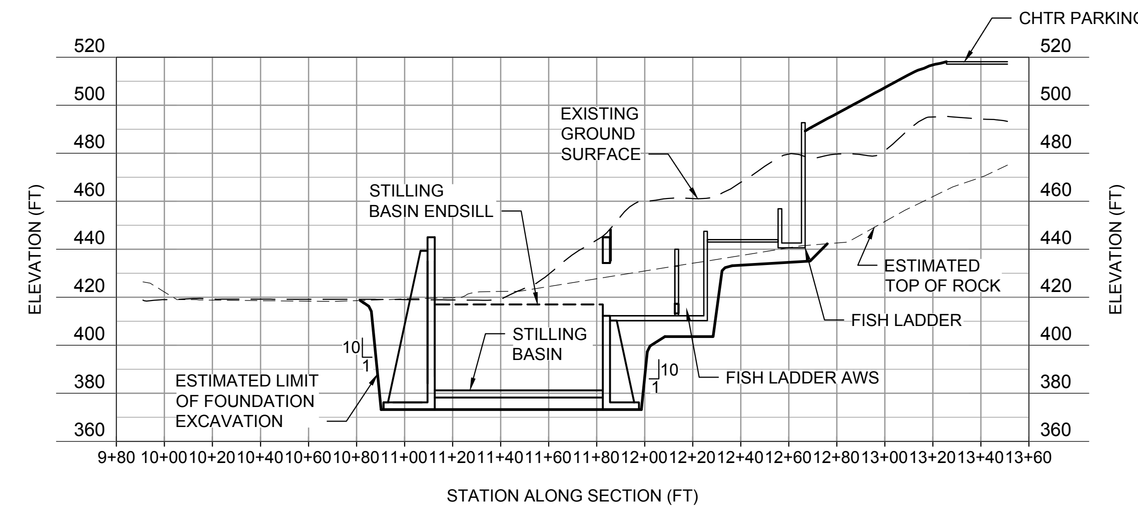
0 20 40 80
SCALE IN FEET

HDR

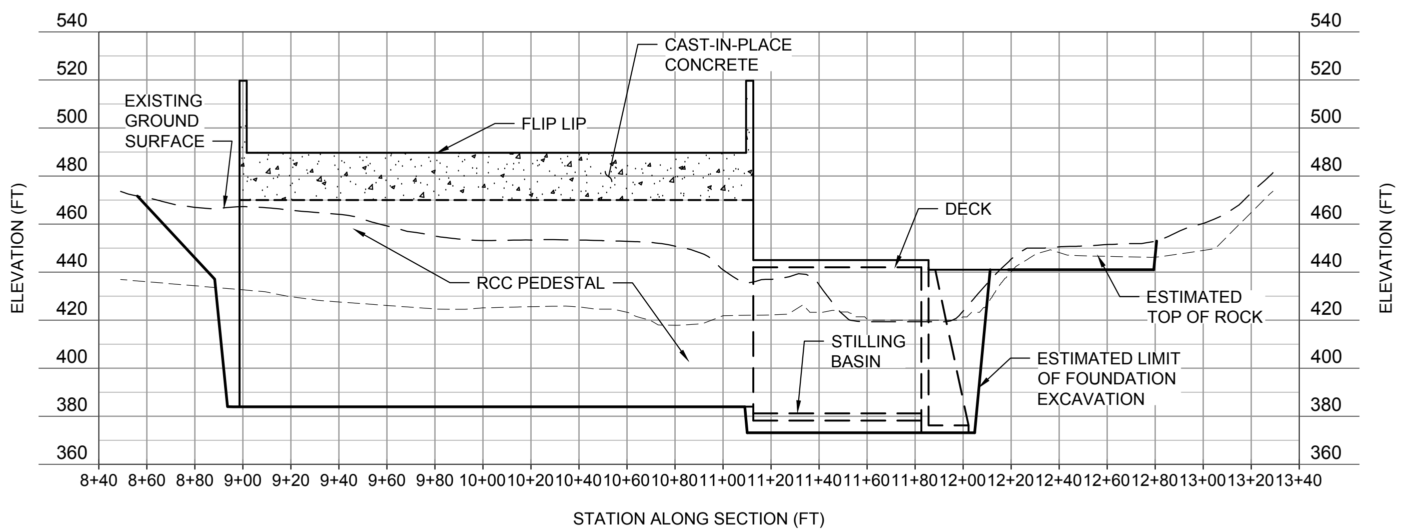
FRO DAM
FLOOD REGULATION AND
FISH PASSAGE SLUICES
SECTIONS 7 AND 8
CHEHALIS BASIN DAM

DATE
JULY 2017

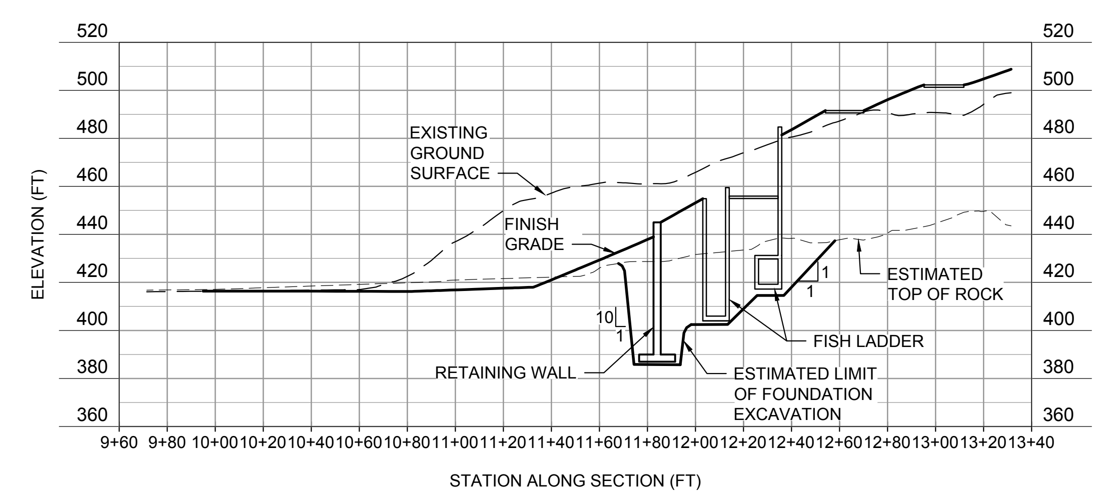
FIGURE
FRO-S-7



SECTION 10 FRO-S-3



SECTION 9 FRO-S-3



SECTION 11 FRO-S-3

0 20 40 80
SCALE IN FEET

HDR

FRO DAM
FLOOD REGULATION AND
FISH PASSAGE SLUICES STILLING BASIN
SECTIONS 9, 10, AND 11
CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRO-S-8

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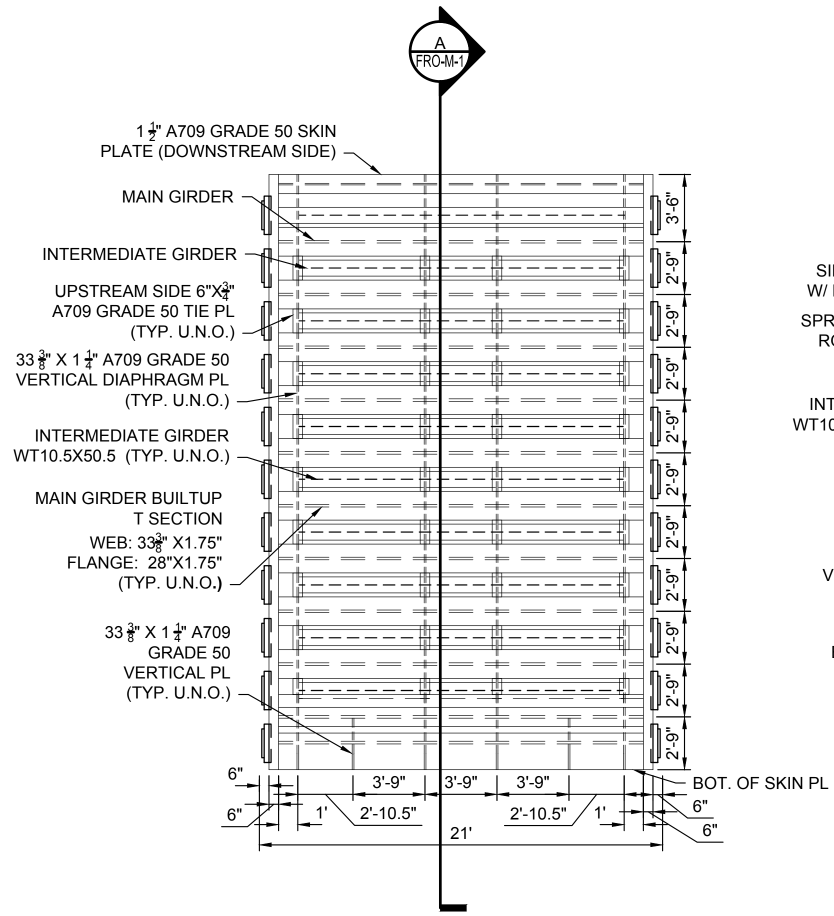
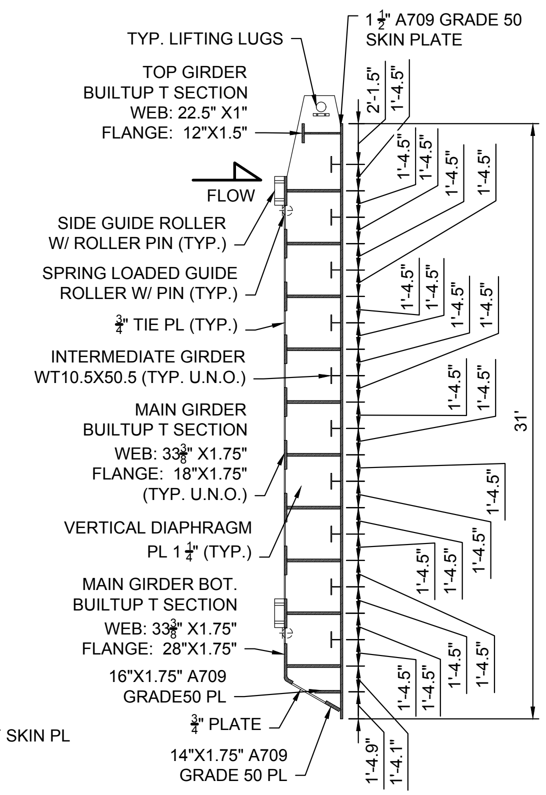
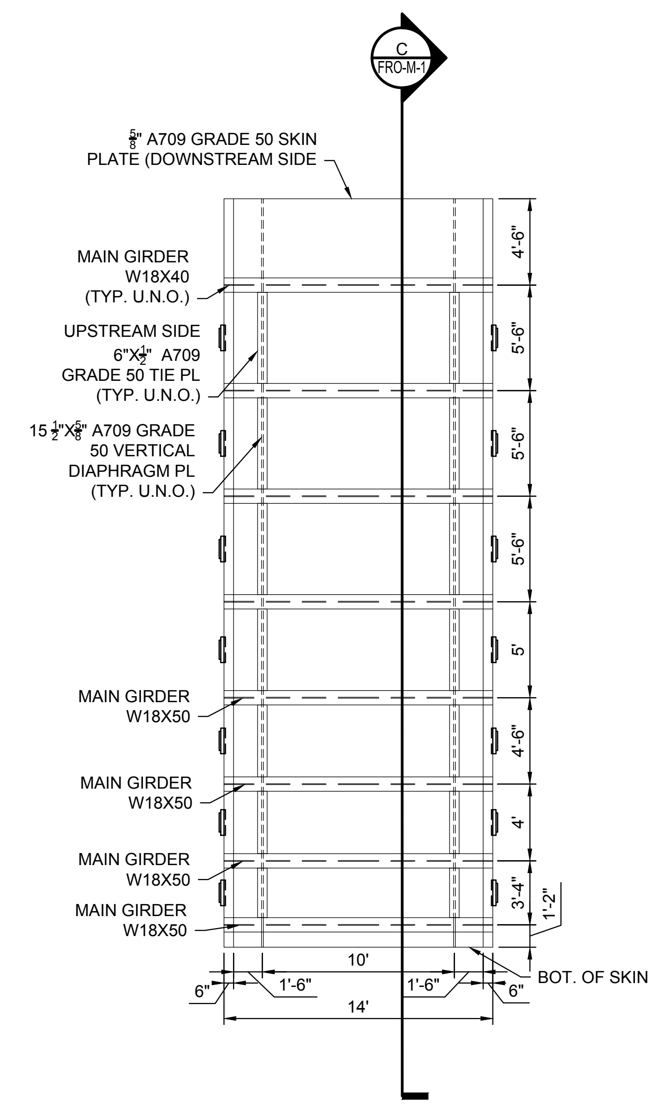


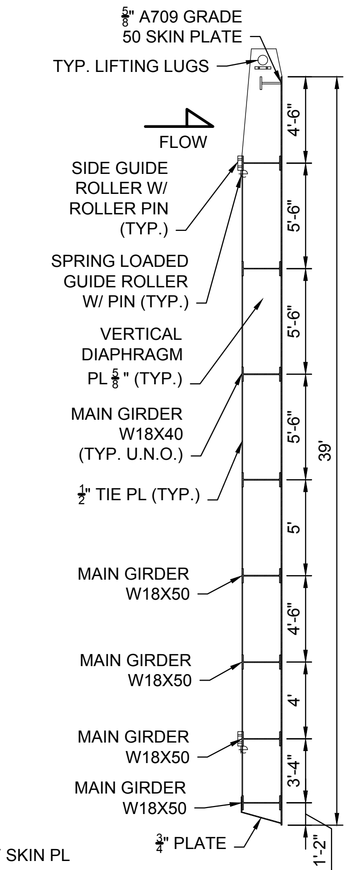
ELEVATION UPSTREAM DEWATERING BULKHEAD (1, FRO-M-1)



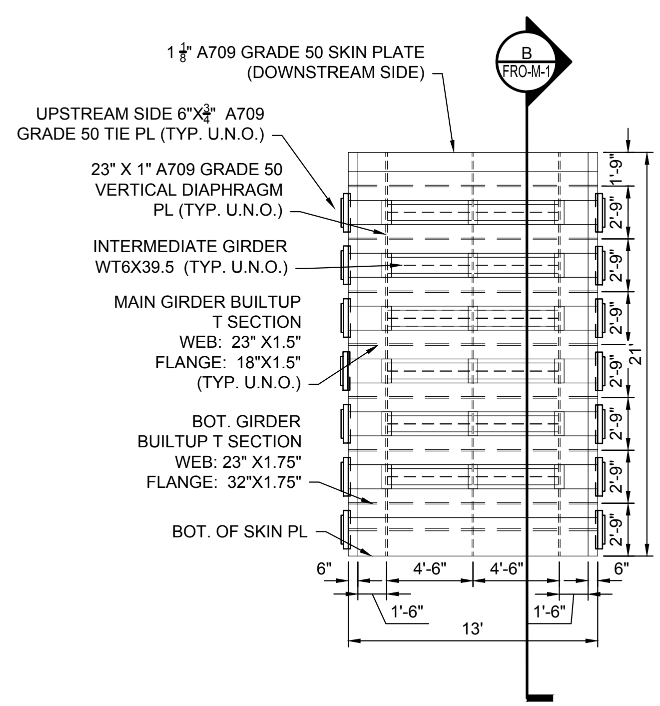
SECTION (A, FRO-M-1)



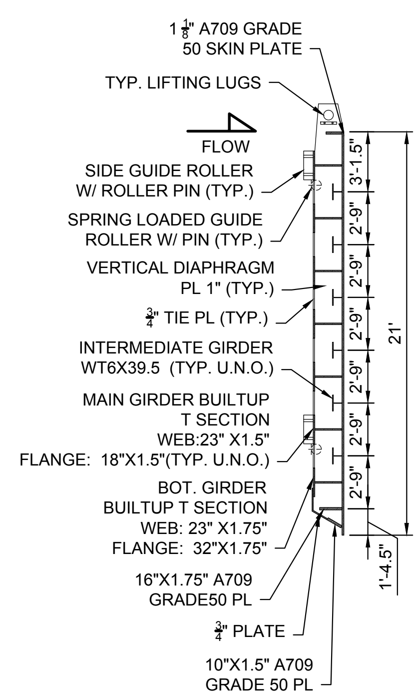
ELEVATION DOWNSTREAM DEWATERING BULKHEAD (3, FRO-M-1)



SECTION (C, FRO-M-1)



ELEVATION EMERGENCY CLOSURE GATE (3, FRO-M-1)



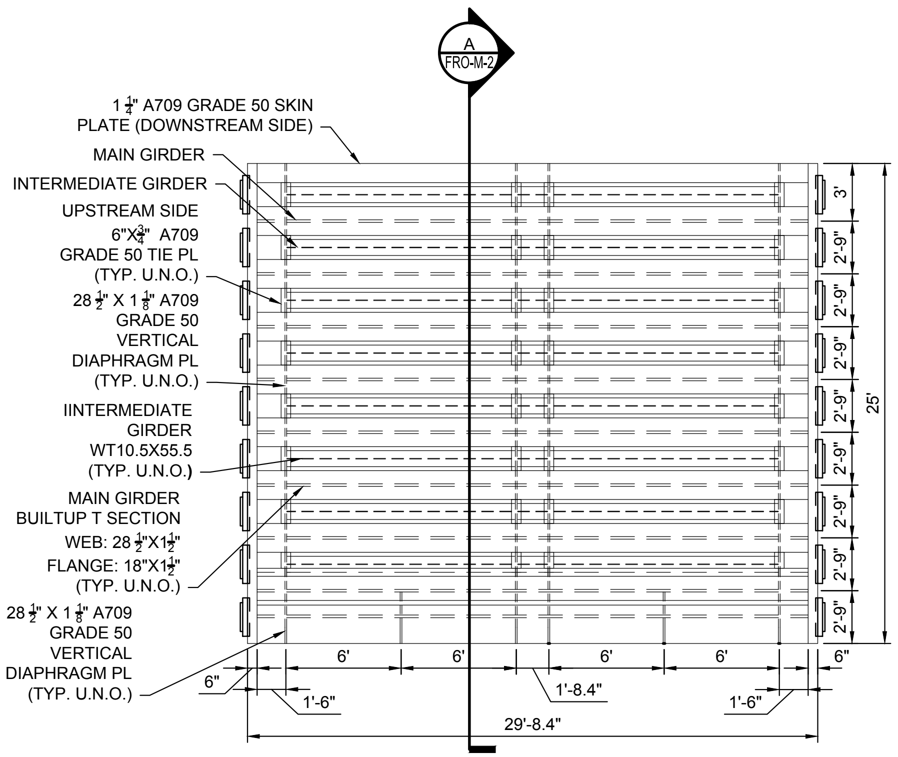
SECTION (B, FRO-M-1)

0 2.5 5 10
SCALE IN FEET

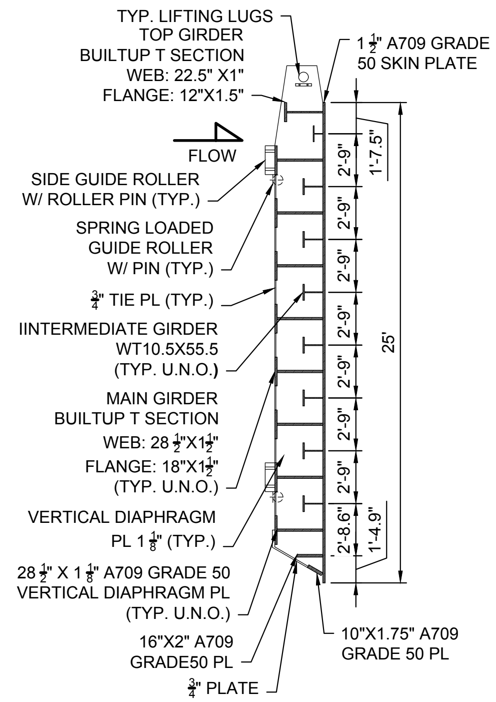
HDR

FRO DAM
FLOOD REGULATION AND FISH PASSAGE
SLUICES GATES AND BULKHEADS DETAILS
12' x 20' SLUICE
CHEHALIS BASIN DAM

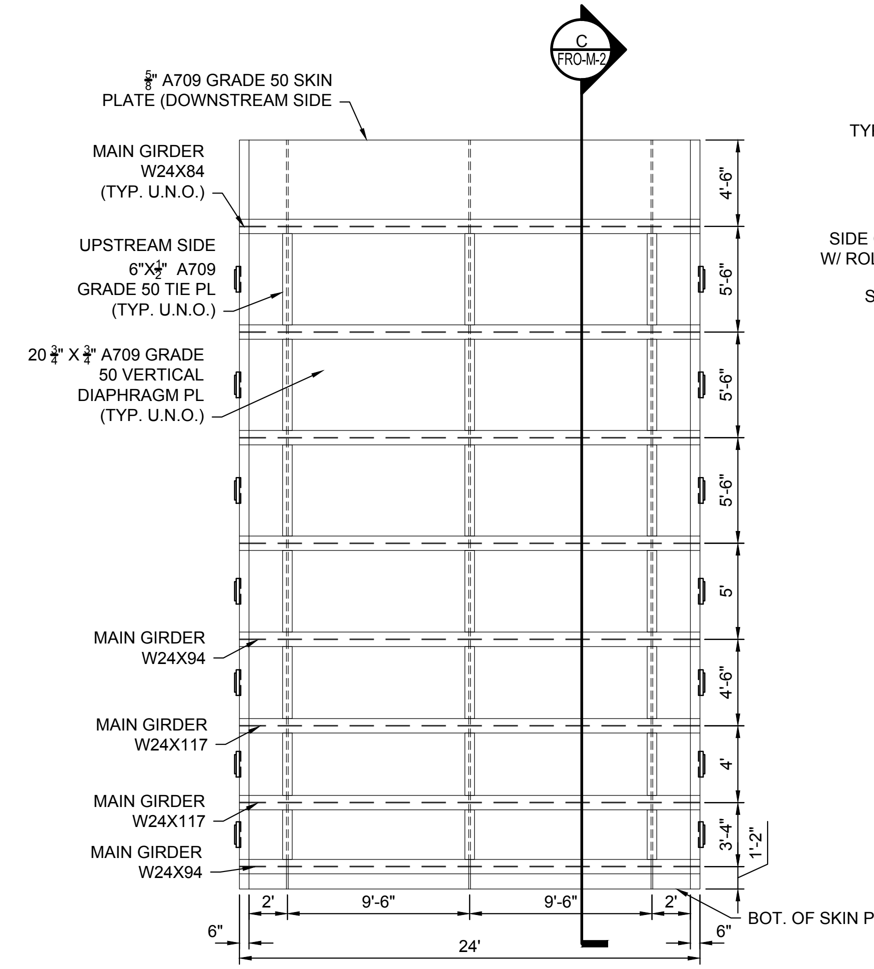
DATE	JULY 2017
FIGURE	FRO-M-1



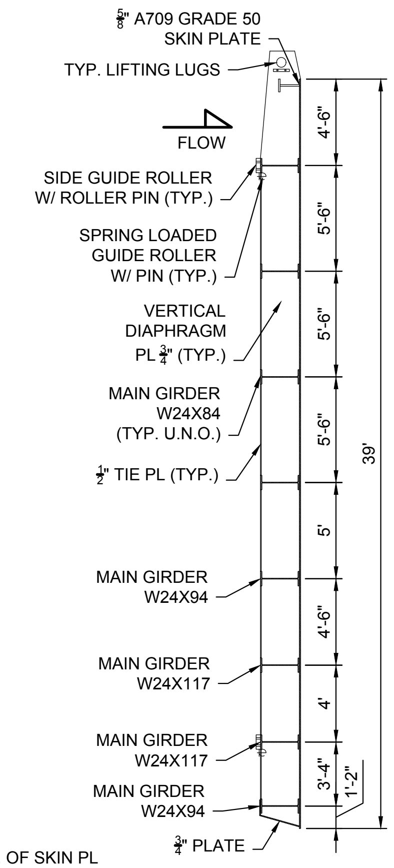
ELEVATION UPSTREAM DEWATERING BULKHEAD (1 / FRO-M-2)



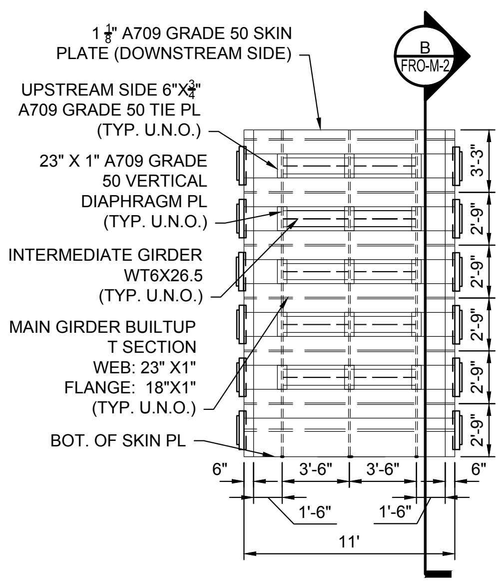
SECTION (A / FRO-M-2)



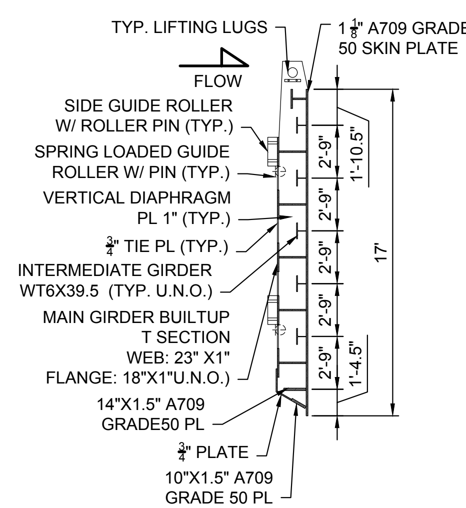
ELEVATION DOWNSTREAM DEWATERING BULKHEAD (3 / FRO-M-2)



SECTION (C / FRO-M-2)



ELEVATION EMERGENCY CLOSURE GATE (3 / FRO-M-2)



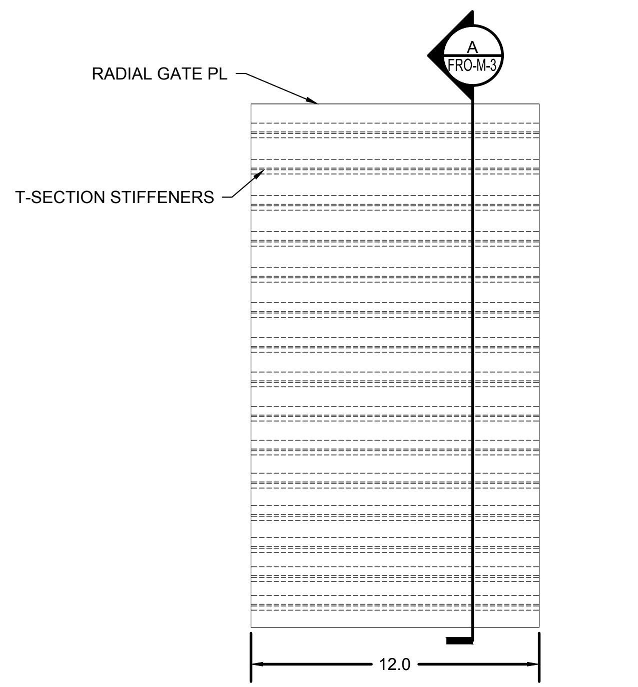
SECTION (B / FRO-M-2)

0 2.5 5 10
SCALE IN FEET

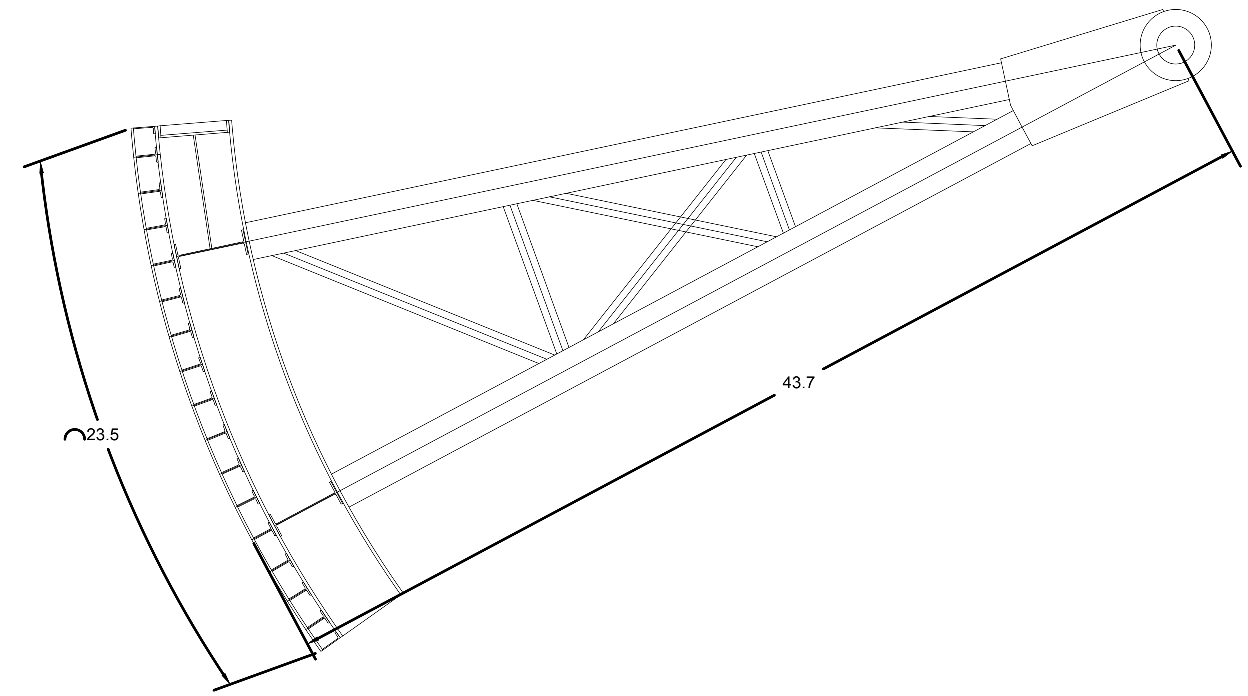
HDR

FRO DAM
FLOOD REGULATION AND FISH PASSAGE
SLUICES GATES AND BULKHEADS DETAILS
10' x 16' SLUICE
CHEHALIS BASIN DAM

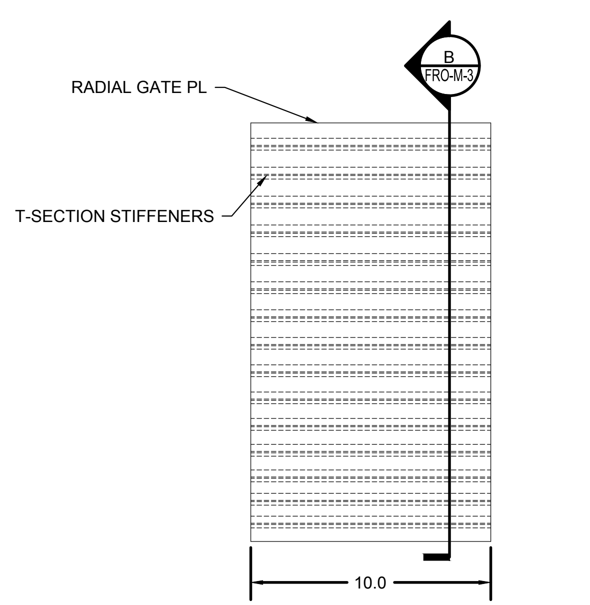
DATE	JULY 2017
FIGURE	FRO-M-2



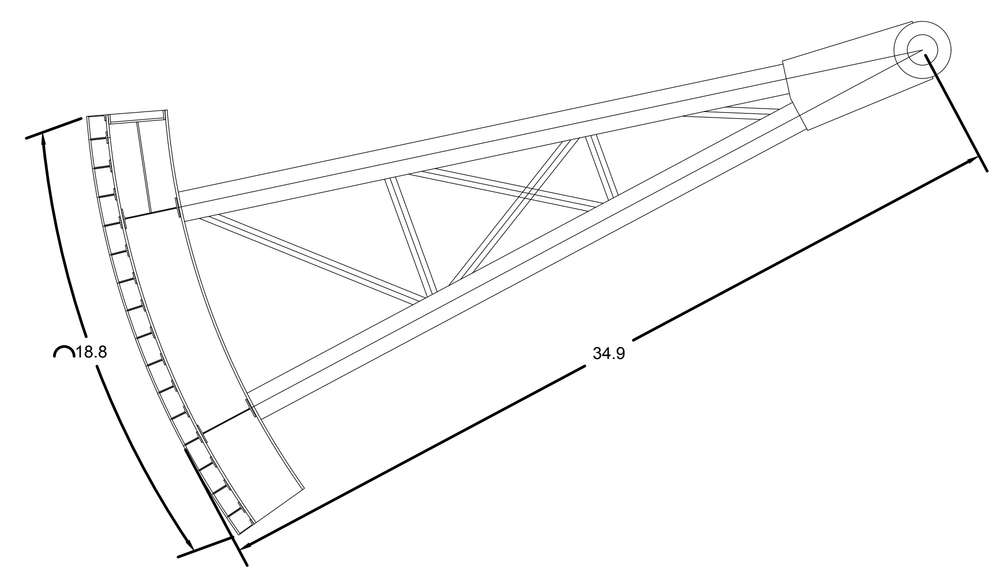
FRONT ELEVATION 12'X20'
RADIAL SLUICE GATE
1
FRO-M-3



SECTION
A
FRO-M-3



FRONT ELEVATION 10'X16'
RADIAL SLUICE GATE
2
FRO-M-3



SECTION
B
FRO-M-3

0 2 4 8
SCALE IN FEET

HDR

FRO DAM
FLOOD REGULATION AND FISH PASSAGE
SLUICES GATES DETAILS
12' x 20' AND 10' x 16' RADIAL GATES
CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRO-M-3

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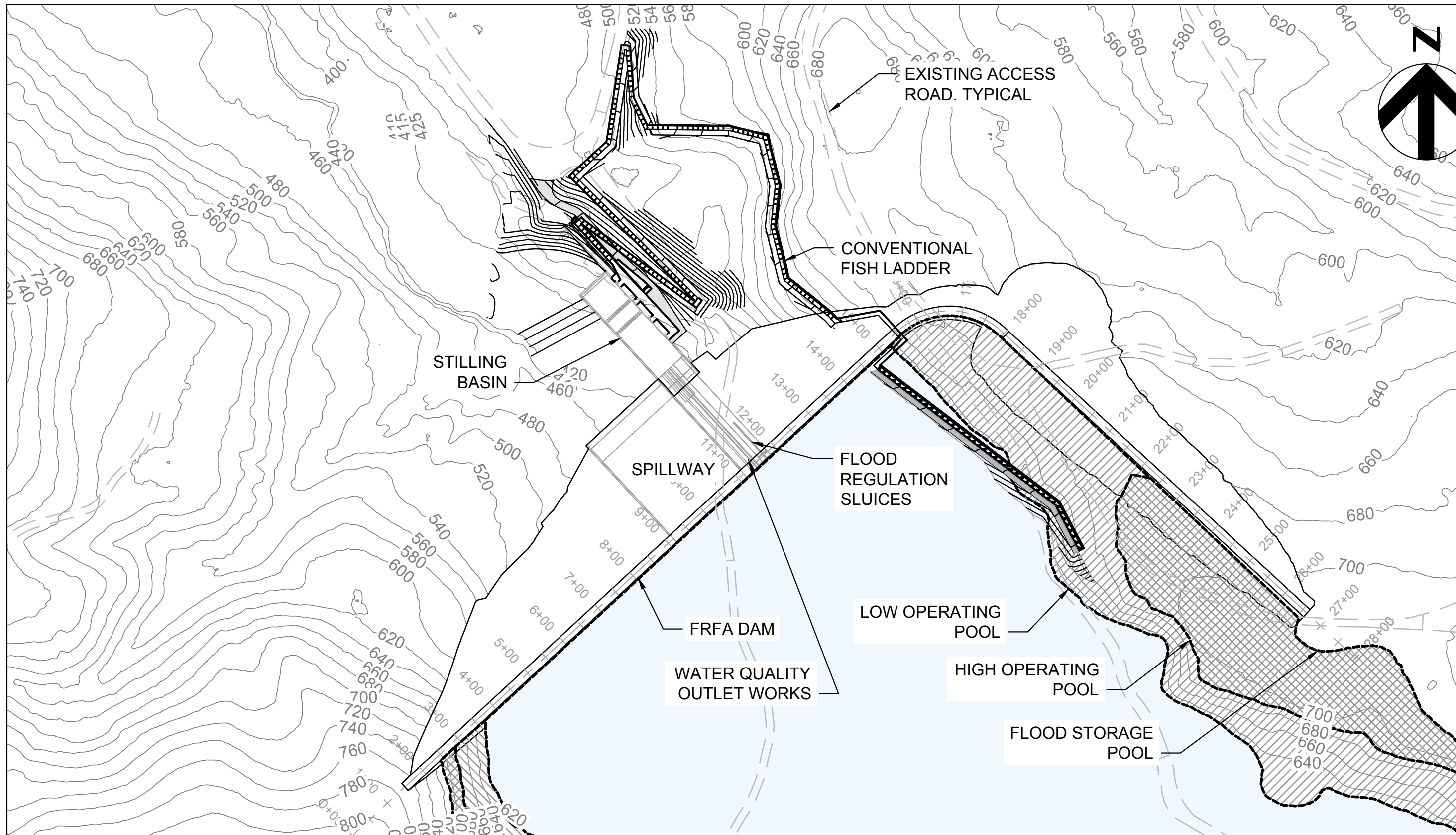
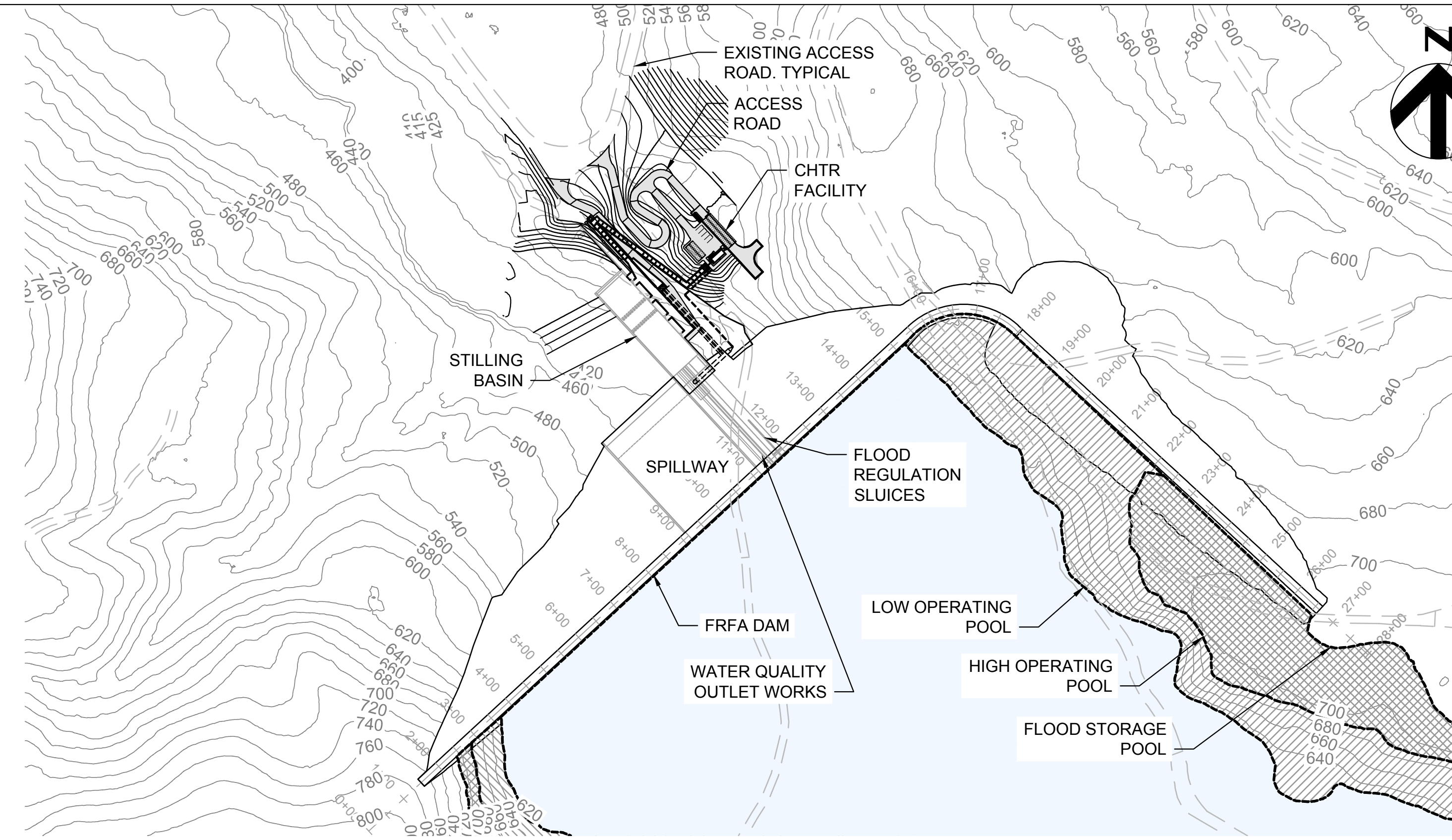


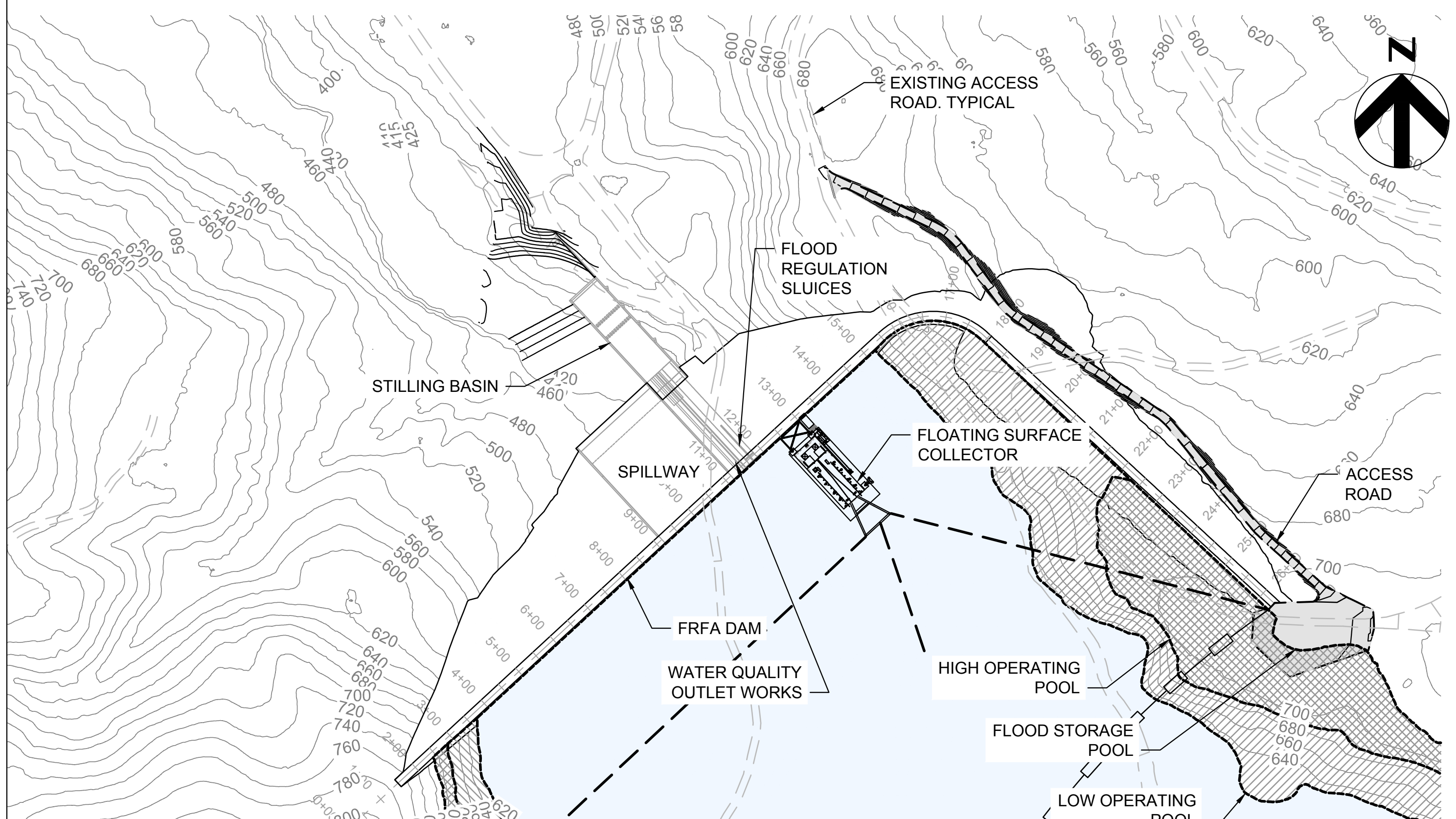
SEE FRFA-C-2 THROUGH FRFA-C-4

UPSTREAM PASSAGE: FISH LADDER PLAN



SEE FRFA-C-5 THROUGH FRFA-C-7

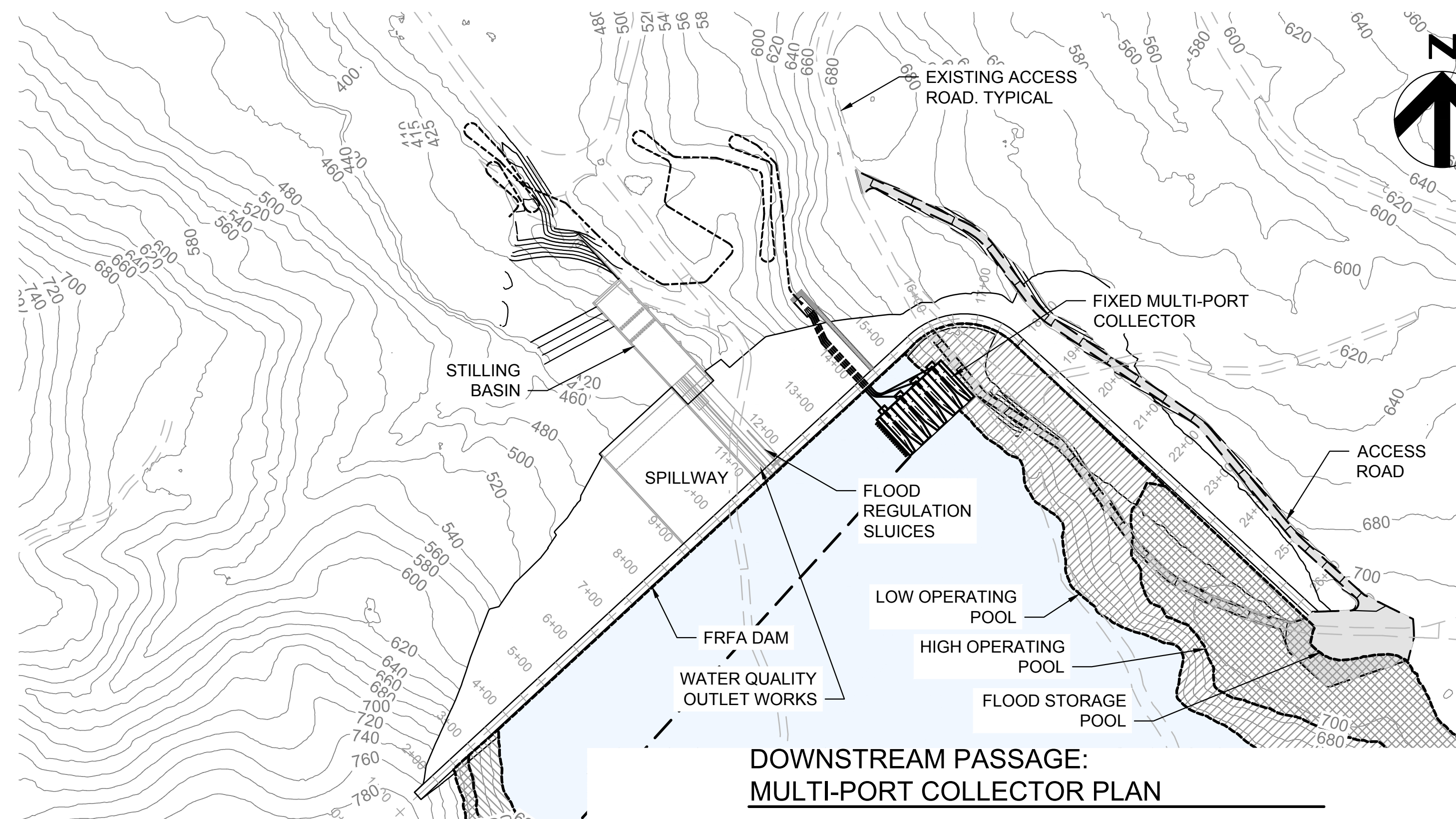
UPSTREAM PASSAGE: CHTR FACILITY PLAN



SEE FRFA-C-8 THROUGH FRFA-C-10

DOWNSTREAM PASSAGE:
FLOATING SURFACE COLLECTOR PLAN

100 0 100 200
SCALE IN FEET



DOWNSTREAM PASSAGE:
MULTI-PORT COLLECTOR PLAN

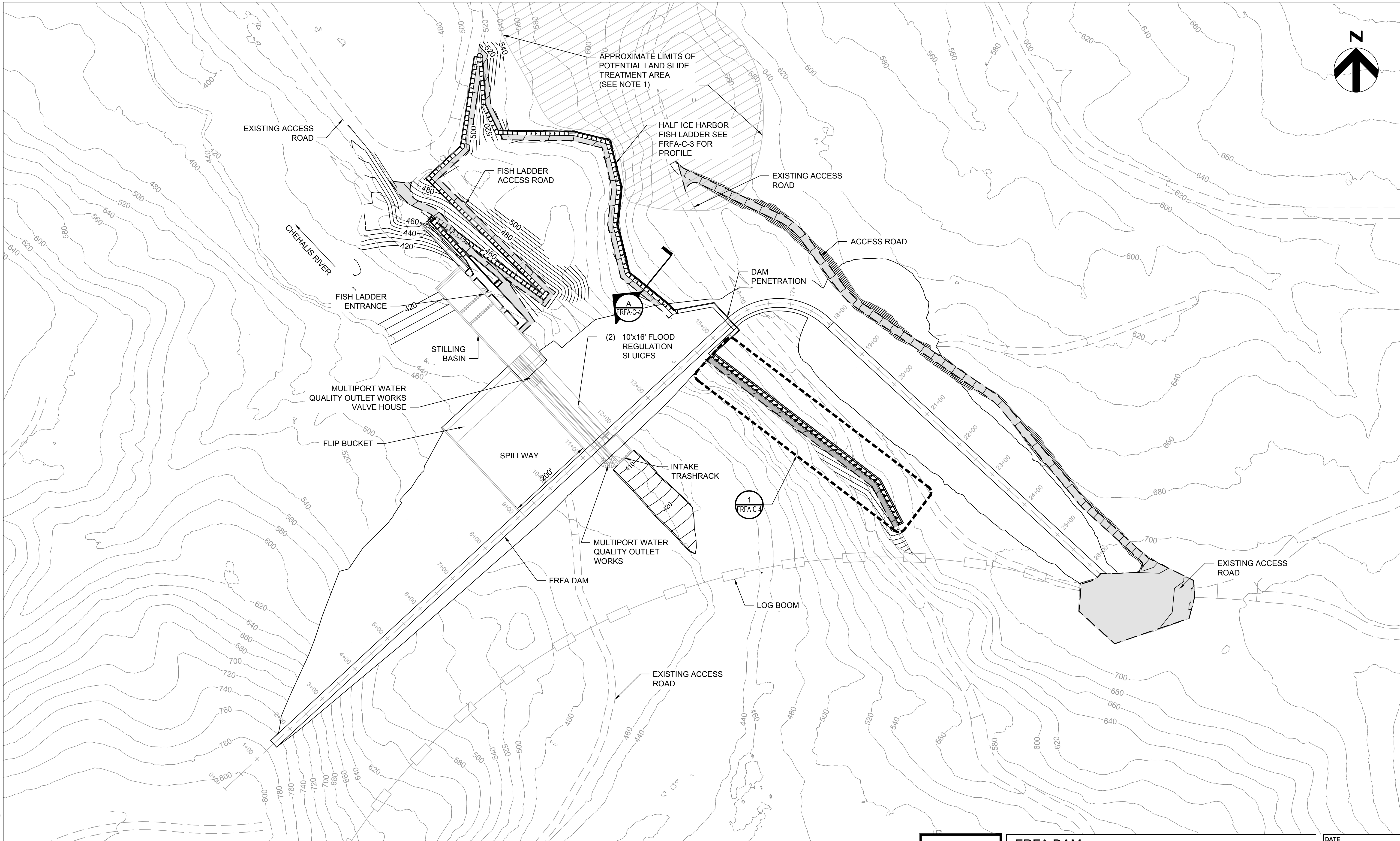
SEE FRFA-C-11 THROUGH FRFA-C-13

HDR

FRFA DAM
FISH PASSAGE FACILITIES LOCATION MAP

CHEHALIS BASIN DAM

DATE	JULY 2017
FIGURE	FRFA-C-1



NOTES:

1. DEPENDING ON THE GEOLOGY OF THE AREA, SLIDE TREATMENT AND ADDITIONAL STRUCTURE PROTECTION ACTIVITIES MAY BE REQUIRED WHICH WILL BE EVALUATED DURING THE PRELIMINARY DESIGN PHASE.

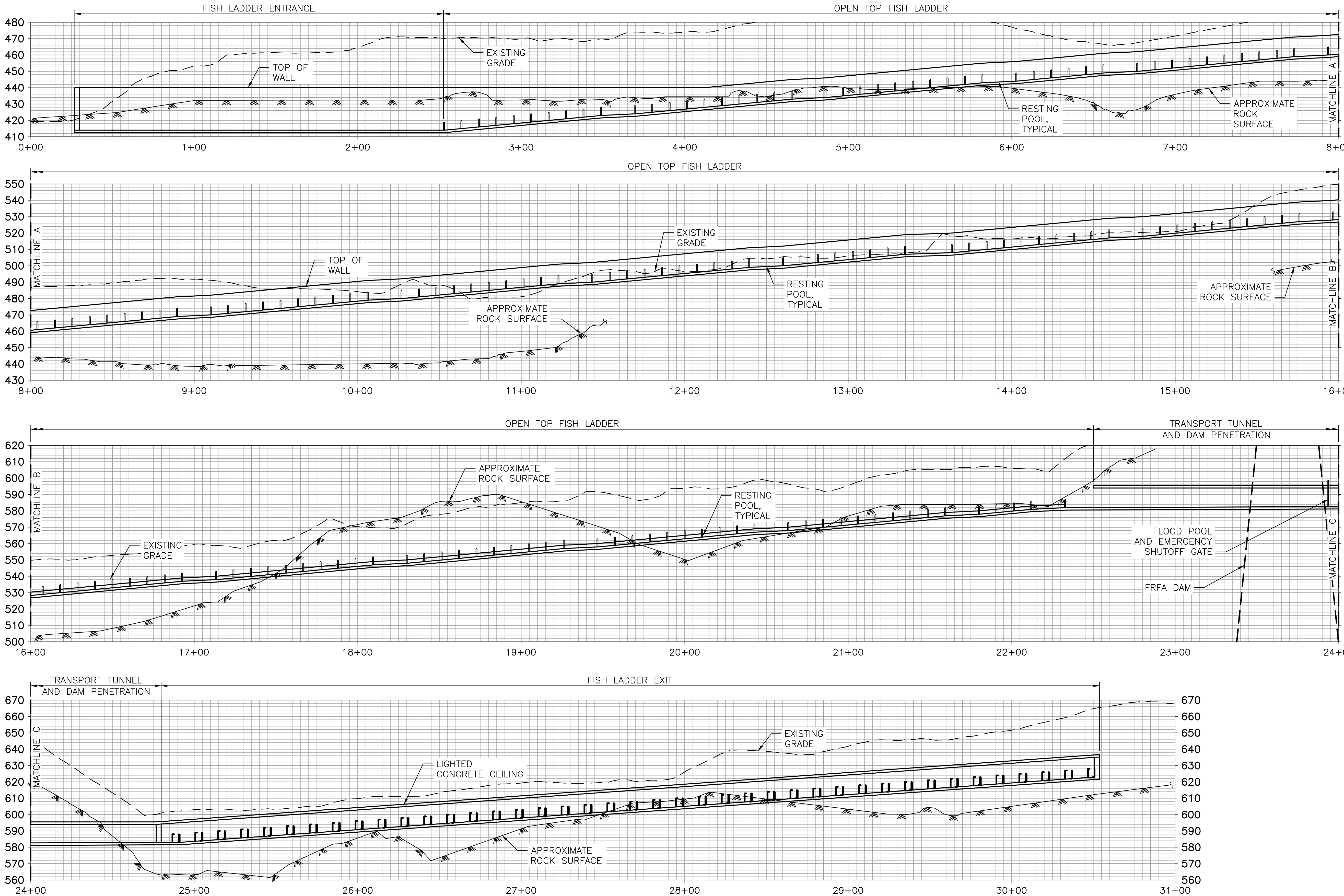
HDR

FRFA DAM
FISH LADDER PLAN

CHEHALIS BASIN DAM

DATE: JULY 2017

FIGURE: FRFA-C-2



30 0 30 60
SCALE IN FEET

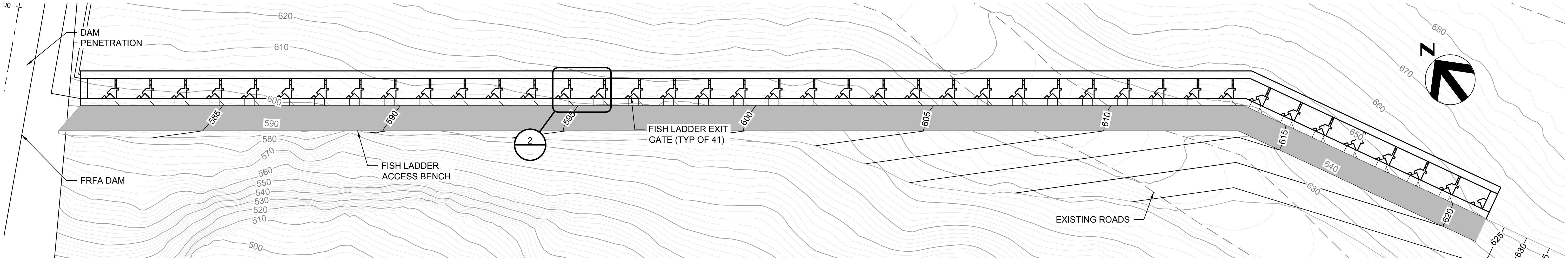
HDR

FRFA DAM
FISH LADDER PROFILE

CHEHALIS BASIN DAM

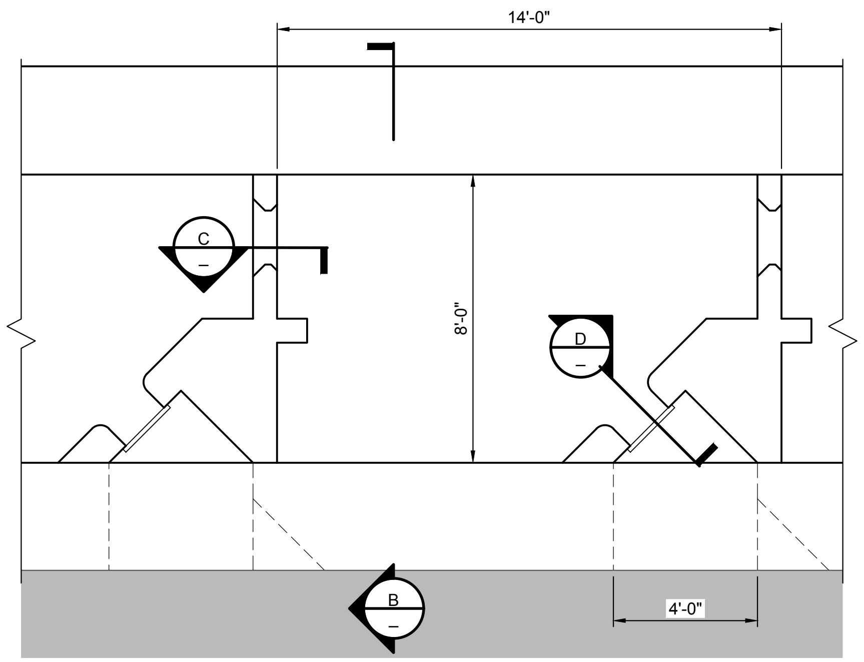
DATE
JULY 2017

FIGURE
FRFA-C-3



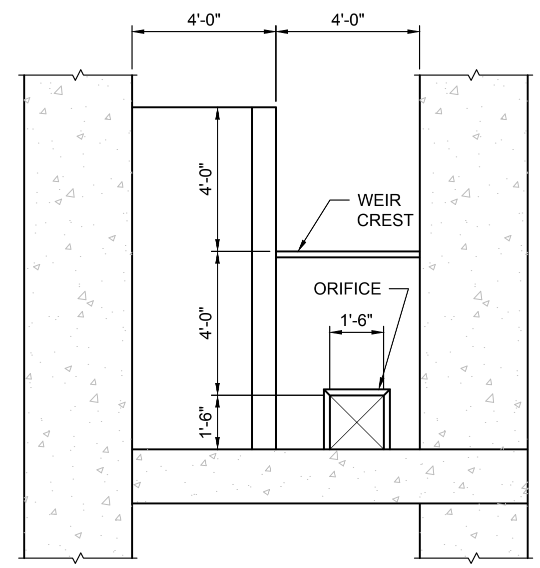
FISH LADDER EXIT (1, FRFA-C-2)

20 0 20 40
SCALE IN FEET



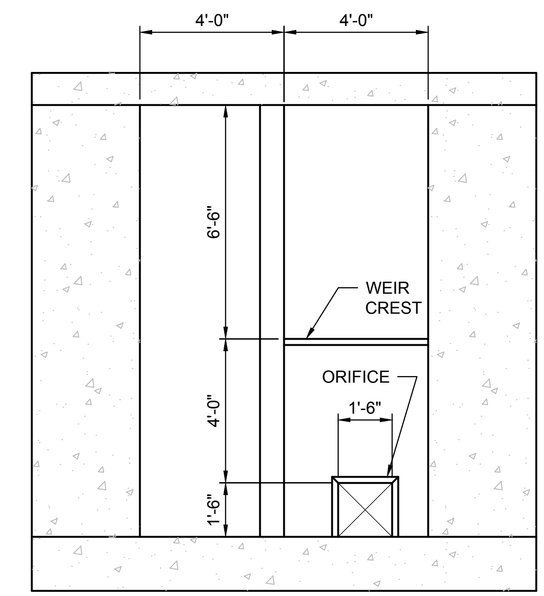
TYPICAL FISH LADDER EXIT POOL (2)

2 0 2 4 6
SCALE IN FEET



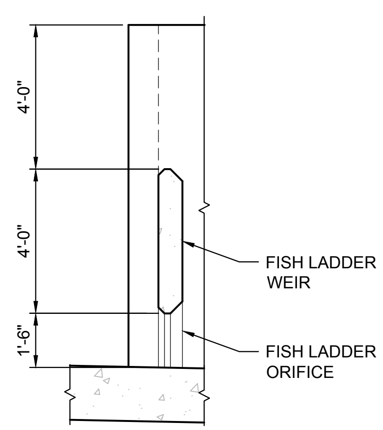
TYPICAL FISH LADDER WEIR ELEVATION (A, FRFA-C-2)

2 0 2 4 6
SCALE IN FEET



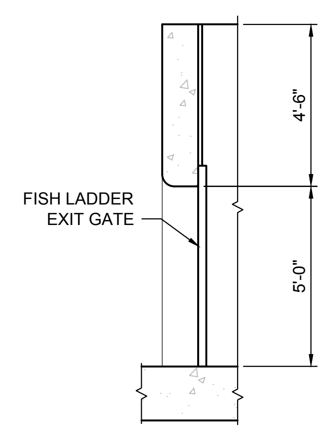
TYPICAL FISH LADDER EXIT ELEVATION (B)

2 0 2 4 6
SCALE IN FEET



2 0 2 4 6
SCALE IN FEET

TYPICAL FISH LADDER WEIR SECTION (C)



TYPICAL FISH LADDER EXIT PORT SECTION (D)

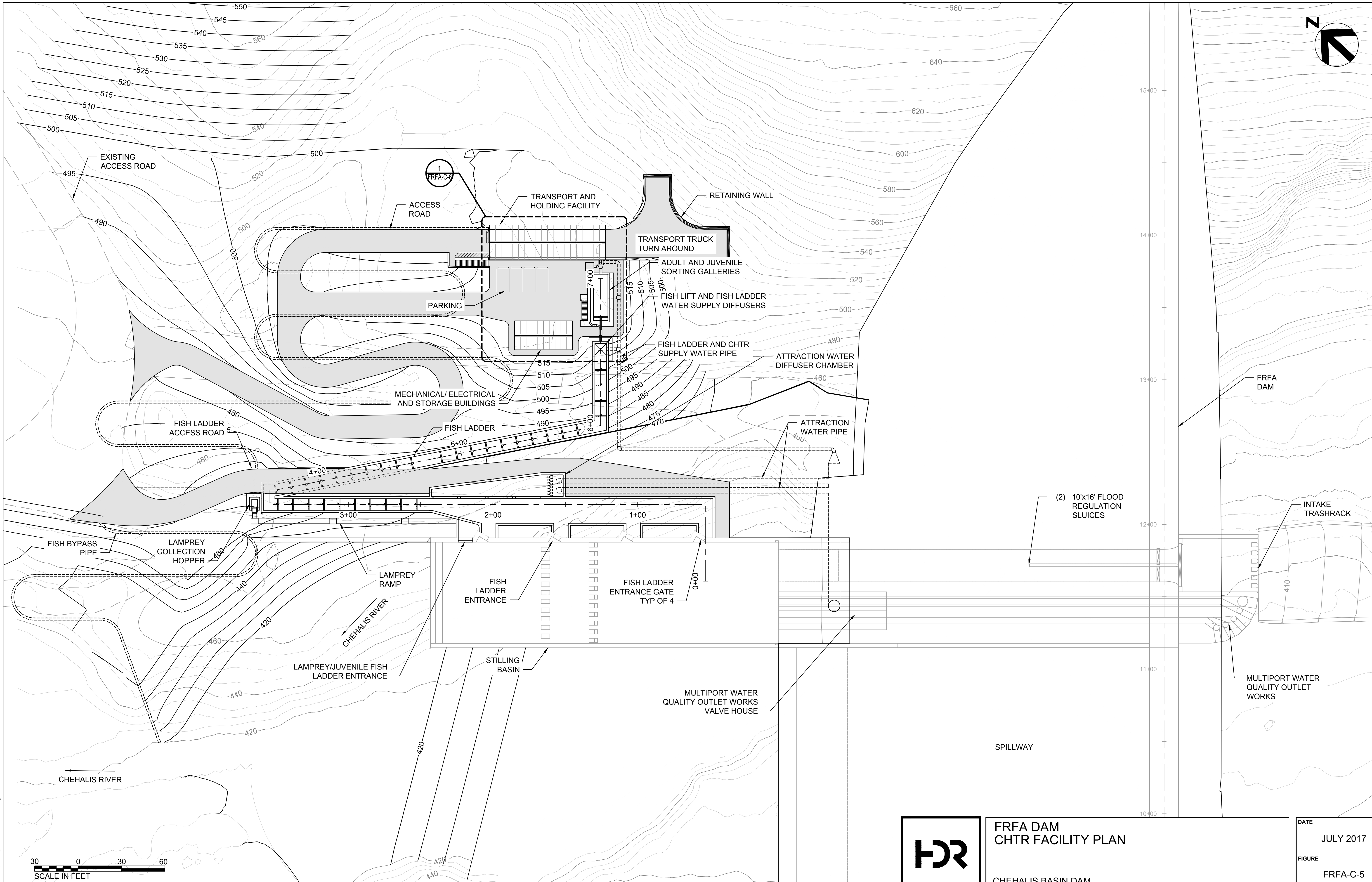
HDR

FRFA DAM
FISH LADDER DETAILS

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-C-4



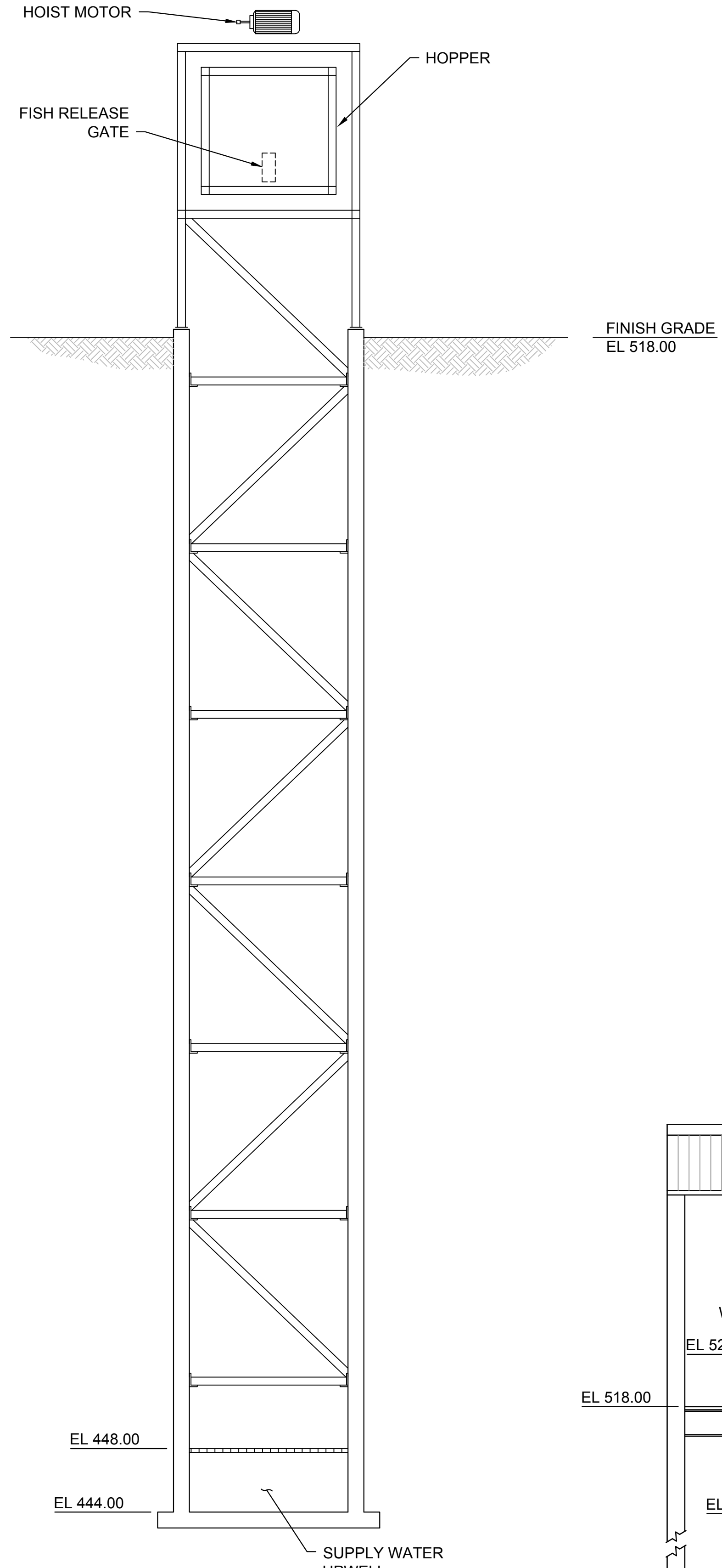
HDR

FRFA DAM
CHTR FACILITY PLAN

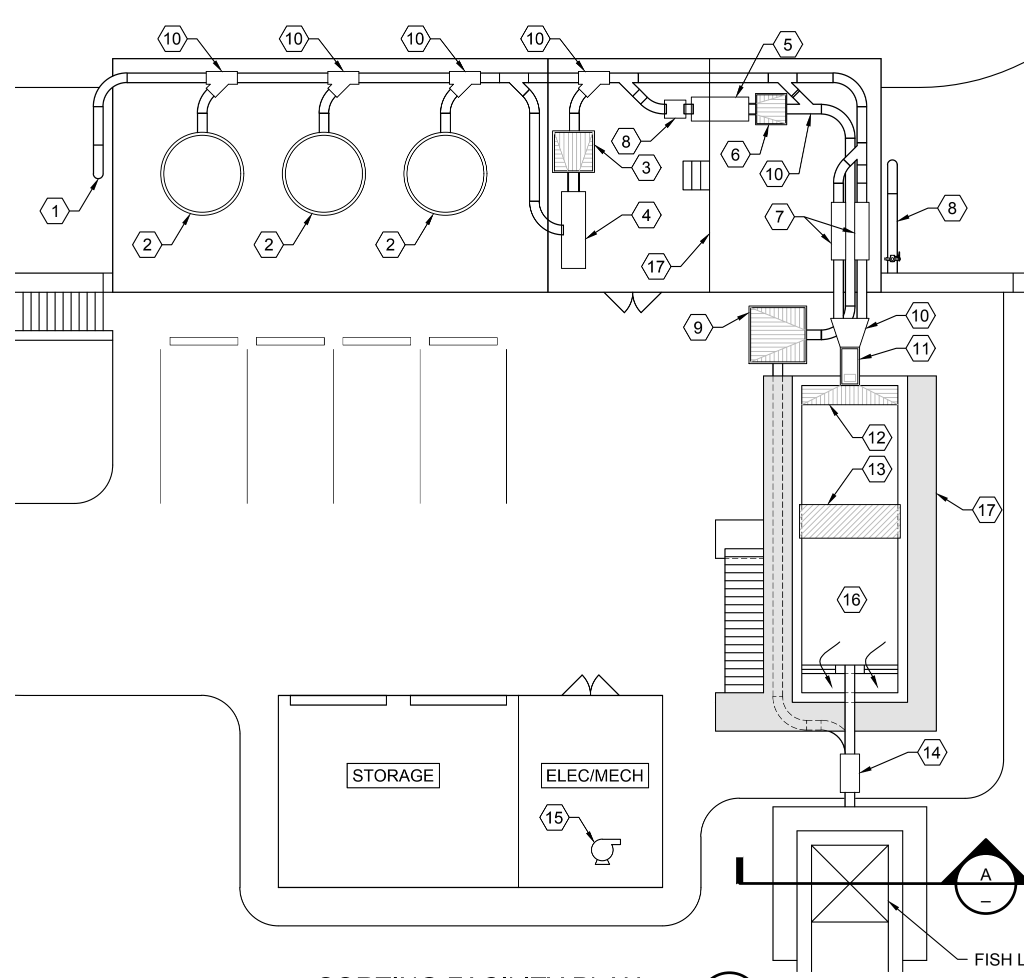
CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-C-5



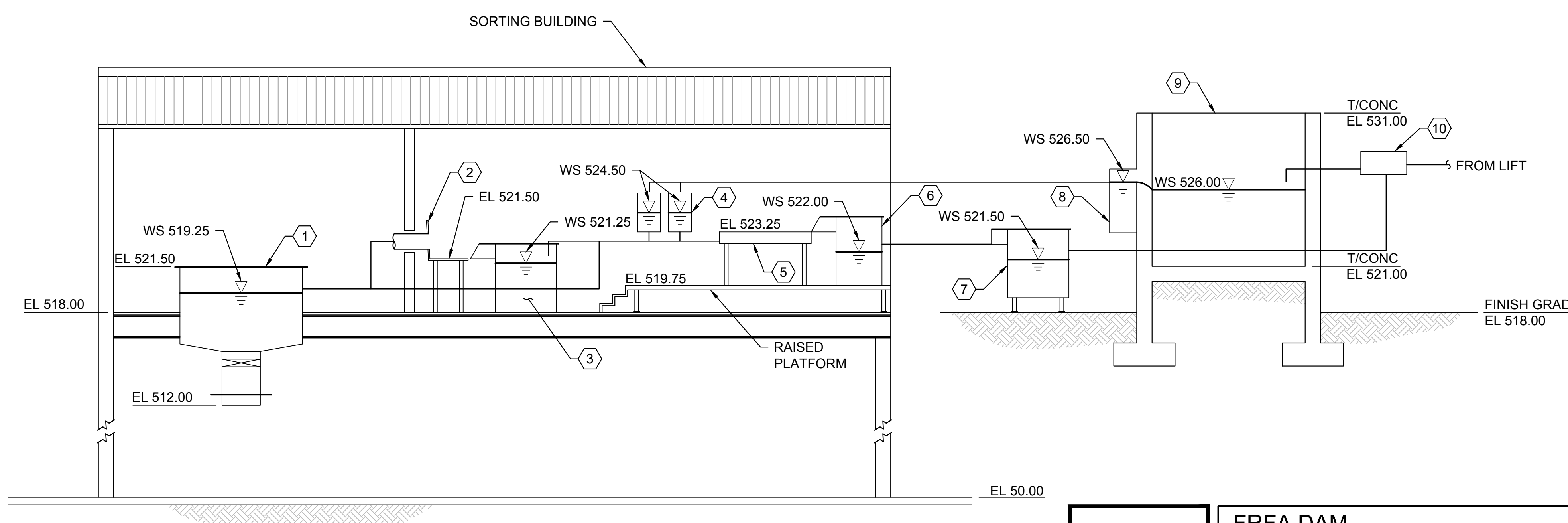
FISH LIFT
SCALE: NTS
A
–



SORTING FACILITY PLAN
SCALE: NTS
1
FRFA-C-5

KEYNOTES:

1. BYPASS PIPE
2. 8' DIA x 5' DEEP CIRCULAR HOLDING TANK
3. ADULT ANESTHESIA TANK WITH BRAIL
4. ADULT WORK-UP TABLE
5. JUVENILE WORK-UP TABLE
6. JUVENILE ANESTHESIA TANK WITH BRAIL
7. VISUAL INSPECTION TANKS
8. RECOVERY TANK
9. JUVENILE HOLDING GALLERY
10. DIVERTER GATE
11. FALSE WEIR
12. BRAIL
13. CROWDER
14. ADULT/JUVENILE GRADER
15. JW BOOSTER PUMP
16. ADULT HOLDING GALLERY
17. ELEVATED PLATFORM



KEYNOTES:

1. CIRCULAR HOLDING TANK
2. ADULT WORK-UP TABLE
3. ADULT ANESTHESIA TANK WITH BRAIL
4. VISUAL INSPECTION TANKS
5. JUVENILE WORK-UP TABLE
6. JUVENILE ANESTHESIA TANK WITH BRAIL
7. JUVENILE HOLDING GALLERY WITH BRAIL
8. FALSE WEIR
9. ADULT HOLDING GALLERY
10. GRADER

SORTING FACILITY
HYDRAULIC PROFILE
SCALE: NTS
2
–

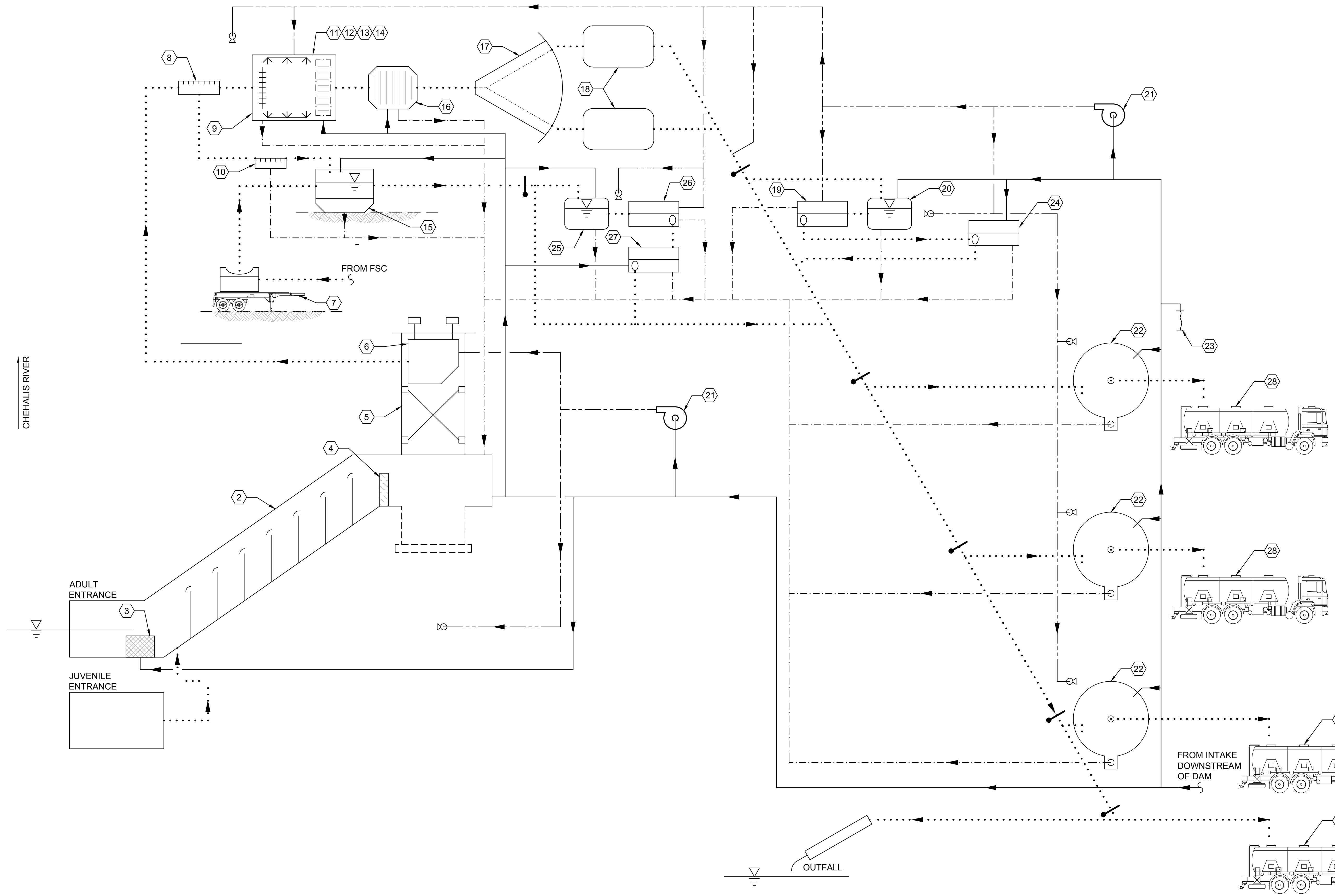
HDR

FRFA DAM
CHTR FACILITY PLAN AND ELEVATION

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-C-6



KEYNOTES:

1. ATTRACTION WATER SUPPLY
2. FISHWAY
3. AUXILIARY WATER SUPPLY
4. TRAP GATE
5. HOPPER LIFT TOWER
6. HOPPER BUCKET
7. JUVENILE TRANSPORT TANK FROM FSC
8. GRADER
9. ADULT HOLDING GALLERY
10. DEWATERING SCREEN
11. SPRINKLER SYSTEM
12. CROWDER
13. HOLDING GALLERY UPWELL
14. HOLDING GALLERY BRAIL
15. JUVENILE HOLDING GALLERY
16. FALSE WEIR
17. AUTOMATIC DIVERTER GATE
18. VISUAL INSPECTION TANKS
19. ADULT WORKUP TABLE
20. ADULT HOLDING TANK
21. JOB WATER BOOSTER PUMP
22. CIRCULAR HOLDING TANK
23. TRUCK FILL
24. ADULT RECOVERY TANK
25. JUVENILE HOLDING TANK
26. JUVENILE WORKUP TABLE
27. JUVENILE RECOVERY TABLE
28. ADULT/ JUVENILE TRANSPORT TRUCK

LEGEND:

FISH FLUME (FF)
DRAIN (DR)
WATER SUPPLY (WS)
JOB WATER (JW)
GATE
HOSE BIB

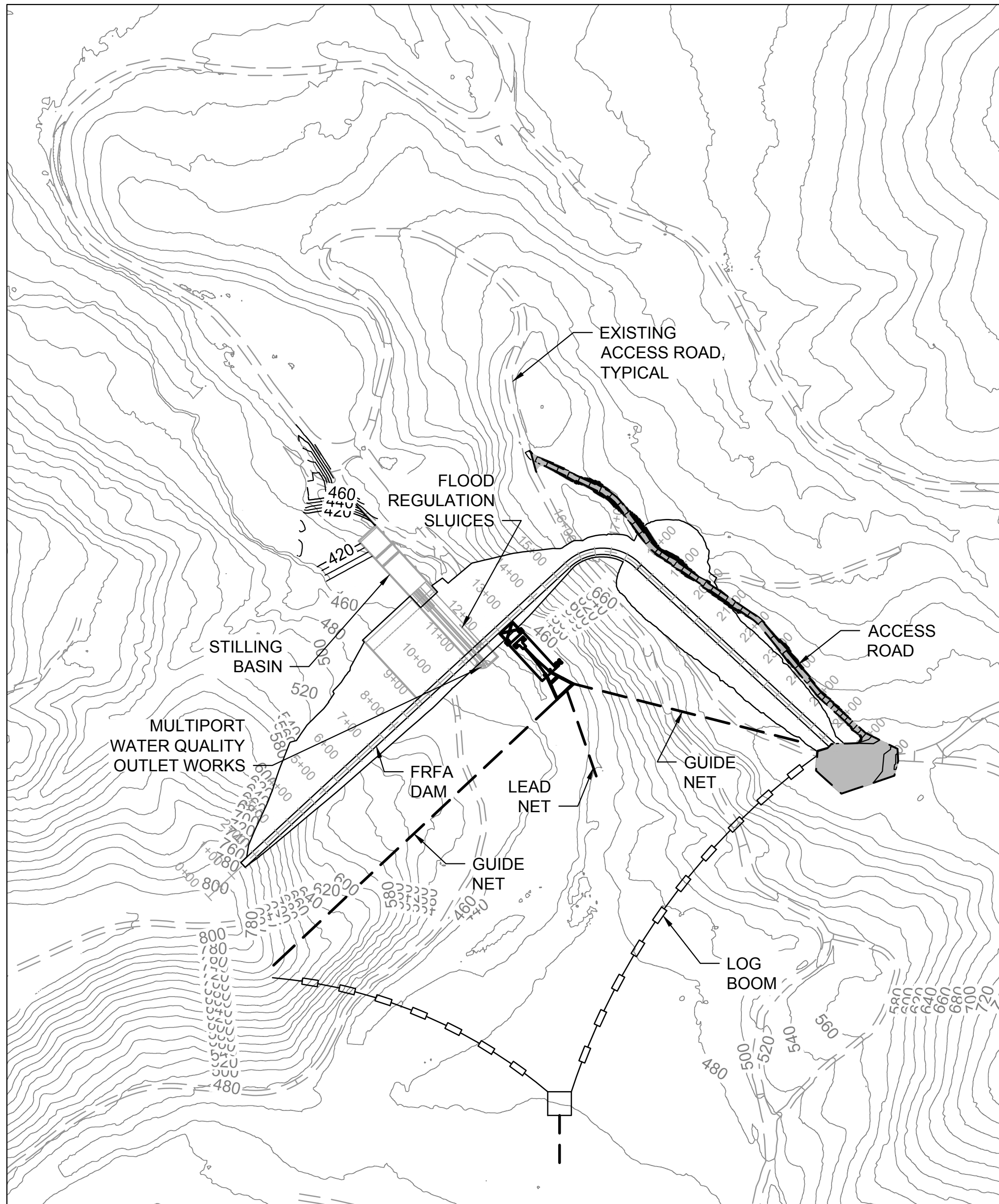
HDR

FRFA DAM
CHTR FACILITY PROCESS DIAGRAM

CHEHALIS BASIN DAM

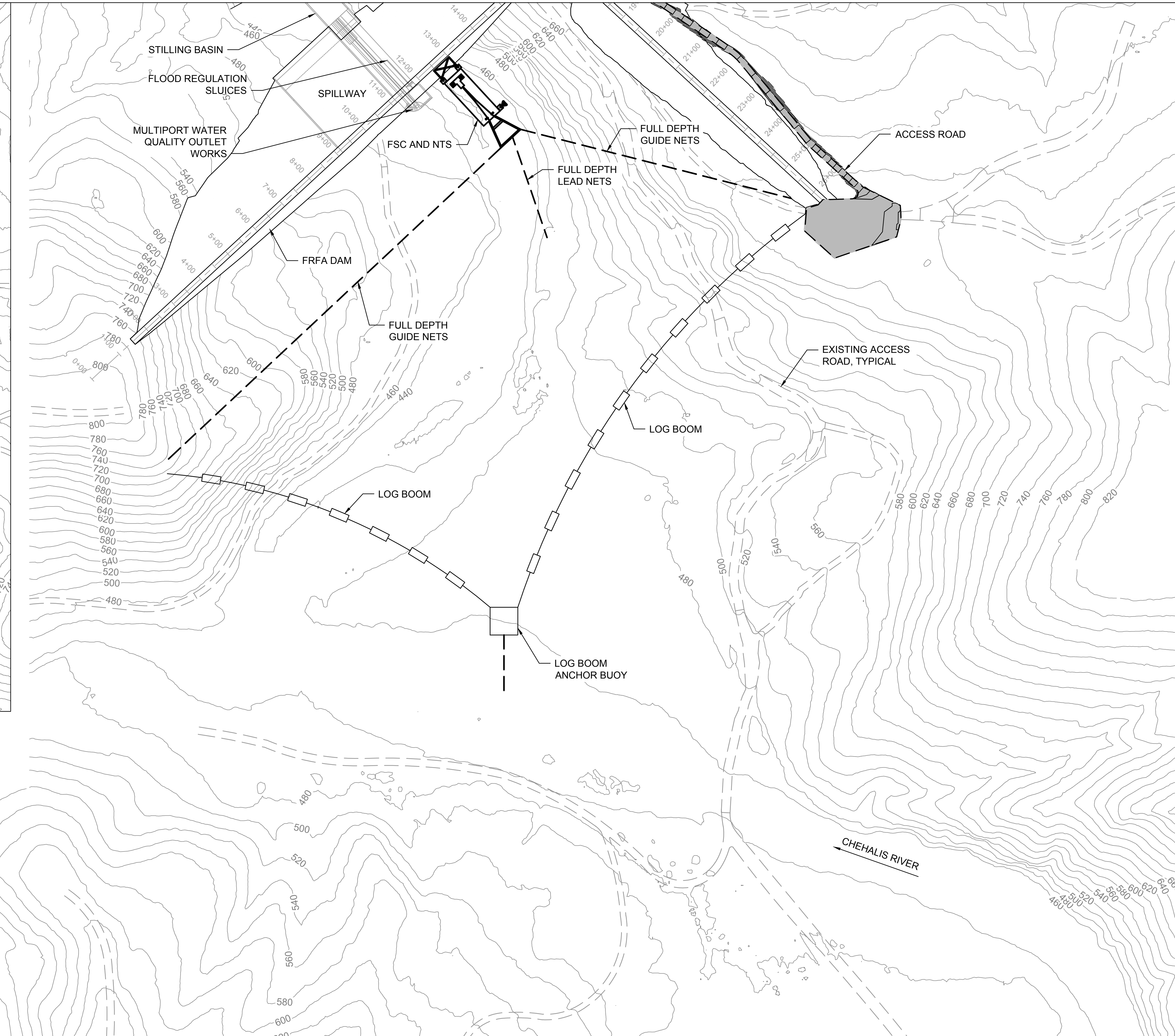
DATE
JULY 2017

FIGURE
FRFA-C-7



ACCESS PLAN

300 0 300 600
SCALE IN FEET



N

SITE PLAN

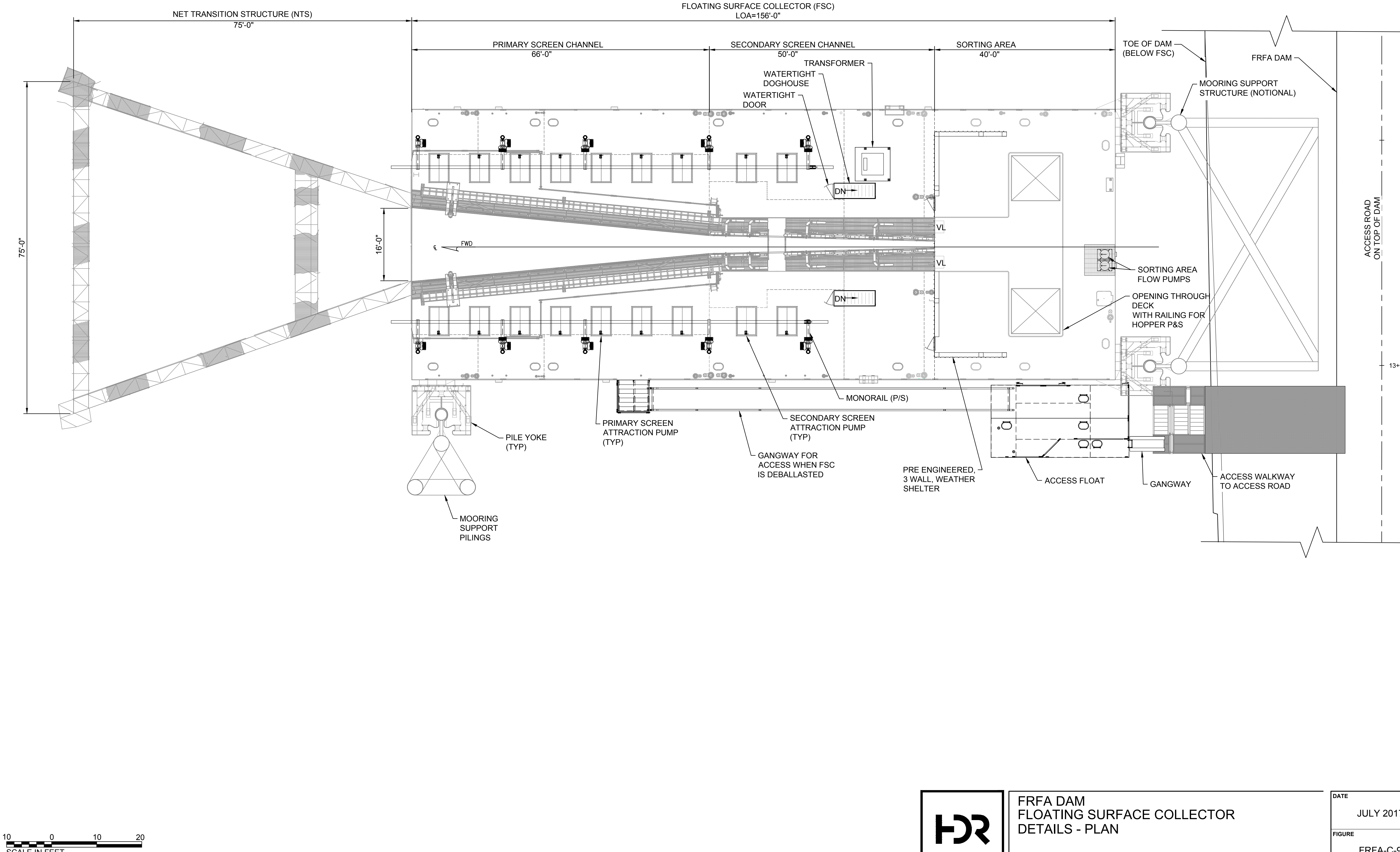
150 0 150 300
SCALE IN FEET

FRFA DAM
FLOATING SURFACE COLLECTOR
AND ACCESS PLAN

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-C-8



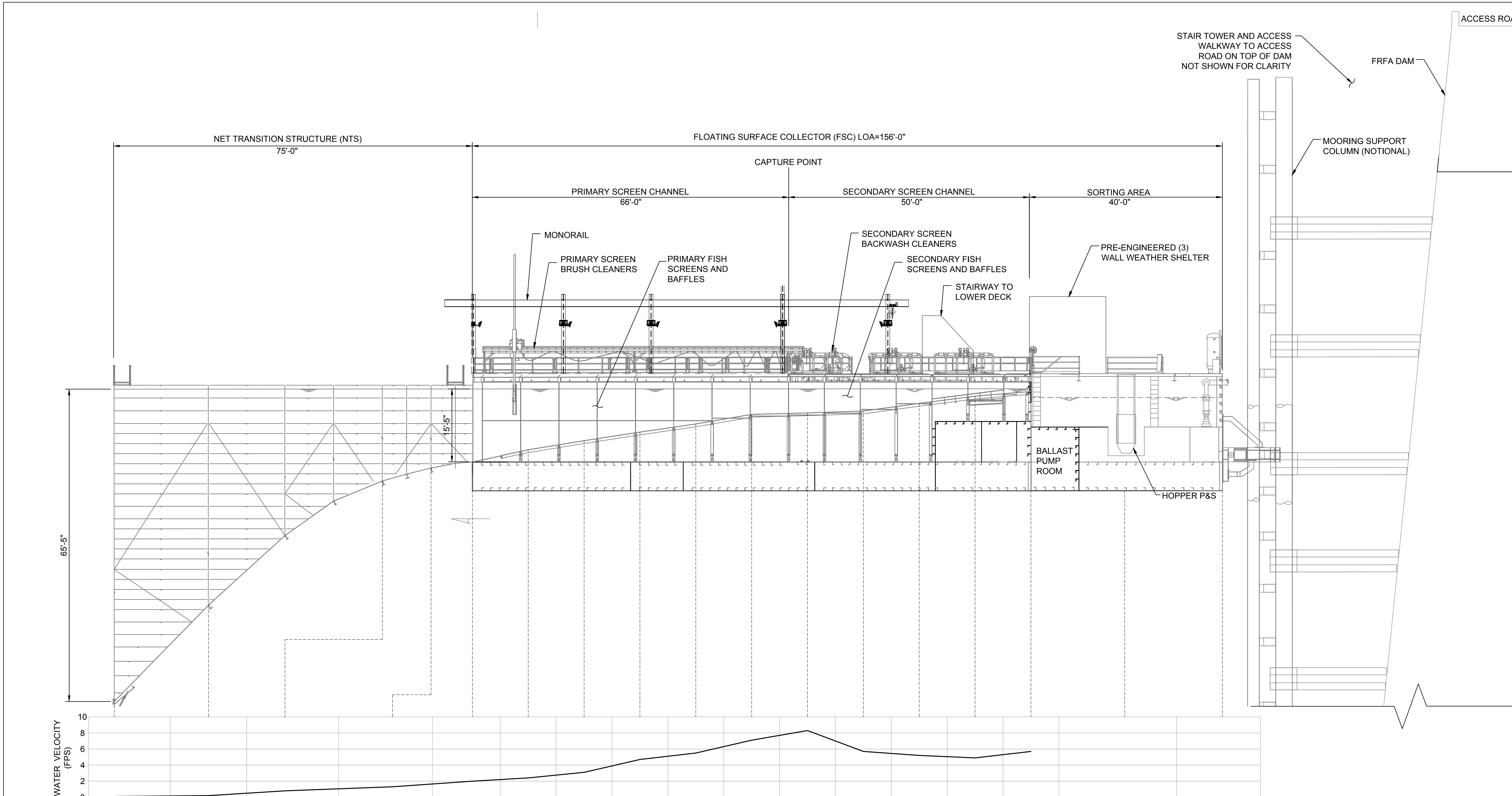
HDR

FRFA DAM
FLOATING SURFACE COLLECTOR
DETAILS - PLAN

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-C-9



HYDRAULIC DATA - BASE CASE																	
FLOW (CFS)	500	500	500	500	500	404	321	250	191	145	105	68	38	17	6	-	-
WIDTH (FT)	75	63.2	31.7	23.9	16.0	12.5	8.9	5.4	4.3	3.3	2.2	2.2	1.8	1.5	1.1	-	-
DEPTH (FT)	65.5	43.4	19.4	16.7	15.4	13.6	11.7	9.9	8.1	6.2	5.9	5.5	3.9	2.4	1.0	~ 5	~ 5

NOTES:

1. FLOATING SURFACE COLLECTOR AND NET TRANSITION STRUCTURE SHOWN AT MAX. NORMAL OPERATING RESERVOIR ELEVATION.

10 0 10 20
SCALE IN FEET

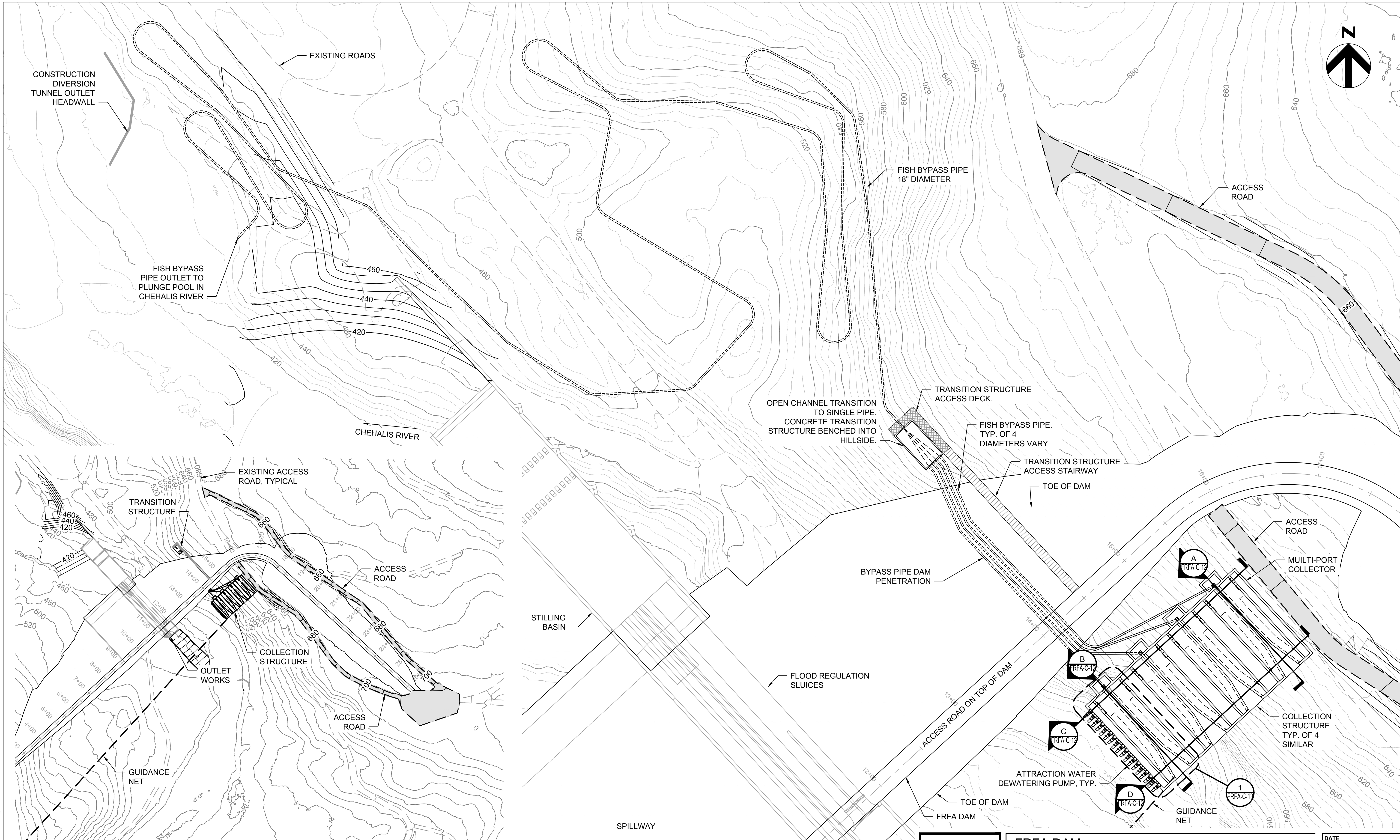
HDR

FRFA DAM
FLOATING SURFACE COLLECTOR PROFILE

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-C-10



ACCESS PLAN

200 0 200 400
SCALE IN FEET

SITE PLAN

40 0 40 80
SCALE IN FEET

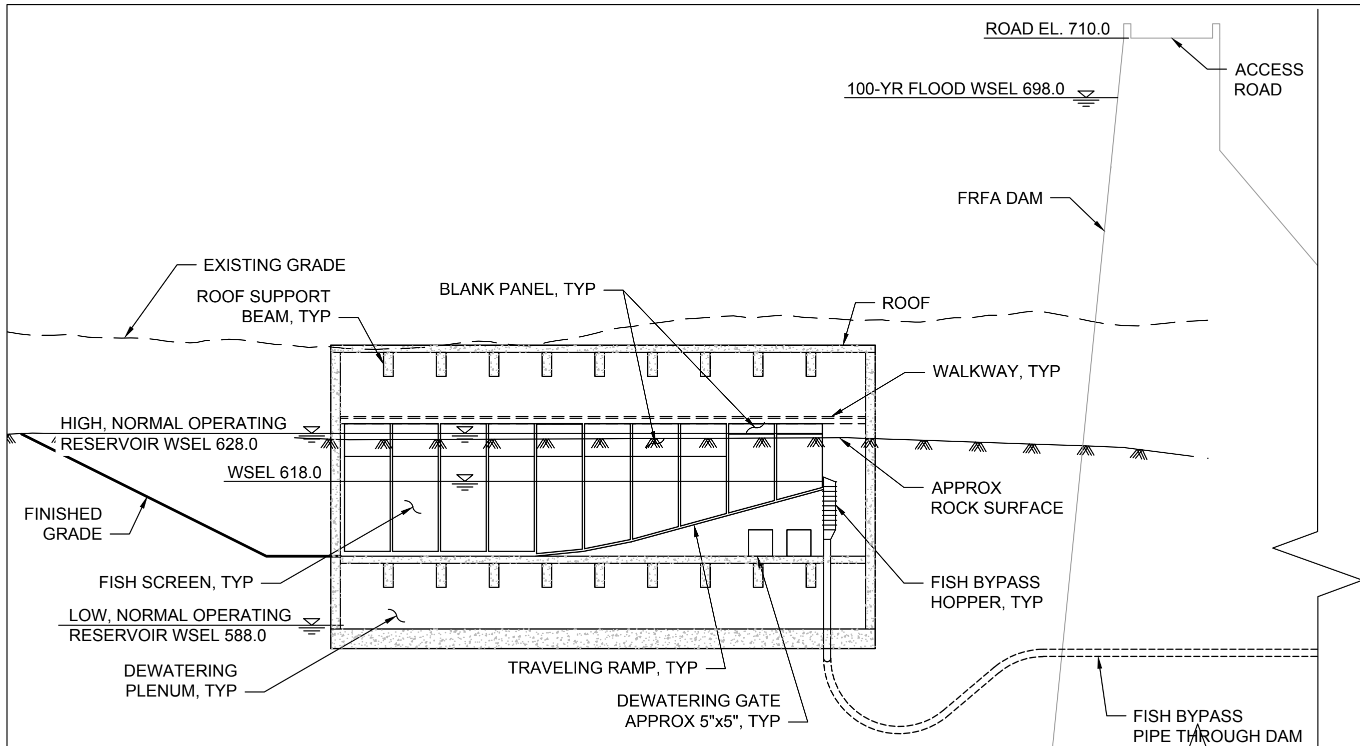
HDR

FRFA DAM
MULTI-PORT COLLECTOR PLAN

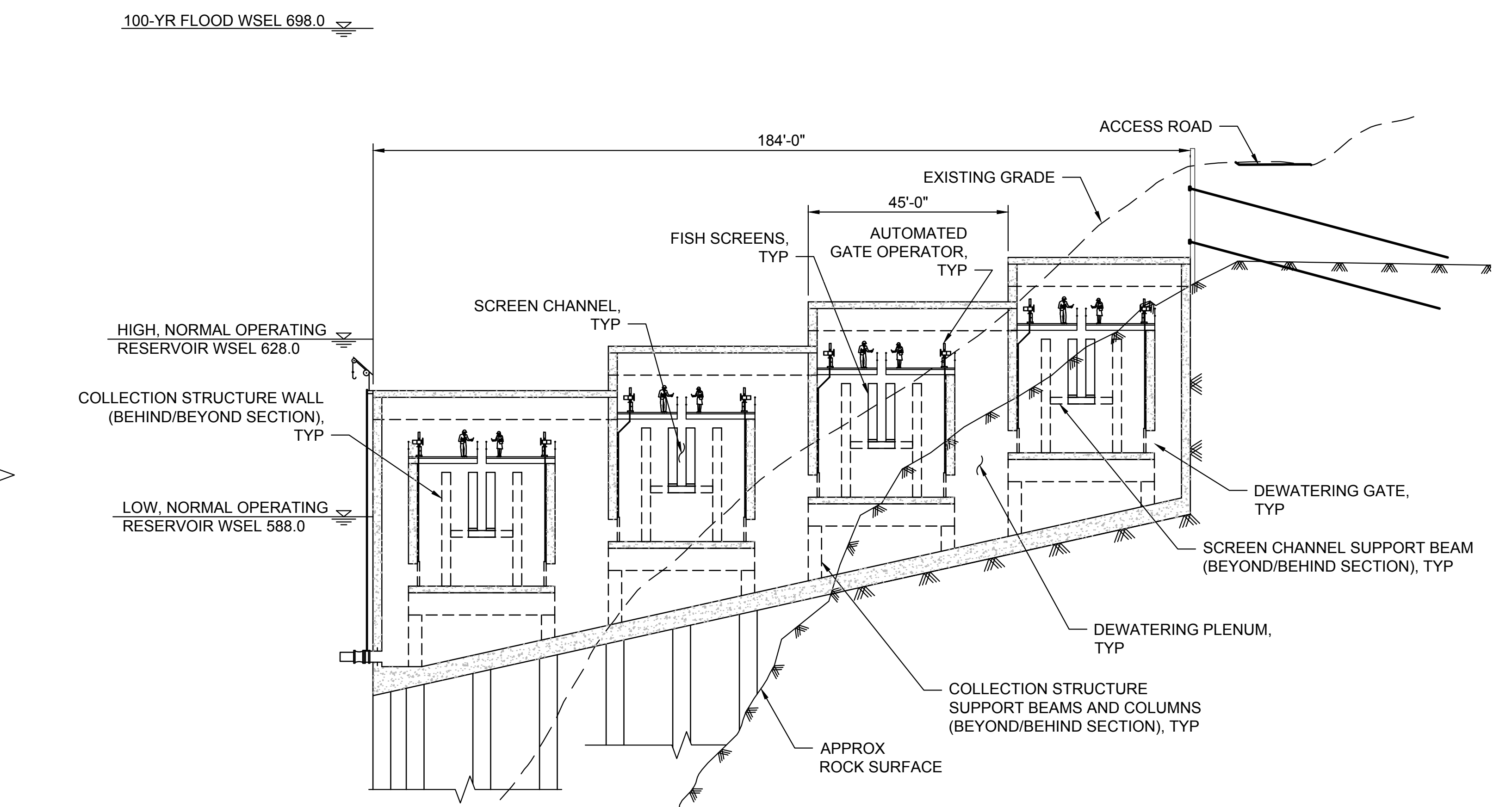
CHEHALIS BASIN DAM

DATE
JULY 2017

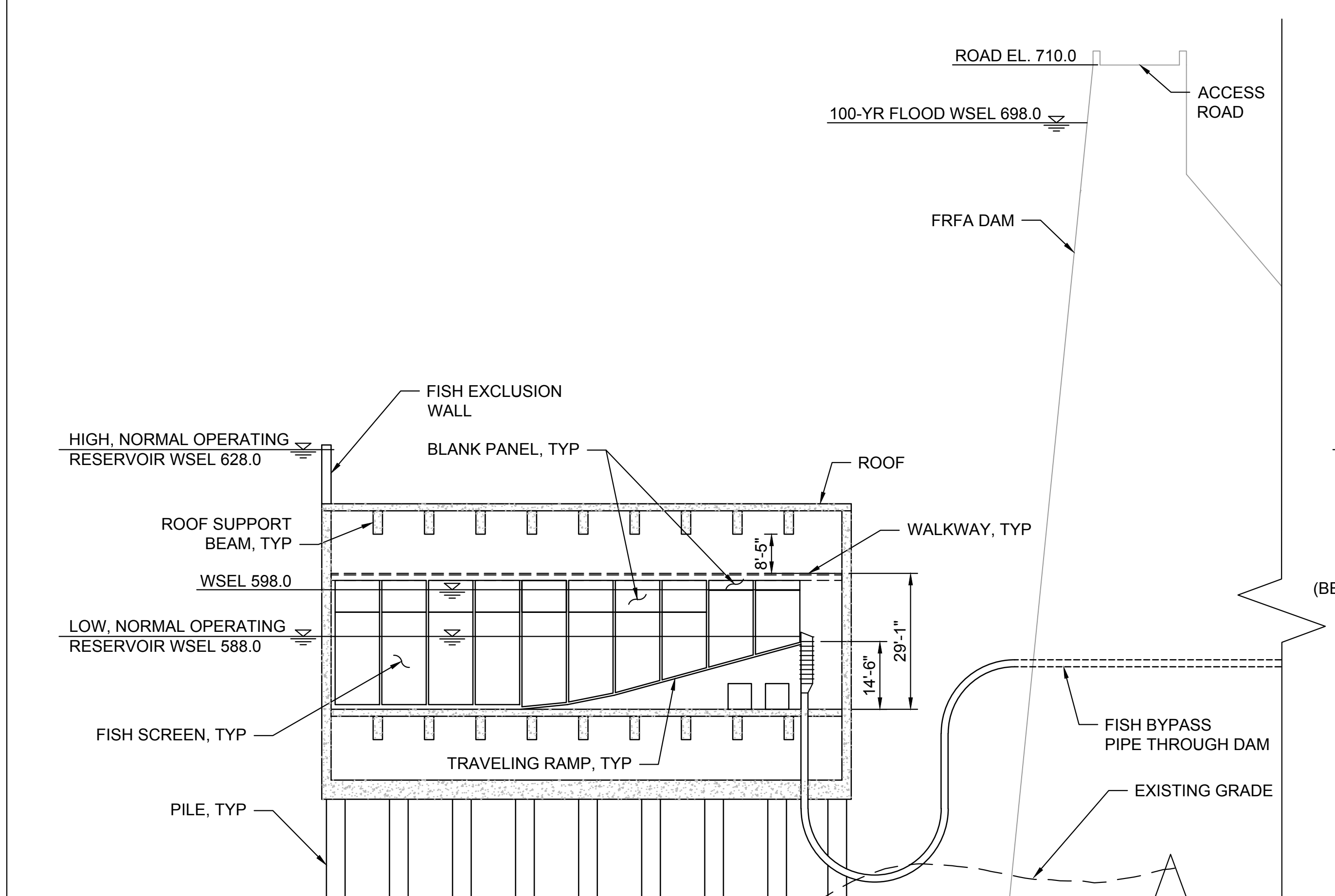
FIGURE
FRFA-C-11



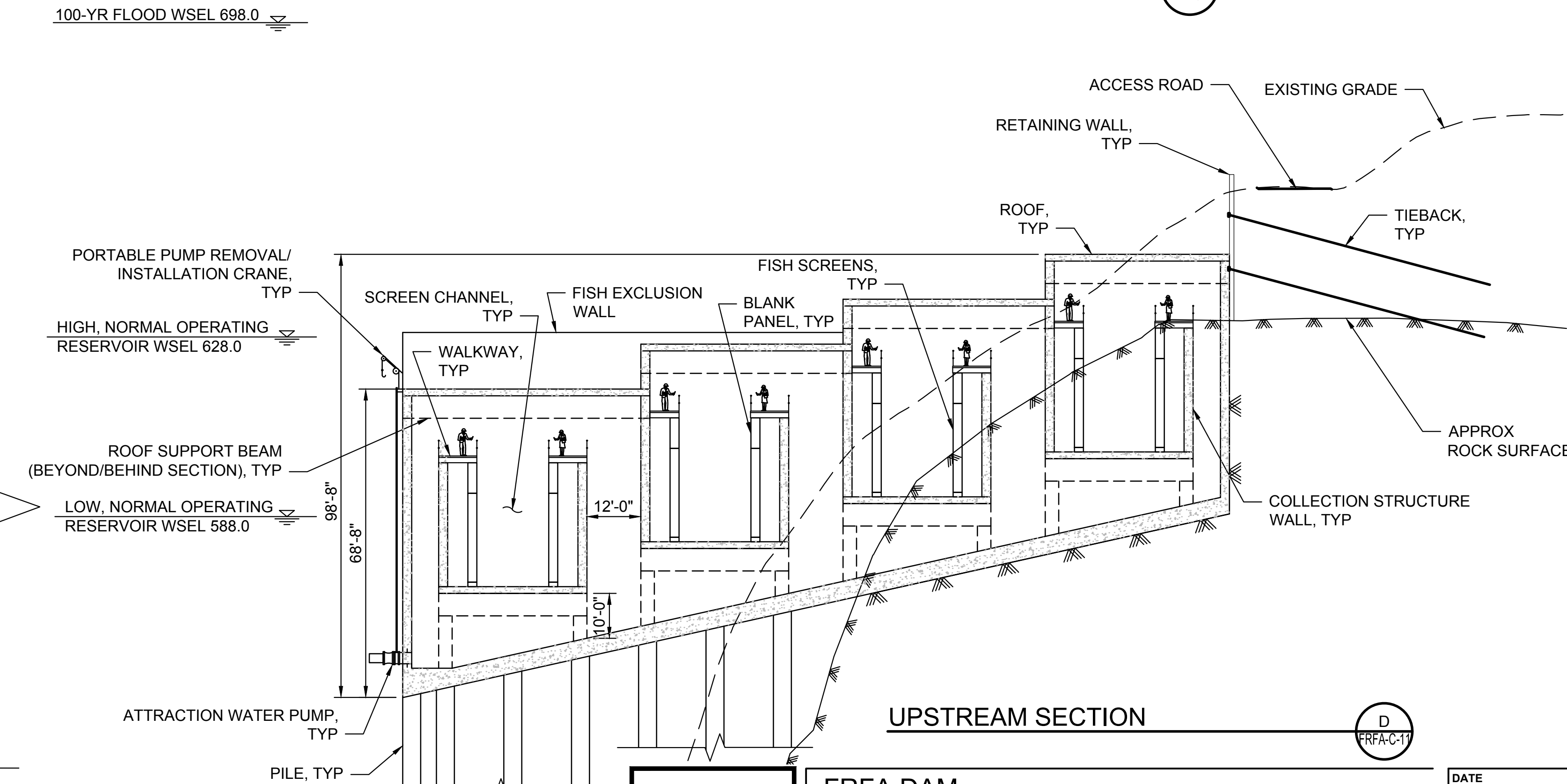
HIGH RESERVOIR COLLECTOR
RESERVOIR WSEL 618-628 - PROFILE (A / FRFA-C-11)



DOWNSTREAM SECTION (C / FRFA-C-11)



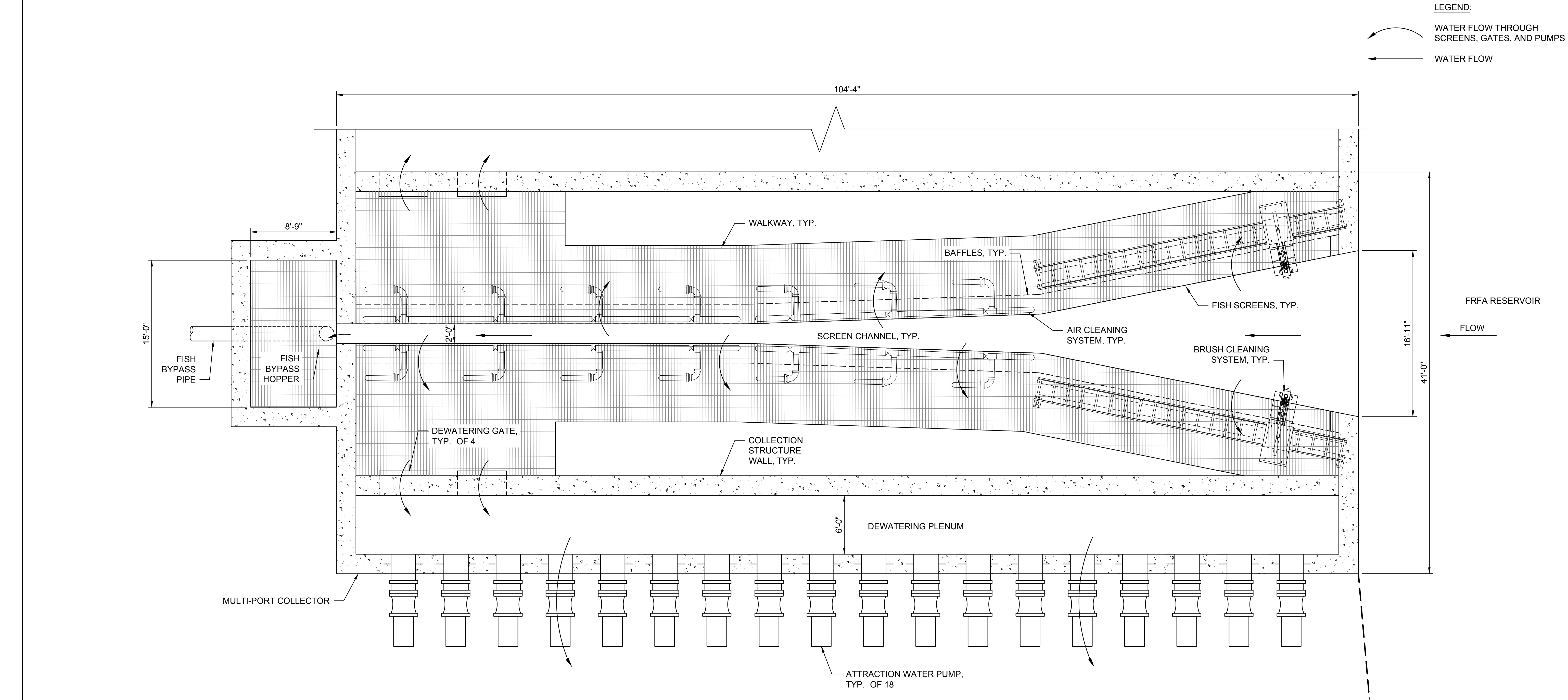
LOW RESERVOIR COLLECTOR
RESERVOIR WSEL 588-598 - PROFILE (B / FRFA-C-11)



UPSTREAM SECTION (D / FRFA-C-11)

20 0 20 40
SCALE IN FEET

HDR	FRFA DAM MULTI-PORT COLLECTOR SECTIONS CHEHALIS BASIN DAM	DATE JULY 2017 FIGURE FRFA-C-12



COLLECTION STRUCTURE DETAIL - PLAN

1
FRFA-C-11

5 0 5 10
SCALE IN FEET

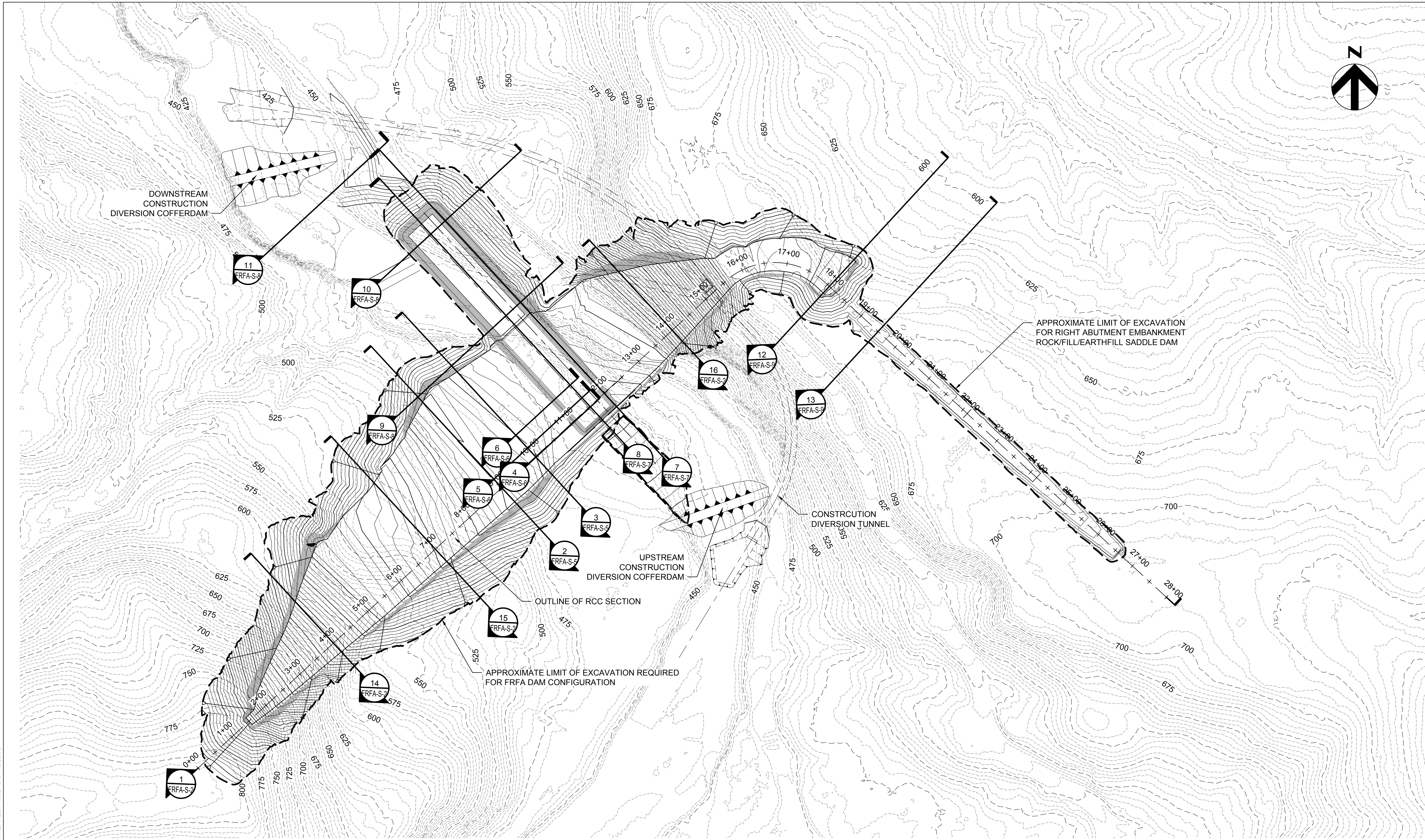
HDR

FRFA DAM
MULTI-PORT COLLECTOR DETAILS

CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-C-13



DAM EXCAVATION PLAN

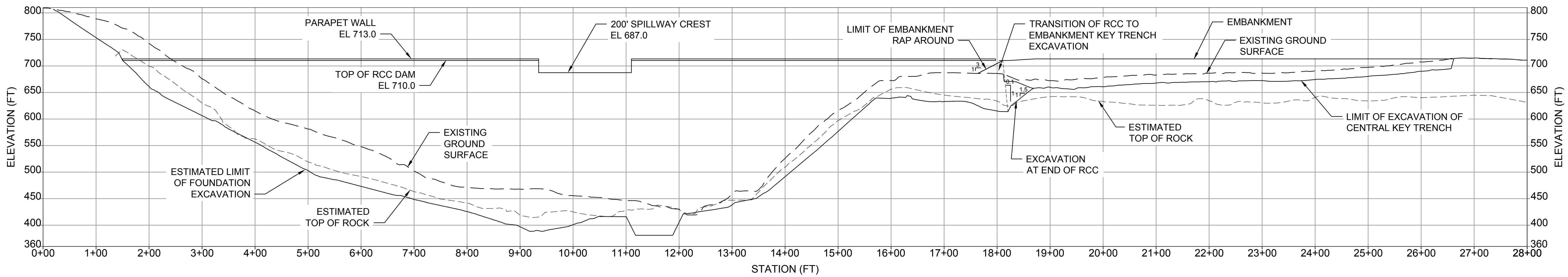
0 50 100 200
SCALE IN FEET

FRFA DAM
DAM EXCAVATION AND
FOUNDATION TREATMENT PLAN

CHEHALIS BASIN DAM

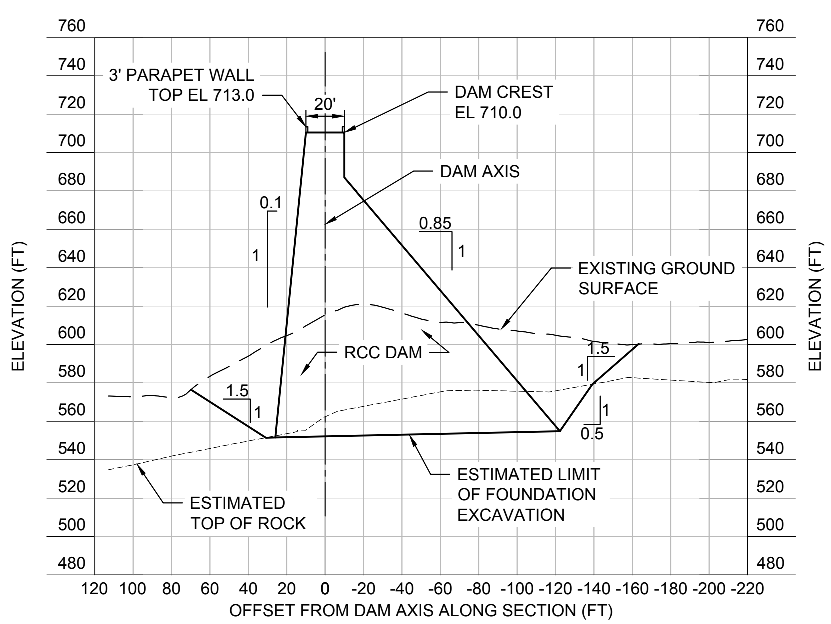
DATE
JULY 2017

FIGURE
FRFA-S-1



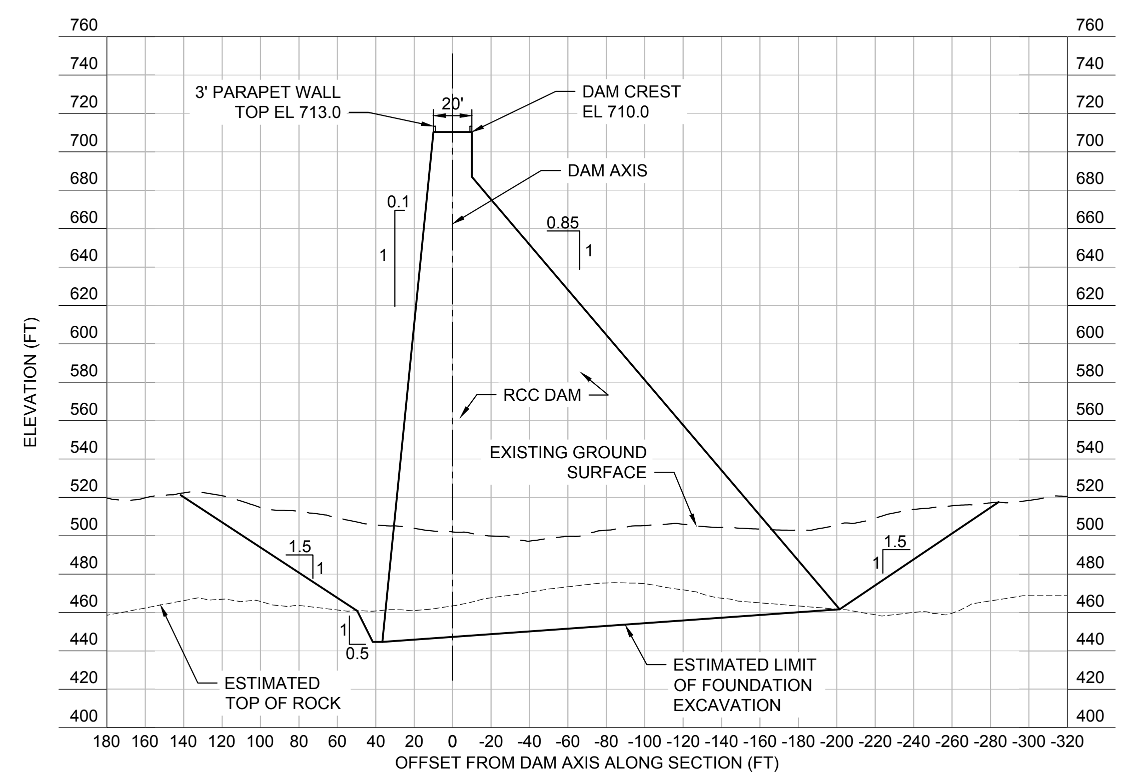
DAM EXCAVATION PROFILE (1 / FRFA-S-1)

0 50 100 200
SCALE IN FEET



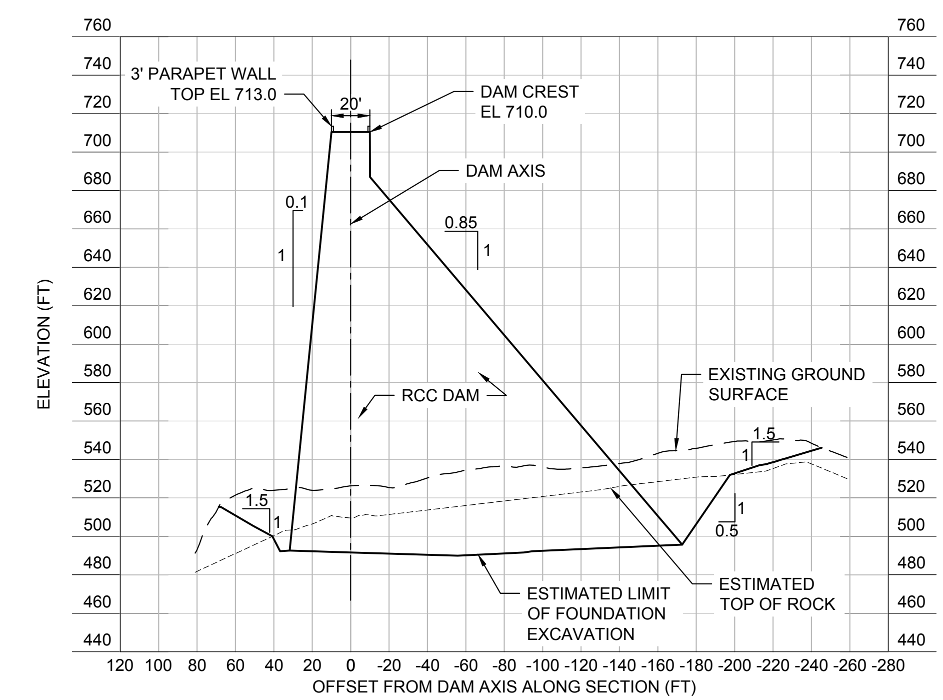
DAM EXCAVATION TYPICAL SECTION (14 / FRFA-S-1)

0 25 50 100
SCALE IN FEET



DAM EXCAVATION TYPICAL SECTION (15 / FRFA-S-1)

0 25 50 100
SCALE IN FEET



DAM EXCAVATION TYPICAL SECTION (16 / FRFA-S-1)

0 25 50 100
SCALE IN FEET

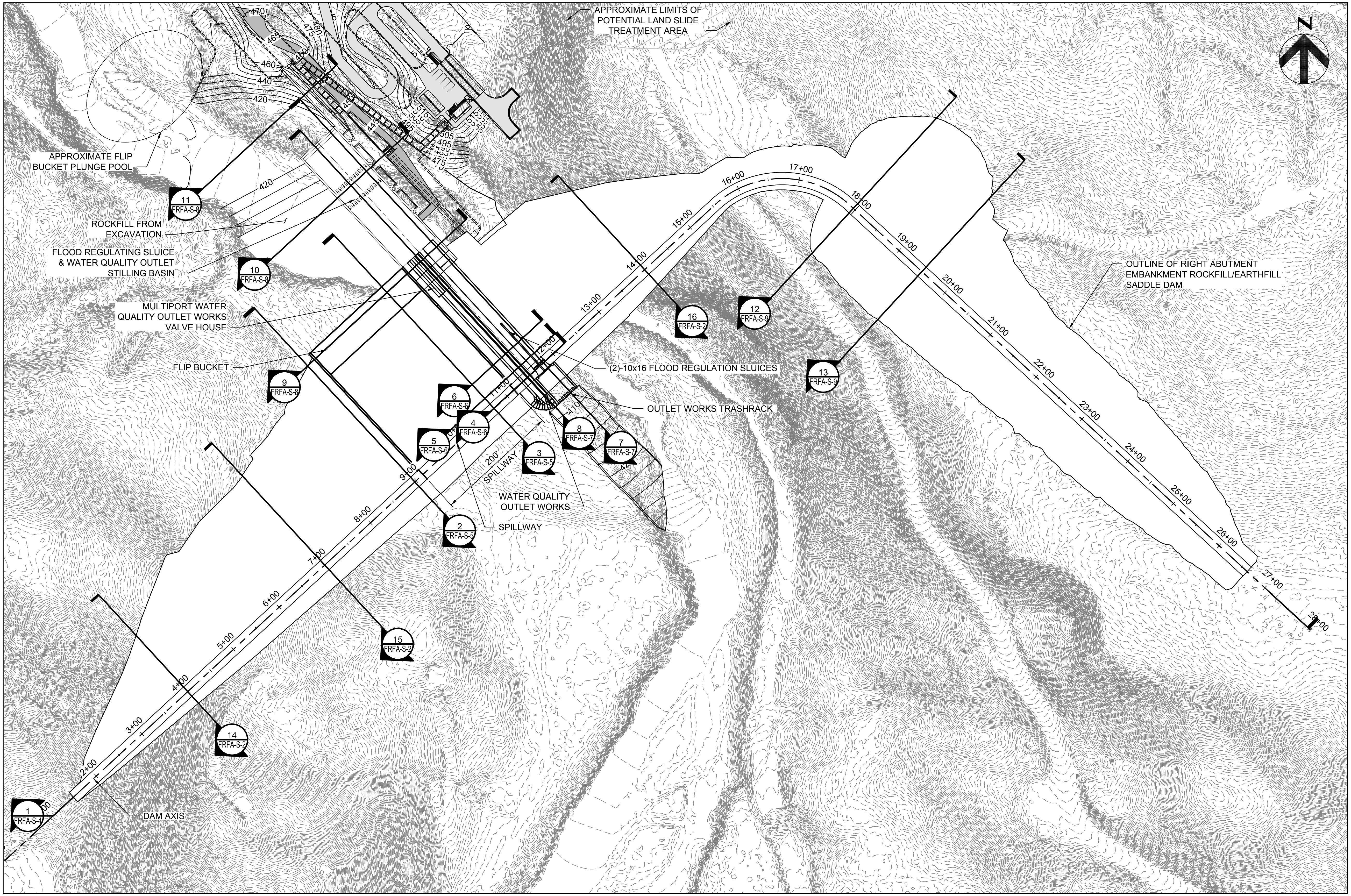
HDR

FRFA DAM
DAM EXCAVATION PROFILE
AND TYPICAL SECTIONS 14, 15 and 16

CHEHALIS BASIN DAM

DATE: JULY 2017

FIGURE: FRFA-S-2



PLAN

0 40 80 160
SCALE IN FEET

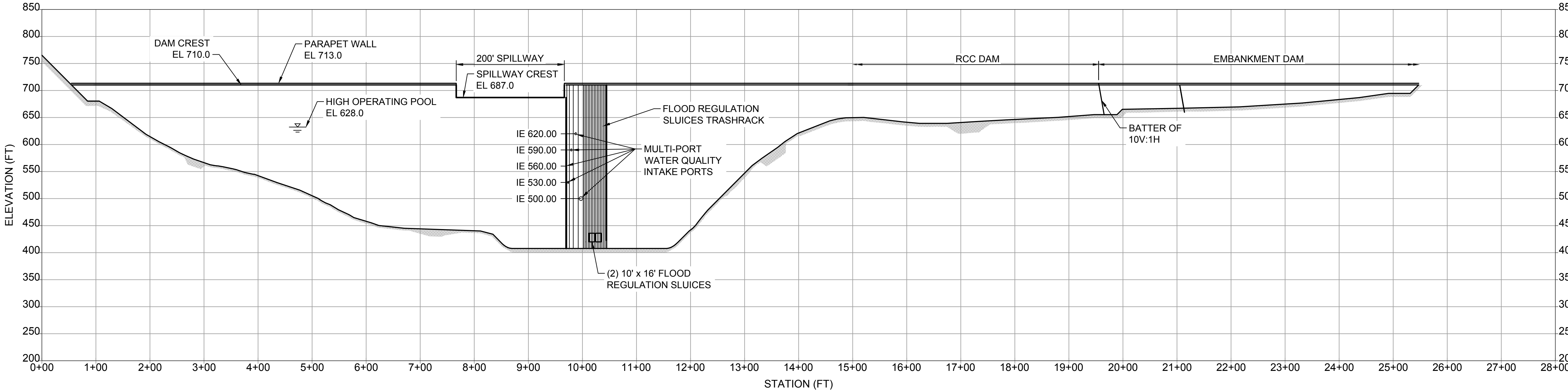
HDR

FRFA DAM
DAM PLAN

CHEHALIS BASIN DAM

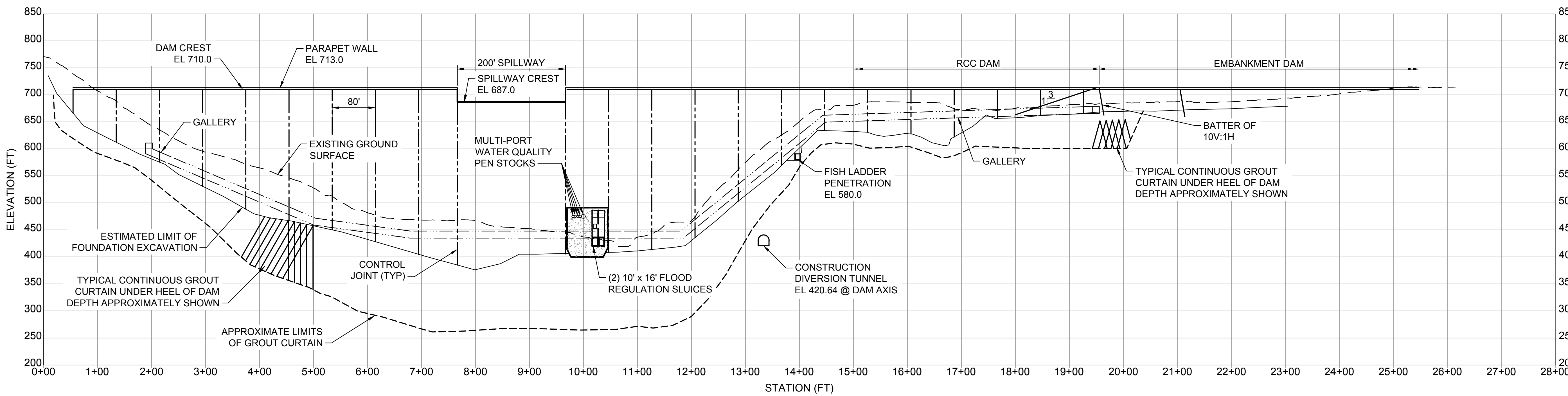
DATE
JULY 2017

FIGURE
FRFA-S-3



DAM ELEVATION VIEW (1 / FRFA-S-3)

0 50 100 200
SCALE IN FEET



SECTION ALONG DAM AXIS (1 / FRFA-S-3)

0 50 100 200
SCALE IN FEET

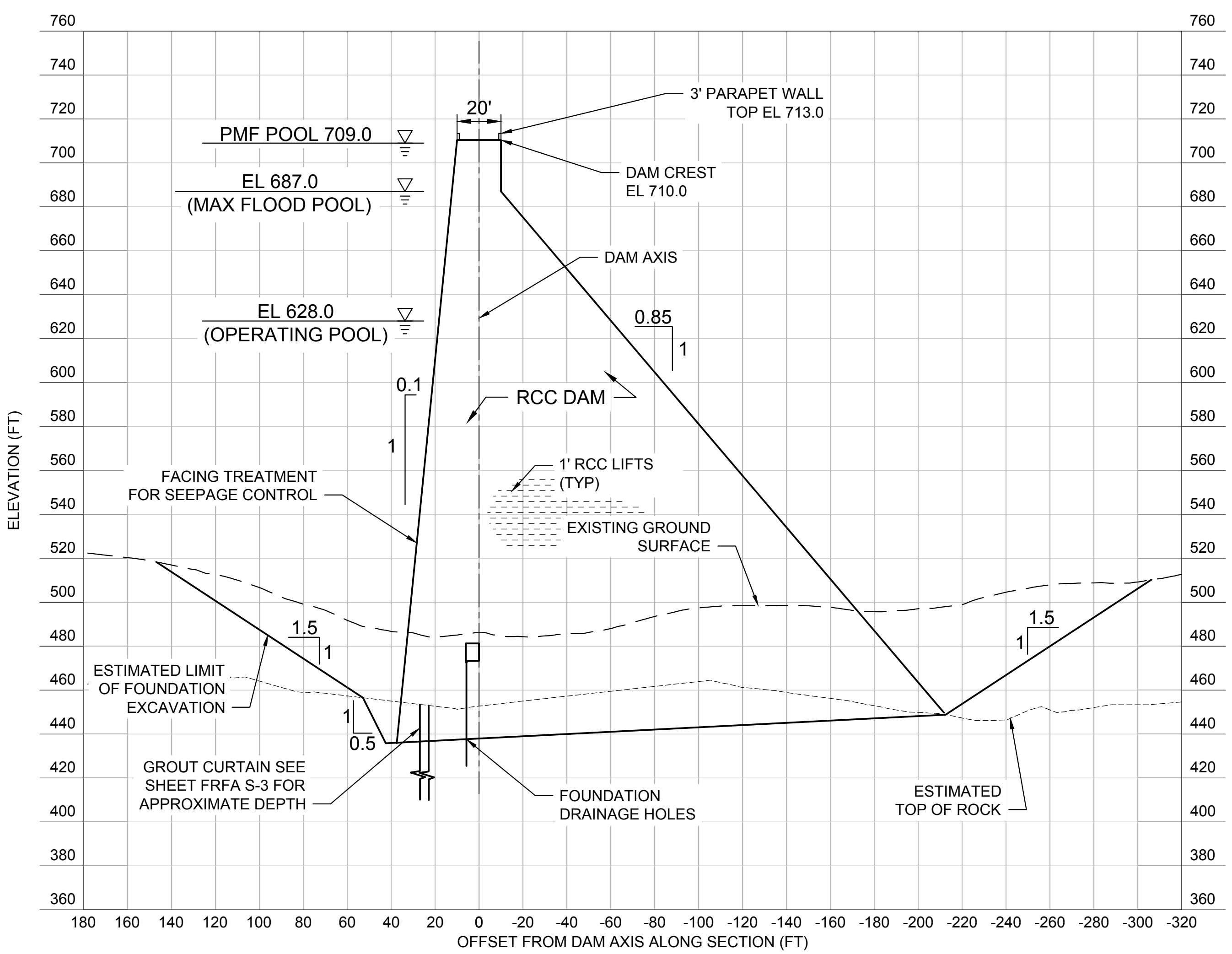
HDR

FRFA DAM
DAM ELEVATION VIEW
AND AXIS SECTION 1

CHEHALIS BASIN DAM

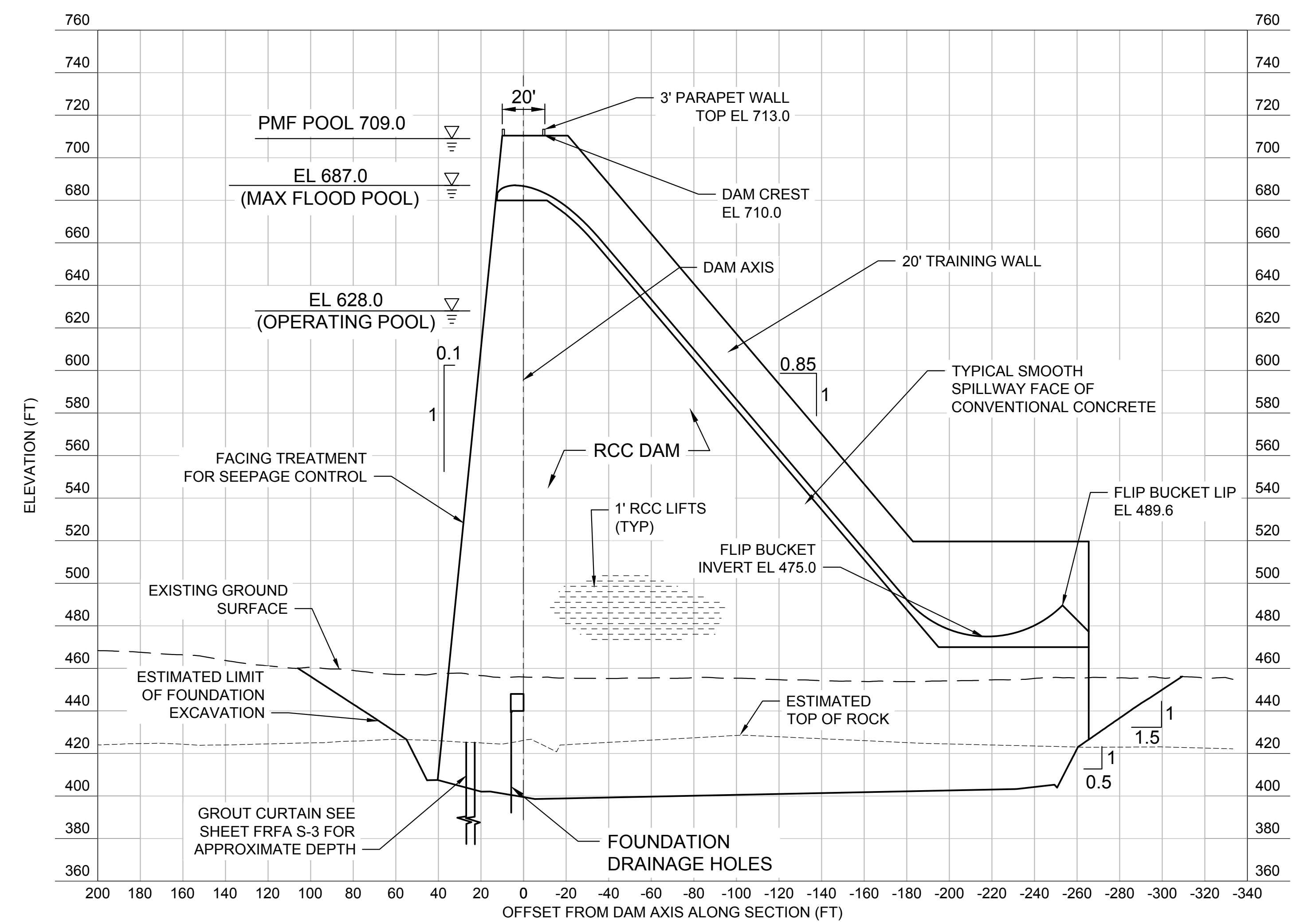
DATE: JULY 2017

FIGURE: FRFA-S-4



SECTION 2 / FRFA-S-3

0 20 40 80
SCALE IN FEET



SECTION 3 / FRFA-S-3

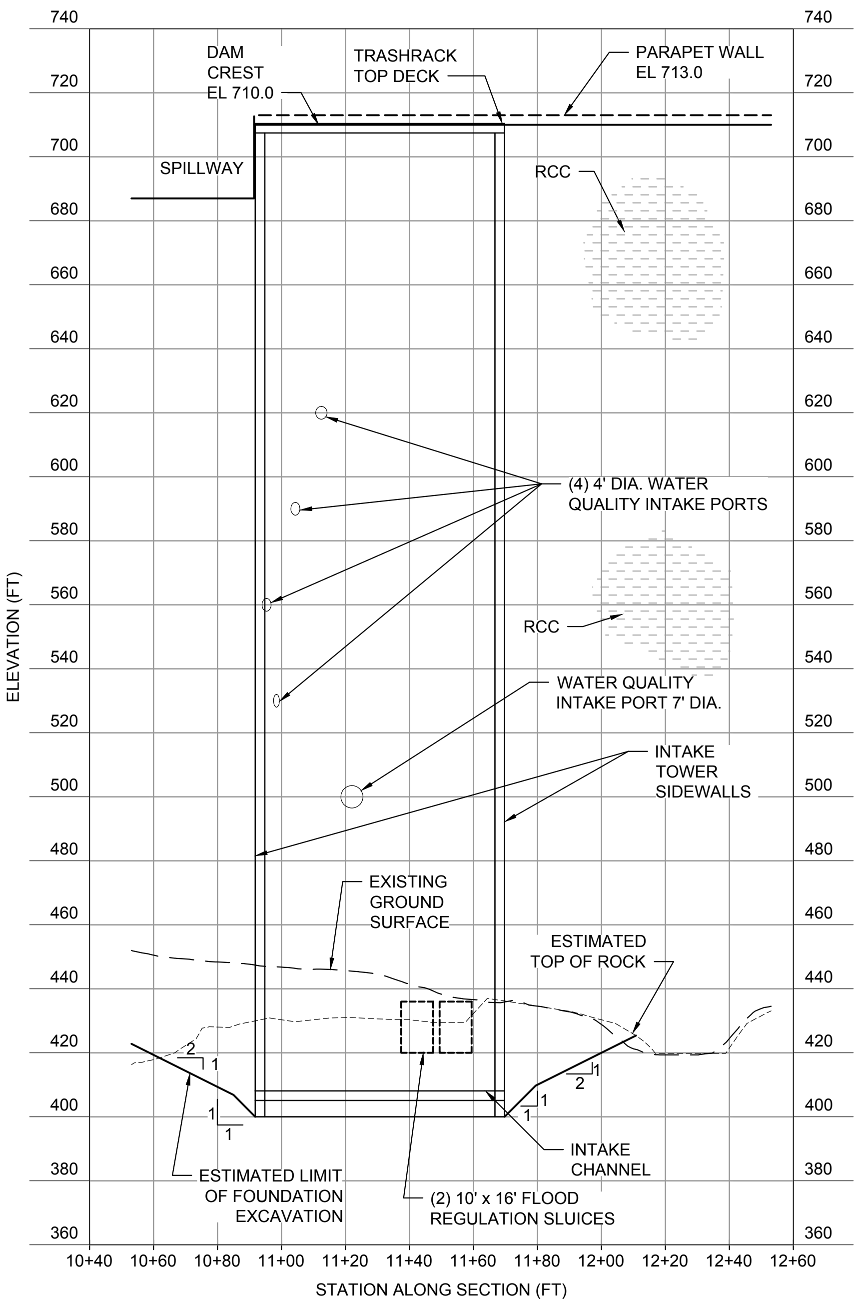
0 20 40 80
SCALE IN FEET

FRFA DAM
TYPICAL DAM AND SPILLWAY
SECTIONS 2 AND 3

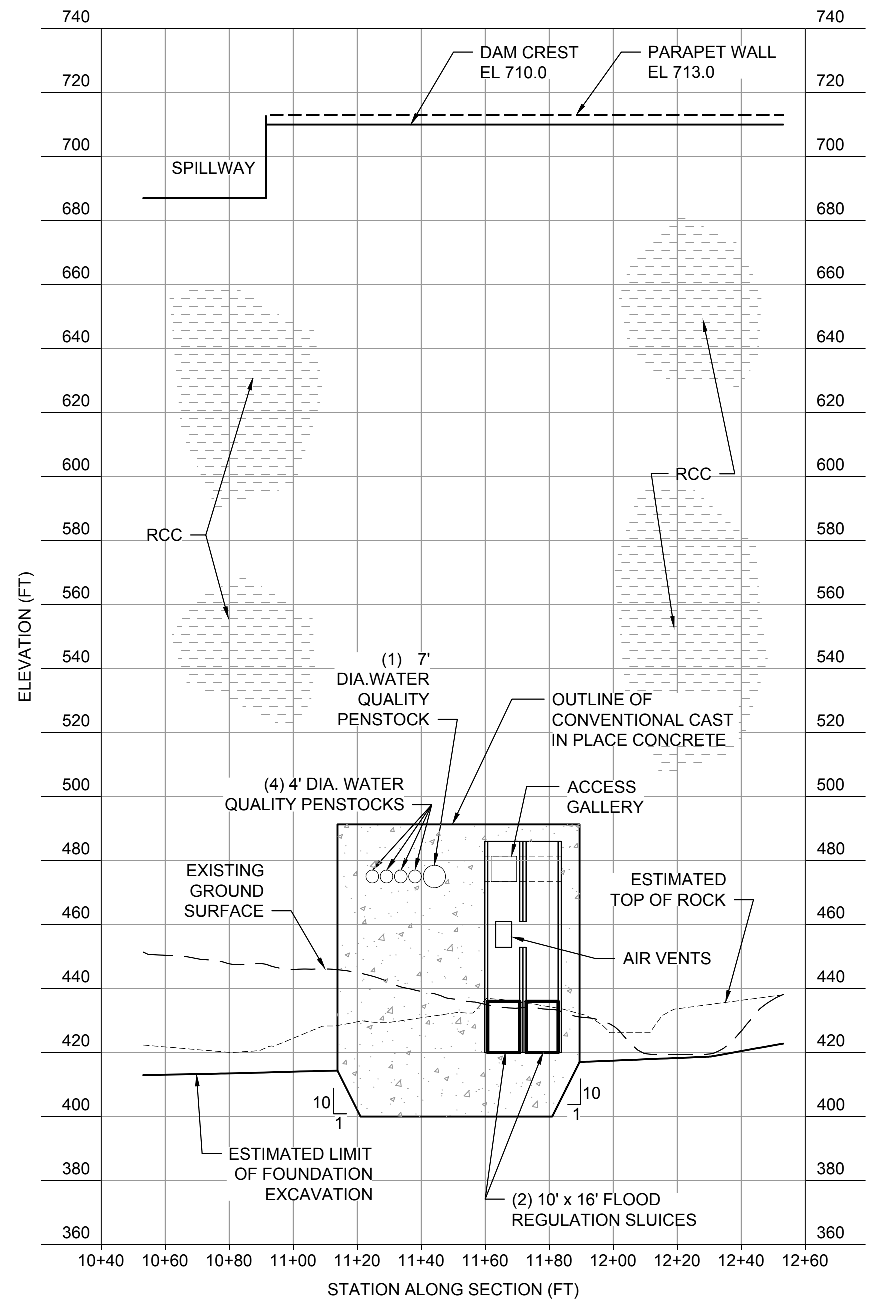
CHEHALIS BASIN DAM

DATE: JULY 2017

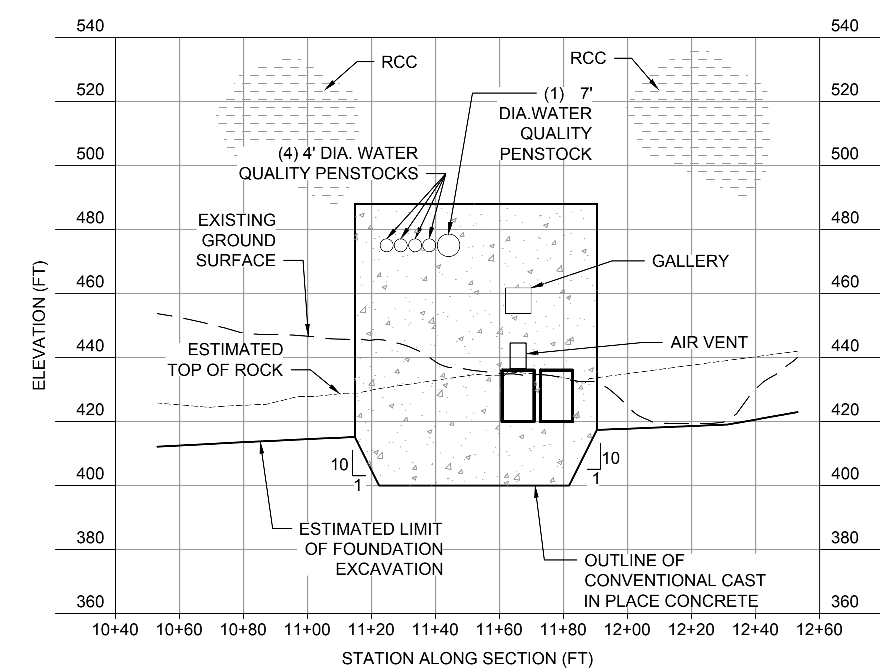
FIGURE: FRFA-S-5



SECTION 4 FRFA-S-3



SECTION 5 FRFA-S-3



SECTION 6 FRFA-S-3

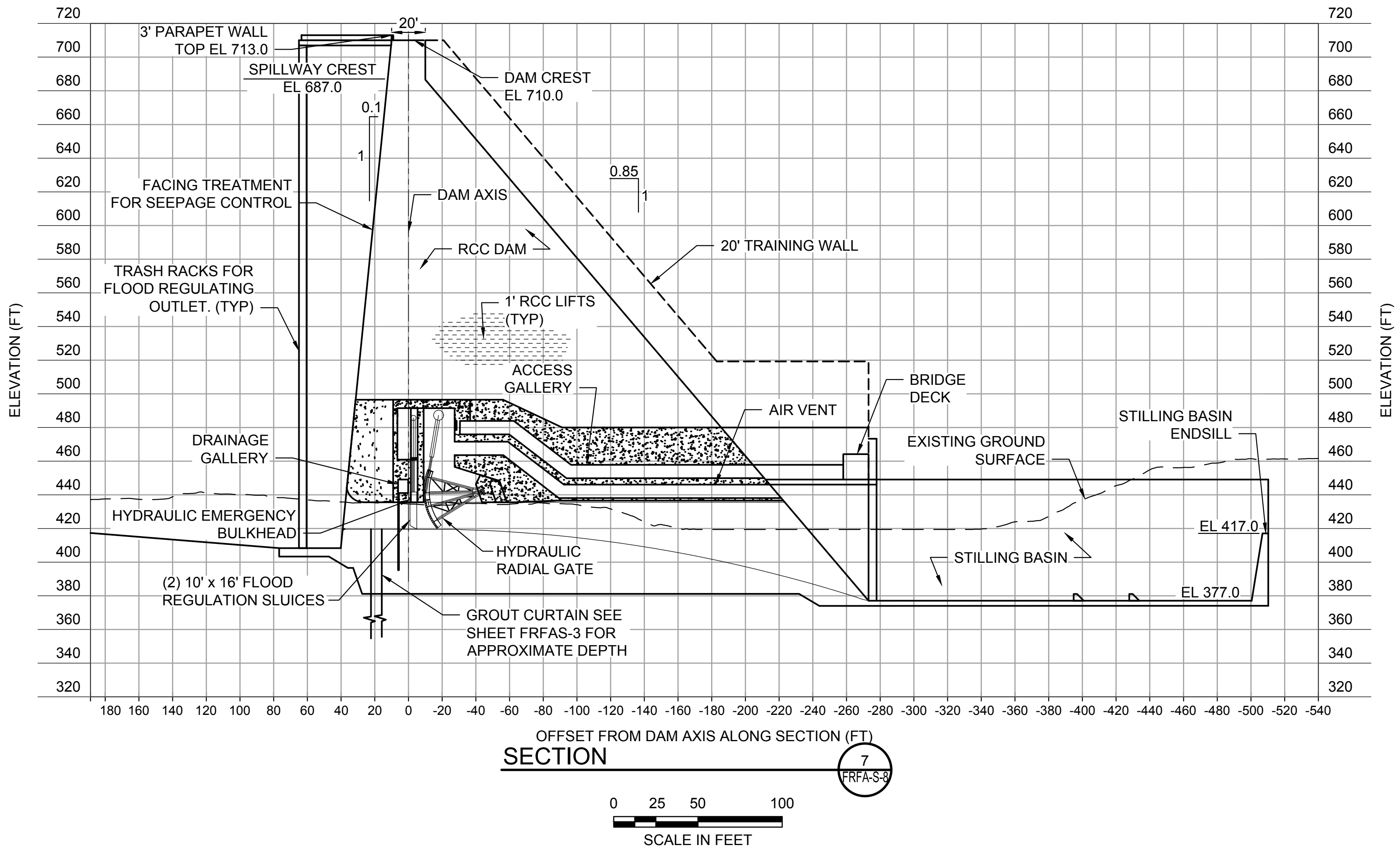
0 15 30 60
SCALE IN FEET

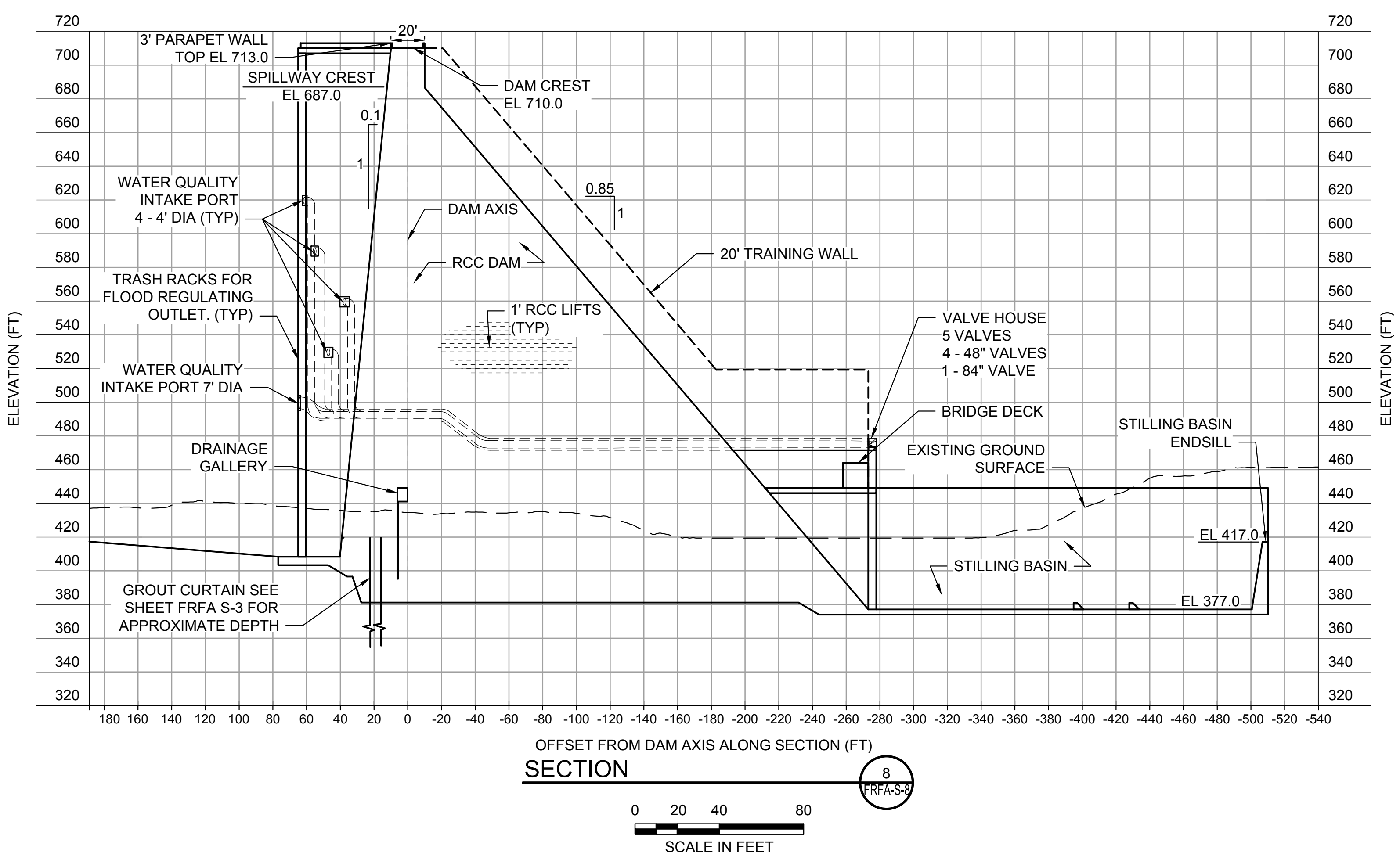
FRFA DAM
INTAKE TOWER AND SLUICES
SECTIONS 4, 5, AND 6

CHEHALIS BASIN DAM

DATE JULY 2017

FIGURE FRFA-S-6

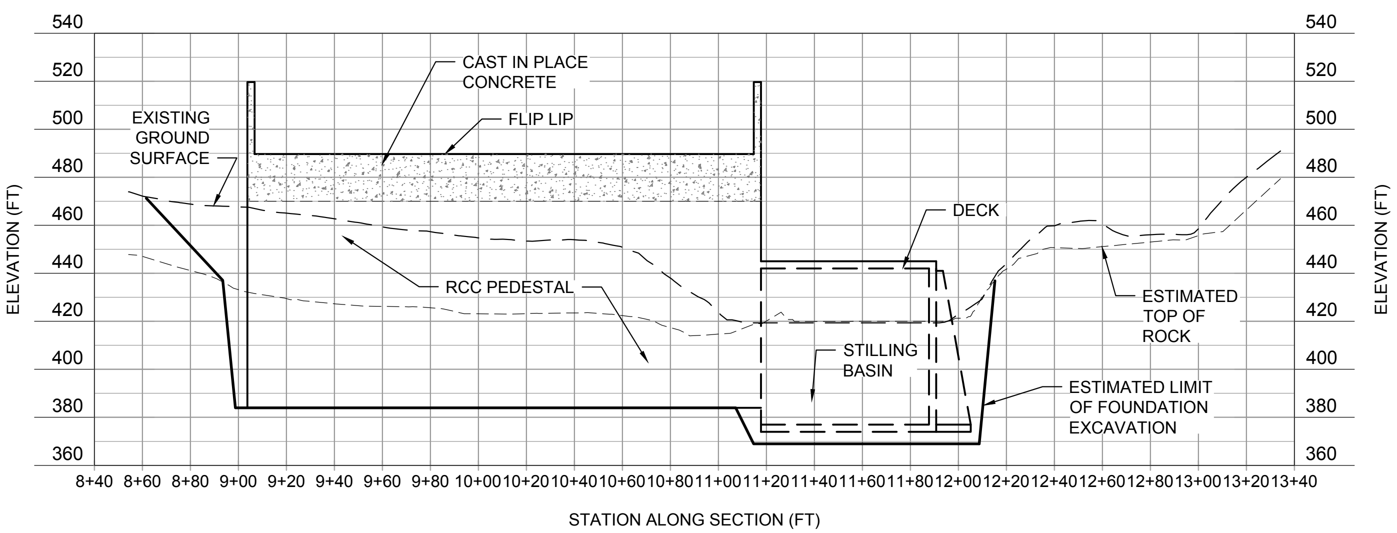




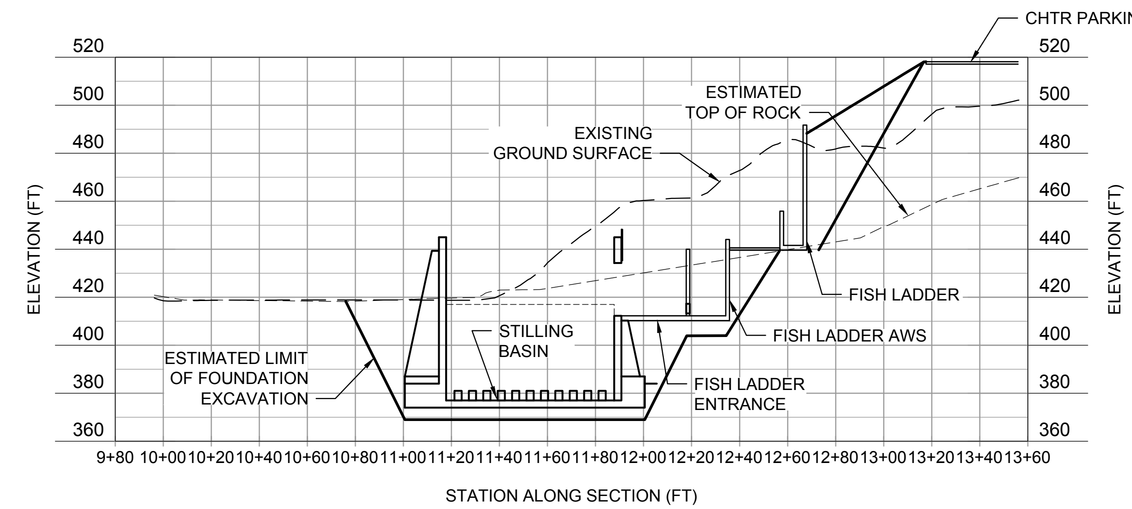
FRFA DAM
FLOOD REGULATION SLUICES AND WATER QUALITY OUTLET WORKS SECTIONS 7 AND 8

CHEHALIS BASIN DAM

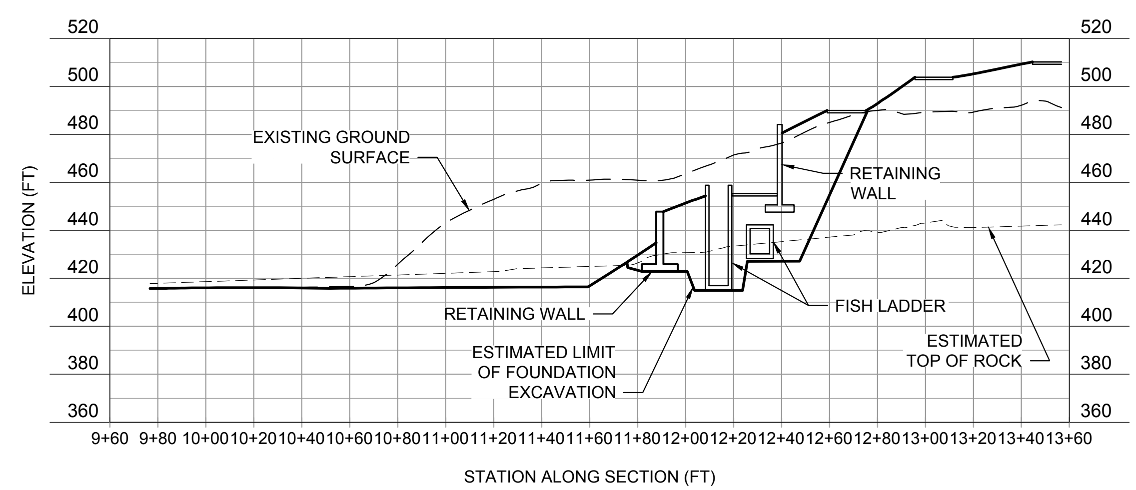
DATE	JULY 2017
FIGURE	FRFA-S-7



SECTION 9 FRFA-S-3



SECTION 10 FRFA-S-3



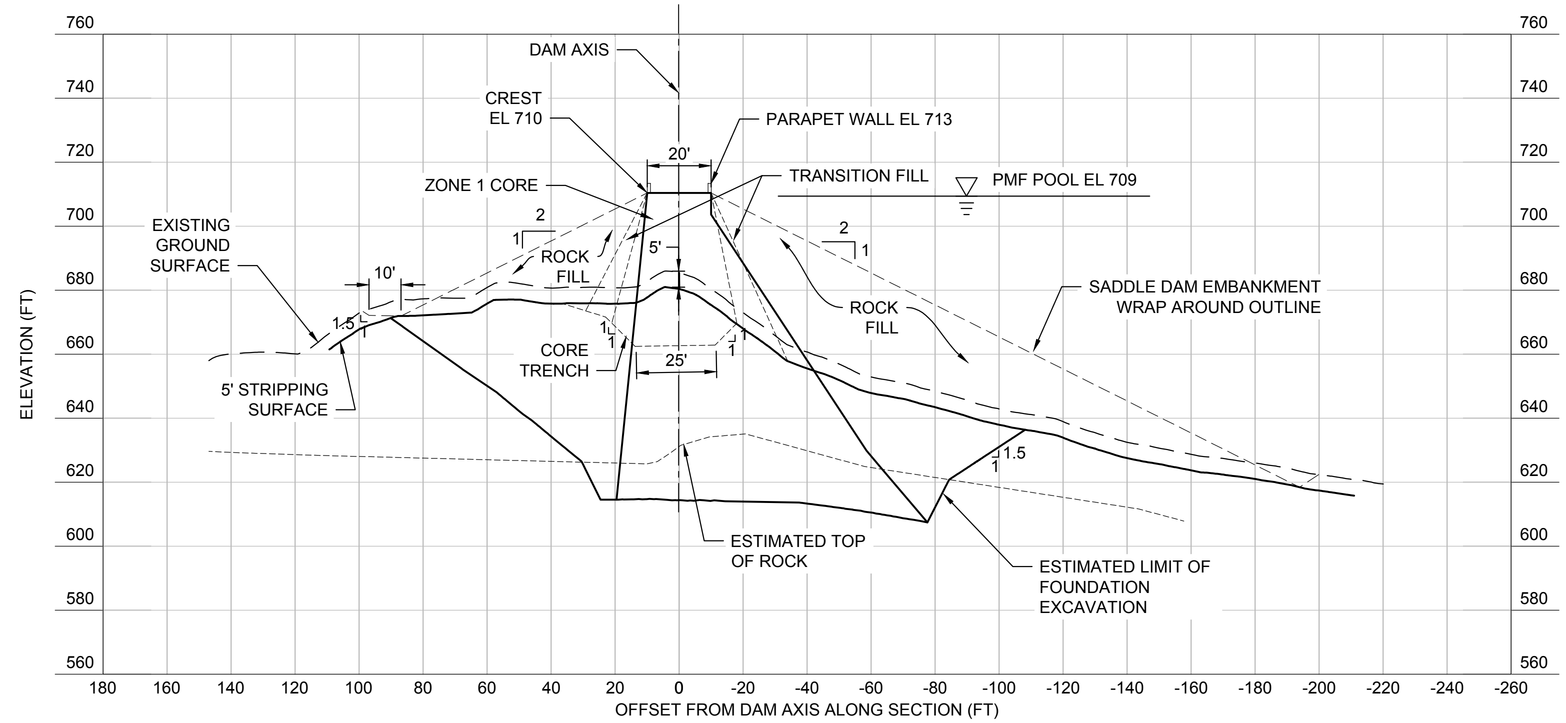
SECTION 11 FRFA-S-3

0 20 40 80
SCALE IN FEET

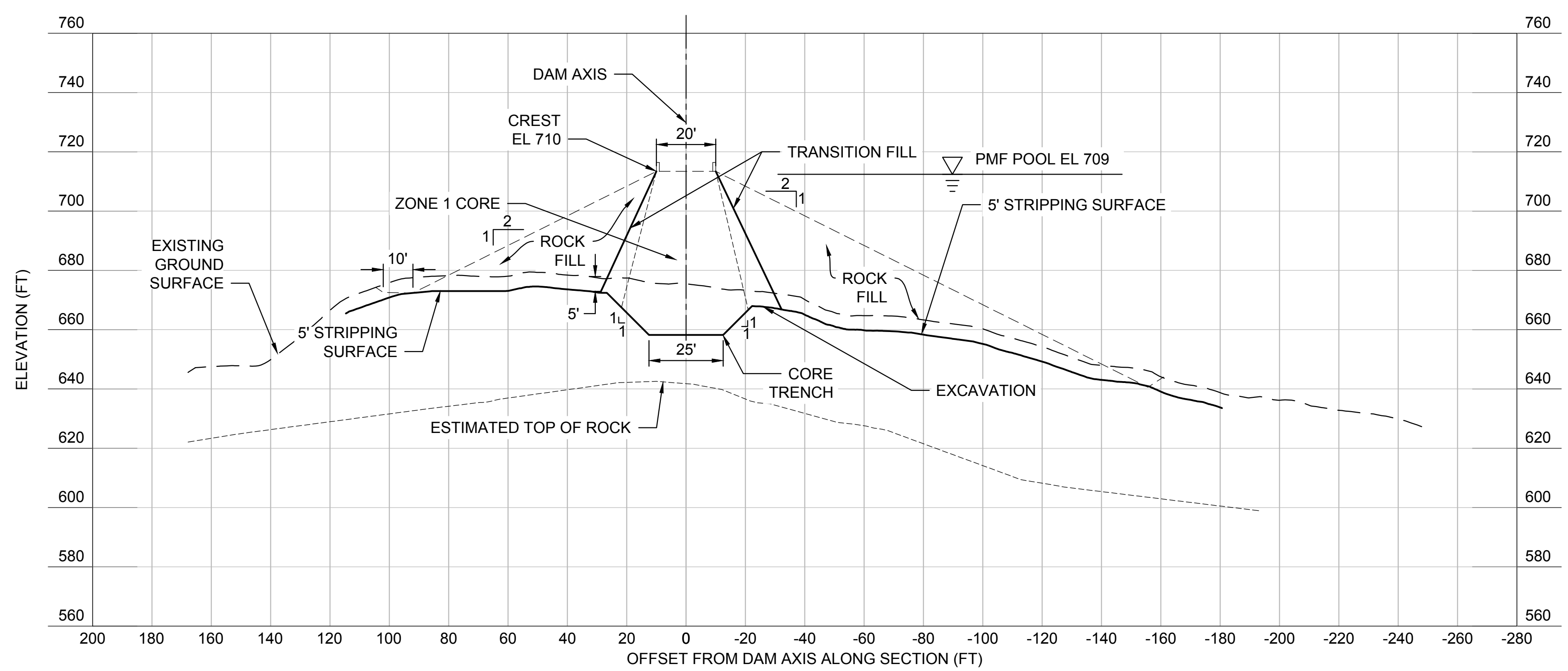
HDR

FRFA DAM
FLOOD REGULATION SLUICES AND WATER QUALITY OUTLET WORKS STILLING BASIN SECTIONS 9, 10, AND 11
CHEHALIS BASIN DAM

DATE: JULY 2017
FIGURE: FRFA-S-8



SECTION 12 FRFA-S-1



SECTION 13 FRFA-S-1

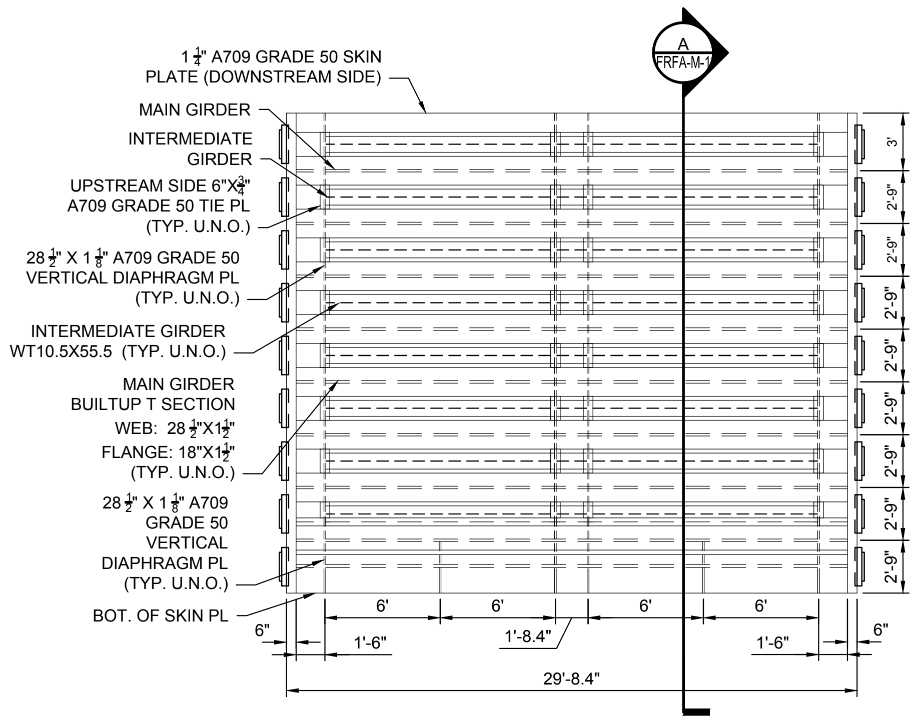
0 15 30 60
SCALE IN FEET

HDR

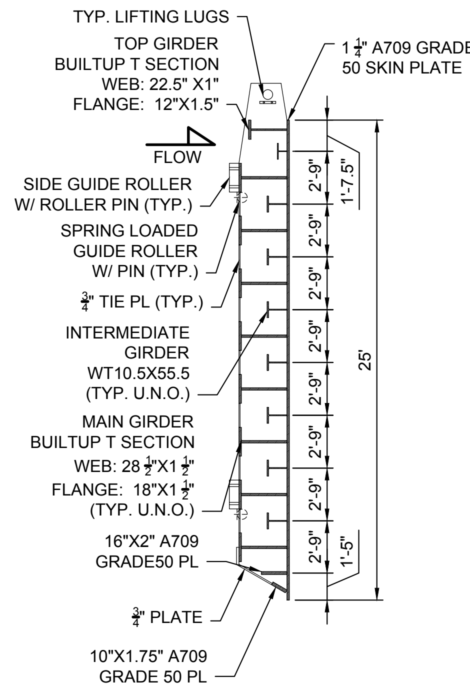
FRFA DAM
RIGHT ABUTMENT
EMBANKMENT DAM TYPICAL
SECTIONS 12 AND 13
CHEHALIS BASIN DAM

DATE
JULY 2017

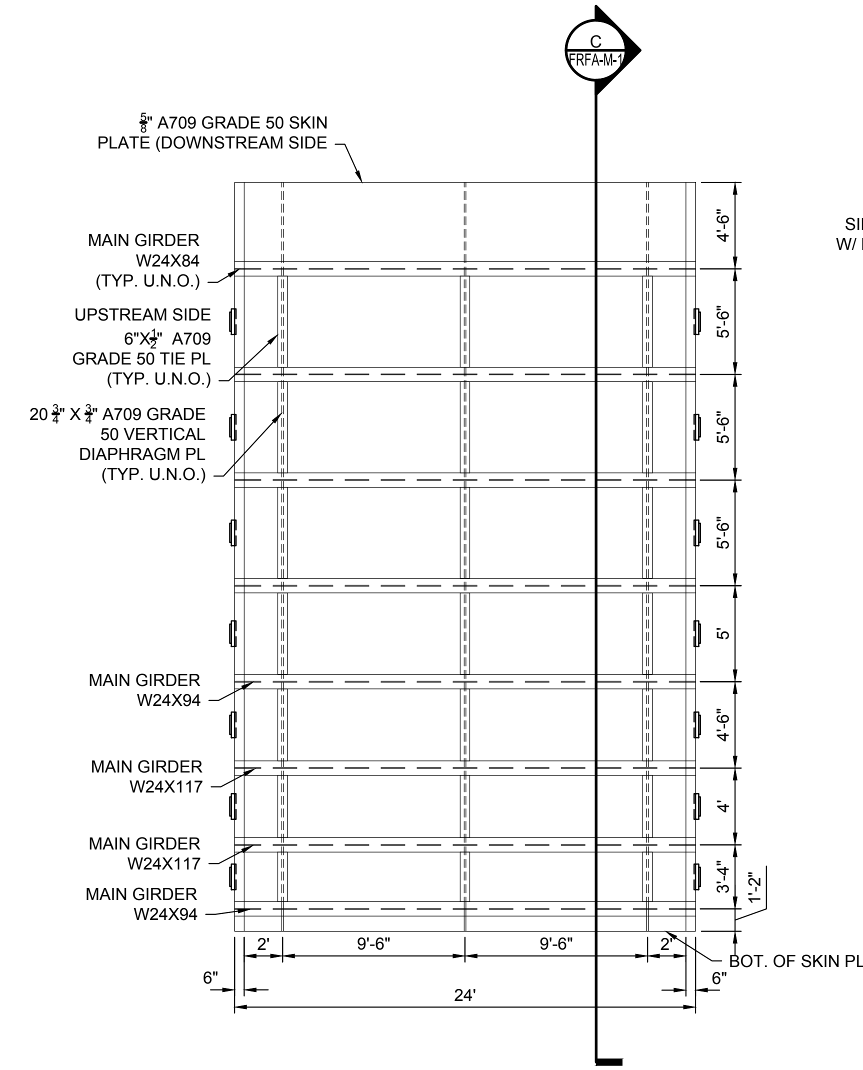
FIGURE
FRFA-S-9



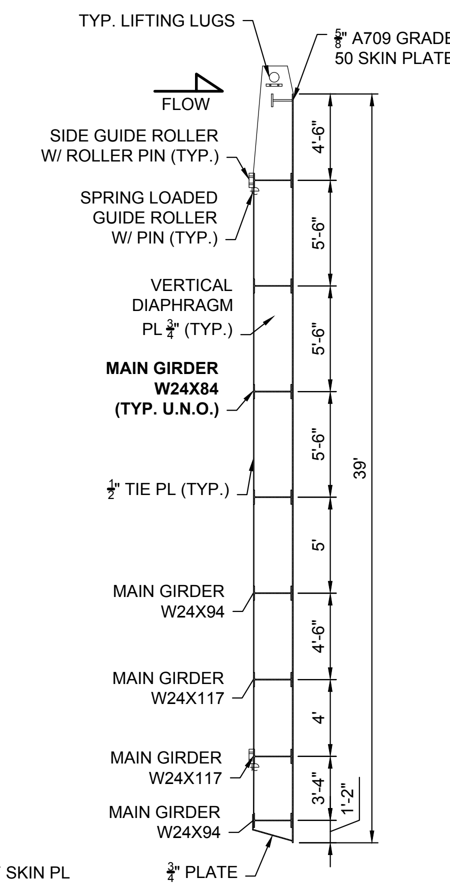
ELEVATION UPSTREAM DEWATERING BULKHEAD (1, FRFA-M-1)



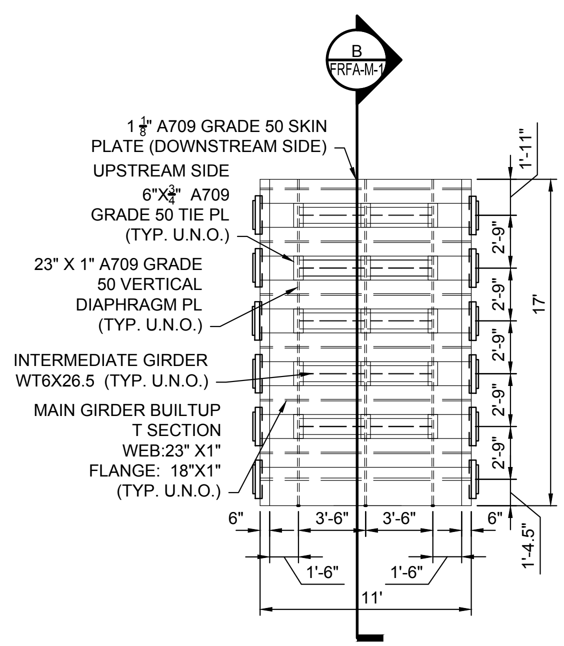
SECTION (A, FRFA-M-1)



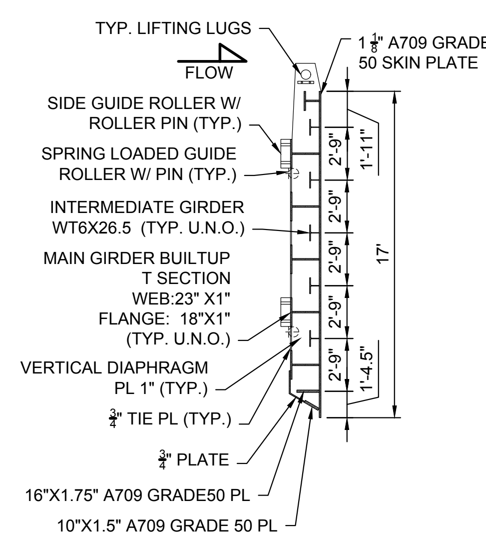
ELEVATION DOWNSTREAM DEWATERING BULKHEAD (3, FRFA-M-1)



SECTION (C, FRFA-M-1)



ELEVATION EMERGENCY CLOSURE GATE (3, FRFA-M-1)



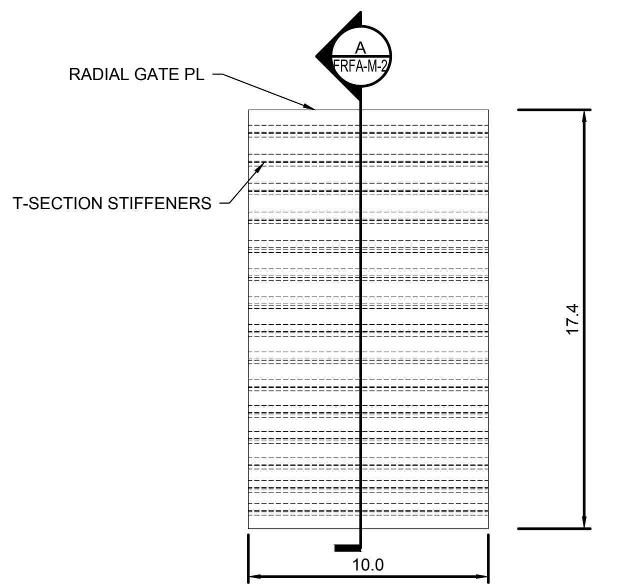
SECTION (B, FRFA-M-1)

0 2.5 5 10
SCALE IN FEET

HDR

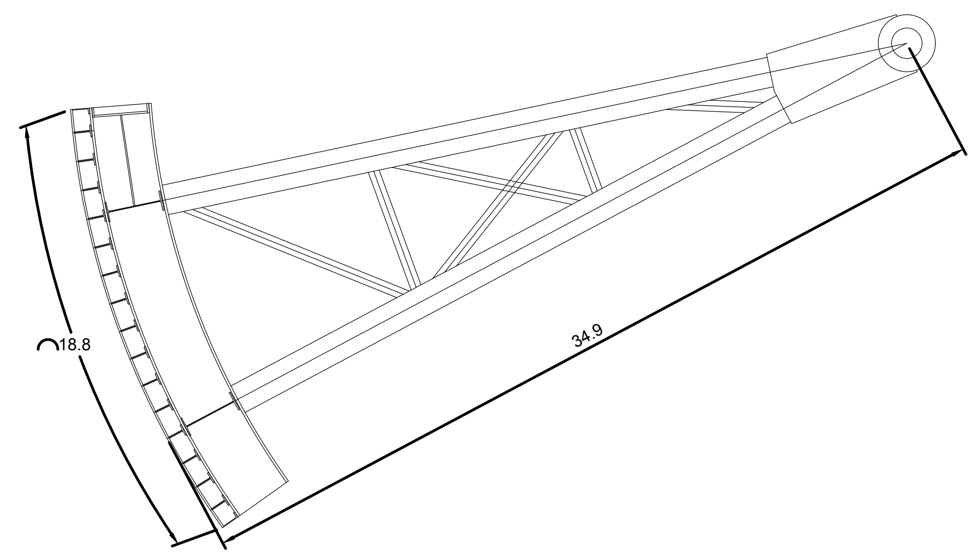
FRFA DAM
FLOOD REGULATION SLUICES
GATES AND BULKHEADS DETAILS
10' x 16' SLUICE
CHEHALIS BASIN DAM

DATE	JULY 2017
FIGURE	FRFA-M-1



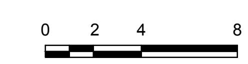
ELEVATION 10'X16' RADIAL SLUICE GATE

1
FRFA-M-2



SECTION

A
FRFA-M-2



HDR

FRFA DAM
FLOOD REGULATION SLUICE GATES DETAILS
10' x 16' RADIAL GATES
CHEHALIS BASIN DAM

DATE
JULY 2017

FIGURE
FRFA-M-2

HDR C:\pwworking\sea\d1810432\FRFA-M-2.dwg PRINTED: 7/11/2017 12:34:33 PM BY: POXBORRO

Appendix B
Hydraulic Design of Spillway and Reservoir Outlet Works

Chehalis Basin Strategy

Appendix B

Hydraulic Design of Spillway and Reservoir Outlet Works

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

Prepared for: State of Washington Recreation and Conservation Office and Chehalis Basin Work Group

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ACRONYMS AND ABBREVIATIONS

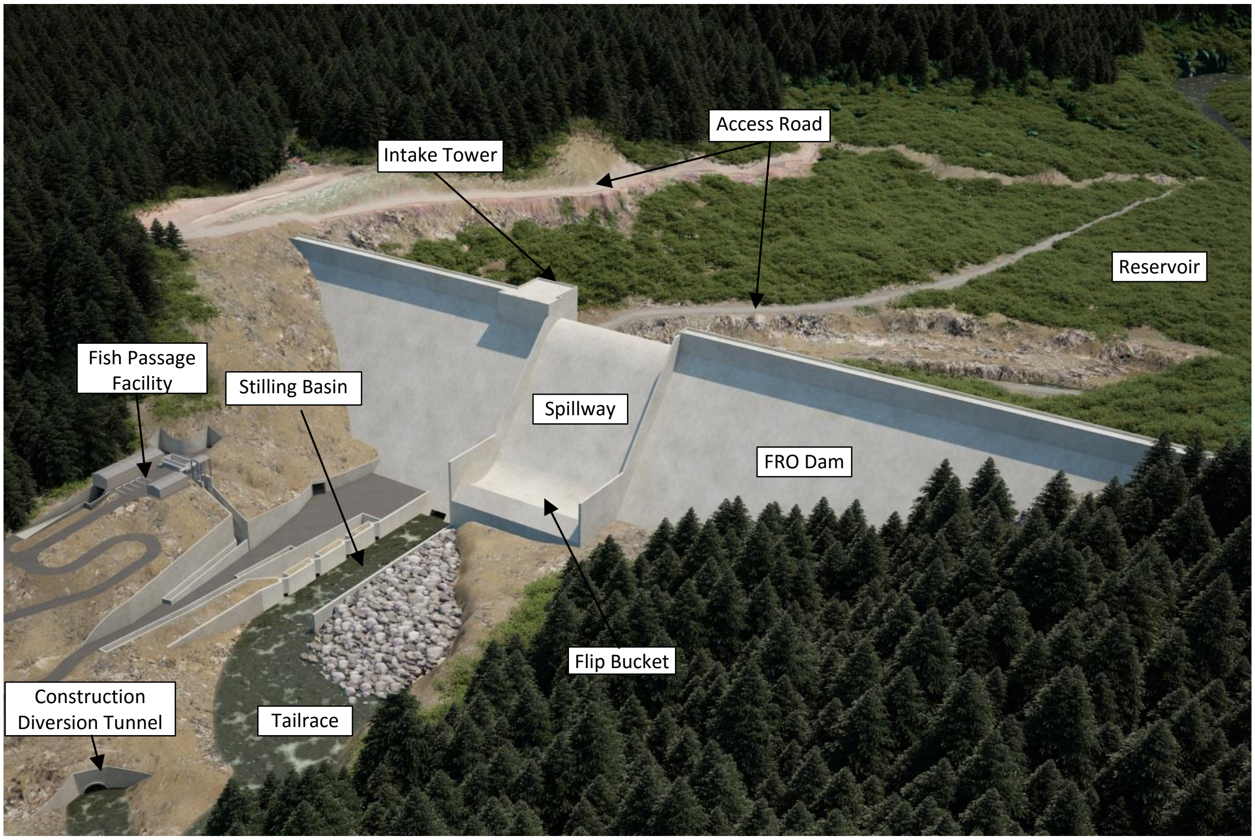
cfs	cubic feet per second
DSO	Dam Safety Office
EM	engineering manual
ER	engineering report
FRFA	flood retention flow augmentation
FRO	flood retention only
HDC	Hydraulic Design Criteria
HEC	Hydrologic Engineering Center
HEC-ResSim	Hydrologic Engineering Center Reservoir System Simulation
LiDAR	Light Detection and Ranging
MPM	Meyer-Peter-Mueller
msl	mean sea level
NRCS	Natural Resources Conservation Service
PMF	probable maximum flood
PMP	probable maximum precipitation
RCC	roller compacted concrete
RM	river mile
SDF	Spillway Design Flood
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USGS	United States Geological Survey
WSE	Watershed Science & Engineering

1 DAM ALTERNATIVES DESCRIPTION

1.1 Flood Retention Only Alternative

The Flood Retention Only (FRO) alternative is comprised of a Roller Compacted Concrete (RCC) gravity dam, to be raised on the mainstem Chehalis River at approximately River Mile (RM) 108 (Figure 1-1). The dam would have a maximum structural height of approximately 255 to 270 feet, depending on final foundation elevation, with a dam crest elevation of about 651 feet (mean sea level [msl]). The spillway would be an uncontrolled ogee crest at about elevation 628 feet (msl) with a crest length of about 200 feet, while primary discharge capacity would be provided by several low level sluice gates extending through the dam structure at an elevation approximately the same as the existing stream channel bed. Under typical operation whenever flood flow regulation is not needed, there would be no reservoir impoundment, as the sluice gates would be held fully open to pass all inflows without retention. The low level sluices would be large enough in size to provide for relatively unimpeded fish and sediment passage through the sluice conduits at flows less than about 2,200 cubic feet per second (cfs). All sluices would remain fully open until the downstream Chehalis River flow at the Grand Mound gage is predicted to rise above 38,000 cfs, at which point flood regulation is required and the sluices would be gradually closed to minimum discharge to retain flood flows. When the Grand Mound gage flow is predicted to fall below 38,000 cfs, the pair of smaller sluices would be gradually opened to draft the reservoir, allowing suspended sediment to pass. When the reservoir elevation is drafted below approximately elevation 500 feet, the large gate would be opened to provide an opportunity for accumulated bed sediment to pass through the dam. The FRO dam operation is patterned after the Mud Mountain Dam on the White River, near Enumclaw, Washington, which is owned by the Seattle District of the U.S. Army Corps of Engineers (USACE).

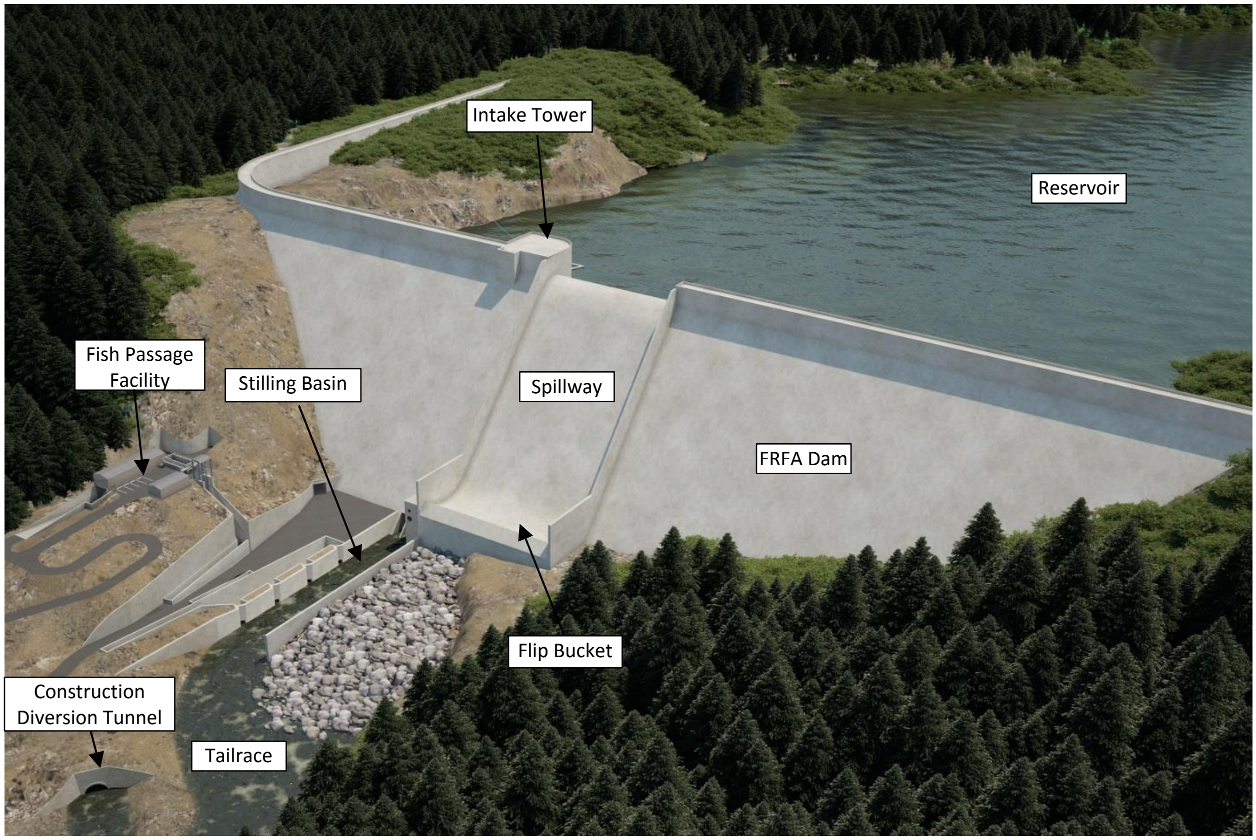
Figure 1-1
FRO Dam Concept



1.2 Flood Regulation Flow Augmentation Alternative

The Flood Retention and Flow Augmentation (FRFA) alternative is comprised of a RCC gravity dam in the same location as the FRO alternative on the mainstem Chehalis River (Figure 1-2). The FRFA dam would have a maximum structural height of 313 to 330 feet, depending on final foundation elevation, with a dam crest elevation of about 710 feet (msl). Similar to the FRO alternative, the spillway would be uncontrolled ogee with a crest length of about 200 feet and primary discharge capacity would be provided by a pair of low level sluices extending through the dam structure and controlled by radial gates.

Figure 1-2
FRFA Dam Concept



2 DAM ALTERNATIVES DESIGN

2.1 Chehalis Flood Storage Dam Alternatives

As discussed in the main body of the report, this study considered two different approaches to provide flood regulation on the Chehalis River. The first option, the FRO, would typically not impound the Chehalis River flows until and unless a large flood is forecasted to occur. The second option, the FRFA, would impound a moderate volume of water each year to aid in providing cool stored water for downstream releases during low flow periods in the summer, in addition to providing a similar amount of flood storage volume as the FRO alternative. The FRO and FRFA are both designed to provide downstream flood protection benefits, but have different dam hydraulic heights, operational approach, and potential gross storage volumes. The specific flow of 38,000 cfs at the Grand Mound gage, or about a 1 in 7 year flood event, has been identified in preliminary assessments as the control flow for the proposed dam flood regulation operation, but that value may be refined and optimized as the economic benefits and costs associated with that particular control flow are more fully evaluated.

2.2 FRO Configuration

The FRO dam consists of a concrete (RCC) gravity dam structure with a right abutment construction diversion tunnel, low-level outlet sluices that provide for both flood regulation and fish passage, full-height trashrack to protect against outlet blockage, an emergency uncontrolled overflow spillway, and supplemental fish passage facilities. An upstream view of the FRO configuration is shown in Figure 2-1 below, and a section view through the proposed dam is shown in Figure 2-2.

The currently envisioned FRO alternative's primary characteristics include the following:

- A RCC dam of 254 to 275 feet estimated maximum dam structural height depending on final foundation elevation
- Dam crest length of approximately 1,225 feet to span the Chehalis valley width along a straight alignment
- Uncontrolled overflow ogee spillway crest approximately 200 feet wide, with crest elevation 628 feet (msl) designed to pass the Probable Maximum Flood (PMF) event with freeboard to the dam crest, but expected to operate very infrequently
- Smooth spillway ogee and chute cast over the RCC dam section. Chute would have training/containment walls approximately 20 feet in height.
- Spillway terminus flip bucket to eject jet well out and away from the dam structure
- Spillway discharge plunge pool well downstream of the toe of the dam
- Single 12 foot wide by 20 foot high low level sluice to pass bed load sediment and low head flood flows, with invert elevation approximately at existing river channel bed elevation. This

sluice floor would be expected to be repaired regularly due to sediment abrasion and erosion, much like Mud Mountain Dam.

- Pair of large 10 foot wide by 16 foot high low level sluices to pass high head flood flows, with invert elevation about 3 feet higher than the existing river channel bed elevation. These would be used to pass flow when the reservoir exceeded about 50 feet of head and bed load sediment would no longer be actively moving through the dam
- Construction diversion tunnel about 20 feet in diameter through the right abutment. The tunnel floor would be lined with concrete to provide a smooth invert wear layer for sediment passage during construction, and would be plugged following completion of the low level outlet sluices but provided with a drain valve to evacuate the reservoir if needed
- Hydraulic jump-type energy dissipating stilling basin approximately 230 feet long by 70 feet wide and 36 feet deep with baffle blocks to capture and dissipate flow energy from the low level outlet sluices. The stilling basin would be concrete lined, and would have an end sill elevation roughly the same elevation as the downstream river channel
- Fish ladder and collection channel with entrances along the right wall of the stilling basin to attract and pass upstream migrating fish to the trap and haul facility
- Initial target flood storage pool volume of 65,000 acre-feet, to be activated in flood events larger than the estimated 7-year recurrence interval event. This value may be refined as the economic benefit-to-cost studies progress to identify the preferred storage volume

The flood regulation operation is achieved by radial sluice gates controlling sluice discharge when required under the prescribed operation plan. Flows would not be impounded unless the Chehalis River at the Grand Mound gage was forecasted to rise above 38,000 cfs. Except during flood control operations, the sluice gates are to remain fully open, freely passing suspended and bed load sediment, smaller woody debris that can readily pass through the trashrack, and fish both upstream and downstream. Larger woody debris that becomes lodged against the trashrack would be removed as needed to keep the channel clear and permit unfettered fish passage and maintain sediment transport through the dam

The Mud Mountain Dam on the White River in western Washington State, owned and operated by the USACE, has been operating successfully since the late 1940s and operates in a very similar fashion. Similar to the proposed FRO dam alternative for Chehalis River, the Mud Mountain dam is a run-of-river type dam which does not typically impound the river flows unless a large flood is forecasted to occur. In this case, the flood regulation operation will commence by closing the low level outlets, holding back water and slowly releasing water back into the river after the flood peak has passed. However, unlike the FRO dam alternative, the Mud Mountain Dam does not pass upstream migrant fish through the low level outlet sluices, and instead utilizes a separate downstream low barrier weir and trap and haul facility operated continuously to collect and transport upstream migrating fish from all five species of Pacific salmonids to the extensive watershed habitat above the dam.

When flood regulation operations are commenced, the sluice gates would be throttled as needed to reduce mainstem flow sufficiently to hold the Grand Mound gage at or below 38,000 cfs. Once flood control operations begin, fish passage through the sluice conduits would be limited or temporarily suspended as a result of the high flow velocities within the low level sluice conduits. However, coincident with the commencement of flood regulation operation, the fish ladder would be opened and fish would be attracted to the ladder and collection facility instead. A trap and haul facility would begin operations to move upstream-migrating fish to a release point above the reservoir. Downstream fish passage would still be possible through the low level sluice conduits, though the rising reservoir would at some point cause the submergence of the sluices to be too excessive for downstream migrating fish to readily find it. Once the flood peak has passed and the reservoir is evacuated, downstream fish passage would resume as the submergence over the low level sluice outlets decreases. Upstream fish passage would be provided by the fish ladder and trap and haul facility until the reservoir was fully drained and woody debris and sediment could be cleared from the trashrack opening to permit free flow again. Larger flood events that carry significant volumes of debris to the reservoir may require that the pool to be maintained for a brief period of time to corral and move floating debris to containment areas before complete draw down.

Figure 2-1
FRO Dam Concept Upstream View

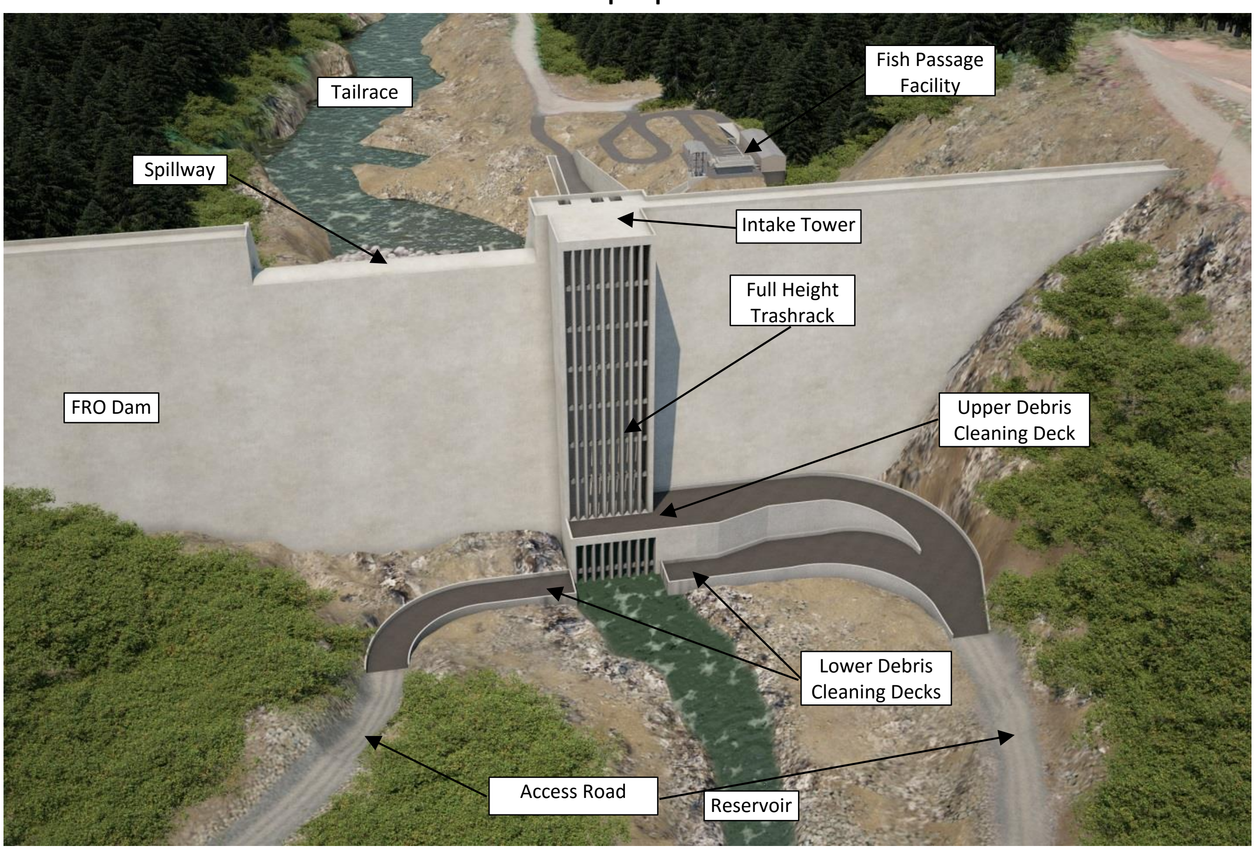
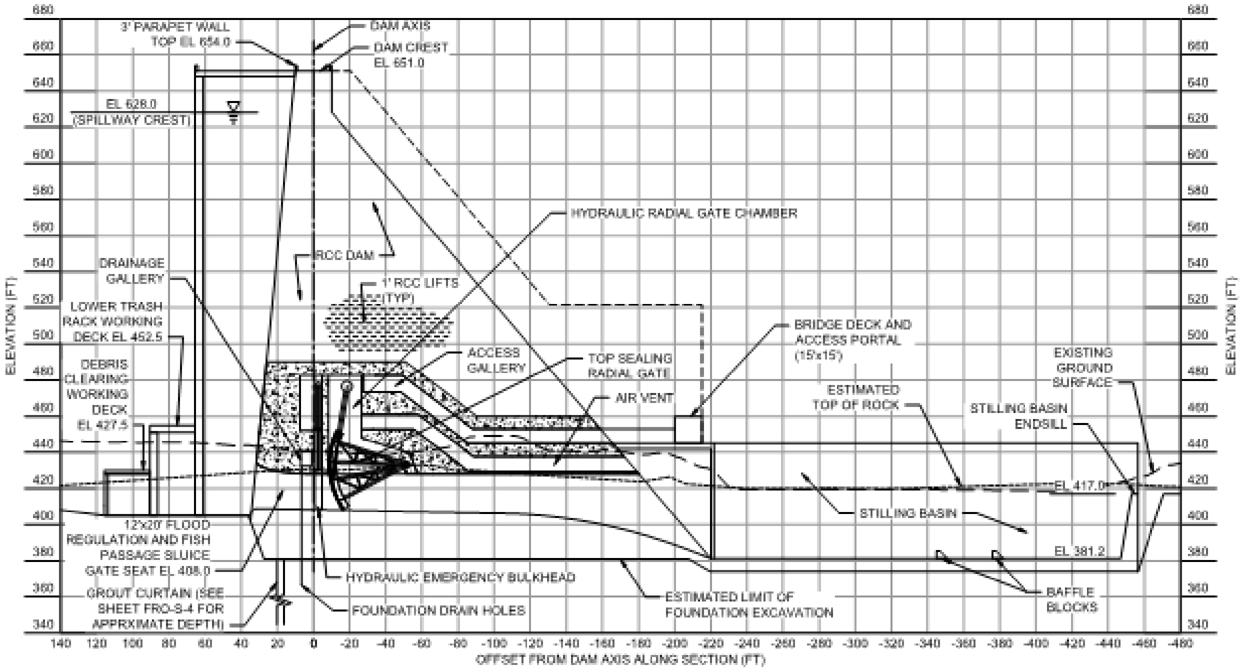


Figure 2-2
Section Through FRO Dam 12 foot by 20 foot Sluice Outlet



2.3 FRFA Configuration

The FRFA dam consists of a concrete (RCC) gravity dam structure with a right abutment construction diversion tunnel, low level outlet sluices that provide for flood regulation, multiple water quality outlet ports at varying elevations to tap various strata within the reservoir during the low flow period, a full height trashrack to protect the low level outlet sluices from debris blockage, an uncontrolled ogee spillway crest, and supplemental fish passage facilities. The FRFA alternative would maintain a permanent pool behind the dam and be designed to provide water storage and releases for flow augmentation from the permanent pool to enhance certain aquatic species habitat, and a flood management pool between the designated permanent pool level and the spillway crest for flood operations. The smaller individual multilevel water quality outlet ports are provided with modular trashracks to prevent debris entrainment. The low level sluices are similar to the pair of 10 foot wide by 16 foot high sluices provided in the FRO alternative, but set at an invert elevation of 420 feet instead of 411.0. This elevation should reduce the amount of sediment bed load entrained into the sluices during flood regulation operation. An upstream elevation of the FRFA configuration is shown in Figure 2- below, and a section view through the proposed dam is shown in Figure 2-.

The currently envisioned FRFA alternative's primary characteristics include the following:

- A RCC dam of 313 to 330 feet estimated maximum dam structural height depending on final foundation elevation

- Dam crest length of approximately 1,600 feet spanning the Chehalis valley and extending along the right abutment ridge with an RCC and rockfill section about 700 feet in length to carry the dam crest closure to high ground
- Uncontrolled overflow ogee spillway crest approximately 200 feet wide, with crest elevation 687 feet (msl) designed to pass the PMF event with freeboard to the dam crest, but expected to operate very infrequently
- Smooth spillway ogee and chute cast over the RCC dam section. Chute would have training/containment walls approximately 20 feet in height
- Spillway terminus flip bucket to eject the jet well out and away from the dam structure
- Spillway discharge plunge pool well downstream of the toe of the dam
- Pair of large 10 foot wide by 16 foot high low level sluices to pass high head flood flows, with invert elevation about 15 to 20 feet higher than the existing river channel bed elevation. These would be used to pass flow whenever the discharge requirements exceeded the capacity of the multilevel outlet ports, and could be used at any reservoir elevation. These gates would typically pass only suspended sediment
- Construction diversion tunnel about 20 feet in diameter through the right abutment. The tunnel floor would be lined with concrete to provide a smooth invert wear layer for sediment passage during construction, and would be plugged following completion of the low level outlet sluices but provided with a drain valve to evacuate the reservoir if needed
- Hydraulic jump-type energy dissipating stilling basin approximately 230 feet long by 70 feet wide and 40 feet deep with baffle blocks to capture and dissipate flow energy from the low level outlet sluices. The stilling basin would be concrete lined, and would have an end sill elevation roughly the same elevation as the downstream river channel
- Multiple outlet works including five water quality inlets/outlets that draw water from multiple levels within the reservoir and a low-level flood control outlet
- Fish ladder and collection channel with entrances along the right wall of the stilling basin to attract and pass upstream migrating fish to the trap and haul facility
- Floating fish collection and dewatering screened facility in the reservoir to collect the downstream migrating fish, transport and release in the river downstream of the dam
- A permanent reservoir storage volume of up to 65,000 acre-feet to be used for flow augmentation in late summer and fall prior to the winter rainy season to enhance fish habitat. This value may change as the biological benefit-to-cost studies progress to identify the preferred storage volume
- Up to 65,000 acre-feet of flood storage volume to be activated in flood events larger than the estimated 7-year recurrence interval event. This value may change as the economic benefit-to-cost studies progress to identify the preferred storage volume

During the flood control season, the low level sluices would typically be used to pass flows that would exceed the capacity of the smaller water quality outlets.

Two dams that have similarities to this configuration are the Wynoochee Dam on the Wynoochee River and the Howard A. Hanson Dam on the Green River, both located in western Washington State. The Wynoochee Dam maintains a summer flow augmentation pool and during the winter flood season the reservoir is drawn down to accommodate peak inflows while controlling downstream discharge to less damaging levels. Wynoochee Dam also is provided with upstream migrant fish collection and truck transport facilities a short distance downstream from the dam. The system has operated successfully since the late 1970s when the dam was completed by the USACE. Ownership of the dam was transferred to the City of Tacoma in the 1990s. The Howard A. Hanson Dam operates in a similar fashion to the Wynoochee Dam, though it has no fish passage facilities.

Seasonal operation of the FRFA dam alternative would typically include adherence to an operational rule curve, which establishes a desired reservoir level during each part of the season, and includes reservoir drawdown and filling rates, as well as limitations on downstream rising and falling ramping rates to protect aquatic species and provide for human safety in the event of ramping operations. When the winter flood season approaches, the reservoir would be drawn down according to the operational rule curve over the course of perhaps a month, in preparation for establishing storage volume for high inflows to the reservoir. Wynoochee Dam's winter flood season drawdown period typically begins in October and reaches the desired flood control pool elevation by November. During the winter, the reservoir storage is used to attenuate reservoir inflows in order to limit discharge to less than damaging flood stage. In the early spring, typically beginning in March, the reservoir is filled gradually over time by storing a portion of inflows as needed to provide the desired volume for flow augmentation later in the summer. Then, as inflows decline during summer and early fall, the stored volume of water would be released gradually through the multilevel outlet ports as needed to control outflow water temperature and discharge to meet the desired downstream objectives.

Since the permanent pool would prevent entirely the free passage of upstream and downstream migrating fish that would be accommodated by the FRO alternative, separate fish passage facilities are planned for both upstream and downstream migrating fish in the FRFA dam design. These include a fish collection channel adjacent to the stilling basin similar to that proposed for the FRO alternative, with a ladder leading to a sorting, holding, and loading facility, and a tank truck hauling operation to move fish into the upper reservoir reaches. Downstream-migrating fish would be collected in the reservoir with a floating collection and dewatering screened facility similar to the upper or lower Baker Lake floating collector, or any one of the several similar fish collectors deployed on a number of Pacific Northwest reservoirs.

Figure 2-3
FRFA Dam Concept Upstream View

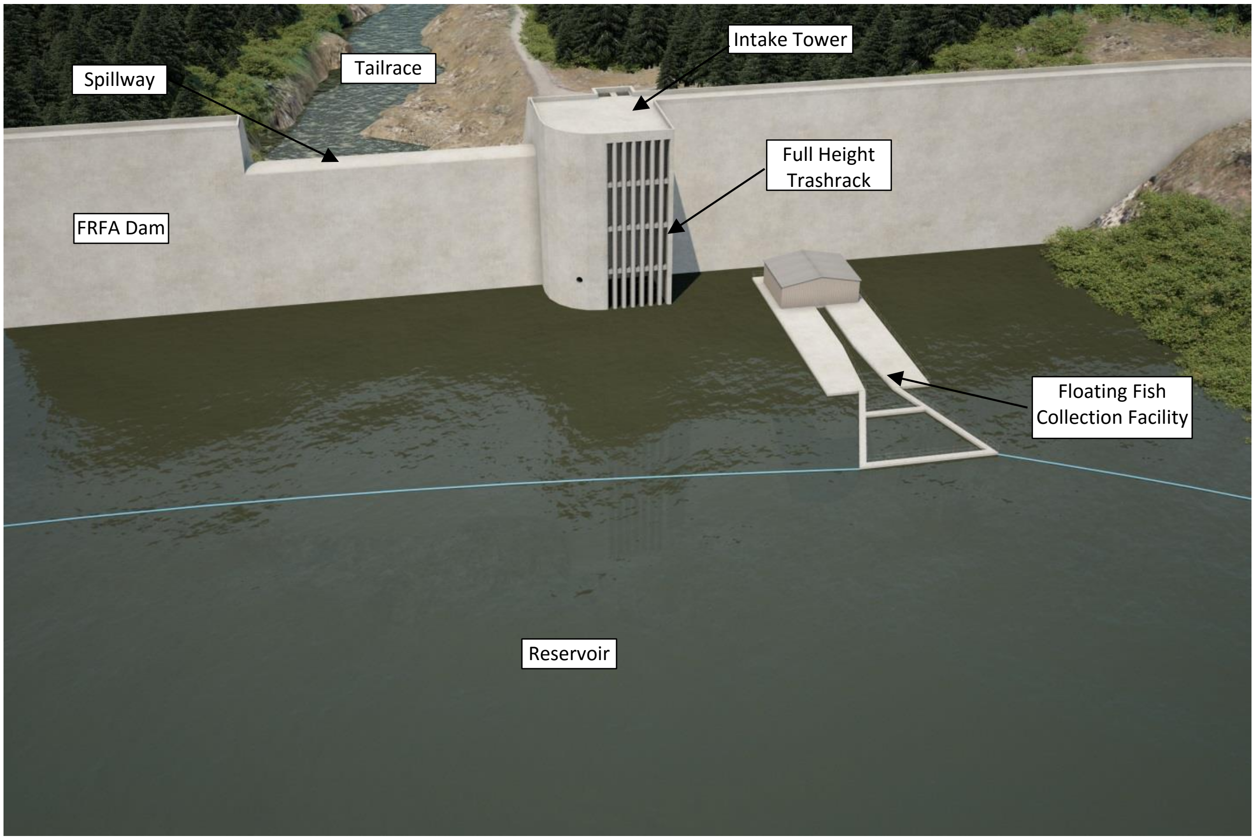
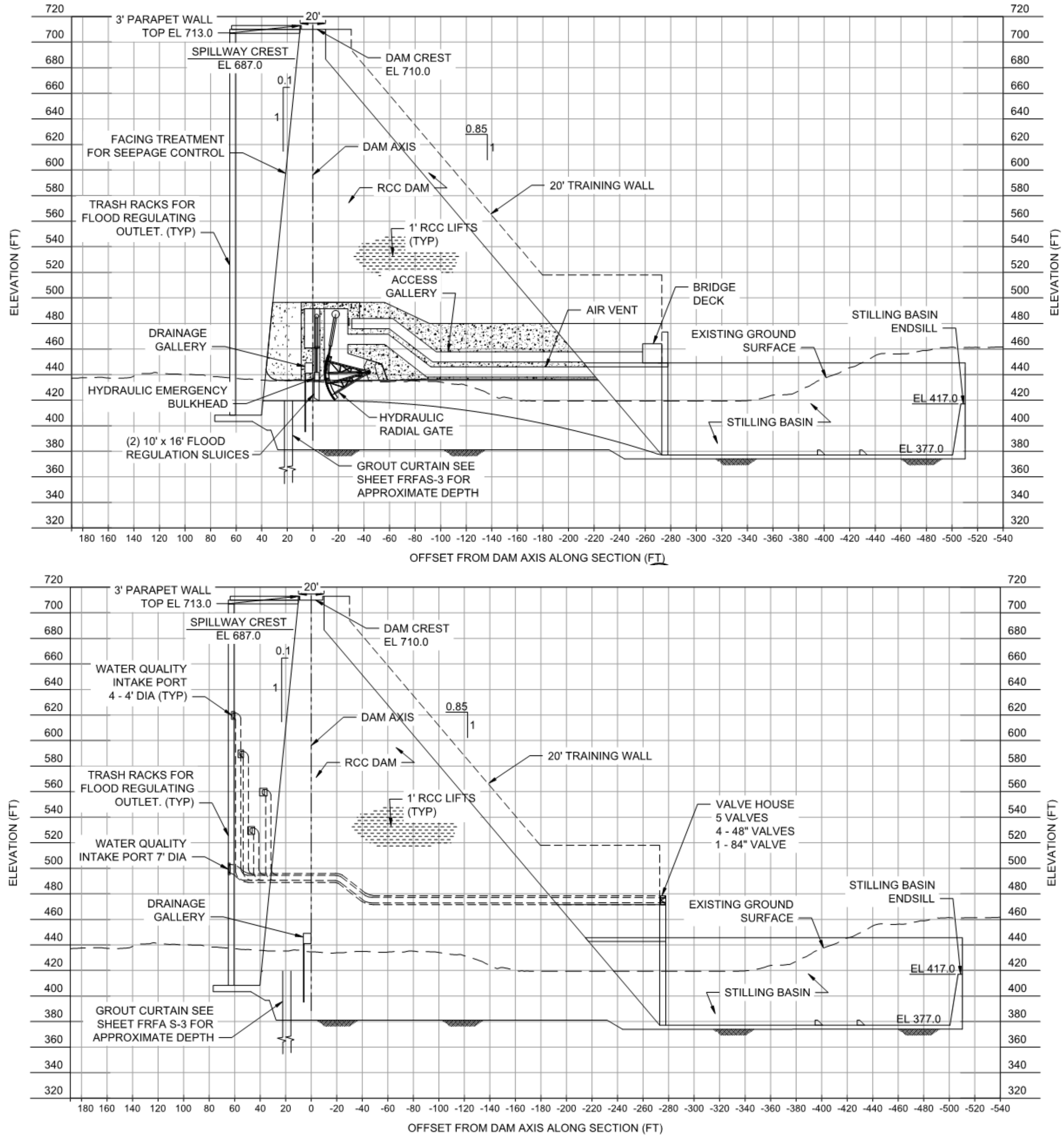


Figure 2-4
Section Through FRFA Dam Concept Sluices and Temperature Withdrawal Outlets



2.4 Design Criteria and Guidelines

2.4.1 Hydrologic Design Criteria

The hydrologic study performed by Watershed Science &Engineering (WSE; 2016b), coupled with the hydrologic modeling of flood storage attenuation by Anchor QEA (Anchor QEA 2014), forms the basis for hydraulic design for the FRO and FRFA design alternatives. The hydrologic design criteria that apply to both configurations are as follows:

- Project design flood event for the spillway is 69,800 cfs (NOTE: this value has been revised downward from 75,000 cfs since the previous estimates were developed)
- The FRO alternative is designed to temporarily store floodwater only during a flood larger than the estimated 7-year recurrence interval flood events, or whenever the downstream gage at Grand Mound is forecasted to rise above 38,000 cfs within 48 hours
- The FRFA alternative is designed to also temporarily store floodwaters only during a flood larger than the estimated 7-year recurrence interval flood events, in addition to the permanent storage of water for augmentation purposes
- Flood operations for either dam configuration would include storage of flood inflows in the reservoir and ramped release of those flows to evacuate the stored volume after the flood peak has passed
- Minimum release of 250 cfs will be maintained throughout any flood event requiring regulation

2.4.2 Hydraulic Design Criteria

2.4.2.1 FRO

- Maximum spillway discharge capacity is 69,800 cfs (NOTE: this value has been revised downward from 75,000 cfs since the previous estimates were developed)
- Maximum design reservoir elevation is 650 feet (msl) under PMF, with minimum one foot freeboard to dam crest
- Spillway crest elevation is 628 feet msl, necessary to provide up to 65,000 acre feet of flood storage volume below that elevation
- Dam crest elevation is 651 feet msl, preventing overtopping of the dam crest under PMF conditions
- Minimum flood storage reservoir elevation = natural riverbed elevation
- Maximum flood storage elevation with no spillway flow is 628 feet msl
- Low-level flood regulation sluice capacity must be at least15,000 cfs at maximum flood control storage reservoir elevation 628 feet msl, though this capacity can be provided at lower reservoir elevation

2.4.2.2 FRFA

- Maximum spillway discharge capacity is 69,800 cfs (NOTE: this value has been revised downward from 75,000 cfs since the previous estimates were developed)
- Maximum design reservoir elevation is 709 feet msl under PMF, with minimum one foot freeboard to dam crest

- Spillway crest elevation is 687 feet msl, necessary to provide up to 65,000 acre feet of flood storage volume between the flow augmentation reservoir elevation of 628 feet msl and the spillway crest
- Dam crest elevation is 710 feet msl, preventing overtopping of the dam crest under PMF conditions
- Minimum flood storage reservoir elevation is 628 feet msl
- Maximum flood storage elevation with no spillway flow is 687 feet msl
- Low-level flood regulation sluice capacity must be at least 15,000 cfs at maximum flood control storage reservoir elevation 687 feet msl, though this capacity can be provided at much lower reservoir elevation
- Maximum flow augmentation storage reservoir elevation is 628 feet msl, to provide 65,000 acre feet of storage below this elevation
- Typical minimum flow augmentation reservoir elevation is 588 feet msl (585 feet msl with climate change scenario), though full volume is achieved between elevation 425 and 628 feet msl (Anchor QEA 2016)

2.4.3 Hydraulic Design References

Federal agencies have well-established guidelines for developing the design of concrete gravity dams such as the RCC dam structure proposed for the Chehalis Flood Storage Dam project. The USACE and the U.S. Bureau of Reclamation (USBR) provide the most applicable and comprehensive design guidance for large concrete gravity dams. Though the Federal Energy Regulatory Commission provides additional dam safety guidance, they are not relevant to a non-hydropower dam and as a consequence are not referenced herein. Similarly, the Natural Resources Conservation Service (NRCS) provides additional guidance for the design of dams. However, the NRCS guidance focuses primarily on embankment dams and is not particularly relevant to the Chehalis Flood Storage Dam project, and therefore the NRCS guidance is also not referenced herein.

2.4.3.1 U.S. Army Corps of Engineers

The USACE has developed comprehensive design guidance in the form of Engineer Manuals (EMs) and Engineer Regulations (ERs) based on decades of experience and many empirical data sets collected at numerous projects around the United States. Those specifically used in this design evaluation of the dam hydraulic structures are indicated in Section 4.

2.4.3.2 U.S. Bureau of Reclamation

In addition to publishing numerous dam design texts and guidelines, the USBR has been a leader in developing and incorporating risk-informed dam safety and design methods and guidelines. As for the USACE guidance, the USBR guidance is based on many decades of direct experience and many constructed dam projects around the United States. Those specifically used in this design evaluation of the dam hydraulic structures are indicated in Section 4.

2.5 Hydrologic Conditions

2.5.1 Basin Hydrology

The hydrologic analysis supporting the development of the Chehalis dam alternatives was conducted by WSE. The following discussion summarizes their analysis and relevant inputs to the dam and hydraulic structures design. This information was provided in three cited sources (WSE 2014, 2016a, 2016b).

The Chehalis River Basin comprises approximately 2,200 square miles within the southwest portion of the State of Washington. The highest elevations of the basin lie within the Willapa Hills at less than 3,500 feet above sea level. In general, the basin is dominated hydrologically by a temperate climate with abundant fall, winter, and spring rainfall, with some occasional snowfall but relatively little consistent snowpack development, and drier, cool summers. Its hydrology is typical of low- to moderate-elevation lowland coastal basins of the northwest Pacific coast, with generous winter base flow and periodic high flows resulting from Pacific storms carrying heavy rainfall from the southwest. Orographic rainfall is significant in the higher elevations of the basin. Average annual rainfall varies from about 45 inches per year in the vicinity of Chehalis and Centralia and up to as much as 120 inches per year at the higher elevations in the Willapa Hills. The bedrock characteristics of the basin result in relatively high rainfall runoff and low groundwater contribution. The drier summer periods generate characteristically low to very low flows throughout the basin. Tributaries to the Chehalis River include the South Fork Chehalis, Newaukum, Skookumchuck, Black, Satsop, and Wynoochee Rivers.

Flood events on the Chehalis are generally the result of warm frontal systems moving across the basin from southwest to northeast. These systems typically drive warm, moisture-laden atmospheric “rivers” from the equatorial regions of the Pacific Ocean to the Pacific Northwest. Rainfall in excess of 12 inches per day can result from these storms. Historic flood events such as the flood of record, which occurred in December 2007, are typically the result of such atmospheric events. The 2007 storm generated rainfall amounts in excess of 1 inch per hour in some areas of the basin, and record amounts for 12- and 24- hour duration. Peak flows exceeded all previously recorded historic levels at a number of gage stations throughout the basin. In particular, the United States Geological Survey (USGS) has estimated the peak discharge at the Doty, Washington gage reached 63,100 cfs, which was more than twice the previous record flow. Farther downstream, however, the peak flows were lower, with the peak flow at Grand Mound exceeding the previous highest flow by only 6 percent. The December 2007 event has been estimated to be greater than a 100-year recurrence interval at the Doty gage (WSE 2016b). Anchor QEA has identified an approximate scale of flow recurrence intervals for various river discharges, as described in Table 2-1 below.

Table 2-1
Estimated Present-day Flood Recurrence Intervals

RETURN PERIOD (YEARS)	PEAK FLOW (CUBIC FEET PER SECOND)
2	6,920
10	13,061
20	16,053
100	24,223
500	35,688

Source: Anchor QEA 2016

Concurrent hydrologic analysis for climate change scenarios has been conducted by the University of Washington's Climate Change Group (UW 2016). They studied possible changes in peak flood flows generated by the Chehalis watershed under four different climate change scenarios using several different hydrologic models. Their results suggest that the peak 100-year recurrence interval flood flows could increase by 30 to 100 percent over the next century (Table 2-2). Under the climate change scenario, the maximum PMF flow would likely increase as well, though the anticipated increase in PMF flow has not been estimated. The spillway has been designed to provide significant freeboard for the current estimate of PMF flow, in anticipation of potential increased PMF flow under the climate change scenario. The spillway design would likely be revisited during the next phase of design once the predicted PMF event has been determined for the climate change scenario, and it may be necessary to increase the design capacity of the spillway to accommodate.

Table 2-2
Estimated Recurrence Interval for Future Flood Events – Existing vs. Climate Change

EST. RECURRENCE INTERVAL	PERCENT INCREASE UNDER CLIMATE CHANGE	PEAK FLOWS (CUBIC FEET PER SECOND)	
		EXISTING FLOOD FLOW	CLIMATE CHANGE SCENARIO FLOW CA. 2099
2-year	16%	6,920	8,027
10-year	35%	16,061	17,633
20-year	45%	16,053	23,276
100-year	66%	24,223	40,211
500-year	94%	35,688	69,234

Source: Table 4.2 from Anchor QEA 2016

2.5.2 Design Flood

The Spillway Design Flood (SDF) for the FRO and FRFA dams was required to satisfy the Washington State Department of Ecology's Dam Safety Office (DSO) criteria, based on their dam safety and risk evaluation methodology. DSO uses a risk-based analysis procedure based on the downstream hazard classification of the proposed dam. Considering the population at risk in the downstream corridor, the proposed dam would be classified as a "high-hazard-potential" structure. The DSO design criterion therefore uses a Step 8 inflow design flood for spillway hydraulic design, which corresponds approximately to the Probable Maximum Precipitation (PMP) with estimated recurrence in the range of annual exceedance probability = $1x10^{-4}$ to $4x10^{-5}$. Additionally, the USACE SDF criteria requires that the spillway capacity must accommodate PMF conditions generated using the PMP.

An RCC dam with sound rock in the foundation will not generally be susceptible to significant erosion from minor overtopping of the dam crest. However, for the Chehalis Dam SDF, the proposed dam is designed to safely pass all the flows up to the SDF within the spillway chute without any overtopping of the dam crest. The USACE SDF criteria specify that no reservoir routing attenuation of the inflow should be accounted for, hence we have selected 69,800 cfs as the SDF event. The spillway is designed to pass at least this SDF and up to a total of 75,000 cfs with no dam crest overtopping. It should be noted that WSE revised their estimate of the PMF event downward to 69,800 cfs from 75,000 cfs after preliminary design had been accomplished (WSE 2016b). The revised design capacity is reflected in the spillway dimensions presented in this report.

An uncontrolled crest was selected to simplify the dam spillway design. Since unregulated discharge would occur under any spill event with this design approach, the reservoir outlet works were designed by routing the present 250-year recurrence interval event through the flood storage capacity such that the full flood storage capacity was utilized up to the spillway crest. The routing analysis conducted by Anchor QEA (Anchor QEA 2016) determined that the flood control outlet works capacity should be at least 15,000 cfs under gate controlled conditions at a reservoir elevation of about 550 feet msl to realize full utilization of the design flood storage volume up to the spillway crest. The current design of the flood control outlet works includes three deeply submerged sluices controlled by radial gates in the FRO design, and two similarly sized deeply submerged sluices in the FRFA design. Gate controlled conditions require that the gate fully regulate the discharge with some reliable and repeatable degree of precision, which is assured at 75 percent of full gate opening or less.

2.5.3 Hydrologic Modeling of Flood Regulation Operations

Modeling of the reservoir operations was conducted by Anchor QEA, and is only briefly summarized here. More detailed information is provided in Anchor's report (Anchor QEA 2016). The reservoir operations were modeled using the USACE Hydrologic Engineering Center (HEC)'s Reservoir System Simulation (HEC-ResSim). HEC-ResSim can simulate single events or a full period of record. Reservoir properties used in the model include reservoir elevation vs. area curves (to determine storage volume), spillway crest elevation (628 feet msl and 687 feet msl, for the FRO alternative and the FRFA alternative, respectively), reservoir length and depth vs. capacity, outlet(s) elevation vs. capacity, and top-of-dam elevation. Only total controlled and uncontrolled outflow was routed through the model, so specific details such as the number of gates or flow through individual gates was not modeled. Concurrent reservoir routing modeling has been conducted by Anchor QEA to revisit these results using the current design rating curves for the flood control outlet sluices.

2.6 Hydraulic Modeling & Sediment Transport

Construction of a dam results in a modification to the hydraulic and sediment transport characteristics of a river. When water is impounded behind the dam, water velocities slow down causing deposition of suspended sediments. Coarser sediments tend to settle out first at the upstream end of the impoundment area. Finer sediments stay in suspension longer and can settle to the floor of the basin closer to the dam. Sediment-laden inflow tends to travel along the bottom of the reservoir toward the dam (beneath clearer upper level water). The design of the low level sluice structures through the dam and operational decisions on how and when to release flows can minimize sediment deposition within the basin by encouraging sediments to pass through the low flow structures (and to the river downstream from the dam).

The FRO dam configuration has low level sluice gates that will be open during normal low to moderate flow conditions. During these periods, sediment transport will be relatively unchanged from existing conditions as water and sediment will be allowed to pass freely through the dam to downstream. During high flow events when the gates are closed to impound water, sediment will be retained upstream of the dam. Some suspended sediments will likely be passed downstream as impounded flows are released, but some will likely be retained upstream of the dam. Some of the retained sediment will likely be re-suspended as the low flow gates are opened and the reservoir is evacuated following regulation of floods.

The FRFA dam will have a permanent pool which will tend to cause settlement of suspended sediment particles. During the winter months, however, suspended sediments will tend to pass through the low flow outlet due to the smaller winter reservoir volume and more rapid reservoir transit time. Passage of sediment could be further encouraged by releasing flow through the dam outlet structure at times when this lower elevation sediment-laden flow is entering the basin and approaching the dam. Experience at Howard Hanson Dam has shown that much of the suspended sediment load is generally passed through the dam, while bed load sediments are generally retained in the reservoir.

The effects of proposed dam alternatives on the hydraulic and sediment transport characteristics of the river reach in the vicinity of the project location were analyzed using computational modeling tools and presented in Section (2.6.1) through Section (2.6.3). As documented in the following sections, these modeling efforts included use of 1-dimensional clear water fixed bed analysis to calculate bed shear stress values, 1-dimensional sediment transport analysis to calculate expected scour and aggradation, and 2-dimensional flow evaluation of the resulting bed elevation as a fixed boundary following the sediment transport analysis.

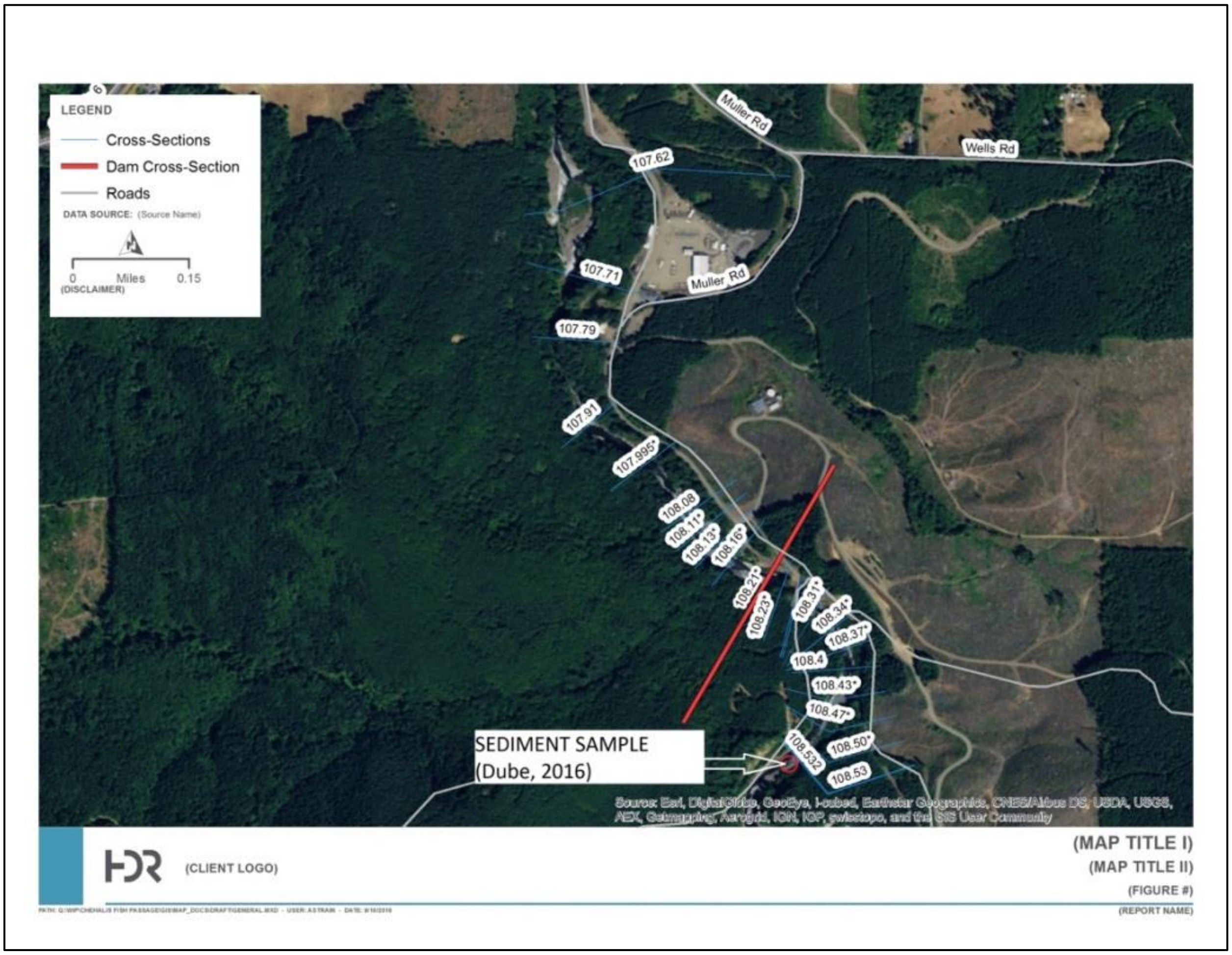
2.6.1 1D HEC-RAS Clear-Water Modeling

The hydraulic modeling analysis was conducted using a combination of tools, including analytical evaluation of outlet works capacity, velocity, gate operation, sediment throughput, as well as computational numerical modeling tools. The hydraulics through the dam reach was assessed using 1D

HEC-RAS, a one-dimensional computer water surface profile modeling tool created by the USACE HEC, and in common use throughout the engineering discipline for flow modeling in preliminary design evaluations. A HEC-RAS model originally created by WSE was updated with additional cross sections for the current proposed dam configurations throughout the reach from RM 108.532 to RM 107.62 using available Light Detection and Ranging (LiDAR) ground topography data (Figure 2-5 and Figure 2-6). Below-water bathymetry was approximated based on visual observations during low flows. Bed roughness was estimated based on visual comparison of the various bed and bank material to typical values for similar channels, materials, and flow depths found in the USGS guide for selecting appropriate Manning's 'n' values (USGS 1967). The model was calibrated roughly to observed water surface elevations at lower flows, while higher flows were calibrated to high water marks estimated from recent higher flood flows. The 2007 record flood event water surface elevations documented from site observations were used where possible by WSE to calibrate the original HEC-RAS model. Figure 3-1 and Figure 3-2 at the end of this Appendix illustrate the water surface profile results of the clear-water modeling for the Existing and With-Project conditions, respectively. Note that the With-Project Condition plot for the initial clear-water model run shows the stilling basin completely clear of sediment. It may take some time for the stilling basin to fill with sediment and reach an equilibrium transport throughput rate, but once filled, it should act as a reservoir of sediment, behaving much like the sediment deposits observed within the narrow bedrock channel in the vicinity of the proposed dam footprint. During flood events, the bed sediment scours within the existing narrow bedrock channel and then refills with bed sediment to varying degrees following the peak of the flood hydrograph, while suspended sediment and wash load passes completely through the reach. The stilling basin is expected to similarly scour and refill with bed sediment, while suspended sediments and wash load would pass through.

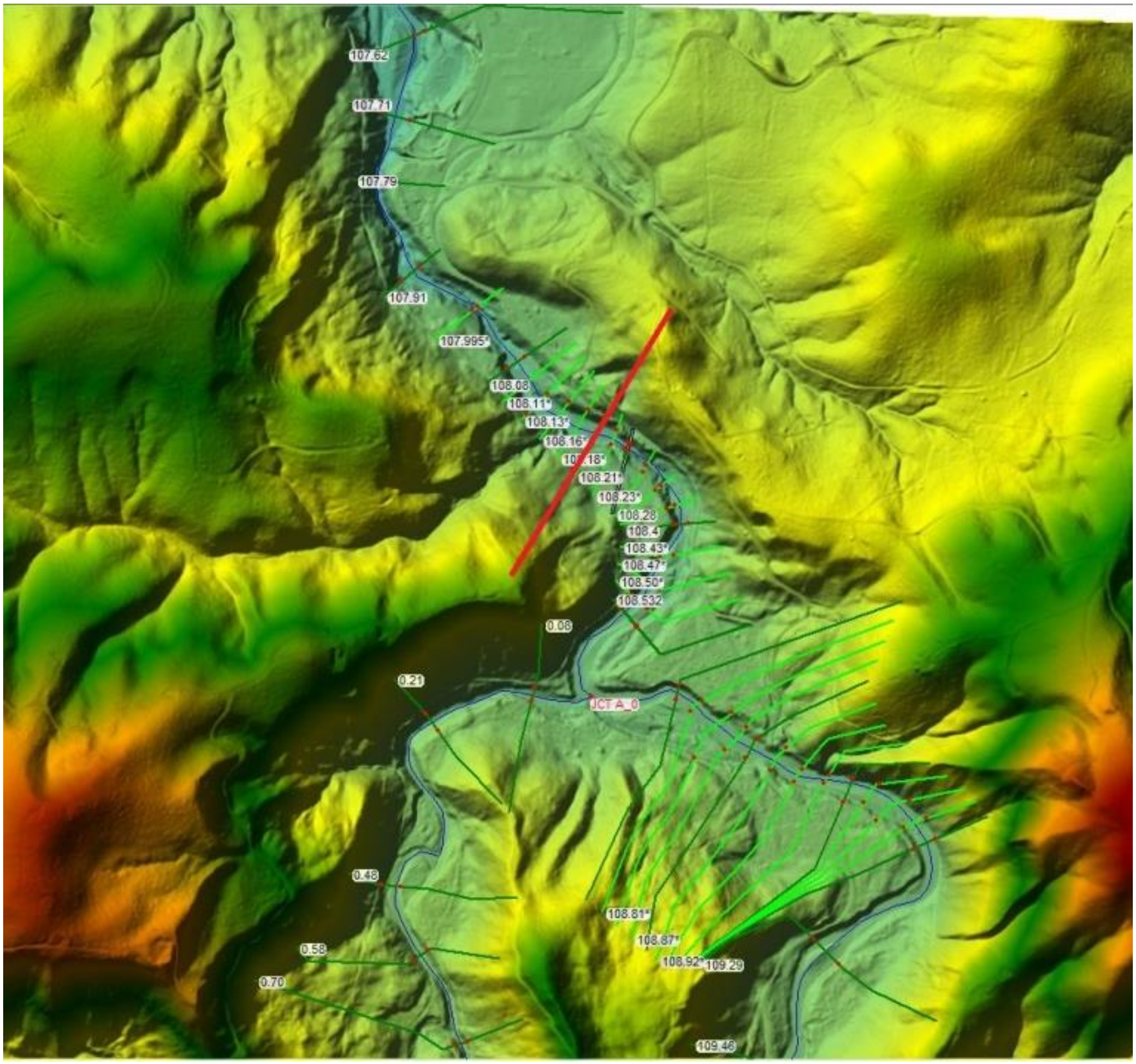
Figure 3-3 and Figure 3-4 illustrate the flow velocity results of the clear-water modeling for the Existing and With-Project conditions, respectively.

Figure 2-5
Sediment Sample Location, HEC-RAS Cross Sections



Note:
Dam axis shown as red line

Figure 2-6
LiDAR Topography with HEC-RAS Sections



Note:
Dam axis shown as red line

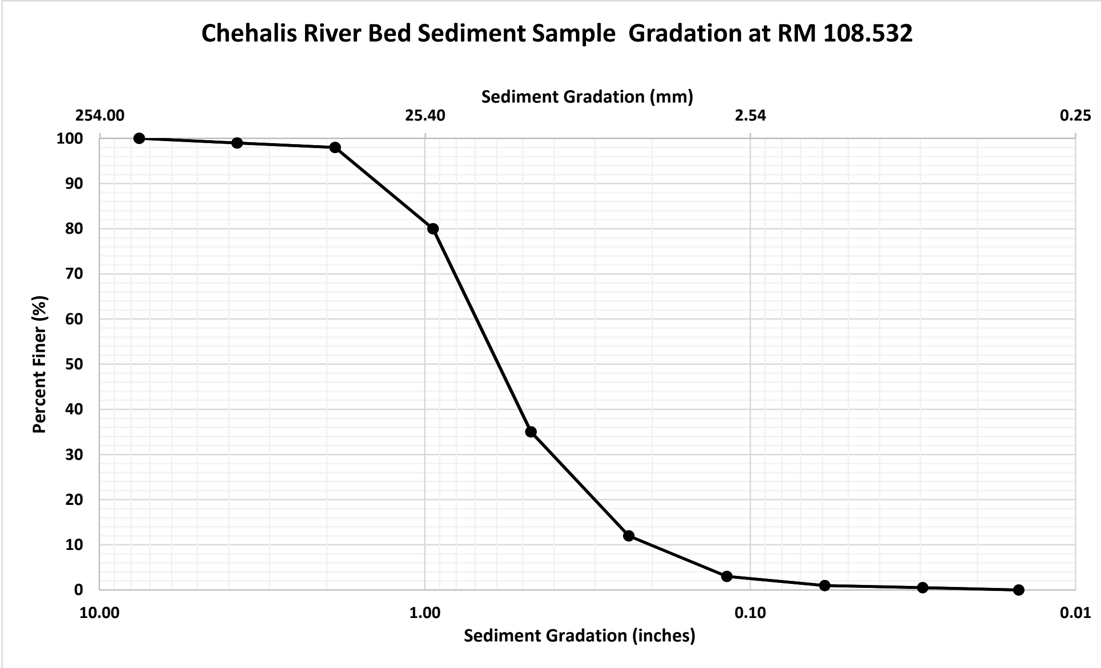
2.6.2 1D HEC-RAS Sediment Transport Modeling

Sediment transport through the dam reach was evaluated using the same 1D HEC-RAS model by activating the sediment transport module available in the software. The purpose of the sediment transport modeling was to compare the bed scour and aggradation behavior of the existing narrow bedrock channel to the narrow sluice conduits and stilling basin of the proposed dam. The objective was to assess and compare the hydraulic parameters important to sediment transport and fish passage for the existing channel with those of the proposed dam and determine if the overall river regime would be altered by the run-of-river dam operations. The model input parameters included a bed sediment sample gradation (Figure 2-) and river hydrograph. The sediment sample gradation was provided by others (Dube 2016), based on sampling data from gravel bars exposed in the vicinity of RM 108.532. The river hydrograph at the dam site was developed by scaling the observed hydrological records observed at the Doty gage using the proportional basin area. The hydrograph was comprised of several years of flow data for the dam site from 1990 to 1994 (Figure 2-). This period was selected for the analysis, as it comprised several larger flood events and the average annual hydrographs over these 4 years of record were fairly typical for the Chehalis River at the dam site. Some adjustments to the minimum bed elevations used in the model cross sections were made to reflect the hard boundary presented by the bedrock channel bathymetry throughout much of the dam reach where the existing channel drops through a bedrock pool and drop bedrock structure, and bedrock channel controls were estimated based on visual observations. A deep scoured trench through the reach immediately upstream of, and through, the proposed dam footprint was observed to have deep deposits of gravel and sand, and the water surface through this natural scour reach appears to be controlled by a downstream bedrock sill, located just downstream of the end sill of the outlet works stilling basin of the proposed dam. During higher flows, the analysis showed that this sediment will mobilize and scour will ensue, possibly to the bedrock limit at an unknown depth. It was estimated that the assumed bedrock elevation within this scour reach through a trial-and-error process to match approximate known water surface elevations observed at moderate to higher flows with an estimated scour depth necessary to generate the observed water surface elevation. No detailed ground penetrating radar or geophysical investigation was conducted through this scour reach in this phase of the study, and as a result we must consider the Existing Conditions Sediment model to be only roughly approximate, and only useful to compare against the With-Project Condition Sediment model which reflects the effect of flow and sediment passing through the proposed dam sluice outlets.

HEC-RAS offers several sediment transport function options for analyzing gravel bed rivers of a similar type to the Chehalis mainstem at the dam location. These include Ackers-White, Meyer-Peter-Mueller (MPM), and several others. The particular sediment gradation samples collected from the river channel and the stability of the MPM method in HEC-RAS suggested MPM would be the most appropriate. The inflow and outflow sediment loads were assumed to be in equilibrium for the purpose of these simulations for both the existing condition and the proposed condition with the dam in place and the proposed FRO operation with fully open gates. This assumption was based on Dube’s assessment that there was no strong indication that the reach was sediment-limited or, conversely, sediment-oversupplied. Additional variables adjusted during the HEC-RAS model construction are not mentioned here, but were modified slightly in both models to achieve a reasonable simulation of transport processes, as evident by the bed profile and sediment gradations observed through the reach.

Figure 2-7

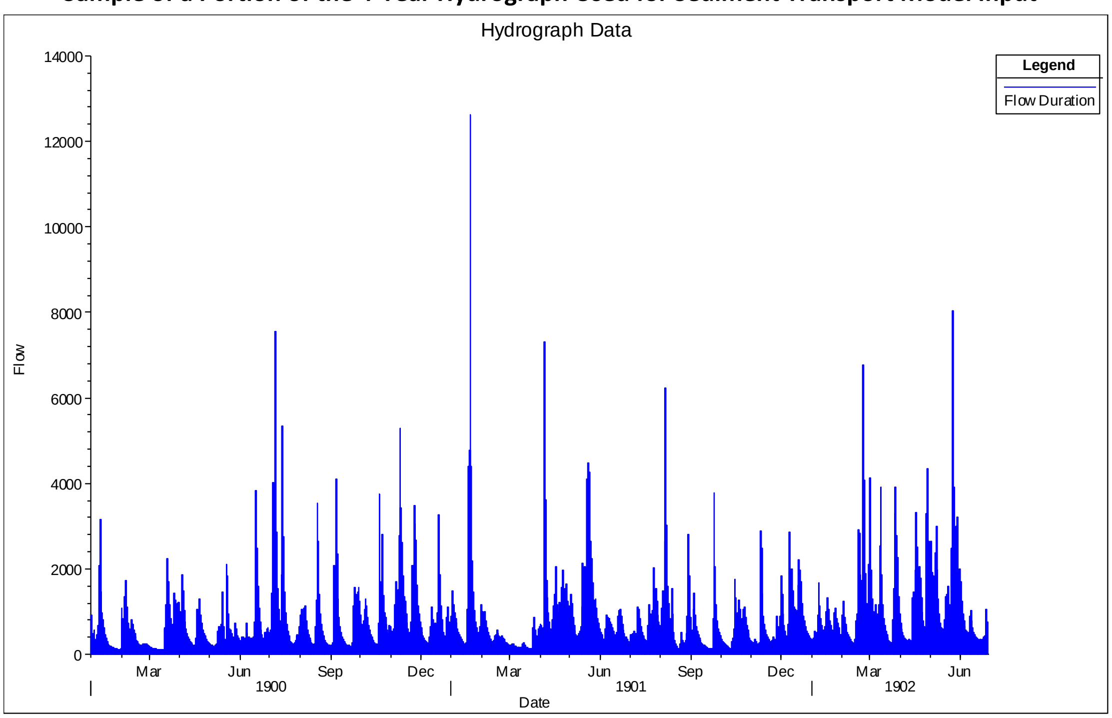
Sediment Sample Gradation Near HEC-RAS Cross Section 108.532 Used for Sediment Transport Model Input



Source: Dube 2016

Figure 2-8

Sample of a Portion of the 4-Year Hydrograph Used for Sediment Transport Model Input



Note:
Sediment model utilized 4 years of hydrologic record from 1990 to 1994

The With-Project Condition Sediment model also applied the same hydrograph used in the Existing Condition Sediment model. These two models allowed comparison of the transport characteristics and expected bed elevation at the end of the model hydrograph of the existing channel (Figure 2-) to that of the sluice outlets (Figure 2-) by mapping the resulting bed elevations. These bed elevation results were then input into the 1D HEC-RAS clear-water model (i.e., with the sediment transport module deactivated) to compare the flow depth and velocity for both the Existing Post Event Condition and the With-Project Post Event Condition. The primary focus of the comparison of hydraulic parameters was on flows within the range of fish passage season values from lowest flows to about 2,200 cfs. For example, the comparison between flow depth and velocity at cross section 108.28 roughly at the existing upstream face of the Tin Bridge and about midway through the proposed low level sluice outlets (Figure 2- and Figure 2-) shows that the expected flow depth in the sluice conduits will be very nearly the same depth as the existing channel, while the expected flow velocity in the sluice conduits will be less than that in the existing channel. According to the results of hydraulic modeling upstream and downstream of the proposed dam and associated features, no change in depth or velocity between the Existing and Proposed Conditions is expected.

Figure 2-9

Bed Profile Following Sediment Transport Hydrograph – Existing Post Event Conditions (1D HEC-RAS)

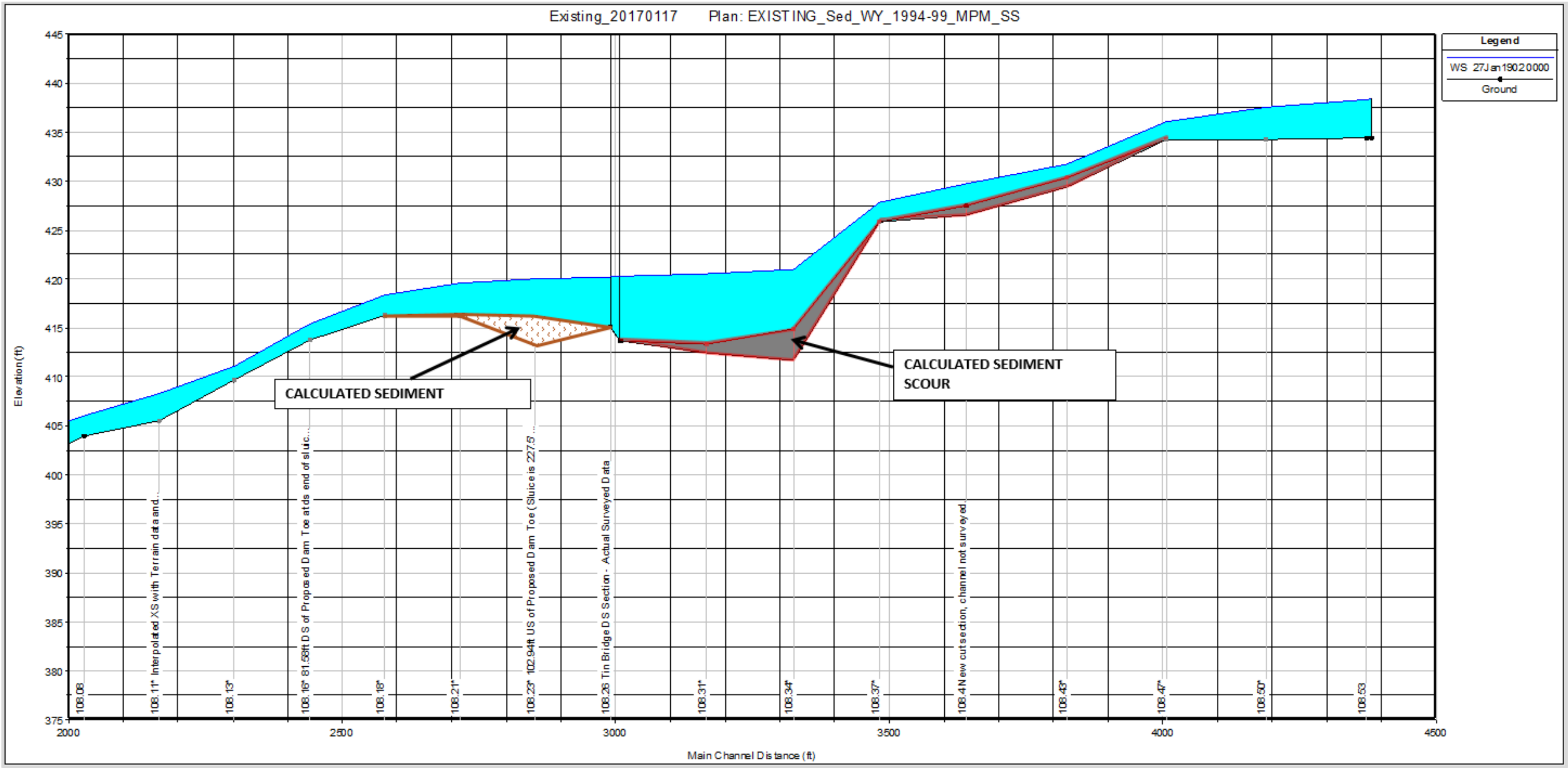


Figure 2-10

Bed Profile Following Sediment Transport Hydrograph – Proposed With-Project Post Event Condition (1D HEC-RAS)

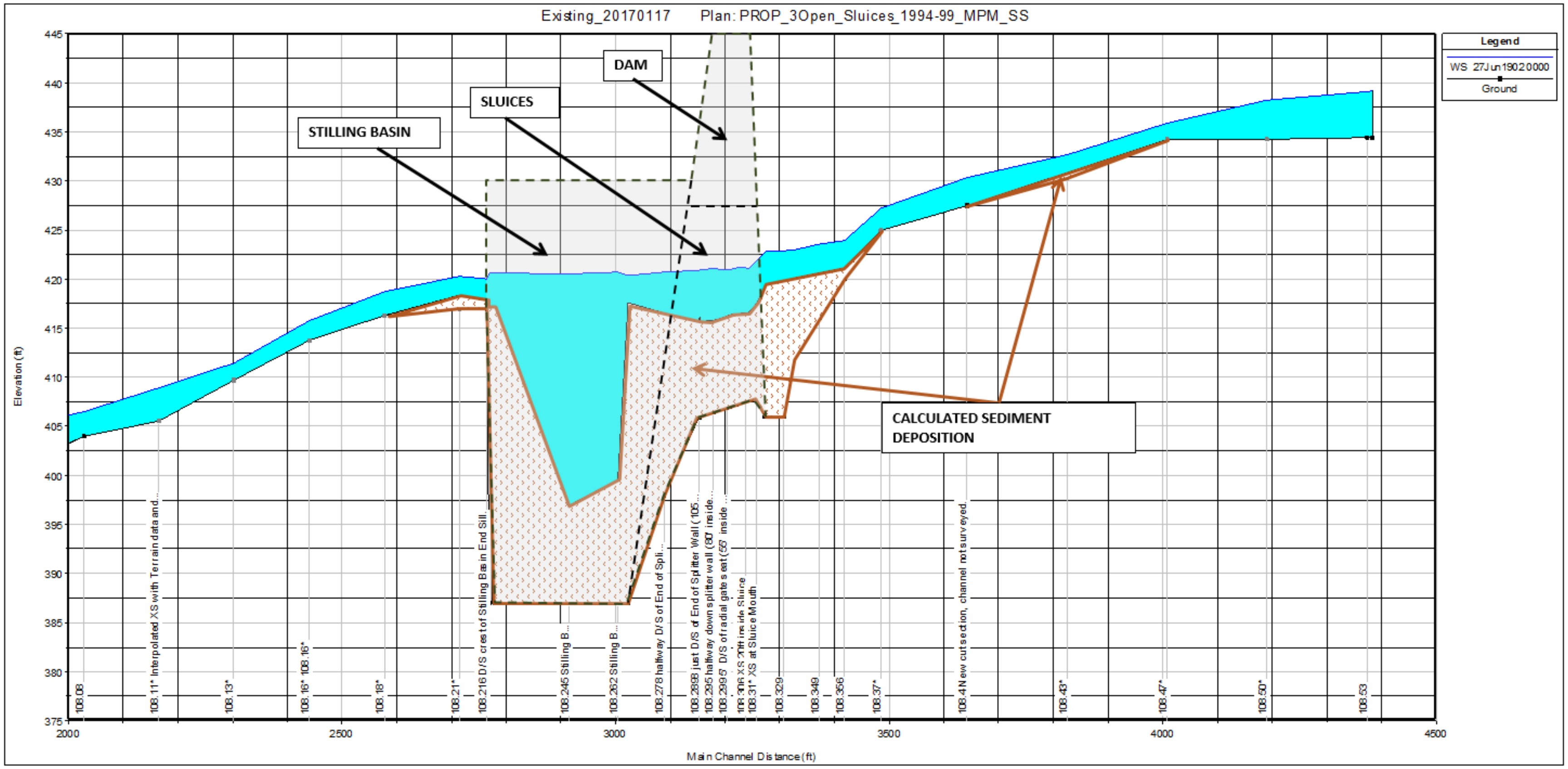


Figure 2-11

Comparison of Flow Depth Rating Curve Between Existing Condition and With Project Condition at About 18 Feet Downstream of the Proposed FRO Dam Toe Inside the Stilling Basin

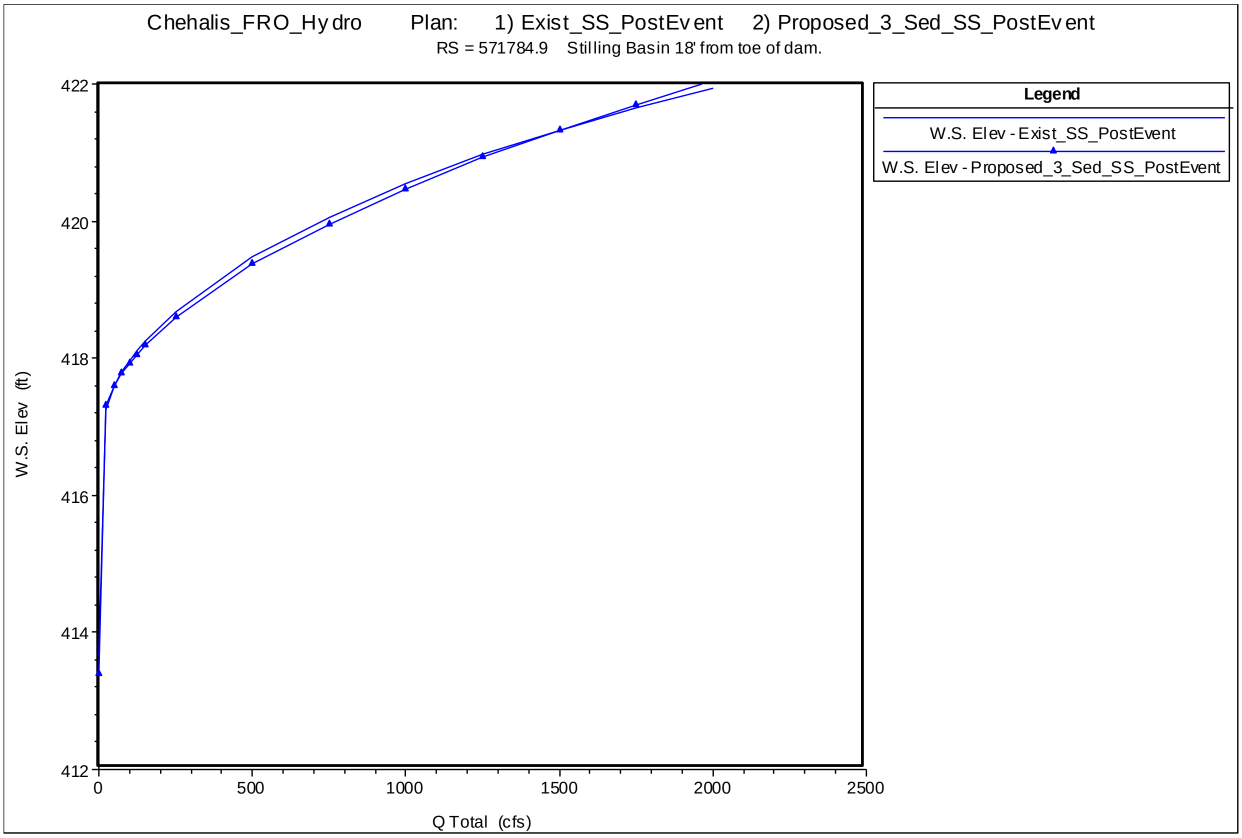
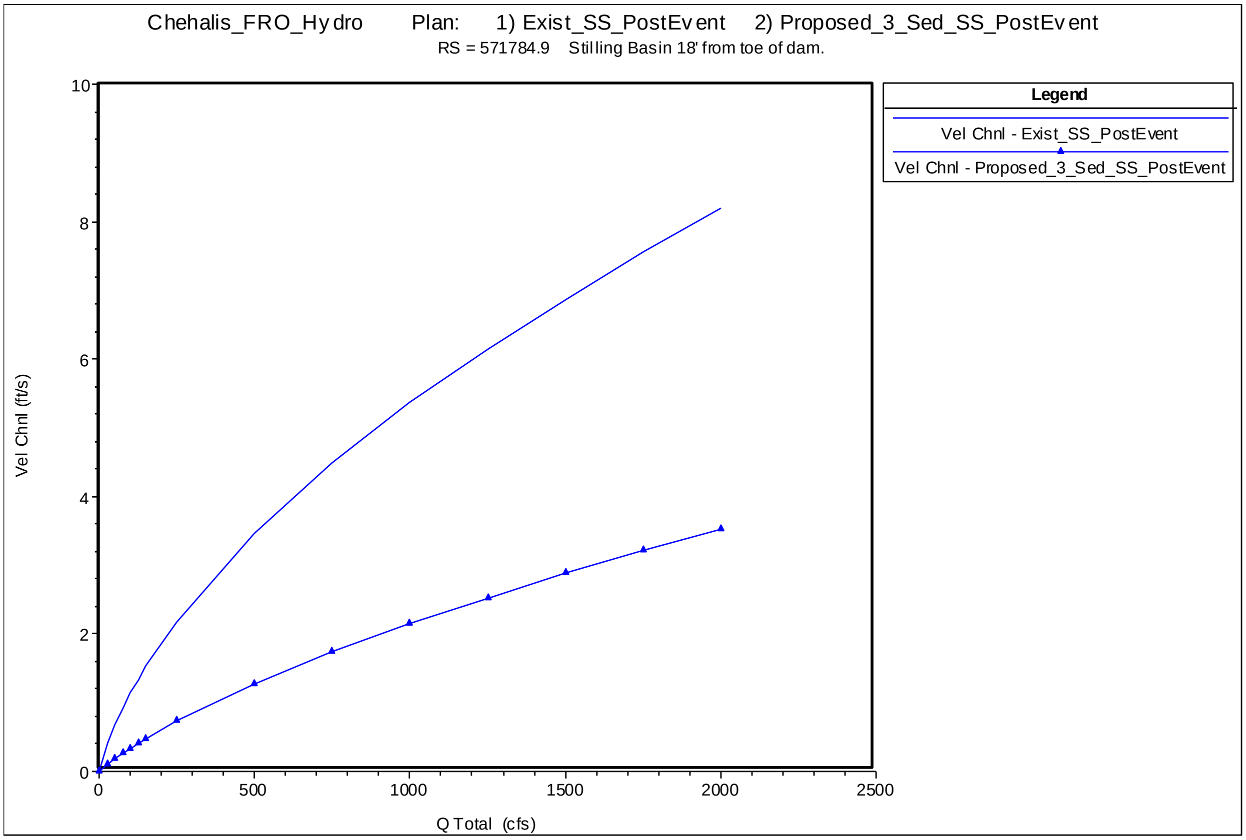


Figure 2-12
Comparison of Flow Velocity Rating Curve between Existing Condition and With-Project Condition at About 18 Feet Downstream of the Proposed FRO Dam Toe Inside the Stilling Basin

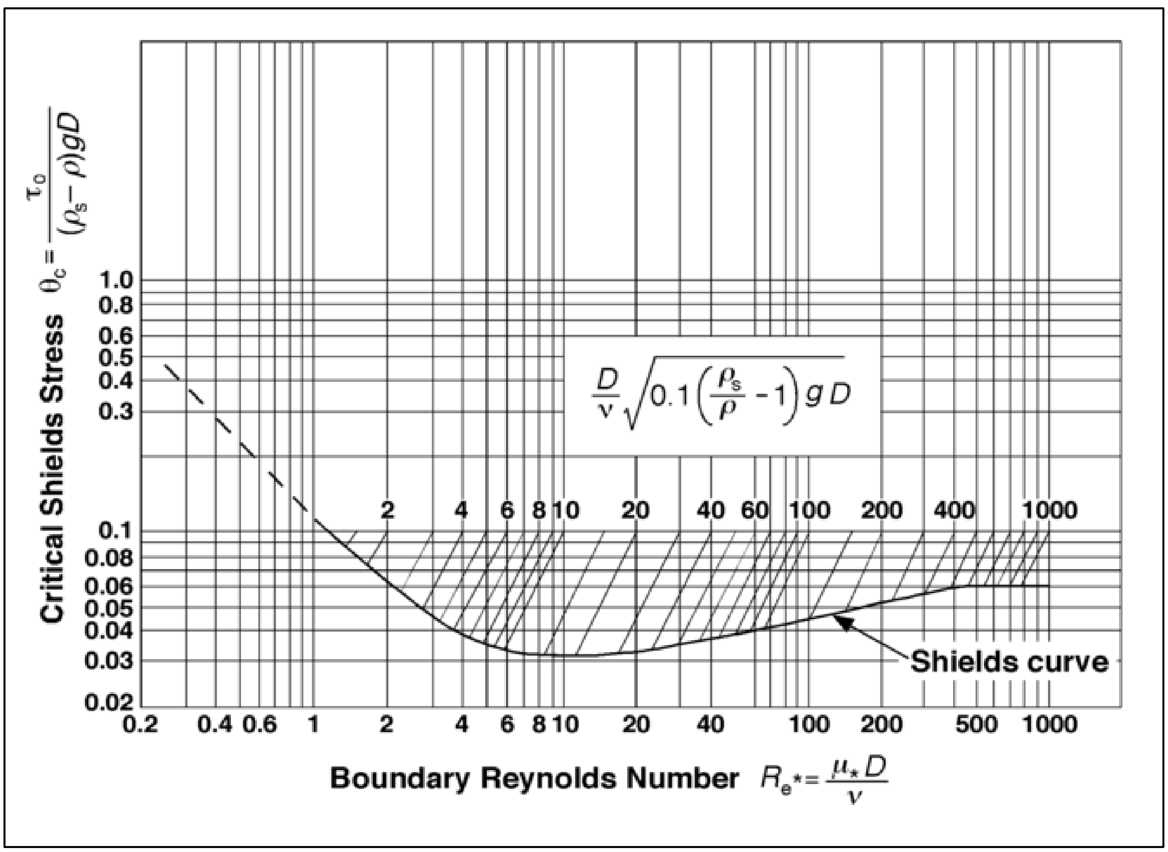


Also, 8 separate model runs were created for the With-Project Condition where the sediment hydrograph was extended 24 additional hours with a steady-state discharge (250 cfs, 500 cfs, 750 cfs, 1000 cfs, 1250 cfs, 1500 cfs, 1750 cfs, and 2000 cfs, respectively). The purpose of this artificial extension of the hydrograph was to evaluate the ability of sediment to clear from the sluice conduit outlets if specific flows continue for a period of time. The results of this separate analysis showed that bed sediment profiles vary throughout the sluice intake approach channel, through the sluice conduits, and in the stilling basin downstream, depending on the steady state discharge held at the end of the natural hydrograph.

The mobility of the sediment deposits would be driven by the transport capacity of the sluice conduit flow. Since the design approach for the sluice conduits was to try to match the same flow depth and flow velocity as the existing channel at all flows up to and including 2,200 cfs, these results are not unexpected. They show that bed sediment generally is not mobilized below 2,200 cfs in either the existing channel or the sluice conduits. It should also be noted that for flows above 2,200 cfs whenever the sluice gates would be used to regulate flow or orifice flow conditions would be attained at the sluice entrance, we calculated that the resulting high velocity jet would cause most of the sediment deposited within the sluice conduit to be swept into the stilling basin downstream.

Useful parameters for comparing the characteristics of the existing channel to the proposed dam and sluice outlets included bed shear stress, flow depth, and flow velocity. Bed shear stress is indicative of the discharge needed to initiate motion of the bed particles over the lower to moderate range of flows associated with typical fish passage periods. Flow depth and flow velocity are useful for evaluating the fish passage corridor characteristics through the sluice conduits compared to the existing channel. The bed shear stress values calculated from the HEC-RAS model were compared to the standard Shields shear stress diagram (Figure 2-**Error! Reference source not found.**) for incipient motion to determine whether sediment particles of several different size ranges would be transported or not. Comparisons of shear stress to the critical shear stress for motion of particle sizes of 0.5, 1, 3, and 6 inches for several typical cross sections are shown in Figure 3-5 through Figure 3-13 at the end of this Appendix. The results show that the calculated bed shear stress necessary to move particles larger than the D_{50} size range (about 0.6 inches) occurs at discharges higher than about 250 cfs in the steep natural bedrock step pool and cascade reach above the proposed dam site as well as the reach below the dam site.

Figure 2-13
Shields' Critical Shear Stress Diagram



Source: Henderson 1966

2.6.3 2D HEC-RAS Clear-Water and Sediment Transport Modeling

A second model was created using 2D HEC-RAS, a two-dimensional computer water surface profile modeling tool also created by the USACE HEC. This was used to evaluate the flow patterns in the natural channel and through the large low level sluices of the FRO Dam alternative over a range of flows up to an including the maximum fish passage flow of 2,200 cfs. The primary purpose of the 2D modeling is to provide more detailed flow pattern information that could further inform sediment transport and fish passage conditions. A physical scale model using mobile bed sediments would be capable of more accurately assessing the sediment transport characteristics for the Existing Condition and the With-Project Condition. In fact, a future physical scale model study will likely result in refinement of the sluice configuration to meet the objectives of the natural regime process continuity.

In the 2D model, the sediment transport module was activated to evaluate the specific deposition and scour patterns throughout the natural channel and proposed FRO dam and sluice conduits and compare to the 1-dimensional results provided by the 1-D HEC-RAS model. A range of flows were run in the 2D sediment transport model that covered the anticipated flows during the fish passage season, from 25 cfs through 2,250 cfs, with the trailing steady state flow as described above. Example comparisons between the 500 cfs and 1,500 cfs steady state trailing flow bed contour plots resulting from the 2D modeling are shown in Figure 2- and Figure 2- **Error! Reference source not found.**below. These plots show the approach channel, the sluice conduits, and the stilling basin.

The 2D modeling results showed that the hydraulic and sediment transport characteristics of channel reaches upstream and downstream of the proposed dam would be unchanged. Similarly, the predicted bed elevation contours throughout the approach channel, sluice conduits, and stilling basin following the 2D sediment transport model simulation for the multi-year hydrograph are illustrated in Figure 3-14 through Figure 3-21 at the end of this Appendix. The resulting flow velocities through the proposed dam sluice conduits following the sediment hydrograph and trailing steady state are plotted as velocity contours for the 8 flows listed above in **Error! Reference source not found.** through **Error! Reference source not found.** at the end of this Appendix.

Figure 2-14
Plot of Bed Contours for 500 cfs Trailing Steady State Flow

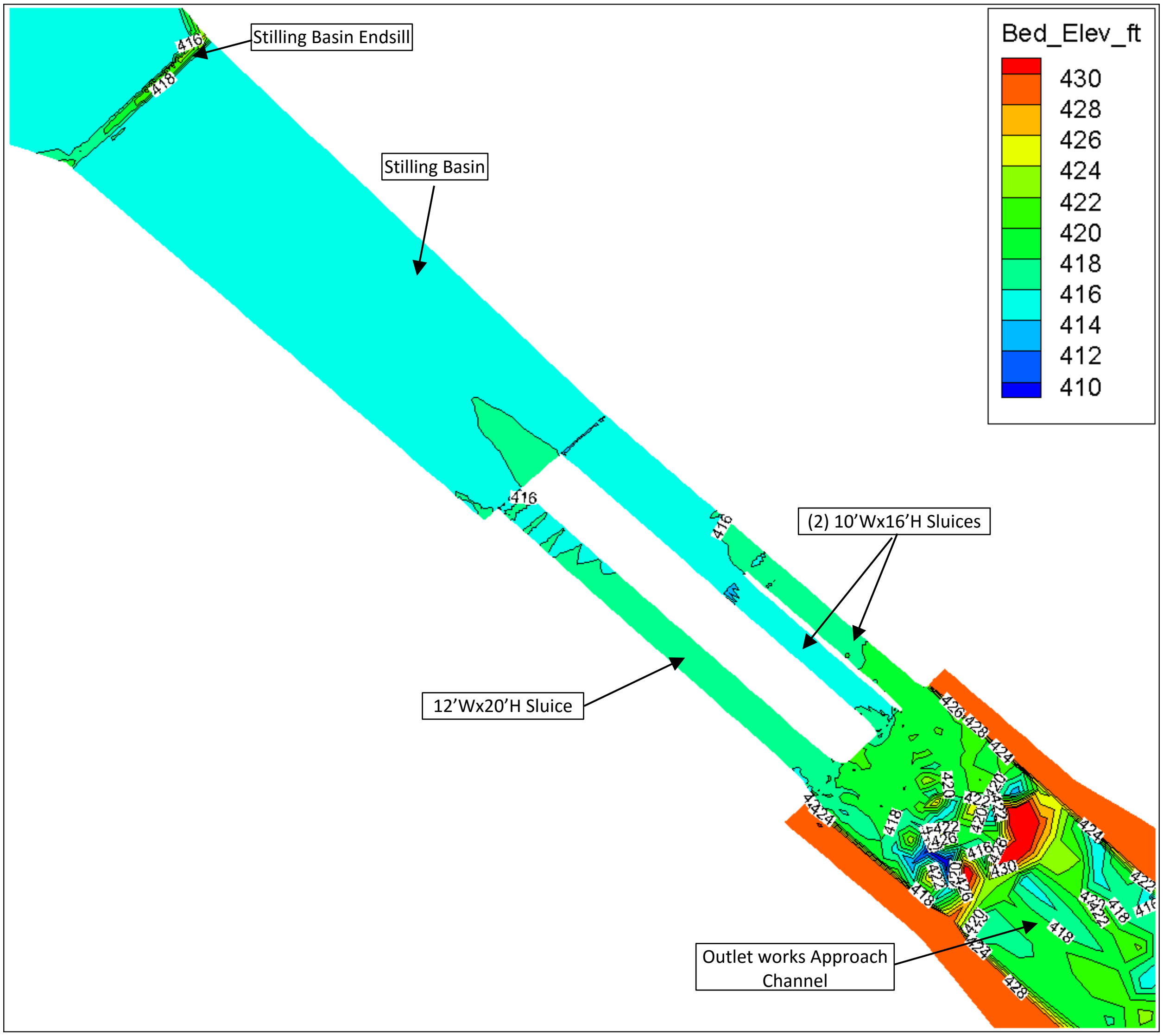
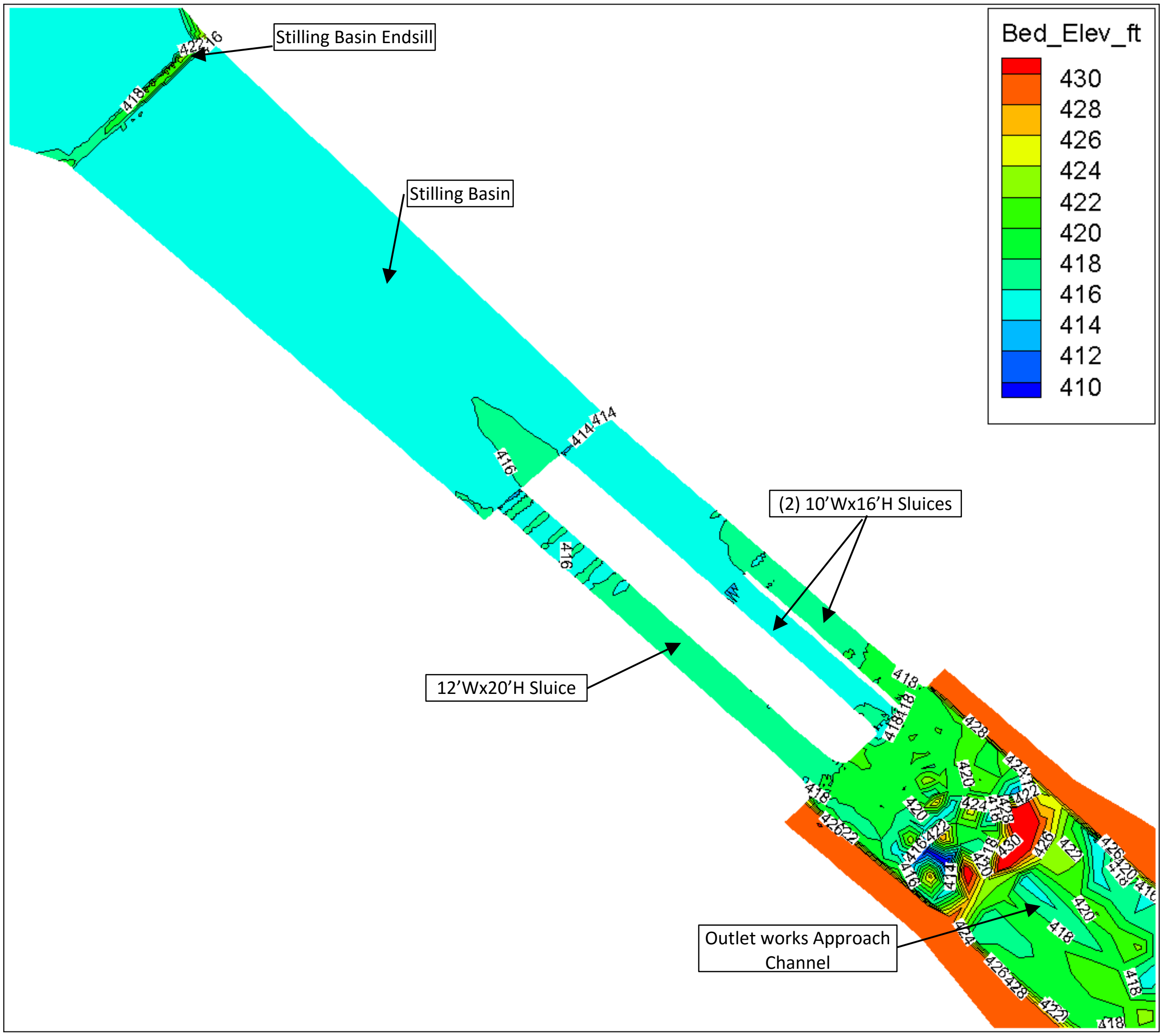


Figure 2-15
Plot of Bed Contours for 1,500 cfs Trailing Steady State Flow



2.7 Fish Passage

The FRO Dam alternative is proposed to permit unimpeded fish passage upstream and downstream through the large low level sluice conduits, achieved by holding the sluice gates fully open under all flow conditions except when anticipated flood discharge is forecast to increase above the specified 38,000 cfs threshold at the Grand Mound gage, as discussed above. At and above this threshold, the low level flood regulating sluice gates would be closed as needed to store flood water in the reservoir. When the low level flood regulating sluice gates are closed or under operation, a fish ladder and trap and truck facility would commence operation to collect fish from the dam stilling basin and move them upstream as needed. No downstream migrant fish collection facilities are proposed for the FRO dam alternative.

The FRFA Dam alternative is proposed to collect upstream migrating fish using a fish ladder and trap and truck facility located downstream of the dam adjacent to the stilling basin, while downstream migrating fish would be collected using a floating collector in the reservoir, then trucked downstream to be released into the river. Both the FRO and FRFA upstream migrating fish facilities are similar in size and configuration, as discussed in the main body of the Combined Dam and Fish Passage Conceptual Design Report. The remainder of this text will focus on the FRO Dam alternative fish passage only.

Early design stages for the FRO dam alternative included the modeling of several different sizes of sluice conduits, each comprised of a pair of sluices sharing a common downstream outlet conduit. These alternative configurations included Alt. 1 - two 12 foot wide by 20 foot high sluice conduits, Alt. 2 - two 14 foot wide by 24 foot high sluice conduits, and Alt. 3 - two 16 foot wide by 28 foot high sluice conduits. Initial evaluation included a simple HEC-RAS model analysis of flow velocity and depth, assuming that the downstream controlling water surface elevation was established at the stilling basin end sill, which was assumed to be at the same elevation as the natural river channel bed at that location. The HEC-RAS results for flow velocity and depth were compared to the existing river channel characteristics and to the fish passage criteria of 2 feet per second flow velocity maximum across the full range of fish passage discharges from 25 to 2,200 cfs. The sluice sizes and invert elevations were refined until the criteria could be met under the clear water condition (no sediment transport considered).

Based on the results from the generated velocity plots during this refinement process, the various alternative configurations were expanded to include options for three and five sluices, with two (or four) of the sluices designed as pair(s) of two sluices discharging into a common downstream discharge conduit, and the additional sluice as a single large outlet at a slightly lower elevation. This concept of a slightly lower, larger sluice gate and conduit was based on the Mud Mountain Dam analogous outlet works, where the lowest sluice intentionally passes the majority of bed load sediment in order to isolate erosion damage to a single outlet that can be readily repaired. This refinement process landed on a preferred configuration of a single large 12 foot wide by 20 foot high sluice at invert elevation 408.0 feet (msl), and a pair of 10 foot wide by 16 foot high sluices at invert elevation 411.0 feet (msl).

These three and five conduit options are the same discussed in the preliminary sediment transport section. The same models that were run through HEC-RAS for sediment transport analysis were used to determine velocities through the proposed fish passage conduits, comparing once again the three and five conduit options to the velocities through the existing channel geometry. During the course of design progress, given that the existing stream channel upstream and downstream of the proposed dam does not meet the stringent fish passage criteria, the Washington Department of Fish & Wildlife modified the criteria to be no higher than the existing channel. The agencies had determined that the criteria of 2 feet per second was no longer necessary and that simply matching the preexisting river channel conditions would be sufficient.

Comparison between the three and five conduit options with the existing channel conditions revealed that the three conduit option with a single 12 foot by 20 foot conduit at invert elevation 408 feet and two 10 foot by 16 foot conduits at elevation 411 feet would best meet both the fish passage criteria and the flood control outlet design discharge.

The refined sluice outlet configuration for the FRO dam alternative was modeled using the HEC-RAS 2D model to examine more detailed velocity and depth characteristics, as discussed above in Section 2.6. In addition, the 2D HEC-RAS model sediment transport module was activated to evaluate potential sediment deposition and scour patterns that might be expected through the low level outlet sluices, the approach channel, and the stilling basin following the multi-year hydrograph simulation discussed above. **Error! Reference source not found.** Figure 3-22 through Figure 3- at the end of this Appendix illustrate the bed elevation contours in the approach channel, through the low level sluices, and in the stilling basin predicted following the sediment transport simulation for the multi-year hydrograph.

2.8 Construction Diversion

Construction diversion design was initially evaluated in the previous studies, and has been revisited in this phase of the project evaluation. The initial reconnaissance study selected a 20 foot diameter diversion tunnel. For this analysis, we have assessed the adequacy of that tunnel dimension to carry anticipated diversion flows over the estimated two to three year construction period. Upstream and downstream construction diversion cofferdams considered the recurrence interval of flood events that might occur during the construction period, and a preferred cofferdam elevation was selected based on the calculated diversion tunnel capacity, as discussed below.

2.8.1 Diversion Tunnel Flow Capacity

A diversion tunnel will carry water around the project site during the construction phase. It is assumed that the tunnel will be a 20 foot modified horseshoe-shaped configuration (developed during previous design evaluations). This tunnel dimension was determined to be a practical size that could be cost effectively advanced using drill and blast techniques and conventional mining equipment. An upstream cofferdam would direct upstream water into the diversion tunnel causing flows to bypass the construction site. The use of this diversion would minimize construction delays due to runoff, improve construction site safety, minimize damage potential to the foundation area under construction, and reduce construction cost increases due to damages and delays. A range of upstream diversion cofferdam elevations and compositions were evaluated. Tunnel capacity calculations showed that free surface flow and critical depth control at the inlet will dictate the discharge through the tunnel for low river elevation upstream of the cofferdam up to about 445 feet msl. For this condition, the tunnel discharge was calculated using the critical flow formula as follow:

$$Q_{cr} = \sqrt{\frac{gA^3}{T}}$$

where Q_{cr}= critical flow in cfs

A= flow area in square feet

T= flow top width in feet
g= 32.18, gravitational acceleration in feet per second squared

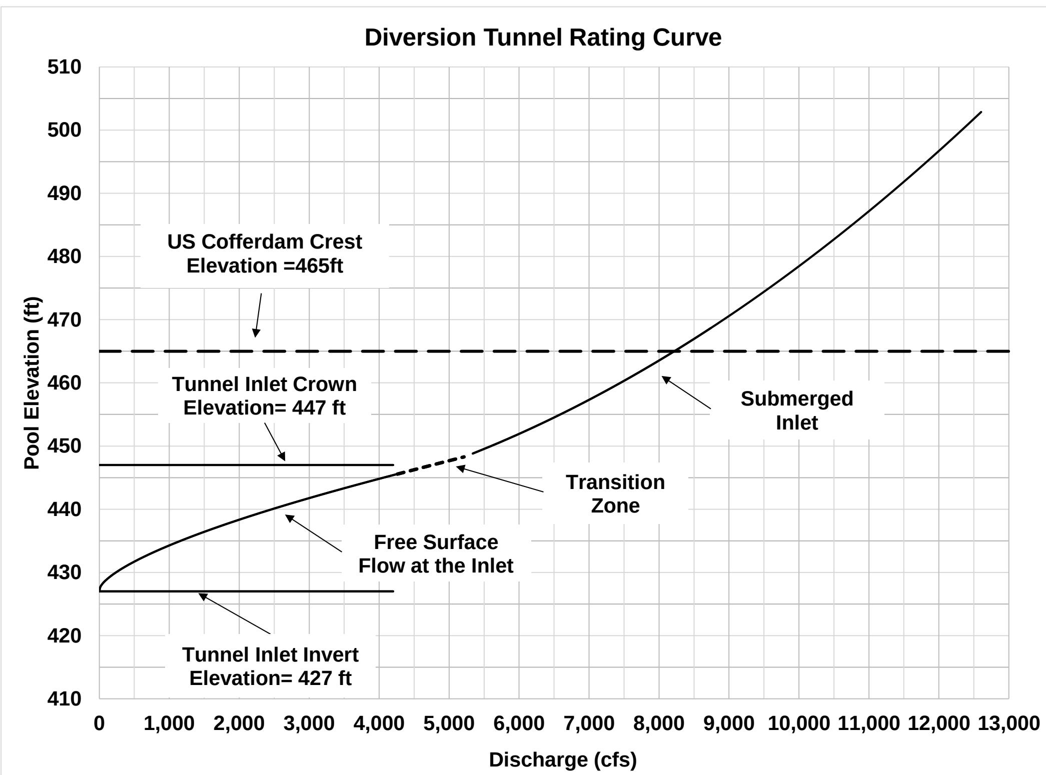
When the river elevation is between 445 feet msl and 449 feet msl, the flow condition is in transition zone from free surface critical depth control at the inlet to submerged inlet and orifice flow condition. Beyond the river elevation of about 449 feet msl, the inlet will be submerged and the flow through the tunnel can be calculated using the orifice equation as follow:

$$Q = C\,A\sqrt{2gH}$$

where Q= discharge in cfs
C= discharge coefficient
A= orifice area in feet squared
g= 32.18, gravitational acceleration in feet per second squared
H= energy head over the opening in feet

Figure 2- shows the diversion tunnel inlet rating curve calculated for different river elevation upstream of the cofferdam.

Figure 2-16
Construction Diversion Tunnel Inlet Rating Curve



2.8.2 Diversion Level of Protection Determination

Varying the upstream cofferdam elevation provides different degrees of protection from flooding during construction. Table 2- indicates the flows that can be accommodated through the diversion tunnel (with 3 feet of freeboard on the upstream cofferdam) by varying the upstream elevation. The upstream cofferdam crest elevation of 465 feet msl was selected for conceptual design to provide up to 7,800 cfs diversion capacity through a partially to fully lined construction diversion tunnel. This corresponds to approximately a 2.8-year recurrence interval flow event.

Table 2-3
Flow Recurrence Interval Table for Upstream Cofferdam (Update)

RIVER DISCHARGE (CFS)	RECURRENCE INTERVAL (YRS)	RIVER WS ELEVATION AT U/S COFFERDAM (FT, MSL)	U/S COFFERDAM ELEVATION (FT, MSL)
12,502	10	501.8	504.8
9,870	5	477.4	480.4
6,580	2	452.1	455.1
4,606	1.25	446.6	449.6

Notes:
3 feet of freeboard assumed
CFS = cubic feet per second
ft, msl = feet mean sea level
u/s = upstream
ws = water surface
yrs = years

A much smaller downstream cofferdam would similarly be constructed to protect the construction area for the stilling basin and fish collection channel. However, because the downstream cofferdam is well downstream of these features, and the receiving river reach is fairly steep, the downstream backwater conditions are not expected to overtop this cofferdam elevation into the construction area. For example, a similar 2-year-recurrence interval level of protection with 3 feet of freeboard requires a cofferdam elevation of just 416.35 feet msl, only 6 feet above the riverbed elevation at the tunnel portal. Table 2-4 below shows approximate downstream cofferdam elevations and flow recurrence interval. The downstream cofferdam crest has been set at 435 feet msl, more than high enough to prevent backwater from the construction diversion tunnel discharge from entering the dam foundation construction area, but also high enough that it can be used as an access road across the river channel to reach the left bank construction area.

Table 2-4
Flow Recurrence Interval Table for Downstream Cofferdam

RIVER DISCHARGE (CFS)	RECURRENCE INTERVAL (YRS)	RIVER WS ELEVATION AT D/S COFFERDAM (FT, MSL)	D/S COFFERDAM ELEVATION (FT, MSL)
12,502	10	417.16	420.16
9,870	5	415.65	418.65
6,580	2	413.35	416.35
4,606	1.25	412.32	415.32

Notes:
3 feet of freeboard assumed
CFS = cubic feet per second
d/s = downstream
ft, msl = feet mean sea level
ws = water surface
yrs = years

RCC was identified as the material that would most appropriately balance cost and durability considerations for the upstream cofferdam. An upstream cofferdam built from RCC would be resistant to erosion in the event that flows during construction exceeded the capacity of the diversion tunnel. The RCC option would also provide better access into the excavated footprint. The cofferdam and diversion tunnel capacity will meet the risk-based design approach used by the USBR as well as the requirements of the DSO. Overtopping of the downstream cofferdam presents less of a risk since inundation of the berm toe would not result in cofferdam failure. Because the downstream cofferdam would be much smaller, rockfill could be more cost effective than RCC.

2.9 Spillway Design

The FRO and FRFA alternatives include an uncontrolled ogee crest spillway, discharging to a smooth-faced conventional concrete chute cast over the top of the RCC mass dam section. The spillway provides safe conveyance from the reservoir to the river channel downstream of the dam for the SDF, though the low level sluices will be the primary flood flow release facility and the spillway will rarely be used. Once the spillway crest is engaged, the low level sluices will be closed to protect them from damage and all flows would be passed over the spillway. Design guidance utilized in the design of the spillway included USACE EM 1110-2-1603, Hydraulic Design of Spillways (1992); the USACE Hydraulic Design Criteria (HDC; 1987); and the USBR Design of Small Dams (1987).

An ogee spillway was considered as the best option for the FRO and FRFA alternatives since it provides high discharge efficiency and a safe and low maintenance spillway structure. The shape of ogee spillway crest is a close approximation of the jet trajectory as water spills over a sharp-crested weir. The crest shape can be defined using the design head, H_d. It should be noted that the design head of spillway can be greater than, equal to, or less than the effective head H_e, which is the sum of actual water depth over the spillway crest and approach velocity head. The spillway crest shape shown in Figure 2-17, below, is used for both the FRO and FRFA designs, assuming the design head is the same for both. The upstream quadrant of the crest (the portion of the surface upstream of the crest centerline) consists of three circular arcs. The downstream quadrant of the crest (the portion of the surface downstream of the crest centerline) is defined by the following equation:

$$X^{1.85} = 2H_d^{0.85}Y$$

where X = horizontal coordinate in feet

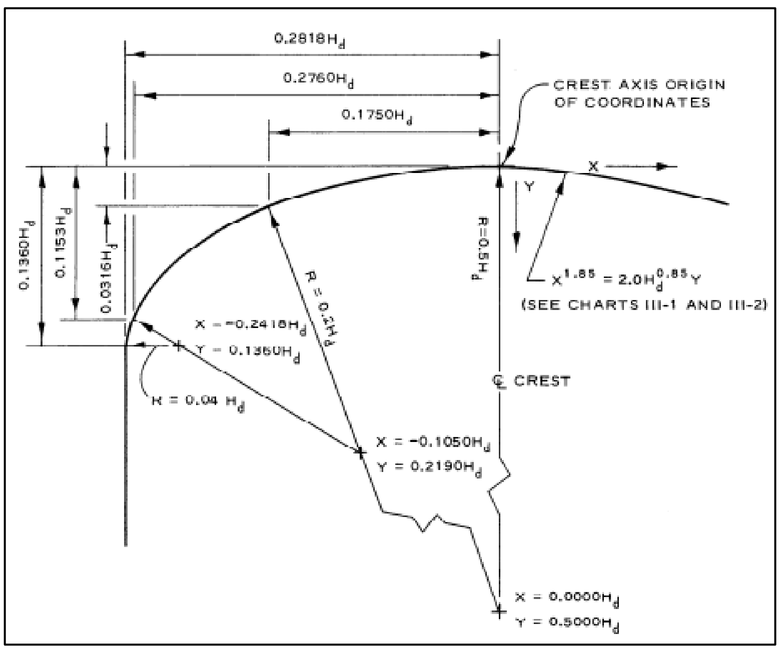
Y = vertical coordinate in feet

H_d = design head in feet

The downstream quadrant equation is used to define the spillway shape down to the point of tangency where the slope of the quadrant is equal to the slope of downstream face of the dam. The location of point of tangency can be determined either analytically or graphically.

Figure 2-17

USACE Hydraulic Design Criteria 111-2/1 Design of Ogee Crest Shape



Note:
'H_d' is the Design Head for the spillway; 'H_e' is the Effective Head for the spillway

The discharge over a free flow uncontrolled spillway crest is computed using the following equation:

$$Q = CL_eH_e^{1.5}$$

where Q= discharge in cfs

C= discharge coefficient

L_e = effective length of the spillway crest in feet
H_e = effective head in feet

The net length of spillway crest is computed using the following equation:

$$L = L_e + 2(NK_p + K_a)H_e$$

where
L= net length of spillway crest in feet
L_e= effective length of spillway crest in feet
N= number of piers
K_p= pier contraction coefficient
K_a= abutment contraction coefficient
H_e= effective head on the spillway in feet

The energy loss, flow profile, velocity and depth over the spillway can also be determined. For flow with fully developed turbulent boundary layer over the spillway, the widely used methods such as Darcy-Weisbach, Chezy, and Manning equations can be used. However, when the turbulent boundary layer is not fully developed, it greatly affects the spillway energy loss and conventional aforementioned methods are not valid anymore. To calculate the turbulent boundary layer development energy loss, the turbulent boundary layer thickness must be computed using the following equation:

$$\frac{\delta}{L} = 0.08(\frac{L}{k})^{-0.233}$$

where
δ=turbulent boundary layer thickness in feet
L= length along the spillway in feet
k= effective roughness in feet

The flow velocity and depth at any location along the spillway can be determined from the following equation using a trial and error method:

$$h_T = d_P \cos\theta + \frac{u^2}{2g}$$

where
h_T= reservoir elevation minus spillway elevation at location T in feet
d_p= potential flow depth at location T in feet
θ= interior angle between spillway face at location T and horizontal in degree
u= potential flow velocity in feet per second
g=32.18, gravitational acceleration in feet per second squared

The spillway energy loss is then defined by the following equation:

$$H_L = \frac{\delta_3 \, u^3}{2gq}$$

where H_L= spillway energy loss head in feet
δ_3= 0.22 δ, energy thickness in feet
q= unit discharge in cfs/feet

The actual flow depth over the spillway is:

$$d = d_p + \delta_1$$

where d_p = potential flow depth in feet
δ_1= 0.18 δ, displacement thickness in feet

The conventional energy loss computations are valid and can be utilized downstream of the location of critical point where the turbulent boundary layer intersects the free surface flow.

2.9.1 FRO

The FRO spillway crest is set at elevation 628 feet msl with a width of 200 feet, and is designed using a design head of 30 feet to pass up to 75,000 cfs with a minimum of 4 feet of freeboard to the top of the upstream crest parapet wall. The equivalent unit discharge at 75,000 cfs is 375 cfs per linear foot. As with the FRFA alternative, the design discharge capacity has been conservatively estimated using a slightly lower discharge coefficient (C_d = 3.84) than is typically found for smooth ogee designs, to ensure adequate capacity without risk of overtopping. Figure 2- shows the design discharge curve of spillway computed using the design discharge coefficient (C_d=3.84).

The flow depth and velocity at the toe of spillway just before entering the energy dissipation structure are estimated using the turbulent boundary layer development equations as explained in previous section. The flow leaving the spillway chute has a depth and velocity of about 4 feet and 100 feet/s, respectively, and an equivalent energy head loss of about 10 feet.

Like that of the FRFA, the FRO crest shape has been designed for a maximum design head of 30 feet, though the maximum anticipated head achieved by the spillway under the PMF event is less than 22 feet. This “overdesign” permits the ogee shape to be cast on top of the underlying RCC dam structural outline and reach tangency with the overall downstream dam structure slope with approximately 3 feet of concrete overlay.

For this evaluation, it is assumed that the RCC construction will proceed in lifts of approximately 1 foot, which would leave a finished concrete face with 1-foot steps at the design face slope of 0.85H:1V. The chute design assumes a structural overlay of concrete on the ogee crest and the face of the chute. Appropriate doweling and structural reinforcement would be required to securely anchor the structural concrete overlay to the RCC dam structure (Figure 2-19).

Figure 2-18
FRO Spillway Discharge Curve

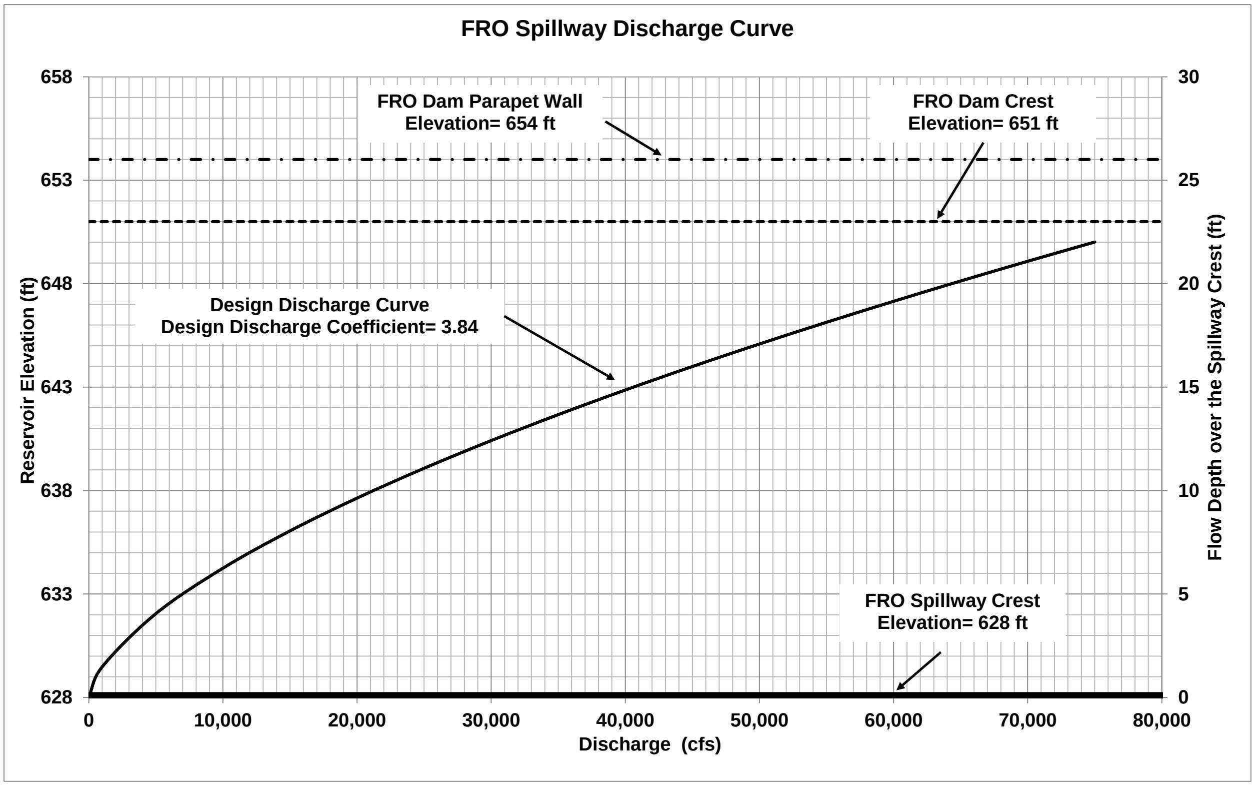
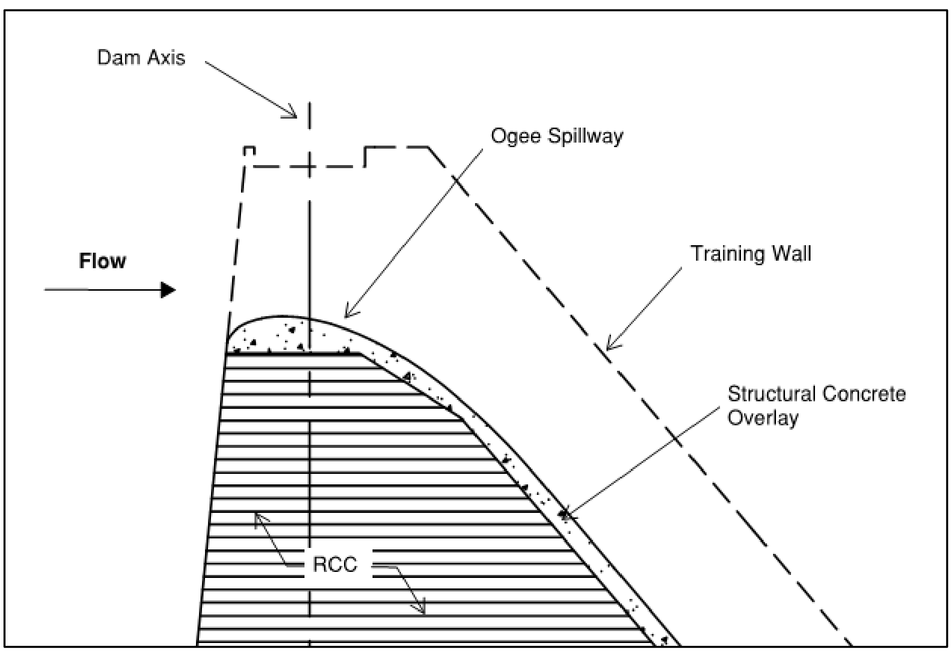


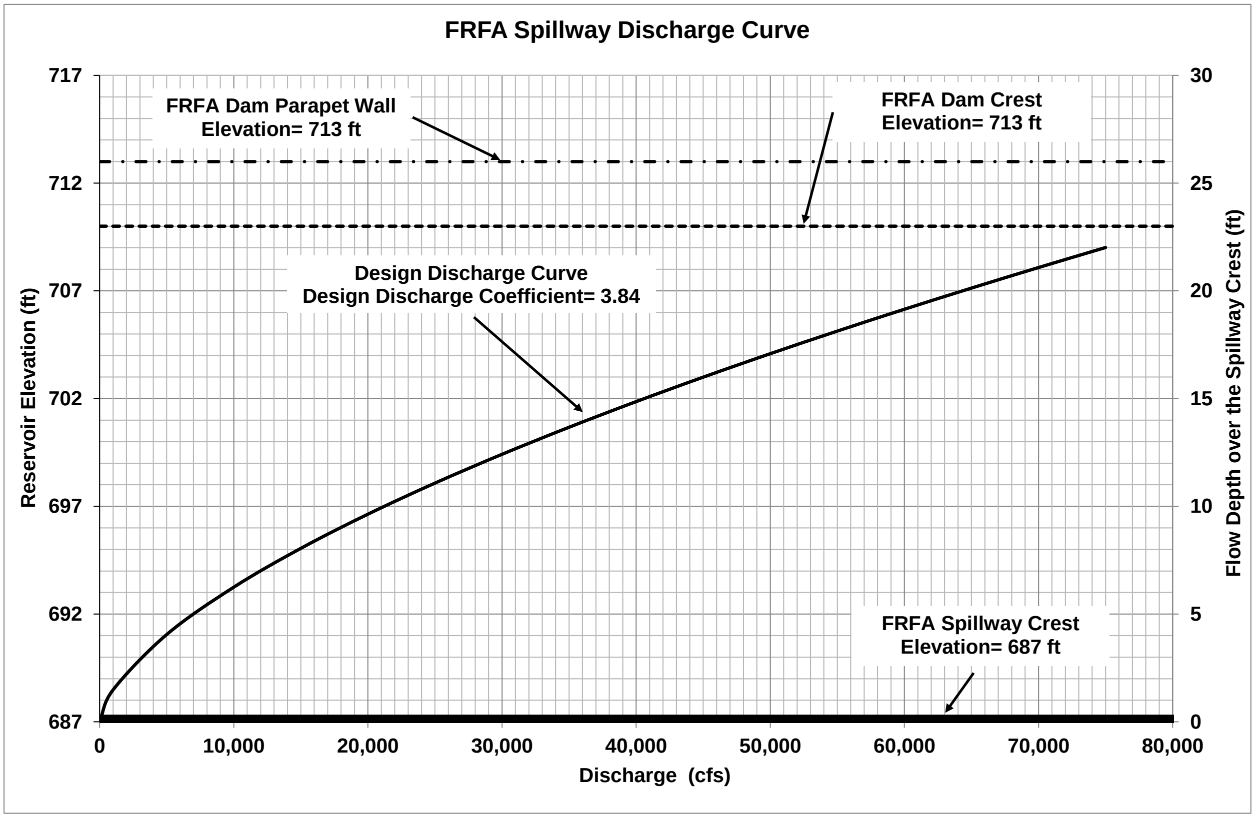
Figure 2-19
Schematic View of the Structural Concrete Overlay to Build the Spillway Face



2.9.2 FRFA

The FRFA spillway crest is set at elevation 687 feet msl with a width of 200 feet, and is designed to pass up to 75,000 cfs with a minimum of 4 feet of freeboard to the top of the upstream crest parapet wall. The equivalent unit discharge at 75,000 cfs is 375 cfs per linear foot. The design discharge capacity has been conservatively estimated using a slightly lower discharge coefficient (C_d = 3.84) than is typically found for smooth ogee designs, to ensure adequate capacity without risk of overtopping. Figure 2-20 shows the design discharge curve of spillway computed using the design discharge coefficient (C_d=3.84).

Figure 2-20
FRFA Spillway Discharge Curve



The flow depth and velocity at the toe of spillway before entering the energy dissipation structure are estimated using the turbulent boundary layer development equations. The flow leaving the spillway chute has a depth and velocity of about 3.4 feet and 117 feet/s, respectively, and an equivalent energy head loss of about 20.9 feet.

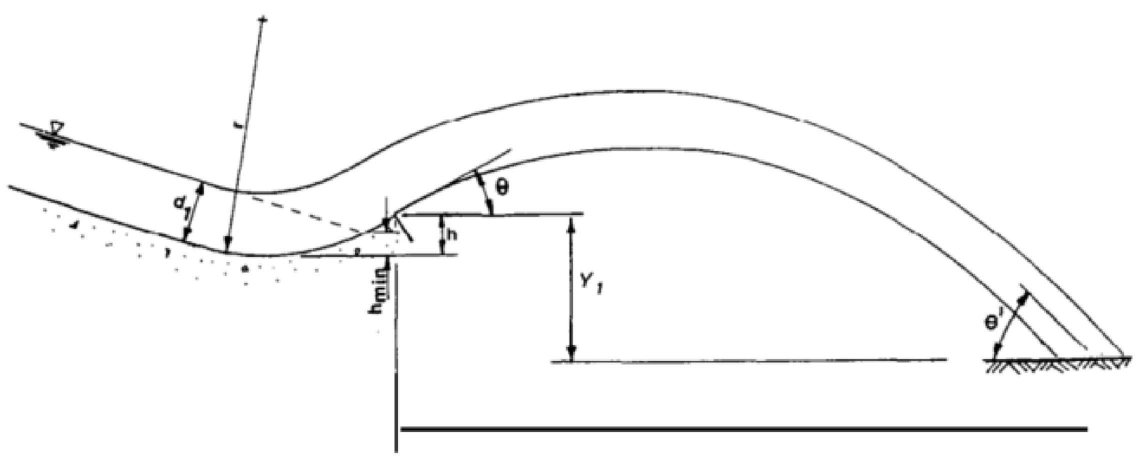
Similar to FRO, for the FRFA design the crest shape has been designed for a maximum head of 30 feet, though the maximum anticipated head achieved by the spillway under the PMF event is only 22 feet. This "overdesign" permits the ogee shape to be cast on top of the underlying RCC dam structural outline and reach tangency with the overall downstream dam structure slope with approximately 3 feet of concrete overlay (see Figure 2-19 above).

2.10 Flip Bucket and Plunge Pool

A combination of flip bucket and plunge pool will be used to dissipate energy from the high velocity spillway flows. The flip bucket directs the incoming high velocity flow of the spillway chute away from the dam by ejecting it upwards and away from the dam as a free jet into the atmosphere. The free jet returns to the river bed well downstream of the dam structure in a plunge pool. The plunge pool for the FRO and FRFA alternatives will consist of an open excavation (with completed pool depth below existing channel bed). Extreme turbulence and entrainment of air help to dissipate energy upon impact.

Although some scour and erosion would likely occur during a discharge event over the spillway, the FRO and FRFA spillways are expected to be used infrequently and for events of very short duration. The reservoir modeling conducted to date indicates that spill events are likely to happen for 300-year to 1,000-year recurrence interval events. Small spill discharges would be expected to cascade from the lip of the flip bucket and pass across the rockfill material to the river below, contained on one side by the low wall adjacent to the stilling basin. Based on the geology of the site, the downstream rock formation appears adequately strong to provide occasional spillway flow dissipation. Additional design refinement in the next phase of the project may include a more detailed evaluation of erosion protection for the rockfill adjacent to the sluice stilling basin. At this stage, a low containment wall about 5 feet high directs these minor spillway flows across the rockfill material adjacent to the stilling basin and to the river channel below. Design guidance used in the design of the flip bucket geometry included USACE EM 1110-2-1603, Hydraulic Design of Spillways (1992) and the USACE HDC (1987). Figure 2-21 illustrates the parameters used in the design of the flip bucket.

Figure 2-21
Spillway Flip Bucket Design



Source: USACE EM 1110-2-1603, Hydraulic Design of Spillways 1992

The minimum radius of flip bucket can be determined using the following equation:

$$r_{min} = \frac{\rho V_1^2 d_1}{P_T - \gamma d_1}$$

where r_{min}= flip bucket minimum radius in feet
ρ= density of water in slug/feet^3
V_1= flow velocity entering the flip bucket in feet per second
d_1= flow depth entering the flip bucket in feet
P_T= allowable theoretical unit load on the flip bucket invert pound per foot
γ= unit weight of the water in pounds per cubic feet

The flip bucket height (h) should be sufficient to prevent the flow from overriding the bucket and provide an optimum trajectory angle which results in the jet impact location well away from any of the dam structures into the plunge pool. To achieve a maximum trajectory distance, the trajectory angle should be a 45 degree from horizontal and the flip bucket height above the bucket invert can be computed using the following equation:

$$h = r - r\ cos\theta$$

where h= flip bucket height above the bucket invert in feet
r = flip bucket design radius in feet
θ= trajectory angle

The jet trajectory leaving the flip bucket can be evaluated using the equation for trajectory of a projectile as follow:

$$y = x \tan\theta - \frac{g\ x^2}{2\ v^2\ (\cos\theta)^2}$$

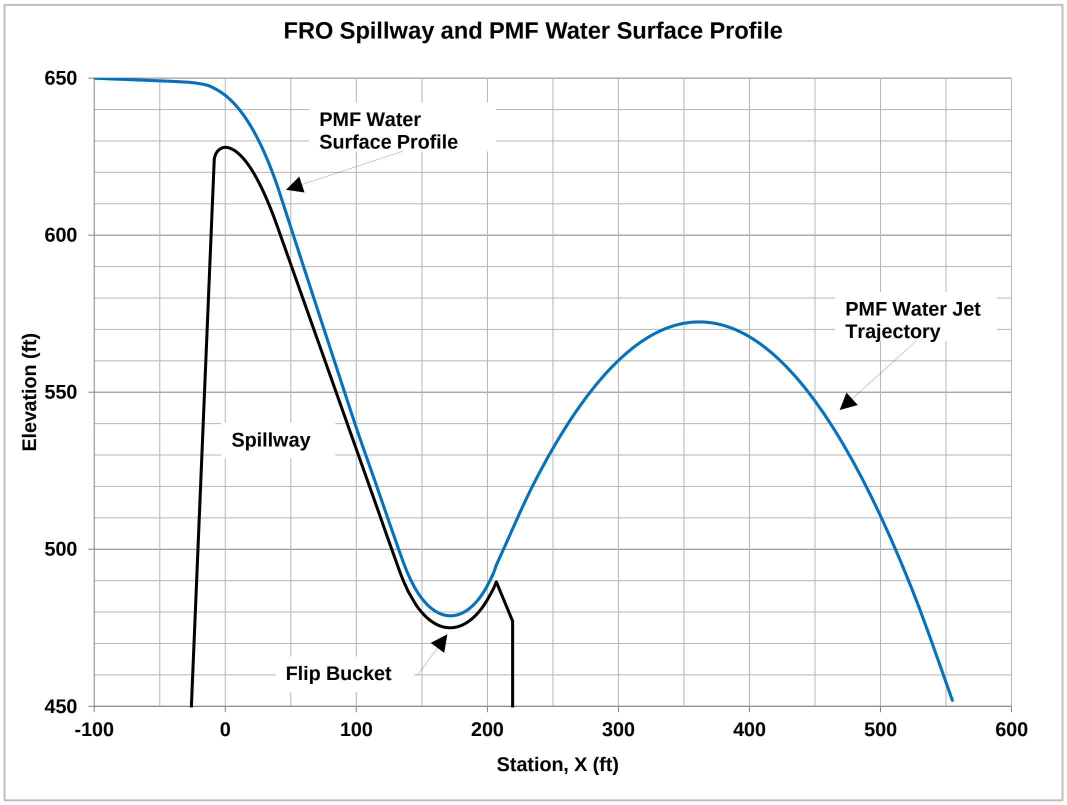
Where x= horizontal coordinate in feet
y= vertical coordinate in feet
θ = trajectory angle
v= velocity of the trajectory in feet per second

2.10.1 FRO

The flip bucket design is based on the maximum unit discharge of 375 cfs per linear foot of width at maximum spillway flow (PMF), with the bucket invert at elevation 475 feet msl and the lip at elevation 489.6 feet msl. The flow profile down the spillway chute was evaluated using the turbulent boundary layer development method, with the result that at maximum discharge the toe velocity is about 100 feet per second and depth of about 4 feet, yielding a minimum bucket radius of 43.6 feet. However, we have used the same 50-foot radius for both the FRFA and FRO alternative flip bucket designs. The trajectory angle of 45 degree was considered to achieve a maximum jet trajectory distance. Figure 2- shows the

PMF water surface profile down the FRO spillway and jet trajectory leaving the flip bucket. Trajectory calculations determined an approximate impact zone of about 375 feet downstream of the bucket lip. The rockfill design below the flip bucket would be developed during the next phase of the study.

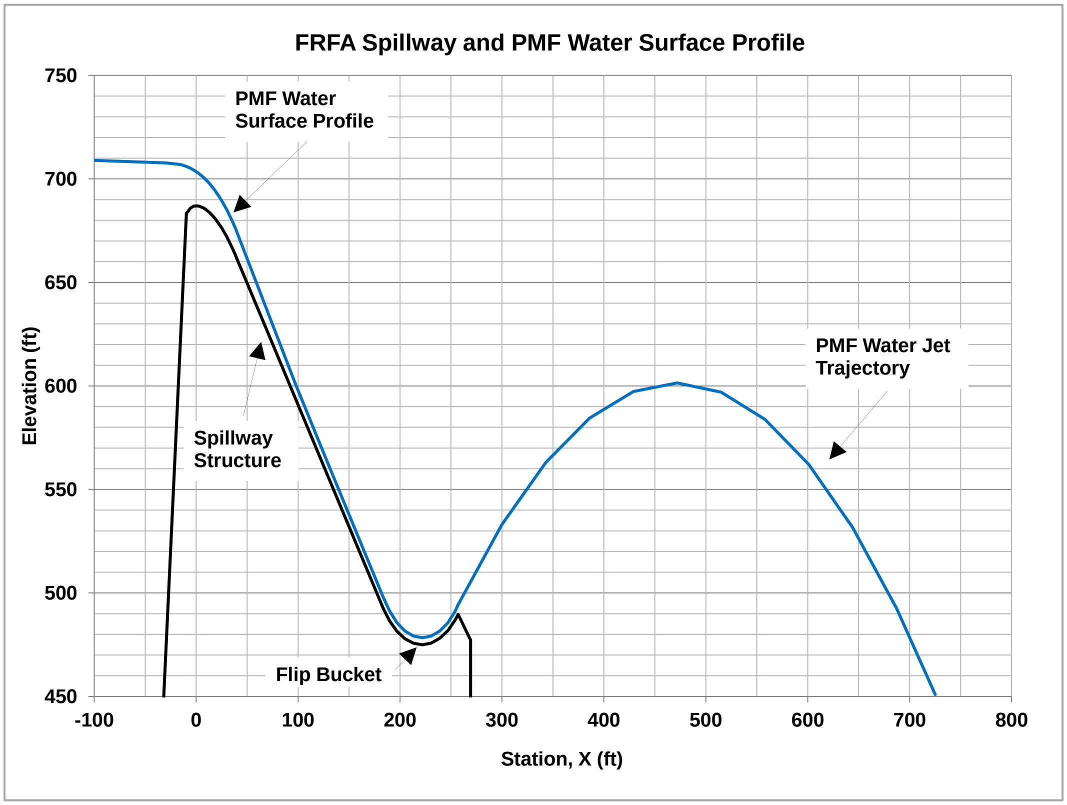
Figure 2-22
FRO Spillway and PMF Water Surface Profile



2.10.2 FRFA

The flip bucket design for the FRFA dam alternative is also based on the maximum unit discharge of 375 cfs per linear foot of width at maximum spillway flow (PMF), with the bucket invert at elevation 475 feet msl and the lip at elevation 489.6 feet msl. The flow profile down the spillway chute was evaluated using the turbulent boundary layer development method, with the result that at maximum discharge the toe velocity is about 117 feet per second and depth of about 3.4 feet, yielding a minimum bucket radius of 51.6 feet. Similar to FRO, a design radius of 50 feet was selected for simplicity. The trajectory angle of 45 degrees was considered to achieve a maximum jet trajectory distance. Figure 2-23 shows the PMF water surface profile down the FRO spillway and jet trajectory leaving the flip bucket. Trajectory calculations determined an approximate impact zone about 400 to 500 feet downstream of the lip. Lesser discharges would result in an impact zone that would be nearer the dam toe. At unit discharges less than about 50 cfs per linear foot, energy losses down the chute would be significant and would reduce the toe velocity appreciably, resulting in impact closer to the dam toe. The rockfill design would accommodate unit discharges of perhaps 30 to 50 cfs per foot without entrainment of stone and plucking or erosion. In the event of higher flows, scour damage (removal of bed material and rock) would occur in the vicinity of the impact area. The specific gradation of the stone surface material has not been calculated in this conceptual design analysis but will be included as a refinement during the next design phase.

Figure 2-23
FRFA Spillway and PMF Water Surface Profile



2.11 Flood Regulation Outlets

Flood control outlets are designed to pass relatively large flows and can be gated to provide close regulation of the flow. USACE EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works (1980), was utilized in the design of the outlet works. Rating curves were generated for each potential sluice size and elevation to determine the proper design that would work best if implemented. These rating curves were compared with the design discharge, and the sluice sizes were optimized to meet the discharge required for flood control outlets while also allowing for fish passage (channel velocities equal to or less than existing channel condition).

The rating curves were calculated using the following equation in the case of submerged inlet condition:

$$Q = C\ GO\ B\ \sqrt{2gH}$$

where
Q = discharge in cfs
C= discharge/contraction coefficient for radial gate
GO= vertical gate opening in feet
B= conduit width in feet
H= energy grade line minus (conduit invert elevation + C*GO) in feet

2.11.1 FRO

The FRO design has three low-level sluice outlets, consisting of a single larger 12 foot wide by 20 foot high sluice at invert elevation 408 feet msl and a pair of 10 foot wide by 16 foot high sluices at invert elevation 411 feet msl. A large, full height trashrack extending from the riverbed to the dam crest will exclude most large trees from the sluice conduits and provide excess open area under all reservoir elevations to pass the desired project outflows. The larger gate will be used to pass the majority of sediment bedload in the river, as well as most small debris. Some sediment is expected to pass through the higher sluice conduits as well, but the lower sluice will intentionally receive the most wear from sediment passage over time. Repairs to the sluice floor are expected every few years to bring the sacrificial concrete floor surface back to original grade.

The partial and full open gate rating curves for the single 12 foot wide by 20 foot high sluice, single 10 foot wide by 16 foot high sluice and pair of the 10 foot wide by 16 foot high sluices for the final design are provided in Figure 2-24 through Figure 2-26. According to the rating curves, with all three FRO low-level flood regulation sluices open, up to approximately 8,500 cfs can be passed through the sluices without transitioning to orifice or pressure flow in any of the sluice openings, with reservoir elevation at 427 feet msl. The 15,000 cfs maximum regulated project outflow can be passed through all three sluice outlets at reservoir elevations greater than about 450 feet msl. The maximum 15,000 cfs project outflow can be passed entirely through the pair of 10 foot by 16 foot sluices at reservoir elevations greater than about 500 feet msl. Typical flood regulation operation would initiate closure of the larger sluice at any time the pool levels exceed about elevation 500 feet msl, to prevent excessive wear on the large sluice floor due to bed sediments entrained in high flow velocity. The pair of 10 foot by 16 foot sluices are expected to entrain considerably less sediment, though the specific elevation details to confirm this and establish the final higher sluice gate seat elevation would have to be evaluated using a physical scale model. Mud Mountain Dam on the White River in western Washington (owned by USACE) is designed similarly, and its three outlet sluices operate much like that proposed for the FRO design alternative.

All three outlets are equipped with radial sluice gates to accommodate fine regulation of the flow. The large lower radial gate has a width of 12 feet, a height of 20 feet, and a radius of about 44 feet. The two smaller radial gates have a width of 10 feet each, a height of 16 feet, and a radius of about 35 feet. The discharge through the large and smaller conduits becomes only a function of the reservoir elevation for the gate openings larger than 18 feet and 15 feet, respectively.

The two smaller sluice gates are designed to pass up to 5,000 cfs each with 61 feet of static head on each gate at the 75 percent open setting, while the larger gate can pass the same 5,000 cfs with 29 feet of static head on the gate at the 75 percent open setting. This ensures that the full 15,000 cfs desired sluice discharge capacity is available at reservoir elevations as low as 462 feet msl in a fully controlled manner, which is about 166 feet below the spillway crest.

At full flood storage reservoir elevation of 628 feet msl, each of the two smaller sluice gates at 75 percent open can pass up to about 9,500 cfs. The larger gate can pass up to about 14,200 cfs when 75% open. As with the FRFA design, the paired design of the two smaller gates was selected to ensure that finely controlled flood regulation would be available with a single gate as needed, given that the larger gate will likely be closed. Adjustment of a single 10 foot wide gate in 6-inch typical lift increments gives just 380 cfs per increment at the maximum flood regulation reservoir elevation of 628 feet msl. Controlling downstream flows is important for both the FRO and FRFA alternatives, so that required ramping rates can be achieved. Flood regulation operation would include operation of the sluices at reservoir elevations up to the spillway crest of 628 feet msl. At reservoir elevation above the spillway crest, sluice operation may be curtailed to avoid adverse flow conditions within the stilling basin.

Each sluice conduit is provided with an emergency bulkhead gate a few feet upstream of the radial gate, as well as dewatering bulkheads at the inlet to the sluice and the outlet to the sluice. The emergency bulkhead gate would be a vertical panel, likely a roller gate, with hydraulic operator, and is designed to close under full flow at maximum reservoir elevation. The upstream and downstream dewatering bulkheads are simple, vertically hung panels that are designed to close under no flow. They are provided to isolate and dewater each sluice conduit so that inspections and repairs can be accomplished in safe working conditions.

Figure 2-24
FRO Single 12 Foot Wide By 20 Foot High Sluice Gate Rating Curves

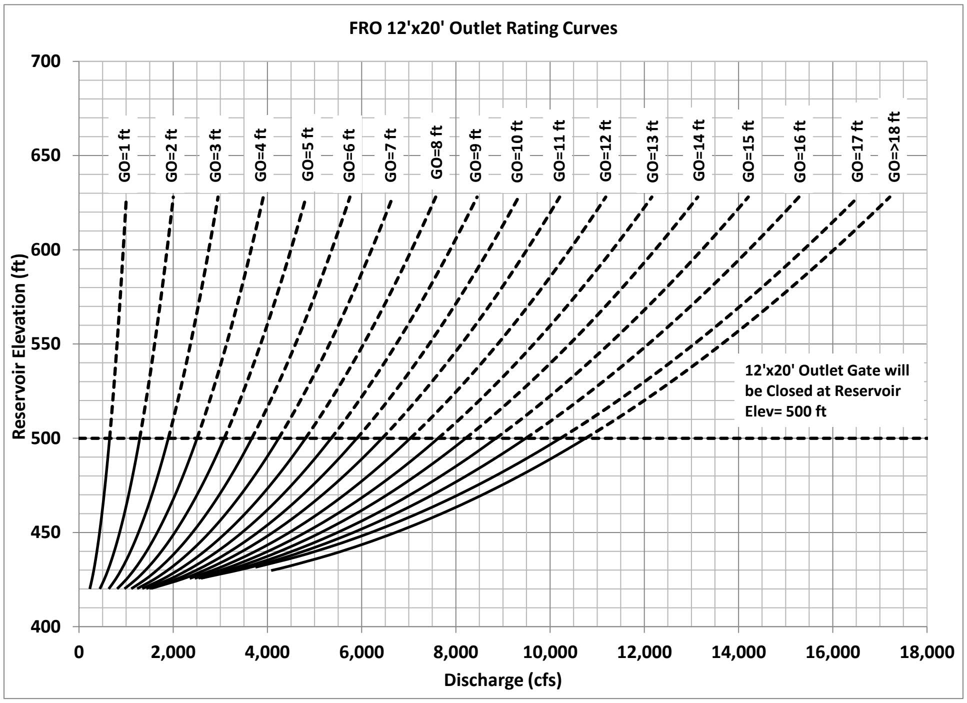


Figure 2-25
FRO Single 10 Foot Wide By 16 Foot High Sluice Gate Rating Curves

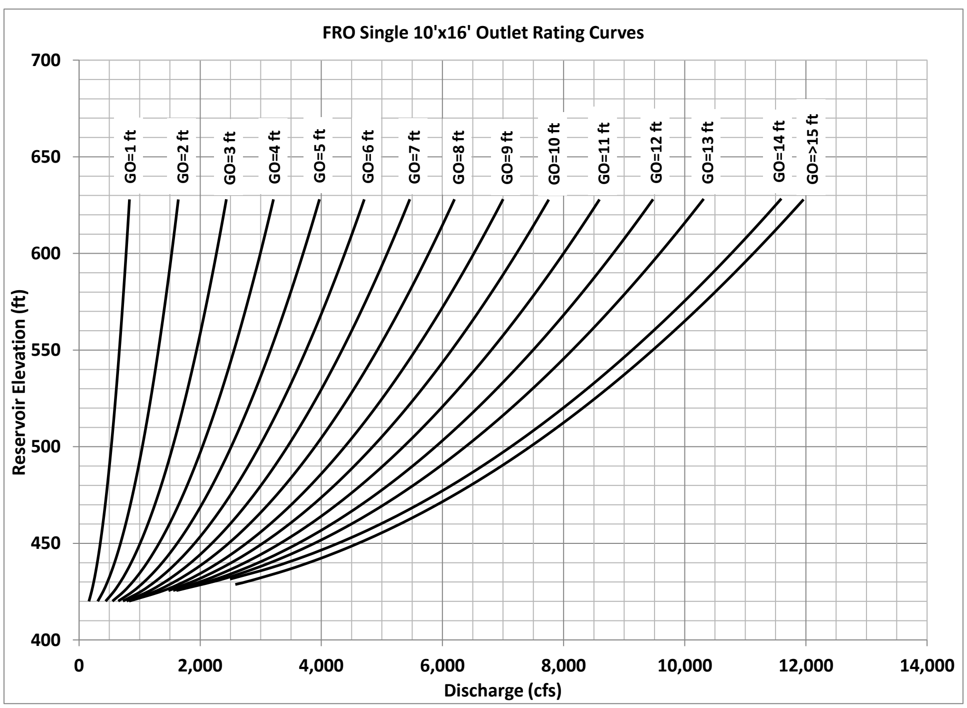
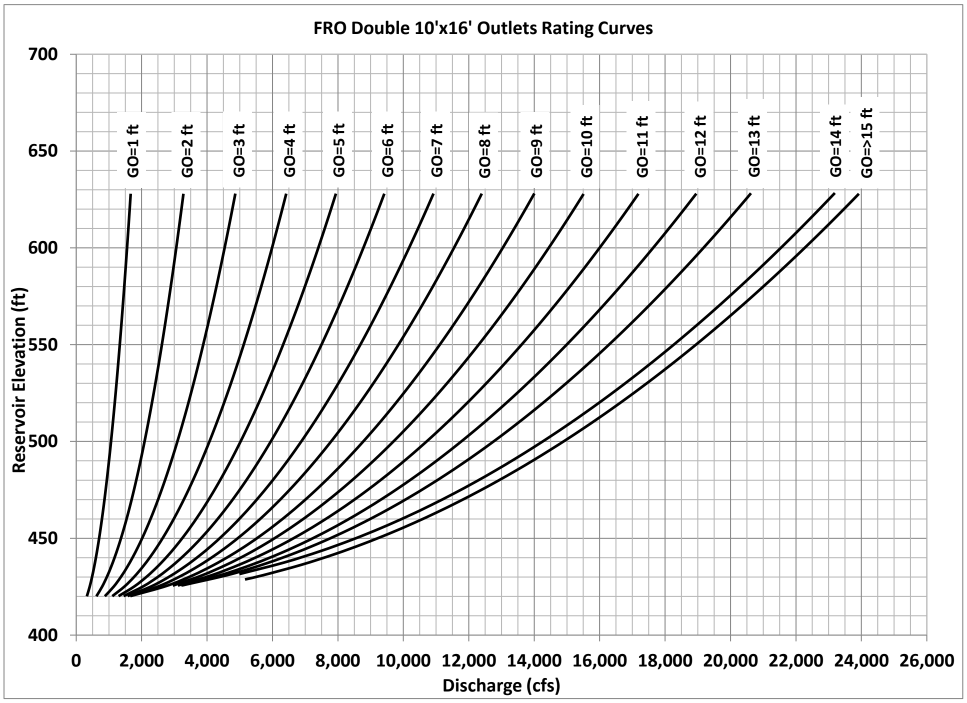


Figure 2-26
FRO Double 10 Foot Wide By 16 Foot High Sluice Gate Rating Curves



2.11.2 FRFA

The FRFA design alternative low-level outlets consist of a pair of 10 foot wide by 16 foot high sluices at invert elevation of 420 feet msl at the base of the dam. A large, full height trashrack protects these sluice gates from entraining large debris that cannot be passed through the dam. As described above, the low-level flood regulation sluices are designed for a maximum controlled discharge of 15,000 cfs at any reservoir elevation within the full operating range of the project (reservoir elevation 588 feet msl to 687 feet msl). The parallel pair of radial sluice gates would be separated by a center divider wall terminating about 100 feet downstream of the gate seats. Downstream of the divider wall, the outflows from both gates would combine in the 22 foot wide by 16 foot high single conduit. A parabolic drop of about 43 feet in the floor of the sluice conduit carries the discharge into the downstream stilling basin floor at elevation 377 feet msl.

The radial gates have a width of 10 feet, a height of 16 feet, and a radius of about 35 feet. Hydraulic cylinder operators would ensure positive closure under all flow conditions. Side seals and top seal would either be inflatable using reservoir static water pressure, or else the gate trunnion would be provided with an eccentric rotator to compress the top seal. Both sealing types have been used with success in high head applications such as this (up to 285 feet). The bottom seal is compressed by the weight of the gate and the operator down force. Radial gates were selected for several reasons:

- They reduce the gate operator load by transmitting the hydrostatic forces to the trunnion

- They eliminate gate slots, which in a sediment- and debris-rich environment can cause problems in fully seating the gate
- They are more reliably and positively controlled than cable-hung vertical gates at these heads
- They do not suffer from pressure regime shifts at small gate openings as do vertical gates

The partial and full open gate rating curves for the single and double 10 foot wide by 16 foot high sluice gates for the final design are provided in Figure 2-27 and **F**igure 2-28. The sluice gates are designed to pass up to 7,500 cfs each with as little as 133 feet of static head on each gate at the 75 percent open setting. This ensures that the full 15,000 cfs desired sluice discharge capacity is available at reservoir elevations as low as 559 feet msl in a fully controlled manner, which is about 29 feet lower than the typical normal low reservoir pool elevation of 588 feet msl. At full flood storage reservoir elevation of 687 feet msl, each gate at 75 percent open can pass up to 10,400 cfs. The paired design was selected to ensure that fairly precise flood regulation was available with a single gate as needed. Adjustment of a single 10 foot wide gate in 6-inch typical lift increments gives just about 380 cfs per increment at the minimum flood regulation reservoir elevation of 628 feet msl. This is important in controlling downstream releases during the flood regulation operation so that required ramping rates can be achieved. Flood regulation operation would include operation of the sluices at reservoir elevations up to the spillway crest of 687 feet msl.

Similar to the FRO alternative, at reservoir elevation above the spillway crest, sluice operation may be curtailed to avoid adverse flow conditions within the stilling basin. Also similar to the FRO alternative, each sluice conduit would be provided with an emergency bulkhead gate a few feet upstream of the radial gate, and dewatering bulkheads at the inlet to the sluice and the outlet to the sluice.

Figure 2-27
FRFA Single 10 Foot Wide By 16 Foot High Sluice Gate Rating Curves

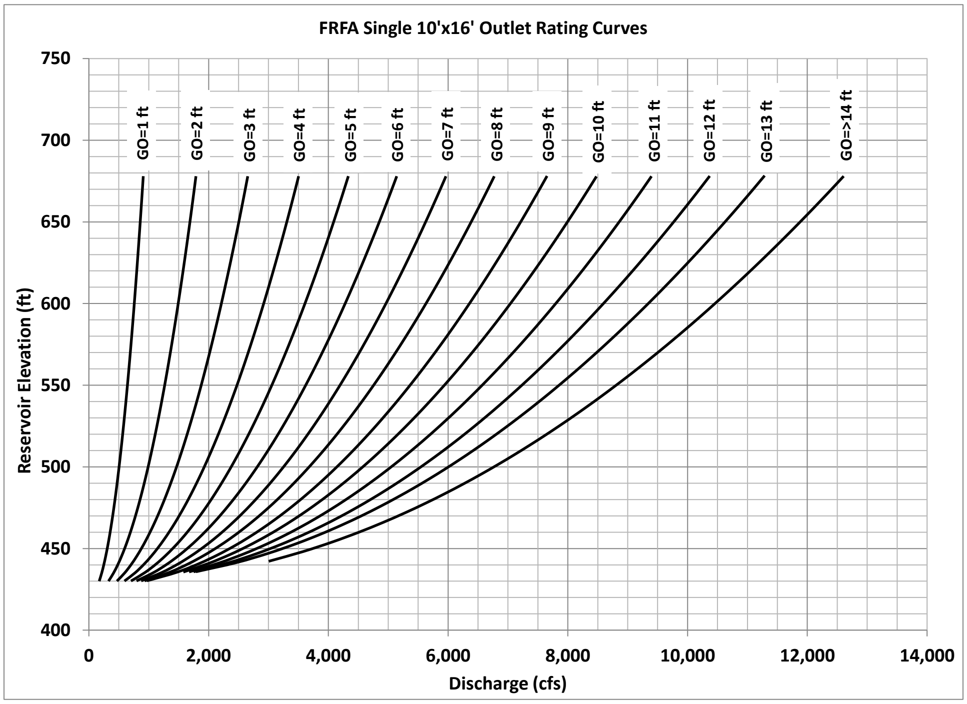
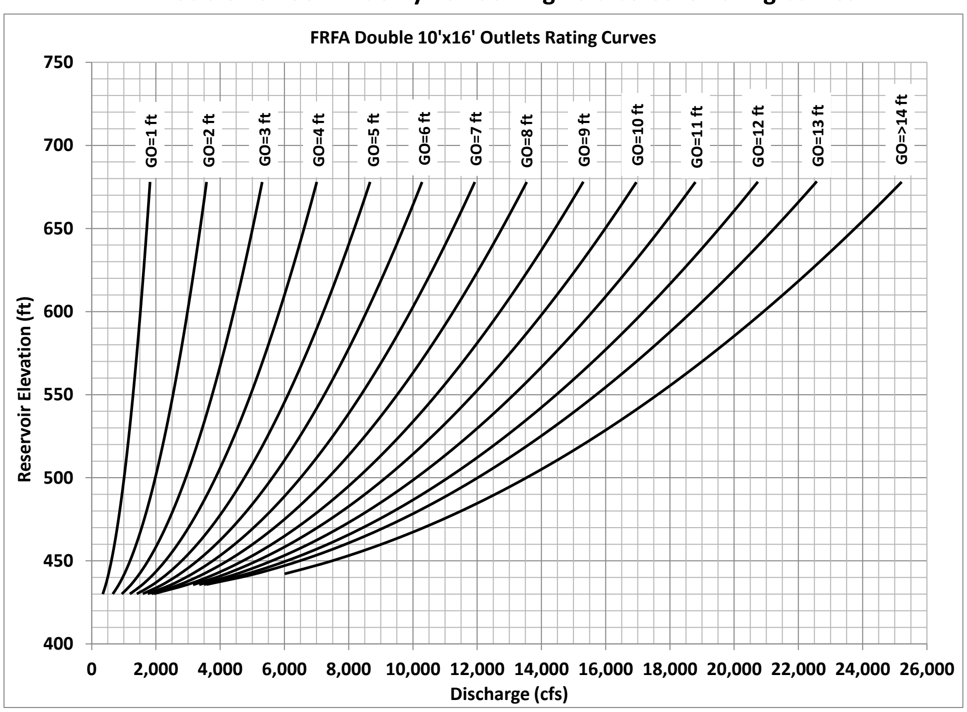


Figure 2-28
FRFA Double 10 Foot Wide By 16 Foot High Sluice Gate Rating Curves



2.12 Stilling Basin

A stilling basin is required to dissipate the high energy of flowing water exiting the outlet work structure. A stilling basin produces a hydraulic jump within a contained and protected area such that the discharge passing from the stilling basin into the river channel downstream is of lower energy and generally flowing under subcritical flow regime, resulting in lower scour and erosion forces. The document USACE EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works (1980) provides guidance for the design of stilling basins.

The stilling basin is designed for the maximum flow to ensure a satisfactory performance under the range of outlet works operational flow. To design the stilling basin size, the formula of hydraulic jump in a rectangular section was utilized:

$$\frac{d_2}{d_1} = \frac{1}{2}\left(\sqrt{1 + 8Fr^2} - 1\right)$$

where d_2 and d_1= conjugate depths of the hydraulic jump in feet

$Fr = \frac{V_1}{\sqrt{gd_1}}$, Froud number of the flow entering the jump

v_1= flow velocity entering the jump in feet per second

d_1: flow depth entering the jump in feet

By assuming a stilling basin floor elevation, the flow depth and velocity entering the jump (d_1 and V_1) can be calculated using the Bernoulli's equation, considering negligible energy loss and jet expansion between the conduit outlet portal and entrance of the stilling basin. Then the conjugate depth of the hydraulic jump will be calculated (d_2) and compared to the available water depth based on the tailwater rating curve. The floor elevation will be accordingly adjusted to ensure a satisfactory hydraulic jump formation within the stilling basin. The length of basin is predicted for the length of the hydraulic jump. For basins with the Froude number between 3 and 12, a length of three times d_2 is recommended. The stilling basin end sill rating curve was calculated using the discharge equation over a broad crest weir as follows:

$$Q = CLH^{1.5}$$

Where Q=discharge in cfs

C= 3.09, broad crest weir discharge coefficient

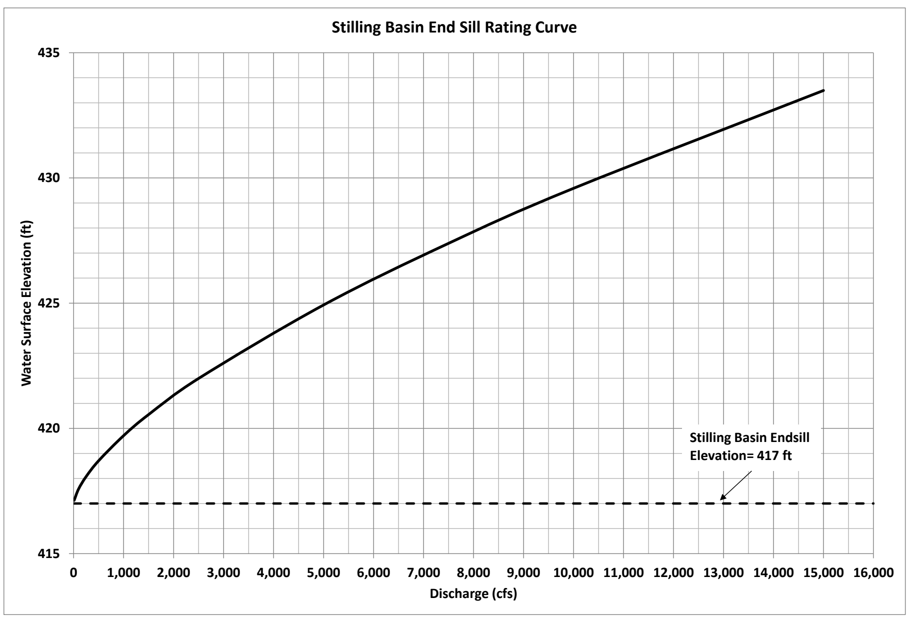
L= stilling basin end sill width in feet

H= water depth over the end sill in feet

2.12.1 FRO

The stilling basin for the FRO dam alternative receives flood regulation outflows from both the two 10 foot by 16 foot sluice gates, up to a design discharge of 15,000 cfs at maximum reservoir elevation at the spillway crest elevation of 628 feet msl, and also discharges from the larger 12 foot wide by 20 foot high gate at reservoir elevations up to about 500 feet msl. The design flow velocity entering the basin is approximately 125 feet per second, with a Froude number of about 10.8 at entry. Since the flow jet would expand quickly following entry into the wider basin, the design energy would be less than the initial entry value. Following USACE guidance, a baffled stilling basin length of approximately 230 feet is obtained, assuming a 70-foot width overall. The end sill elevation was selected to be commensurate with the natural bedrock-controlled stream bed elevation of about 417 feet msl, and the width of 70 feet provides a water surface elevation of about 433.5 feet at the full sluice outlet discharge of 15,000 cfs. **Error! Reference source not found.**Figure 2-29 shows the stilling basin end sill rating curve. The downstream conjugate depth at 15,000 cfs with the expanded jet is approximately 52 feet, yielding a basin floor elevation of about 381.2 feet, which provides adequate energy dissipation within the basin. HEC-RAS modeling of the natural downstream channel indicates that the natural water surface at the end sill location is about 433.5 feet msl at the maximum stilling basin capacity of 15,000 cfs, ensuring hydraulic control by the end sill, since submergence of the elevation 417 feet msl end sill is just 5 feet against a driving head of 16.5 feet.

Figure 2-29
Stilling Basin End Sill Rating Curve



2.12.2 FRFA

The stilling basin for the FRFA design receives flood regulation outflows from the two 10 foot by 16 foot sluice gates, up to a design discharge of 15,000 cfs at maximum reservoir elevation at the spillway crest elevation of 687 feet msl. The design flow velocity entering the basin is approximately 140.4 feet per second, with a Froude number of about 12.6 at entry. As for the FRO, since the flow jet would expand quickly following entry into the wider basin, the design entry energy would be less than the initial entry value. Following USACE guidance, a baffled stilling basin length of approximately 230 feet is obtained, assuming a 70-foot width overall. Similar to FRO, the end sill elevation was selected to be commensurate with the natural bedrock-controlled stream bed elevation of about 417 feet msl, and the width of 70 feet provides a water surface elevation of about 433.5 feet at the full sluice outlet discharge of 15,000 cfs. The downstream conjugate depth at 15,000 cfs with the expanded jet is approximately 56 feet, yielding a basin floor elevation of 377 feet, which provides adequate energy dissipation within the basin. HEC-RAS modeling of the natural downstream channel indicates that the natural water surface at the end sill location is about 433.5 feet msl at the maximum stilling basin capacity of 15,000 cfs, ensuring hydraulic control by the end sill, since submergence of the elevation 417 feet msl end sill is just 5 feet against a driving head of 16.5 feet. The multiport low-flow outlet conduits would discharge through individual valves into the stilling basin from a valve setting above the maximum expected regulating flow stilling basin water surface elevation of 433.5 feet msl. We anticipate these valves would likely be of the hollow cone type, such as a Howell-Bunger design, or perhaps fixed-cone or plunger type valves. The design of the discharge valves for the multiport outlets will be refined in the next phase of designs. For cost estimation purposes, we have assumed Howell-Bunger valves will be selected.

3 CALCULATIONS, TABLES, AND FIGURES

3.1 Hydraulic Modeling and Sediment Transport

3.1.1 1D HEC-RAS Clear-Water Modeling

The results of 1 D HEC-RAS clear water modeling are provided in this section in Figure 3-1 through Figure 3-4.

Figure 3-1
HEC-RAS 1D Water Surface Profiles for Existing Condition

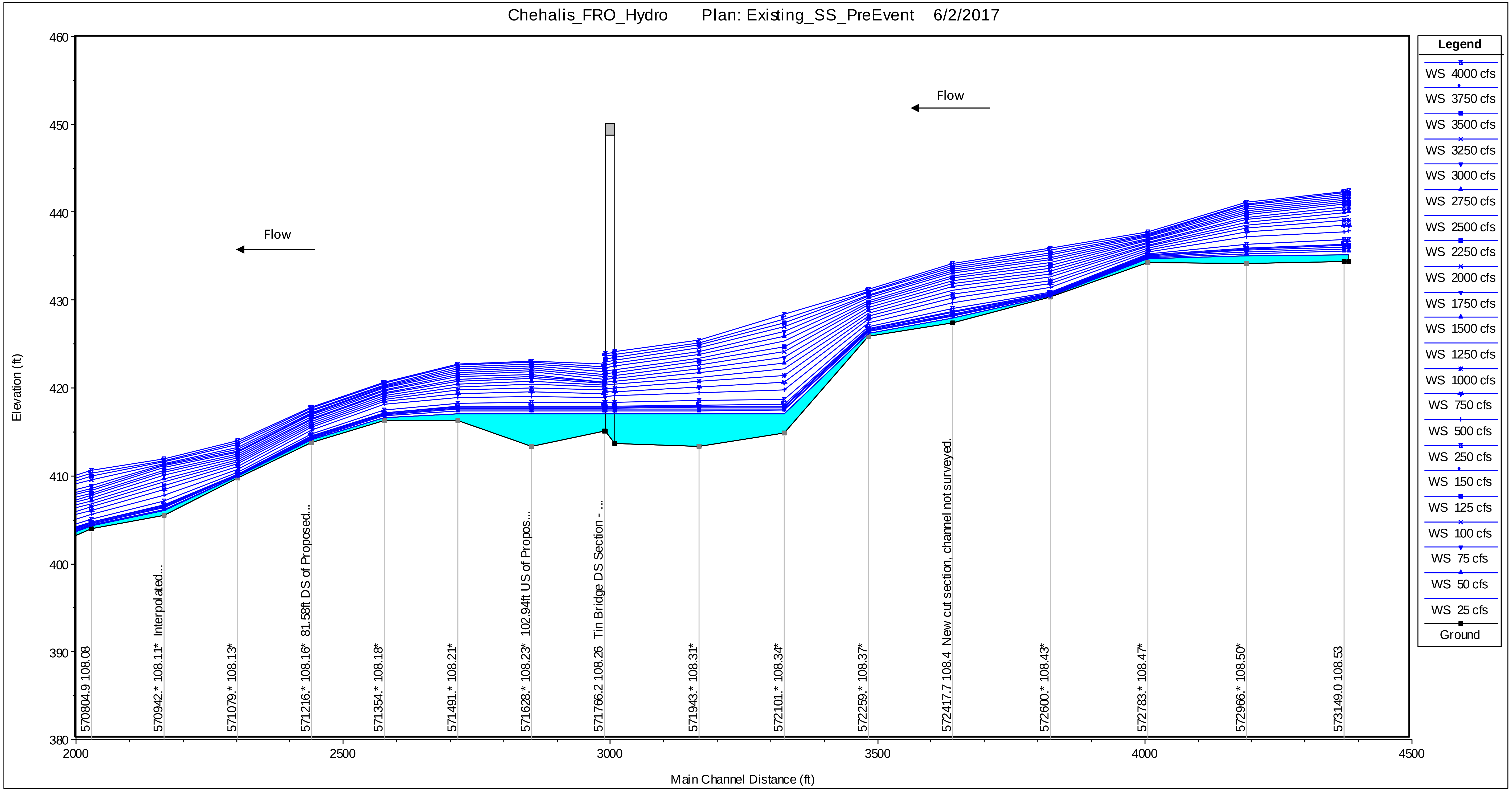


Figure 3-2

HEC-RAS 1D Water Surface Profiles for With-Project Condition

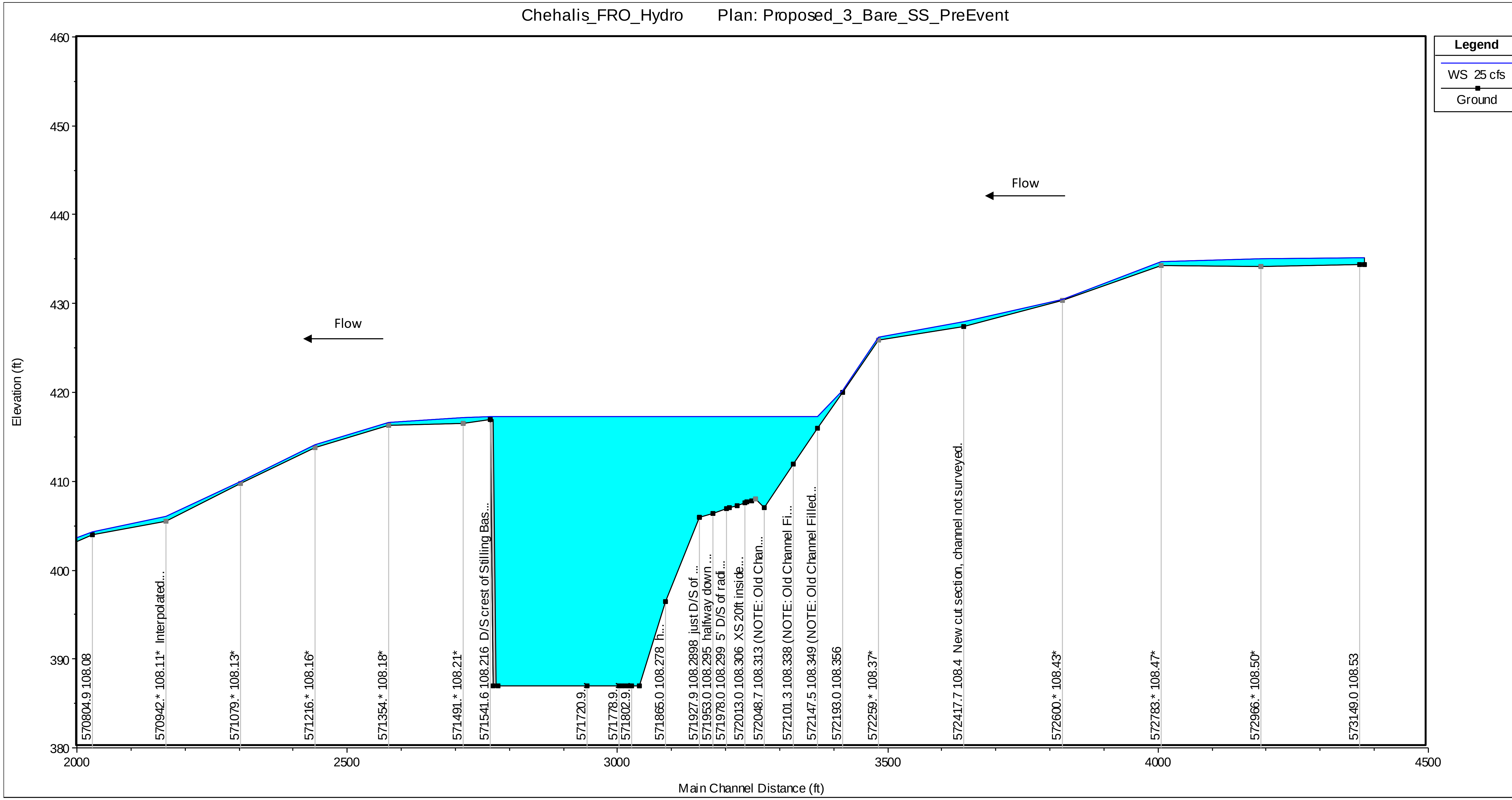


Figure 3-3
HEC-RAS 1D Flow Velocity Profiles for Existing Condition

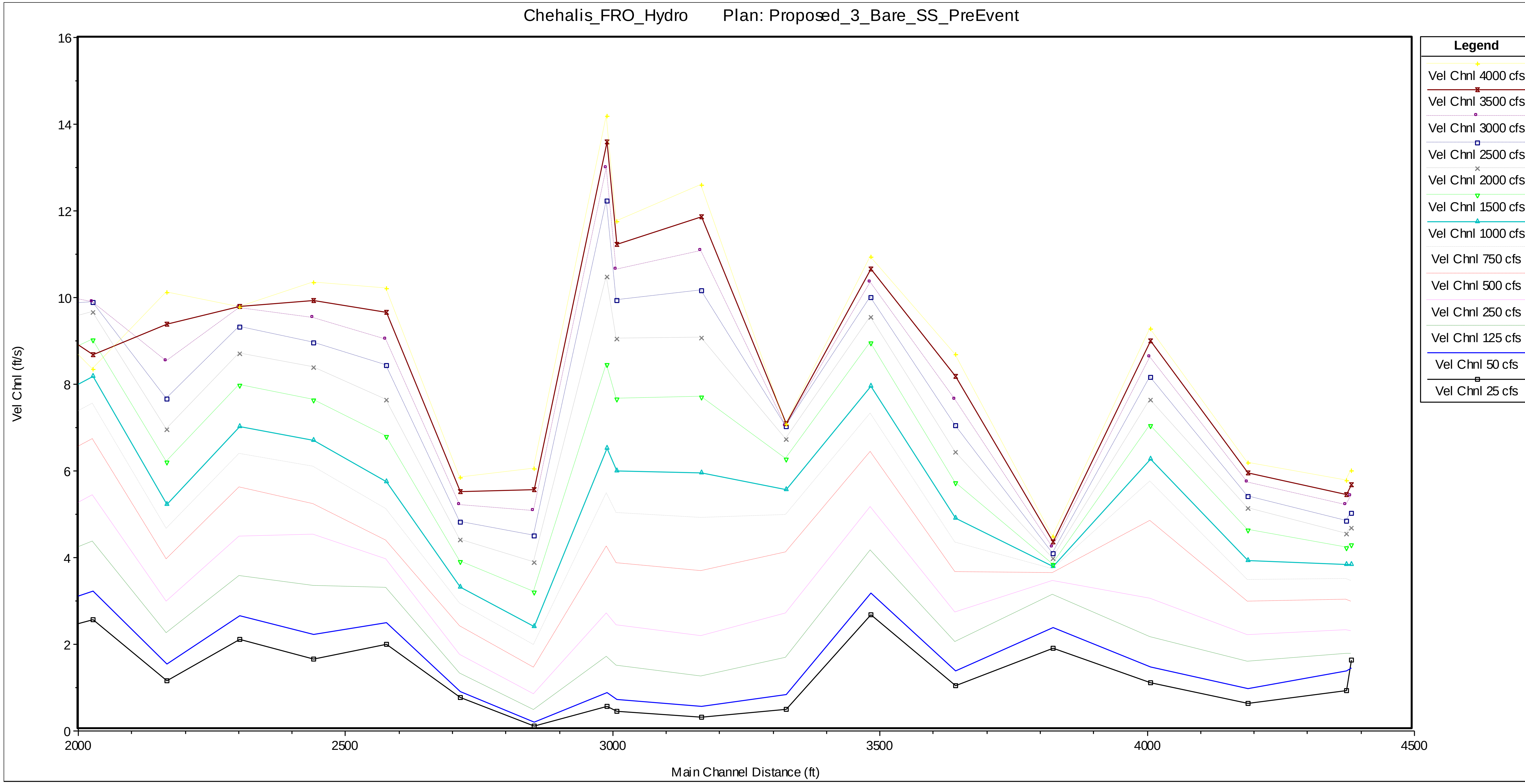
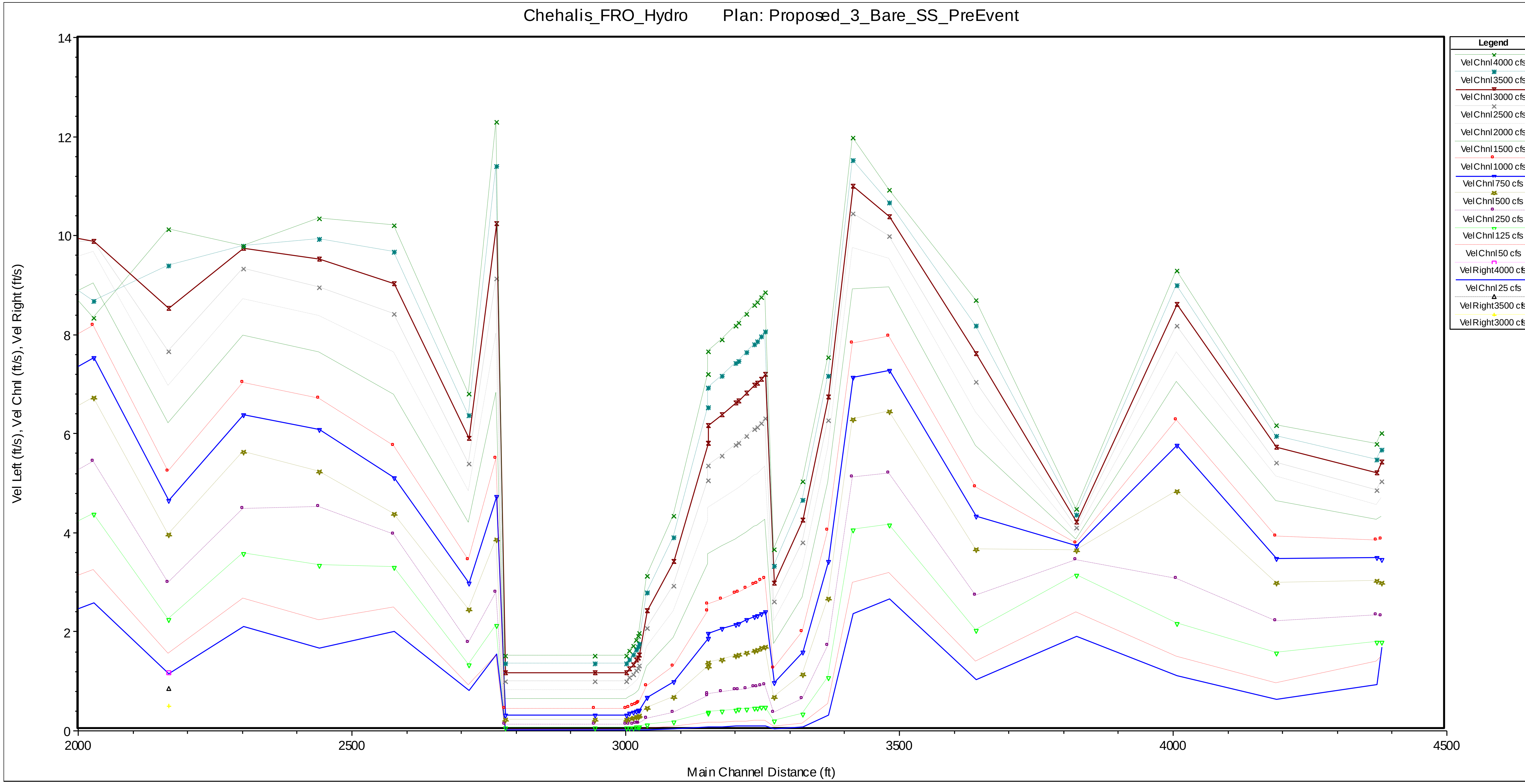


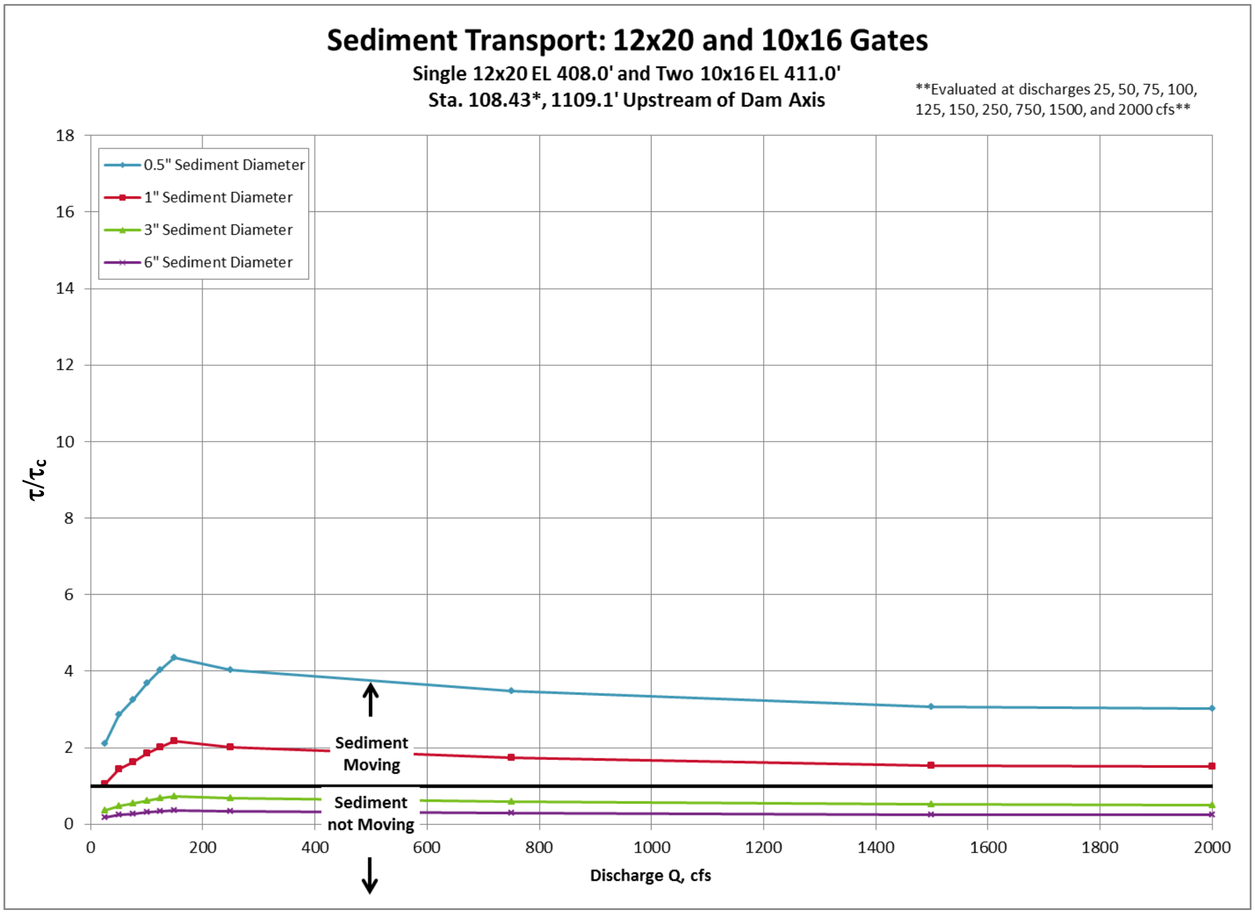
Figure 3-4
HEC-RAS 1D Flow Velocity Profiles for With-Project Condition



3.1.2 1D HEC-RAS Sediment Transport Modeling

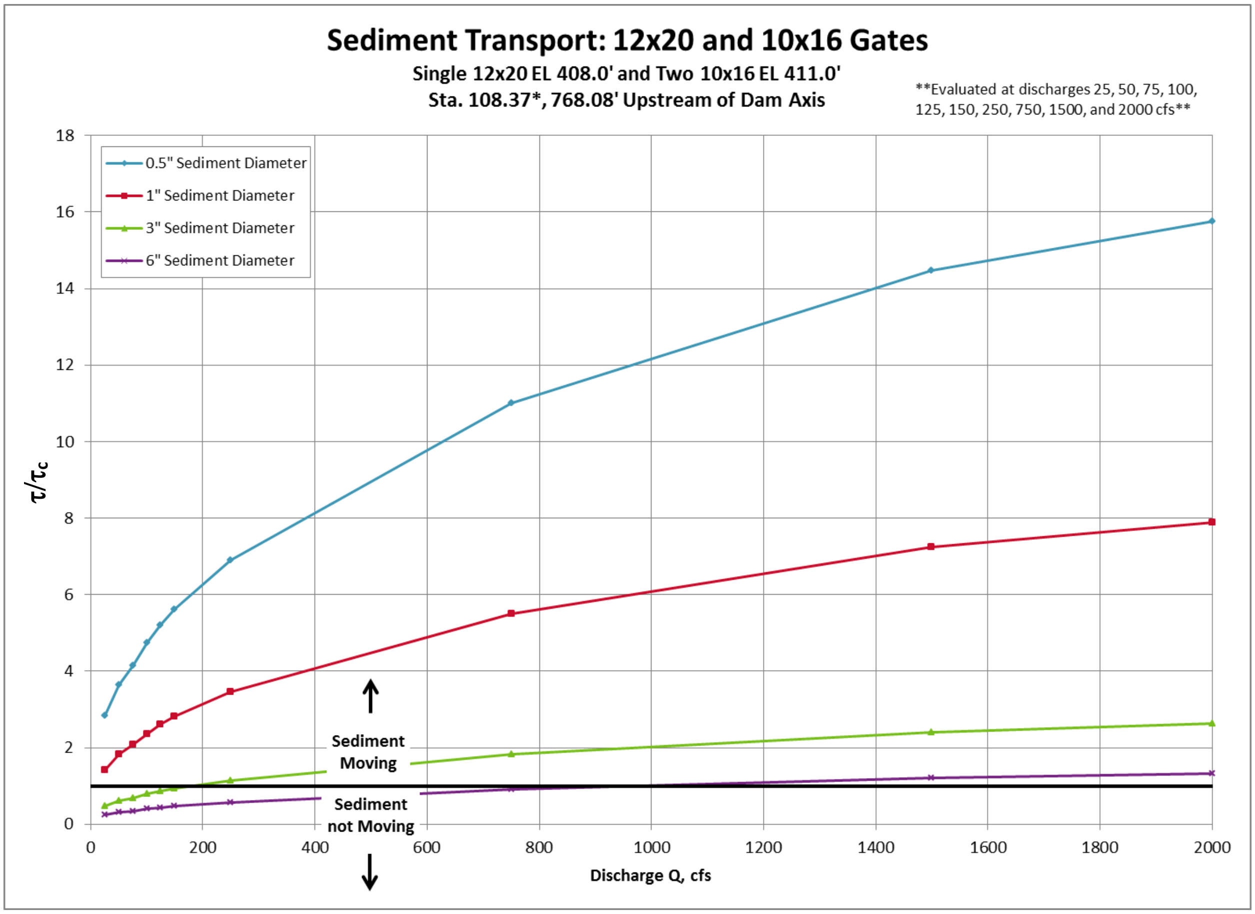
The results of the 1-D HEC-RAS sediment transport modeling is shown in Figure 3-5 through Figure 3-13.

Figure 3-5
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.43



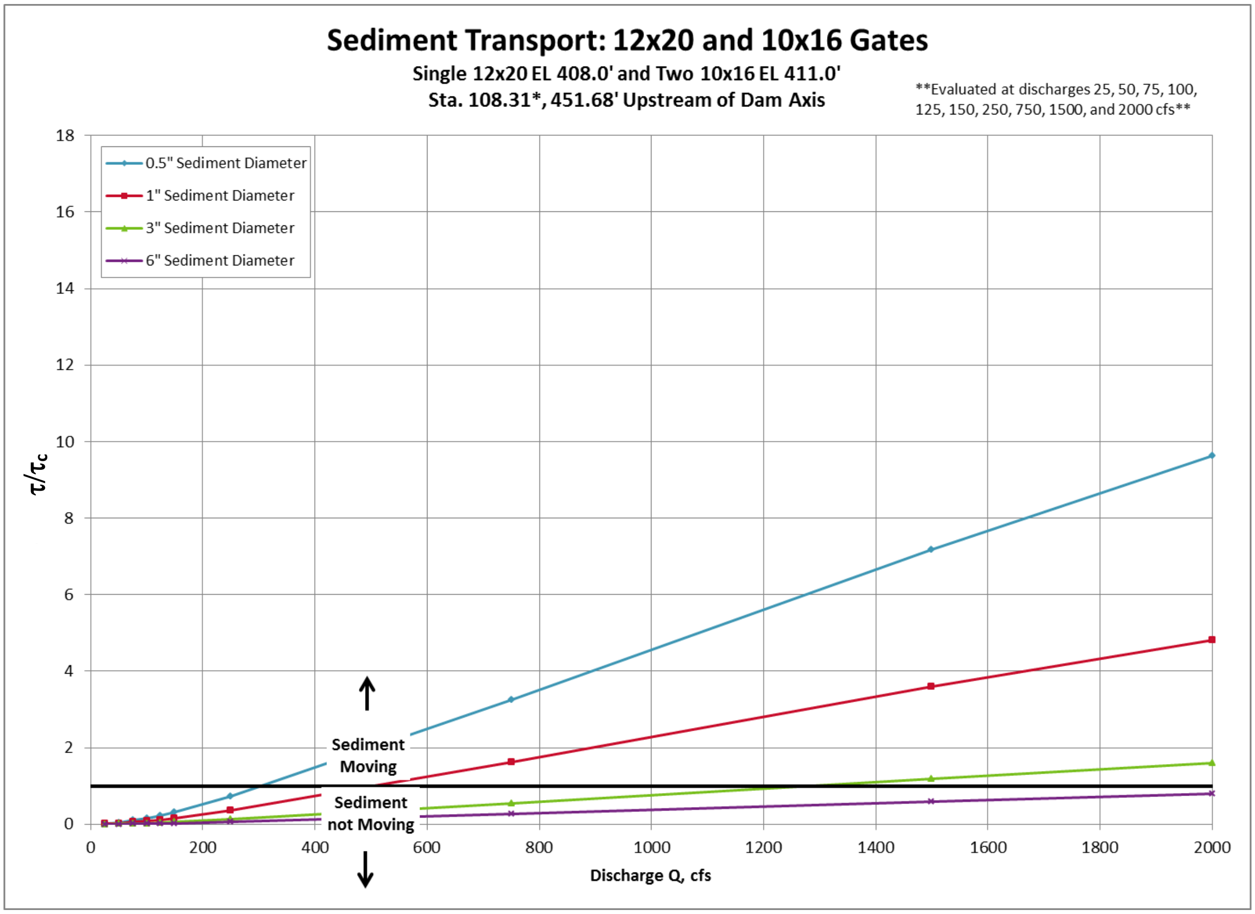
Note:
About 1,110 feet upstream of dam

Figure 3-6
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.37



Note:
About 770 feet upstream of dam

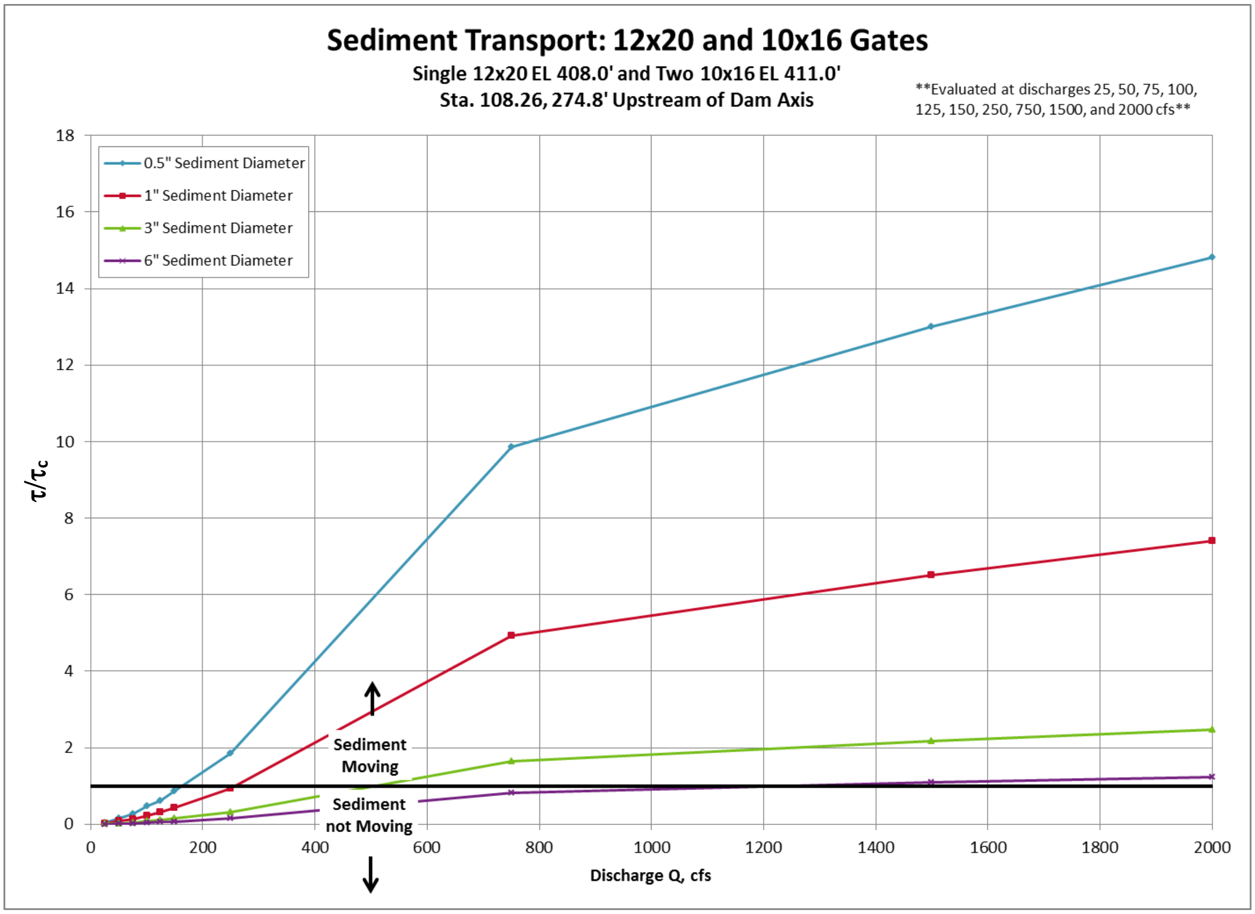
Figure 3-7
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.31



Note:
About 450 feet upstream of dam

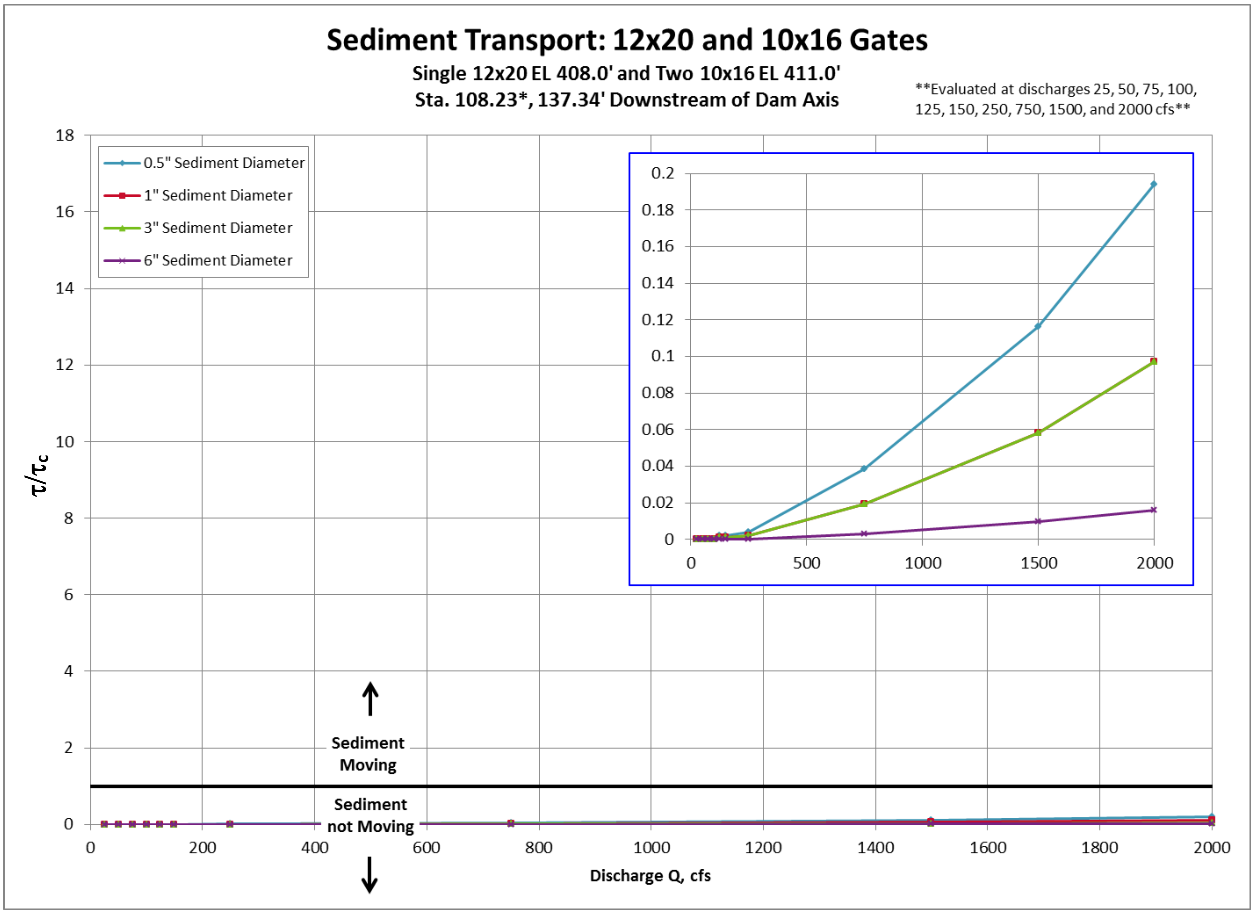
Figure 3-8

Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.26



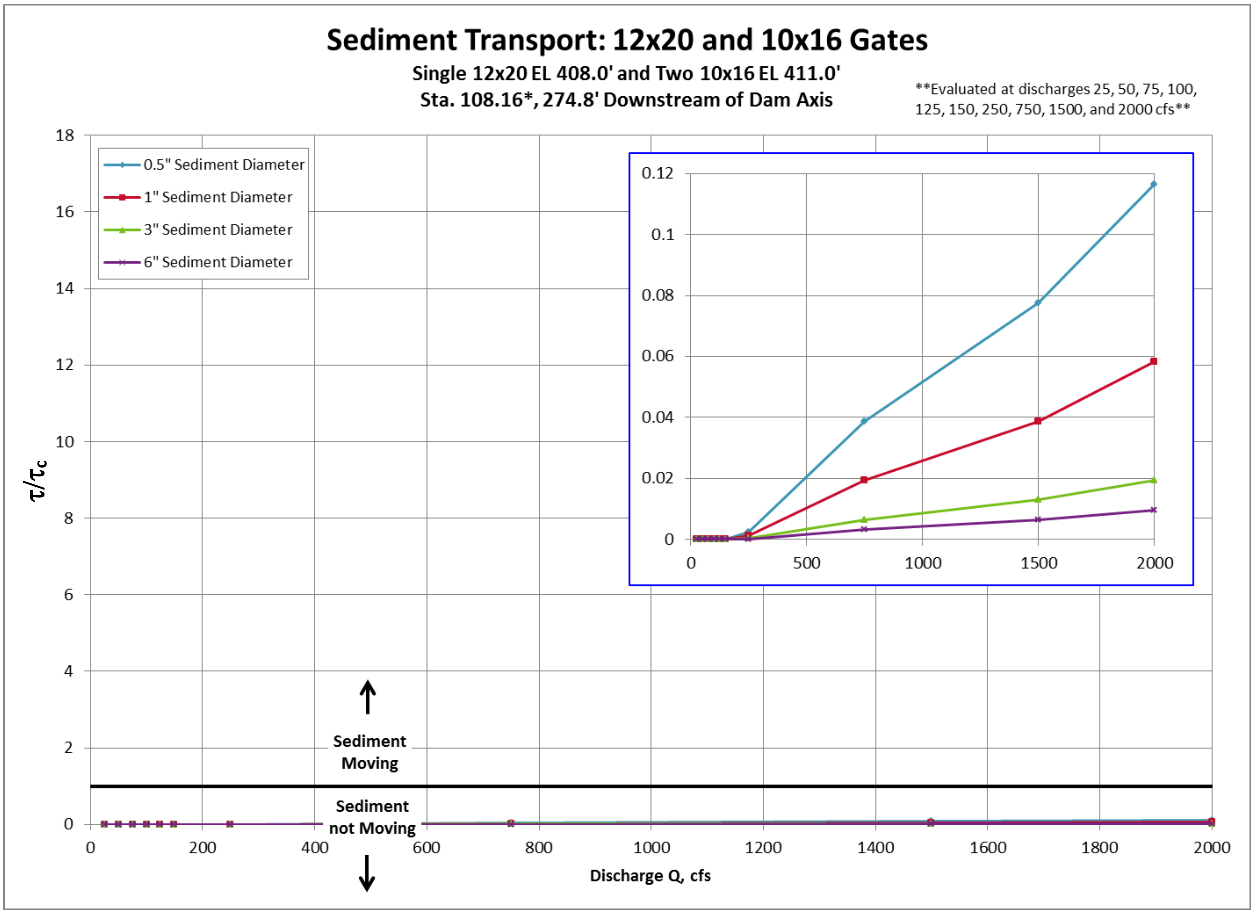
Note:
About 275 feet upstream of dam

Figure 3-9
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.23



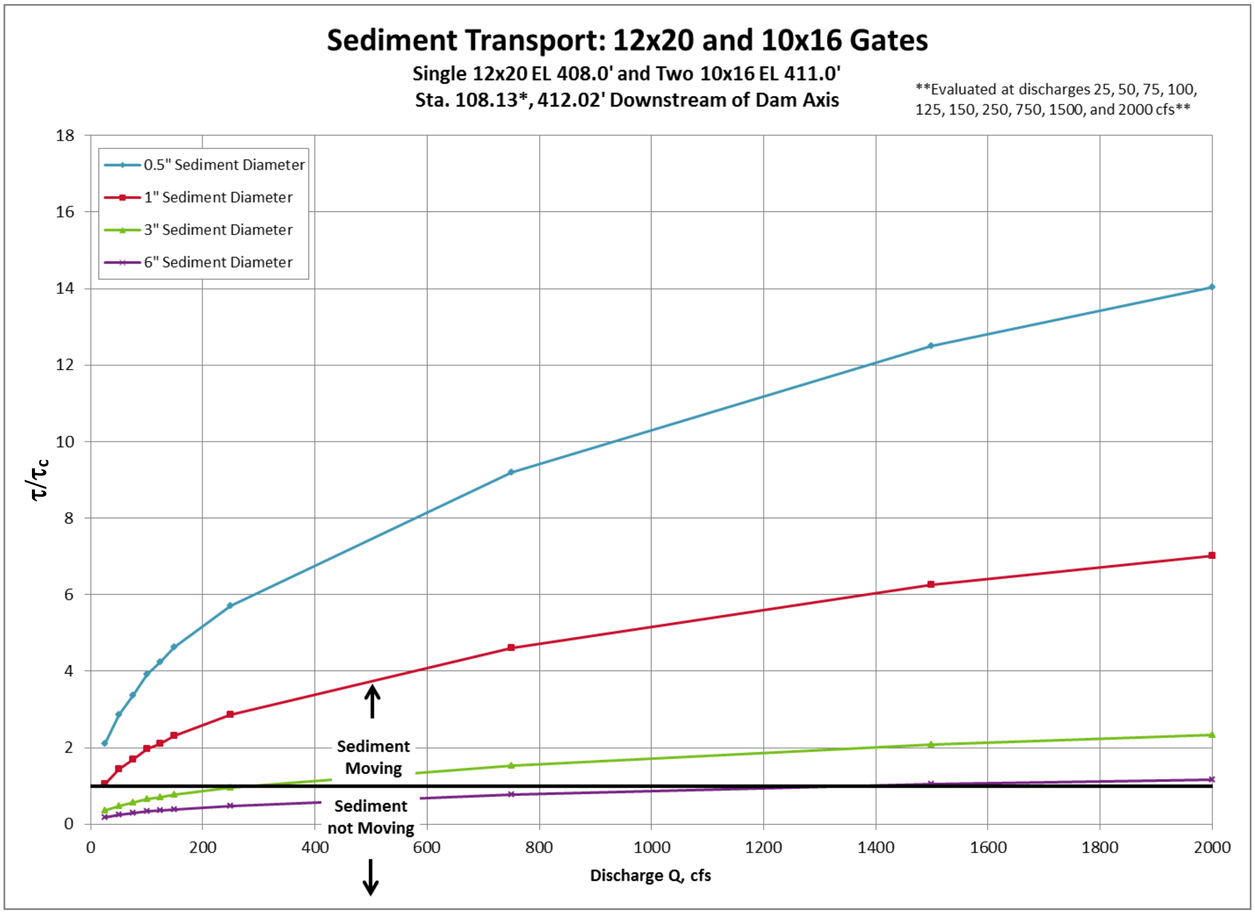
Note:
About 140 feet downstream of dam axis, within sluice conduits

Figure 3-10
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.16



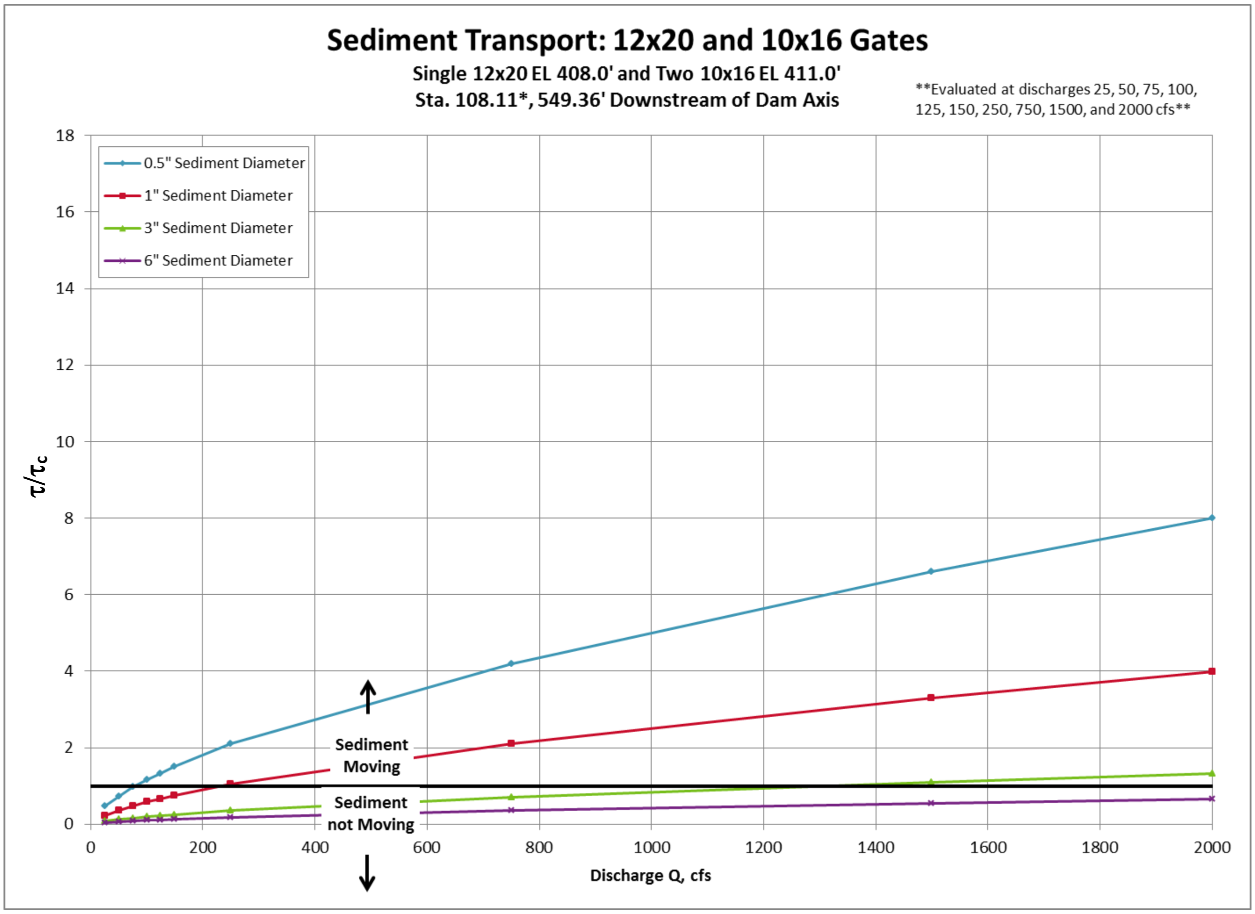
Notes:
About 275 feet downstream of dam axis, within stilling basin

Figure 3-11
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.13



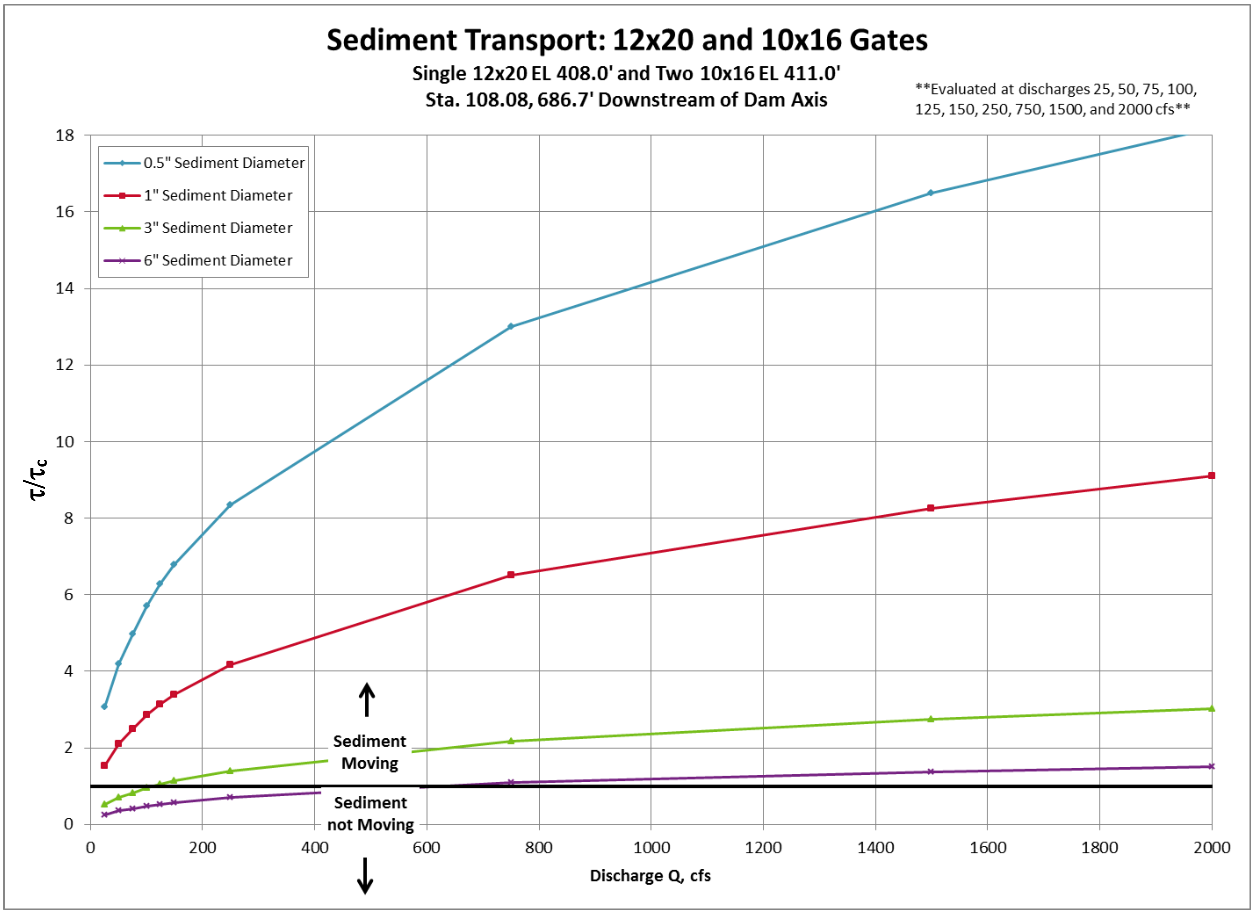
Note:
About 410 feet downstream of dam axis, within natural reach downstream of dam

Figure 3-12
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.11



Note:
About 550 feet downstream of dam axis, within natural reach downstream of dam

Figure 3-13
Comparison of Bed Shear Stress to Critical Shear Stress for Cross Section 108.08



Note:
About 690 feet downstream of dam axis, within natural reach downstream of dam

3.1.3 2D HEC-RAS Clear-Water and Sediment Transport Modeling

Figure 3-14

250 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

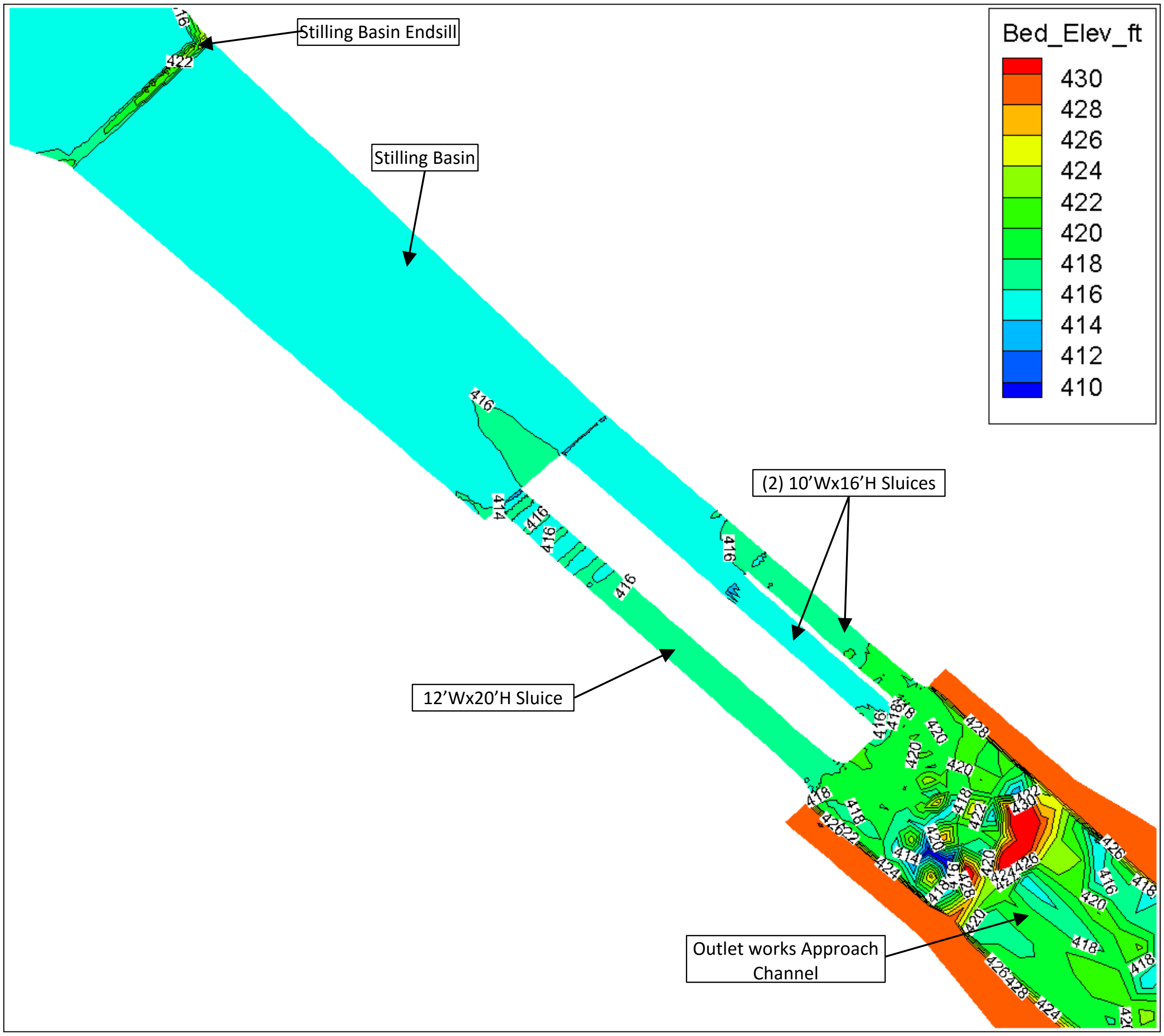


Figure 3-15
500 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

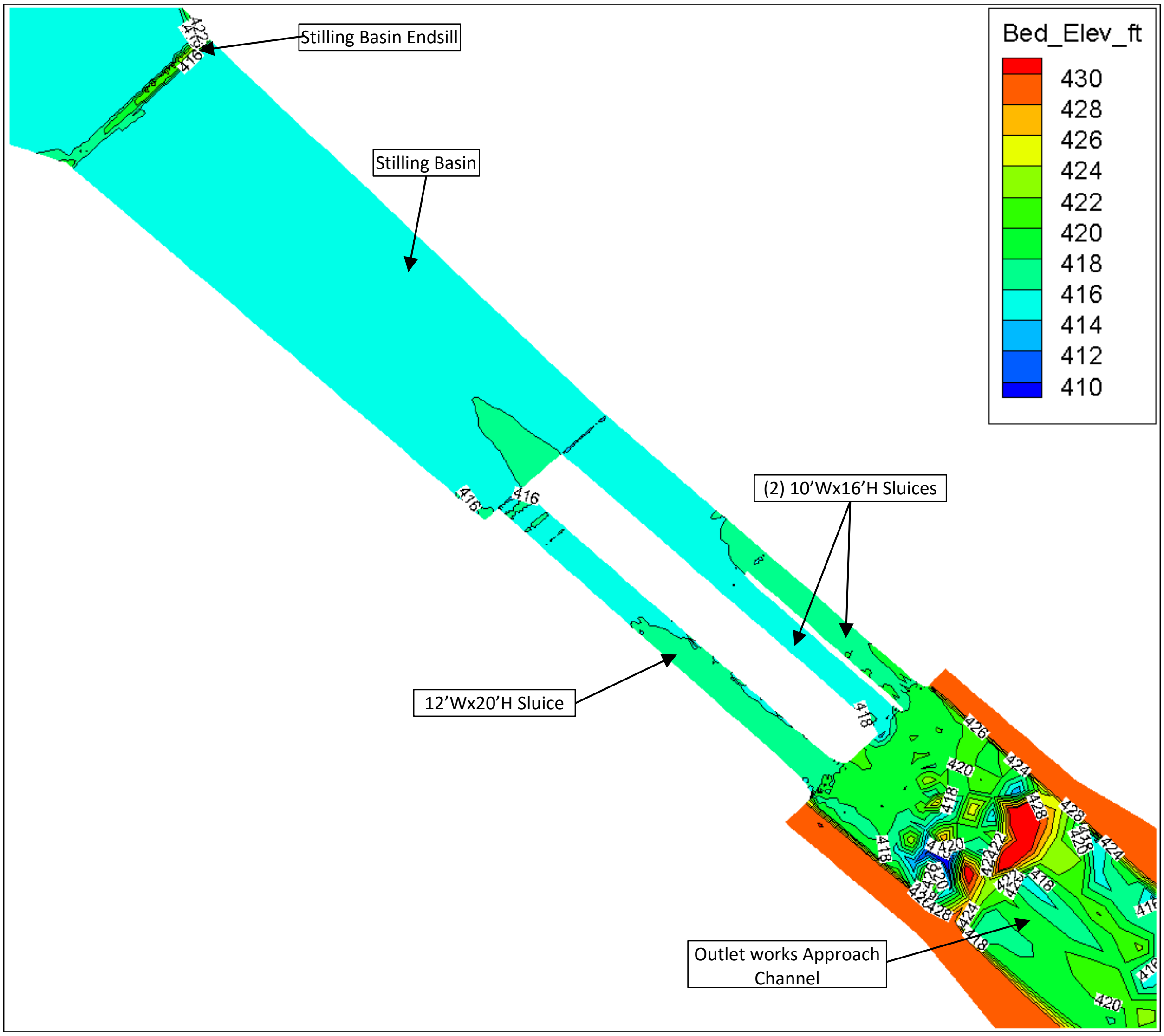


Figure 3-16
750 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

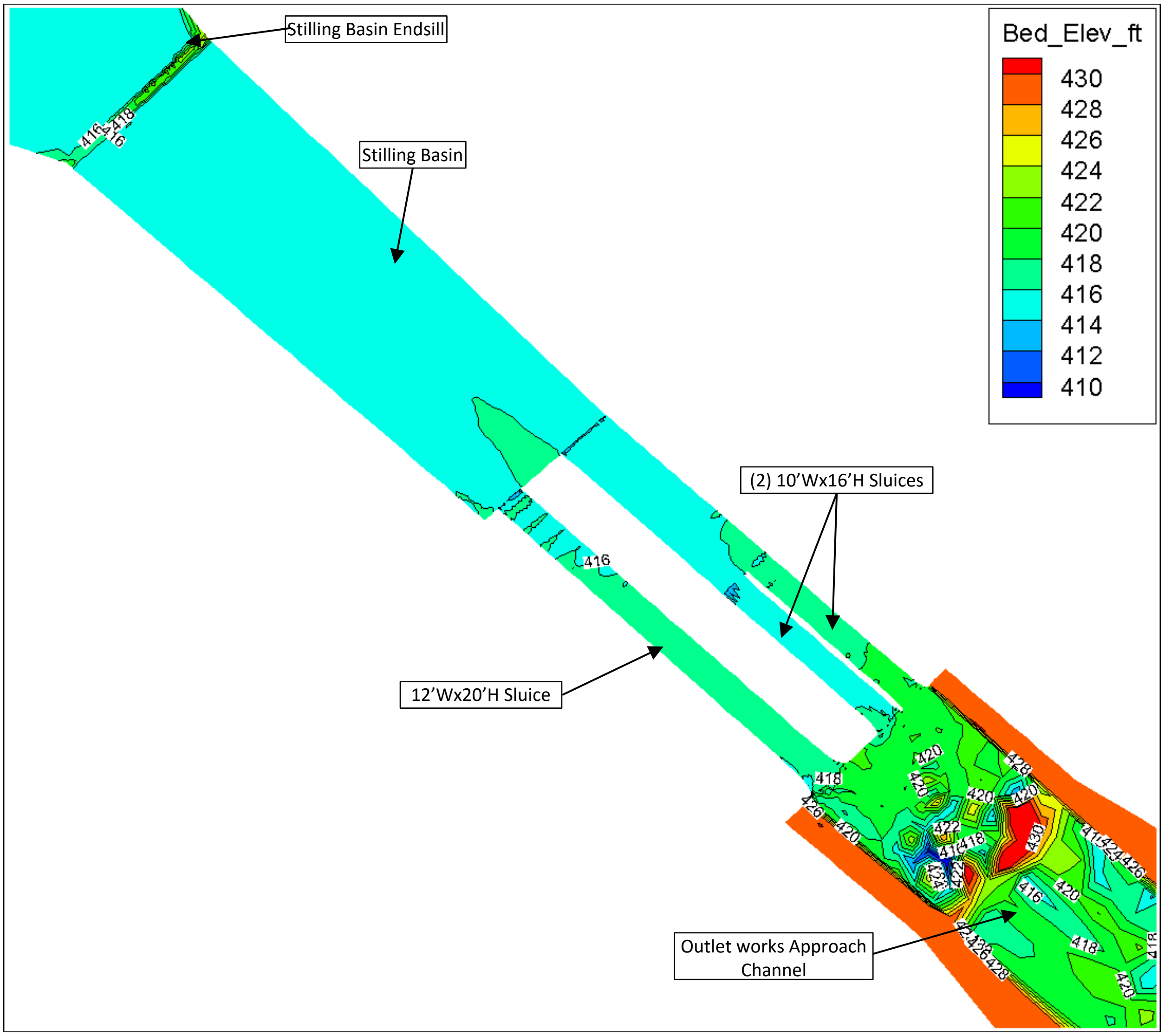


Figure 3-17
1,000 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

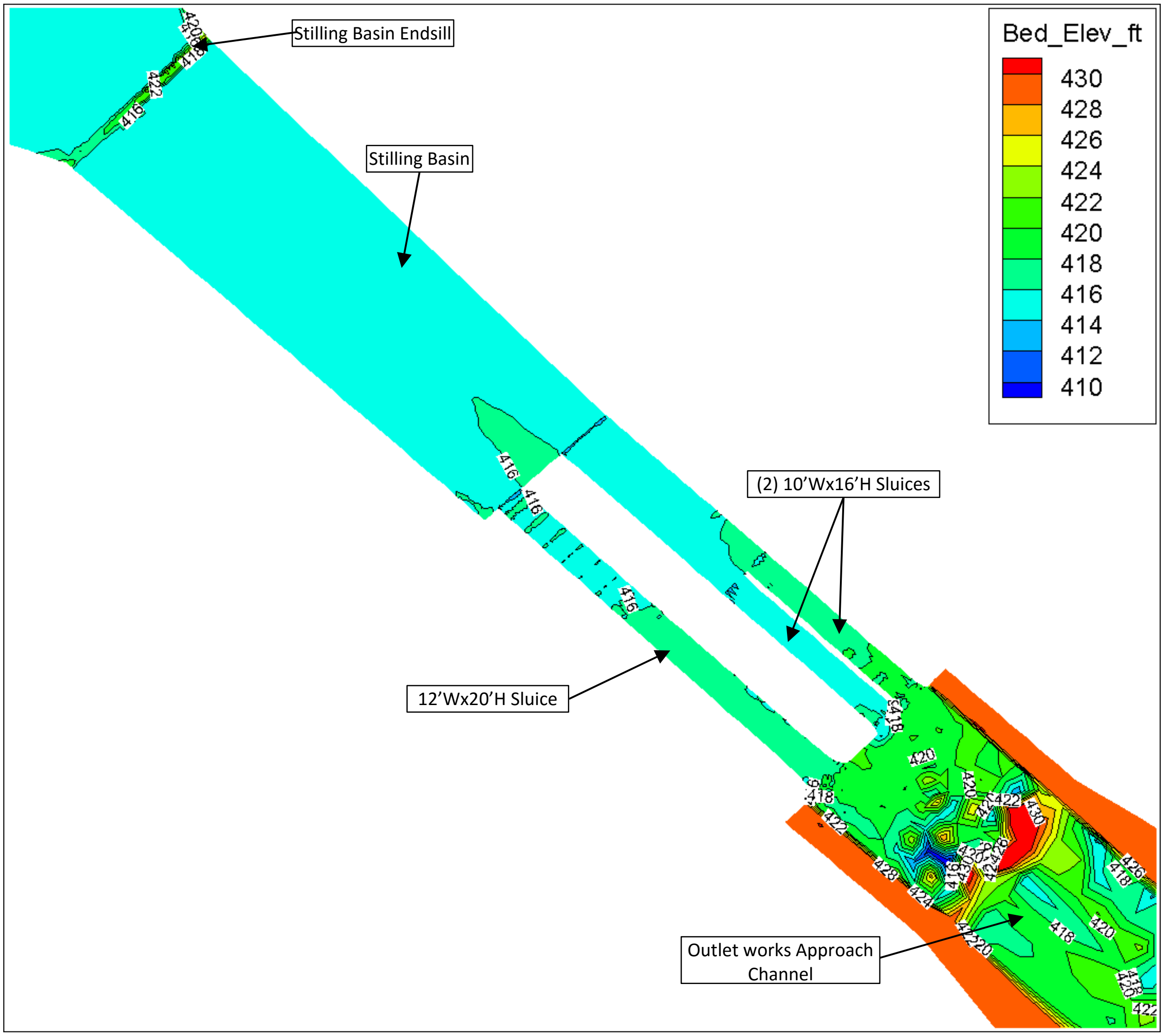


Figure 3-18
1,250 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

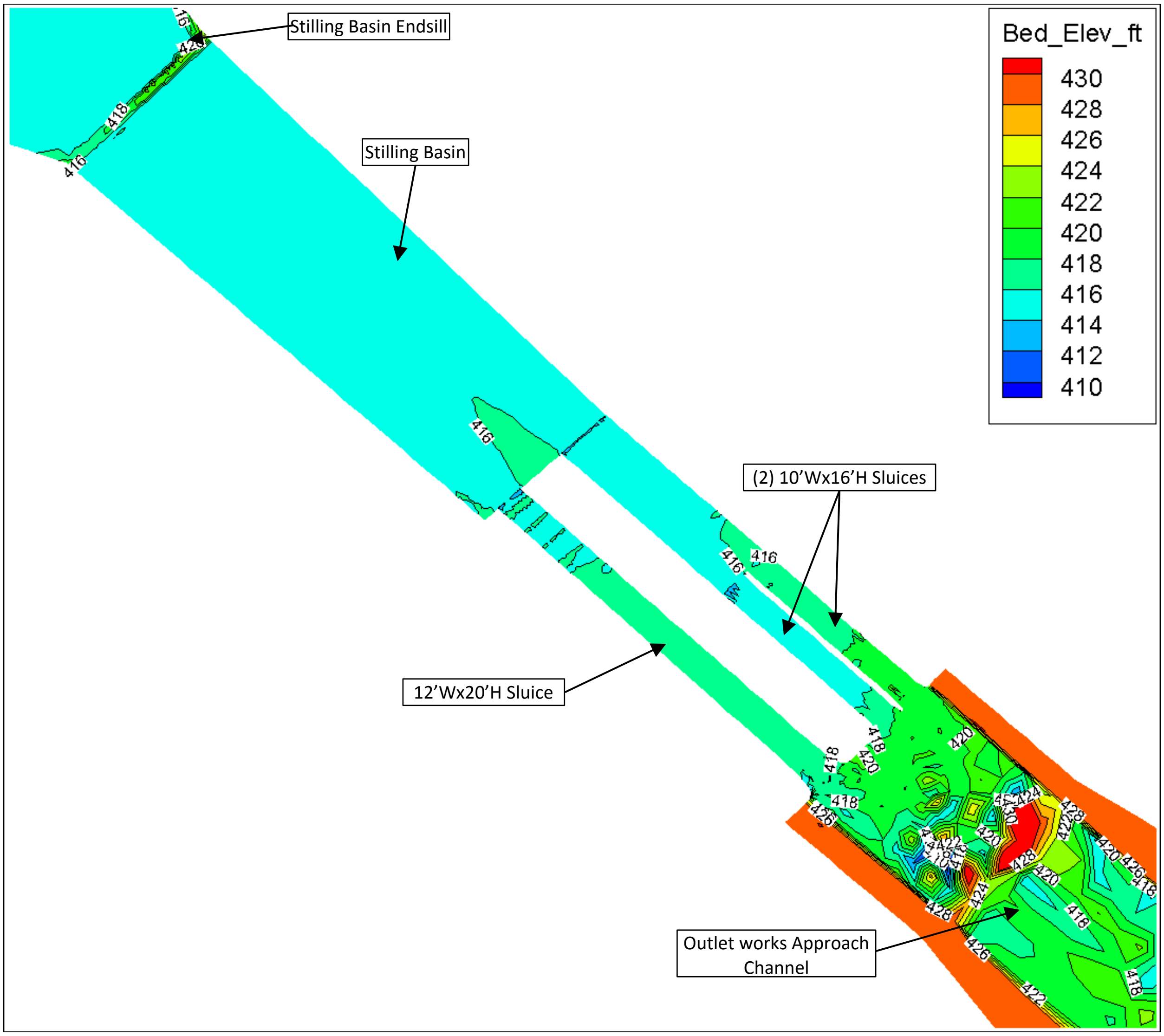


Figure 3-19

1,500 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

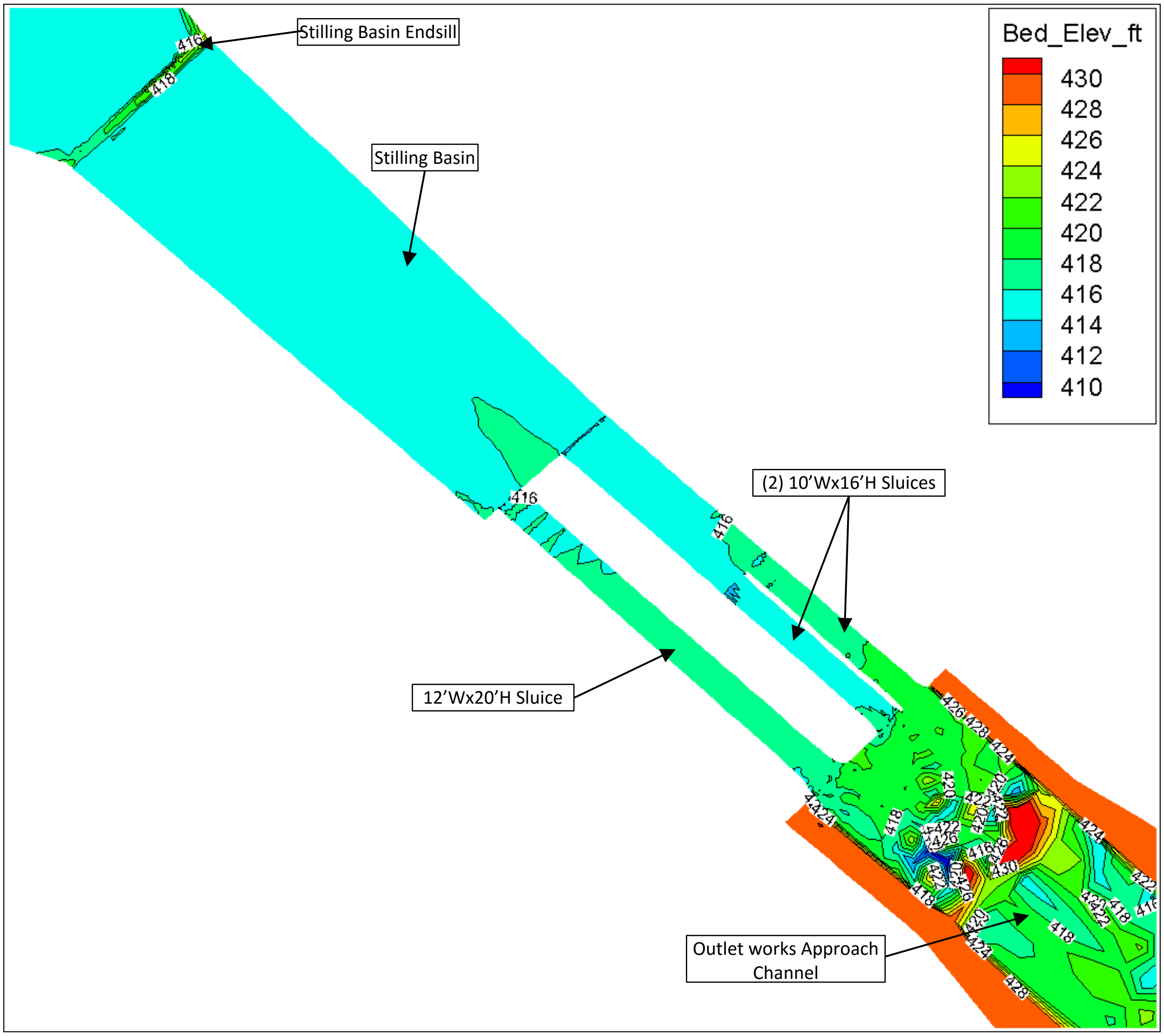


Figure 3-20
1,750 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

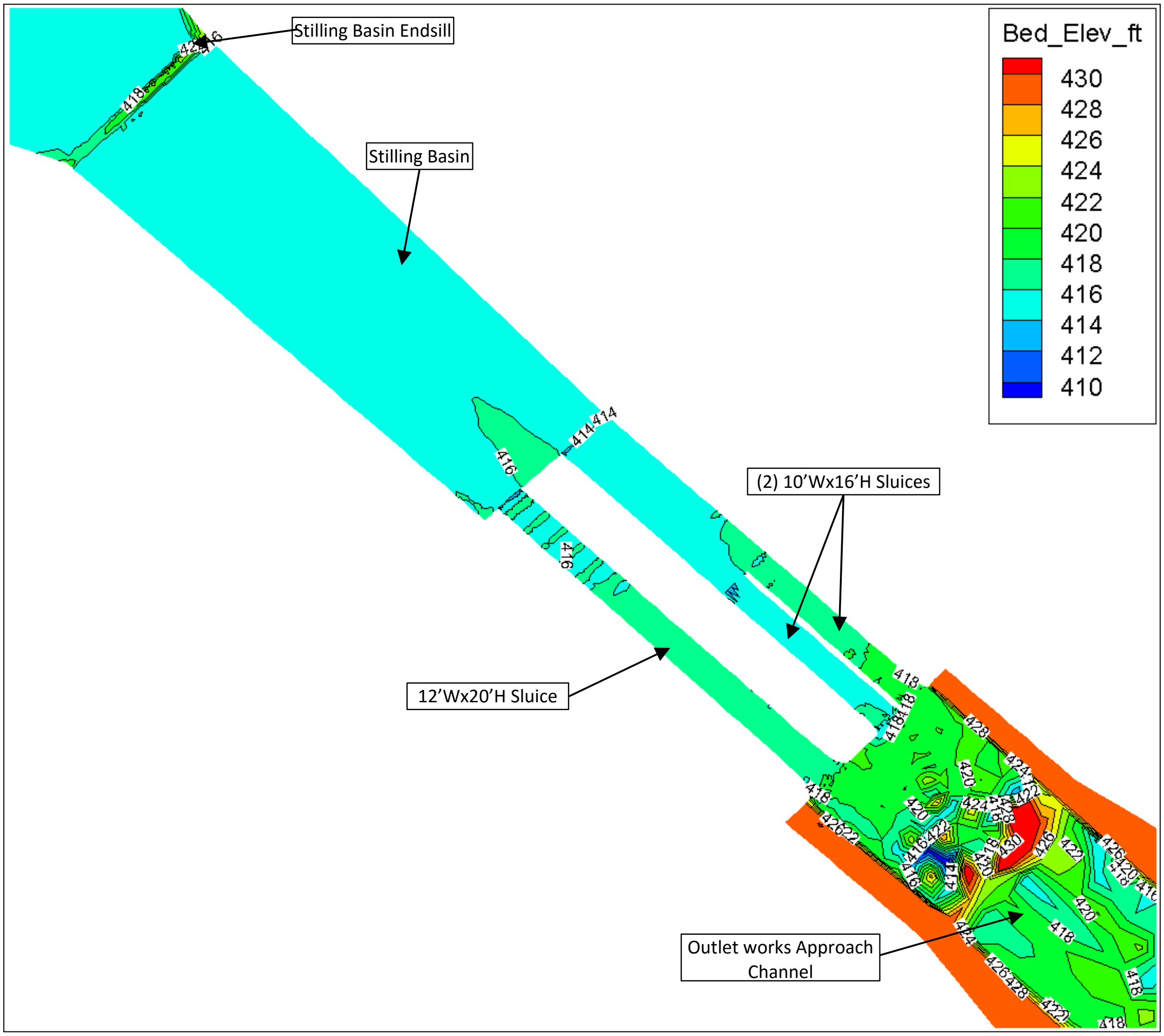


Figure 3-21
2,000 cfs Bed Elevation Contour Plot for With Project Post Event Sediment

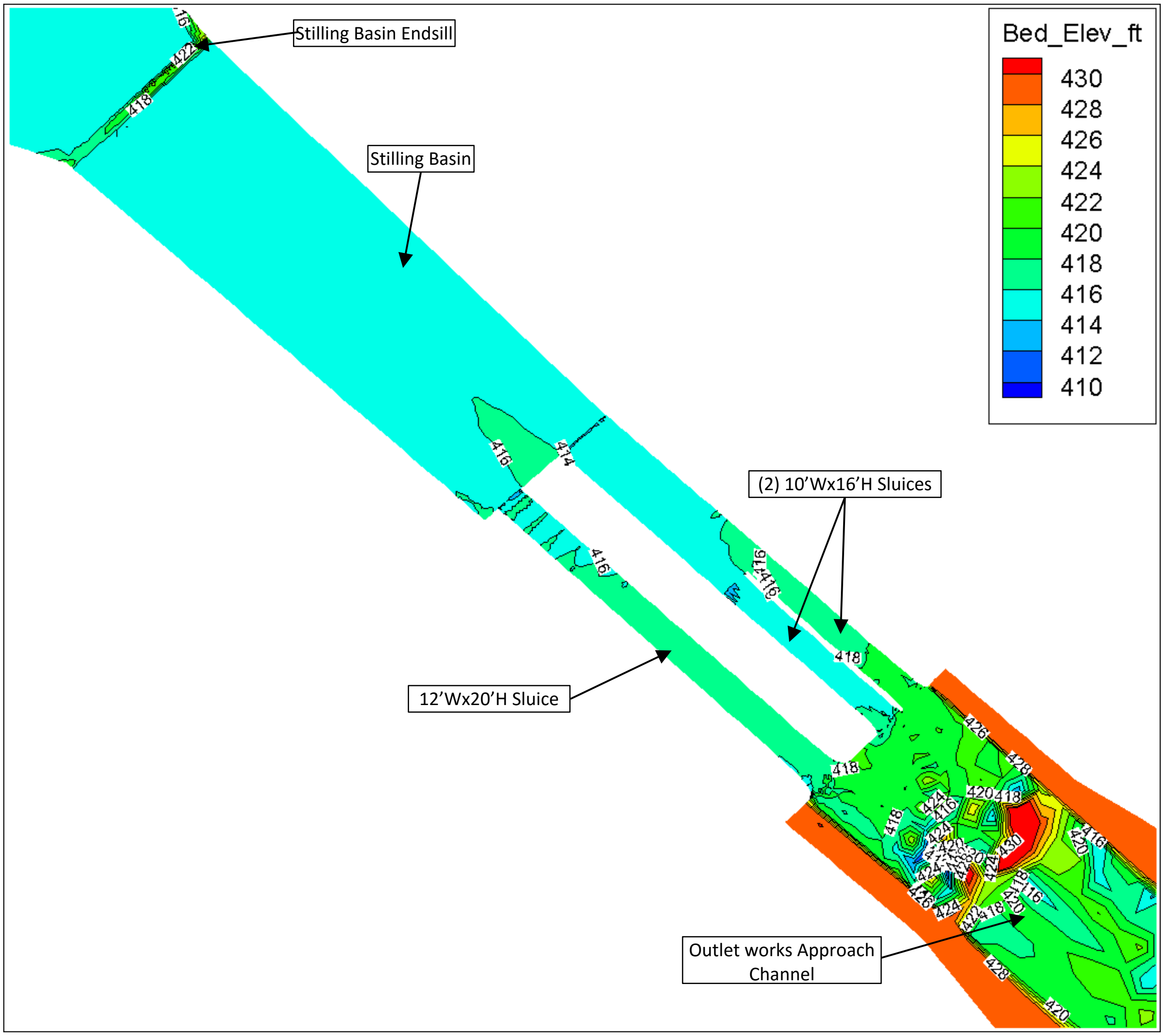


Figure 3-22
250 cfs Flow Velocity Contour Plot for With Project Post Event Sediment

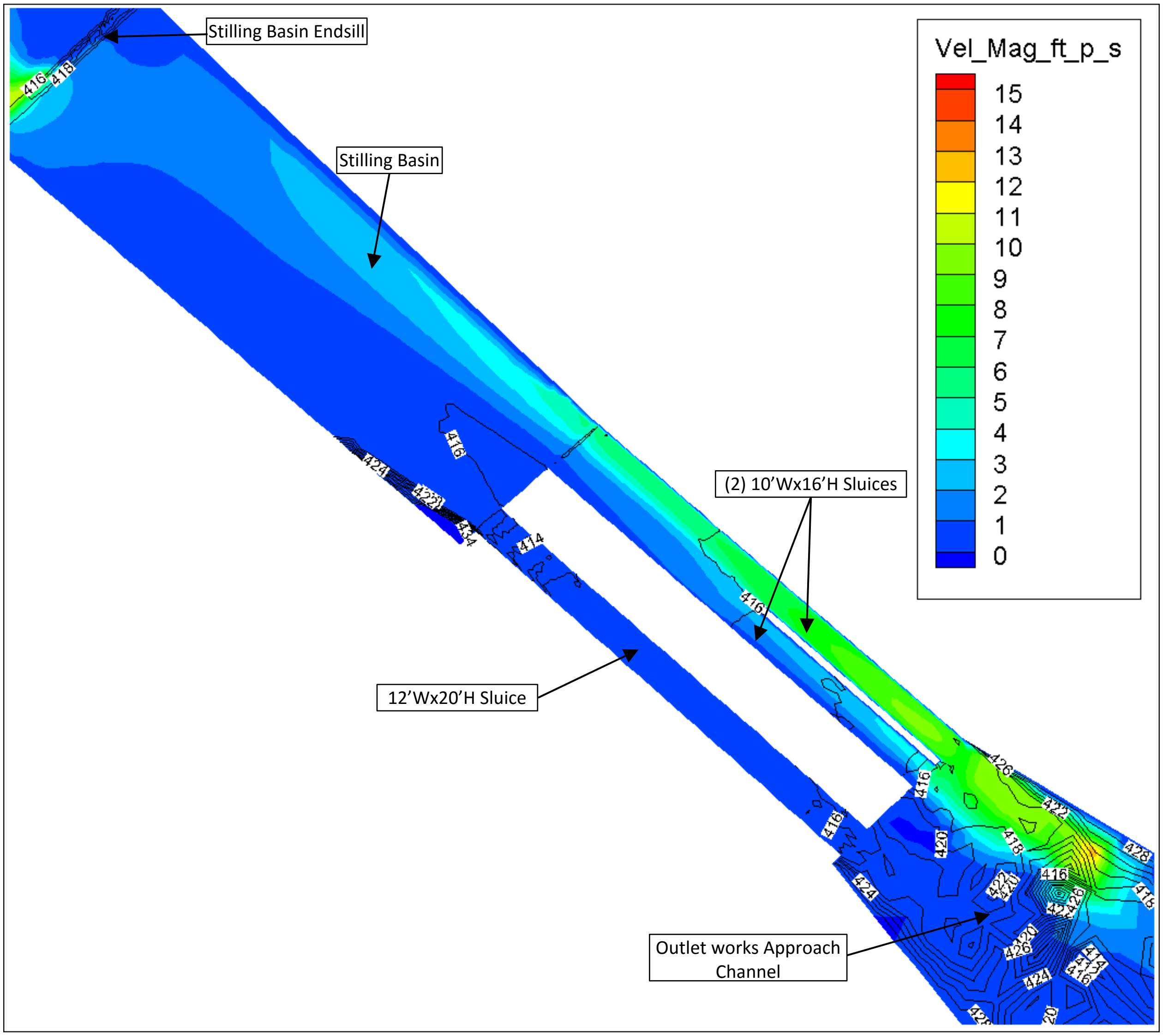


Figure 3-23
500 cfs Flow Velocity Contour Plot for With Project Post Event Sediment

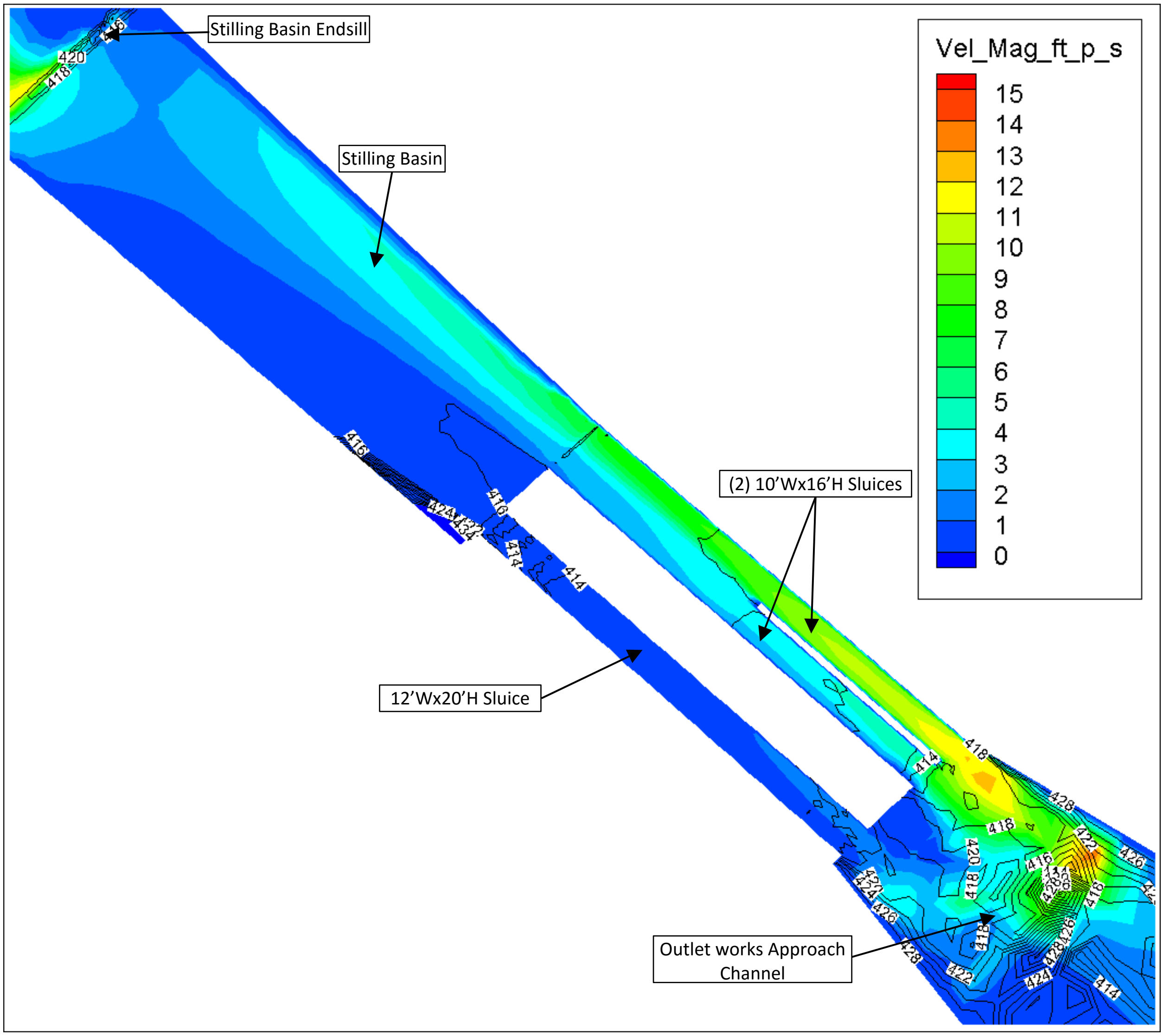


Figure 3-24
750 cfs Flow Velocity Contour Plot for With Project Post Event Sediment

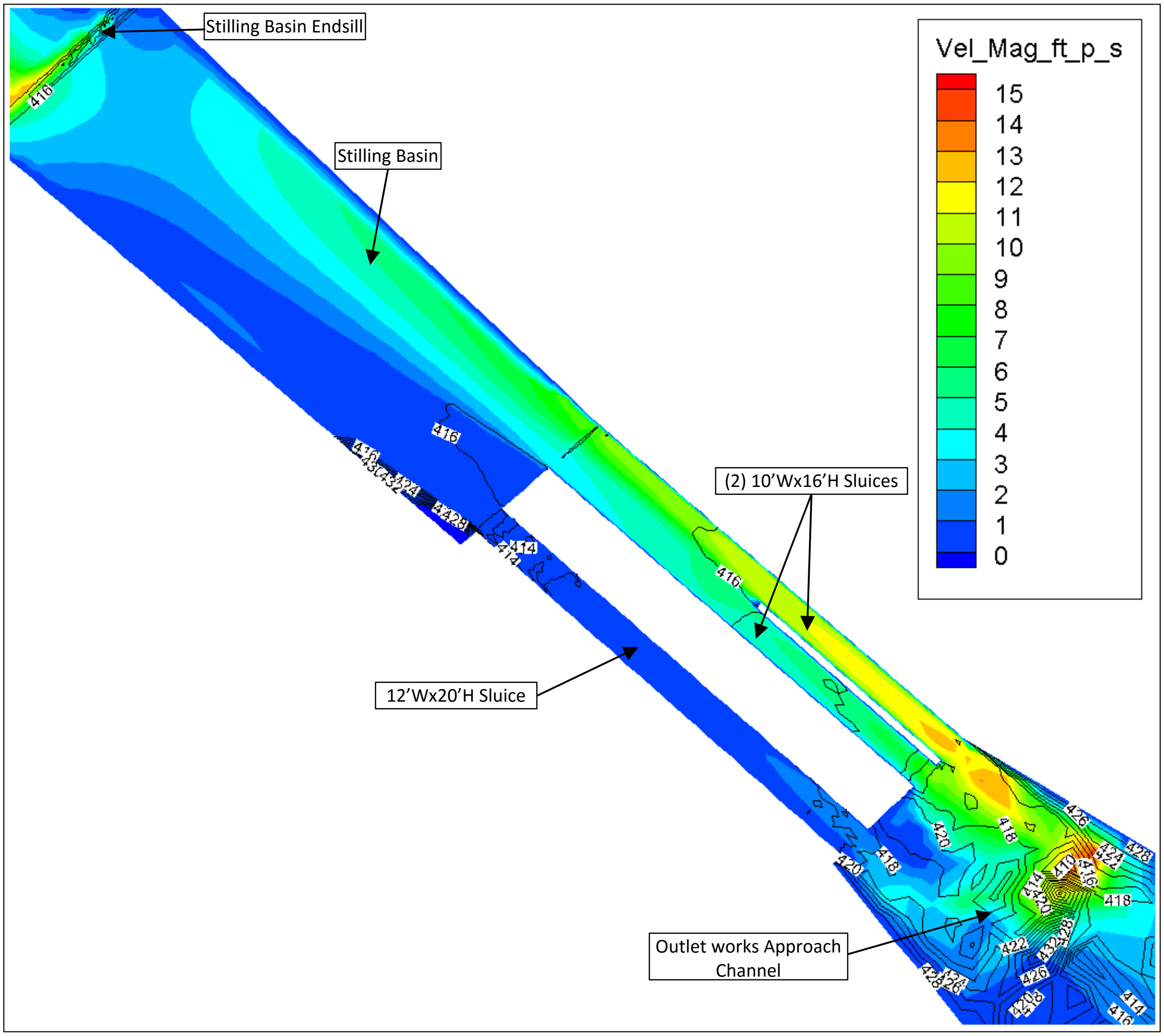


Figure 3-25
1,000 cfs Flow Velocity Contour Plot for With Project Post Event Sediment

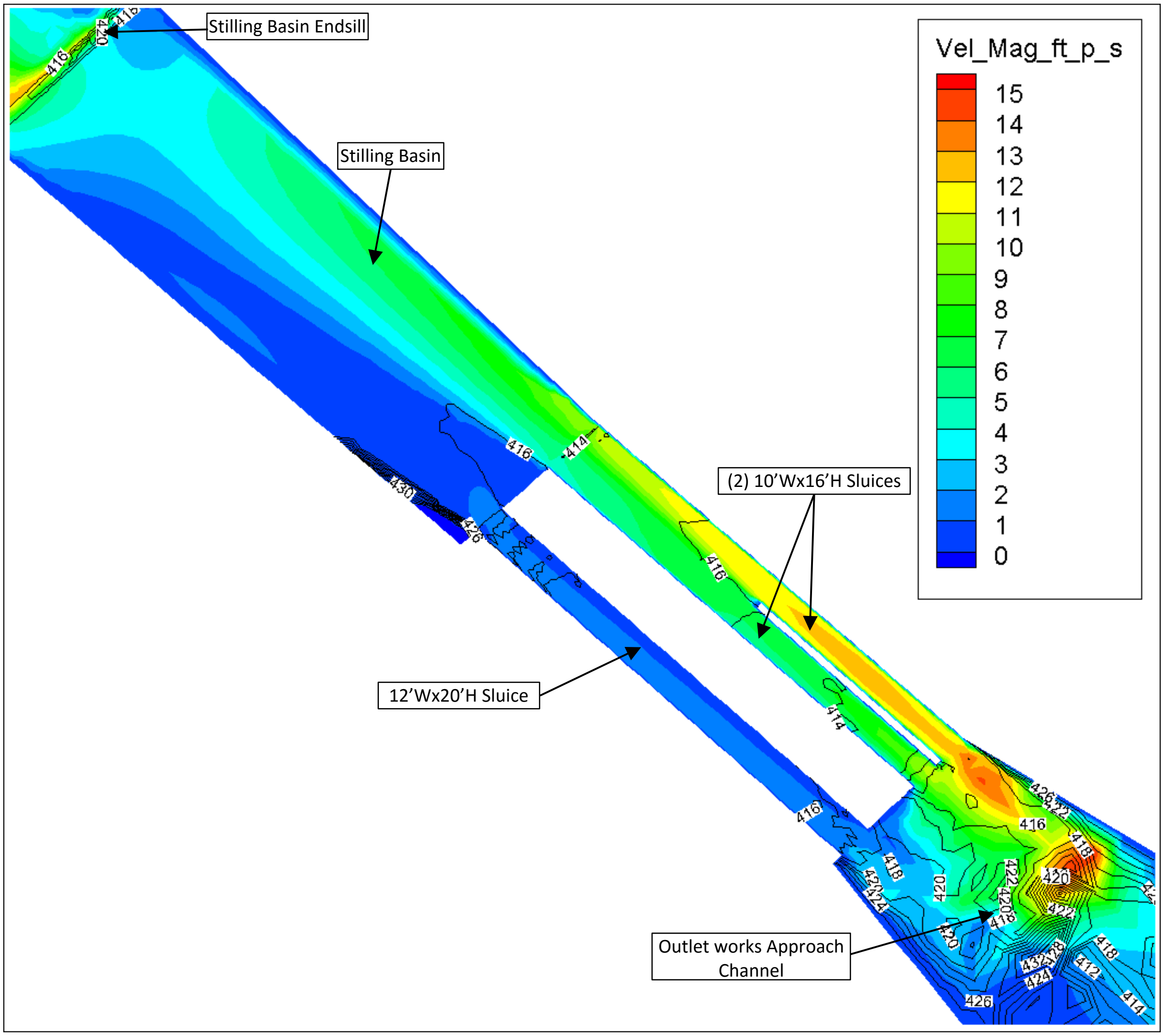


Figure 3-26

1,250 cfs Flow Velocity Contour Plot for With Project Post Event Sediment

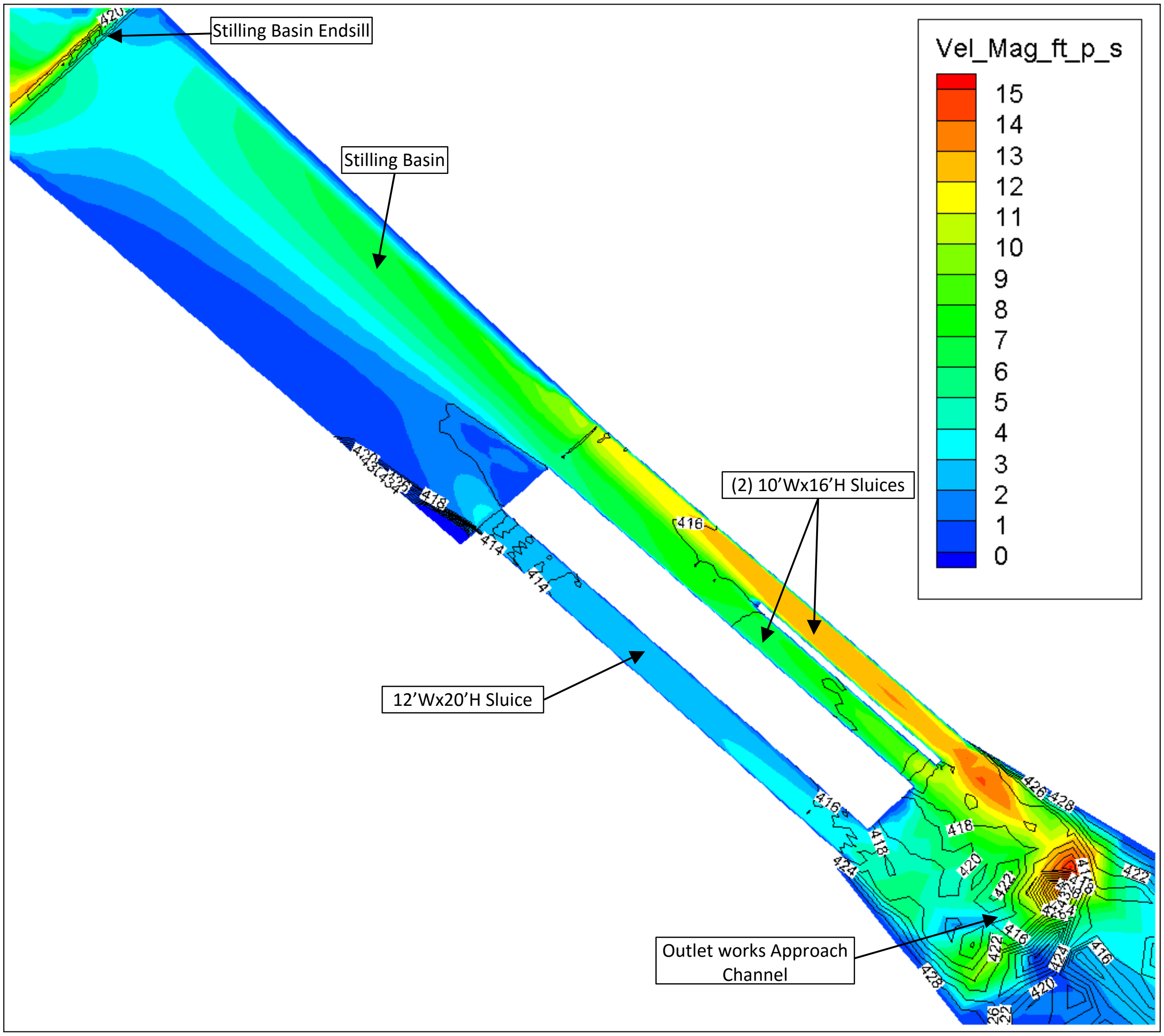


Figure 3-27
1,500 cfs Flow Velocity Contour Plot for With Project Post Event Sediment

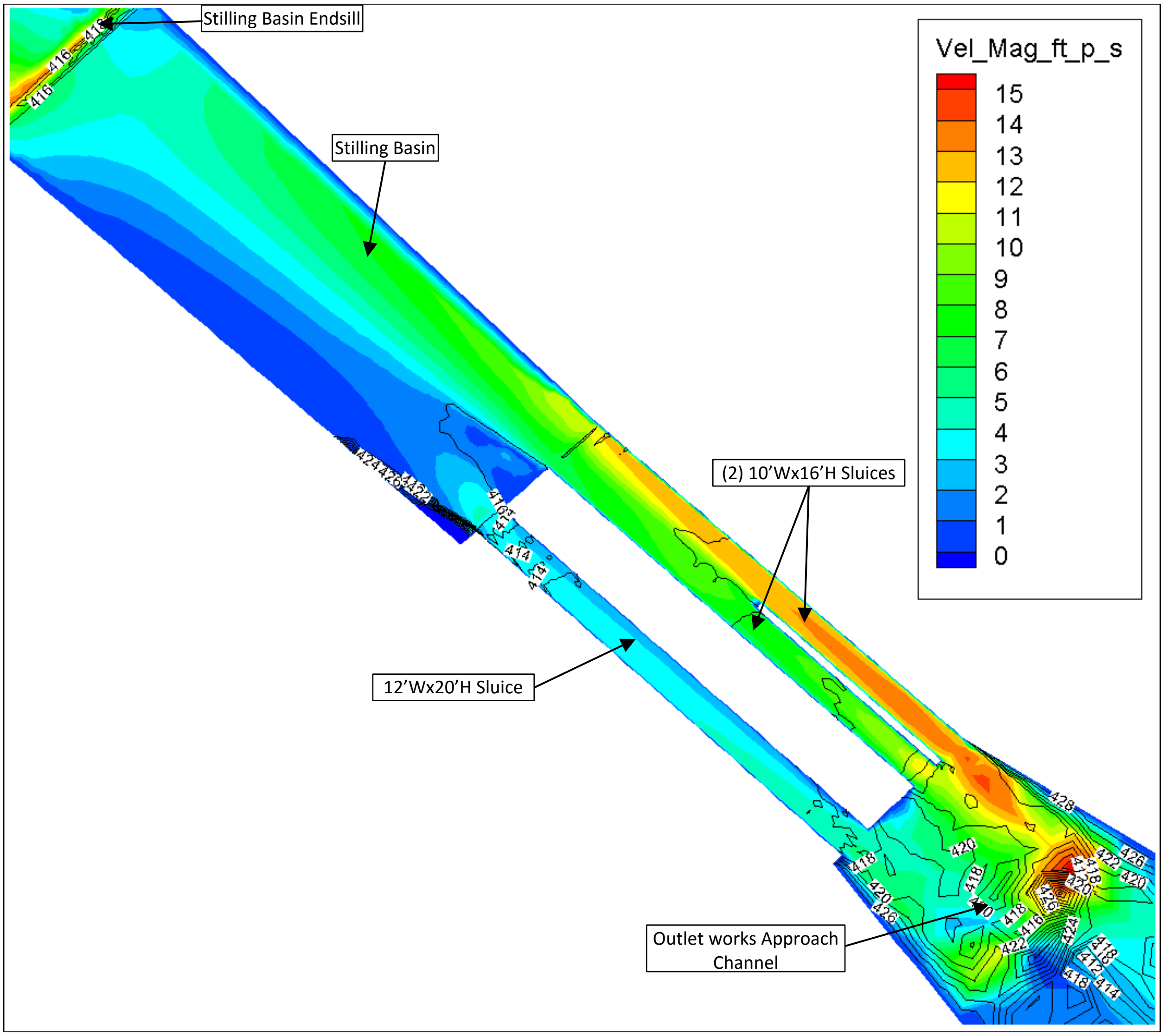


Figure 3-28
1,750 cfs Flow Velocity Contour Plot for With Project Post Event Sediment

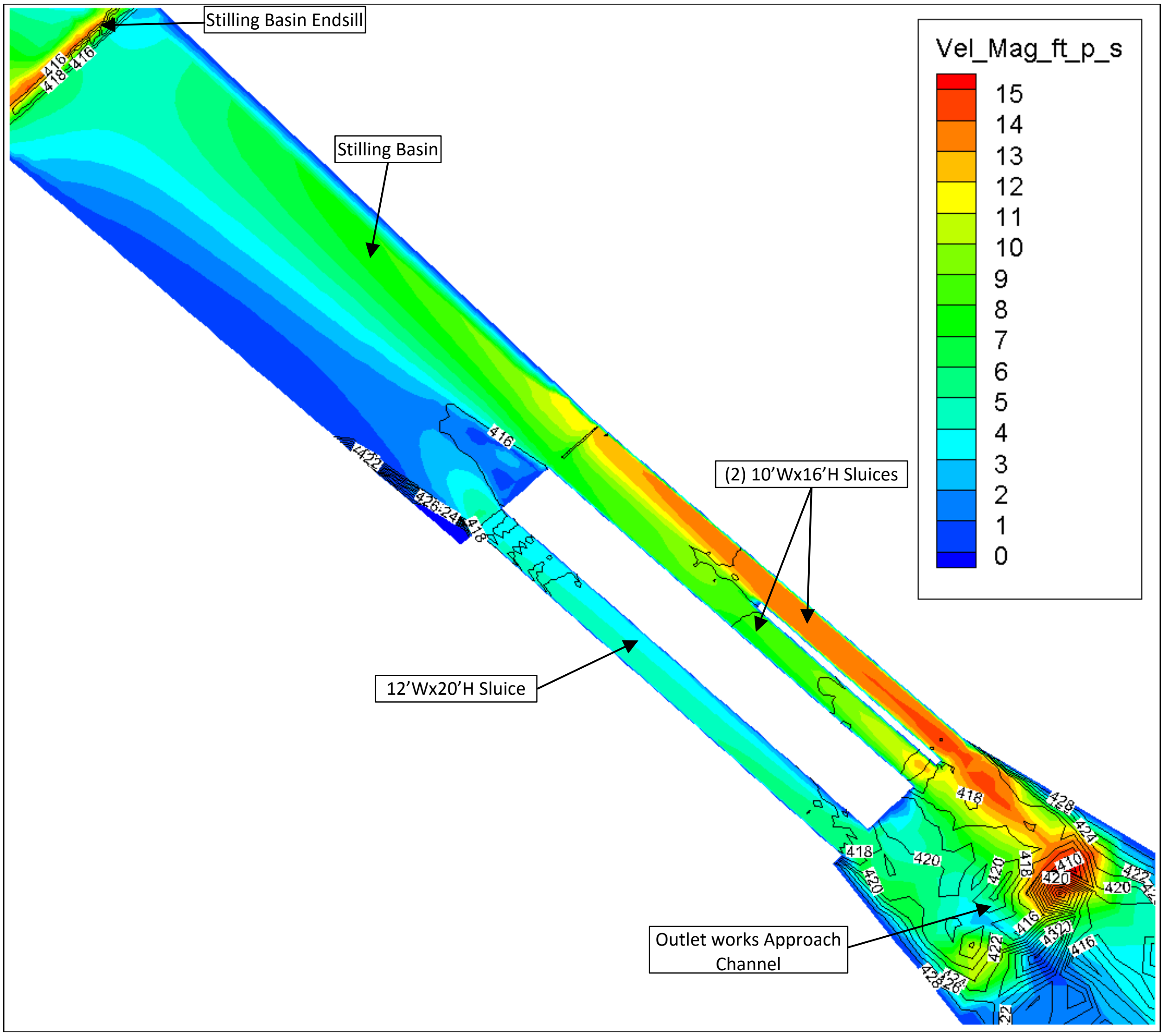
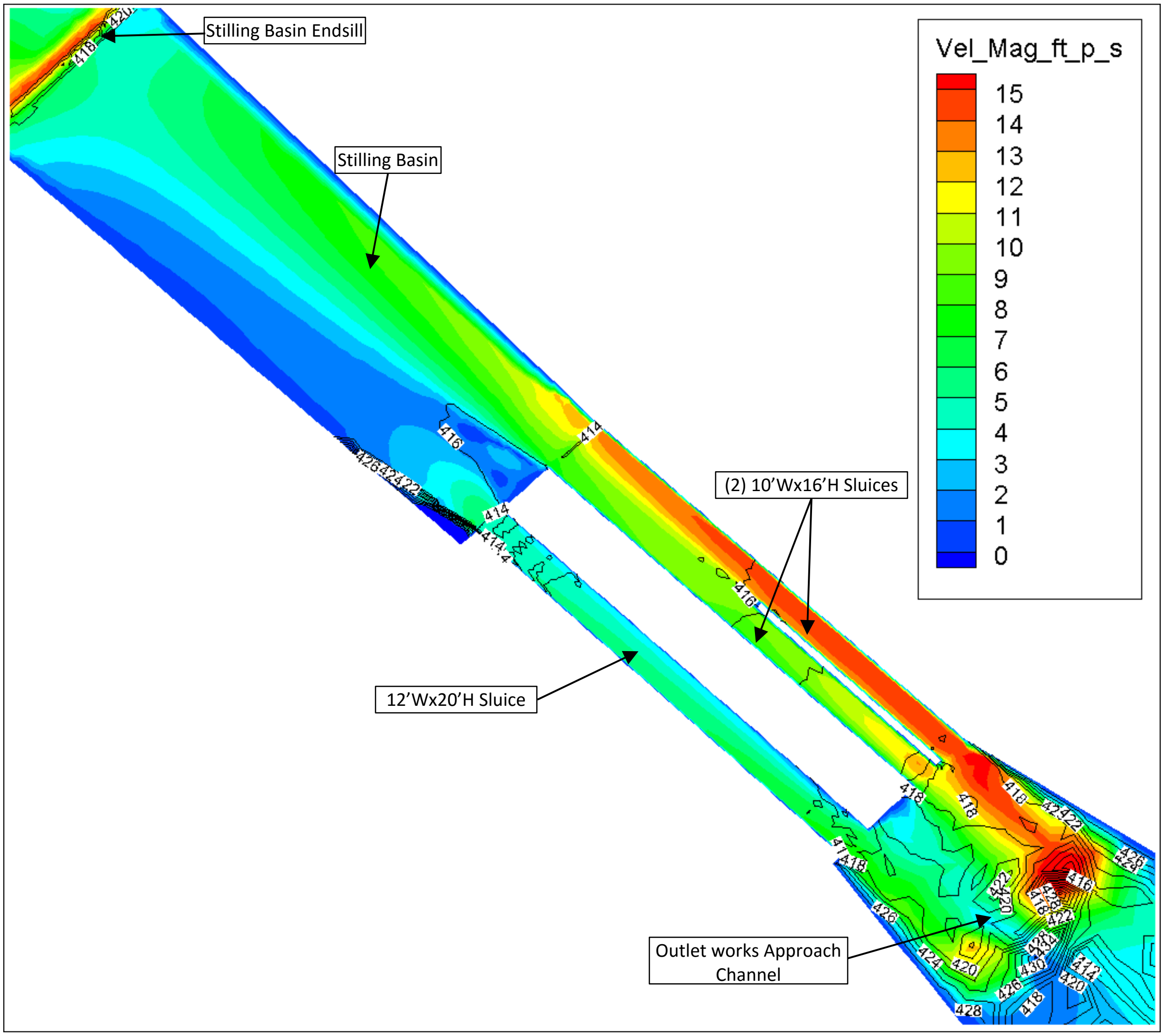


Figure 3-29

2,000 cfs Flow Velocity Contour Plot for With Project Post Event Sediment



3.2 Diversion Tunnel Rating

The hydraulic characteristic curves of the diversion tunnel were established and developed to aid with determining the flow condition and control type. Figure 3-30 and

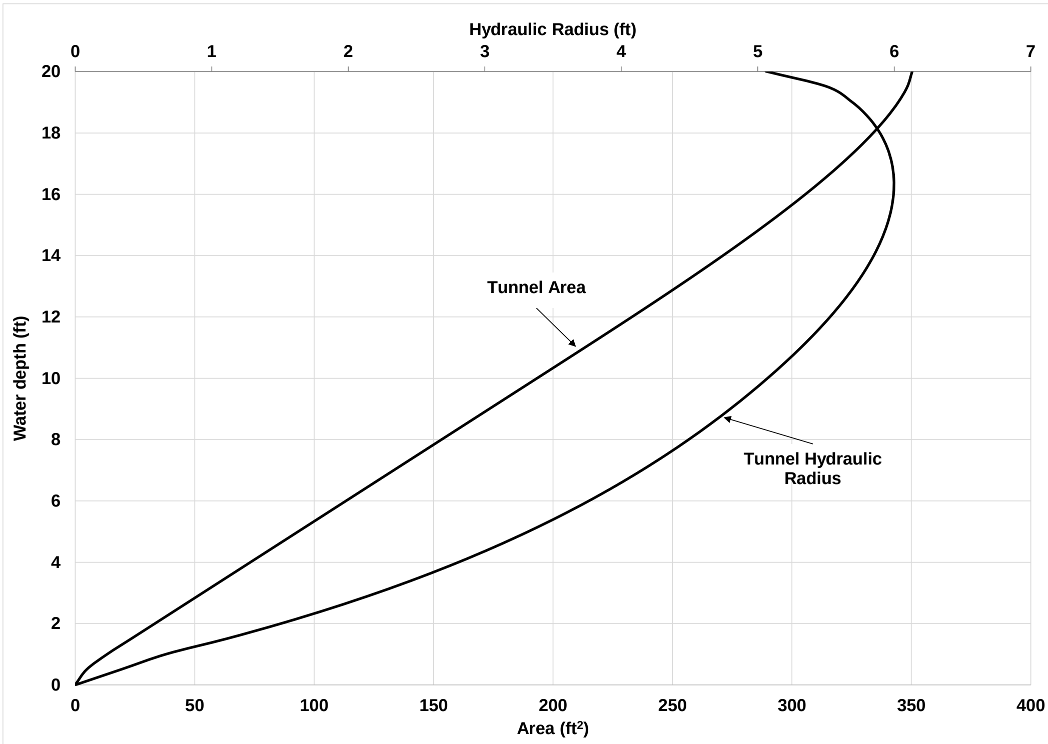


Figure 3-31 show the hydraulic characteristic curves for the horseshoe-shaped tunnel cross section. Based on the discharge curves (

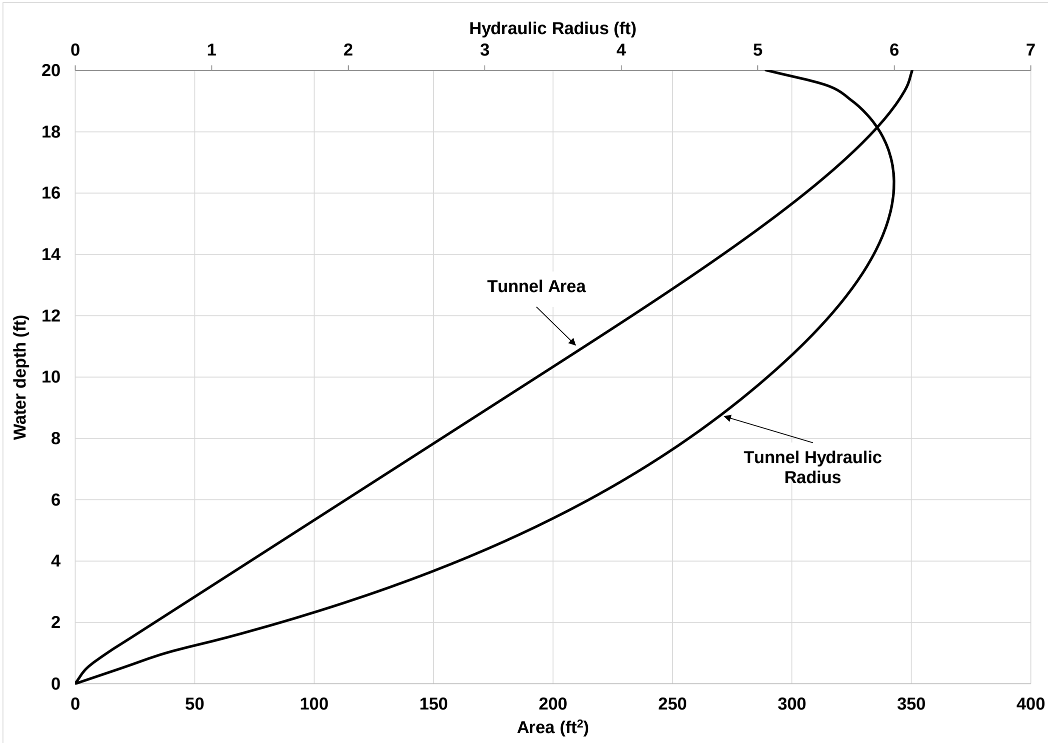


Figure 3-31), when open channel flow occurs in the tunnel, normal depth is smaller than critical depth for a given discharge. Therefore, the tunnel is a steep slope and critical depth control occurs at the inlet. The discharge through the tunnel can be calculated using the critical flow formula when free surface flow at the inlet exists.

Figure 3-30
Area and Hydraulic Radius of the Tunnel for Various Water Depths

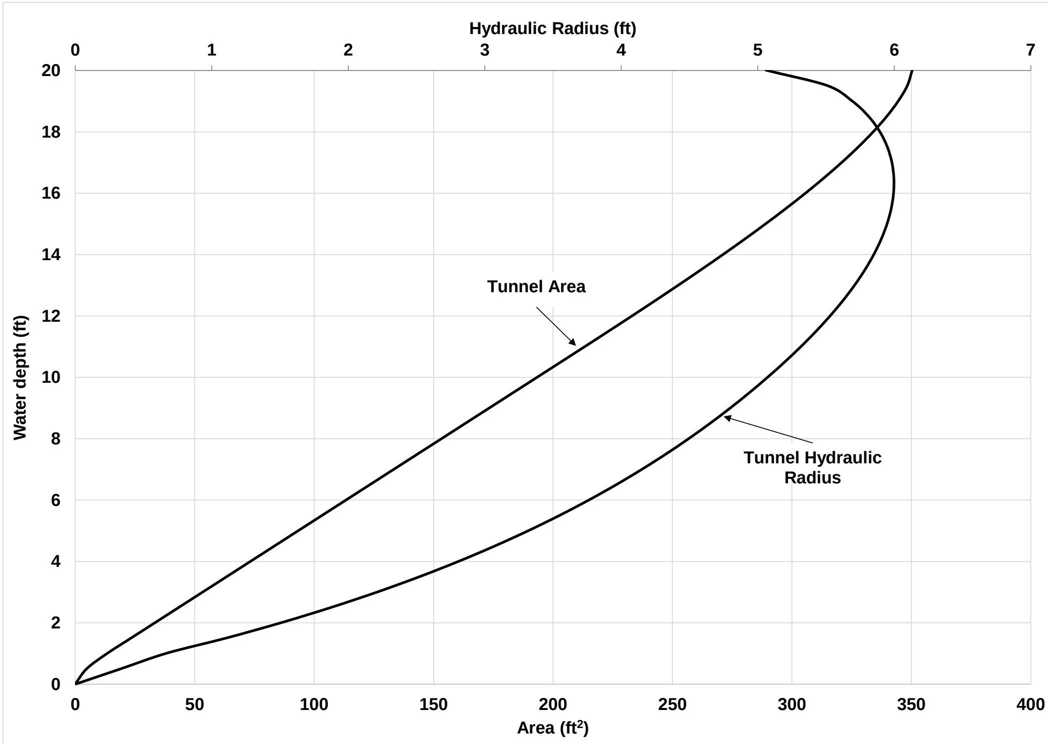


Figure 3-31
Discharge Curves for Various Water Depths for Open Channel Flow Condition

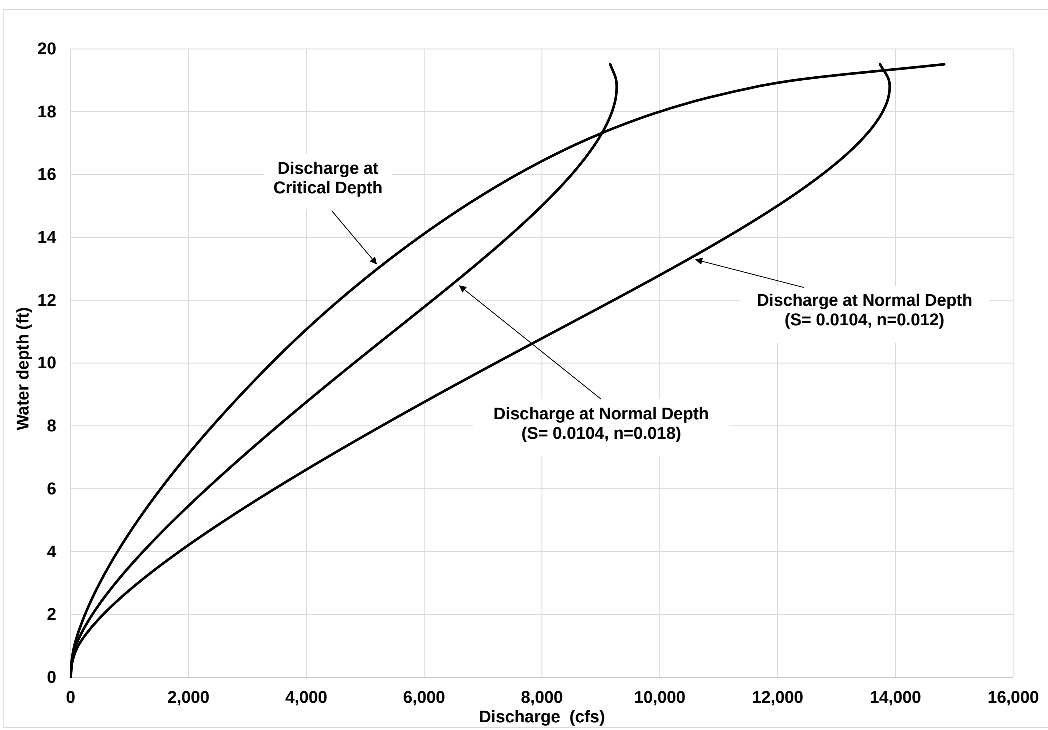


Table 3-1 shows the computation of diversion tunnel inlet rating curve when free surface flow exists at the inlet.

Tunnel invert elevation = 427 feet msl

Tunnel crown elevation= 447 feet msl

K_e= 0.25, entrance loss coefficient

Pool elevation = conduit invert elevation + y_c + $(K_e+K_v)\ V^2/2g$

Table 3-1
Diversion Tunnel Inlet Rating Calculation for Free Surface Flow Condition

Q (cfs)	y_c (ft)	A (ft^2)	V (ft/s)	1.25 $V^2/2g$ (ft)	Pool Elevation (ft)
0.0	0.0	0.0	0.5	0.0	427.0
16.0	0.5	4.8	3.3	0.2	427.7
62.9	1.0	13.5	4.7	0.4	428.4
144.4	1.5	23.5	6.1	0.7	429.2
245.8	2.0	33.5	7.3	1.0	430.1
363.8	2.5	43.5	8.4	1.4	430.9
496.2	3.0	53.5	9.3	1.7	431.7
641.7	3.5	63.5	10.1	2.0	432.5
799.1	4.0	73.5	10.9	2.3	433.3
967.6	4.5	83.5	11.6	2.6	434.1
1146.6	5.0	93.5	12.3	2.9	434.9
1335.4	5.5	103.5	12.9	3.2	435.7
1533.5	6.0	113.5	13.5	3.5	436.6
1740.6	6.5	123.5	14.1	3.9	437.4
1956.2	7.0	133.5	14.7	4.2	438.2
2180.1	7.5	143.5	15.2	4.5	439.0
2412.0	8.0	153.5	15.7	4.8	439.8
2651.5	8.5	163.5	16.2	5.1	440.6
2898.4	9.0	173.5	16.7	5.4	441.4
3152.6	9.5	183.5	17.2	5.7	442.2
3413.8	10.0	193.5	17.6	6.0	443.1
3684.1	10.5	203.5	18.1	6.4	443.9
3965.7	11.0	213.5	18.6	6.7	444.7
4258.9	11.5	223.4	19.1	7.1	445.6
4564.2	12.0	233.2	19.6	7.4	446.4

Notes:
Q= discharge
Y_c= critical depth
A= flow area
V=average velocity

When the tunnel inlet becomes submerged, the rating curve can be computed using orifice formula. The rating curve computation for the submerged inlet condition is provided in Table 3-2.

C= 0.6, discharge coefficient, slightly less than common discharge coefficient for orifice (C=0.67) to account for floating debris

K_e= 0.5, entrance loss coefficient

E=Energy grade line at the inlet

Pool elevation= EGL+ $K_e V^2/2g$

Table 3-2

Diversion Tunnel Inlet Rating Calculation for Submerged Inlet Condition

EGL(ft)	Q(cfs)	A (ft^2)	V (ft/s)	$V^2/2g$ (ft)	0.5 $V^2/2g$ (ft)	Pool Elevation (ft)
447.0	5337.5	350.4	15.2	3.6	1.8	448.8
448.0	5595.3	350.4	16.0	4.0	2.0	450.0
449.0	5841.6	350.4	16.7	4.3	2.2	451.1
450.0	6078.0	350.4	17.3	4.7	2.3	452.3
451.0	6305.5	350.4	18.0	5.0	2.5	453.5
452.0	6525.1	350.4	18.6	5.4	2.7	454.7
453.0	6737.6	350.4	19.2	5.7	2.9	455.8
453.9	6943.5	350.4	19.8	6.1	3.0	457.0
454.9	7143.6	350.4	20.4	6.5	3.2	458.2
455.9	7338.1	350.4	20.9	6.8	3.4	459.3
456.9	7527.7	350.4	21.5	7.2	3.6	460.5
457.9	7712.5	350.4	22.0	7.5	3.8	461.7
458.9	7893.1	350.4	22.5	7.9	3.9	462.8
459.9	8069.6	350.4	23.0	8.2	4.1	464.0
460.9	8242.3	350.4	23.5	8.6	4.3	465.2
461.9	8411.5	350.4	24.0	8.9	4.5	466.3
462.9	8577.4	350.4	24.5	9.3	4.7	467.5
463.8	8740.1	350.4	24.9	9.7	4.8	468.7
464.8	8899.8	350.4	25.4	10.0	5.0	469.8
465.8	9056.7	350.4	25.8	10.4	5.2	471.0
466.8	9212.5	350.4	26.3	10.7	5.4	472.2
467.8	9365.7	350.4	26.7	11.1	5.5	473.4
468.8	9516.4	350.4	27.2	11.5	5.7	474.5
469.8	9664.8	350.4	27.6	11.8	5.9	475.7
470.8	9810.9	350.4	28.0	12.2	6.1	476.9
471.8	9954.9	350.4	28.4	12.5	6.3	478.1
472.8	10096.9	350.4	28.8	12.9	6.4	479.3
473.8	10236.8	350.4	29.2	13.3	6.6	480.4
474.8	10374.9	350.4	29.6	13.6	6.8	481.6
475.8	10511.2	350.4	30.0	14.0	7.0	482.8
476.8	10645.7	350.4	30.4	14.3	7.2	484.0
477.8	10778.6	350.4	30.8	14.7	7.3	485.2
478.8	10909.8	350.4	31.1	15.1	7.5	486.3
479.8	11039.4	350.4	31.5	15.4	7.7	487.5
480.8	11167.6	350.4	31.9	15.8	7.9	488.7
481.8	11294.3	350.4	32.2	16.1	8.1	489.9
482.8	11419.6	350.4	32.6	16.5	8.2	491.1
483.8	11543.6	350.4	32.9	16.9	8.4	492.2
484.8	11666.2	350.4	33.3	17.2	8.6	493.4
485.8	11787.5	350.4	33.6	17.6	8.8	494.6
486.8	11907.6	350.4	34.0	17.9	9.0	495.8
487.8	12026.6	350.4	34.3	18.3	9.1	497.0
488.8	12144.3	350.4	34.7	18.7	9.3	498.1
489.8	12260.9	350.4	35.0	19.0	9.5	499.3
490.8	12376.4	350.4	35.3	19.4	9.7	500.5
491.8	12490.9	350.4	35.7	19.7	9.9	501.7
492.8	12604.3	350.4	36.0	20.1	10.0	502.9

Notes:
EGL= energy grade line
Q= discharge
A= flow area
V=average velocity

3.3 Spillway Design

The spillway design procedure for FRFA and FRO dam alternatives are similar. Therefore, only FRFA spillway calculation is presented in this section.

Base of the dam elevation= 400 feet msl

Spillway crest elevation = 687 feet msl

Dam crest elevation=710 feet msl

PMF water surface elevation =709 feet msl

Q Design =75,000 cfs

Q PMF = 69,800 cfs

H_e= PMF water surface elevation - spillway crest elevation = 22 feet, effective head

P = spillway crest elevation – dam base elevation= 287 feet, dam height

H_d= 30 feet, design head (selected); $\frac{P}{H_d} = 9.6$ → C= 3.84, spillway discharge coefficient per plate 3-4 EM 1110-2-1603 (USACE 1992)

$Q = CL_eH_e^{1.5}$; $L_e = \frac{Q}{CH_e^{1.5}} = 189\ ft$ → effective spillway crest length

$L = L_e + 2(nk_p + k_a)H_e$; n=0, K_p= 0.08 per plate 3-11 EM 1110-2-1603 (USACE 1992)→ L= 193 feet; used 200 feet for net spillway length

After determining the spillway design head and crest length, the ogee face shape can be computed following the procedure in USACE HDC 111-2/1, Design of Ogee Crest Shape. Table 3-3 provides the spillway shape upstream quadrant calculation for FRO and FRFA dam alternatives. Table 3-4 provides the spillway shape downstream quadrant calculation for FRO and FRFA dam alternatives.

Table 3-3
Spillway Shape Upstream Quadrant Profiles for FRO and FRFA Dam Alternatives

FRO	
R_{CL}(ft)	15.0
X_{CL} (ft)	0.0
Y_{CL}(ft)	613.0
$R_{2,3}$(ft)	6.0
$X_{2,3}$ (ft)	-3.2
$Y_{2,3}$ (ft)	621.4
R_1(ft)	1.2
$X_{1,CEN}$ (ft)	-7.3
$Y_{1,CEN}$ (ft)	623.9
X_1(ft)	8.5
Y_1 (ft)	623.9
X_2(ft)	8.3
Y_2(ft)	624.5
X_3(ft)	5.3
Y_3 (ft)	627.1

FRFA	
R_{CL}(ft)	15.0
X_{CL} (ft)	0.0
Y_{CL}(ft)	672.0
$R_{2,3}$(ft)	6.0
$X_{2,3}$ (ft)	-3.2
$Y_{2,3}$ (ft)	680.4
R_1(ft)	1.2
$X_{1,CEN}$ (ft)	-8.5
$Y_{1,CEN}$ (ft)	682.9
X_1(ft)	8.5
Y_1 (ft)	682.9
X_2(ft)	8.3
Y_2(ft)	683.5
X_3(ft)	5.3
Y_3 (ft)	686.1

Note:
Refer to Figure 2-17 for parameter definition

Table 3-4
Spillway Shape Downstream Quadrant for FRFA Dam Alternative

FRO					FRFA				
X (ft)	Y (ft)	Elevation (ft)	Slope	Location	X (ft)	Y (ft)	Elevation (ft)	Slope	Location
0.0	0.0	628.0	--	Downstream Qudrant	0.0	0.0	687.0	--	Downstream Qudrant
2.0	0.1	627.9	19.99		2.0	0.1	686.9	19.99	
2.9	0.2	627.8	9.09		2.9	0.2	686.8	9.09	
3.6	0.3	627.7	7.13		3.6	0.3	686.7	7.13	
4.2	0.4	627.6	6.09		4.2	0.4	686.6	6.09	
4.8	0.5	627.5	5.42		4.8	0.5	686.5	5.42	
5.3	0.6	627.4	4.94		5.3	0.6	686.4	4.94	
5.7	0.7	627.3	4.58		5.7	0.7	686.3	4.58	
6.2	0.8	627.2	4.28		6.2	0.8	686.2	4.28	
6.6	0.9	627.1	4.04		6.6	0.9	686.1	4.04	
6.9	1.0	627.0	3.84		6.9	1.0	686.0	3.84	
7.3	1.1	626.9	3.67		7.3	1.1	685.9	3.67	
7.7	1.2	626.8	3.52		7.7	1.2	685.8	3.52	
8.0	1.3	626.7	3.39		8.0	1.3	685.7	3.39	
8.3	1.4	626.6	3.27		8.3	1.4	685.6	3.27	
8.6	1.5	626.5	3.16		8.6	1.5	685.5	3.16	
8.9	1.6	626.4	3.07		8.9	1.6	685.4	3.07	
9.2	1.7	626.3	2.98		9.2	1.7	685.3	2.98	
9.5	1.8	626.2	2.90		9.5	1.8	685.2	2.90	
9.8	1.9	626.1	2.83		9.8	1.9	685.1	2.83	
10.1	2.0	626.0	2.76		10.1	2.0	685.0	2.76	
10.4	2.1	625.9	2.70		10.4	2.1	684.9	2.70	
12.8	3.1	624.9	2.43		12.8	3.1	683.9	2.43	
14.9	4.1	623.9	2.09		14.9	4.1	682.9	2.09	
16.7	5.1	622.9	1.86		16.7	5.1	681.9	1.86	
18.4	6.1	621.9	1.70		18.4	6.1	680.9	1.70	
20.0	7.1	620.9	1.58		20.0	7.1	679.9	1.58	
21.5	8.1	619.9	1.48		21.5	8.1	678.9	1.48	
22.9	9.1	618.9	1.40		22.9	9.1	677.9	1.40	
24.2	10.1	617.9	1.33		24.2	10.1	676.9	1.33	
25.5	11.1	616.9	1.27		25.5	11.1	675.9	1.27	
26.7	12.1	615.9	1.22		26.7	12.1	674.9	1.22	
27.9	13.1	614.9	1.17		27.9	13.1	673.9	1.17	
29.0	14.1	613.9	1.13		29.0	14.1	672.9	1.13	
30.1	15.1	612.9	1.09		30.1	15.1	671.9	1.09	
31.2	16.1	611.9	1.06		31.2	16.1	670.9	1.06	
32.2	17.1	610.9	1.03		32.2	17.1	669.9	1.03	
33.2	18.1	609.9	1.00		33.2	18.1	668.9	1.00	
34.2	19.1	608.9	0.98		34.2	19.1	667.9	0.98	
35.1	20.1	607.9	0.96		35.1	20.1	666.9	0.96	
36.1	21.1	606.9	0.93		36.1	21.1	665.9	0.93	
37.0	22.1	605.9	0.91		37.0	22.1	664.9	0.91	
37.9	23.1	604.9	0.90		37.9	23.1	663.9	0.90	
38.8	24.1	603.9	0.88		38.8	24.1	662.9	0.88	
39.8	25.3	602.7	0.85	Point of Tangancy	39.8	25.3	661.7	0.85	Point of Tangancy
58.5	47.3	580.7	0.85	Spillway Chute	68.5	59.1	627.9	0.85	Spillway Chute
77.2	69.3	558.7	0.85		97.3	92.9	594.1	0.85	
95.9	91.4	536.6	0.85		126.0	126.8	560.2	0.85	
114.7	113.4	514.6	0.85		154.8	160.6	526.4	0.85	
133.4	135.4	492.6	0.85		183.5	194.4	492.6	0.85	

The spillway rating curve is calculated following the procedure provided in USACE HDC Sheet 111-3/3 (1987). Table 3-5 presents the spillway rating curve calculations for FRO and FRFA dam alternatives.

Q =0-75,000 cfs

$$Q = CLH_d^{1.5} = 123{,}091\ cfs$$

$$H_e = H_d\left(\frac{Q}{Q_d}\right)^{\frac{1}{1.6}}$$

Water surface elevation= H_e + spillway crest elevation

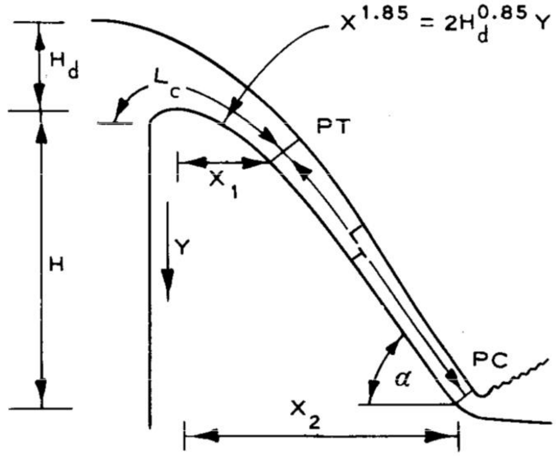
Table 3-5
Spillway Rating Curve for FRFA Dam Alternative

FRO			FRFA		
Q (cfs)	**H_e (ft)**	**WSE (ft)**	**Q (cfs)**	**H_e (ft)**	**WSE (ft)**
75000	22.0	650.0	75000	22.0	709.0
70000	21.1	649.1	70000	21.1	708.1
65000	20.1	648.1	65000	20.1	707.1
60000	19.1	647.1	60000	19.1	706.1
55000	18.1	646.1	55000	18.1	705.1
50000	17.1	645.1	50000	17.1	704.1
45000	16.0	644.0	45000	16.0	703.0
40000	14.9	642.9	40000	14.9	701.9
35000	13.7	641.7	35000	13.7	700.7
30000	12.4	640.4	30000	12.4	699.4
25000	11.1	639.1	25000	11.1	698.1
20000	9.6	637.6	20000	9.6	696.6
15000	8.0	636.0	15000	8.0	695.0
10000	6.2	634.2	10000	6.2	693.2
5000	4.1	632.1	5000	4.1	691.1
1000	1.5	629.5	1000	1.5	688.5
0	0.0	628.0	0	0.0	687.0

Notes:
Q= discharge
He= effective head
WSE= water surface elevation

Turbulent boundary layer development method was utilized to calculate the flow velocity and depth at the toe of the spillway. Figure 3-32 shows the parameters used in this calculation.

Figure 3-32
Parameters Used in Calculating the Flow Depth and Velocity at the Toe of Spillway



Source: USACE Hydraulic Design Criteria 111-18 to 111-18/5 1987

H_d= 30 feet; H=P=287 feet; k=0.002 feet (surface roughness); face slope α= 0.85H:1V

Coordinates of point of tangency:

$$X_1 = 1.096H_d(\frac{1}{\alpha})^{1.176} = 39.8\ feet;\ Y_1 = 0.592H_d(\frac{1}{\alpha})^{2.176} = 25.3\ feet$$

$\frac{X_1}{H_d} = 1.33; \frac{L_c}{H_d} = 1.93$ per plate HDC 111-18/1; L_c= 57.9 feet

Y_2= 194.4 foot spillway toe; Y_2-Y_1= 169.1 feet

$L_T = \frac{Y_2 - Y_1}{Sin\ \alpha} = 221.9\ ft$; L= L_c+L_T= 279.8 feet

$$\frac{\delta}{L} = 0.08(\frac{L}{k})^{-0.233} = 0.005;\ \delta = 1.42\ ft$$

h_T= H_d+H= 216.4 feet

$h_T = d_P \cos\alpha + \frac{u^2}{2g}$; trial and error method --> d_p=3.2 feet, u= 117.4 feet/s

$\delta_1 = 0.18\delta = 0.25\ feet$, displacement thickness

$\delta_3 = 0.22\delta = 0.31\ feet$, energy thickness

$d = d_p + \delta_1 = 3.45\ feet$, actual flow depth at the toe

$$H_L = \frac{\delta_3\, u^3}{2gq} = 20.9\ feet, \text{ energy loss}$$

The water surface profile over the spillway to the point of tangency was evaluated using upper nappe profiles for spillway without piers presented in USACE HDC plate 111-11. The water surface profile over the spillway crest and nappe are presented in Table 3-6 and Table 3- for FRO and FRFA dam alternatives. The continuation of water surface profile over the spillway chute was computed using the turbulent boundary development method and is presented in Table 3-8 for FRO and FRFA dam alternatives.

Table 3-6
Water Surface Profile over the Spillway to the Crest for FRO and FRFA Dam Alternatives

FRO						
Location	**X (ft)**	**SE (ft)**	**X/H_d**	**Y/H_d**	**Y (ft)**	**WSE (ft)**
Upstream	-100.00	--	--	--	--	650.00
	-30.00	--	-1.00	-0.69	-20.77	648.77
	-15.00	--	-0.50	-0.66	-19.90	647.90
Circle 1	-8.45	623.92	-0.28	-0.63	-18.80	646.80
	-8.45	623.92	-0.28	-0.63	-18.80	646.80
	-8.44	624.13	-0.28	-0.63	-18.80	646.80
	-8.38	624.33	-0.28	-0.63	-18.79	646.79
	-8.29	624.52	-0.28	-0.63	-18.77	646.77
Circle 2,3	-7.75	625.29	-0.26	-0.62	-18.65	646.65
	-7.01	626.03	-0.23	-0.62	-18.48	646.48
	-6.15	626.63	-0.21	-0.61	-18.28	646.28
Circle CL	-5.33	627.02	-0.18	-0.60	-18.08	646.08
	-2.60	627.77	-0.09	-0.58	-17.34	645.34
	0.00	628.00	0.00	-0.55	-16.53	644.53

FRFA						
Location	**X (ft)**	**SE (ft)**	**X/H_d**	**Y/H_d**	**Y (ft)**	**WSE (ft)**
Upstream	-100.00	--	--	--	--	709.00
	-30.00	--	-1.00	-0.69	-20.77	707.77
	-15.00	--	-0.50	-0.66	-19.90	706.90
Circle 1	-9.65	682.92	-0.32	-0.63	-19.04	706.04
	-9.64	683.13	-0.32	-0.63	-19.04	706.04
	-9.58	683.33	-0.32	-0.63	-19.03	706.03
	-9.49	683.52	-0.32	-0.63	-19.01	706.01
	-9.48	683.54	-0.32	-0.63	-19.01	706.01
Circle 2,3	-7.75	684.29	-0.26	-0.62	-18.65	705.65
	-7.01	685.03	-0.23	-0.62	-18.48	705.48
	-6.15	685.63	-0.21	-0.61	-18.28	705.28
Circle CL	-5.30	686.03	-0.18	-0.60	-18.07	705.07
	-2.60	686.77	-0.09	-0.58	-17.34	704.34
	0.00	687.00	0.00	-0.55	-16.53	703.53

Notes:
SE= spillway elevation
WSE= water surface elevation

Table 3-7
Water Surface Profile Over the Spillway Nappe for FRO and FRFA Dam Alternatives

FRO						
Location	X (ft)	SE (ft)	X/H_d	Y/H_d	Y (ft)	WSE (ft)
Nappe	2.00	627.90	0.07	-0.53	-15.84	643.84
	2.91	627.80	0.10	-0.52	-15.50	643.50
	3.62	627.70	0.12	-0.51	-15.23	643.23
	4.23	627.60	0.14	-0.50	-14.98	642.98
	4.77	627.50	0.16	-0.49	-14.76	642.76
	5.27	627.40	0.18	-0.49	-14.56	642.56
	5.72	627.30	0.19	-0.48	-14.37	642.37
	6.15	627.20	0.21	-0.47	-14.18	642.18
	6.56	627.10	0.22	-0.47	-14.00	642.00
	6.94	627.00	0.23	-0.46	-13.83	641.83
	7.31	626.90	0.24	-0.46	-13.67	641.67
	7.66	626.80	0.26	-0.45	-13.51	641.51
	8.00	626.70	0.27	-0.44	-13.35	641.35
	8.33	626.60	0.28	-0.44	-13.20	641.20
	8.64	626.50	0.29	-0.43	-13.04	641.04
	8.95	626.40	0.30	-0.43	-12.90	640.90
	9.25	626.30	0.31	-0.43	-12.75	640.75
	9.54	626.20	0.32	-0.42	-12.61	640.61
	9.82	626.10	0.33	-0.42	-12.47	640.47
	10.10	626.00	0.34	-0.41	-12.33	640.33
	10.36	625.90	0.35	-0.41	-12.19	640.19
	12.79	624.90	0.43	-0.36	-10.89	638.89
	14.88	623.90	0.50	-0.32	-9.68	637.68
	16.74	622.90	0.56	-0.28	-8.53	636.53
	18.45	621.90	0.61	-0.25	-7.41	635.41
	20.02	620.90	0.67	-0.21	-6.32	634.32
	21.50	619.90	0.72	-0.17	-5.24	633.24
	22.90	618.90	0.76	-0.14	-4.19	632.19
	24.23	617.90	0.81	-0.10	-3.14	631.14
	25.49	616.90	0.85	-0.07	-2.10	630.10
	26.71	615.90	0.89	-0.04	-1.07	629.07
	27.88	614.90	0.93	0.00	-0.04	628.04
	29.01	613.90	0.97	0.03	0.98	627.02
	30.11	612.90	1.00	0.07	2.01	625.99
	31.17	611.90	1.04	0.10	3.02	624.98
	32.20	610.90	1.07	0.13	4.04	623.96
	33.21	609.90	1.11	0.17	5.06	622.94
	34.19	608.90	1.14	0.20	6.08	621.92
	35.14	607.90	1.17	0.24	7.09	620.91
	36.08	606.90	1.20	0.27	8.11	619.89
	36.99	605.90	1.23	0.30	9.13	618.87
	37.89	604.90	1.26	0.34	10.15	617.85
	38.76	603.90	1.29	0.37	11.17	616.83
	39.80	602.69	1.33	0.41	12.41	615.59

FRFA						
Location	**X (ft)**	**SE (ft)**	**X/H_d**	**Y/H_d**	**Y (ft)**	**WSE (ft)**
Nappe	2.00	686.90	0.07	-0.53	-15.84	702.84
	2.91	686.80	0.10	-0.52	-15.50	702.50
	3.62	686.70	0.12	-0.51	-15.23	702.23
	4.23	686.60	0.14	-0.50	-14.98	701.98
	4.77	686.50	0.16	-0.49	-14.76	701.76
	5.27	686.40	0.18	-0.49	-14.56	701.56
	5.72	686.30	0.19	-0.48	-14.37	701.37
	6.15	686.20	0.21	-0.47	-14.18	701.18
	6.56	686.10	0.22	-0.47	-14.00	701.00
	6.94	686.00	0.23	-0.46	-13.83	700.83
	7.31	685.90	0.24	-0.46	-13.67	700.67
	7.66	685.80	0.26	-0.45	-13.51	700.51
	8.00	685.70	0.27	-0.44	-13.35	700.35
	8.33	685.60	0.28	-0.44	-13.20	700.20
	8.64	685.50	0.29	-0.43	-13.04	700.04
	8.95	685.40	0.30	-0.43	-12.90	699.90
	9.25	685.30	0.31	-0.43	-12.75	699.75
	9.54	685.20	0.32	-0.42	-12.61	699.61
	9.82	685.10	0.33	-0.42	-12.47	699.47
	10.10	685.00	0.34	-0.41	-12.33	699.33
	10.36	684.90	0.35	-0.41	-12.19	699.19
	12.79	683.90	0.43	-0.36	-10.89	697.89
	14.88	682.90	0.50	-0.32	-9.68	696.68
	16.74	681.90	0.56	-0.28	-8.53	695.53
	18.45	680.90	0.61	-0.25	-7.41	694.41
	20.02	679.90	0.67	-0.21	-6.32	693.32
	21.50	678.90	0.72	-0.17	-5.24	692.24
	22.90	677.90	0.76	-0.14	-4.19	691.19
	24.23	676.90	0.81	-0.10	-3.14	690.14
	25.49	675.90	0.85	-0.07	-2.10	689.10
	26.71	674.90	0.89	-0.04	-1.07	688.07
	27.88	673.90	0.93	0.00	-0.04	687.04
	29.01	672.90	0.97	0.03	0.98	686.02
	30.11	671.90	1.00	0.07	2.01	684.99
	31.17	670.90	1.04	0.10	3.02	683.98
	32.20	669.90	1.07	0.13	4.04	682.96
	33.21	668.90	1.11	0.17	5.06	681.94
	34.19	667.90	1.14	0.20	6.08	680.92
	35.14	666.90	1.17	0.24	7.09	679.91
	36.08	665.90	1.20	0.27	8.11	678.89
	36.99	664.90	1.23	0.30	9.13	677.87
	37.89	663.90	1.26	0.34	10.15	676.85
	38.76	662.90	1.29	0.37	11.17	675.83
	39.80	661.69	1.33	0.41	12.41	674.59

Notes:
SE= spillway elevation
W_SE = water surface elevation

Table 3-8
Water Surface Profile Over the Spillway Chute for FRO and FRFA Dam Alternatives

FRO											
Location	**X (ft)**	**SE (ft)**	**Slope**	**L (ft)**	**δ/L**	**δ (ft)**	**u (ft/s)**	**d_p (ft)**	**q (cfs/ft)**	**δ_1 (ft)**	**d(ft)**	**WSE (ft)**
Spillway Chute	58.52	580.67	0.85	86.79	0.0066	0.58	64.96	5.77	375.0	0.10	5.88	589.75
	77.23	558.66	0.85	115.69	0.0062	0.72	75.30	4.98	375.0	0.13	5.11	566.55
	95.95	536.64	0.85	144.58	0.0059	0.85	84.31	4.45	375.0	0.15	4.60	543.75
	114.66	514.63	0.85	173.48	0.0057	0.98	92.42	4.06	375.0	0.18	4.23	521.16
	133.37	492.61	0.85	202.37	0.0055	1.10	99.86	3.76	375.0	0.20	3.95	498.72

FRFA												
Location	**X (ft)**	**SE (ft)**	**Slope**	**L (ft)**	**δ/L**	**δ (ft)**	**u (ft/s)**	**d_p (ft)**	**q (cfs/ft)**	**δ_1 (ft)**	**d(ft)**	**WSE (ft)**
Spillway Chute	68.55	627.87	0.85	102.28	0.0064	0.65	70.71	5.30	375.0	0.12	5.42	636.24
	97.29	594.06	0.85	146.66	0.0059	0.86	84.93	4.42	375.0	0.16	4.57	601.12
	126.04	560.24	0.85	191.04	0.0055	1.06	97.01	3.87	375.0	0.19	4.06	566.50
	154.78	526.43	0.85	235.43	0.0053	1.24	107.72	3.48	375.0	0.22	3.70	532.15
	183.52	492.61	0.85	279.81	0.0051	1.42	117.44	3.19	375.0	0.25	3.45	497.93

Notes:
SE = spillway elevation
L = length along the spillway
▯ = boundary layer thickness
u =velocity
dp = potential depth
▯1 =displacement thickness
q = unit discharge
d = water depth
W_SE = water surface elevation.

3.4 Flip Bucket

Flip bucket radius and height were computed for various bucket invert elevations and 475 feet msl was selected for the final design. The flip bucket design calculation for FRFA dam alternative is presented here:

V_1=117.4 feet; d_1= 3.45 feet; P_T=2000 lb/feet, allowable bearing pressure on the bucket; θ=45 degree

$$r_{min} = \frac{\rho V_1^2 d_1}{P_T - \gamma d_1} = 51.6\ feet;$$ Bucket radius of 50 feet was selected.

$$h = r - r\ cos\theta = 14.6\ feet$$

Bucket lip elevation = bucket invert elevation + h = 489.6 feet msl

The jet trajectory leaving the flip bucket was evaluated using the equation for trajectory of a projectile. Table 3-9 presents the water jet trajectory for FRO and FRFA dam alternatives.

Table 3-9
Water Jet Trajectory Leaving the Flip Buck for FRO and FRFA Dam Alternatives

FRO		FRFA	
X (ft)	Elevation (ft)	X (ft)	Elevation (ft)
206.84	494.99	257.06	494.37
235.84	521.28	300.06	533.05
264.84	542.13	343.06	563.10
293.84	557.55	386.06	584.52
322.84	567.54	429.06	597.30
351.84	572.09	472.06	601.44
380.84	571.22	515.06	596.96
409.84	564.91	558.06	583.83
438.84	553.17	601.06	562.08
467.84	535.99	644.06	531.69
496.84	513.39	687.06	492.66
525.84	485.35	725.06	450.99
554.84	451.89		

3.5 Flood Regulation Outlets Rating Curves

The rating curves for flood regulating outlet works were calculated using the radial gate discharge equation when inlet control exists at the gate location. Depending on the size of gate opening (i.e., large gate openings) the control section may shift to the upstream at the conduit entrance location. Therefore, discharge through the gate is independent from the gate opening and can be calculated using the orifice equation. A sample calculation for the FRFA dam alternative flood regulation outlet works is presented here.

Outlet size= 10 foot wide by 16 foot high equipped with radial gate

Conduit invert elevation= 420 feet msl

Gate 75% open → GO= 12 feet

Angle between the radial gate lip and horizontal at 12 foot open setting= 78.3 degree

Gate opening/conduit height= 0.75

C= 0.72 per plate C-24 EM 1110-2-1602 (USACE 1980)

EGL= 500 feet, energy grade line at the conduit inlet

H= EGL-(conduit invert elevation+ C*GO)= 500-(420+0.72*12)=71.36 feet

g= 32.18 feet/s^2, gravitational acceleration

$$Q = C\ GO\ B\ \sqrt{2gH} = 5855.3\ cfs$$

V=Q/A=5855.3/(10*16)= 36.6 feet/s

K_e= 0.4, entrance loss coefficient considering both conduit entrance and trashrack effects

Pool elevation= EGL+ K_e $V^2/2g$= 508.4 feet msl

For the FRFA outlet works at gate opening greater than 14 feet, the control section shifts to the conduit entrance and discharge will be calculated using orifice equation.

Outlet size= 10 foot wide by 16 foot high equipped with radial gate

Conduit invert elevation= 420 feet

Gate 94% open → GO= 15 feet > GO=14 feet

C= 0.67 for orifice

EGL= 500 feet, energy grade line at the conduit inlet

H= EGL-(conduit invert elevation+ 0.5* conduit height)= 500-(420+0.5*16)=72 feet

g= 32.18 feet/s^2, gravitational acceleration

$$Q = CA\sqrt{2gH} = 7297.4\ cfs$$

V=Q/A=7297.4/ (10*16)=45.6 feet/s

K_e= 0.4, entrance loss coefficient considering both conduit entrance and trashrack effects

Pool elevation= EGL+ K_e $V^2/2g$= 512.9 feet msl

3.6 Stilling Basin

Stilling basin is designed for the maximum flow to ensure a satisfactory performance under the range of outlet works operational flow. The stilling basin floor elevation of 377 feet msl was selected for the final design calculation for the FRFA dam alternative, while 381.2 feet msl was selected for the FRO dam alternative.

Stilling basin floor elevation =377 feet msl

Q= 15,000 cfs

Pool elevation= 687 feet msl

Conduit invert elevation = 420 feet

Both 10 foot wide by 16 foot high outlets open at 8.75 foot gate opening (GO) setting

C=0.69; contraction coefficient of the gate at 8.75 foot gate opening

$d = C\ GO = 6.0\ feet\ ;$ water depth downstream of the gate

H= pool elevation - conduit invert elevation - d=267 feet

$V = \sqrt{2gh} = 129.6\ feet\ per\ second;$ jet velocity downstream of the gate

d_1= 3.8 feet, V_1=140.4 feet per second; using Bernoulli equation between sections downstream of the gate and stilling basin entrance assuming negligible energy loss and jet expansion.

$$Fr = \frac{V_1}{\sqrt{gd_1}} = 12.6$$

$d_2 = \frac{d_1}{2}\left(\sqrt{1+8Fr^2} - 1\right) = 66.6\ feet,$ conjugate depth of the hydraulic jump at initial entry used to calculate the stilling basin length

Jet expansion following entry to full 70 foot width of stilling basin resulted in conjugate depth of 56 feet to calculate the stilling basin floor elevation

Stilling basin length = 3.5d_2= 233.1 feet, stilling basin length of 230 feet was selected

The stilling basin end sill rating curve was calculated using the discharge equation over a broad crest weir and is presented in Table 3-10.

Table 3-10
Stilling Basin End Sill Rating Curve

Discharge (cfs)	H (ft)	WSE (ft)
10	0.1	417.1
100	0.6	417.6
250	1.1	418.1
500	1.7	418.7
1000	2.7	419.7
1500	3.6	420.6
2500	5.0	422.0
5000	7.9	424.9
7500	10.4	427.4
10000	12.6	429.6
15000	16.5	433.5

Notes:
H = water head
W_SE = water surface elevation

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Appendix C
Foundation Design

Chehalis Basin Strategy

Appendix C

Foundation Design

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

Prepared for: State of Washington Recreation and Conservation Office and Chehalis Basin Work Group

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ACRONYMS AND ABBREVIATIONS

3-D	three-dimensional
FRO	flood retention only
FRFA	flood retention flow augmentation
ft/s	feet per second
Lu	Lugeon
Qa	Quaternary alluvium
Qao	Quaternary alluvium older
Qc	Quaternary colluvium
Qls	Quaternary landslide deposit
Qos	Quaternary overburden soil
RCC	roller-compacted concrete
RMR	rock mass rating
RQD	rock quality designation
Tig	intrusive igneous volcanics
Tcb	Crescent Formation basalt
Tcs	Crescent Formation siltstone
Tml	McIntosh Formation lower

1 GENERAL

In general, there is good quality foundation bedrock available at the Chehalis dam site. The Chehalis roller-compacted concrete (RCC) dam will be founded on competent bedrock, and the right abutment saddle embankment dam will likely be founded on lower quality but adequate bedrock. The foundation areas will be excavated to remove soil materials and unsuitable rock. The foundation surface will be shaped and treated as needed. The rock foundation will be grouted to reduce seepage and foundation drains will be provided to reduce uplift forces.

Proper design of the Chehalis flood retention only (FRO) and flood retention/flow augmentation (FRFA) RCC dam foundation is critical for meeting safety standards that are part of standard practice, and for meeting regulatory agency guidance and criteria. Historical case histories of concrete dam maintenance, safety incidents, and failures have typically involved foundation deficiencies. Over the past several decades the dam engineering industry has learned from these case histories. While concrete dam foundation engineering requires a high level of experience and expertise, state of the practice site characterization techniques, design techniques, modeling capabilities, construction technologies, and a respect for rigorous construction monitoring will allow the construction of a dam that meets rigorous present-day safety standards at the Chehalis RCC dam site.

Foundation design for the Chehalis RCC dam must include several primary elements:

- Foundation excavation objective: establishing an excavation surface that provides the appropriate geologic and engineering characteristics to properly found the dam.
- Foundation surface treatment: surface treatment to improve the rock foundation surface to enable better contact with the concrete, eliminate minor surficial weak zones and shape the foundation to reduce the possibility for stress concentrations and the potential for uncontrolled cracking in the concrete dam.
- Grouting: drilling and pumping concrete based grout into rock discontinuities (joints, fractures, shears, etc.) to reduce seepage through the foundation.
- Foundation drainage: drilling permanent holes through the bottom of the dam and into the foundation to allow uplift pressures to drain, enhancing the stability of the structure.
- Stabilization of potential areas of kinematic instability: foundation excavation cut slopes need to be checked to avoid adverse rock stability situations that could result in a release of a foundation block or cause excavation slope instability.

Each of these design elements is discussed in this document. Supporting geologic data that is necessary to develop the foundation design is provided.

From a design and cost estimating perspective, it is critical to evaluate the volume of overburden soil and inadequate quality rock to be removed. Proper characterization of the site and subsurface conditions will be critical to establishing designs with appropriate risk management strategies that result in qualified contractors submitting competitive bids, and establishing design and construction contingencies that support successful completion of the project construction.

2 EXPLORATION AND TESTING

2.1 Exploration Program

The evaluation documented by this report builds on two previously completed phases of site exploration. Information obtained from the Phases 1 and 2 site investigation programs, including the in-situ testing and laboratory testing results which are presented in the Site Characterization Report, was considered as part of the foundation design. One of the first considerations in the engineering process of foundation design was to identify the top of rock at each exploration location and use that information to estimate the top of rock surface across the site. By creating a top of rock "surface" the volume of material between the ground surface and top of rock surface can be estimated. This significant volume of "common excavation" will need to be handled (excavated, transported, stockpiled, etc.) by the contractor building the dam, and will be an important consideration during the cost estimating process.

Figure 2-1 provides a site exploration plan that shows the information used to develop the foundation design. Table 2-1, from the Phase 2 Site Characterization Report (HDR 2016) includes the depth to top of rock at each borehole location.

Figure 2-1

Chehalis Dam Project Phase 2 Site Characterization Site Investigation Plan Layout

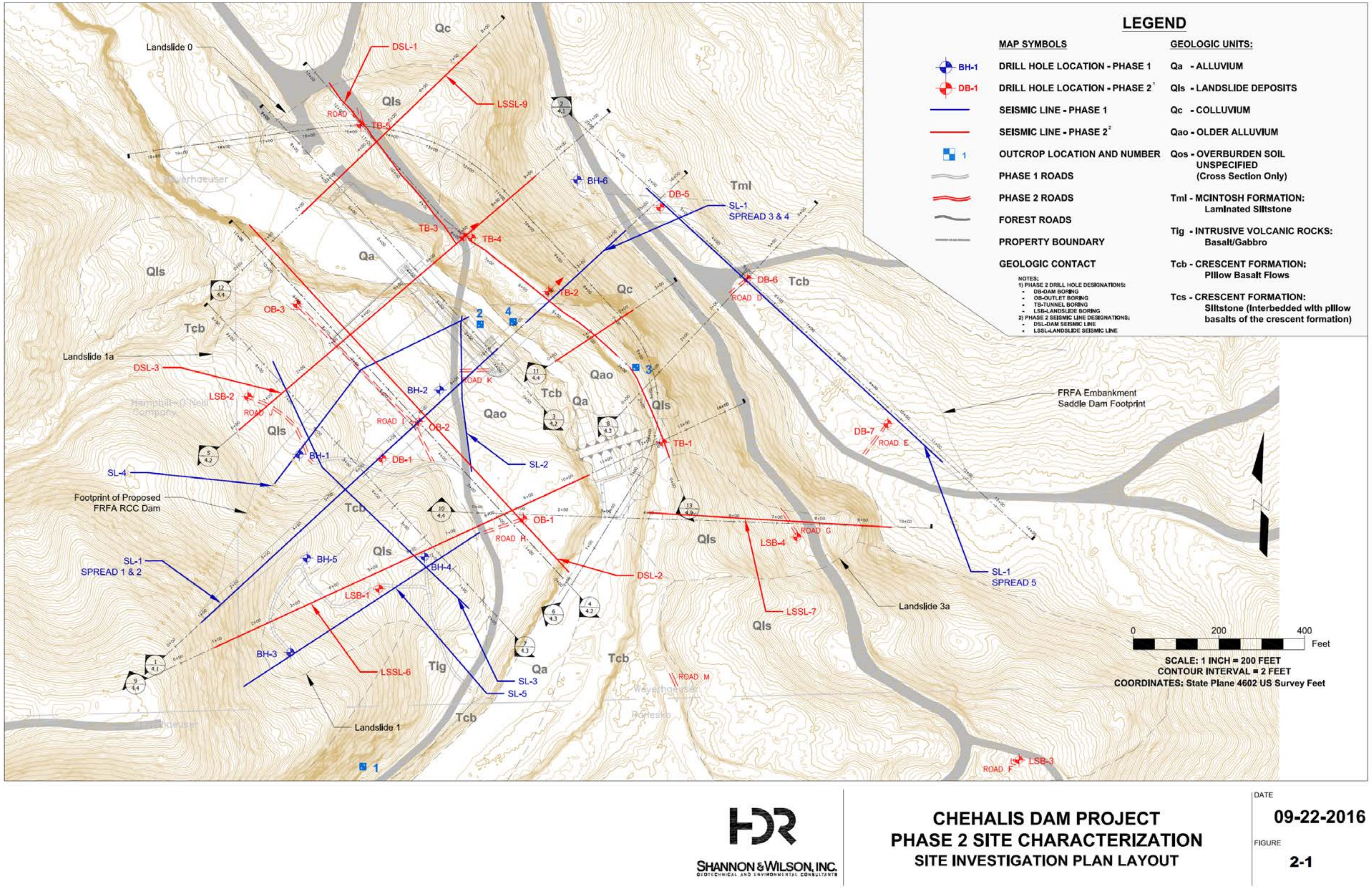


Table 2-1

Summary of Completed Borehole Information

PHASE	BOREHOLE NUMBER	LOCATION	EASTING[1]	NORTHING[1]	ELEVATION[1] (FEET)	DEPTH (FEET)	SOIL/ WEATHERED ROCK DEPTH (FEET)	ANGLE OF HOLE FROM HORIZONTAL (DEGREES)	AZIMUTH (DEGREES)	WATER PRESSURE TESTS		GROUNDWATER[2]	
										HIGHEST LUGEON VALUE	DEPTH (FEET)	DEPTH (FEET)	ELEVATION (FEET)
1	BH-1	Left abutment downstream landslide	936024.4	454038.5	507	140.4	35	90	NA	17.3	118	53	455
	BH-2	Dam alignment centerline - near maximum section	936325.6	454246.7	456	241	30	90	NA	75.4	178	23	432
	BH-3	Left abutment upstream landslide	935961.8	453694.0	577	150	35	90	NA	12.4	47	103	499
	BH-4	Left abutment upstream landslide	936303.2	453852.2	516	120	62.5	90	NA	22.7	87	60	452
	BH-5	Dam alignment centerline - left abutment	936012.7	453826.0	560	250	47.5	90	NA	67.5	57	127	432
	BH-6	Dam alignment centerline - right abutment	936660.0	454684.9	664	350	32.5	90	NA	165.8	137	243	422
2	DB-1	Alignment Sta. 6+50	936187.86	454055.83	470.7	240	30	90	NA	26.9	95	43	428
	DB-5	Alignment Sta. 15+36	936836.907	454641.444	686.8	200	40	90	NA	40.1	76	194	492.5
	DB-6	Saddle Dam Alignment Sta. 4+15	937040.256	454475.286	673.8	200	31.5	90	NA	22.3	186	192	482
	DB-7	Saddle Dam Alignment Sta. 9+00	937367.468	454137.034	691.8	200	37	90	NA	9.7	56	196	496
	OB-1	Upstream of Dam along FRFA dam flood control outlet	936516.79	453914.79	464.68	100	29.5	90	NA	12.3	65	36	429
	OB-2	Along Dam Alignment and FRFA dam flood control outlet	936273.08	454141.41	467.84	240	38	90	NA	2.7	55	40	428
	OB-3	Downstream of Dam along FRFA dam flood control outlet	935988.82	454415.80	466.20	100	34.3	90	NA	4.3	65	36	430
	TB-1	Upstream of Dam along FC dam flood control outlet	936842.9557	454093.63	465.44	100	25	90	NA	2.8	35	38	427.5
	TB-2	Alignment Sta. 11+68	936576.19	454445.04	467.85	200	0	22	N47E	50.6	176.6	33	435
	TB-3	Downstream of Dam Alignment along road	936377.8776	454573.328	455.04	200	0	18	N47E	213.9	42.5	25	430
	TB-4	Downstream of Dam Alignment along road	936397.08	454570.29	458.53	100	10	90	NA	65.7	45	40	419
	TB-5	Downstream of dam alignment in the landslide	936137.8354	454834.305	488.206	100	60	90	NA	5.4	75	70	418
	LSB-1	Landslide 1	936180.9439	453752.078	524.69	105	40	90	NA	NA	NA	16	509
	LSB-2	Landslide 1a	935874.80	454200.11	520.71	110	40	90	NA	NA	NA	21	500
	LSB-3	Landslide 3	937673.92	453352.60	556.59	100	57	90	NA	NA	NA	36	521
	LSB-4	Landslide 3a	937156.48	453873.02	586.29	50	30	90	NA	NA	NA	29	557.5
	QB-1	15NE1	939529.47	448782.15	804.20	150	35	90	NA	NA	NA	NA	NA
	QB-2	23SE1(P)	914366.39	439616.43	2564.25	150	0	90	NA	NA	NA	NA	NA

Notes:

1. Coordinate System: State Plane 4602 US Survey feet; Datum: WGS84. Coordinates and elevation obtained from survey data
2. Groundwater measurements were last taken for BH-2 and BH-6 taken in November 2015 and BH-3, BH-4 and BH-5 in March 2016 due to data logger failure; all other measurements taken in May 2016

2.2 Laboratory Testing

Laboratory testing of materials from the dam foundation included representative overburden soil and bedrock samples from the test boreholes. Laboratory soil testing results are documented in the Site Characterization Report. Testing of bedrock samples provided bedrock parameters including bulk density, lithology classification, and unconfined compressive strength, Young's modulus, and Poisson's ratio. Slake durability test results were performed to evaluate the potential for potential foundation materials to degrade when exposed in excavations or in drain holes. Appropriate treatment precautions may be required when certain foundation materials are encountered in excavations, grout holes, and foundation drainage holes.

3 FOUNDATION GEOLOGIC CONDITIONS

3.1 General Conditions

Most of the dam foundation area consists of soils of varying geologic origin overlying weathered rock that will be excavated to reach a suitable foundation surface. The soils include stream alluvium, colluvium, landslide deposits and residual overburden soil from in place weathered rock. Each of these soil types is described in the Site Characterization Report and is summarized below.

Most of the RCC dam will be founded on competent basalt bedrock that is part of the Crescent formation. Various rock types were encountered in the site explorations and the Site Characterization Report includes descriptions of the geologic units and overall site geology. The bedrock types are summarized below. Details from the site explorations are included on the composite boring logs in the Phase 2 Site Characterization Technical Memorandum (HDR 2017).

3.2 Soils

3.2.1 Stream Alluvium

Stream alluvium is located along the valley floors of the Chehalis River and its tributary streams and consists primarily of very loose to loose, light to dark brown, stratified slightly silty fine sand, gravelly sand and sandy gravel. 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. Qao tends to be denser than Qa and contains subrounded to subangular gravel.

3.2.2 Colluvium

Colluvium (Qc) consists of poorly sorted, loose to dense, light-brown to reddish-brown, sandy to gravelly clay or silt deposited on or at the base of hill slopes, primarily through gravity-driven transport of weathered rock and soil. These deposits may contain high percentages of subangular boulders consisting of basalt and gabbro ranging widely in size, and could be more than 2 feet in maximum dimension. Seismic refraction surveys suggest that these materials range from 0 to 25 feet in vertical thickness along the upper slope of the right abutment near DB-5.

3.2.3 Landslide Deposits

Quaternary landslide deposits (Qls) are made up of heterogeneous, mostly unsorted and unstratified debris that is often characterized by hummocky topography, closed depressions, springs or seeps, and a lobate form. The soil in landslide deposits is highly variable. They were observed to consist of loose to very dense, reddish brown to dark gray 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 direction.

Landslide thickness can vary considerably depending on the configuration and depth of the failure plane, ranging from relatively thin (less than 10 feet thick) to more than 100 feet thick at the toe in a deep-seated failure. The two landslides observed in the left abutment by BH-1 and BH-4 completed in Phase 1 and by LSB-1 and LSB-2 completed in Phase 2 appears to be deep-seated failures with a maximum thickness of about 70 feet.

3.2.4 Overburden Soil

Where soil has not been deposited by fluvial processes or gravity it is considered residual soil, developed in place from weathering of the bedrock beneath it. Quaternary overburden soil (Qos) consists of medium dense to very dense, red-brown or yellow-brown to dark gray 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 in the Qos range from low to high plasticity except at about 30 feet depth in OB-2 where the sandy silt was non-plastic. Overburden soils frequently contain highly weathered clasts of bedrock that are angular to subangular.

3.3 Bedrock

3.3.1 Intrusive Igneous Volcanics

Intrusive volcanic rocks (Tig) have been identified in the vicinity of the dam site by geologic maps and previous studies as primarily gabbro; which typically is high to very high strength, dark gray to black, occasionally white or black-speckled, aplanatic to medium grained and massive to columnar or block jointed rock. However, Tig was only encountered in one boring to date at the site (BH-4) and the material there may be part of the disturbed landslide complex.

3.3.2 McIntosh Formation

The McIntosh Formation (Tml) represents a thick sequence of locally tuffaceous marine siltstone and claystone with interbedded arkosic sandstone and basaltic sandstone. In the vicinity of the dam site, McIntosh claystone is interbedded with Crescent formation basalt (described in the next section), that creates the steep hill slope of the right abutment and underlies the saddle dam footprint. The McIntosh formation has only been observed in BH-6 and is locally very weak to weak, gray, very fine grained, and slightly to moderately weathered with completely weathered zones and cross-bedded sandy siltstone interbeds. Due to the contemporaneous depositions with the crescent formation it is difficult to determine if the claystone units found in the right abutment (DB-5, DB-6, and DB-7) are McIntosh formation or Crescent formation claystones, as discussed in the following section.

3.3.3 Crescent Formation Basalt and Siltstone/Claystone

The Crescent Formation (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 basalts resulting in interbeds of siltstone and claystone. 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 that were lithified to siltstone/claystone (Tcs) and occasionally claystone breccias consisting of basalt clasts in a claystone matrix by subsequent events of pillow basalt flow deposition. Both the Crescent and McIntosh Formations were deposited in the early to middle Eocene age contemporaneously.

The Crescent formation basalts were found in every borehole and ranged from weak to very strong. The strength of these materials generally increases 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 and the basalt is locally slightly vesicular. The Crescent Formation basalt makes up a large portion of the subsurface lithology at the potential dam site.

In between basalt flows, local volcanic rocks weathered and were eroded and deposited as silt and clay interbedded units, ultimately becoming siltstone (claystone) within the Crescent Formation as a sub-unit (Tcs). These materials have some similar characteristics to the McIntosh formation since they were derived the same way however Tcs is older, more consolidated and generally less weathered than the McIntosh formation

3.4 Groundwater

Groundwater levels were measured in the boreholes and are tabulated in Table 2-1 above. Groundwater levels are variable across the site, sometimes being observed relatively close to the ground surface (20 to 30 feet) and sometimes being observed very deep in the bedrock.

4 DAM FOUNDATION

4.1 General Approach

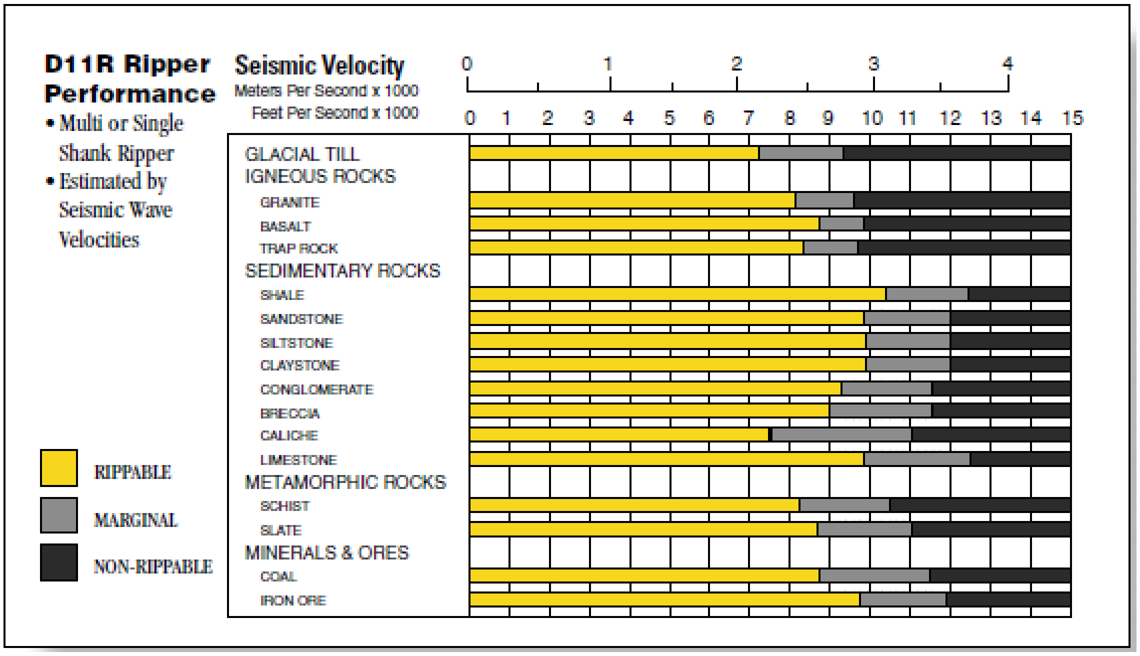
4.1.1 Bedrock Quality and Rock Rippability Approach

A commonly used approach to modern dam foundation design and construction on sites where bedrock quality tends to increase with depth is to establish the initial foundation excavation objective based on construction equipment performance such as "rippability" of rock. Rippability is the ease with which soil or rock can be mechanically excavated. According to Bieniawski (1989), rippability of rock is assessed by numerous parameters including uniaxial strength, degree of weathering, abrasiveness, and spacing of discontinuities. The geophysical field exploration testing method called "seismic refraction tomography" has historically been the geophysical method utilized to indirectly estimate the degree of rock rippability.

Ripping is typically performed by tractor-mounted equipment. The size or model of the required tractor (dozer) is determined by the ripping assessment of the rock. The hardness and competency of each individual material will determine the ease of rippability. Rock that is too hard to be ripped can be locally shaped, and fragmented with explosives. Rock types vary in rippability, depending on their degree of stratification or foliation. Sedimentary rocks are generally the most rippable. Highly stratified or laminated rocks and rocks with extensive fracturing are usually rippable also.

For example, as shown in Figure 4-1, a Caterpillar D11R tractor operating with a single shank ripper should be capable of ripping through basalt with a compression wave velocity (i.e., seismic velocity) less than about 8,000 feet per second (ft/s). The same equipment has marginal ripping capability with a seismic velocity of about 9,000 ft/s, and cannot rip basalt with a seismic velocity of 10,000 ft/s or greater. Combining the use of compression wave velocity with results of borehole explorations provided the basis to develop a three-dimensional (3-D) model of the estimated depth and shape of excavation required to create an adequate foundation for the dam. In addition, this information will provide a proven basis for construction contractors to estimate and bid construction costs with an understanding of the rock rippability and type of equipment needed to achieve the foundation excavation objective.

Figure 4-1
D11R Ripper Performance



Source: Caterpillar Handbook of Ripping (12th Edition)

For this conceptual design phase of the project, there was adequate foundation rock information obtained from the Phase 1 and Phase 2 site investigation programs to estimate the extent of foundation excavation required to achieve the foundation objective. However, interpretation of subsurface conditions over significant distances between borings was necessary, even with the existing geophysical testing, to estimate the foundation objective.

A number of areas near the Chehalis River and in the right and left abutments will have relatively shallow excavations to achieve the required foundation excavation. Portions of these areas will likely require controlled blasting for practical removal. Considerable attention must be given to blasting operations so that blasting produces a properly shaped foundation surface and to prevent damage of the rock beyond the specified limits of the excavation. Localized shaping of the foundation excavation will also likely require controlled blasting operations including presplitting. Detailed blasting requirements will be developed as part of later stages of design.

4.1.2 Geologic Data Used to Inform the Excavation Objective

The engineering process of foundation design involved evaluating all of the rock properties and characteristics collected during the site investigations to assess the depth to suitable and acceptable quality rock for the dam foundation. As mentioned above in the rippability discussion, the seismic velocity value obtained from geophysical testing is a key characteristic, but other information was also considered as described below.

The following characteristics and criteria were used to evaluate the foundation excavation depth at each borehole location:

- Rock type (basalt preferred over siltstone and claystone)
- Increasing Rock Quality Designation (RQD; greater than 50)
- Visual examination of fractures and fracture spacing from televiewer logs
- Low weathering index
- Rock Mass Rating (RMR) of about 55 or higher
- Engineering/geologic judgment

After estimating the rock foundation elevation at each borehole location, the excavation depths were compared to the compression wave velocities where they crossed the boreholes. Based on this evaluation and the equipment rippability described above, the approximate depth corresponding to a seismic (compression wave) velocity of 9,000 ft/s was established as the depth where suitable dam foundation rock would be located between boreholes. This process was then applied to the area beneath the entire footprint of the RCC dam to establish a 3-D dam foundation excavation surface.

A significant amount of engineering judgment was applied in estimating the foundation excavation extent in locations where no subsurface information was available. In addition, in some places acceptable foundation rock was encountered or estimated to be at shallower depths compared to adjoining areas. Some areas of acceptable foundation rock may need to be removed to achieve proper foundation shaping considering 3-D layout so that an adverse geometry is avoided under the dam. Adverse geometry could result in stress concentrations in the dam, uncontrolled cracking, or differential movements between monoliths during seismic loadings. Proper foundation design involves shaping of the rock to a gradually varying surface, elimination of abrupt geometry changes, and a final optimizing of the amount of rock shaping and blasting, which adds costs. The foundation shape can be estimated with more confidence as more subsurface information is obtained, but the final configuration will not be known until construction is underway.

4.2 Excavation Objective

Establishing the appropriate foundation excavation objective (i.e., the estimated limit of excavation or foundation surface that provides suitable quality, strength and deformability characteristics to meet design requirements) is a critical element of the dam design. Information presented below has been utilized to develop an excavation objective surface meeting the rock quality objectives presented in this report. The conceptual design excavation configuration for FRO and FRFA dam alternatives are shown in drawing sheets FRO S-1 and S-2; and Drawings FRFA S-1 and S-2 of Conceptual Combined Dam and Fish Passage Design Report-Appendix A.

The excavation of overburden soils and highly weathered rock (i.e., common excavation) will involve removal of a significant amount of material, particularly on the left abutment. Excavation with typical earthmoving equipment will be feasible in the overburden soils and weathered bedrock materials. The geophysical data and interpretations indicate that a large portion of the upper bedrock excavation required to reach the foundation excavation objective limits can be achieved with a single ripper D-9R Caterpillar bulldozer (or equivalent) to depths ranging from 20 to over 50 feet. Excavation of soil and rock on the right abutment will require a greater degree of design and planning due to the steep topography. Table 4-1 provides a summary of the excavation objective depth at various exploration locations. Site characterization indicates the potential for localized areas of decreased rock quality. These areas may require localized over-excavation or foundation treatment which may include the use of dental concrete, shaping blocks, consolidation grouting, or combinations of these methods to create a suitable foundation for the dam. Subsequent site characterization work should be targeted to address areas where explorations indicate the potential for deeper excavations in order to better define the costs and risks associated with lower quality rock and to further identify the extent and methods for additional treatment. Such evaluations will be necessary to complete the preliminary design, and develop cost estimates meeting a suitable level of accuracy for final decision making and budgeting.

The large landslide in the left abutment upstream of the dam axis does not fall within the dam footprint. This landslide will likely require some combination of removal and stabilization as part of the dam construction work due to its proximity to the dam and outlet works facilities. A smaller landslide in the left abutment is within the dam footprint. This landslide will likely be completely removed to competent rock as part of the dam excavation.

The dam foundation area was subdivided into four general areas in order to assess the foundation excavation objective as follows: (1) left abutment of the RCC dam, (2) central portion of the RCC dam, (3) right abutment of the RCC dam, and (4) earthfill/rockfill saddle dam foundation. Geotechnical and geological information for the selection of excavation objective in each area is described in separate sections below.

Table 4-1
Summary of Excavation Objective Evaluations at Exploration Locations

BOREHOLE	ELEVATION	LOCATION	TOP OF ROCK	SELECTION, LIMIT OF RIPABILITY (9,000 FEET/SECOND)	FOUNDATION OBJECTIVE DEPTH (FEET)
Phase 1 SL-1 Spread 1,2	775 to 450	dam axis		Varying between 80 ft., 50 ft., 70 ft., 60 ft. and 80 ft. at stations 0+50, 1+20, 1+60, 2+70 and 3+50, respectively.	
BH-3	607	u/s offset left abutment	35.5 ft. basalt, 133 ft. claystone	60 based on SL-1 35-40 ft. based on seismic SL-5 and LSSL-6 37 ft. based on strength and weathering index, RQD 95% and RMR 60 on surface	37

BOREHOLE	ELEVATION	LOCATION	TOP OF ROCK	SELECTION, LIMIT OF RIPABILITY (9,000 FEET/SECOND)	FOUNDATION OBJECTIVE DEPTH (FEET)
BH-5	566	u/s offset left abutment	47.5 ft. siltstone, 70 ft. basalt	70 based on SL-1 60 ft. based on log, minor fractures/joints	60
Phase 1 SL-1 Spread 1,2	775 to 450	dam axis		Varying between 70 ft., 60 ft., 60 ft. and 50 ft. at stations 4+50, 5+10, 5+50, and 6+00, respectively. SL-3 which cross SL-1 around station 5+10 also confirm the 9,000 ft/s velocity depth around 60 ft.	
BH-1	504	d/s offset left abutment	35.2 ft. basalt, 102.4 ft. claystone	35 ft. based on seismic line SL-1, 35 ft. SL-3 and SL-4 37 ft. based on strength and weathering index, RQD 90% and RMR 65 on surface, weathering index	37
DB-1	470	dam axis, middle	30 ft. basalt, 77 ft. claystone	60 ft. based on SL1 low RQD between 30 to 35, RMR and RQD reach 60 and 90% and strength index and weathering index improve at 36 ft.	36
Phase 1 SL-1 Spread 1,2	775 to 450	dam axis, middle		90 ft. at station 7+00	
OB-2	468	dam axis, middle, FRFA Outlet works tunnel	38 basalt, 95 claystone	85 ft. on DSL2; -50 ft. on SL1 RMR and RQD reach to 55 and 85% at a depth of about 65.5 ft. depth and weathering index 2. RMR and RQD dip to 50 and 30% within 70 ft. depth; it remains relatively low permeability at 1E-7 cm/s. High RQD and RMR values between 80 and 95 ft. depth.	81
Phase 1 SL-1 Spread 1,2	775 to 450	dam axis, middle		50 ft. at station 8+20	
BH-2	452	slightly d/s offset, middle	30 ft. basalt, 81 ft. claystone	50 based on SL1 RMR and RQD reach to 60 and 90% at a depth of about 40.5 ft. depth low quality rock 85 to 90	40.5
Phase 1 SL-1 Spread 1,2	775 to 450	dam axis, middle		varying between 50 ft., 30 ft., and 20 ft. at station 9+00, 9+50, and 10+00, respectively	
Phase 1 SL-1 Spread 3,4	450 to 680	dam axis, middle		10 ft. at station 11+25	

BOREHOLE	ELEVATION	LOCATION	TOP OF ROCK	SELECTION, LIMIT OF RIPABILITY (9,000 FEET/SECOND)	FOUNDATION OBJECTIVE DEPTH (FEET)
TB-2	468	dam axis, middle, flood control tunnel at the dam alignment	0 basalt, 19.1 ft. claystone, 23.6 ft. basalt	30 based on SL1 and DSL1 RMR and RQD reach 70 and 75%, respectively at a depth of about 12.5 ft. depth. RMR and RQD dip to 50 and 25% within 17.2 ft. depth yet it remains relatively low permeability at 1E-7cm/s	12.5
TB-4	459	d/s offset, middle, FRFA hydro power outlet	10 ft. basalt, 44 ft. claystone	0 based on DSL1 RMR and RQD reach 70 and 85% at a depth of about 10 ft. depth	10
Phase 1 SL-1 Spread 3,4	450 to 680	dam axis, middle		varying between 10 ft., 30 ft., and 30 ft. at station 12+50, 13+50, and 14+50 respectively	
DB-5	687	slightly d/s offset, right abutment	40 ft. basalt, 90 ft. claystone	80 based on SL1 RMR and RQD reach 60 and 95% at about 51 ft. depth. RMR and RQD dip to 55 and 80% within 65 ft. depth yet it remains with high strength index and low weathering index.	51
Phase 1 SL-1 Spread 5	685 to 705	dam axis, right abutment		varying between 80 ft., 70 ft., 70 ft., 110 ft., and 120 ft. at station 15+25, 16+00, 16+75, 17+25, and 17+70, respectively	
DB-6	674	dam axis, right abutment	31.5 claystone, 144 basalt	120 based on SL1 low RQD between 31.5 to 120 basalt at 144	120

Notes:
cm/s = centimeters per second
d/s = downstream
ft. = feet
ft/s = feet per second
RMR = rock mass rating
RQD = rock quality designation
u/s = upstream

4.2.1 Left Abutment of the RCC Dam (Stations 0+00 to 6+00)

Available geotechnical/geologic information in this area is based on boreholes BH-5, BH-3, BH-1, LSB-1 and geophysical seismic lines SL-1 (spreads 1 and 2), SL-3, SL-4, SL-5, and LSSL-6. Based on SL-1 which is on the dam centerline, a velocity of 9,000 ft/s is observed at depths of 80, 50, 70, 60, and 80 feet at stations 0+50, 1+20, 1+60, 2+70 and 3+50, respectively.

BH-3 is on the left abutment offset upstream at station 2+00. Basalt rock was encountered at 35.5 feet depth with high RQD (95%), high RMR (60) with minor joints/fractures. Based on BH-3, the best estimate for the foundation objective is at 37 feet depth, since the weathering index and strength index both improve within the top 2 feet. Geophysical lines SL-5 and LSSL-6 indicate a velocity of 9,000 ft/s at about 35 to 40 feet depth also in this area.

BH-5 is on the left abutment near station 3+70. Basalt rock was encountered at 47.5 feet depth with a relatively low RQD (60%), low RMR (42) with major and minor joints/fractures. Based on BH-5, the best estimate for the excavation objective is at 60 feet depth. RMR reaches 55 at a depth of about 60 feet Although RMR and RQD drop to 45 and 80% at about 65 feet depth, minor open joints/fractures remain. Geophysical lines SL-1 indicate 9,000 ft/s velocity is observed around 70 feet depth in this area.

Based on SL-1 which is on the dam axis, a velocity of 9,000 ft/s is observed at depths varying between 70, 60, and 50 feet at stations 4+50, 5+10, 5+50, and 6+00, respectively. SL-3 which crosses SL-1 around station 5+10 also confirms the 9,000 ft/s velocity depth around 60 feet.

BH-1 is on the downstream left abutment near station 5+20. Basalt rock was encountered at 35.2 feet depth with high RQD (90%), and high RMR (65). Based on BH-1 the best estimate for the foundation objective is at 37 feet depth, since the weathering index improves within the top 2 feet. Geophysical lines SL-3 and SL-4 indicate a velocity of about 9,000 ft/s is observed around 35 feet depth in this area.

4.2.2 Central Portion of the RCC Dam (Station 6+00 to 12+00)

The central portion of the RCC dam includes hydraulic structures such as the stilling basin, spillway, fish ladder, and diversion tunnel. Available geotechnical/geologic information in this area is based on boreholes DB-1, OB-2, BH-2, TB-2 and geophysical lines SL-1 (spreads 1, 2, 3 and 4), SL-3, SL-4, SL-5, DSL-1, DSL-2.

DB-1 is on the dam axis around station 6+40. Basalt rock was encountered at 30 feet depth with moderately low RQD (70%) and low RMR (50) with major to minor joints/fractures. Based on DB-1 the best estimate for the foundation excavation objective is 36 feet depth due to the strength and weathering index values. RMR and RQD reach 60 and 90, respectively at a depth of about 36 feet depth and do not decrease below 60 and 35, respectively until the claystone unit is encountered at 77 feet depth. Geophysical line SL-1 indicates 9,000 ft/s velocity is observed around 60 feet depth also in this area. The current foundation model was established with a foundation depth at 50 feet, although this depth could likely be reduced with additional subsurface exploration information targeted in this area.

OB-2 is on the dam axis at about station 7+80. Basalt rock was encountered at 38 feet depth with low RQD (0 to 75%) and low RMR (30-50) with minor joints/fractures. Based on OB-2 one estimate for the foundation excavation objective is at 65.5 feet depth. RMR and RQD reach 55 and 85% at a depth of about 65.5 feet depth and weathering index improves. Although RMR and RQD dip to 50 and 30% within 70 feet depth, permeability remains relatively low at 1E-7 centimeters/second (cm/s). RQD and RMR values remain high between 80 and 95 feet depth. The alternative estimate for the excavation objective is 81 feet depth, below the fracture zone and low weathering index. Claystone was encountered at 95 feet depth. Geophysical lines SL-1 and DSL-2 indicate a velocity of 9,000 ft/s is observed around 50 and 85 feet depth respectively.

Based on SL-1 located on the dam axis at station 8+20, a velocity of 9,000 ft/s is observed at depth of 50 feet. BH-2 is close to the centerline with a slight downstream offset at about station 8+60. Basalt rock was encountered at 30 feet depth with low RQD (20 to 55%), low RMR (35-50) with minor joints/fractures, high weathering and low strength index. Based on BH-2 the best estimate for the foundation excavation objective is at 41 feet depth. RMR and RQD reach to 60 and 90% at a depth of about 40 feet depth. Although RMR and RQD dip to 40 and 35% within 85 feet depth within the claystone layer, it was judged that the material properties sufficiently account for the claystone at this depth. Geophysical line SL-1 indicates a velocity of 9,000 ft/s was observed at about 50 feet depth.

Based on SL-1, spreads 1 and 2, which are on the dam axis a velocity of 9,000 ft/s is observed at depths varying between 50, 30, and 20 feet at stations 9+00, 9+50, and 10+00, respectively. The shallower depths are near the current river channel where very strong, high quality rock is exposed in outcrops. Similarly, based on SL-1 spreads 3 and 4, which are on the dam axis farther to the right of spreads 1 and 2, a velocity of 9,000 ft/s is observed at a depth of 10 feet at station 11+25.

4.2.3 Right Abutment of RCC Dam (Station 12+00 to 16+50)

The right abutment of the dam is very steep terrain. No borings have been advanced on the right abutment between the river channel and the upper ridge due to extremely difficult access. The upper right abutment of the FRFA dam will include the transition from RCC dam to an earthfill/rockfill saddle dam. The interpretation of conditions in this area is based on geotechnical/geologic information from boreholes DB-5, DB-6, DB-7 (which are located on the right abutment ridge) and geophysical lines SL-1 spreads 3, 4, and 5.

The only subsurface information available from the steep terrain portion of the right abutment is spreads 3 and 4 of SL-1. Based on SL-1, a velocity of 9,000 ft/s is observed at depths varying between 10 and 30 feet. DB-5 is on the downstream side close to the bend in the alignment at about station 15+50. Basalt was encountered at 40 feet depth with medium RQD (80%), medium RMR (55) with minor joints/fractures. Based on DB-5 the best estimate for the foundation excavation objective is 51 feet depth. RMR and RQD reach 60 and 95% at about 51 feet depth. Although RMR and RQD dip to 55 and 80% within 65 feet depth, the strength index remains high with a low weathering index. Farther up on the right abutment, SL-1 indicates a velocity of 9,000 ft/s at depths between 70 and 120 feet.

4.2.4 Earthfill/Rockfill Embankment Dam Foundation (Station 16+00 to 26+15)

A topographic low area exists in the upper right abutment ridge of the dam site. A saddle dam will be needed in this area to achieve the FRFA dam crest elevation of 713.4 feet. An earthfill dam consisting of a central zone of lower permeability embankment materials with upstream and downstream rockfill shells will wrap around the end of the RCC dam section at about dam axis station 16+50. The overburden soil and weathered bedrock on the top of the ridge tends to be thicker than across the valley of the main dam.

DB-6 is on the downstream side close to the bend in the alignment at about station 15+00. Claystone was encountered at 31.5 feet depth with low RQD (0 to 60%), low RMR (30-45) with minor joints/fractures and higher weathering index and low strength index. Based on DB-6 the best estimate for the foundation excavation objective for the RCC dam would be about 120 feet depth or deeper. A weaker claystone locally extends to a depth of 144 feet with low RQD (50%) and RMR (45) values. RMR and RQD reach 60 and 75% of about 120 feet depth. Geophysical line SL-1 indicates a velocity of 9,000 ft/s is observed around 120 feet depth. Basalt was encountered at 144 feet depth.

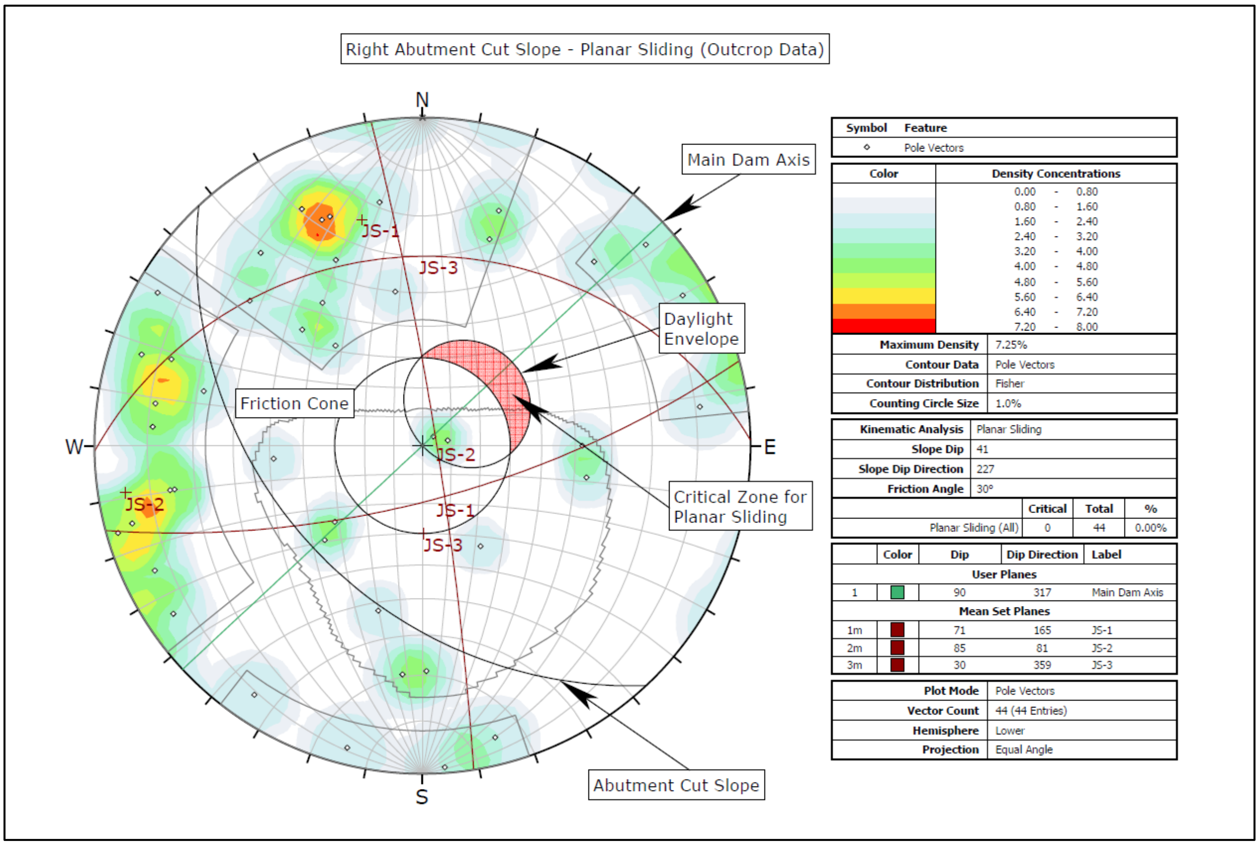
This large zone of weaker material around DB-6 is unsuitable for RCC foundation and excavation to depths of 120 to 140 feet would be extremely costly considering the relatively small heights needed for the saddle dam. Therefore, with the information obtained from the Phase 2 site investigation, the transition from RCC to embankment has been shifted closer to the RCC dam axis. An embankment dam in this area would be more tolerant to settlement of the foundation and a camber can be included to accommodate the relatively small about of settlement and deformations that may occur during construction and first reservoir filling. The saddle dam embankment will be constructed of rockfill materials with a central earth core. The embankment will have side slopes of 2H:1V and could have a maximum structural height of up to 50 feet in the vicinity of DB-6 depending on the final foundation excavation and treatment requirements. The foundation for the earthfill does not need to meet the deformation and strength requirements defined for RCC foundation or a very high embankment dam. For the initial design it is assumed that embankment excavation will be 5 feet under the embankment rockfill shells and 15 feet under the central embankment earthfill core. The deeper core excavation and fine grained core will provide a seepage barrier. An additional cutoff wall may be necessary for the seepage concerns but more subsurface information on the right abutment is necessary to advance seepage control designs for the saddle dam.

4.2.5 Excavation Cut Slopes and Kinematic Stability

Temporary excavation slopes are anticipated to be 1.5H:1V through overburden soils and weathered rock, and 0.5H:1V through sound rock. The kinematic stability of the dam excavation slopes on the right and left abutments was reviewed by evaluating joint data from both the dam site outcrops and the televiewer structure data. The method of evaluation is based on an analysis methodology described by Goodman (1980) which evaluates the potential for localized slope failures by three different mechanisms; toppling, planar sliding and wedge sliding. These mechanisms would most likely be associated with excavation slopes along the upstream or downstream sides of the excavation for the dam or hydraulic structures.

The results of the kinematic analysis indicate that left abutment excavation has no significant potential for any failure mechanism to develop. For failure to occur, sliding on either joint sets J1 and J3, or the intersection of J1 and J3 would have to occur. Only two of the three potential failure mechanisms showed any risk at all. Joint sets are plotted on the stereonet on Figure 4-2 below. These mechanisms were found in the right abutment and include a planar sliding failure along J5 that has a very low risk of occurring. A toppling failure associated with J2 has a low risk based on outcrop data. Risk ratings were assigned based on the percentage of data points that appear within the critical zone as described in the Site Characterization Report. However; since a majority of the boreholes were vertical the televiewer data may contain a structural bias that should be further evaluated and additional angled boreholes to intersect the major joint sets should be included in future site investigation work. Additional details of the kinematic stability analysis including all stereonets are included in the Site Characterization Report.

Figure 4-2
Example Stereonet – Right Abutment Cut Slope



4.2.6 Special Considerations Related to Siltstone and Claystone

As discussion in section 3.3, there are interbedded siltstone and claystone materials within the Crescent Formation Basalts that introduce some important design considerations. When confined within the foundation, these materials have low to moderate strength properties that are suitable for the foundation of the dam. However, when exposed in excavations or foundation grout and drain holes, these materials are prone to slaking and degradation. Special treatment is necessary to minimize the slaking and degradation. These issues and likely treatment strategies are summarized below.

4.2.6.1 Slaking and Degradation in Exposed Excavations

The initial 3D geologic model of the site and the proposed excavation objective and configuration presented in this technical memorandum have identified that exposure of the siltstone and claystone materials will be limited to the foundation of the outlet works discharge stilling basin. However, there is some risk of localized exposure in other locations. In order to preserve the foundation excavation surface, a dental concrete mat, and shotcrete cover of sloping excavation surfaces where the siltstone and claystone materials are exposed will be required. In addition, any borehole excavation for foundation anchors will have to be installed within specified timeframes. Some testing of anchors under variable installation timeframes will likely be required to establish the final anchor requirements relative to spacing and length. It is important to note that the presence of the siltstones and claystones does not preclude construction of a safe and robust dam, it simply requires a different level of foundation treatment that is not uncommon in the dam design and construction industry.

This design consideration will have important impact to the planning of future site characterization studies. Additional borings are needed in the dam foundation to continue the development of the excavation objective and configuration for the dam as well as to manage the design and construction risks associated claystone and siltstone materials.

4.2.6.2 Deterioration of the Foundation Drain Holes

Foundation drain holes installed for control of uplift pressures acting on the base of the dam will likely encounter siltstone and claystone materials. Left untreated in the drain holes, the siltstone and claystone materials will deteriorate impacting the function of the drains and the frequency of drain maintenance. To minimize these detrimental effects, drain holes will need to be properly drilled and logged during installation and well screens with filter packs will be required. These provisions will result in an increase in the installation costs for the drains but should provide suitable protection of long-term drain function and minimize long-term maintenance requirements.

4.3 Foundation Surface Treatment

The process of excavation of rippable rock will require a higher level of engineering geology inspection compared to common excavation. Some highly jointed but strong rock may be easily excavated with equipment, and other stronger rock may not be rippable, but still may require removal by other means to achieve an acceptable foundation shape. After the initial excavation is completed, the exposed foundation rock should be cleaned, carefully examined, and mapped to verify that the excavation objective has been reached.

The initial excavation may reveal fractures, defects, and zones of unsuitable rock that will have to be selectively removed and/or treated. Replacing portions of the foundation that exhibit less favorable conditions with dental concrete, consolidation grouting, or construction of shaping blocks will improve the stability and performance of the dam and will be an important part of foundation preparation. Dental concrete will be required to complete the shaping of the foundation surface and reduce damage and/or deterioration of exposed foundation rock during construction activities, particularly if any claystone or siltstone materials are exposed during construction activities.

The final excavated surface for the dam footprint will be variable across the site. It is important that the foundation be shaped so that a gradually varying surface is created along the dam axis, and from the upstream to the downstream toe of the dam. The prepared surface should be free of offsets or sharp breaks. Sharp breaks in the excavation can cause marked changes in stresses in both the dam and the foundation and have the potential to lead to adverse cracking and seepage from the RCC structure.

Once an acceptable foundation excavation and treatment are confirmed, the surface should undergo a final cleaning to remove remaining loose material. Cleaning will involve hand removal of unsuitable loose material, washing of the foundation rock surface, and removal of wash water and debris.

4.4 Foundation Material Properties for Analysis

With an understanding of the quality of rock that will serve as the foundation for the RCC dam, foundation bedrock material properties were estimated for use in structural analyses. Foundation material properties used in the seismic structural response evaluation analysis are provided in Table 4-2 below. Three moduli values were used in the analysis: (1) a lower bound to conservatively represent a hypothetical siltstone/claystone only foundation, (2) upper bound to represent a basalt only foundation, and (3) a conservatively weighted average of basalt/siltstone, which is the expected condition. The friction angle assumptions for sliding varied between 35 and 55 degrees for the analysis, but the main SAP2000 analysis considered only a 45 degree friction angle. The post-earthquake sliding stability factor of safety calculation considered friction angles between 30 and 55 degrees. The material properties for RCC lift joints are also summarized in Table 4-2 because the analysis of the dam considered sliding at the rock-foundation contact as well as within RCC lift joints low in the dam near the dam foundation

Table 4-2
Foundation Material Properties

PROPERTY	RCC/LIFT JOINTS	ROCK /ROCK-CONCRETE INTERFACE
Unit weight (pcf)	150	165 (Rock)
Compressive strength (psi)	3000	N/A*
Direct tensile strength (psi)	150 (intact RCC)	0
Apparent static tensile strength (psi)	270 (lift joint)	0
Actual dynamic tensile strength (psi)	300 (lift joint)	0
Apparent Dynamic tensile strength (psi)	400 (lift joint)	0
Dynamic Mod. of Elasticity (psi)	3.24 x 106	1.39 x 106 (lower bound) 3.15 x 106 (upper bound) 2.61 x 106 (weighted average)
Poisson's Ratio	0.20	0.33
Hysteresis damping (percent)	0	1
Intact bonded friction angle (degrees)	65	55

PROPERTY	RCC/LIFT JOINTS	ROCK /ROCK-CONCRETE INTERFACE
Friction angle for cracked concrete, but no sliding has occurred (degrees) *	55	55
Residual friction angle of smoothed sliding planes (for sliding of more than two feet) (degrees) **	40	35
Average friction angle of rock-concrete interface or RCC lift joints after for sliding of less than one foot (degrees)	45	45
Average friction angle of rock-concrete interface or RCC lift joints after for sliding between one to two feet (degrees)	45	35 to 55

Notes:
* Cracked but with no sliding (limited the force resultant location to the base but no sliding during earthquake.
**No cohesion was assumed along the lift joints, concrete to rock interface, of bedding planes in the foundation.
pcf: pounds per cubic foot
psi: pounds per square inch

4.5 Grouting and Drainage

4.5.1 In Situ Testing

Hydraulic conductivity is a measure of the rate that water flows through interconnected joint and fracture systems within a rock mass. Highly fractured zones or locations where large open fractures occur in the bedrock were identified in the Phase 2 program. Unless treated (or excavated), these fractures could act as preferential seepage pathways beneath the dam foundation and abutment. This may result in excessive water pressures acting on the base of the dam or other structure/tunnel elements, or result in unwanted loss of stored water. The Phase 1 and 2 explorations, review of rock cores, down-hole geophysical tests, and water pressure tests have identified that foundation grouting to treat and reduce flow through fractures in the rock will be an important component of the dam design.

To evaluate the permeability and groutability of the bedrock, water pressure testing was conducted during the field explorations. It was noted that both the siltstone/claystone and basalt tend to have higher hydraulic conductivity values with increasing depth. A wide range and large variability of values occurs for both basalt and siltstone/claystone, especially among the bedrock found at depths ranging from about 100 to 150 feet.

Groutability was evaluated using Lugeon values calculated from hydraulic conductivity measurements. Lugeon (Lu) values are empirically defined as the hydraulic conductivity required to achieve a flow rate of 1 liter/minute per meter of test interval under a reference water pressure equal to 1 megapascal (Houlsby 1976). Ewert (2003) used a comparative analysis of data from a number of projects, together with lab test data, and presented the following generalized correlations of Lu values and groutability:

- Small Lu values (less than 2 to 5) usually indicates ungroutable rock.
- Moderate Lu values (less than 5 to 10) usually indicate poor groutability.

- Large Lu values (greater than 10) may indicate a groutable rock. However, joint frequency and fissure widths must be evaluated.

Weaver and Bruce (2007) provide commentary related to the Ewert groutability scale. Specifically, these values may correlate to “old” grouting technology and are not wholly valid for modern technology where it may be possible to grout to a 1-Lu closure standard.

4.5.2 Grouting Design

Based on the in situ test results, evaluation of core samples from the boreholes, and our general experience on other similar project sites, a multi-line grout curtain extending to the approximate limits shown on drawing sheets FRO S-4 and FRFA S-4 (Conceptual Combined Dam and Fish Passage Design Report-Appendix A) should be included in design to reduce seepage through joints and other discontinuities in the bedrock foundation. Some highly fractured bedrock zones have been observed in the borings. Some of these fractured zones will be excavated because they are above the estimated excavation objective, but some fracture zones exist well below zones of competent less fractured rock. These zones will act as preferential seepage pathways beneath the dam foundation and abutment unless they are treated. Grouting will be performed to fill these rock discontinuities with cement grout and seal the highly fractured rock zones.

Based on initial structural information from the boreholes, it is anticipated that grout holes should be inclined 15 to 30 degrees in an orientation along the grout curtain alignment to intercept the maximum number of sub-vertical fractures in the basalt of the Crescent Formation. The number of grout lines, spacing and inclination of the grout holes, and sequence of grouting operations may vary along the dam based on results from supplemental site characterization studies. Grouting procedures should be based on a Lu closure criteria between 1 and 5. Additional site characterization work along the dam axis, including the upper right abutment, should be performed to better define the extent and depth of the foundation grouting program.

Consolidation grouting of the upper portion of the dam foundation bedrock is not anticipated at this time as the foundation surface can be effectively treated with localized dental excavations to provide an acceptable foundation contact surface for the RCC.

Foundation drains will be required to control uplift pressures on the base of the dam and within the foundation rock mass in order to achieve the stability and seismic design criteria for the project.

Oversized drainholes that are properly developed with well screens and sand packs across intervals of claystone/siltstone will be required to provide for long-term stability of the drain holes.

5 DEWATERING

Groundwater levels were measured in the boreholes and are tabulated on Table 2-1 above. Groundwater levels are variable across the site, sometimes being observed relatively close to the ground surface (20 to 30 feet) and sometimes being observed very deep in the bedrock. During the excavation for the dam foundation, it is likely that groundwater will be encountered in soil and rock units. Groundwater and surface water runoff will need to be managed to allow work in the excavation and particularly during final foundation cleaning, inspection and placement of the RCC on the dam foundation. Managing groundwater may involve dewatering soil and rock units using deep wells and/or drains. Control and care of water during construction will need to be planned and managed, particularly given the environmental sensitivity of the area.

6 USE OF EXCAVATED MATERIALS

As previously described, the excavation of overburden soils and highly weathered rock (i.e., common excavation) will involve removal of a significant amount of material, particularly on the left abutment of the dam. Common excavation materials will need to be stockpiled for use elsewhere on the project. Possible uses for these materials include constructing buttress fills to stabilize existing landslides, saddle dam earth and rockfill materials, and general site grading and hydraulic structure backfill. Some construction planning and design may enable common excavation materials to be hauled directly to final locations such as landslide buttresses, but some materials would also need to be efficiently stockpiled for future possible use. Appropriate stockpiling would include separate stockpiles for soil materials and weathered rock materials. Compacted weathered rock can provide a strong, free draining rockfill that can be used as embankment dam shell material that would add stability and provide significant resistance against seismic shaking.

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Appendix D
Seismic Evaluation

Chehalis Basin Strategy

Appendix D

Seismic Evaluation

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

Prepared for: State of Washington Recreation and Conservation Office and Chehalis Basin Work Group

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ACRONYMS AND ABBREVIATIONS

ACI	American Concrete Institute
AEP	annual exceedance probability
AFP	annual failure probability
CMS	conditional mean spectrum
CSZ	Cascadia Subduction Zone
DSO	Dam Safety Office
EM	engineering manual
EPRI	Electric Power Research Institute
ER	engineering report
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FRO	flood retention only
FRFA	flood retention flow augmentation
FT	feet
HCRCC	High-cementitious RCC
ICOLD	International Commission on Large Dams
kg/m^3	kilograms per cubic meter
lbs/yd^3	pounds per cubic yard
MCE	maximum credible earthquake
MCRCC	medium-cementitious RCC
NRCS	Natural Resources Conservation Service
pcf	pounds per cubic foot
PGA	peak ground acceleration
PMF	probable maximum flood
psi	pounds per square inch

RCC	roller-compacted concrete
RMR	Rock Mass Rating
TVA	Tennessee Valley Authority
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation

1 INTRODUCTION

1.1 Project Description

The proposed project dam site is located south of State Route 6 in Lewis County, Washington, on the South Fork of the Chehalis River, about 1 mile upstream of the community of Pe Ell, Washington. The site is located in a narrow bedrock gorge upstream of a much wider floodplain and valley opening to the Chehalis River valley. The dam is proposed as a gravity concrete (roller-compacted concrete [RCC]) dam supported on a basalt bedrock foundation with minor amounts of siltstone and claystone interbeds typical of submarine basalt depositional environments. The analyses presented in this memorandum can be used for alternative dam configurations being considered for the project including the Flood Retention Only (FRO) or Flood Retention Flow Augmentation (FRFA) options.

The FRO alternative consists of a concrete gravity dam structure with low-level fish passage and flood control outlet works, fish passage facilities, and a spillway at the crest of the dam. The FRO alternative is designed to store floodwaters only for events larger than the typical 5-year recurrence interval flood events. At all other times, the project is expected to freely pass all river flows with only minor restriction of flow.

The FRFA alternative consists of a concrete gravity dam structure with low-level flood control outlets and mid-level water quality outlets, fish passage facilities, and a spillway over the crest of the dam. The FRFA alternative is designed to provide some reservoir storage volume for flow augmentation in late summer and fall prior to the winter rainy season, primarily for fish habitat enhancement, as well as flood storage volume to be activated as needed in flood events larger than, approximately, the 5-year recurrence interval event.

The following summary of the FRO and FRFA alternatives is based on the studies performed for the 2014 report and is consistent with the design revisions that have taken place through development of the Combined Dam and Fish Passage Conceptual Design Technical Memorandum.

- FRO
  - Reservoir storage of up to 65,000 acre feet of water
  - Dam sized for flood storage with an estimated maximum dam structural height of about 260 feet, and a dam crest length of about 1,380 feet (HDR 2014)
  - Outlet works including a lower right abutment tunnel for construction diversion, and a low-level outlet through the base of the dam for flood control operations
  - A central overflow spillway designed to safely pass probable maximum flood (PMF) without dam failure. The spillway would include an ungated Ogee overflow crest structure, spillway chute, flip bucket, and plunge pool

  - Fish passage designed for free passage upstream and downstream prior to and after flood, and trap and haul during flood regulation periods
- FRFA
  - Reservoir storage of up to 130,000 acre feet, with 65,000 acre feet available for flood storage regulation and 65,000 acre feet for active water quality augmentation
  - Estimated maximum dam structural height of about 310 feet, and a dam crest length of about 1,650 feet (HDR 2016) for the RCC portion of the dam. A central core rockfill dam section that is approximately 930 feet long would be constructed in the upper right abutment area. This rockfill section of the dam will be joined to the RCC section with a typical wrap-around detail
  - Multiple outlet works, including a lower right abutment tunnel for construction diversion, a water quality inlet/outlet that draws water from multiple levels within the reservoir, and a low-level flood control outlet through the base of the dam
  - A central overflow spillway designed to safely pass the PMF without dam failure. The spillway includes an ungated Ogee overflow crest structure, spillway chute, flip bucket, and plunge pool
  - Upstream fish passage by trap and haul fishway; downstream fish passage by trap and haul

1.2 Project Team

The HDR team members that performed the analyses and prepared this technical memorandum were as follows:

Scott Anderson, P.E.	Structural Analysis Modeling
Farzad Abedzaduh, PhD, P.E.	Consultant on Modeling and Model Review
Keith A. Ferguson, P.E.	Principal Engineer
Dan Osmun, P.E.	State and Federal Dam Design Approach
Matt Redington, P.E.	Report QA

2 ANALYSIS APPROACH

2.1 Dam Design Guidance

The project is being funded by Washington State. Final design and construction may include federal funding and/or federal reviews. The strategy for identification of design criteria outlined in this document is to develop a dam design that will satisfy state and federal requirements.

2.1.1 State Guidelines and Requirements

The Washington State Department of Ecology Dam Safety Office (DSO) uses a risk-based procedure for design that is tied to the consequences of failure in downstream areas. The procedure can also be tied to a downstream hazard classification (consistent with federal guidelines) of the proposed dam.

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 based on Table 1 in the guidelines. A total of six separate parameters are assessed under three consequence categories; 1) capital value of the project, 2) potential for loss of life, and 3) potential for property damage.

Both dam configurations present unusual considerations in assigning an appropriate consequence rating as a result of the amount of the reservoir storage pool that is dedicated to flood storage only and the very limited amount of time that the flood storage is used. A conservative approach under these conditions is to set design criteria to a standard that corresponds to "full pool" storage condition. This approach would involve following the practices of federal agencies and other state agencies for "detention dam" structures. Table 2-1 below presents a summary of the estimated consequence rating for the proposed Chehalis Dam project FRO and FRFA configurations using this "detention dam" approach.

Table 2-1
Summary of Preliminary Consequence Rating for Chehalis FRO and FRFA Dam Configurations

CONSEQUENCE CATEGORY	INDICATOR PARAMETER	RATING – FRO	RATING – FRFA
Capital Value of Project	Dam Height	120	130
Capital Value of Project	Project Benefits	20	25
Potential for Loss of Life	Catastrophic Index	60	70
Potential for Loss of Life	Population at Risk	60	70
Potential for Loss of Life	Adequacy of Warning	70	80
Potential for Property Damage	Items Damaged or Services Disrupted	80	100
Total Rating		410	475

Using these consequence ratings and Figure 1 in the DSO guidelines, the following are the design step and corresponding annual exceedance probability (AEP) requirements proposed for the conceptual/ preliminary design of the alternative dam configurations:

FRO	Design Step = 4	AEP = 1x10-4
FRFA	Design Step = 5	AEP = 4x10-5

Based on our current knowledge of the population at risk in the downstream corridor, as well as our experience with the federal hazard classification system for dams, the proposed FRO and FRFA dam configurations will likely classify as "high" hazard potential structures. The DSO-related design step and AEP requirements outlined above are consistent with a "high" hazard potential and a downstream hazard classification of 1C or 1B, as shown in Table 2 in the DSO guidelines.

It should be noted that the DSO design step and AEP requirements outlined above will continue to be evaluated through the project until such time as final designs are underway. The design step and AEP classifications will be subject to change based on input from the state.

The DSO guidelines are focused primarily on analysis of existing low embankment structures, which comprise the majority of the dams under state jurisdiction. Gravity and RCC dams and barriers are not discussed in detail.

With regard to large concrete dam structures, the Dam Safety Guidelines include the following statement:

"Concrete structures present a number of unique design problems. Generally, only specialty firms, well versed in the peculiarities posed by such structures, are qualified to formulate a suitable design. It would be misleading to imply that these guidelines could somehow substitute for the requisite experience and judgment necessary to design a suitable concrete impounding structure." With regard to this statement in the Guidelines we offer the paragraph below.

A unique attribute of concrete dams is that they provide a basis for adaptation of more "risk-informed" design criteria. In high seismic hazard areas, earthquake loading typically governs the cross-sectional requirements of the structure. A risk-informed design involves setting the cross-section of the dam for a loading condition that is something less than the AEP with the expectation that there should be no to minimal damage to the structure during such an event. Using this cross-section configuration for the dam, potential loading up to and including the AEP condition are then analyzed, and the expected structure performance is evaluated. Tolerable deformations and post-earthquake stability with a suitable margin of safety must then be demonstrated. If deformations are small, and the post-earthquake stability indicates a very low potential for a catastrophic failure of the dam, the cross-section is adopted. If deformations are large, or if the post-earthquake stability of the structure is not acceptable, the cross-section of the dam is modified until acceptable performance is obtained.

Additional discussion of the design criteria and methodology for the Chehalis dam configurations are provided in a later section of this document.

2.1.2 Federal Requirements and Guidelines

Federal agencies have well established guidelines for evaluating the safety of concrete dams such as the RCC configuration proposed for the Chehalis project. The federal agencies that have established design criteria and guidelines include the U.S. Army Corps of Engineers (USACE), U.S. Bureau of Reclamation (USBR), the Federal Energy Regulatory Commission (FERC), and the Natural Resources Conservation Service (NRCS). Although there are some differences in the details of the federal agencies' guidelines, the general approach is relatively consistent and the agencies often refer to the guidelines developed by the other agencies. As noted above, the intent of the designs under the current scope of work will be to satisfy design criteria of all federal agencies where it is reasonable to do so; however, when in question, the USACE guidance will be given precedence in deference to possible future review/involvement with the project by the Corps.

2.1.3 USACE

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

- EM 1110-2-2200, Gravity Dam Design, June 1995
- EM 1110-2-2006, Roller-Compacted Concrete, January 2000
- EM 1110-2-2100, Stability Analysis of Concrete Structures, December 2005
- ER 1110-2-1156, Safety of Dams – Policy and Procedures, March 2014

From ER 1110-2-1156:

> *"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 for USACE criteria require written concurrence from the USACE SDO."*

The USACE is in the process of updating some of its guidance documents, including several of those listed above, to incorporate more risk-informed criteria and methodology.

2.1.4 USBR

In addition to publishing numerous dam design manuals and guidelines, the USBR has been a leader in developing and incorporating risk-informed dam safety and design methods and guidelines.

- Design Standard No. 2 – Concrete Dams
- Design Standard No. 3 – Water Conveyance and Fish Facilities
- Design of Small Dams, 1987

- Design of Gravity Dams, 1976
- Roller-Compacted Concrete, Design and Construction Considerations for Hydraulic Structures, 2005
- Interim -Dam Safety Public Protection Guidelines, A Risk Framework to Support Dam Safety Decision-Making, August 2011
- Reclamation Consequence Estimating Methodology, Interim, Guidelines for Estimating Life Loss for Dam Safety Risk Analysis, February 2014
- USBR and USACE, Dam Safety Risk Analysis Best Practices Training Manual, 2012

2.1.5 FERC

The FERC regulates 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 highly evolved methodology and guidance on the design and safety evaluation of concrete structures: FERC Guidelines, Chapter 3, Gravity Dams, revised October 2002

2.2 Risk-Based Approach to Design and Construction

There is a growing body of evidence under both state and federal dam safety programs that criteria-based design and safety evaluation programs for dams in the United States and around the world can lead to significant gaps and deficiencies. Hence, there has been a strong movement toward incorporation of risk-informed evaluation and design approaches into dam safety programs. This approach is most notable in the major federal agencies including the USBR, USACE, FERC, and the Tennessee Valley Authority (TVA). The Federal Emergency Management Agency (FEMA) recently issued risk-informed evaluation guidance that all federal agencies are required to incorporate into their dam safety programs. The FEMA, USACE, and USBR guidance is as follows:

- FEMA P-1025, Federal Guidelines for Dam Safety Risk Management, January 2015
- USACE, ER 1110-2-1156, Safety of Dams – Policy and Procedures, March 2014
- USBR, Interim Dam Safety Public Protection Guidelines, August 2011

FEMA and USACE leadership are in the process of updating the overarching Federal Guidelines for Dam Safety (FEMA – 93). The update will incorporate a wide range of advances in dam safety engineering across the United States since this original guideline was developed in 1978.

State agencies including the Washington DSO are making important advances to incorporate "risk-informed" methods into their programs. This trend will likely continue to develop over the next 5 to 10 years as the Chehalis project completes the planning and design phases.

Due to current trends toward risk-based evaluation, the updated designs for the Chehalis FRO and FRFA dam configurations will be based on a combination of DSO and federal guidelines that include both "standard" and "risk-informed" methods. In general, a combination of the state AEP requirements outlined above and the federal "risk-informed" guidelines will be considered. The FEMA dam safety risk guideline chart (also known as the "f-N" chart) is presented in Figure 2-1. This chart, and other, similar charts, is used by federal agencies including the USACE to help manage their dam safety programs that use risk to inform decisions on existing dams. The annual failure probability (AFP) is plotted on the vertical access of Figure 2-1, and life loss on the horizontal axis. For illustrative purposes, we have superimposed the state AEP safety requirement on the federal f-N chart. Because it is important to help illustrate the concepts, we have made some key assumptions including AEP equals AFP (which might be the case for embankment dams, where flood exceedance will likely lead to failure of the structure) and estimated life loss ranges from single digits up to just under 100. The green box on the diagram illustrates the DSO AEP requirements based on the estimated consequence rating summarized in Table 2-1 of Section 2.1.1, above.

To use a risk-informed approach, estimates of the annual failure probability and life loss consequences are made for each potential failure mode. The following overall risk equation to quantify annualized life loss will be used in the design evaluations:

Risk (Annualized Life Loss) = PL * PF * C

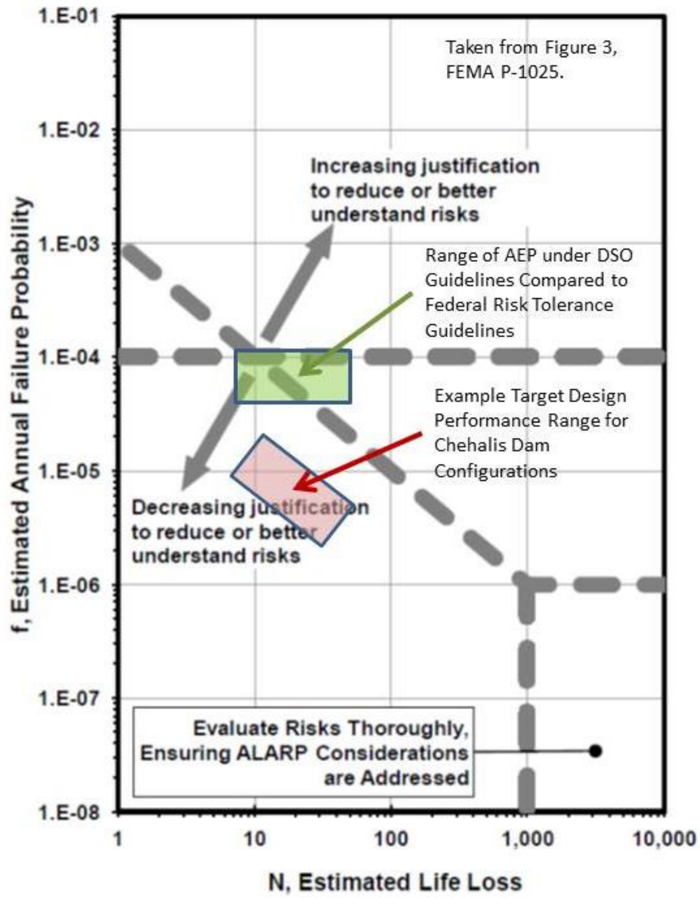
Where: PL = Probability of the Loading Condition

PF = Probability of Failure Given the Loading Condition (System Response)

C = Consequences of Failure in terms of Potential for Loss of Life

For new dams (and major modifications to existing dams) a typical target level of risk is about one order of magnitude below the annualized life loss guideline value of 1x10-3 lives/year, as shown on Figure 2-1 for each individual potential failure mode with some added margin of safety. For example, design of the dam cross-section will be confirmed by evaluating a range of potential dam and dam foundation failure modes and achieving an estimated total annual AFP of about 0.5 to 1 order of magnitude below the sloping dashed line on the figure. This is illustrated by the red shaded box on the figure. The red box is sloped parallel to the annualized life loss line (societal risk line) because the design risk level would be lower as the life loss consequence estimate increases.

Figure 2-1
Summary of "Risk-Informed" Criteria for Chehalis Project



2.3 Conceptual Dam Design Approach

The concept-level design of the dam cross-section was developed based on a risk-informed design approach. This approach is intended to provide a safe but cost-effective configuration given the very large range of, and uncertainties associated with, potential seismic loadings that may occur at the Chehalis dam site. The representative maximum non-overflow and spillway overflow sections of the FRFA were selected for structural analyses because these sections represent the highest structural height and largest normal reservoir loading conditions being considered for the site. A cross-section meeting the risk-informed criteria for these maximum sections will be capable of seismic performance equal to or better than that of lower dam and section heights.

Overall, the risk-informed design approach used for the Chehalis dam includes four separate but related design criteria A through D, generally outlined as follows:

A. **Elastic Structural Response:** All normal and flood loads (including PMF), and Earthquake loads with estimated recurrence intervals of 500 to 2,500 years
B. **Non-linear but Limited Damage Response:** Earthquake loads with estimated recurrence intervals of between 5,000 and 10,000 years
C. **Non-linear Limited to Moderate Damage Response**: Earthquake loads with estimated recurrence intervals of up to 25,000 years
D. **Post-earthquake Stability:** Dam cross-section would have a factor of safety greater than about 1.1 under conservative (degraded) shear strength properties assumptions, and uplift pressures so that catastrophic failure of the dam would not occur under the full range of potential seismic loadings including earthquakes with estimated recurrence intervals up to the 25,000-year event

This approach represents an extension of the risk-evaluation criteria for large concrete dams under the new federal guidelines for dam safety (USBR Public Protection Guidelines, 2011, and USACE, EM 1110-2-1156). This approach is currently being used for a new RCC dam at the Scoggins Dam site in northwest Oregon under USBR Public Protection Guidelines (HDR, 2016). The recently completed appraisal level designs (USBR design detail and cost estimate accuracy classification) for the Scoggins Dam project have included a detailed quantitative risk analysis for a similar large new RCC dam where potential sliding failure modes and failure consequences have been used to verify the suitability of a similar risk-based four-part design criteria used for cross-section development.

2.4 Evaluation Steps for Cross-section Verification

The overall rationale for the risk-based design approach summarized in the preceding section is that the dam should remain operational with no damage for smaller return period seismic events, but that some progressively increasing but tolerable damage can be allowed for more remote earthquake events provided the dam continues to hold the reservoir and catastrophic failure is avoided. Allowing some damage for the more remote earthquake events typically results in substantial reduction in the cross-section requirements and associated costs for construction of the dam. Uncertainties associated with seismic loads, material properties, hydrodynamic and uplift pressures acting on the dam must be considered in the risk-based design.

Although no gravity dam damage is known to have resulted in a catastrophic loss of reservoir during or immediately after an earthquake, some minor to moderate damage has been reported for certain cases in the literature. Most of this damage was limited to the location of a sharp variation in the dam geometry. Damage has included opening of joints, minor displacements, development of small cracks, and increases in seepage. The most significant damage to a concrete dam occurred to the 70-foot-high Shih Kang Dam in Taiwan, where 29 feet of vertical differential movement and 23-foot horizontal offset occurred as a result of a fault rupture orthogonal to the dam axis. The fault rupture occurred in the foundation of a gated spillway section of the dam (Nuss et al, 2012).

There is no direct evidence of the presence of a fault in the foundation area of the proposed dam site. Hence, the adopted evaluation criteria are based not on potential fault rupture under the dam section but on estimated ground motion hazards in the general vicinity of the site.

To address the risk-based design criteria outline above, a stepwise process is required using multiple two-dimensional finite element models of the RCC dam as follows:

Step 1: The first step in the evaluation uses the finite element program SAP2000 and a response spectrum analysis method to assess the "elastic response" of the structure for the smaller earthquakes. The response of the dam is modeled for loadings from relatively frequent earthquakes beginning with the estimated 500-year recurrence interval event. When the response spectrum model indicates that tensile stresses in the heel area such as the concrete/bedrock interface of the dam likely exceed the dynamic tensile strength of the concrete, the dam is assumed to crack through the entire section of the dam, and analyses move to the second step summarized below. Using response spectrum analyses of the dam cross-section is a conservative approach to the verification of cross-section requirements because the response spectrum analyses only identify the maximum compression and tensile stresses that occur. The number of times that the maximum stress condition occurs and the process of crack propagation is not considered. More rigorous analyses completed during later stages of the design process will include actual time history evaluations and may allow for additional cross-section optimization associated with both the initiation and propagation of cracking.

Step 2: This step of the analysis uses an alternative finite element model developed with the EAGD-SLIDE computer program that is capable of simulating deformations along an assumed cracked surface within the base area of the dam or along the dam/foundation contact. This finite element model is analyzed with zero cohesion and friction only along the cracked surface and neglects any resistance along the sides of a monolith or other 3-dimensional influences on monolith cracking and deformations.

Overall, the section performance is considered acceptable when it meets all of the response criteria introduced in Section 2.3. The cross-section is modified by either decreasing or increasing the external slopes of the cross-section until a cross-section is identified that fulfills ***all*** these criteria:

A. **Normal loading, PMF loading and up to 500-year seismic event**: The section shall remain in the linear elastic range (i.e. stresses do not exceed the tensile capacity of the concrete) and the result of all forces acting above any horizontal plane through the dam intersects that plane inside the kern (the middle third).
B. **5,000-year seismic event**: Stresses are allowed to begin to exceed the tensile strength in the base of the dam or at the dam/foundation contact at the heel (located where the upstream slope meets the foundation contact), resulting in the potential for initiation and propagation of

cracking through the entire section. Assuming that the section fully cracks, acceptable and limited damage to the structure would occur that allows operation of the dam and reservoir after the earthquake. Damage at joints, grout curtain, and drains, along with intrusion of water in the damaged interface, is expected. Acceptable seismic deformations would be limited to less than 1 foot based on the assumption of an average degraded rock-concrete interface friction angle and drain performance reduced or eliminated in post-earthquake stability evaluations. The earthquake force resultant at the base is restricted to the limits of the base of structure in order to limit the foundation compression at the downstream toe, limit the rock-concrete interface damage, and allow the induced earthquake base crack to close after the earthquake. For this particular case, with the uplift pressure based on linear distribution of uplift from upstream to downstream and the estimated final degraded friction angle (associated with one foot sliding) the post-earthquake factor of safety should be greater than 1.5 to limit aftershock damage and allow safe operation during repair work that would be required.

C. **10,000-year seismic event:** Seismic deformations must be less than 2 feet for a 10,000-year return period earthquake. Such deformations would be considered limited to moderate and would result in a low to very low potential for release of water flow into the downstream channel that is no more than the maximum safe channel capacity.

D. **Extreme seismic event:** For extreme events such as a 25,000-year earthquake, damage along the base of the dam can possibly yield to full reservoir pressures across the entire base. For this case, the post-earthquake stability factor of safety is based on a condition of fully degraded shear strength along the entire cracked surface (residual strength), and uplift pressures equal to full reservoir pressure along the entire cracked surface. A minimum post-earthquake factor of safety greater than 1 must be demonstrated to reduce the possibility of a sudden release of the reservoir.

2.5 Cross-section Geometry and Computer Models

Two cross-sections through the FRFA dam were used in the structural analyses: a section through the main body of the dam (a non-overflow section) and a section through the overflow spillway. A plan view of the FRFA-configuration RCC dam configuration evaluated is provided in Figure 1. The locations of the two representative maximum cross-sections used for structural evaluations and analyses are shown in Figure 1, and the cross-sections are illustrated in Figures 2 and 3 for the non-overflow and overflow (spillway) sections, respectively.

The computer program SAP2000 was used to perform preliminary response spectra analyses of the two cross-sections selected for evaluation. Modeling results for the non-overflow section were checked against a simplified spreadsheet model (Appendix A) based on Fenves and Chopra (1987), and hand calculations (Appendix B) estimating maximum stresses during earthquake loading, overturning, and sliding stability.

Nonlinear analyses of each cross-section were performed using the program EAGD-SLIDE (Chavez and Fenves, 1994). This program allows modeling the compressible water-foundation-structure dynamic response with base sliding for combined loading of hydrostatic pressures, uplift pressure distributions, and horizontal and vertical earthquake input motions. The program internally accounts for hydrostatic forces from given reservoir and tailwater elevations, uplift, hydrodynamic forces with compressible water model, damping, and flexibility of the foundation and structure along with the dynamic response of the dam.

2.6 Material Properties

To complete the analyses of the dam cross-section described in the preceding sections, knowledge of the material properties for bedrock materials beneath the dam, RCC properties within the dam, and the contact between the bedrock and dam are required. A summary of the materials properties used in the analyses is presented in Table 1. The subsections below provide additional information on the derivation of these properties.

Table 2-2
Summary of Material Properties Used in Structural Analyses

PROPERTY	RCC/LIFT JOINTS	ROCK /ROCK-CONCRETE INTERFACE
Unit weight (pounds per cubic foot - pcf)	150	165 (Rock)
Unconfined compressive strength (pounds per square inch - psi)	3,000 to 4,000	N/A*
Direct tensile strength (psi)	150 (intact RCC)	0
Apparent static tensile strength (psi)	270 (lift joint)	0
Actual dynamic tensile strength (psi)	300 (lift joint)	0
Apparent dynamic tensile strength (psi)	400 (lift joint)	0
Dynamic mod. of elasticity (psi)	3.24×10^6	2.61×10^6 (weighted average)
Poisson's ratio	0.20	0.33
Hysteresis damping (percent)	0	1
Intact bonded friction angle (degrees)	65	45
Friction angle for cracked concrete, but no sliding has occurred (degrees) *	55	55
Average friction angle of rock-concrete interface or RCC lift joints after for sliding of less than 1 foot (degrees)	45	45
Average friction angle of rock-concrete interface or RCC lift joints after for sliding between one to two feet (degrees)	45	45
Residual friction angle of smoothed sliding planes (for sliding of more than 2 feet) (degrees) **	40	35

Notes:
* Cracked but with no sliding (limited the force resultant location to the base but no sliding during earthquake).

** No cohesion was assumed along the lift joints, concrete to rock interface, of bedding planes in the foundation.

2.6.1 Bedrock Material Properties

The engineering properties of the bedrock beneath the dam were estimated based on site characterization work (borings, surface, and downhole geophysical testing) completed to date for this project (Chehalis Dam Site Characterization Report, HDR, 2016), and the estimated depth of excavation that will be required at the maximum cross-section locations developed as part of other geotechnical engineering evaluations of the dam site (Chehalis Dam Foundation Design Technical Memorandum, HDR, 2016). Bedrock properties required for the structural analyses include 1) rock modulus for the bedrock profile beneath the dam estimated from a Rock Mass Rating (RMR) evaluation of core samples, 2) Poisson's ratio, and 3) an estimate of the rock shear strength (friction angle and cohesion) along any potential sliding surfaces.

Three deformation moduli were estimated and used in the analyses based on the RMR results. RMR analysis performed as part of the site characterization is provided in Appendix C. The weighted average value was considered for the majority of analyses, and the upper and lower bound values were used to confirm that the dam performance would not be significantly affected by the modulus assumptions. The weighted average value represents what is believed to be the actual bedrock profile beneath the dam in the vicinity of the maximum section locations.

lower bound modulus (siltstone/claystone only);	$E_D = 1.39 \times 10^6$ psi
upper bound modulus (basalt only);	$E_D = 3.15 \times 10^6$ psi
weighted average (combined basalt/siltstone profile);	$E_D = 2.61 \times 10^6$ psi

An average Poisson ratio of 0.33 was assumed for the bedrock profile of any of the material profiles that may be present.

A strength degradation model was included in our evaluation for both the bedrock/concrete contact and the RCC materials. Strength degradation was considered for the RCC/bedrock contact after cracking was assumed to occur and sliding could develop along this cracked surface. Estimates of the shear strength along this surface were based on the fundamental strength of the rock and some component of micro and macro roughness along the crack surface. Large irregularities or undulations of the rock surface that typically exist along the dam/foundation contact were neglected. The bonded peak strength of the contact was assumed to be 65 degrees with zero cohesion based on published results of laboratory tests on a concrete/sandstone contact (Electric Power Research Institute [EPRI], 1992). After a crack develops along the entire contact of the dam and foundation, the shear strength was progressively decreased from 55 degrees (full crack but no sliding) to 35 degrees (sliding of 2 feet or more).

2.6.2 RCC Material Properties

The RCC dam would be constructed with modern mix design and construction methods using a medium to high cementitious cement content (International Commission on Large Dams [ICOLD], 2013 and 2014) as summarized below:

- Medium-cementitious RCC (MCRCC); cementitious content (cement and fly ash/pozzolan) between 100 and 150 kilograms per cubic meter (kg/m^3) (170 and 250 pounds per cubic yard [lbs/yd^3])
- High-cementitious RCC (HCRCC); cementitious content > 150 kg/m^3 (>250 lbs/yd^3)

RCC materials with this range of cementitious material will provide adequate shear strength and tensile strength and also allow for the use of a grout-enriched method of facing system development for both the upstream and downstream exterior faces of the dam. Aggregate materials obtained from a local quarry would be used to produce the RCC materials.

Unconfined Compressive Strength: A mix design program with corresponding laboratory testing has not been completed for the RCC materials at this time. Typical properties for the RCC have been based on USACE EM 1110-2-2006 (2006). The mix would be prepared with a set retarder, and mortar would be applied if needed between lifts to achieve a high degree of lift surface bonding. The 1-year compressive strength would be in the range of 3,000 to 4,000.

Tensile Strength: The estimated static direct tensile strength for intact RCC materials having a compressive strength (f_c') of 3,000 psi would be 177 psi according to the equation:

$$f_t = 0.85 f_c'^{2/3}$$ (Raphael, 1984)

Tensile strengths used in the structural analysis of the dam response to seismic loading were estimated as follows:

1. A more conservative static tensile strength value of 150 psi (5 percent of compressive strength) was assumed.
2. Ratios of 2.3/0.85, 2.6/0.85, and 3.4/0.85, respectively, were applied to the static tensile strength to estimate the apparent tensile strength, dynamic tensile strength, and apparent dynamic tensile strength as proposed by Raphael.
3. An additional factor of 2/3 was applied to the intact RCC tensile strengths computed above to estimate the tensile strength of lift joints.

The various estimated RCC tensile strengths are shown in Table 1.

Lift Joint Shear Strength: Information on the left side of Figure 4 shows the peak shear strength of bonded lift joints from laboratory tests from numerous concrete dams (EPRI, 1992). According to this figure, the lower range of friction angle of an intact lift joint before loss of bond can be assumed to be 65 degrees (zero cohesion). Information on the right side of Figure 4 shows that the lower bound of residual shear strength of concrete lift joints can be assumed to be about 45 degrees. According to the American Concrete Institute (ACI), the peak friction coefficient for an intact cracked concrete is about 1.4 (friction angle of about 55 degrees). Finally, 40 degrees was used as a worst-case estimate of a lower bound residual strength of RCC lift joint after 2 or more feet of displacement along a joint surface.

Modulus: The RCC modulus was estimated based on an ACI equation:

$$E = 57000\sqrt{f_c'}$$

2.7 Other Loading Condition Considerations

The following subsections provide a summary of the other loading conditions evaluated in the analysis of the FRFA RCC dam section:

2.7.1 Uplift Pressures on the Base of the Dam

A foundation drain efficiency of 50.0% was assumed based on design guidance of the USACE (EM 1110-2-2100, 2005). The USBR design criteria allows consideration of a maximum suggested drain efficiency (USBR, 1976); however, this was not included in our analyses at this time. The efficiency of foundation drains extending from the gallery into the foundation will be a recurring design consideration as the project progresses. Key design considerations include the spacing and orientation of the drains and the potential for long-term drainage degradation due to lack of maintenance.

The uplift conditions consisted of “normal” uplift, calculated as shown in USACE EM 1110-2-2200, Gravity Dam Design, 1995, for the geometry of the dam provided in Figure 2. Full uplift was calculated assuming a linear distribution from full head at the upstream heel to the downstream tailwater condition, and extreme uplift was calculated assuming full upstream head exists all the way to the downstream toe.

A section with zero drain efficiency after a large earthquake and significant sliding that could disrupt the function of the foundation drains was also evaluated as part of post-earthquake stability analyses of the 10,000-year or larger earthquake event.

2.7.2 Hydrostatic Load

A normal maximum operating pool elevation of 628 feet and a tailwater elevation of 415 feet were used in the analyses for the overflow and non-overflow sections.

2.7.3 Hydrodynamic Load

Hydrodynamic forces from the reservoir pool were considered using the Westergaard added mass formulation with the normal maximum operating pool. This formulation is typically used for concrete structures with vertical to near vertical faces subject to the hydrodynamic forces, similar to the proposed configurations of the FRO and FRFA dams.

2.7.4 Silt Pressure

The buildup of sediment against the upstream face of the dam is expected to be minimal. Therefore, silt pressures acting on the upstream face of the dam were not used in the structural analysis models.

2.7.5 Earthquake (Extreme) Load

Site-specific seismic hazard design criteria have been developed for the site (Shannon and Wilson, September 2015). Response spectrum analyses were initially performed to verify the initial cross-section properties of the dam. Subsequently, time history analyses for Cascadia Subduction Zone (CSZ) events with 500, 2,500, 5,000 and 10,000 recurrence intervals were evaluated along with a crustal fault motion maximum credible earthquake (MCE) event. The event chosen for analysis was the Curico EW motion, which had the highest peak ground acceleration (PGA) and the most excursions close to the PGA. Scaling factors provided by Shannon and Wilson (September 2015) were used to adjust the base 2,500-year return period motion for events with return periods lower and higher than the 2,500-year return period base motion. Figures 5 to 9 show the response spectrum developed for the site and the time histories and associated parameters used in our structural analyses. Figures 5 to 9 are based on the 2,500-year return period earthquake time histories. Scaling factors were used to represent the ground motions and response spectra at different return periods. Response spectra were used with the SAP2000 software. Time histories were used with the EAGD-SLIDE cracked base model to estimate sliding deformations.

2.7.6 Flood (Unusual) Hydraulic Loading

Probable maximum flood (PMF) loading was considered in the analysis of the FRFA configuration. The routed PMF results in an approximate 81- foot difference between the normal maximum operating pool elevation (628 feet) and the estimated maximum routed PMF elevation (709 feet).

3 ANALYSIS RESULTS

The following sections provide a summary of results for the following analyses:

- Normal and PMF Loading Analyses
- Earthquake Loading Analyses
- Earthquake Sliding Analyses
- Post-Earthquake Uplift Analyses

3.1 Normal and PMF Loading Analysis

Plots from the response spectrum analysis results showing estimated maximum stresses within the dam sections under reservoir loading at the maximum normal operating pool level are shown in Figures 10 and 11 and under PMF loading of the non-overflow section on Figure 12. The analyses results of loadings at the maximum normal operating pool and the PMF pool elevations indicate that the dam contact at the foundation as well as all other portions of the dam remain in compression and the resultant force remains in the middle third of the base of the dam. Hence these cross-sections meet the criteria A requirements summarized in Sections 2.3 and 2.4.

3.2 Earthquake Loading Analysis (Elastic Response)

The results of SAP2000 response spectra analyses for earthquakes with estimated return periods of 500, 2,500, 5,000, and 10,000 years are presented on Figures 13 to 17, respectively, for the non-overflow section and on Figures 18 through 22 for the spillway overflow section. A summary of the maximum estimated tensile stresses in the heel area of the dam is presented in Table 3.1. The effect of a reduced foundation modulus was also evaluated in the analyses. A reduction in the foundation modulus reduces the peak stresses in the upstream heel and downstream toe areas and also results in a smaller area of concentrated stresses. Figures 23 and 24 show the stresses in the dam as a result of reducing the deformation modulus from 2.61×10^6 to 1.39×10^6 psi.

Table 3-1
Summary of Maximum Estimated Tensile Stresses in the Heel of the FRFA Maximum Non-Overflow and Spillway Overflow Dam Sections

EARTHQUAKE RETURN PERIOD	NON-OVERFLOW SECTION MAXIMUM TENSILE STRESS (PSI)	SPILLWAY OVERFLOW SECTION MAXIMUM TENSILE STRESS (PSI)	COMMENTS1
500-year	122.1	138.0	0% of Base/0% of Base
2500-year	447.3	484.2	6% of Base/4% of Base
5000-year	626.3	674.0	7% of Base/8% of Base
10,000-year	829.2	888.9	9% of Base/11% of Base
25,000-year	1,446.5	1,561.4	44 % of Base/24% of Base
2500-year	57.7	Not run	0% of base reduced foundation modulus
10,000-year	108.7	Not run	reduced foundation modulus

Note:
1. Percentage of base with tensile stress of greater than 400 psi for non-overflow and overflow sections

These analyses suggest that the tensile strength of the RCC would be exceeded, and base cracking of the dam would begin to occur at both the upstream heel and downstream toe of the cross-sections at about the 2,500-year event, consistent with criteria B summarized in Section 2.3. The elastic response of the dam would be expected for all earthquake loading with estimated return periods of less than 5,000 years.

Modal shapes associated with elastic response of the non-overflow section are shown on Figures 25 to 34. These shapes were used to further assess the types of structural displacements occurring at different frequencies. The scale for Figures 25 to 34 is exaggerated to illustrate the displacement characteristics for the different frequencies within the response spectrum. Figures 35 to 39 show the magnitude of elastic displacement expected for each of the return periods. These displacements are based on linear and elastic material property assumptions without regard to the potential for cracking. As noted above, cracking would begin to occur for loading conditions associated with the 5,000-year event and the mode of deformation would change to a nonlinear sliding response along an assumed crack surface (as described in the next section of this report).

3.3 Earthquake Sliding Analysis (Nonlinear Response Analysis)

Nonlinear time-history analyses of sliding were performed using EAGD-SLIDE (Earthquake Analysis of Concrete Gravity Dams including Base Sliding), Chavez and Fenves (1994). A general schematic of the model as analyzed by EAGD-SLIDE is shown on Figure 40. Analyses were performed for a range of foundation modulus; a lower bound modulus of 1.39 x 10^6 psi, assuming a siltstone foundation; an upper bound modulus of 3.15 x 10^6 psi, assuming a basalt foundation; and a weighted average modulus of 2.61 x 10^6 psi, assuming a relative mix of both basalt and siltstone in the foundation, which was based on RMR analysis performed as part of the site characterization (see Appendix C). Friction angles between the bedrock and the dam were varied, with angles of 35, 45, and 55 degrees considered in the analysis.

In addition to consideration of friction angle degradation for the foundation/dam contact, and variable foundation modulus (flexible foundation condition), the foundation condition was also considered rigid. This assumption results in the largest estimated sliding displacements and represents a highly unlikely bounding condition that could be considered in a more rigorous quantitative risk analysis.

The analyses were performed for one-time history record representing the CSZ source. This record has been spectrally matched to the 0.2-second conditional mean spectrum (CMS) time history and is expected to produce the maximum number of PGA excursions that would cause sliding.

Figures 6 and 7 show the horizontal component and the time history and associated plots. Figures 8 and 9 show the vertical time history and associated plots.

To further identify the maximum potential sliding response from the time history selected, the time history was run for the following conditions:

- Normal polarity in horizontal and vertical directions
- Normal polarity horizontal coupled with reverse polarity vertical
- Reverse polarity horizontal and normal polarity vertical
- Reverse polarity for both horizontal and vertical

The analyses results showed that the normal polarity horizontal coupled with the reverse polarity vertical motions yielded the largest displacements when all other variables were held constant. An example plot of the different polarity runs is shown in Figure 41.

A summary of the maximum estimated sliding displacements from the analyses is presented in Tables 3.2 and 3.3. It should be noted that an analysis of the 2,500-year return period earthquake was included in the analysis although crack for this event is highly unlikely based on the results of the response spectrum analyses described in the preceding section.

Figures 41 to 47 show the estimated sliding response for different assumptions in the foundation modulus, earthquake return period, and rock/dam interface friction angle. Figure 41 shows the response of the dam with the assumption of a rigid foundation and a 45-degree (base assumption) friction angle with the return period varied from 500 to 25,000 years. Figures 42 to 47 show the response of the dam to the 10,000-year return period motion in response to changing of the base friction angle. Each figure also shows the response based on the assumption of a rigid foundation, the moduli from the siltstone, basalt, and the weighted average of the basalt/siltstone. The flexible foundation assumption results in lower estimated displacements. Although the rigid foundation assumption results in much higher displacement estimates, the higher displacements still meet criteria.

Table 3-2
Sliding Displacements for Varying Friction Angles and EQ Return Periods, Flexible Foundation Assumption

SLIDING DISPLACEMENT (FEET) FOR RETURN PERIOD[1]					
FRICTION ANGLE	500-YR	2,500-YR	5,000-YR	10,000-YR	25,000-YR
35	0.00	0.31	1.12	2.73	6[2]
45	0.00	0.00	0.08	0.53	2.17
55	0.00	0.00	0.00	0.02	0.483

Notes:
1. Flexible foundation, weighted average of siltstone/basalt
2. Estimate-Calculation won't complete

Table 3-3
Estimated Sliding Displacements for Varying Friction Angles and EQ Return Periods, Rigid Foundation Assumption

SLIDING DISPLACEMENT (FEET) FOR RETURN PERIOD[1]					
FRICTION ANGLE	500-YR	2,500-YR	5,000-YR	10,000-YR	25,000-YR
35	0.08	1.32	2.73	4.91	9.60
45	0.00	0.42	1.07	2.20	4.66
55	0.00	0.07	0.34	0.86	2.20

Note:
1. Rigid foundation, flexible foundation has less displacement

3.4 Post-Earthquake Factor of Safety

Post-earthquake factor of safety was calculated for three different uplift conditions and for friction angles that ranged from 30 to 55 degrees in 5-degree increments. The uplift conditions consisted of "normal" uplift, calculated as shown in USACE EM 1110-2-2200, Gravity Dam Design, for the geometry of the dam provided in Figure 2. Full uplift was calculated assuming a linear distribution from full head at the upstream heel to the downstream tailwater condition, and extreme uplift was calculated assuming full upstream head exists all the way to the downstream toe. For all but the 30- and 35-degree friction angles for the extreme uplift condition, the factor of safety was above 1.5, and even for the lower friction angles and the extreme uplift condition the factor of safety remained above 1.0.

Table 3-4

Estimated Post Earthquake Factor of Safety for Varying Friction Angle and Uplift Pressure Distribution Assumptions

POST EARTHQUAKE FACTOR OF SAFETY			
FRICTION ANGLE	NORMAL UPLIFT	FULL UPLIFT	EXTREME UPLIFT[1]
30	2.08	1.77	1.06
35	2.52	2.15	1.28
40	3.02	2.57	1.54
45	3.60	3.06	1.83
50	4.29	3.65	2.18
55	5.14	4.38	2.62

Note:
1. See definitions of uplift conditions above

4 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the preliminary analyses detailed in Section 2, the following conclusions and recommendations should be considered for the proposed RCC dam cross-section:

- The preliminary results show that a cross-section for the RCC dam meeting the criteria set forth in Sections 2.1 and 2.3 of this document has been developed.
- The diameter of the drain holes should be sized to allow for the anticipated displacements while retaining a minimum level of efficiency to reduce the uplift potential for the 10,000-year recurrence interval event.
- The base of the dam shows minimal tensile stresses above the apparent dynamic tensile stress of 400 psi for the 2,500-year return period, indicating essentially elastic response at this level. At the 5,000-year return period the tensile stresses are higher than for the 2,500-year return period; however the percentage of the base that exceeds the 400 psi apparent dynamic tensile stress is nearly the same. Hence elastic response of the structure is anticipated for all seismic loadings up to a 5000-year earthquake event
- The sliding displacements for both a “rigid” and “flexible” foundation condition have been estimated. The “rigid” condition should be considered an upper bound estimate of displacements whereas the “flexible” foundation condition should be considered the best estimate of sliding displacements. Estimated displacements for a flexible foundation condition and earthquake events larger than a 5000-year earthquake meet the displacement criteria outline in Section 2.4. This range of displacements identified for both analysis conditions could be considered in a quantitative risk analyses.
- Further refinement of the dam cross-section with a representative nonlinear time-history analysis of a three dimensional (3D) model of the dam should be developed and used during preliminary design to verify that the dam cross-section meets the criteria.

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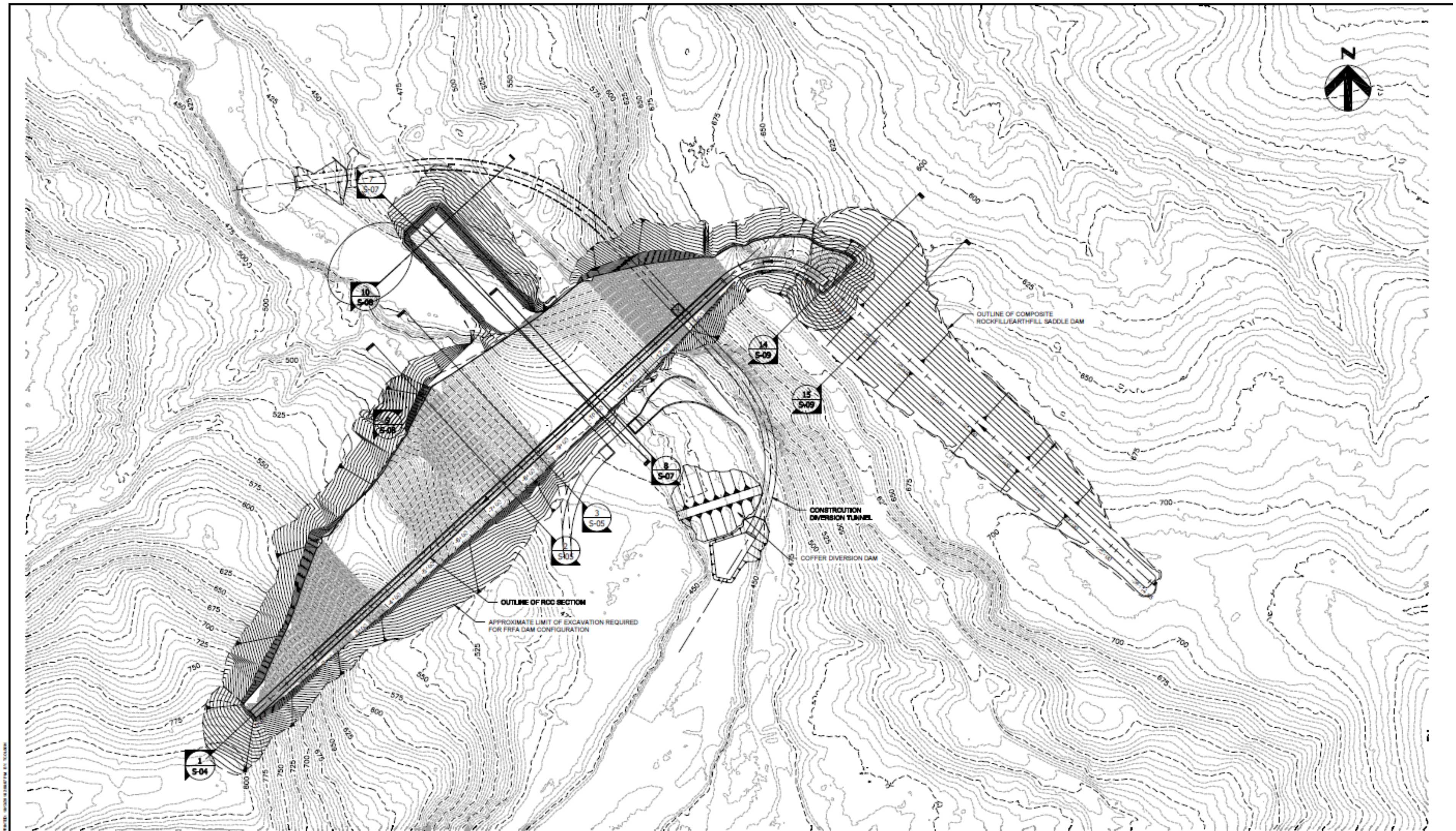
Figures



DAM EXCAVATION PLAN

HDR	PROJECT NO.	10026522	Plan View of Dam		FIGURE
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	DRAWN BY:	STA			
	CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
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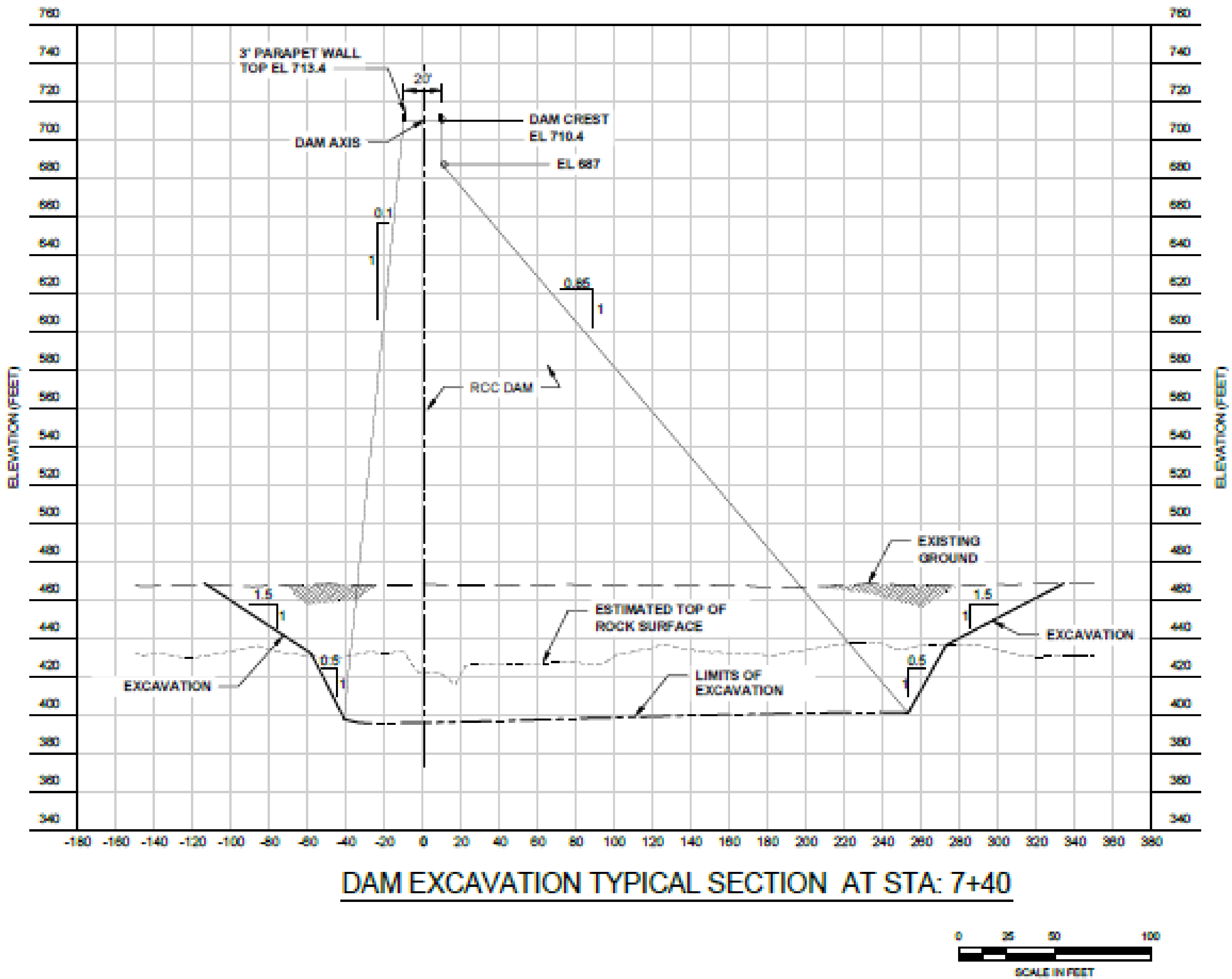

DAM EXCAVATION TYPICAL SECTION AT STA: 7+40

PROJECT NO.	10026522	Non-Overflow Section Schematic		FIGURE
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DRAWN BY:	STA			
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
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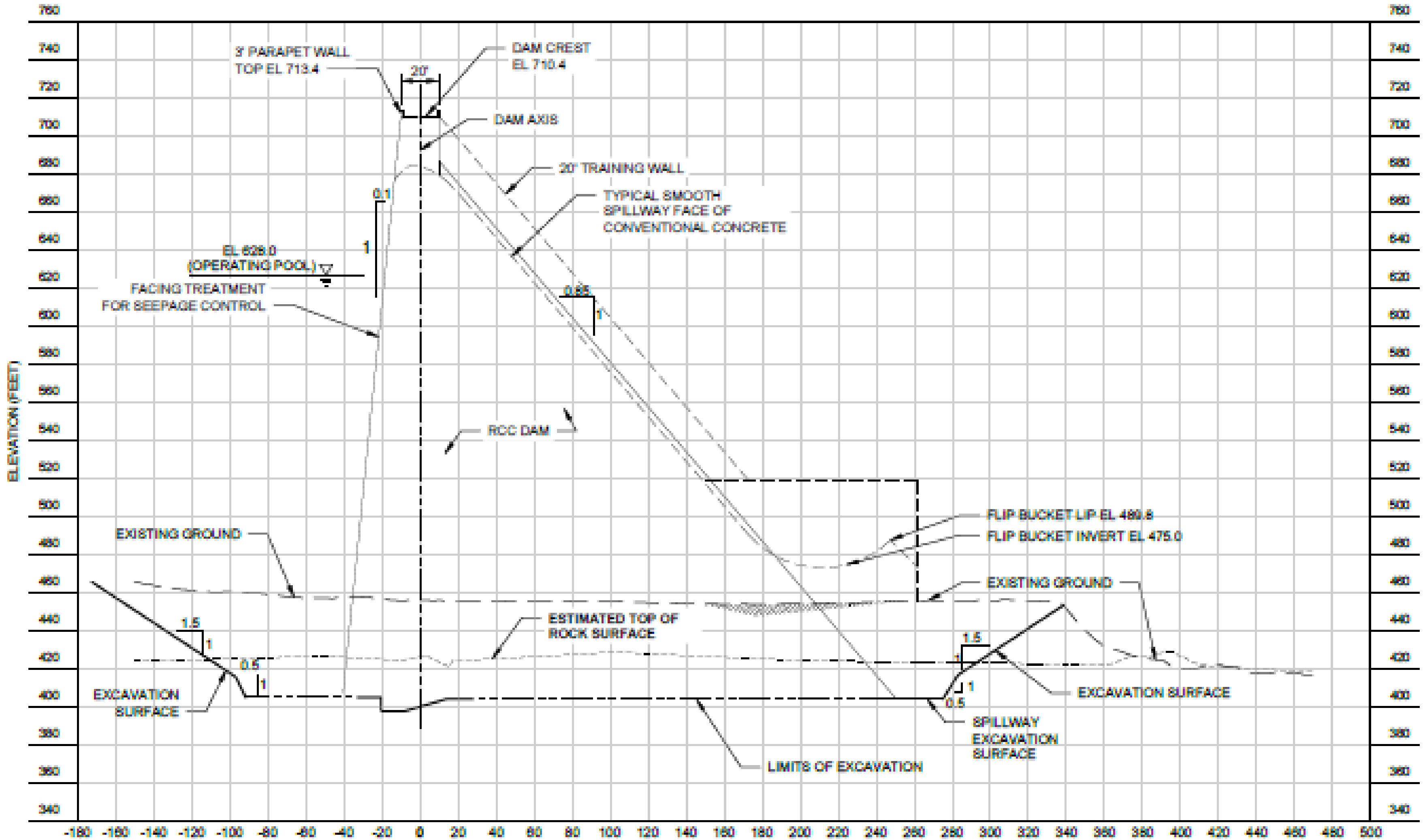

DAM EXCAVATION TYPICAL SECTION AT SPILLWAY STA: 9+20

HDR

PROJECT NO. 10026522	Overflow Section Schematic			FIGURE
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DRAWN BY: STA				
CHECKED BY: FA				
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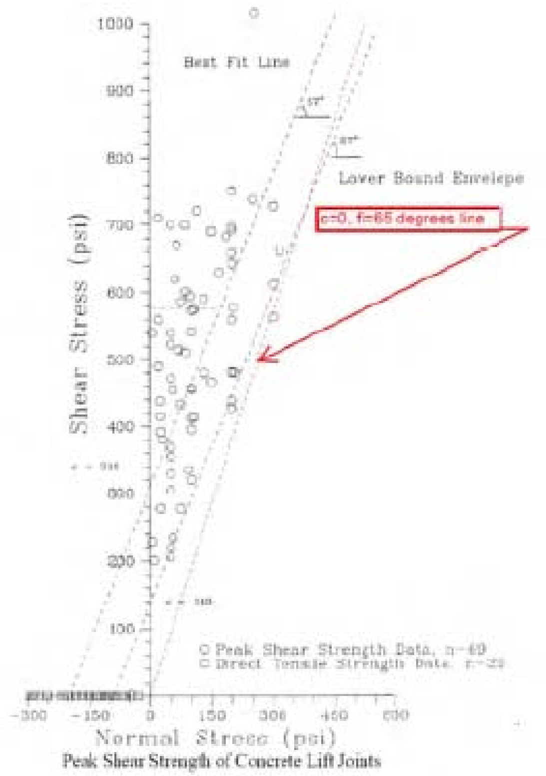
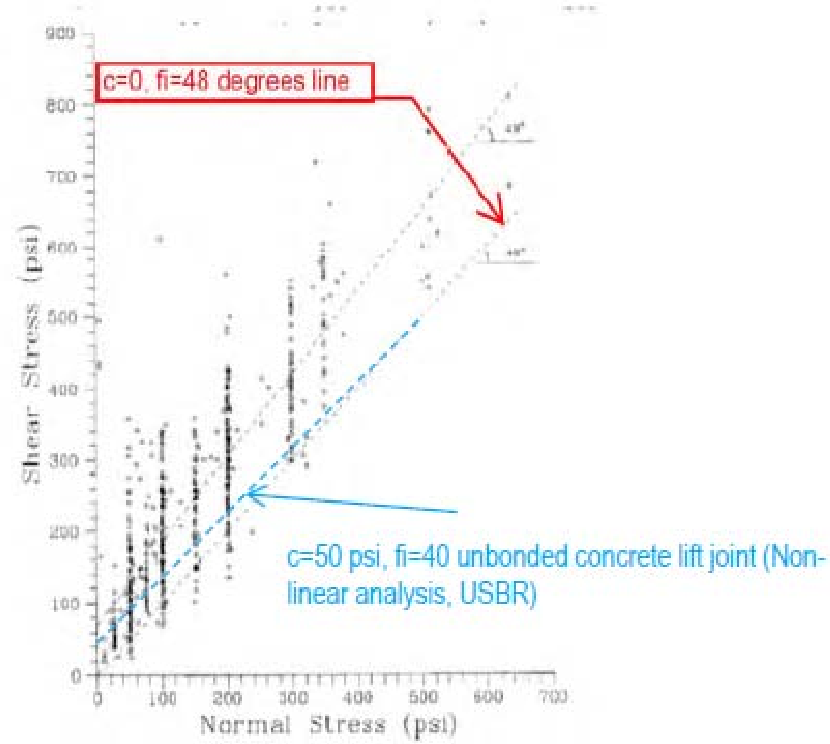


Peak Shear Strength of Concrete Lift Joints



Residual Shear strength of Concrete lift joints = Peak shear strength of unbonded lift joints (EPRI)

Source: Concrete lift joints
Ref. (EPRI 1992)

PROJECT NO.	10026522	
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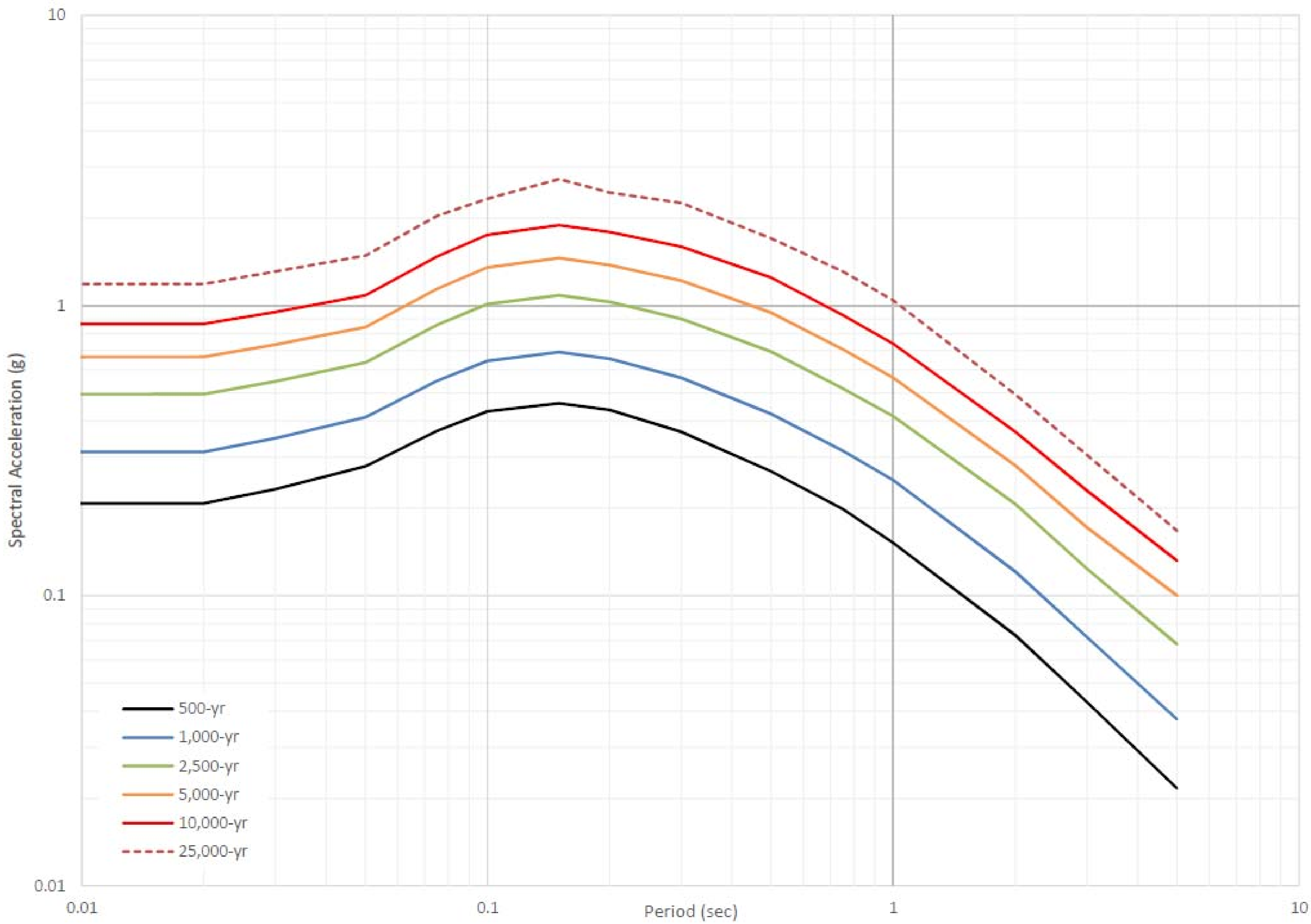
Strength of Concrete Lift Joints

CHEHALIS DAM
PE ELL, WASHINGTON

ORIGINATOR: STA
APPROVED BY: KAF/KM
DRAWING CATEGORY:

FIGURE 4



HDR	PROJECT NO. 10026522	Horizontal Response Spectra Plot		FIGURE
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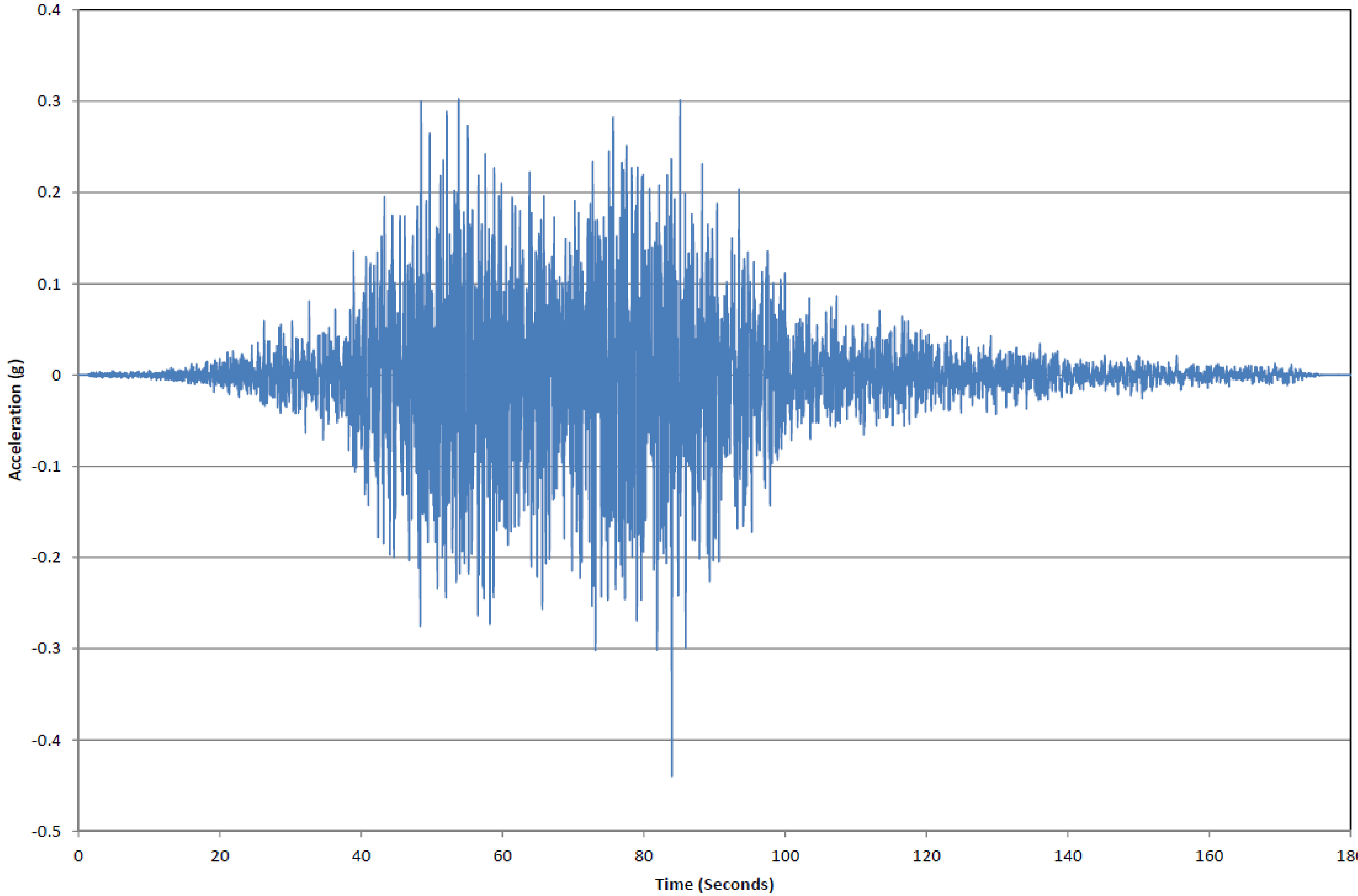

2,500—Year Return Period

PROJECT NO. 10026522	Curico EW Horizontal Acceleration Record	FIGURE
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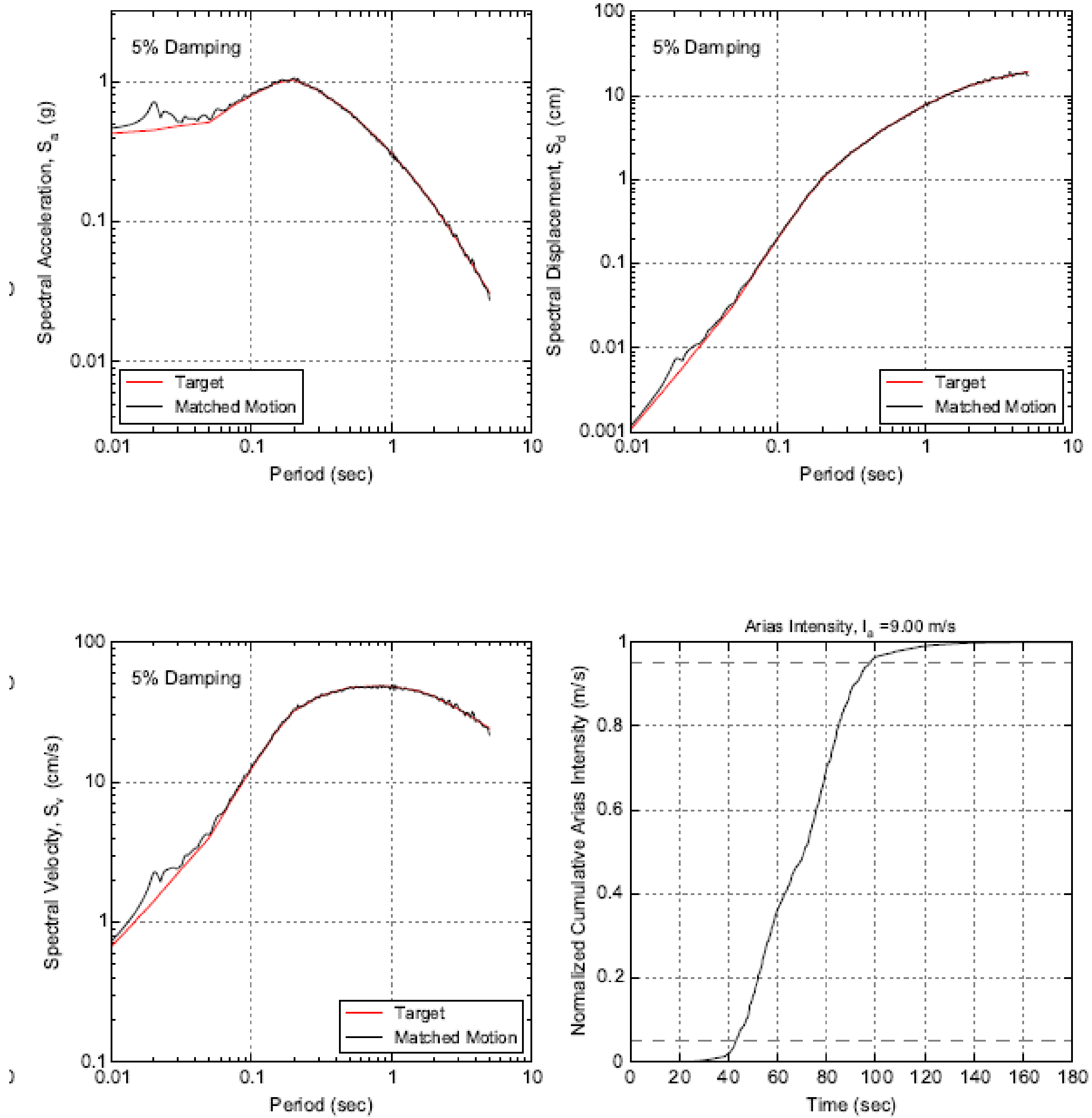
FILE NAME:		



2,500—Year Return Period

PROJECT NO.	10026522	
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FILE NAME:		

Curico EW Horizontal Acceleration Record

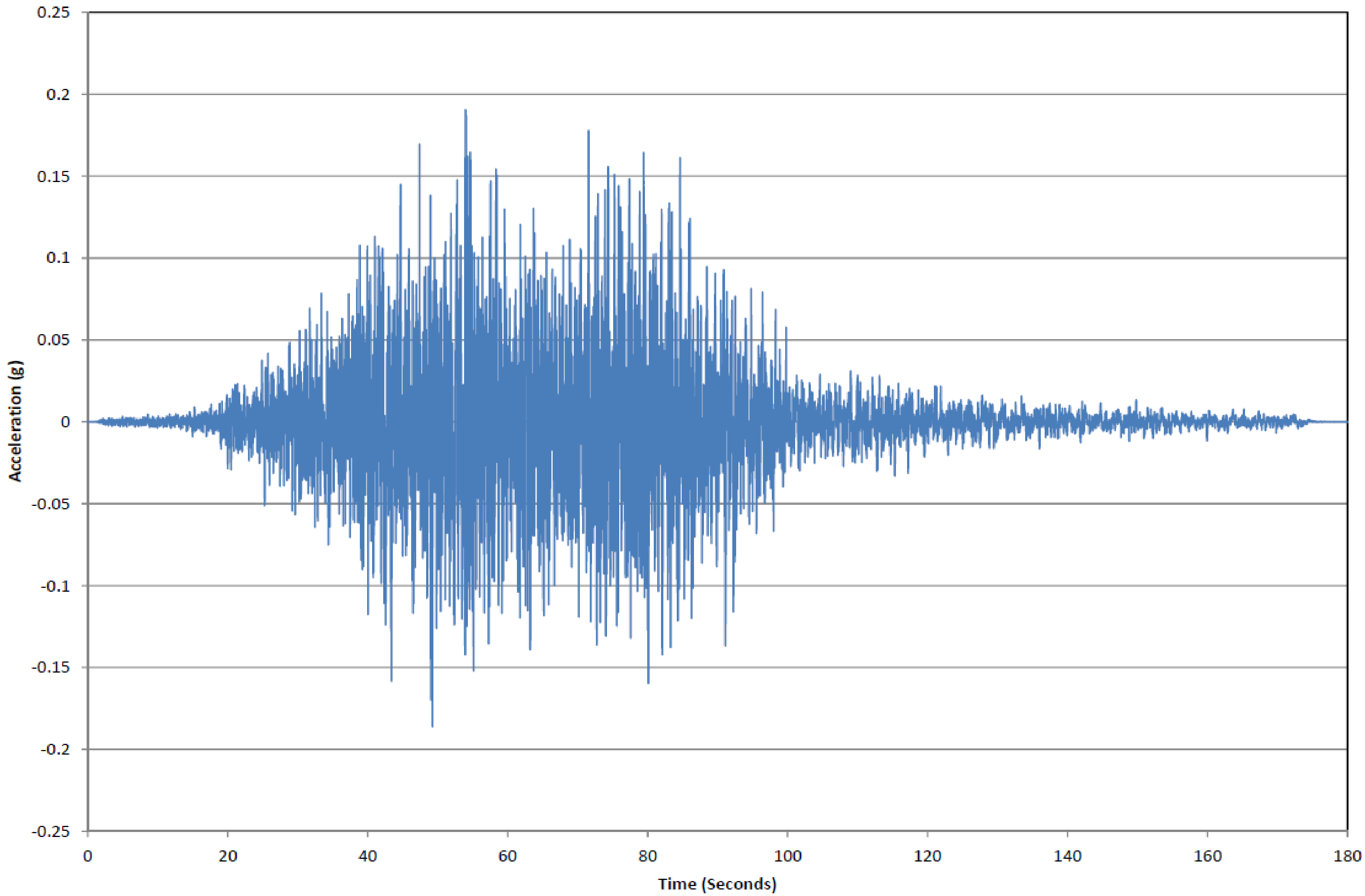
CHEHALIS DAM
PE ELL, WASHINGTON

ORIGINATOR: STA

APPROVED BY: KAF/KM

DRAWING CATEGORY:

FIGURE 7



2,500—Year Return Period

PROJECT NO. 10026522	Curico Vertical Acceleration Record			FIGURE
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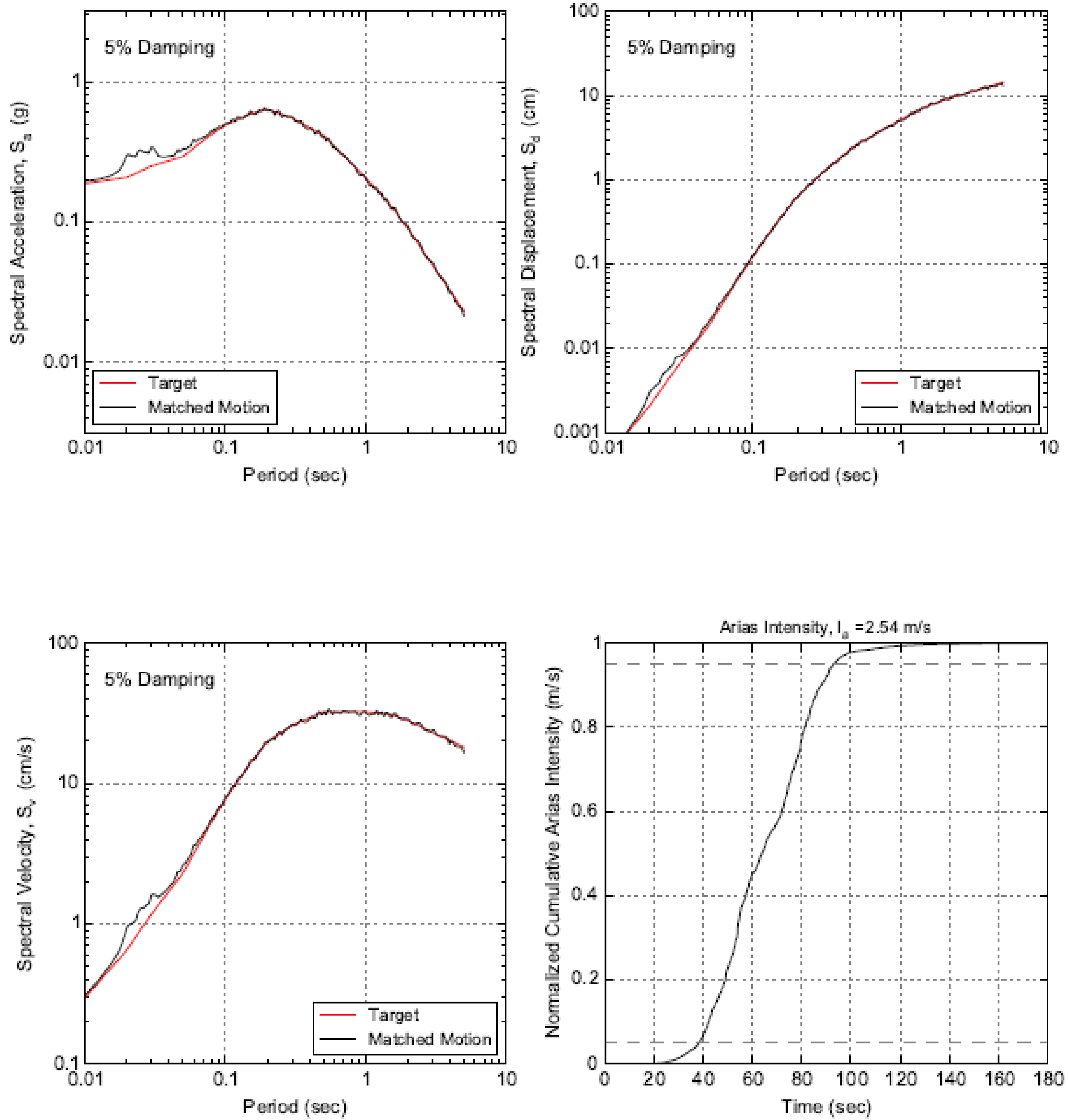

2,500—Year Return Period

PROJECT NO. 10026522	Curico Vertical Acceleration Record		FIGURE
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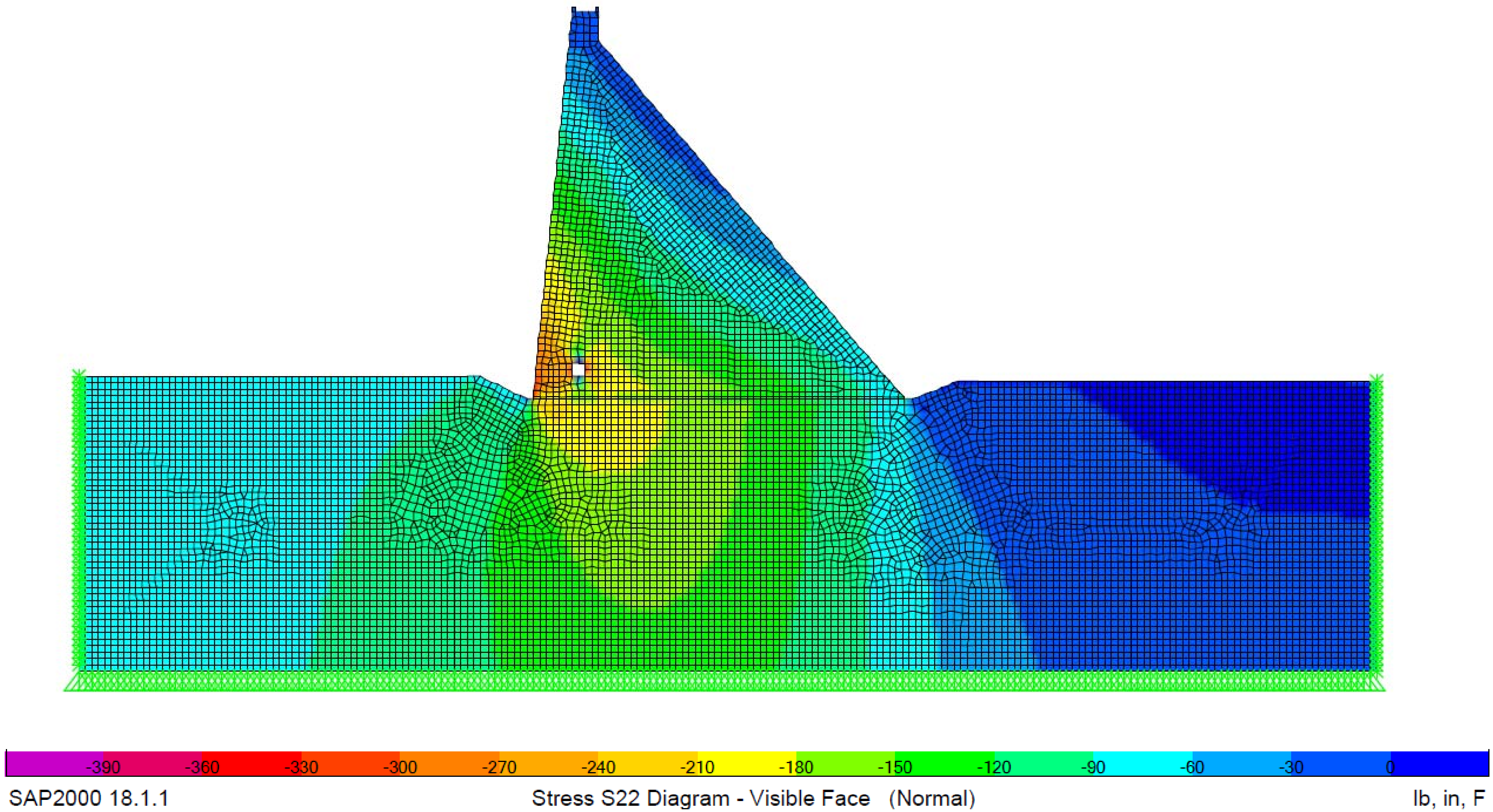
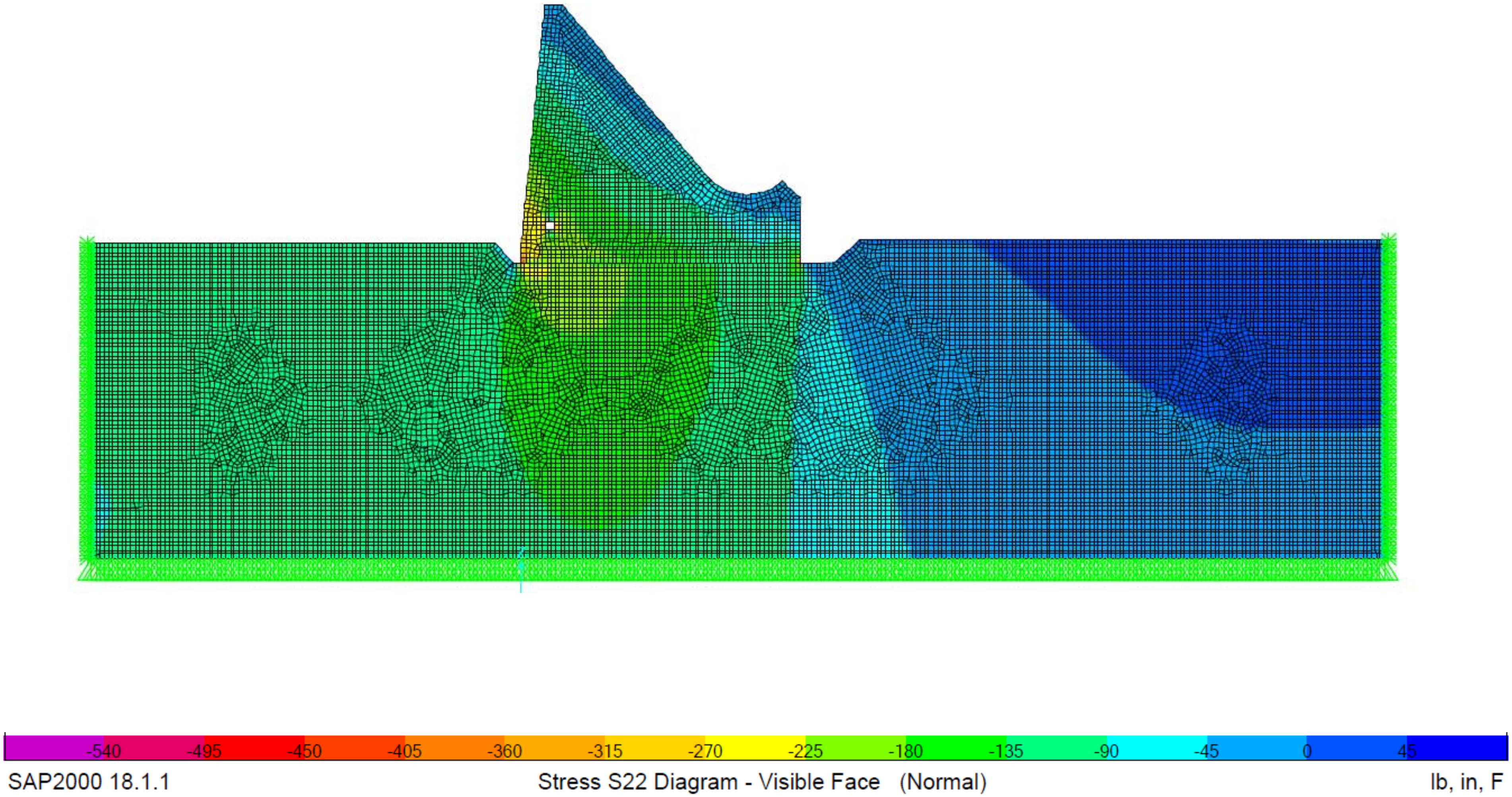
DRAWN BY: STA			
CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:	APPROVED BY: KAF/KM		



Scale is psi

Non-Overflow Section

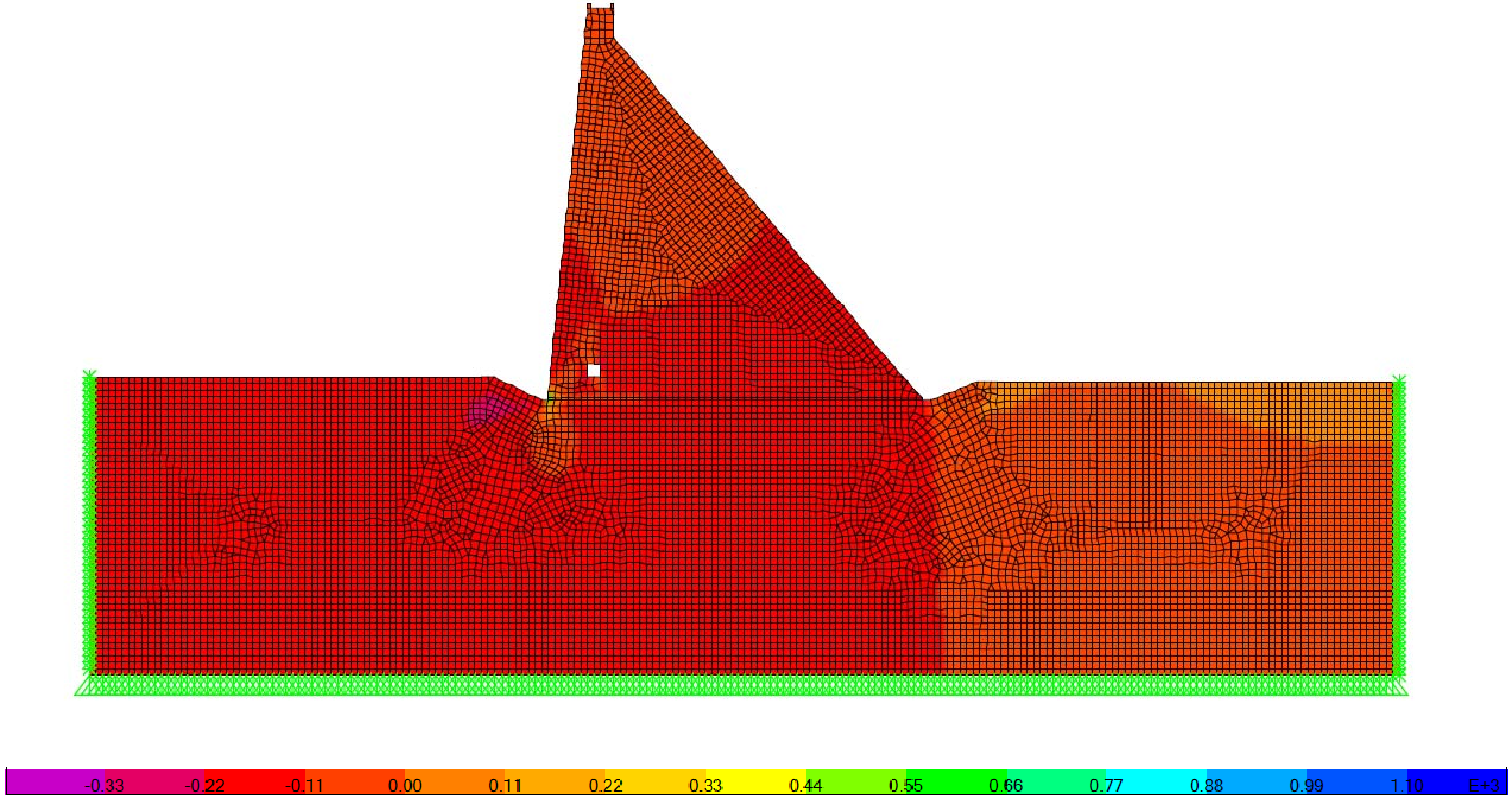
HDR			
PROJECT NO. 10026522	Normal Loading		FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		10
DRAWN BY: STA			
CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:	APPROVED BY: KAF/KM		



Scale is psi

Overflow Section

HDR	PROJECT NO. 10026522	Normal Loading	FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	11
	DRAWN BY: STA		
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:
	FILE NAME:	APPROVED BY: KAF/KM	

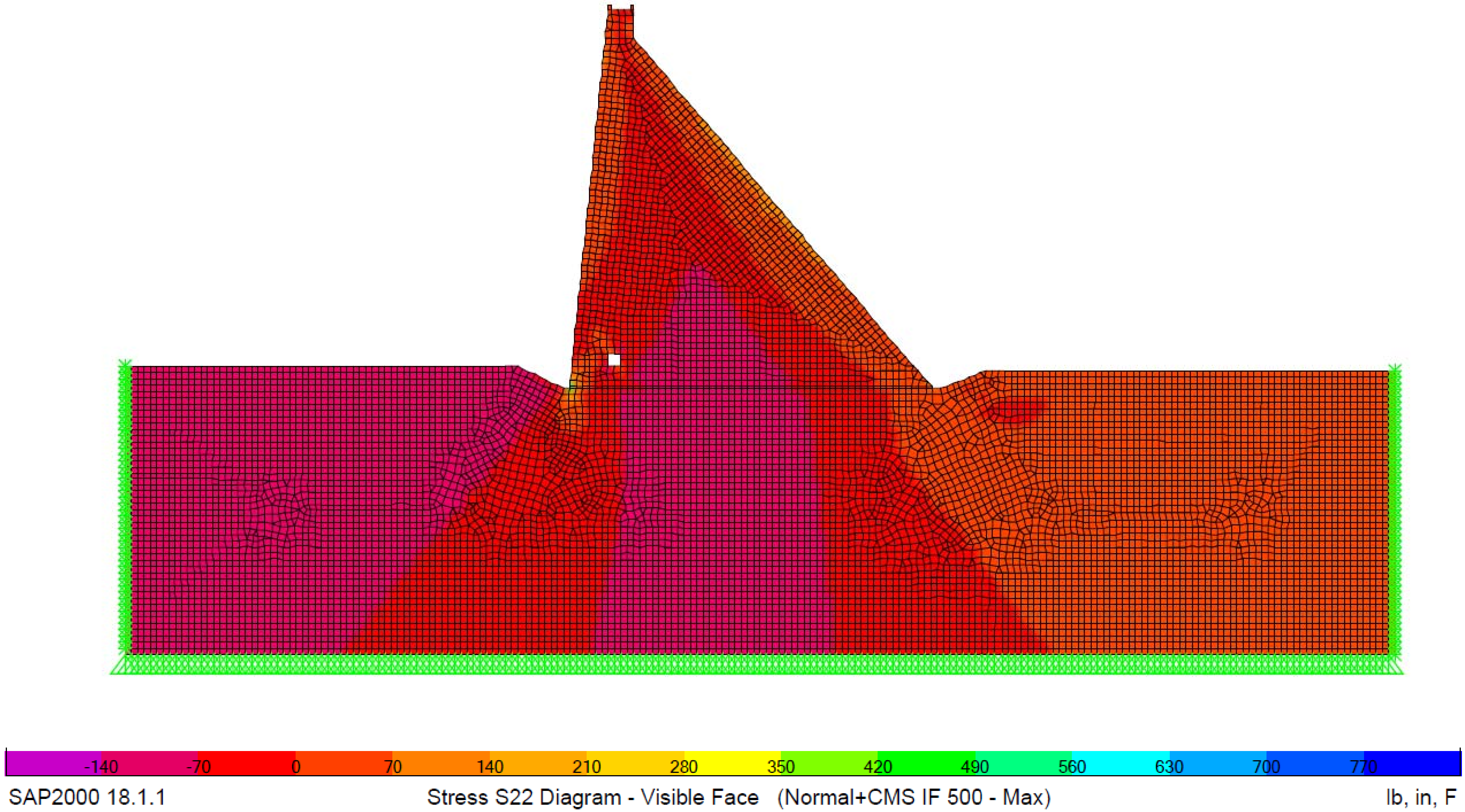


SAP2000 18.1.1 Stress S22 Diagram - Visible Face (Normal+PMF) lb, in, F

Scale is x 10^3 psi

Non-Overflow Section

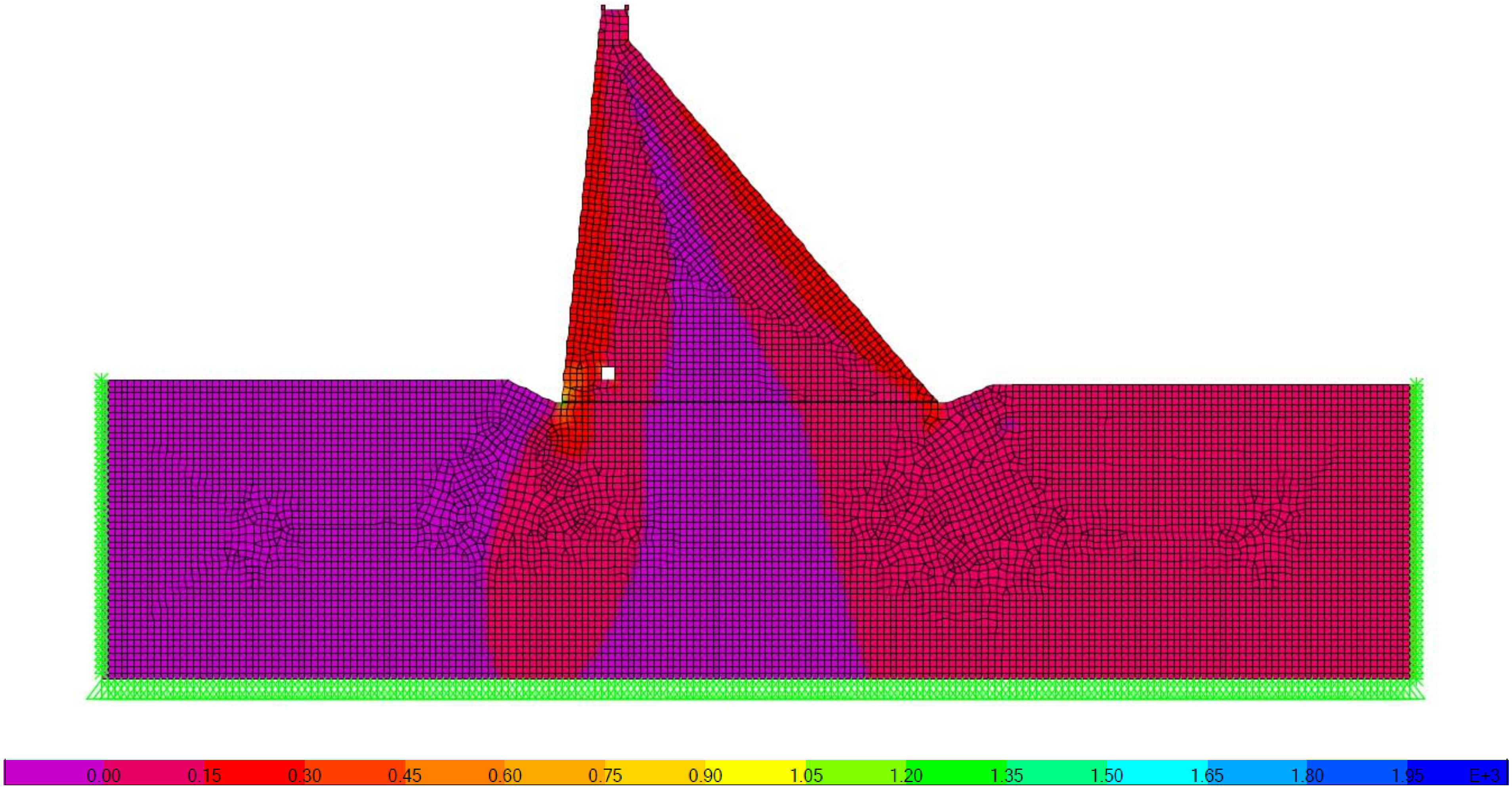
PROJECT NO. 10026522	Normal+PMF Loading		FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		12
DRAWN BY: STA			
CHECKED BY: FA			
FILE NAME:	ORIGINATOR: STA	DRAWING CATEGORY:	
	APPROVED BY: KAF/KM		



Scale is psi

Non-Overflow Section

HDR	PROJECT NO. 10026522	Normal+ CMS Interface 500-yr Loading	FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	13
	DRAWN BY: STA		
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:
	FILE NAME:	APPROVED BY: KAF/KM	

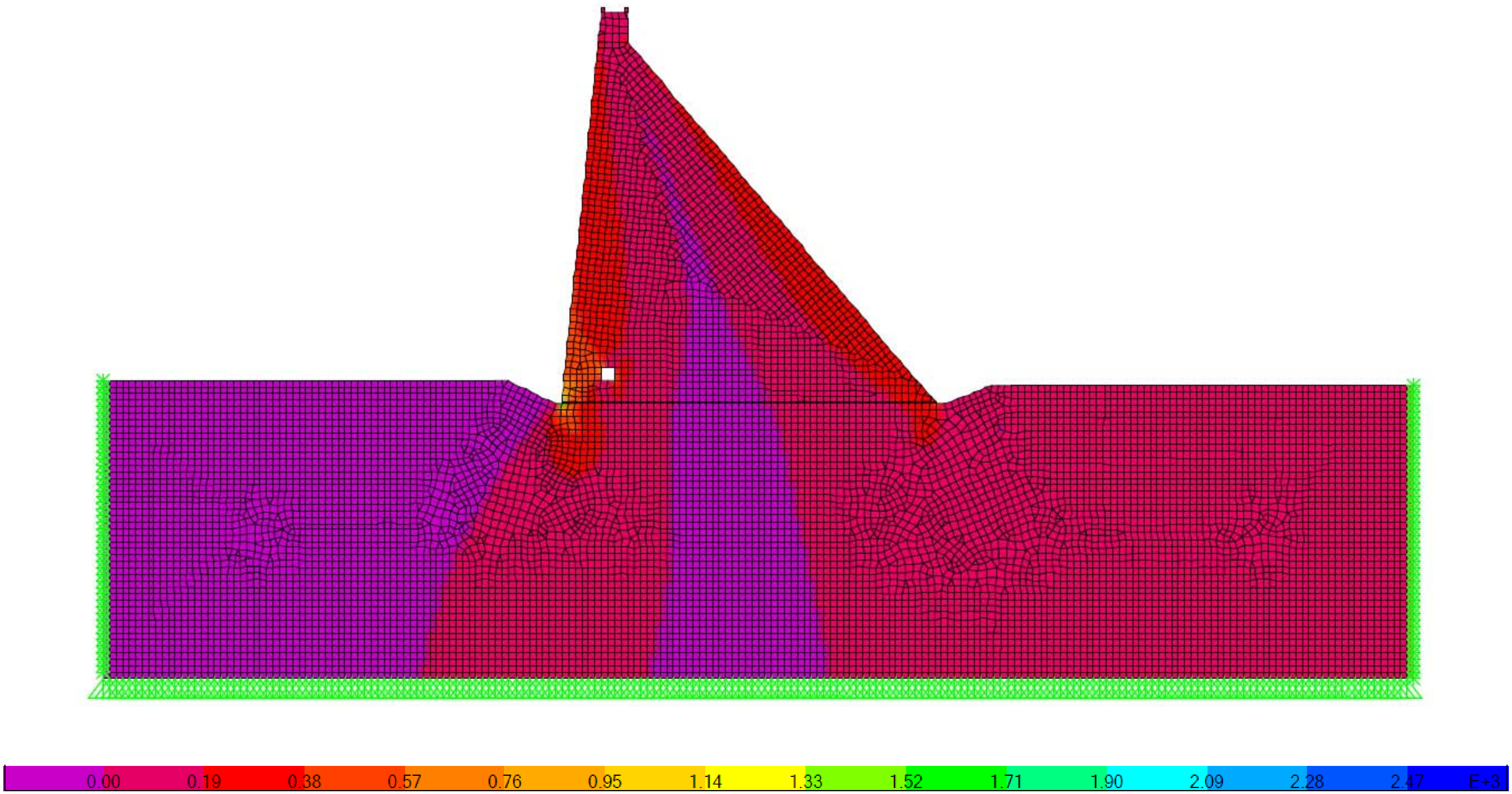


SAP2000 18.1.1 Stress S22 Diagram - Visible Face (Normal+CMS IF 2500 - Max) lb, in, F

Scale is x 10^3 psi

Non-Overflow Section

PROJECT NO.	10026522	Normal+ CMS Interface 2500-yr Loading		FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		14
DRAWN BY:	STA			
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:		APPROVED BY: KAF/KM		

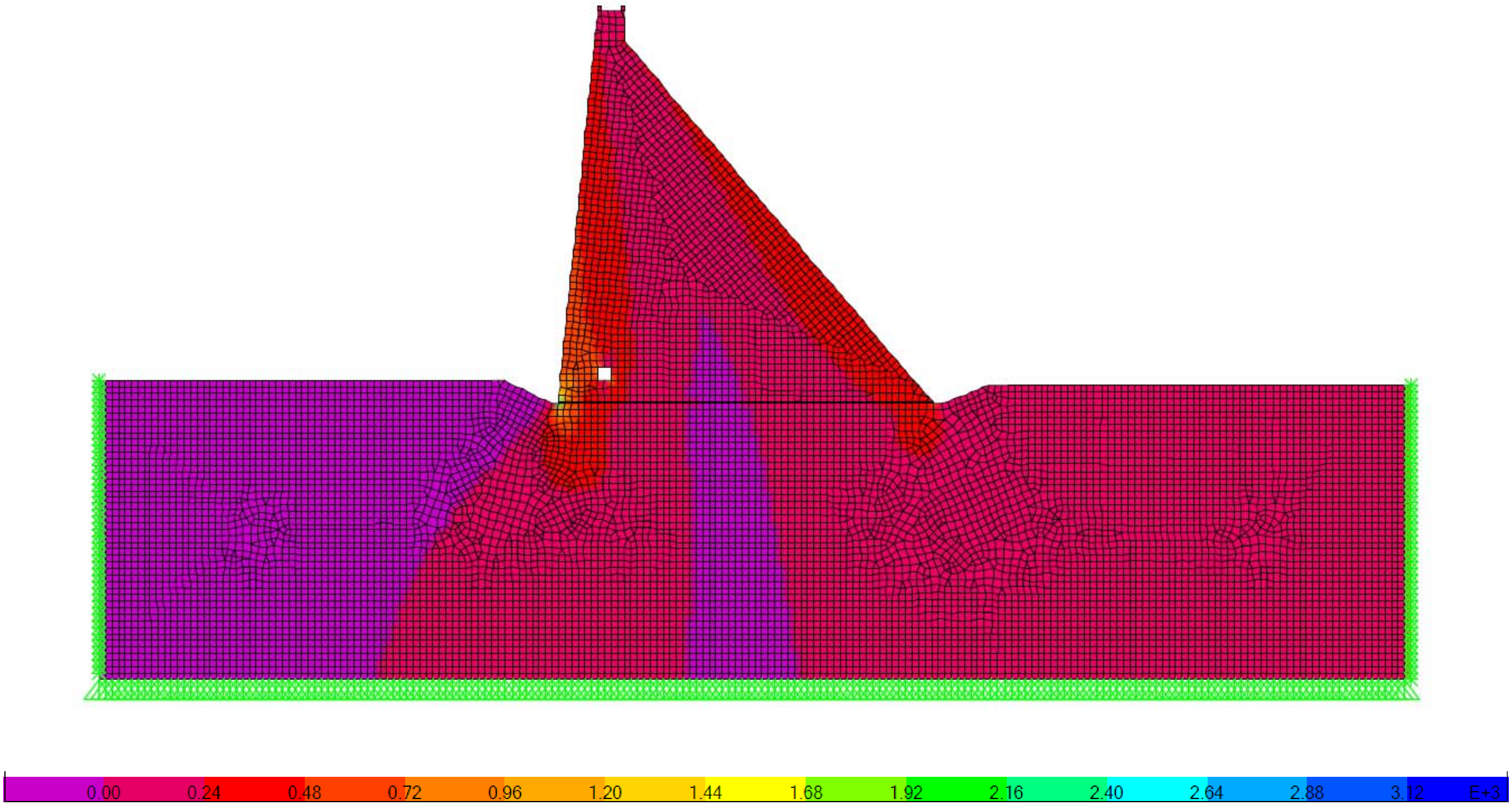


SAP2000 18.1.1 | Stress S22 Diagram - Visible Face (Normal+ CMS IF 5000 - Max) | lb, in, F

Scale is x 10^3 psi

Non-Overflow Section

PROJECT NO.	10026522	Normal+ CMS Interface 5000-yr Loading	FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	15
DRAWN BY:	STA		
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:
FILE NAME:		APPROVED BY: KAF/KM	



SAP2000 18.1.1

Stress S22 Diagram - Visible Face (Normal+CMS IF 10000 - Max)

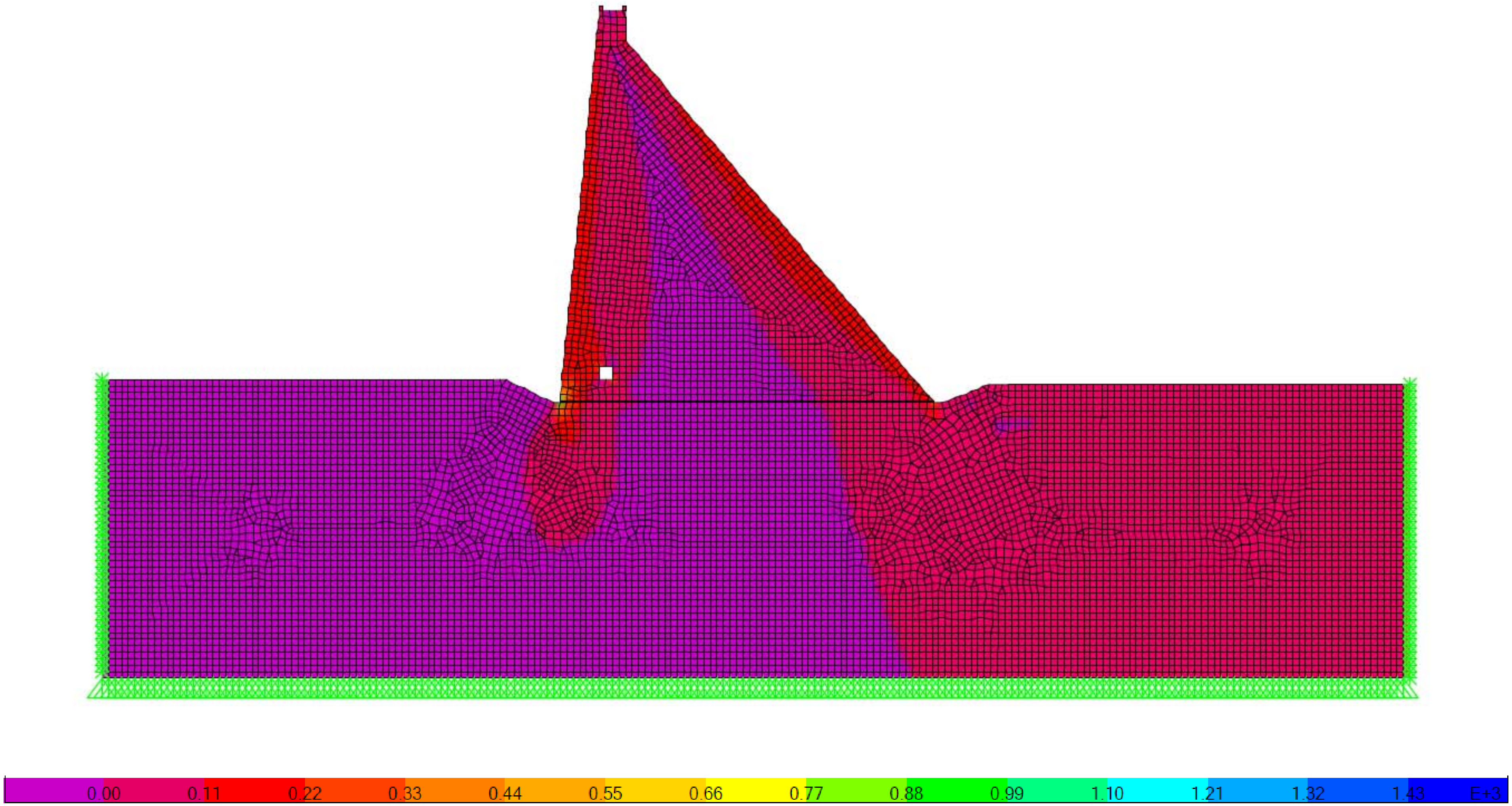
lb, in, F

Scale is x 10^3 psi

Non-Overflow Section

HDR

PROJECT NO.	10026522	Normal+ CMS Interface 10000-yr Loading	FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	16
DRAWN BY:	STA		
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:
FILE NAME:		APPROVED BY: KAF/KM	



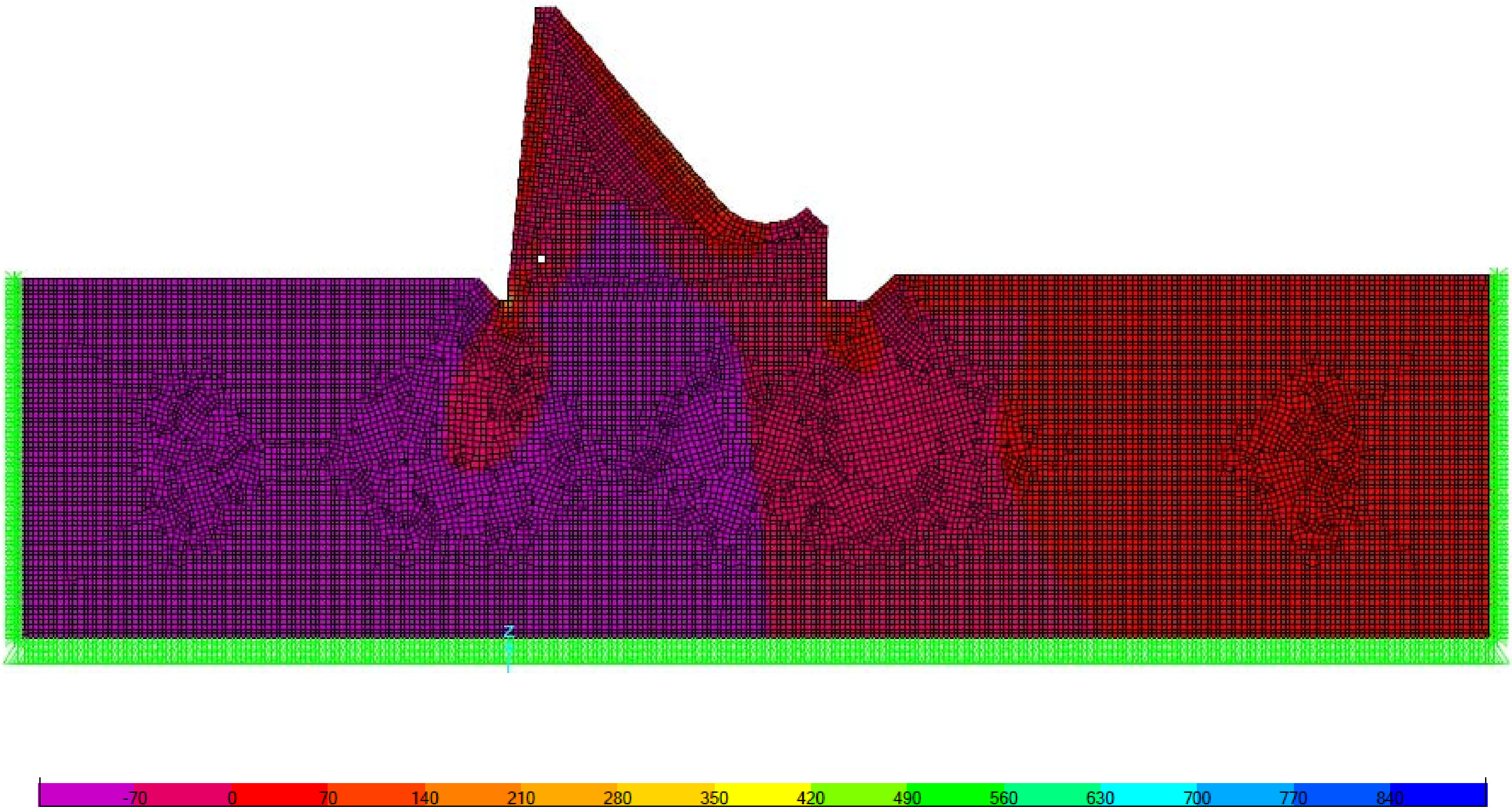
SAP2000 18.1.1 Stress S22 Diagram - Visible Face (Normal+MCE-CSZ-IF - Max) lb, in, F

Scale is x 10^3 psi

Non-Overflow Section

HDR

PROJECT NO.	10026522	Normal+ CMS MCE Doty Loading	FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	17
DRAWN BY:	STA		
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:
FILE NAME:		APPROVED BY: KAF/KM	

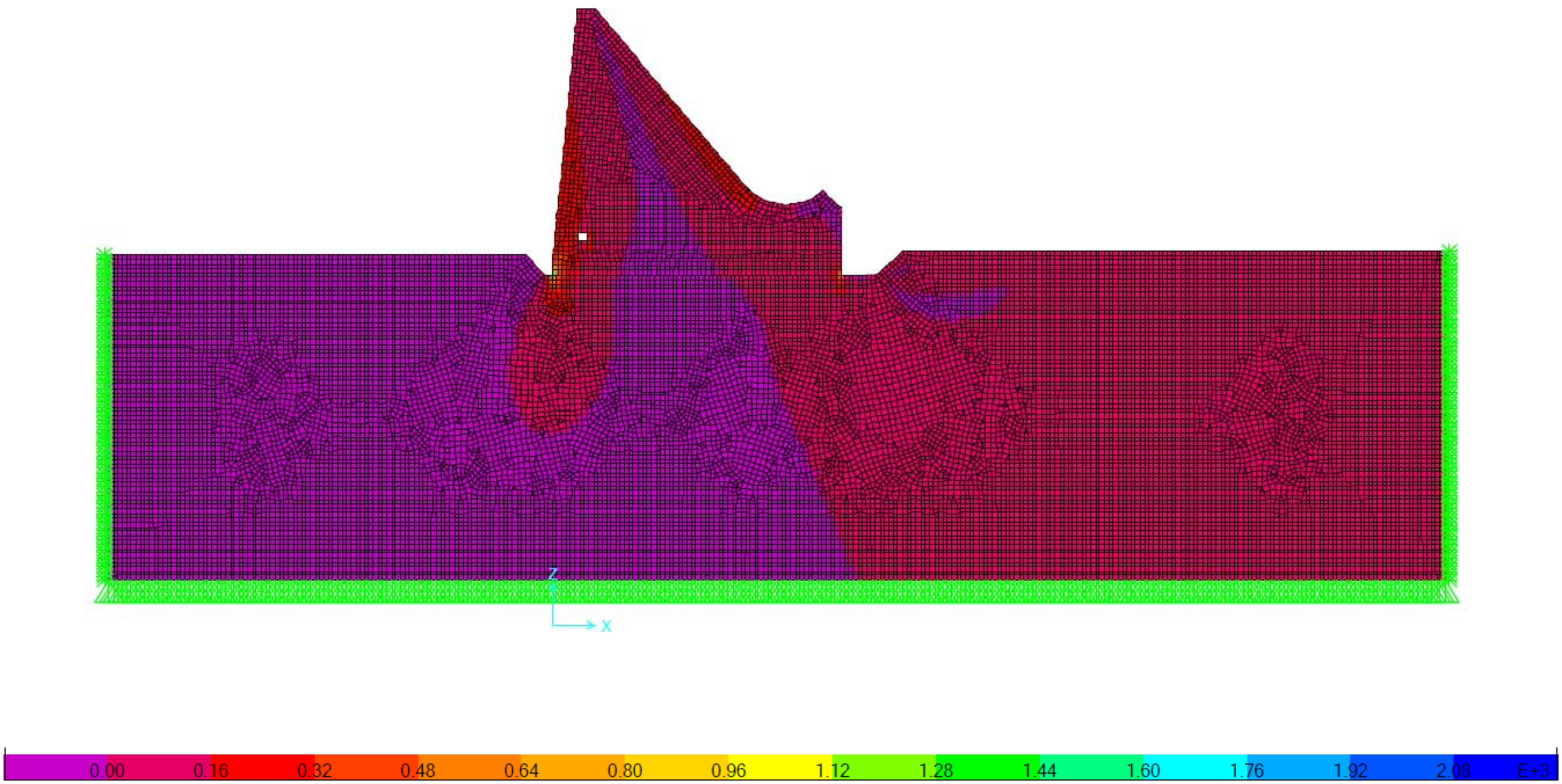


SAP2000 18.1.1 Stress S22 Diagram - Visible Face (Normal+IF500 - Max) lb, in, F

Scale is psi

Overflow Section

HDR	PROJECT NO. 10026522	Normal+ CMS Interface 500-yr Loading	FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL WASHINGTON	18
	DRAWN BY: STA		
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:
	FILE NAME:	APPROVED BY: KAF/KM	



SAP2000 18.1.1

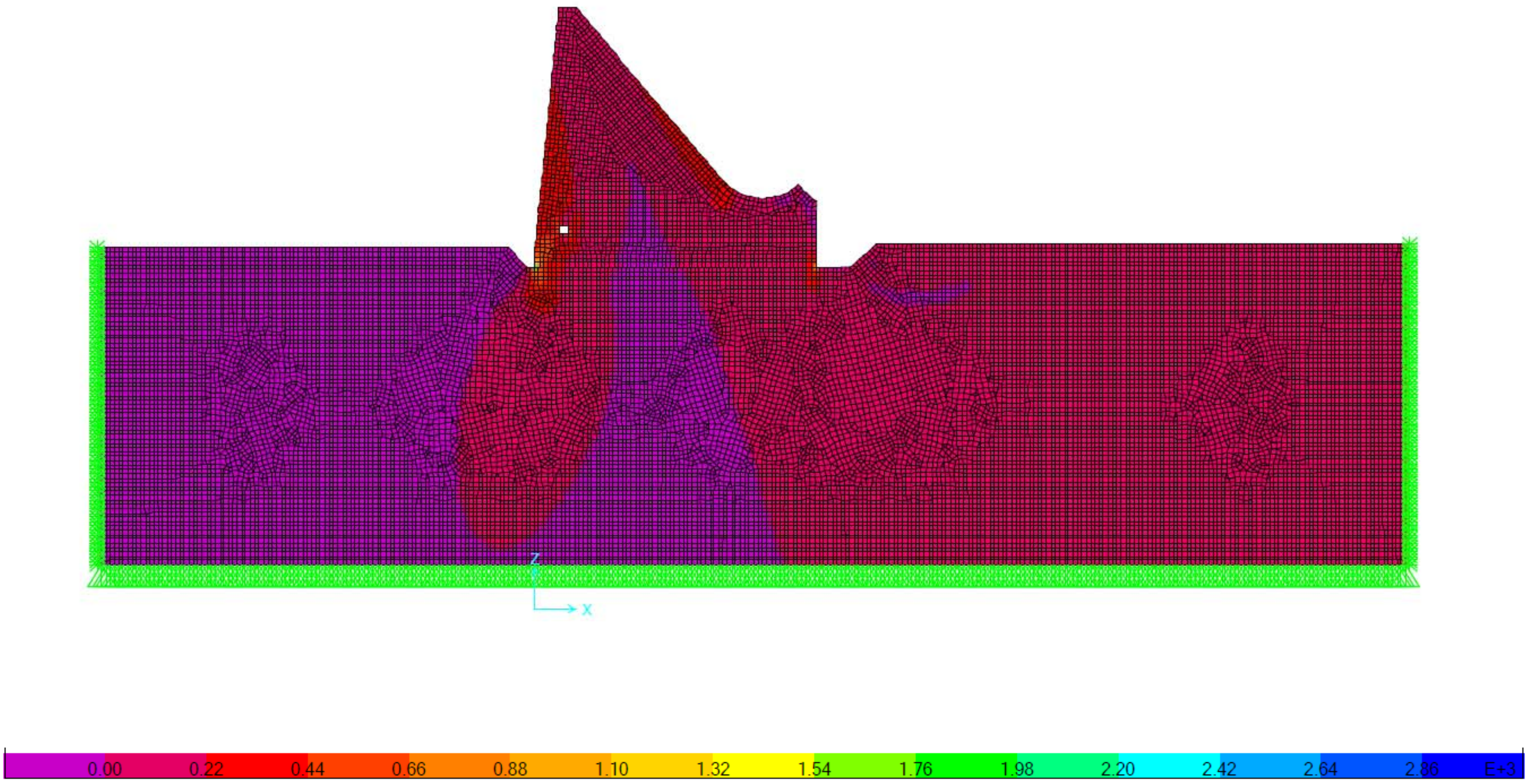
Scale 10^3 psi

Stress S22 Diagram - Visible Face (Normal+IF2500 - Max)

lb, in, F

Overflow Section

PROJECT NO.	10026522	Normal+ CMS Interface 2500-yr Loading	FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	19
DRAWN BY:	STA		
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:
FILE NAME:		APPROVED BY: KAF/KM	



SAP2000 18.1.1

Stress S22 Diagram - Visible Face (Normal+IF5000 - Max)

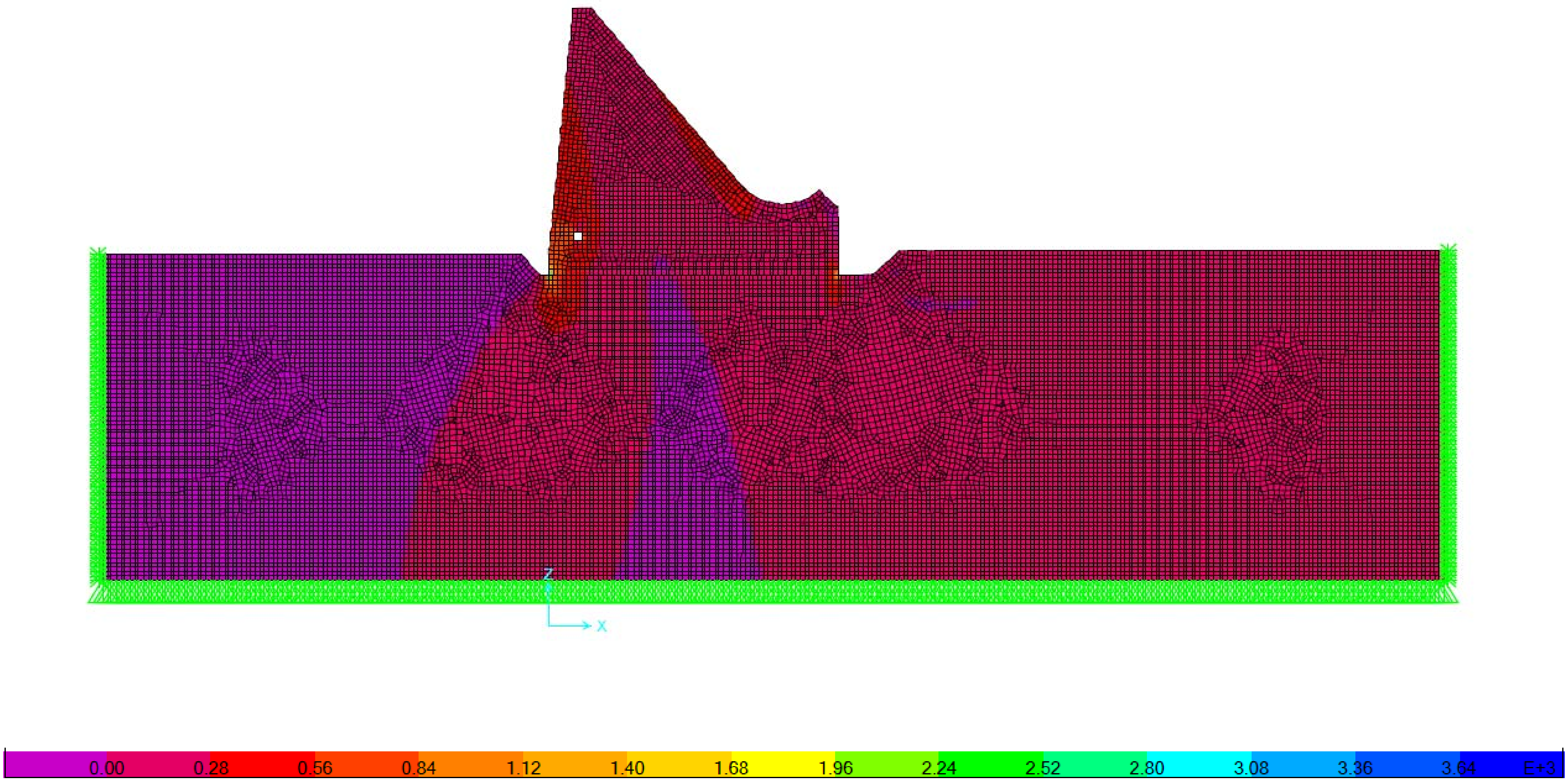
lb, in, F

Scale 10^3 psi

Overflow Section

HDR

PROJECT NO. 10026522	Normal+ CMS Interface 5000-yr Loading		FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		20
DRAWN BY: STA			
CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:	APPROVED BY: KAF/KM		

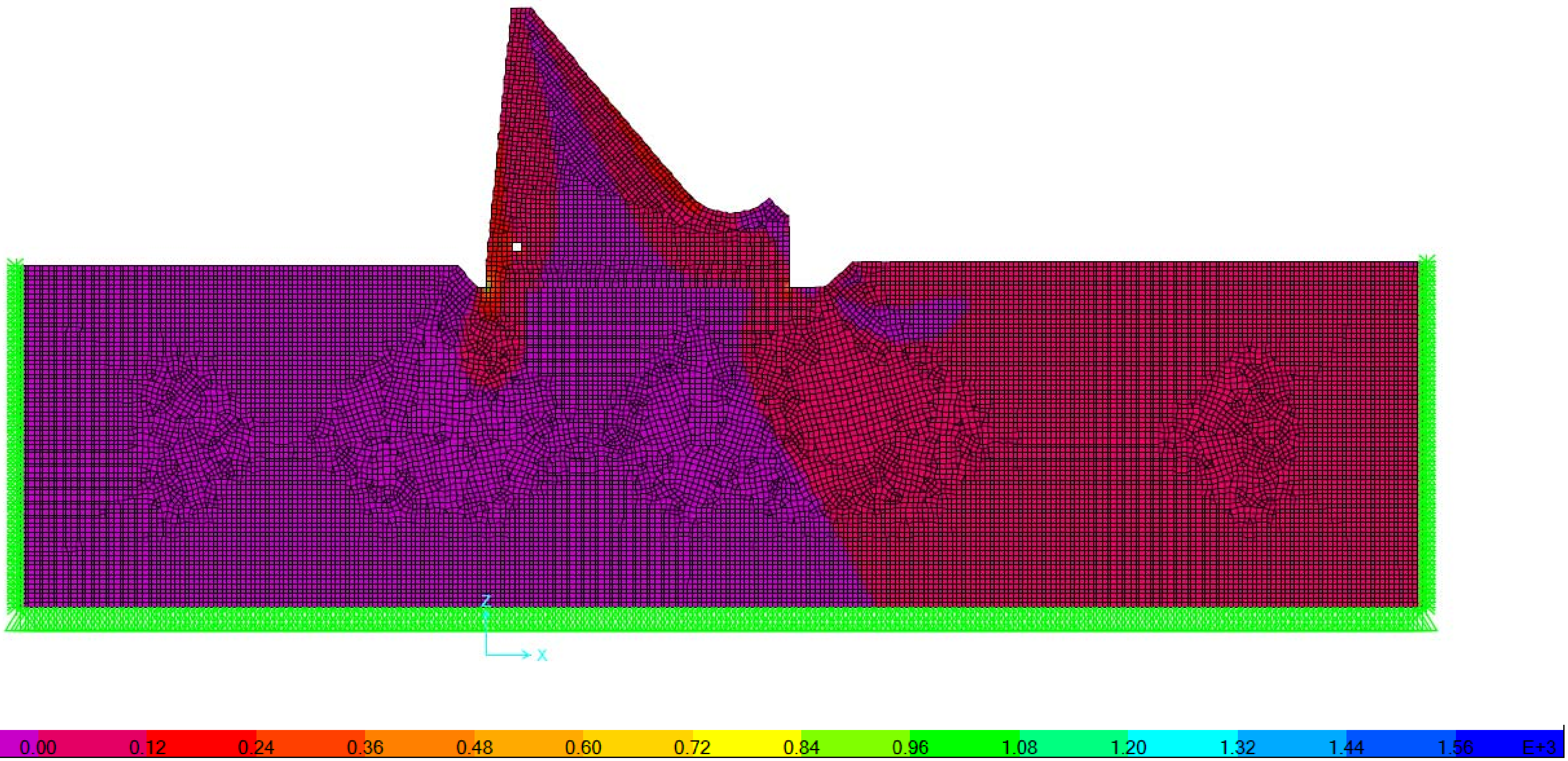


SAP2000 18.1.1 Stress S22 Diagram - Visible Face (Normal+IF10000 - Max) lb, in, F

Scale 10^3 psi

Overflow Section

PROJECT NO.	10026522	Normal+ CMS Interface 10000-yr Loading		FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		21
DRAWN BY:	STA			
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:		APPROVED BY: KAF/KM		

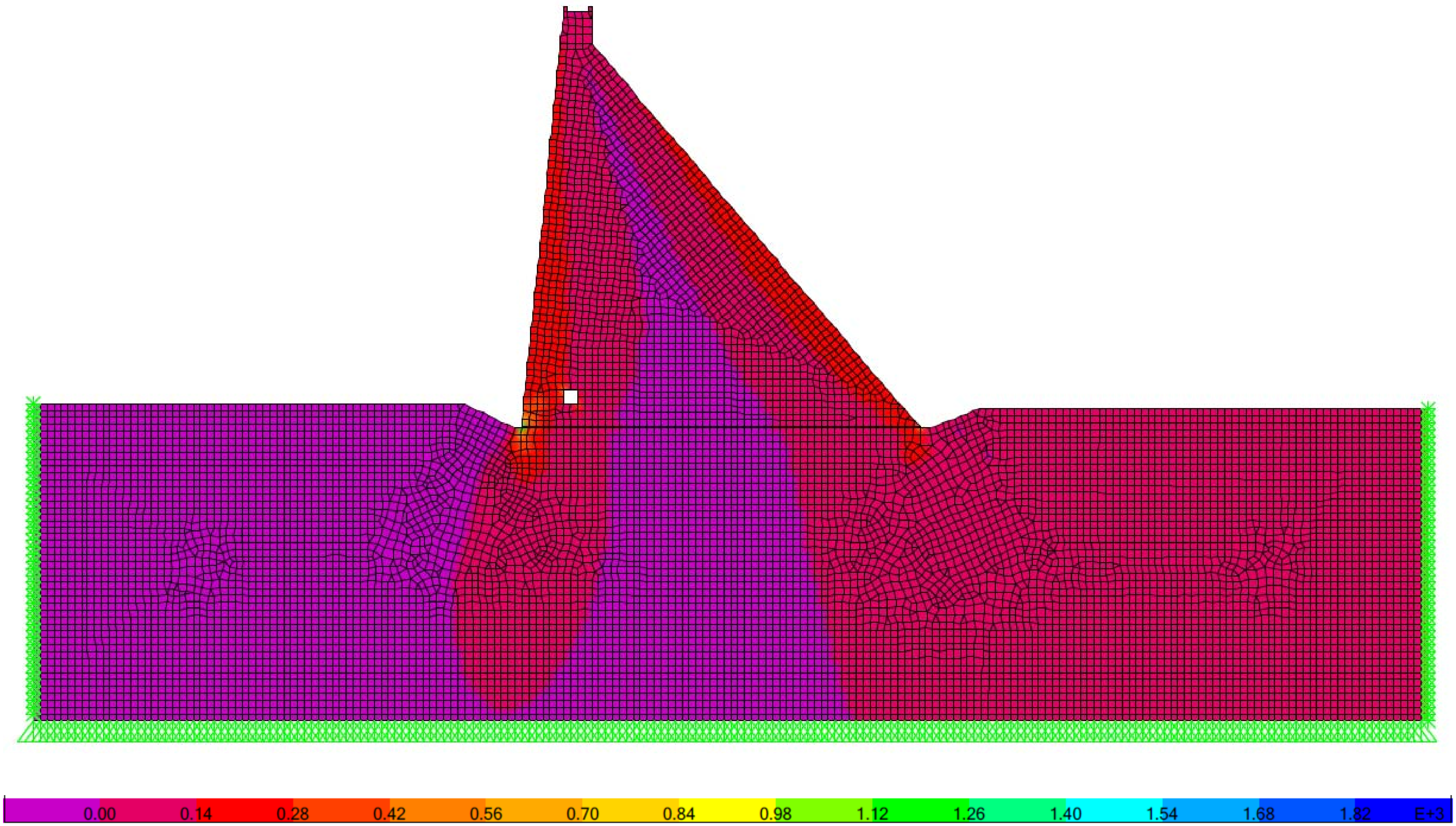


SAP2000 18.1.1 Stress S22 Diagram - Visible Face (Normal+MCE CSZ IF - Max) lb, in, F

Scale 10^3 psi

Overflow Section

PROJECT NO.	10026522	Normal+ CMS Interface MCE-Doty Loading		FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		22
DRAWN BY:	STA			
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:		APPROVED BY: KAF/KM		



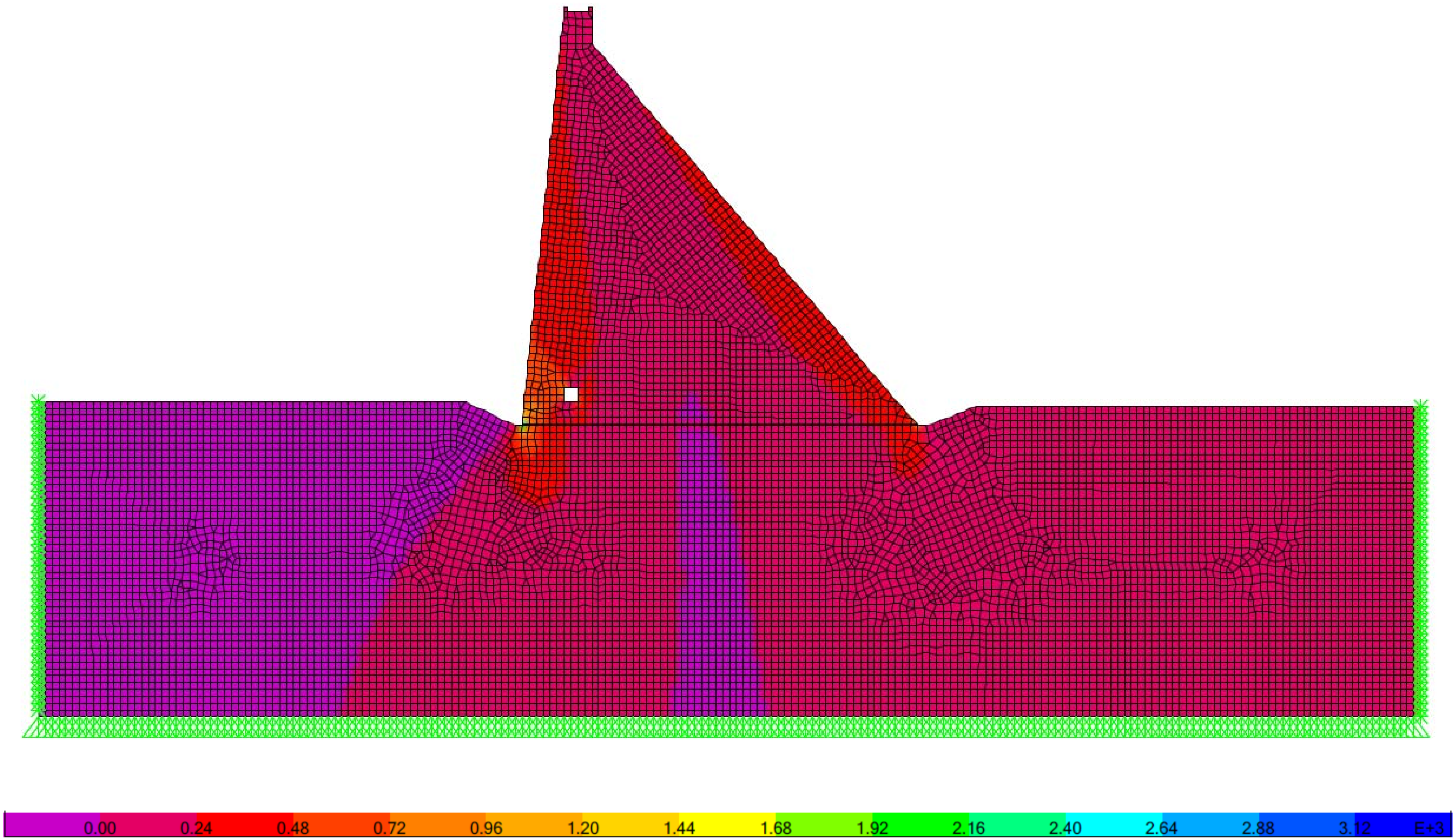
SAP2000 18.1.1 | Stress S22 Diagram - Visible Face (Normal+CMS IF 2500 - Max) | lb, in, F

Scale is x 10^3 psi

Non-Overflow Section

HDR

PROJECT NO.	10026522	Normal+ CMS Interface 2.5k Loading Reduced Modulus	FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	23
DRAWN BY:	STA		
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:
FILE NAME:		APPROVED BY: KAF/KM	

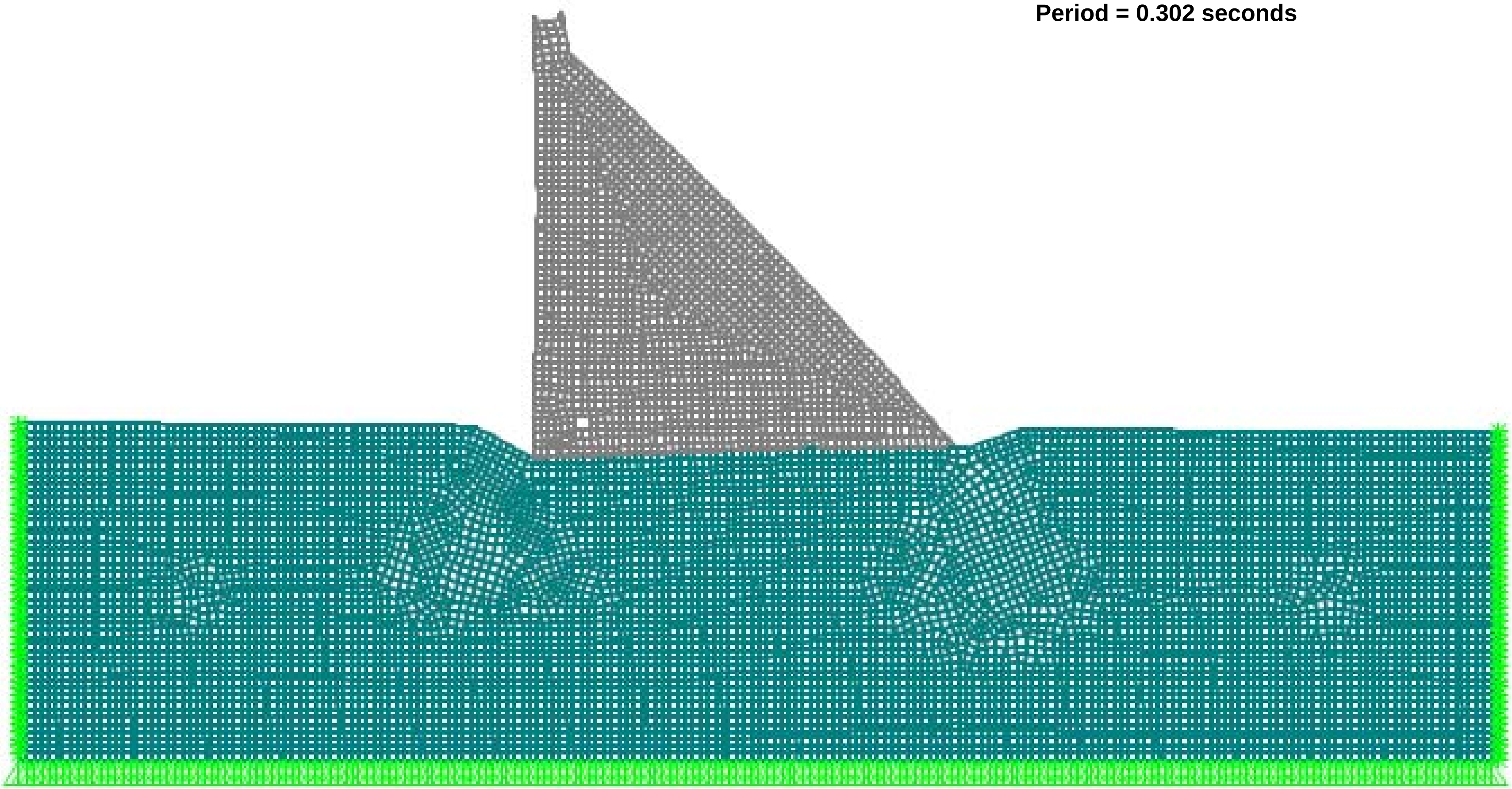


SAP2000 18.1.1 Stress S22 Diagram - Visible Face (Normal+CMS IF 10000 - Max) lb, in, F

Scale is x 10^3 psi

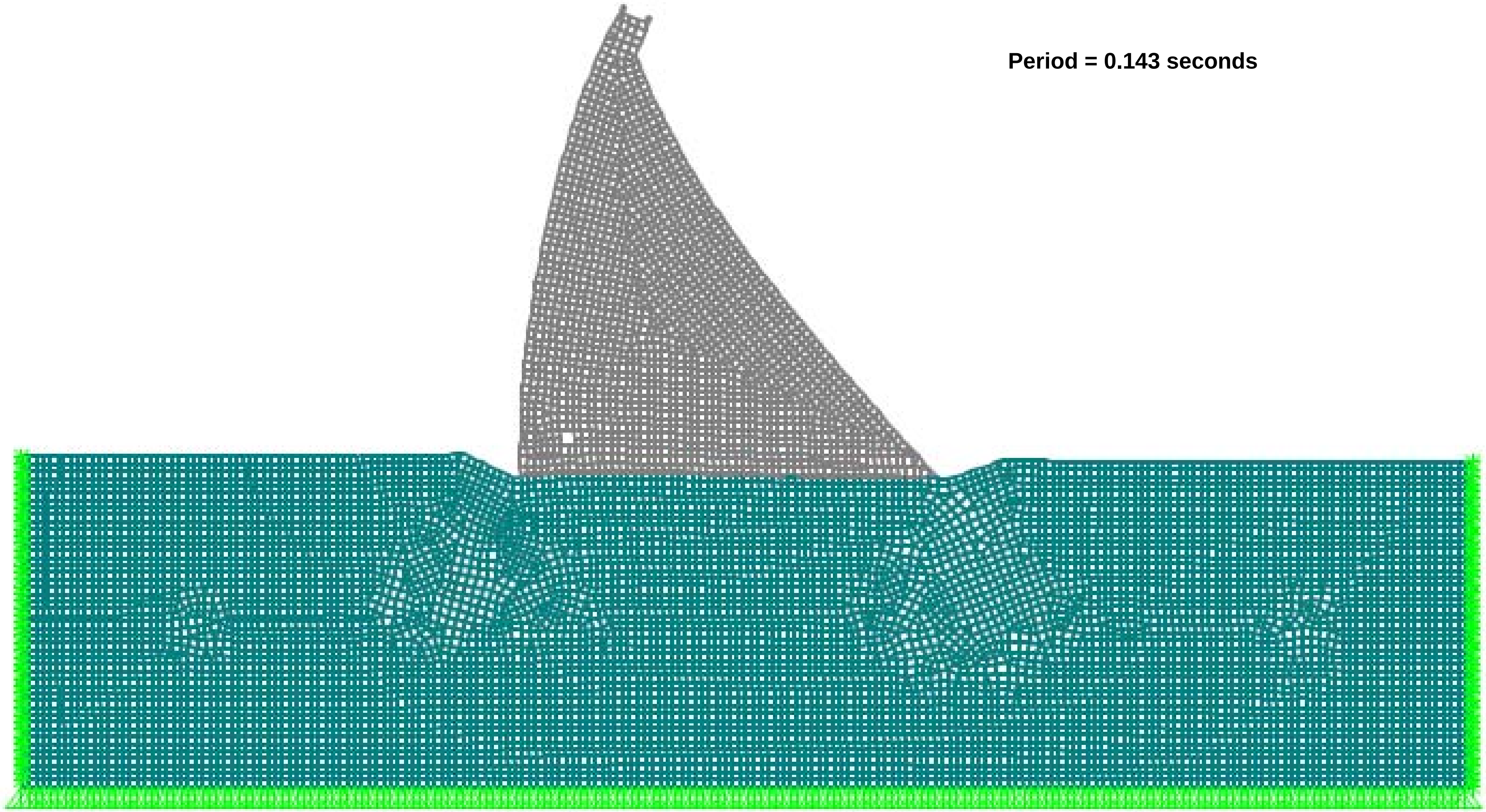
Non-Overflow Section

HDR	PROJECT NO. 10026522	Normal+ CMS Interface 10k Loading Reduced Modulus		FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		24
	DRAWN BY: STA			
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
	FILE NAME:	APPROVED BY: KAF/KM		



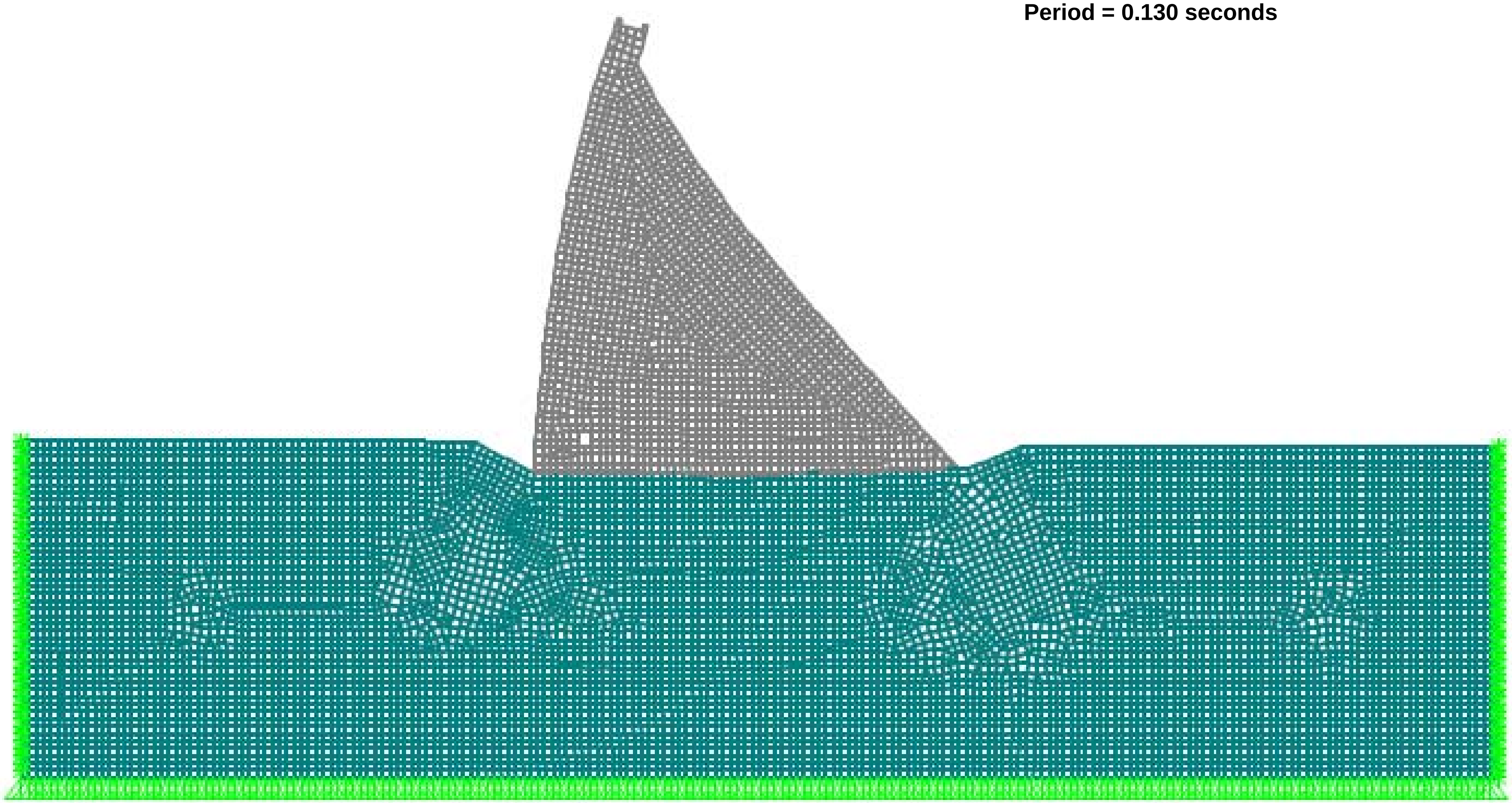
Non-Overflow Section

PROJECT NO.	10026522	Mode 1 Shape		FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		25
DRAWN BY:	STA			
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:		APPROVED BY: KAF/KM		



Non-Overflow Section

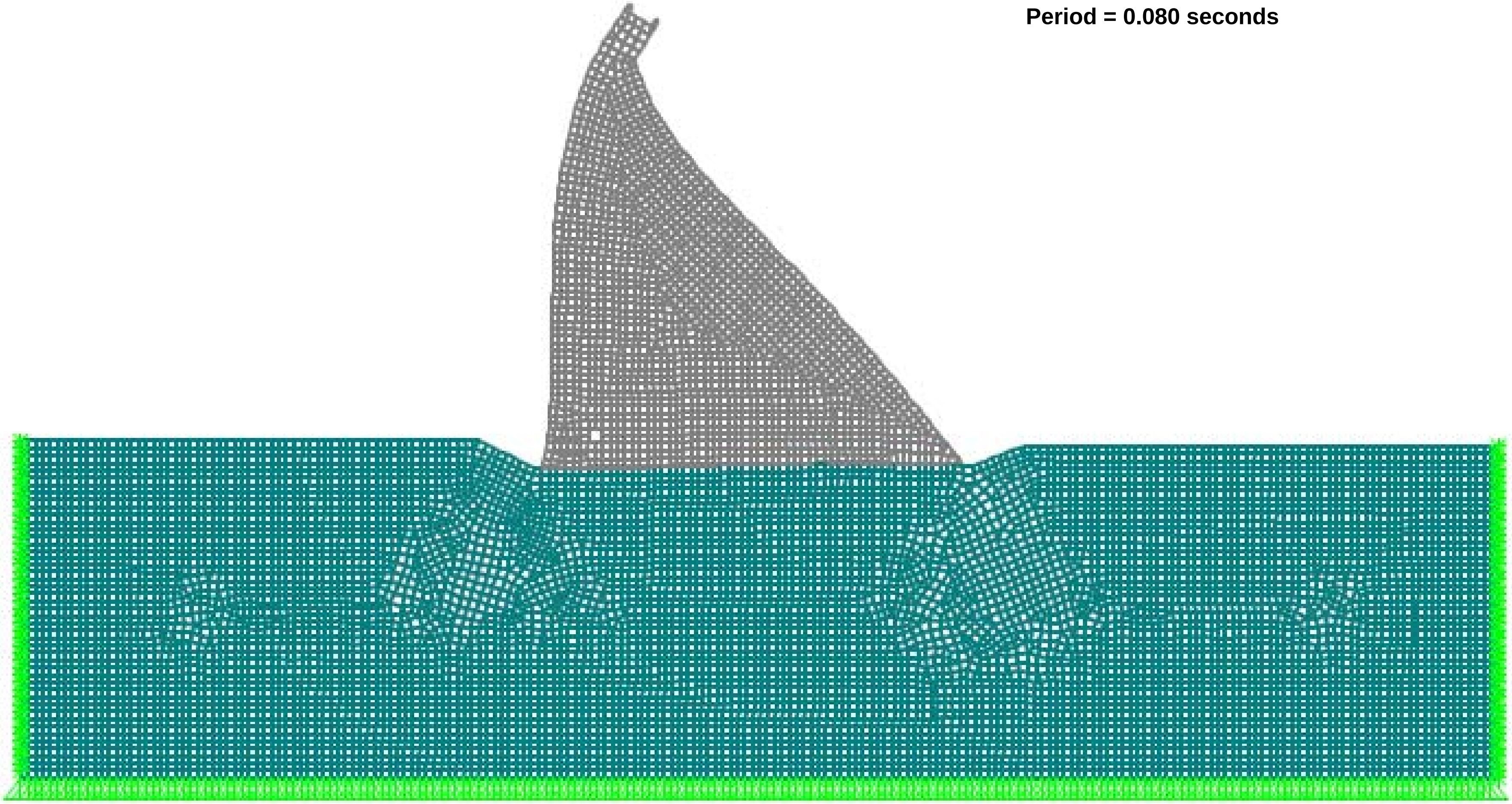
HDR	PROJECT NO.	10026522	Mode 2 Shape		FIGURE
	DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		26
	DRAWN BY:	STA			
	CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
	FILE NAME:		APPROVED BY: KAF/KM		



Non-Overflow Section

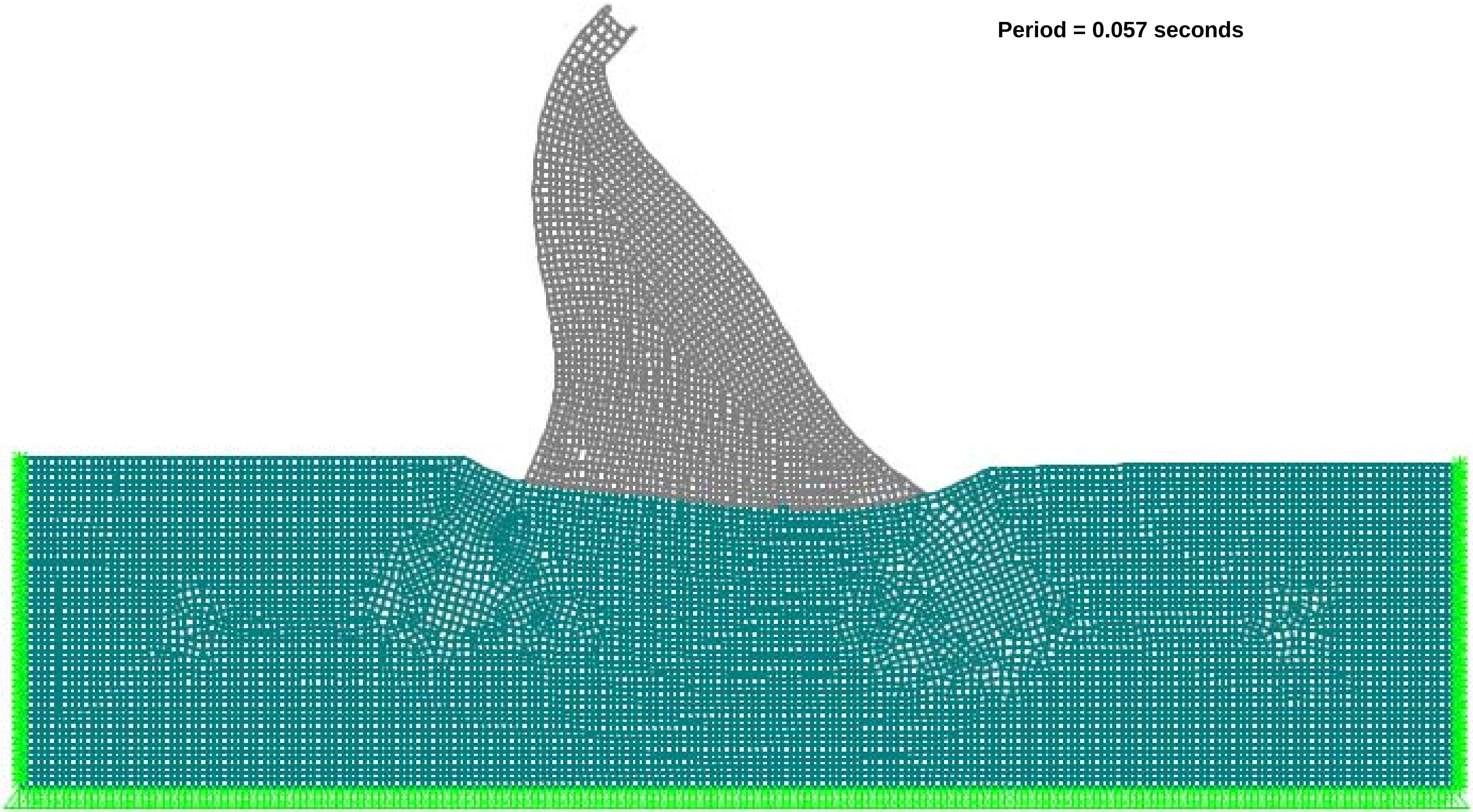
PROJECT NO.	10026522	Mode 3 Shape		FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		27
DRAWN BY:	STA			
CHECKED BY:	FA			
FILE NAME:		ORIGINATOR: STA	DRAWING CATEGORY:	
		APPROVED BY: KAF/KM		

Period = 0.080 seconds



Non-Overflow Section

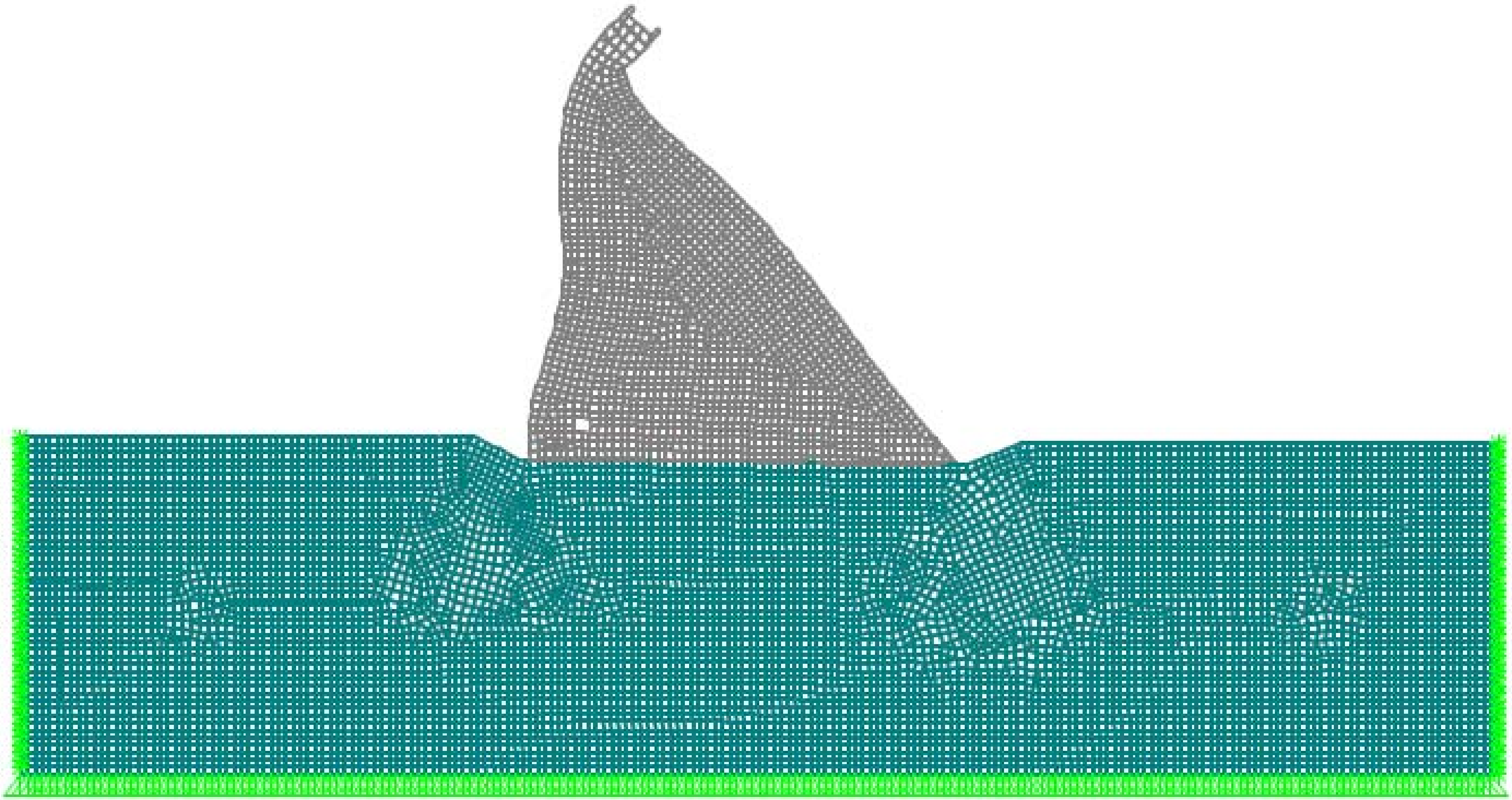
HDR	PROJECT NO. 10026522	Mode 4 Shape		FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		28
	DRAWN BY: STA			
	CHECKED BY: FA			
	FILE NAME:	ORIGINATOR: STA	DRAWING CATEGORY:	
		APPROVED BY: KAF/KM		



Non-Overflow Section

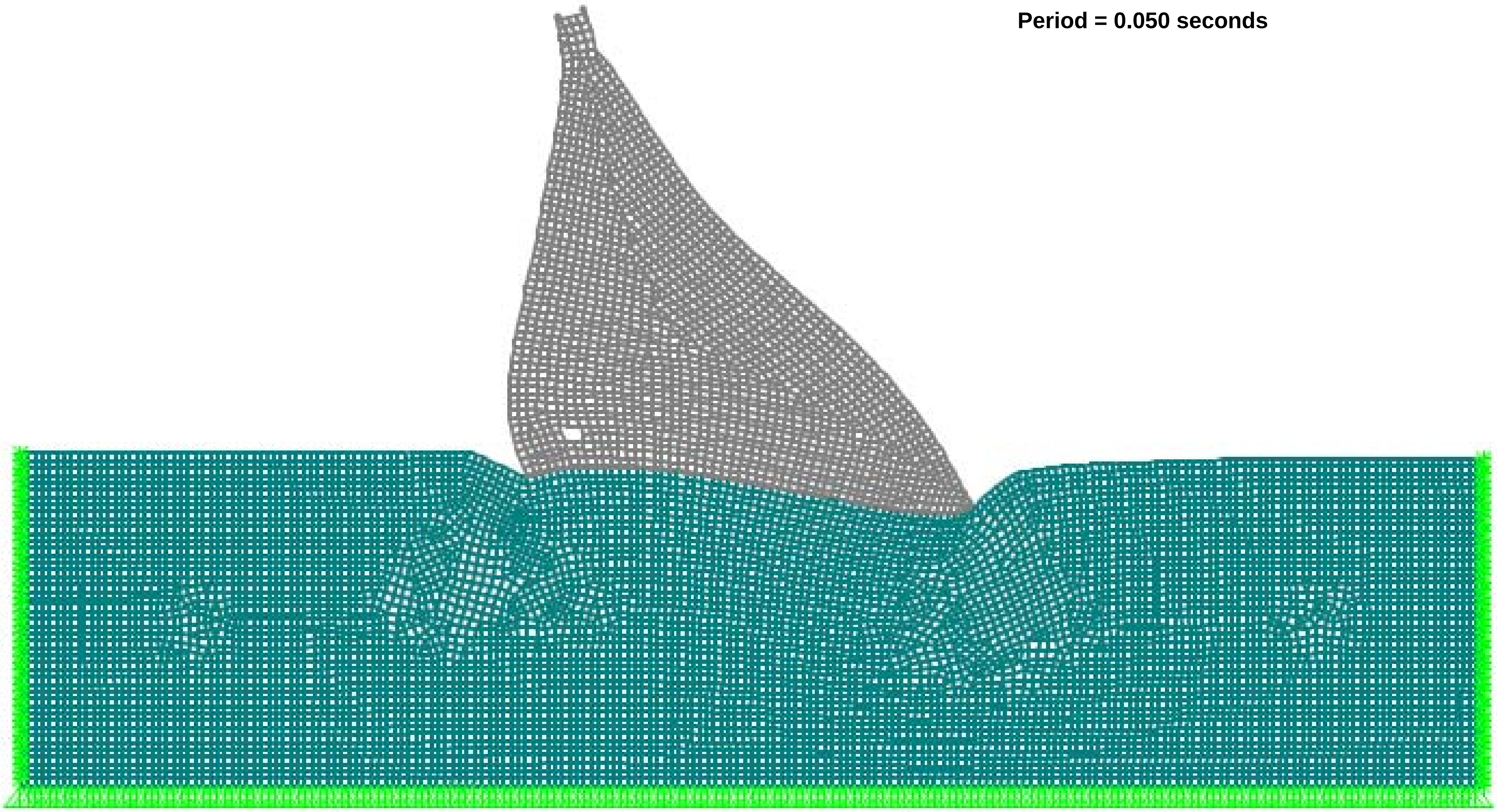
PROJECT NO. 10026522	Mode 5 Shape		FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		29
DRAWN BY: STA			
CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:	APPROVED BY: KAF/KM		

Period = 0.053 seconds



Non-Overflow Section

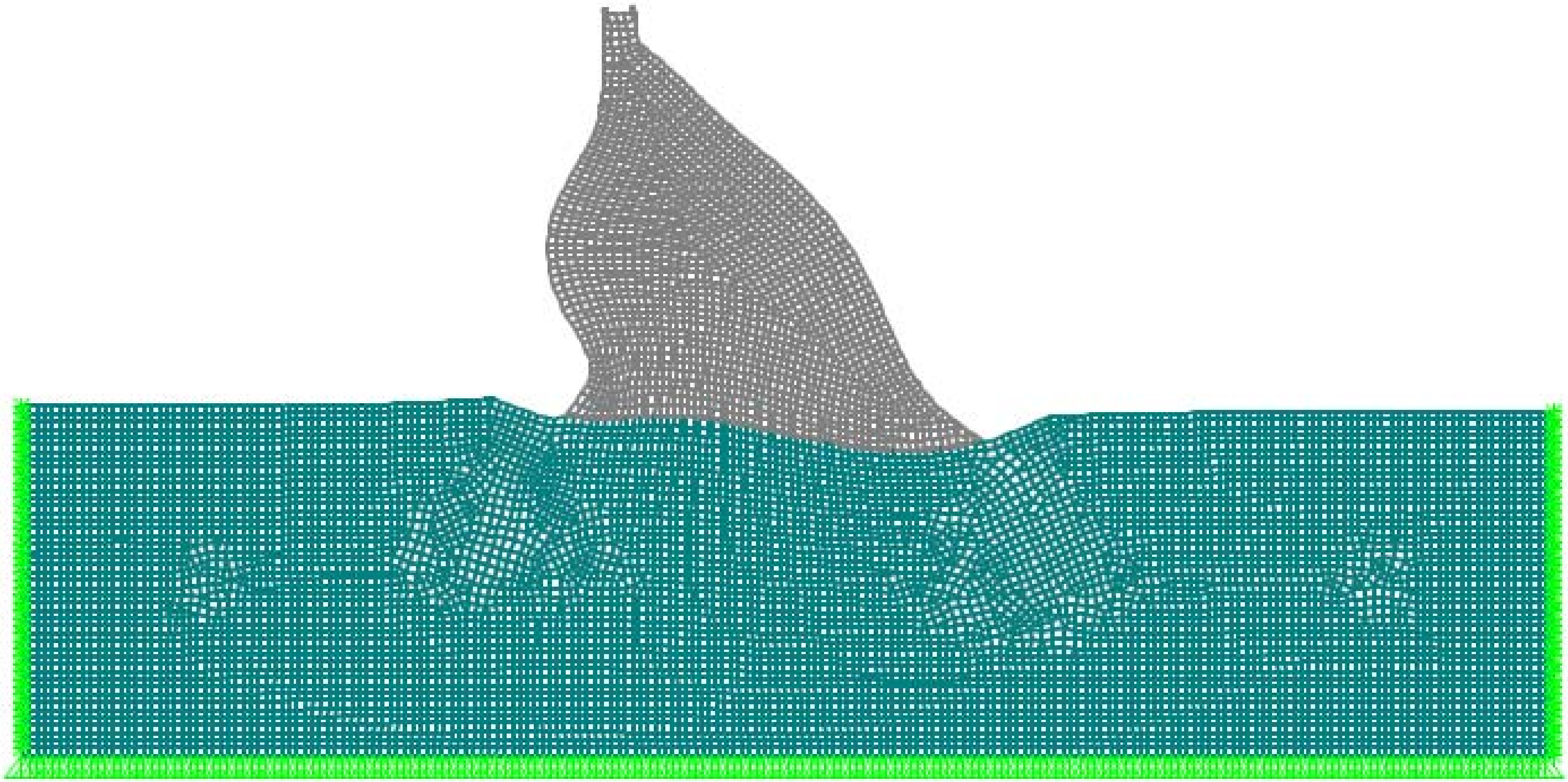
HDR	PROJECT NO. 10026522	Mode 6 Shape		FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		30
	DRAWN BY: STA			
	CHECKED BY: FA			
	FILE NAME:	ORIGINATOR: STA	DRAWING CATEGORY:	
		APPROVED BY: KAF/KM		



Non-Overflow Section

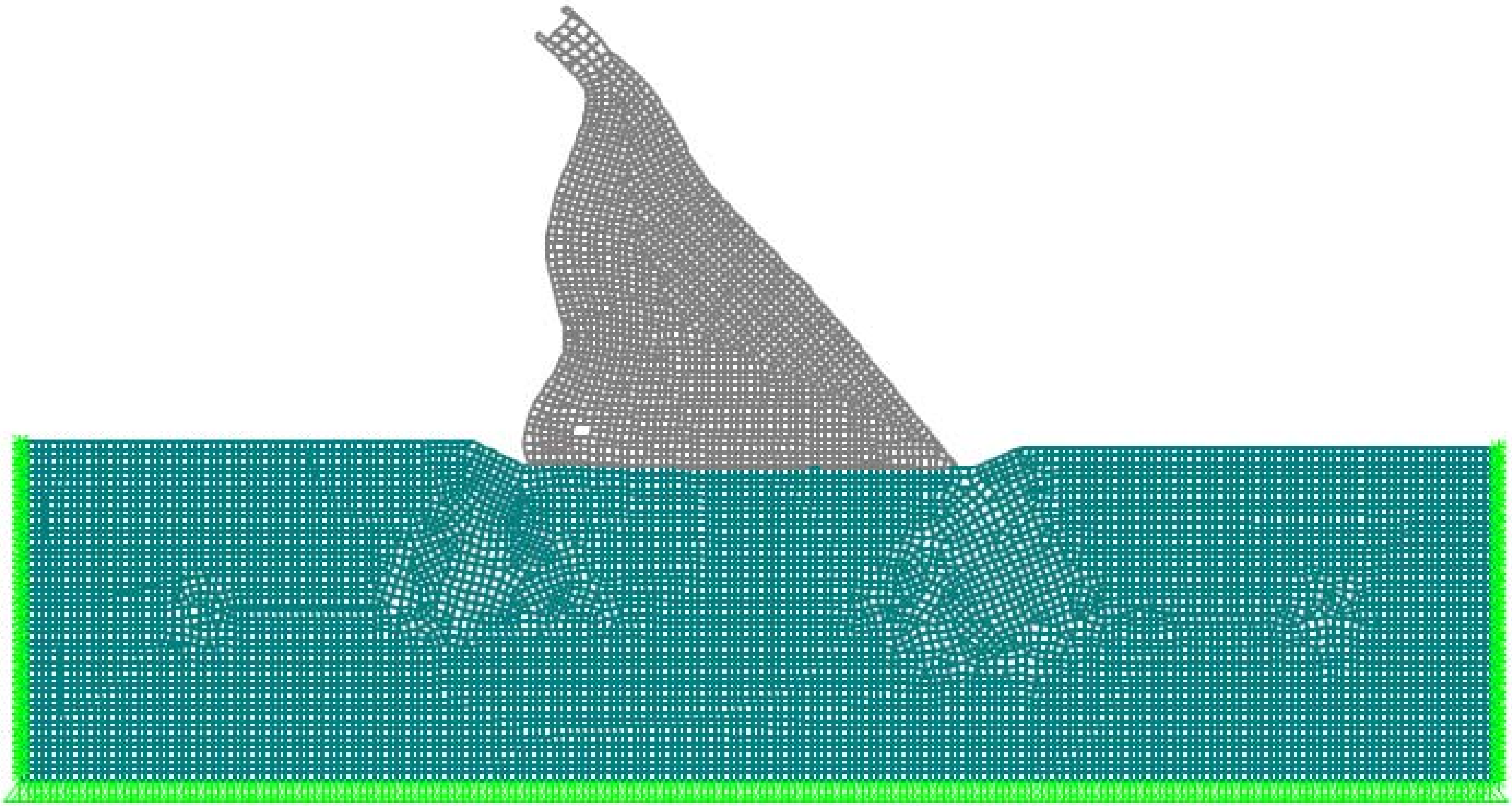
HDR	PROJECT NO. 10026522	Mode 7 Shape		FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		31
	DRAWN BY: STA			
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
	FILE NAME:	APPROVED BY: KAF/KM		

Period = 0.046 seconds



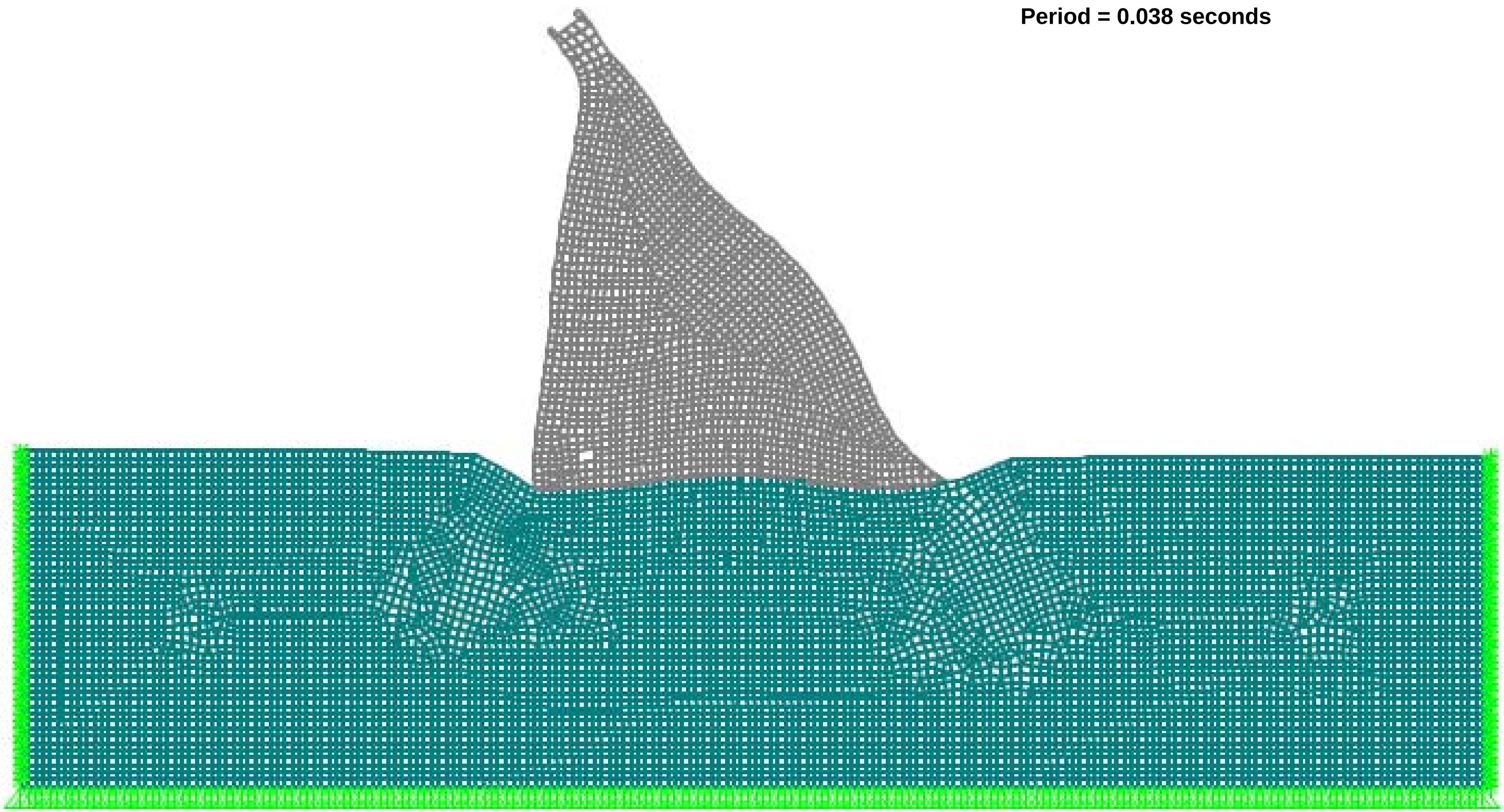
Non-Overflow Section

PROJECT NO. 10026522	Mode 8 Shape		FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		32
DRAWN BY: STA			
CHECKED BY: FA			
FILE NAME:	ORIGINATOR: STA	DRAWING CATEGORY:	
	APPROVED BY: KAF/KM		



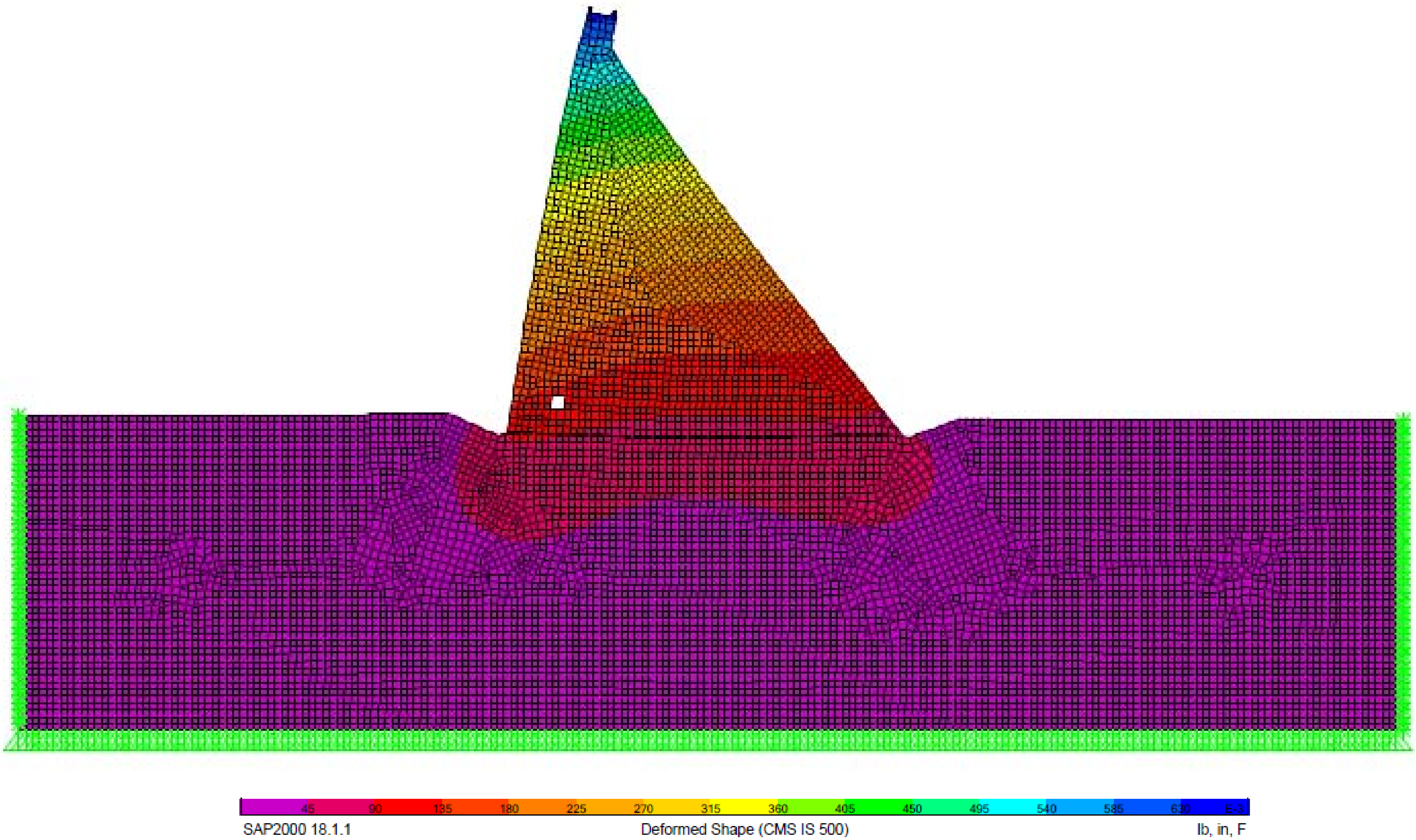
Non-Overflow Section

PROJECT NO.	10026522	Mode 9 Shape	FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	33
DRAWN BY:	STA		
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:
FILE NAME:		APPROVED BY: KAF/KM	



Non-Overflow Section

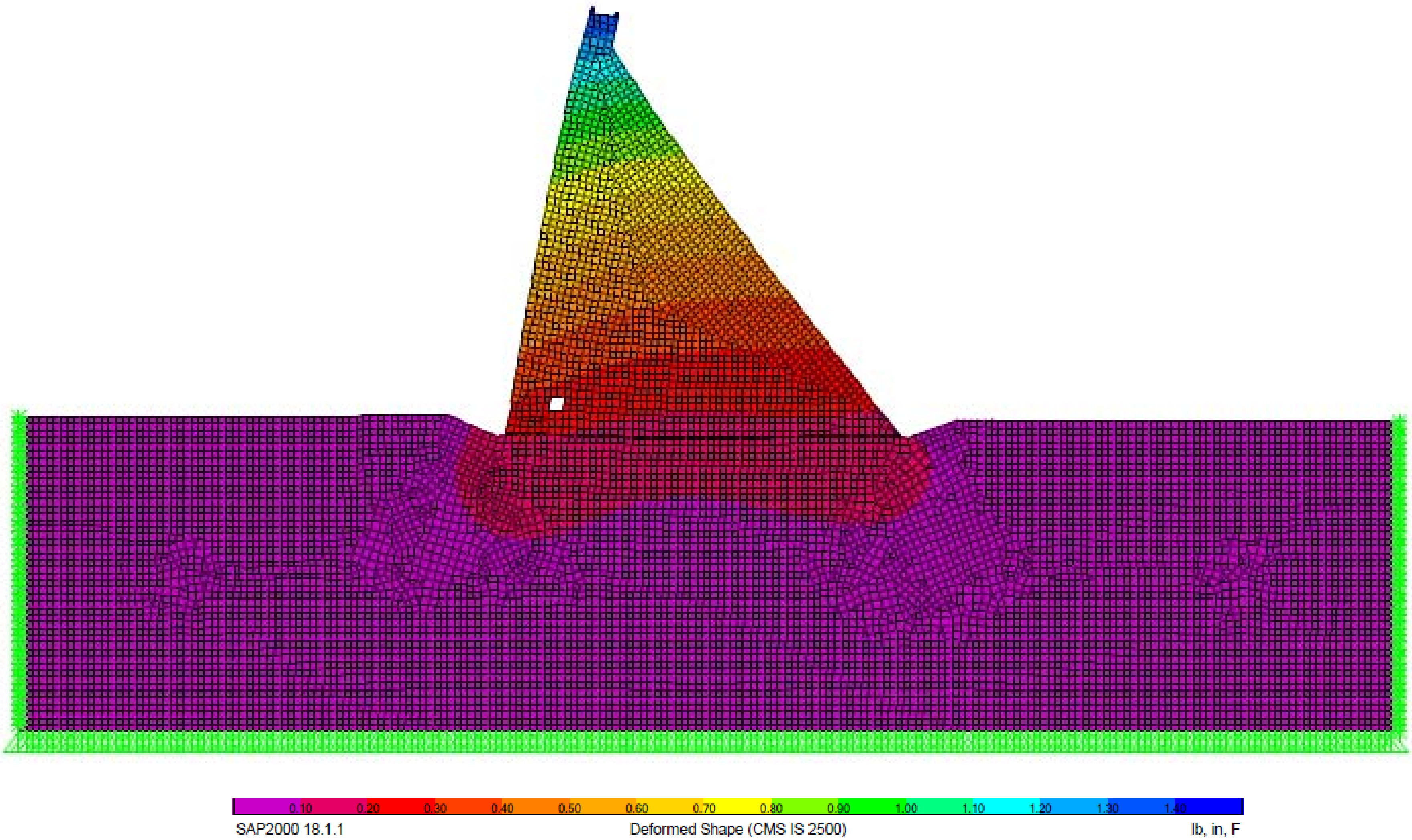
HDR	PROJECT NO. 10026522	Mode 10 Shape		FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		34
	DRAWN BY: STA			
	CHECKED BY: FA			
	FILE NAME:	ORIGINATOR: STA	DRAWING CATEGORY:	
		APPROVED BY: KAF/KM		



Scale is 10^{-3} inches

Non-Overflow Section

HDR	PROJECT NO. 10026522	Displacements CMS IF 500		FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		35
	DRAWN BY: STA			
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
	FILE NAME:	APPROVED BY: KAF/KM		

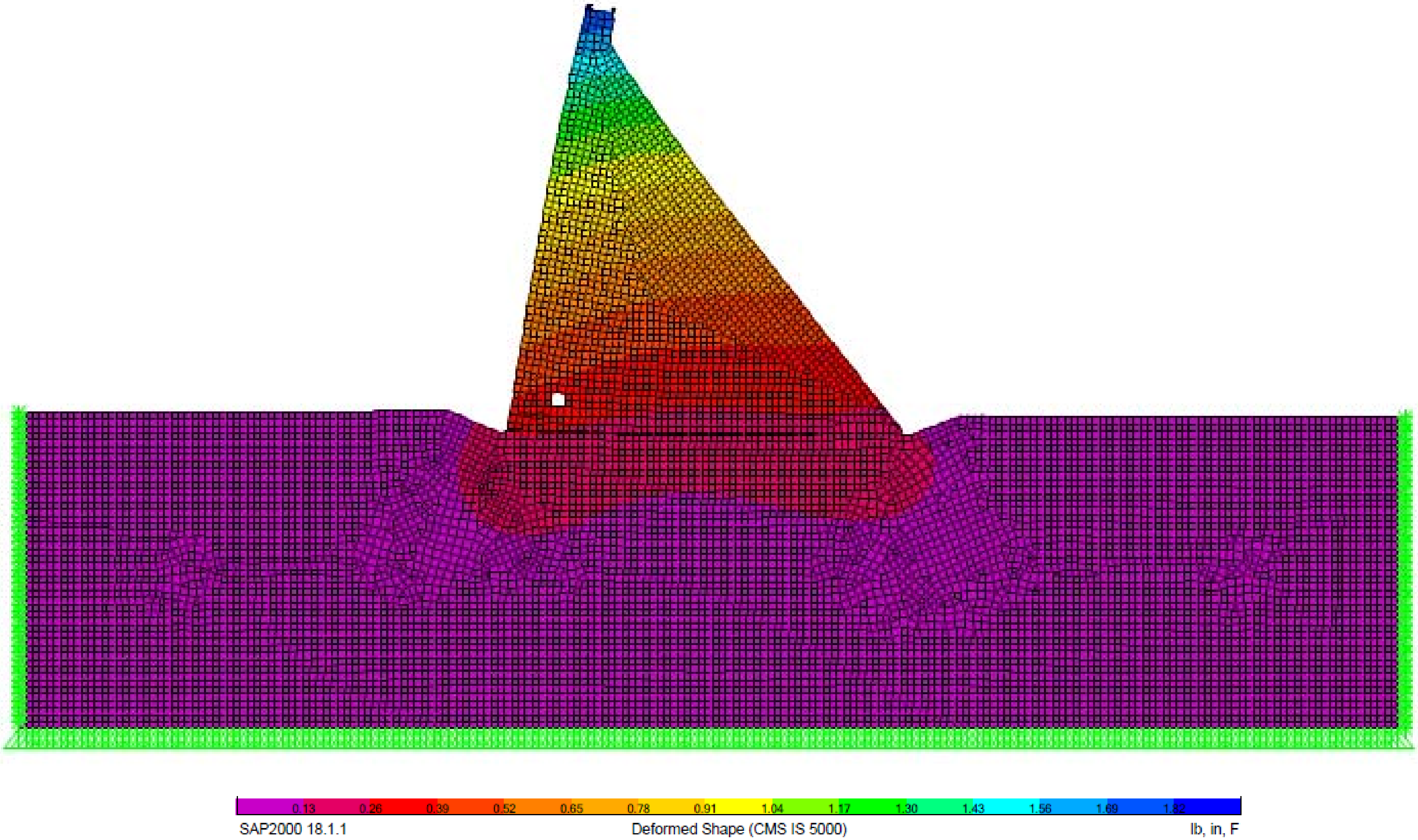


Scale is inches

Non-Overflow Section

HDR

PROJECT NO. 10026522	Displacements CMS IF 2500		FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		36
DRAWN BY: STA			
CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:	APPROVED BY: KAF/KM		

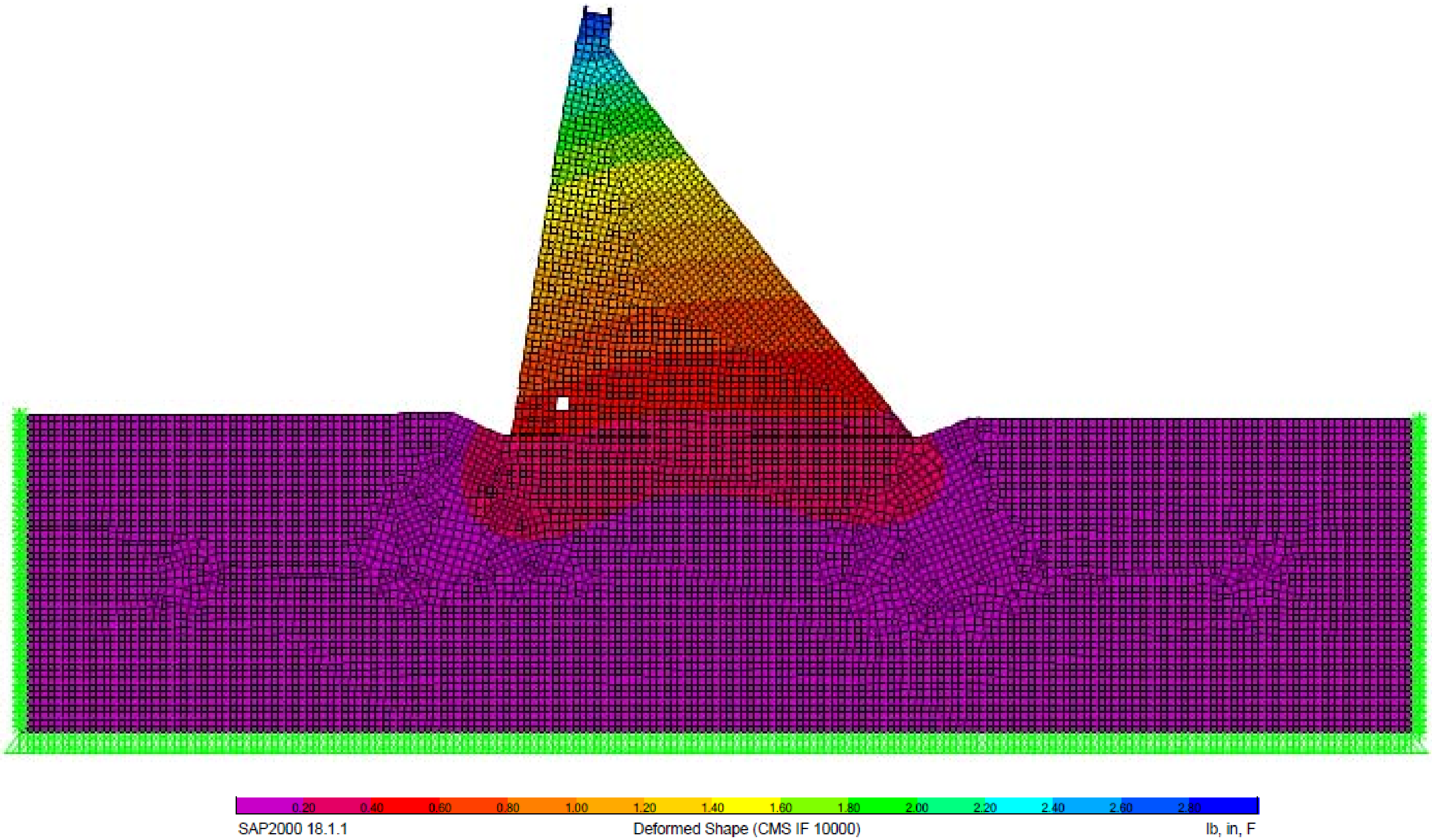


Scale is inches

Non-Overflow Section

PROJECT NO.	10026522	Displacements CMS IF 5000
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON
DRAWN BY:	STA	
CHECKED BY:	FA	ORIGINATOR: STA
FILE NAME:		APPROVED BY: KAF/KM
		DRAWING CATEGORY:

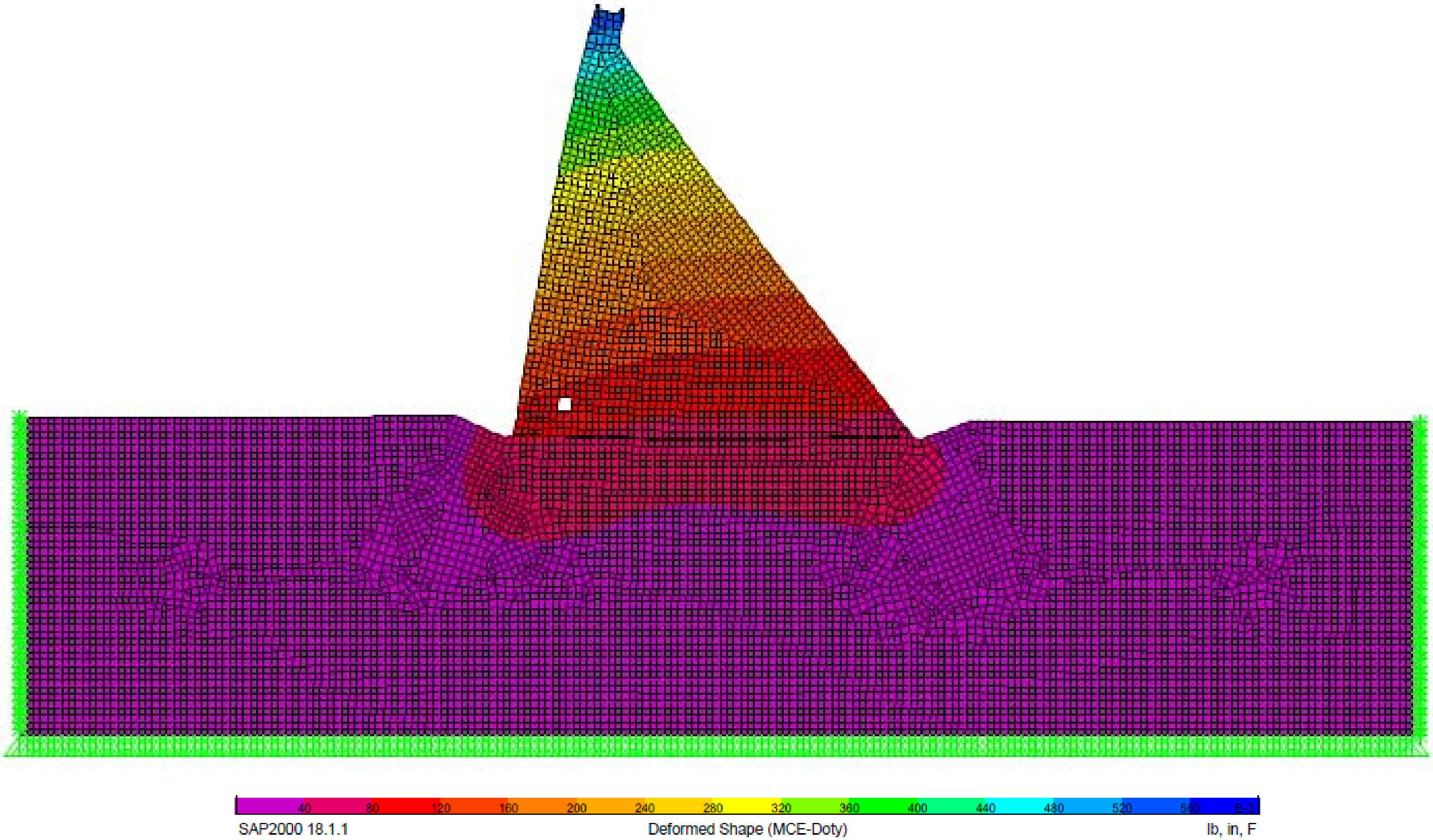
FIGURE 37



Scale is inches

Non-Overflow Section

HDR	PROJECT NO. 10026522	Displacements CMS IF 10000	FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	38
	DRAWN BY: STA		
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:
	FILE NAME:	APPROVED BY: KAF/KM	

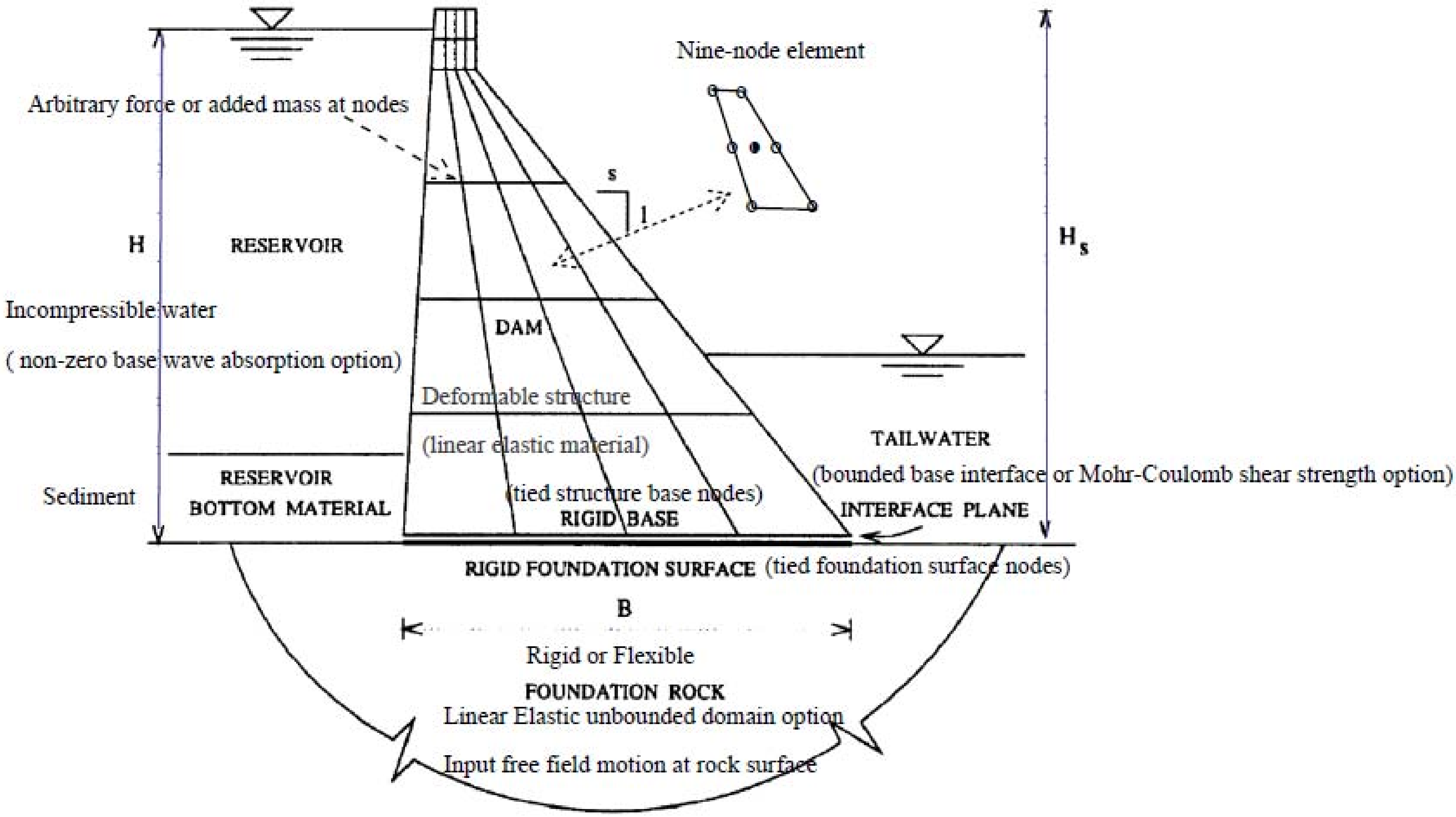


Scale is 10^{-3} inches

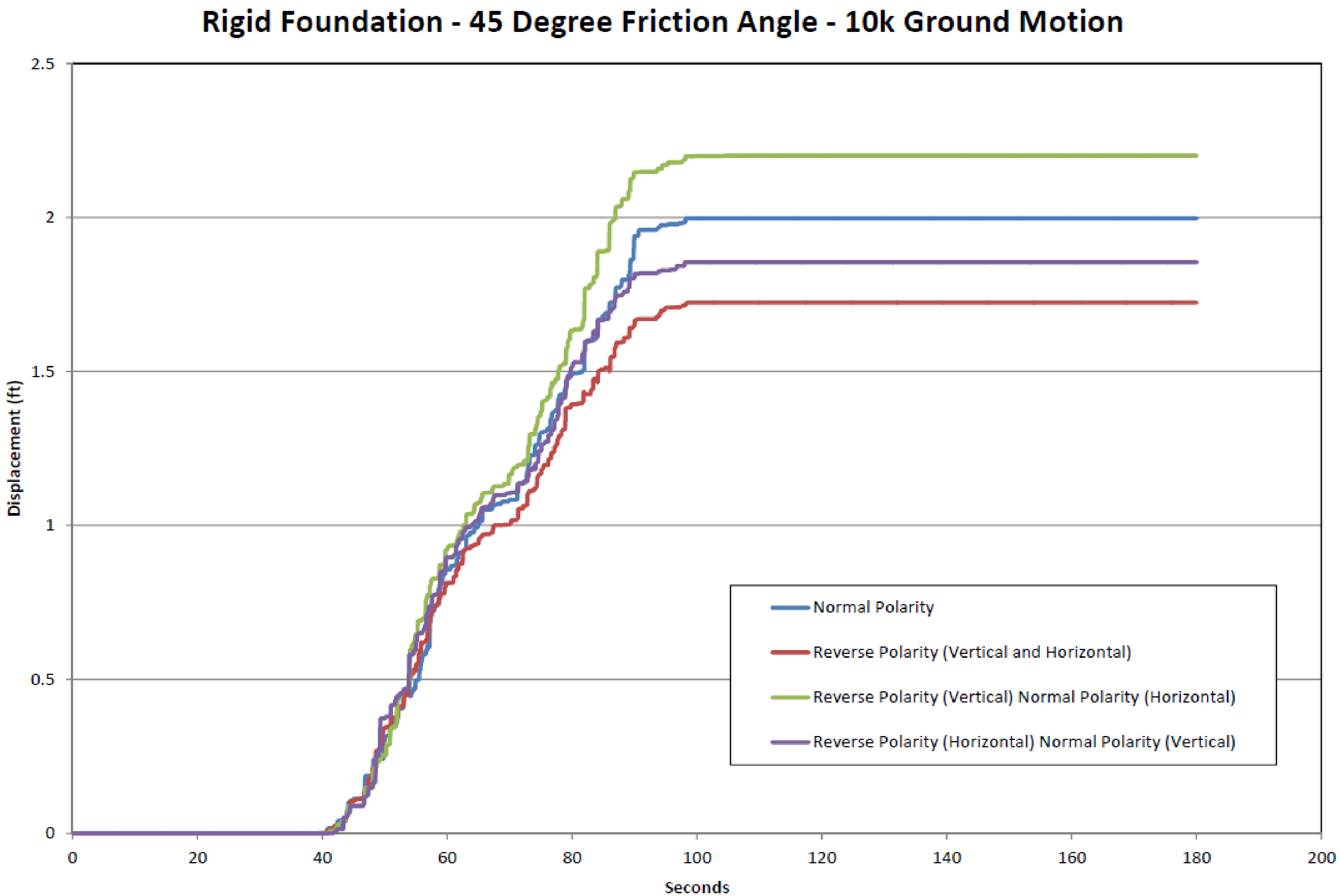
Non-Overflow Section

HDR

PROJECT NO. 10026522	Displacements MCE-Doty		FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		39
DRAWN BY: STA			
CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:	APPROVED BY: KAF/KM		

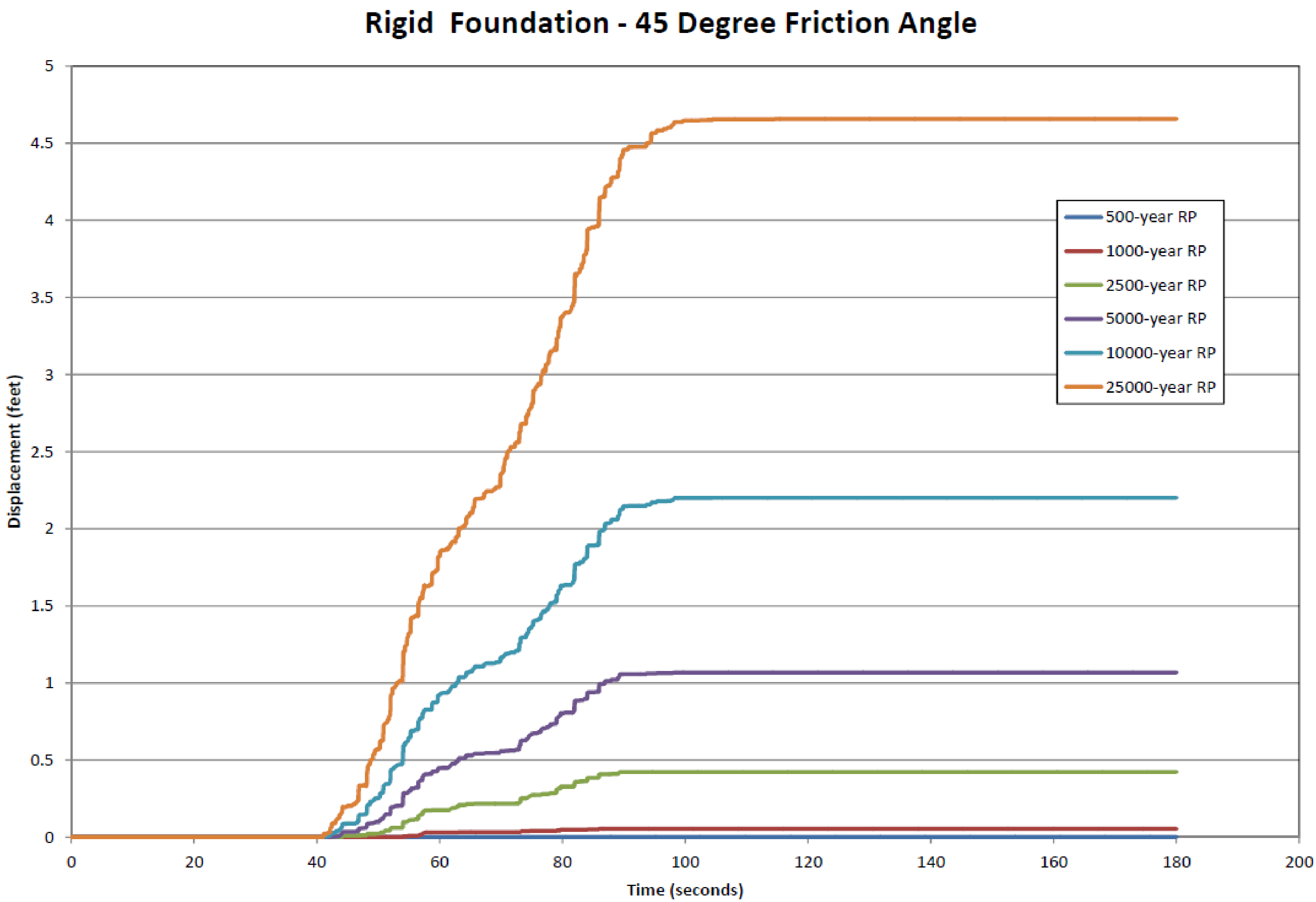


PROJECT NO.	10026522	Schematic of EAGD-SLIDE			FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON			40
DRAWN BY:	STA				
CHECKED BY:	FA	ORIGINATOR:	STA	DRAWING CATEGORY:	
FILE NAME:		APPROVED BY:	KAF/KM		



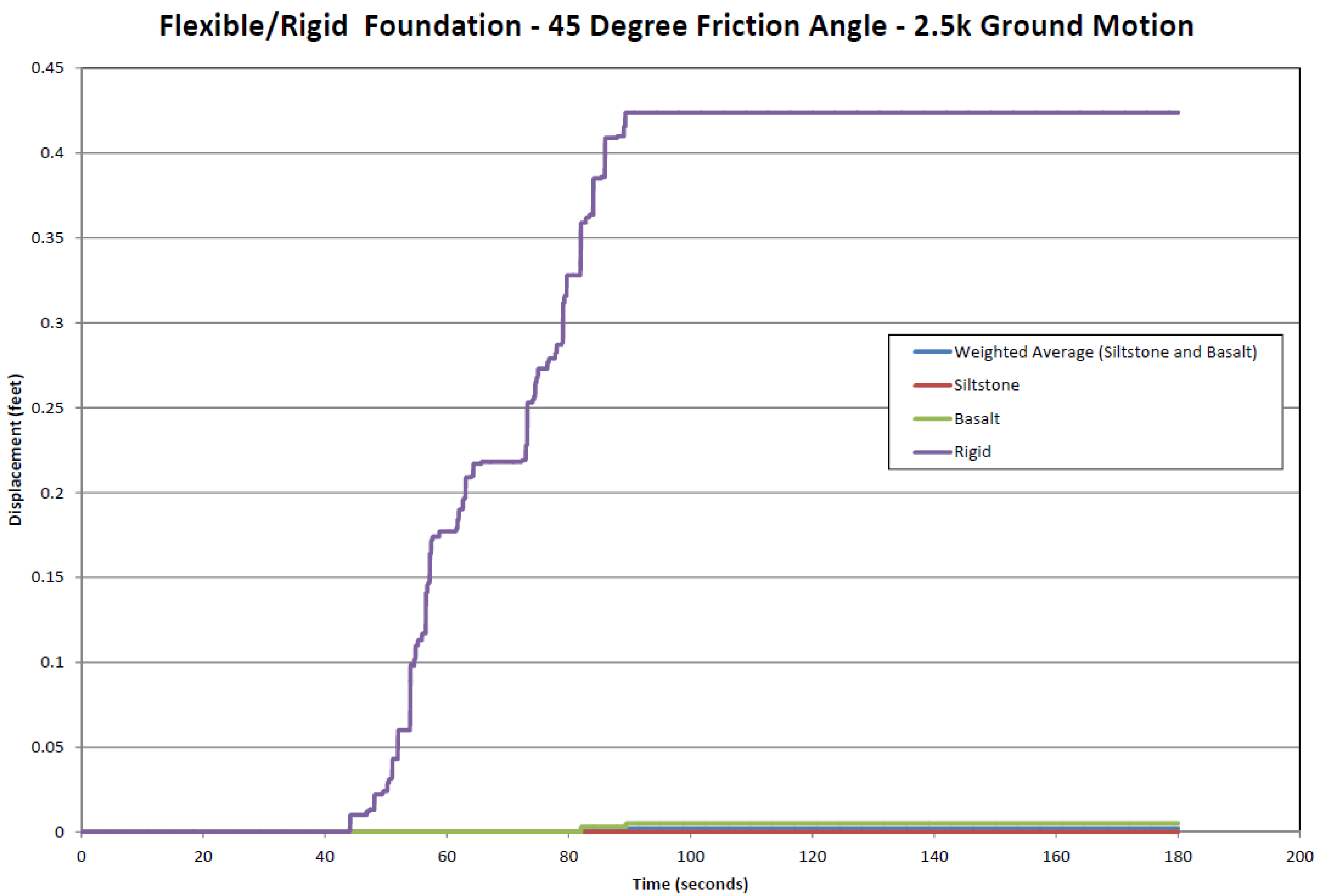
Non-Overflow Section

HDR	PROJECT NO. 10026522 DRAWN: 8/25/2016 DRAWN BY: STA CHECKED BY: FA FILE NAME:	Polarity Plot 45 Friction Angle 10k RP CHEHALIS DAM PE ELL, WASHINGTON ORIGINATOR: STA APPROVED BY: KAF/KM DRAWING CATEGORY:	FIGURE 41



Non-Overflow Section

PROJECT NO.	10026522	Rigid Foundation w/ Different Return Periods		FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		42
DRAWN BY:	STA			
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:		APPROVED BY: KAF/KM		



Non-Overflow Section

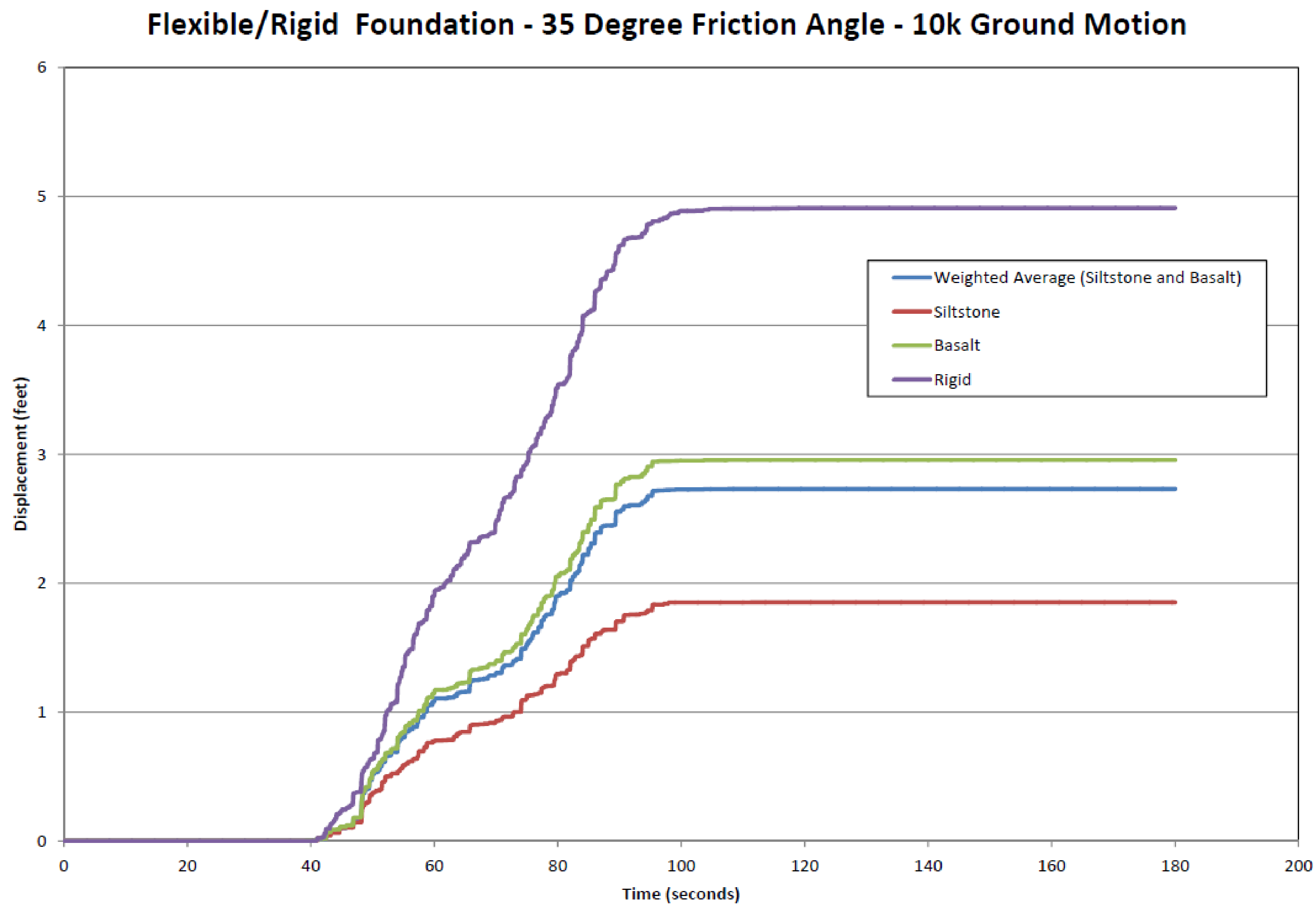
PROJECT NO.	10026522
DRAWN:	8/25/2016
DRAWN BY:	STA
CHECKED BY:	FA
FILE NAME:	

45 degree Friction Angle—2.5k RP

CHEHALIS DAM
PE ELL, WASHINGTON

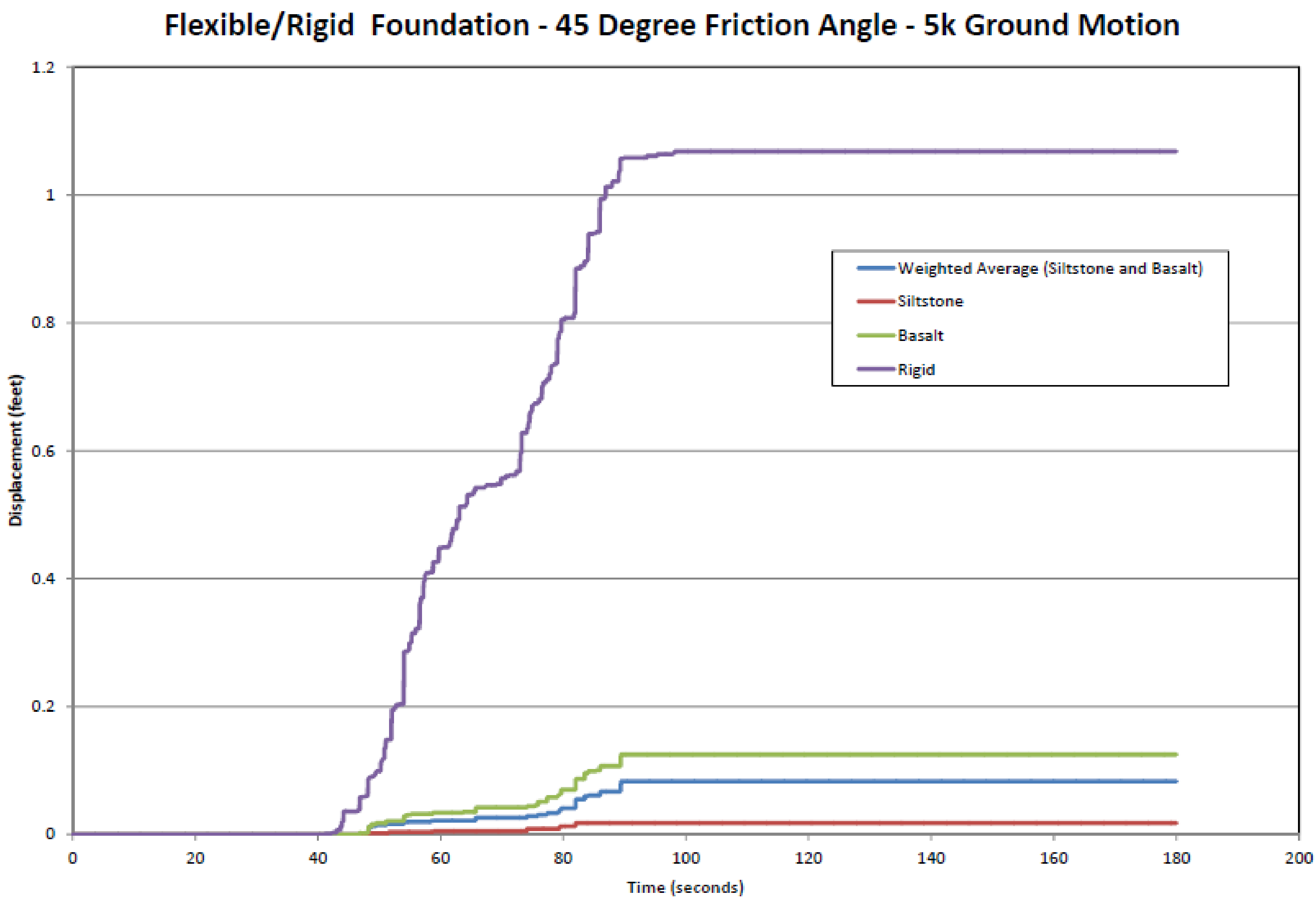
ORIGINATOR:	STA	DRAWING CATEGORY:
APPROVED BY:	KAF/KM	

FIGURE
43



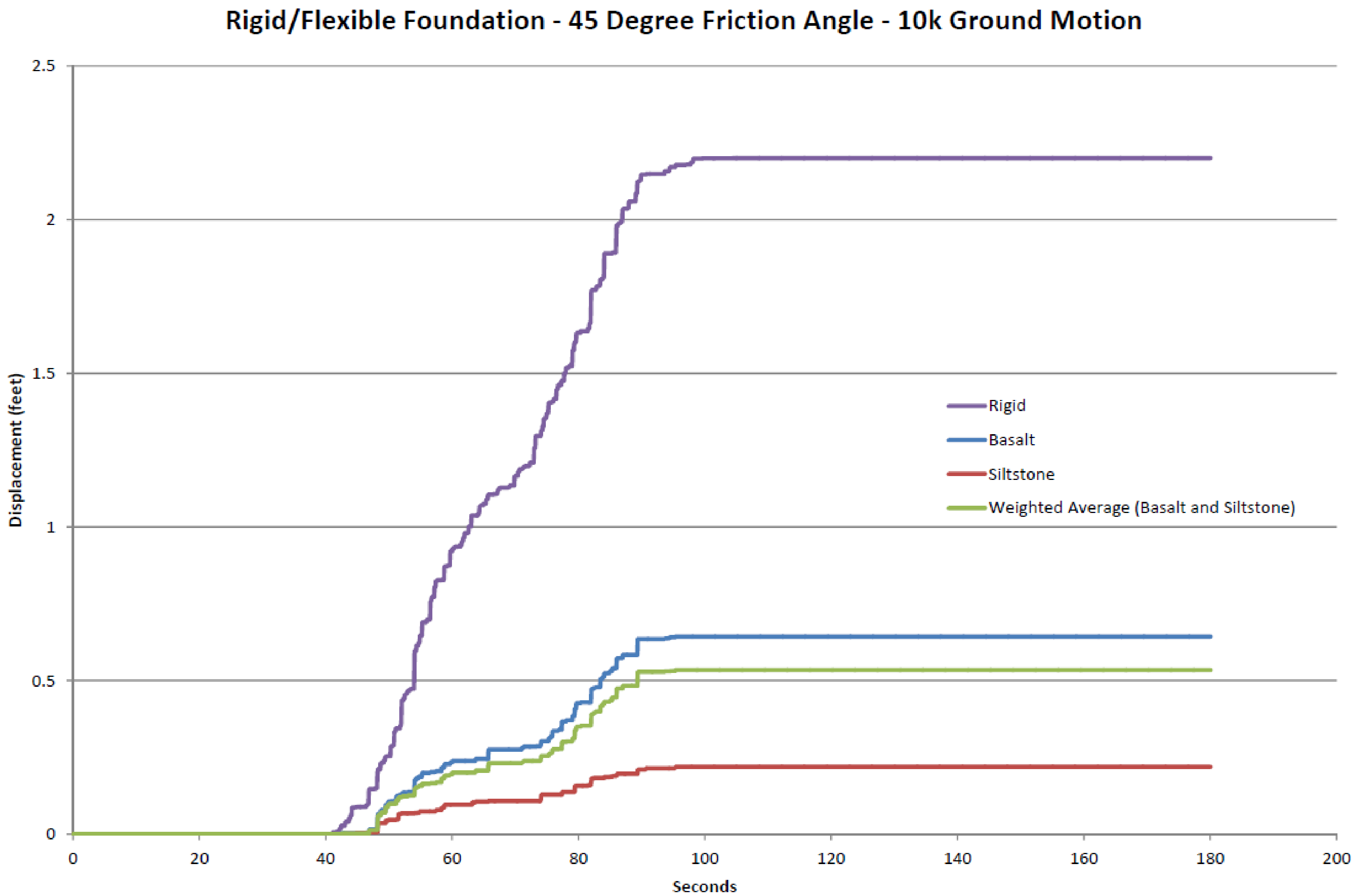
Non-Overflow Section

PROJECT NO.	10026522	35 degree Friction Angle—10k RP		FIGURE
DRAWN:	8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON		45
DRAWN BY:	STA			
CHECKED BY:	FA	ORIGINATOR: STA	DRAWING CATEGORY:	
FILE NAME:		APPROVED BY: KAF/KM		



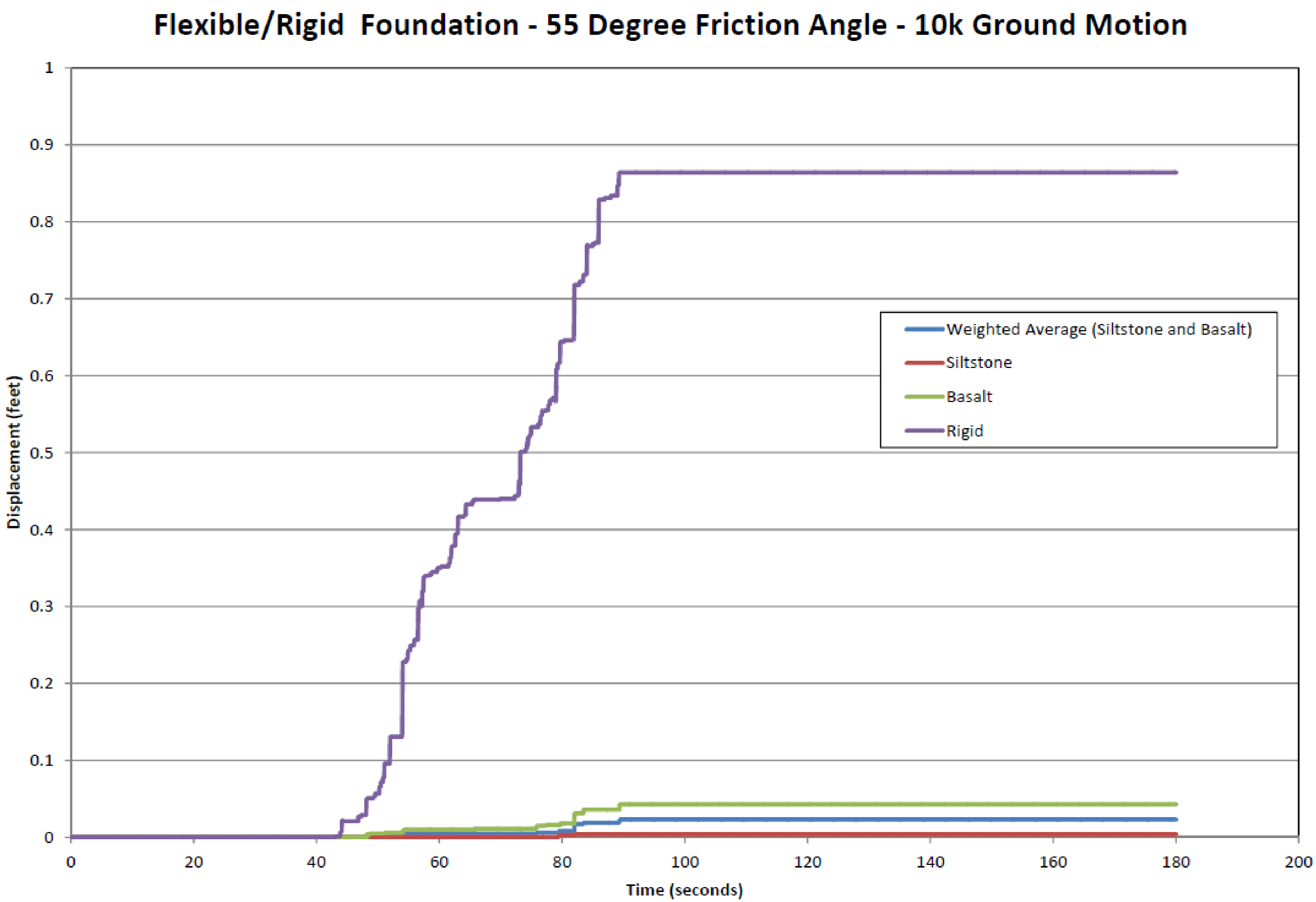
Non-Overflow Section

HDR	PROJECT NO. 10026522	45 degree Friction Angle—5k RP	FIGURE
	DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	44
	DRAWN BY: STA		
	CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:
	FILE NAME:	APPROVED BY: KAF/KM	



Non-Overflow Section

PROJECT NO. 10026522	45 degree Friction Angle—10k RP	FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON	46
DRAWN BY: STA		
CHECKED BY: FA	ORIGINATOR: STA	DRAWING CATEGORY:
FILE NAME:	APPROVED BY: KAF/KM	



Non-Overflow Section

PROJECT NO. 10026522	55 degree Friction Angle—10k RP			FIGURE
DRAWN: 8/25/2016	CHEHALIS DAM PE ELL, WASHINGTON			47
DRAWN BY: STA				
CHECKED BY: FA	ORIGINATOR:	STA	DRAWING CATEGORY:	
FILE NAME:	APPROVED BY:	KAF/KM		

Attachment A
Chopra Analysis

GRAVITY DAM STABILITY ANALYSIS

CLIENT:
PROJECT: Chehalis RCC dam
SUBJECT: Crustal source 4975 year — FILE: Block 5 Dynamic.xls
INPUT BY: STA — CHECKED BY: — DATE: 6/2/2016
REV. NOTE: L-tilde_1/M-tilde_1 value is 3 or more.

Simplified Earthquake Analysis of Concrete Gravity Dam

Chehalis RCC dam

FILE:

17-Jan-17
11:50 AM

==

Input Data

==

A. Shape and Size of Dam

Height	=======>	300.00	ft
Kink Height	=======>	287.00	ft
Crest Width	=======>	20.00	ft
Upstream Slope	=======>	0.10	
Downstream Slope	=======>	0.85	
Water Depth	=======>	218.00	ft

*spreadsheet template set tailwater zero, FEM takes 25 feet tailwater

B. Material Properties

Unit Wt. of Water	0.0624	kips/cu ft
Unit Wt. of Concrete	0.1500	kips/cu ft
Concrete Modulus	3.28E+06	psi
Foundation Modulus	2.61E+06	psi

C. Dynamic Stuff

Viscous Damping (xi_1)	0.05
Damping Coeff (eta_f)	0.10
Wave Reflection (alpha)	0.70

D. Anchor Data

Anchor force	0.00	kips/ft
Pos. D/S of U/S crest	3.33	ft

E. Shear Strength Data (to calculate post-earthquake sliding stability)

Lift Joints		
Cohesion	0.00	psi
Friction Angle	45.00	deg
Foundation Contact		
Cohesion	0.00	psi
Friction Angle	30.00	deg

Static Analysis

1. Concrete Weights and Moment Arms

Block	y/H_s	y	w1	w2	w3	w	CL	W1	x1	W2	x2	W3	x3	Total Wt.
1	0.90	270.00	3.00	20.00	14.45	37.45	18.73	6.75	-16.73	90.00	-5.73	18.42	9.09	115.17
2	0.80	240.00	6.00	20.00	39.95	65.95	32.98	27.00	-28.98	180.00	-16.98	140.82	6.34	232.65
3	0.70	210.00	9.00	20.00	65.45	94.45	47.23	60.75	-41.23	270.00	-28.23	377.97	3.59	360.90
4	0.60	180.00	12.00	20.00	90.95	122.95	61.48	108.00	-53.48	360.00	-39.48	729.87	0.84	489.15
5	0.50	150.00	15.00	20.00	116.45	151.45	75.73	168.75	-65.73	450.00	-50.73	1196.52	-1.91	617.40
6	0.40	120.00	18.00	20.00	141.95	179.95	89.98	243.00	-77.98	540.00	-61.98	1777.92	-4.66	745.65
7	0.30	90.00	21.00	20.00	167.45	208.45	104.23	330.75	-90.23	630.00	-73.23	2474.07	-7.41	873.90
8	0.20	60.00	24.00	20.00	192.95	236.95	118.48	432.00	-102.48	720.00	-84.48	3284.97	-10.16	1002.15
9	0.10	30.00	27.00	20.00	218.45	265.45	132.73	546.75	-114.73	810.00	-95.73	4210.62	-12.91	1130.40
10	0.00	0.00	30.00	20.00	243.95	293.95	146.98	675.00	-126.98	900.00	-106.98	5251.02	-15.66	1258.65

2. Reservoir and Uplift Forces and Moment Arms, Anchor Force and Moment Arm

Block	y/H_s	y	P	yP	U	xU	WH	xWH	||	Anchor Force	xA
1	0.90	270.00	0.00	0.00	0.00	0.00	0.00	0.00	||	0.00	-12.40
2	0.80	240.00	0.00	0.00	0.00	0.00	0.00	0.00	||	0.00	-23.65
3	0.70	210.00	2.00	2.67	23.57	-15.74	0.20	-46.96	||	0.00	-34.90
4	0.60	180.00	45.05	12.67	145.77	-20.49	4.51	-60.21	||	0.00	-46.15
5	0.50	150.00	144.27	22.67	321.32	-25.24	14.43	-73.46	||	0.00	-57.40
6	0.40	120.00	299.64	32.67	550.22	-29.99	29.96	-86.71	||	0.00	-68.65
7	0.30	90.00	511.18	42.67	832.47	-34.74	51.12	-99.96	||	0.00	-79.90
8	0.20	60.00	778.88	52.67	1168.07	-39.49	77.89	-113.21	||	0.00	-91.15
9	0.10	30.00	1102.73	62.67	1557.02	-44.24	110.27	-126.46	||	0.00	-102.40
10	0.00	0.00	1482.75	72.67	1999.33	-48.99	148.27	-139.71	||	0.00	-113.65

3. Total Forces, Overturning Moment and Streses

Block	y/H_s	y	SigFH	SigFV	SigM	e	U/S SIG_st (ksf)	U/S SIG_st (psi)	D/S SIG_st (ksf)	D/S SIG_st (psi)	TAU (ksf)	TAU (psi)
1	0.90	270.00	0.00	115.17	-461	-4.00	5.05	35.04	1.10	7.67	0.00	6.15
2	0.80	240.00	0.00	347.82	-2945	-8.47	9.34	64.84	1.21	8.41	0.00	10.55
3	0.70	210.00	2.00	685.35	-8401	-12.26	12.91	89.63	1.61	11.15	0.02	14.51
4	0.60	180.00	45.05	1056.61	-16086	-15.22	14.98	104.02	2.21	15.34	0.37	17.19
5	0.50	150.00	144.27	1508.38	-25880	-17.16	16.73	116.18	3.19	22.15	0.95	19.92
6	0.40	120.00	299.64	2040.67	-37004	-18.13	18.20	126.37	4.48	31.14	1.67	22.68
7	0.30	90.00	511.18	2653.48	-48680	-18.35	19.45	135.08	6.01	41.72	2.45	25.46
8	0.20	60.00	778.88	3346.79	-60129	-17.97	20.55	142.71	7.70	53.46	3.29	28.25
9	0.10	30.00	1102.73	4120.62	-70570	-17.13	21.53	149.53	9.51	66.07	4.15	31.05
10	0.00	0.00	1482.75	4974.97	-79226	-15.92	22.43	155.74	11.42	79.33	5.04	33.85

==
Dynamic Analysis -- Part I: Loads Due to Fundamental Mode
==

Step I-1: Fundamental Period, Rigid Foundation

1. Result:

T1 =			0.2319	seconds

Step I-2: Fundamental Period, Influence of Reservoir

1. Input:

Period Ratio R_r (Table 2a) =	for H/Hs=	0.73	**1.054**	nondimensional
Added Damping xi_r (Table 2a) =	with alpha=	0.70	**0.030**	nondimensional

2. Result:

T-tilde_r =			0.2444	seconds

Step I-3: Period Ratio, Reservoir-Dam

1. Result:

T_1^r =			0.1847	seconds
R_w =	for Ef/Es=	0.80	**0.7558**	nondimensional

Step I-4: Fundamental Period, Influence of Foundation & Reservoir

1. Input:

Peroid ratio for fndn R_f (Table 3) =	eta_f=	0.10	**1.248**	nondimensional
Damping from fndn. xi_f (Table 3) =			**0.093**	nondimensional
Damping coeff. at fndn. eta_f (Table 3) =			0.1000	

2. Result:

T-tilde_1 =			**0.3050**	seconds

Step I-5: Damping Ratio, Influence of Foundation & Reservoir

1. Result:

xi-tilde_1 = **0.1474** nondimensional

Step I-6: Hydrodynamic Pressure Function

1. Input: Fill in column D using data from Table 4

y/Hs	y	y-hat	gp(y-hat) / wH	gp(y,T-tilde_r)	phi	
=======	=======	=======	=======	=======	=======	
1.00	300.00	1.38	0.000	0.000	1.000	0.00
0.90	270.00	1.24	0.118	0.848	0.735	6.11
0.80	240.00	1.10	0.144	1.034	0.530	5.37
0.70	210.00	0.96	0.161	1.156	0.389	4.41
0.60	180.00	0.83	0.159	1.142	0.284	3.18
0.50	150.00	0.69	0.159	1.142	0.200	2.24
0.40	120.00	0.55	0.150	1.077	0.135	1.43
0.30	90.00	0.41	0.146	1.049	0.084	0.86
0.20	60.00	0.28	0.139	0.998	0.047	0.46
0.10	30.00	0.14	0.136	0.977	0.021	0.20
0.00	0.00	0.00	0.133	0.955	0.000	0.00

2. Result: gp(y,T-tilde_r) in column E

Step I-7: Generalized Mass

1. Result: M_1, L_1 and M-tilde_1 below

y/Hs	y	w_s * t	phi	w_s * phi^2	w_s * phi
=======	=======	=======	=======	=======	=======
0.95	285.00	104.400	0.866	78.295	90.410
0.85	255.00	232.650	0.619	89.142	144.010
0.75	225.00	360.900	0.455	74.715	164.210
0.65	195.00	489.150	0.334	54.568	163.376
0.55	165.00	617.400	0.240	35.562	148.176
0.45	135.00	745.650	0.165	20.300	123.032
0.35	105.00	873.900	0.108	10.193	94.381
0.25	75.00	1002.150	0.065	4.234	65.140
0.15	45.00	1130.400	0.034	1.307	38.434
0.05	15.00	1258.650	0.010	0.126	12.587
				=======	=======
				368.443	1043.756

M_1 = 368.44 kips/gravity
L_1 = 1043.76 kips/gravity
M-tilde_1 = 409.31 kips/gravity

Step I-8: Genaralized Earthquake Force Coefficient

1. Input: Hydrodynamic Force Coeff Ap (Table 5) = **0.279** nondimensional

2. Result:

Hydrodyanam force Fst = 1482.75 kips/ft of width
L-tilde_1 = 1262.20 kips/gravity
Conservative L-tilde_1/M-tilde_1= 3.08

Step I-9: Equivalent Lateral Earthquake Forces

1. Input:

Pseudo Accel. Ordinate Sa = **0.901** g
(FOR CHECK IT IS FROM 5% response spectrum, can be assigned with higher damping)

2. Result: Equivalent Lateral Force in column F

y/Hs	y	w_s	phi	gp(I-6)	f_1
=======	=======	=======	=======	=======	=======
1.00	300.00	3.000	1.000	0.000	8.34
0.90	270.00	5.618	0.735	0.848	13.83
0.80	240.00	9.893	0.530	1.034	17.44
0.70	210.00	14.168	0.389	1.156	18.53
0.60	180.00	18.443	0.284	1.142	17.73
0.50	150.00	22.718	0.200	1.142	15.80
0.40	120.00	26.993	0.135	1.077	13.12
0.30	90.00	31.268	0.084	1.049	10.21
0.20	60.00	35.543	0.047	0.998	7.42
0.10	30.00	39.818	0.021	0.977	5.04
0.00	0.00	44.093	0.000	0.955	2.65

Step I-10: Response due to Fundamental Vibration Mode

1. Result: Shear force F_H1 and SIG_1 in table below

Block	y/H_s	y	w(y)	S(y)	F_H1	Moment	SIG_1 (ksf)	SIG_1 (psi)
1	0.90	270.00	37.45	233.75	332.43	4482	19.2	133.14
2	0.80	240.00	65.95	724.90	801.46	20844	28.8	199.68
3	0.70	210.00	94.45	1486.80	1340.96	52500	35.3	245.22
4	0.60	180.00	122.95	2519.45	1884.73	100529	39.9	277.09
5	0.50	150.00	151.45	3822.85	2387.58	164353	43.0	298.56
6	0.40	120.00	179.95	5397.00	2821.31	242300	44.9	311.77
7	0.30	90.00	208.45	7241.90	3171.25	332060	45.9	318.42
8	0.20	60.00	236.95	9357.55	3435.66	431079	46.1	319.91
9	0.10	30.00	265.45	11743.95	3622.45	536898	45.7	317.48
10	0.00	0.00	293.95	14401.10	3737.83	647273	44.9	312.13

Computation of Moments for column G in table above:

	Block	Arm==>	13.48	13.62	14.11	14.34	14.48	14.57	14.63	14.68	14.72	14.75
Moment	Number	Force=>	332.43	469.02	539.51	543.77	502.85	433.73	349.94	264.40	186.80	115.38
4482	1	||	4482									
20844	2	||	14455	6389								
52500	3	||	24428	20460	7613							
100529	4	||	34401	34530	23798	7800						
164353	5	||	44374	48601	39984	24113	7282					
242300	6	||	54346	62672	56169	40427	22367	6319				
332060	7	||	64319	76742	72354	56740	37452	19331	5121			
431079	8	||	74292	90813	88539	73053	52538	32343	15619	3881		
536898	9	||	84265	104884	104724	89366	67623	45355	26117	11814	2749	
647273	10	||	94238	118954	120909	105679	82708	58367	36616	19746	8353	1701

===

Dynamic Analysis -- Part II: Loads Due to Higher Modes of Vibration

===

Step II-11: Lateral Forces for Higher Modes

1. Input:

a_g from response spectrum = **0.435** g
Enter gP_0/wH (Table 6) in column F

2. Result:

B_1 = 156.59
<A> = w_s *[1 - (L_1/M_1)*phi]
<B> = (B_1/M_1) * w_s * phi

==> f_sc = (a_g/g) * [<A> + gP_0 - <B>] (Column J)

y/Hs	y	w_s	phi	y/H	gP_0 / wH	gP_0	<A>	<B>	f_sc
1.00	300.00	3.000	1.000	1.38	**0.000**	0.000	-5.499	1.275	-2.947
0.90	270.00	5.618	0.735	1.24	**0.137**	1.864	-6.079	1.755	-2.597
0.80	240.00	9.893	0.530	1.10	**0.350**	4.761	-4.960	2.228	-1.056
0.70	210.00	14.168	0.389	0.96	**0.456**	6.203	-1.445	2.342	1.051
0.60	180.00	18.443	0.284	0.83	**0.300**	4.081	3.605	2.226	2.375
0.50	150.00	22.718	0.200	0.69	**0.580**	7.890	9.846	1.931	6.875
0.40	120.00	26.993	0.135	0.55	**0.659**	8.965	16.670	1.549	10.477
0.30	90.00	31.268	0.084	0.41	**0.690**	9.386	23.827	1.116	13.962
0.20	60.00	35.543	0.047	0.28	**0.722**	9.822	30.810	0.710	17.366
0.10	30.00	39.818	0.021	0.14	**0.737**	10.026	37.449	0.355	20.497
0.00	0.00	44.093	0.000	0.00	**0.742**	10.094	44.093	0.000	23.571

Step II-12: Response due to Higher Modes of Vibration

1. Result: Shear force F_Hsc and normal stress SIG_sc in table below

Block	y/H_s	y	w(y)	S(y)	F_Hsc	Moment	SIG_sc (ksf)	SIG_sc (psi)
=========	=======	=======	=======	=======	=======	=======	=======	=======
1	0.90	270.00	37.45	233.75	-83.154	-1121	-4.8	-33.30
2	0.80	240.00	65.95	724.90	-137.950	-4362	-6.0	-41.79
3	0.70	210.00	94.45	1486.80	-138.026	-8502	-5.7	-39.71
4	0.60	180.00	122.95	2519.45	-86.639	-11905	-4.7	-32.81
5	0.50	150.00	151.45	3822.85	52.114	-12495	-3.3	-22.70
6	0.40	120.00	179.95	5397.00	312.400	-7139	-1.3	-9.19
7	0.30	90.00	208.45	7241.90	678.989	7597	1.0	7.28
8	0.20	60.00	236.95	9357.55	1148.911	34865	3.7	25.87
9	0.10	30.00	265.45	11743.95	1716.851	77690	6.6	45.94
10	0.00	0.00	293.95	14401.10	2377.866	138943	9.6	67.00

Computation of Moments for column G in table above:

	Block	Arm==>	13.48	13.62	14.11	14.34	14.48	14.57	14.63	14.68	14.72	14.75
Moment	Number	Force=>	-83.15	-54.80	-0.08	51.39	138.75	260.29	366.59	469.92	567.94	661.02
------	------	------	------	------	------	------	------	------	------	------	------	------
-1121	1	||	-1121									
-4362	2	||	-3616	-746								
-8502	3	||	-6110	-2390	-1							
-11905	4	||	-8605	-4034	-3	737						
-12495	5	||	-11100	-5678	-6	2279	2009					
-7139	6	||	-13594	-7322	-8	3820	6172	3792				
7597	7	||	-16089	-8966	-10	5362	10334	11601	5364			
34865	8	||	-18583	-10610	-13	6904	14497	19409	16362	6898		
77690	9	||	-21078	-12254	-15	8445	18660	27218	27360	20996	8358	
138943	10	||	-23573	-13897	-17	9987	22822	35027	38357	35094	25396	9747

==
Part III: Response due to Static and Earthquake Loads
==

III-1: Sliding Stability

1. Result:

Maximum horizontal shear force Fmax and shear stress TAUmax
Note: TAUmax assumes uniform stress along entire length of surface w

Block	y/H_s	y	w	F_Hst	F_H1	F_Hsc	F_Hmax	TAUmax
=========	=======	=======	=======	=======	=======	=======	=======	=======
1	0.90	270.00	37.45	0.00	332.43	-83.154	342.67	9.15
2	0.80	240.00	65.95	0.00	801.46	-137.950	813.24	12.33
3	0.70	210.00	94.45	2.00	1340.96	-138.026	1350.04	14.29
4	0.60	180.00	122.95	45.05	1884.73	-86.639	1931.78	15.71
5	0.50	150.00	151.45	144.27	2387.58	52.114	2532.42	16.72
6	0.40	120.00	179.95	299.64	2821.31	312.400	3138.20	17.44
7	0.30	90.00	208.45	511.18	3171.25	678.989	3754.31	18.01
8	0.20	60.00	236.95	778.88	3435.66	1148.911	4401.55	18.58
9	0.10	30.00	265.45	1102.73	3622.45	1716.851	5111.44	19.26
10	0.00	0.00	293.95	1482.75	3737.83	2377.866	5912.83	20.12

III-2: Maximum and Minumum Normal Stress at U/S and D/S Face

1. Result:

SY_max and SY_min at U/S and D/S face (Columns I,J,K, and L)

			U/S	D/S	Earthquake Loads			U/S Face		D/S Face		D/S	U/S
Block	y/H_s	y	SIG_st	SIG_st	SIG_1	SIG_sc	SIG_d	SY_max	SY_min	SY_max	SY_min	Crack	Crack
=========	=======	=======	=======	=======	=======	=======	=======	=======	=======	=======	=======	=======	=======
1	0.90	270.00	5.05	1.10	19.17	-4.80	19.76	24.81	-14.72	20.87	-18.66	0.00	0.00
2	0.80	240.00	9.34	1.21	28.75	-6.02	29.38	38.71	-20.04	30.59	-28.16	0.00	0.00
3	0.70	210.00	12.91	1.61	35.31	-5.72	35.77	48.68	-22.86	37.38	-34.16	38.95	0.00
4	0.60	180.00	14.98	2.21	39.90	-4.73	40.18	55.16	-25.20	42.39	-37.97	50.13	0.00
5	0.50	150.00	16.73	3.19	42.99	-3.27	43.12	59.85	-26.39	46.31	-39.93	60.61	0.00
6	0.40	120.00	18.20	4.48	44.90	-1.32	44.91	63.11	-26.72	49.40	-40.43	70.27	0.00
7	0.30	90.00	19.45	6.01	45.85	1.05	45.86	65.32	-26.41	51.87	-39.86	79.00	0.00
8	0.20	60.00	20.55	7.70	46.07	3.73	46.22	66.77	-25.67	53.92	-38.52	86.69	0.00
9	0.10	30.00	21.53	9.51	45.72	6.62	46.19	67.73	-24.66	55.71	-36.68	93.26	0.00
10	0.00	0.00	22.43	11.42	44.95	9.65	45.97	68.40	-23.54	57.39	-34.55	98.65	0.00

III-3: Maximum and Minimum Principal Stresses at U/S and D/S Faces

1. Hydrostatic and Hydrodynamic Loads at Upstream Face
===> p_d is resultant pressure at elevation y

Block	y/H_s	y	p_st	p_1	p_sc	p_d
1	0.90	270.00	-3.24	2.36	0.05	-0.89
2	0.80	240.00	-1.37	2.87	1.10	1.71
3	0.70	210.00	0.50	3.21	1.68	4.12
4	0.60	180.00	2.37	3.17	0.81	5.65
5	0.50	150.00	4.24	3.17	2.59	8.34
6	0.40	120.00	6.12	2.99	3.23	10.52
7	0.30	90.00	7.99	2.91	3.60	12.62
8	0.20	60.00	9.86	2.77	3.96	14.70
9	0.10	30.00	11.73	2.71	4.21	16.74
10	0.00	0.00	13.60	2.65	4.39	18.73

Maximum and minimim principal stresses SIG_max and SIG_min
at U/S and D/S faces of the dam

	<<<<<<<<<<<<<<<< U/S Face >>>>>>>>>>>>>>>>								<<<<<<<<< D/S Face >>>>>>>>				
Block	y/H_s	y	Theta	p_d	SY_max	SY_min	SIG_max	SIG_min	Theta	SY_max	SY_min	SIG_max	SIG_min
1	0.90	270.00	0.10	-0.89	24.81	-14.72	25.07	-14.86	0.4489	20.87	-18.66	25.71	-22.99
2	0.80	240.00	0.10	1.71	38.71	-20.04	39.08	-20.26	0.7045	30.59	-28.16	52.69	-48.51
3	0.70	210.00	0.10	4.12	48.68	-22.86	49.12	-23.13	0.7045	37.38	-34.16	64.38	-58.85
4	0.60	180.00	0.10	5.65	55.16	-25.20	55.65	-25.51	0.7045	42.39	-37.97	73.02	-65.40
5	0.50	150.00	0.10	8.34	59.85	-26.39	60.36	-26.73	0.7045	46.31	-39.93	79.76	-68.77
6	0.40	120.00	0.10	10.52	63.11	-26.72	63.64	-27.09	0.7045	49.40	-40.43	85.09	-69.64
7	0.30	90.00	0.10	12.62	65.32	-26.41	65.84	-26.80	0.7045	51.87	-39.86	89.35	-68.65
8	0.20	60.00	0.10	14.70	66.77	-25.67	67.29	-26.07	0.7045	53.92	-38.52	92.87	-66.35
9	0.10	30.00	0.10	16.74	67.73	-24.66	68.24	-25.07	0.7045	55.71	-36.68	95.96	-63.18
10	0.00	0.00	0.10	18.73	68.40	-23.54	68.89	-23.97	0.7045	57.39	-34.55	98.86	-59.51

==

Post Earthquake Static Analysis

==

1. Concrete Weights and Moment Arms

Block	y/H_s	y	w1	w2	w3	w	CL	W1	x1	W2	x2	W3	x3	Total Wt.
1	0.90	270.00	3.00	20.00	14.45	37.45	18.73	6.75	-16.73	90.00	-5.73	18.42	9.09	115.17
2	0.80	240.00	6.00	20.00	39.95	65.95	32.98	27.00	-28.98	180.00	-16.98	140.82	6.34	232.65
3	0.70	210.00	9.00	20.00	65.45	94.45	47.23	60.75	-41.23	270.00	-28.23	377.97	3.59	360.90
4	0.60	180.00	12.00	20.00	90.95	122.95	61.48	108.00	-53.48	360.00	-39.48	729.87	0.84	489.15
5	0.50	150.00	15.00	20.00	116.45	151.45	75.73	168.75	-65.73	450.00	-50.73	1196.52	-1.91	617.40
6	0.40	120.00	18.00	20.00	141.95	179.95	89.98	243.00	-77.98	540.00	-61.98	1777.92	-4.66	745.65
7	0.30	90.00	21.00	20.00	167.45	208.45	104.23	330.75	-90.23	630.00	-73.23	2474.07	-7.41	873.90
8	0.20	60.00	24.00	20.00	192.95	236.95	118.48	432.00	-102.48	720.00	-84.48	3284.97	-10.16	1002.15
9	0.10	30.00	27.00	20.00	218.45	265.45	132.73	546.75	-114.73	810.00	-95.73	4210.62	-12.91	1130.40
10	0.00	0.00	30.00	20.00	243.95	293.95	146.98	675.00	-126.98	900.00	-106.98	5251.02	-15.66	1258.65

2. Reservoir and Uplift Forces and Moment Arms, Anchor Force and Moment Arm

Block	y/H_s	y	P	yP	U_1	xU_1	U_2	xU_2	WH	xWH	||	Anchor Force	xA
1	0.90	270.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	||	0.00	-12.40
2	0.80	240.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	||	0.00	-23.65
3	0.70	210.00	2.00	2.67	23.57	-15.74	0.00	-47.23	0.20	-46.96	||	0.00	-34.90
4	0.60	180.00	45.05	12.67	145.77	-20.49	0.00	-61.48	4.51	-60.21	||	0.00	-46.15
5	0.50	150.00	144.27	22.67	321.32	-25.24	0.00	-75.73	14.43	-73.46	||	0.00	-57.40
6	0.40	120.00	299.64	32.67	550.22	-29.99	0.00	-89.98	29.96	-86.71	||	0.00	-68.65
7	0.30	90.00	511.18	42.67	832.47	-34.74	0.00	-104.23	51.12	-99.96	||	0.00	-79.90
8	0.20	60.00	778.88	52.67	1168.07	-39.49	0.00	-118.48	77.89	-113.21	||	0.00	-91.15
9	0.10	30.00	1102.73	62.67	1557.02	-44.24	0.00	-132.73	110.27	-126.46	||	0.00	-102.40
10	0.00	0.00	1482.75	72.67	1999.33	-48.99	0.00	-146.98	148.27	-139.71	||	0.00	-113.65

3. Total Forces, Overturning Moment and Streses

Block	y/H_s	y	SigFH	SigFV	SigM	e	U/S SIG_st (ksf)	U/S SIG_st (psi)	D/S SIG_st (ksf)	D/S SIG_st (psi)	C (ksf)	FS
1	0.90	270.00	0.00	115.17	-461	-4.00	5.05	35.04	1.10	7.67	0.00	#DIV/0!
2	0.80	240.00	0.00	347.82	-2945	-8.47	9.34	64.84	1.21	8.41	0.00	#DIV/0!
3	0.70	210.00	2.00	685.35	-8401	-12.26	12.91	89.63	1.61	11.15	0.00	343.22
4	0.60	180.00	45.05	1056.61	-16086	-15.22	14.98	104.02	2.21	15.34	0.00	23.45
5	0.50	150.00	144.27	1508.38	-25880	-17.16	16.73	116.18	3.19	22.15	0.00	10.46
6	0.40	120.00	299.64	2040.67	-37004	-18.13	18.20	126.37	4.48	31.14	0.00	6.81
7	0.30	90.00	511.18	2653.48	-48680	-18.35	19.45	135.08	6.01	41.72	0.00	5.19
8	0.20	60.00	778.88	3346.79	-60129	-17.97	20.55	142.71	7.70	53.46	0.00	4.30
9	0.10	30.00	1102.73	4120.62	-70570	-17.13	21.53	149.53	9.51	66.07	0.00	3.74
10	0.00	0.00	1482.75	4974.97	-79226	-15.92	22.43	155.74	11.42	79.33	0.00	1.94

Attachment B
Hand Calculations

This calculation provides the input parameters for the FEM analysis of the FRFA non-overflow section of the proposed Chehalis RCC dam.

Geometry

See Figure MP-RCC-2 for geometry/dimensions

Material Properties

The model is a linear elastic model with the following properties

RCC Material

$f'_c = 3{,}000\,psi \quad \nu = 0.2 \quad \gamma = 150\,pcf$

$E = E_{dyn} = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{3{,}000} = 3.12 \times 10^6\,psi$

$f'_{c\,dyn} = 1.2 f'_c = 1.2(3{,}000) = 3{,}600\,psi$

Tensile strength direct static $- 0.85 f'^{2/3}_c = 0.85(3{,}000)^{2/3} = 176.8\,psi$

direct dynamic $= 1.3 f'^{2/3}_c = 1.3(3{,}000)^{2/3} = 270.4\,psi$

Splitting static $= 1.7 f'^{2/3}_c = 1.7(3000)^{2/3} = 353.6\,psi$

Splitting dynamic $= 2.6 f'^{2/3}_c = 2.6(3000)^{2/3} = 540.8\,psi$

Intact Shear Strength $c = 0.1 f'_c = 0.1(3000) = 300\,psi$

$\phi = 45°$

Bonded lift line shear strength

$c = 0.085 f'_c = 0.085(3000) = 255\,psi \quad \phi = 45°$

Unbonded lift line shear strength

$c = 0$ to $50\,psi \quad \phi = 40°$

Damping (Hysteretic) = 10% (Viscous) 5 to 10%

Ref. State of Practice for Nonlinear Analysis of Concrete Dams @ USBR

Material Properties

Rock Foundation

Two rock types: Siltstone and Basalt

Three modulus values, one for siltstone (lower bound) one for basalt (upper bound) and one a weighted average of the siltstone/basalt (best estimate)

Siltstone $E_{RMR} = 1.39 \times 10^6$ psi

Basalt $E_{RMR} = 3.15 \times 10^6$ psi

Siltstone/Basalt $E_{RMR(AVG)} = 2.61 \times 10^6$ psi

$\gamma = 165$ pcf (foundation massless)

$\nu = 0.33$ estimate

Loadings

Gravity = 32.2 ft/sec^2

Normal Pool El 628 ft Tailwater El 415 ft

Flood Pool Storage El 687 ft TW - TBD

PMF El 709.4 ft TW - TBD

Uplift

$L = 285$ ft $x = 28$ ft

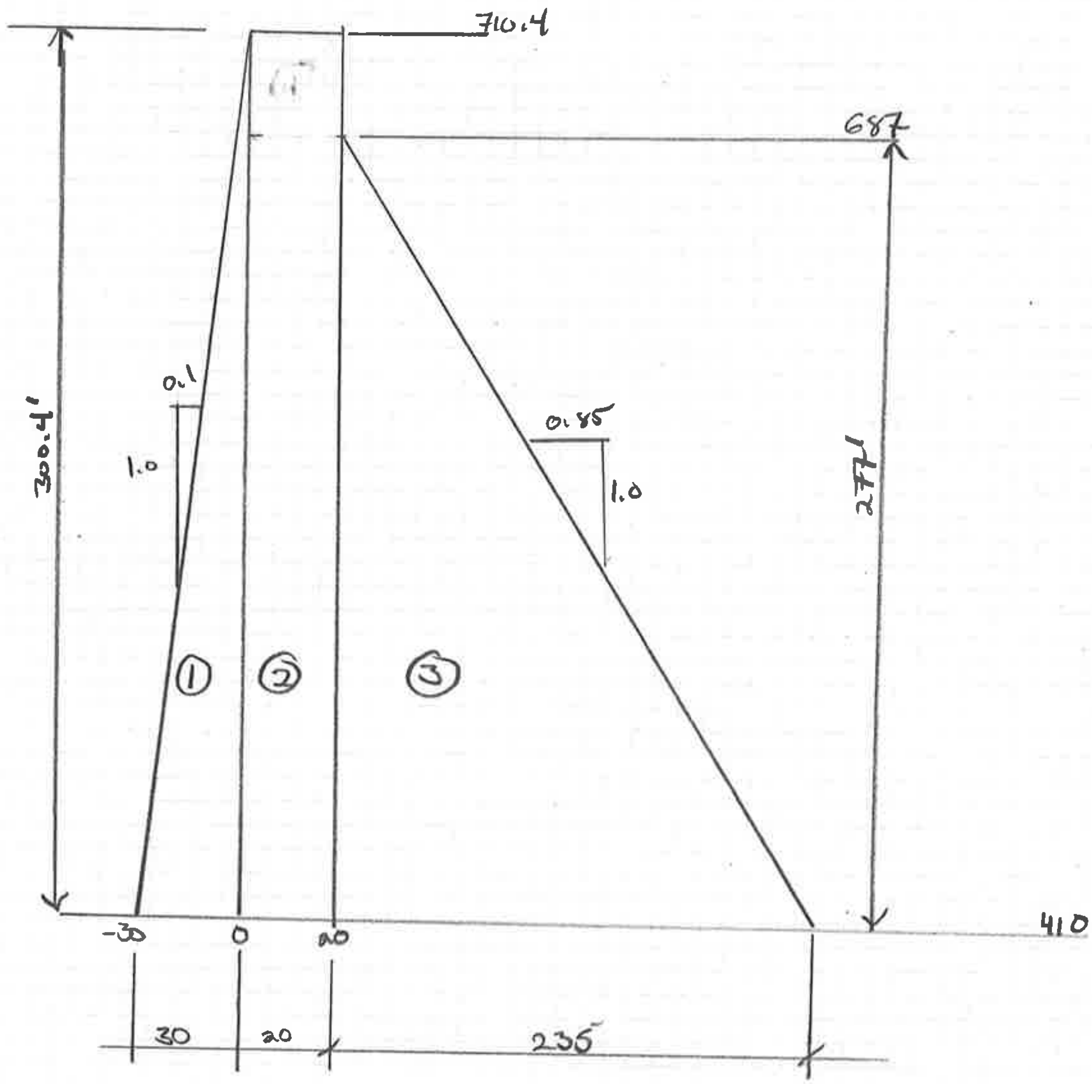
$H_1 = 628 - 410 = 218$ ft $K = 1 - E$ $E = 50\% = 0.5$

$H_2 = 15$ ft $= 1 - 0.5 = 0.5$

$H_4 = 18$ ft $H_4 > H_2$ so,

$$H_3 = K(H_1 - H_2)\frac{(L-x)}{L} + H_2$$

$$= 0.5(218 - 15)\frac{(285 - 28)}{285} + 15 = 106.5 \text{ ft}$$



① $A_1 = \frac{1}{2}(30)(300.4) = 4512\ ft^2$ $\gamma = 150\ lb/ft^3$ $\gamma A_1 = 676801.2\ lb/ft$

② $A_2 = 20(300.4) = 6008\ ft^2$ $\gamma A_2 = 901200\ lb/ft$

③ $A_3 = \frac{1}{2}(235)(277) = 32547.2$ $\gamma A_3 = 4882125\ lb/ft$

Parapets ④ $A_4 = 2(3)(2) = 12$ $\gamma A_4 = 1800\ lb/ft$

$\gamma A_1 + \gamma A_2 + \gamma A_3 + \gamma A_4 = 6461926.2\ lb/ft$

$= 6461.9\ kips/ft$ ok checks w/FEM

Project: Chehalis	Computed: STA	Date: 3-31-16	
Subject: Uplift Non Overflow	Checked:	Date:	
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Job #:	No:		

$L = 50 + 235 = 285$ ft $\quad x = 28$ ft.

Normal $H_1 = 628 - 410 = 218$ ft

PMF $H_1 = 709.4 - 410 = 299.6$ ft

$H_2 = 15$ ft.

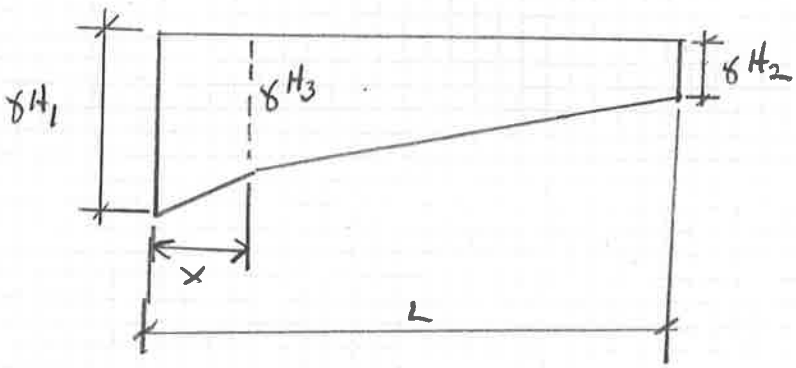
$K = 1 - E \quad E = 50\% = 0.5$

$= 1 - 0.5 = 0.5$

$H_4 = 18$ ft $\qquad H_4 > H_2$ so,

$$H_3 = K(H_1 - H_2)\frac{(L - x)}{L} + H_2$$

$$= 0.5(218 - 15)\frac{(285 - 28)}{285} + 15 = 106.5$$



$\gamma H_1 = 62.4(218) = 13603.2$

$\gamma H_3 = 62.4(106.5) = 6645.6$

$\gamma H_2 = 62.4(15) = 936$

Using coordinate system in model

~~$H_1 = 218 + 410 = 628$~~ $\quad$ ~~$\gamma H_1 = 39187.2$~~

~~$H_2 = 15 + 410 = 425$~~ $\quad$ ~~$\gamma H_2 = 26520.0$~~

~~$H_3 = 106.5 + 410 = 516.5$~~ $\quad$ ~~$\gamma H_3 = 32229.6$~~

Project: Chehalis	Computed: STA	Date: 3-31-2016	
Subject: Uplift NonOverflow	Checked:	Date:	
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For joint pattern in SAP2000

$-50 < x - 15 \quad p = 3663.2 - 198.8x$

$-15 < x < 235 \quad p = 6298.904 - 23.0864x$

For PMF $H_1 = 709.4 \text{ ft} - 410 - 299.6 \text{ ft}$

$\gamma H_1 = 62.4(299.6) = 18695.$

Assume $H_2 = 15 \text{ ft}$

$H_3 = K(H_1 - H_2)\frac{(L-x)}{L} + H_2$

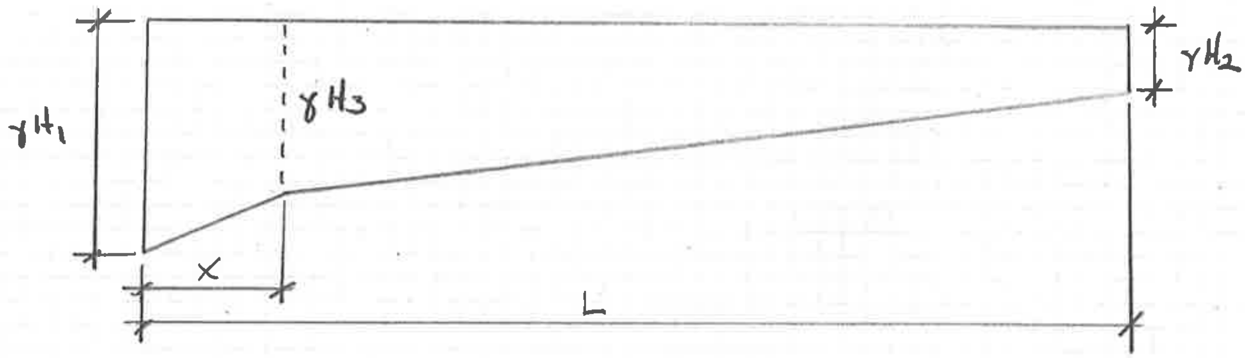
$0.5(299.6 - 15)\frac{(285 - 28)}{285} + 15 = 143.3 \text{ ft}$

$\gamma H_3 = 62.4(143.3) = 8943.2$

$-50 < x - 15 \quad p = 4762.45 - 278.65x$

$-15 < x < +235 \quad p = 1416.372 + 32.0248x$

Project:	Chehalis	Computed:	Date:
Subject:	Uplift Calcs	Checked:	Date:
Task:	Non Overflow	Page: 3	of: 4
Job #:	10026522	No:	



$\gamma H_1 = 62.4(218) = 13603.2$ psf

$\gamma H_2 = 62.4(15) = 936$ psf

$\gamma H_3 = 62.4(106.5) = 6645.6$ psf

For uplift in FEM

$-50 < x < -15 \quad p = 3663.2 - 198.8x$

$-15 < x < 235 \quad p = 6298.904 - 23.0864x$

Earthquake Loading

Use response spectrum analysis for one MCE CSZ IF event and 500, 2500 and 5000 interface events
We have both horizontal + vertical spectra

Hydrodynamic - Westergaard

See spreadsheet for added mass calcution

Reservoir pressure

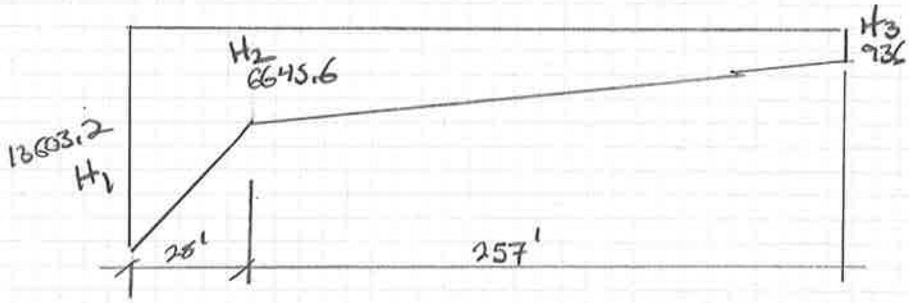
Joint patterns for SAP2000

U/S $39187.2 - 62.4y$

D/S Tailwater @415 $25896 - 62.4y$

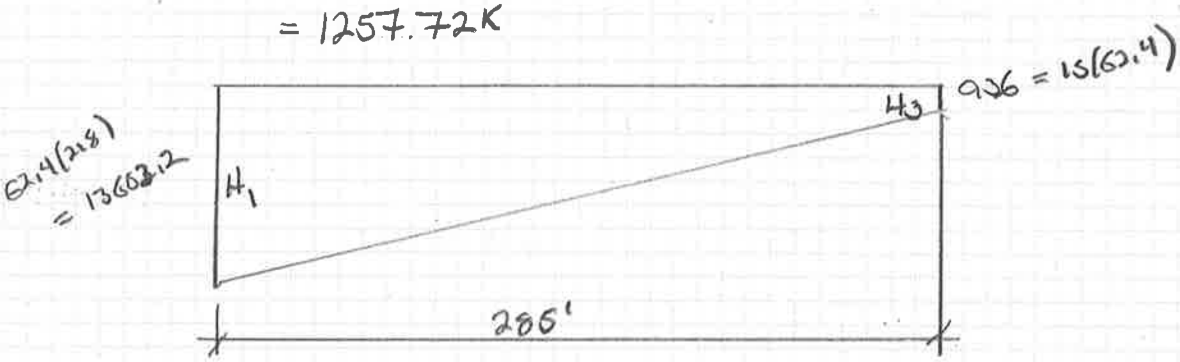
PMF U/S $(709.4)(62.4) - 62.4 = 44266.6 - 62.4y$

Project: Chehalis		Computed: STA	Date:
Subject: Uplift Calcs		Checked:	Date:
Task: Non Overflow		Page: 4	of: 4
Job #:		No:	

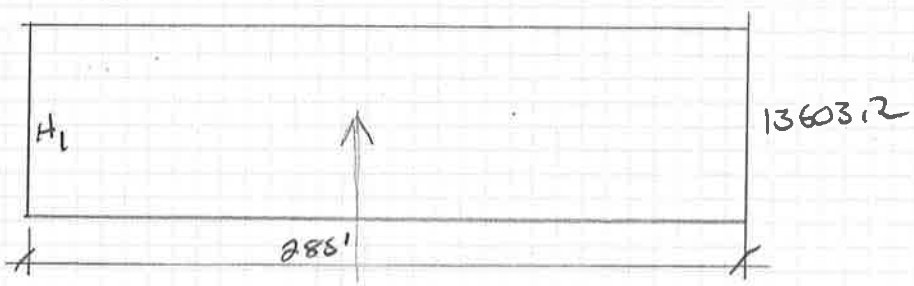


$$A = \left[\frac{13603.2 + 6645.6}{2}(28) + \frac{6645.6 + 936}{2}(257)\right]/1000$$

$$= 1257.72^{K}$$



$$A = \left[\frac{13603.2 + 936}{2}(285)\right]/1000 = 2071.84^{k}$$



$$A = 13603.2(285) = 3876.91^{K}$$

$H_1 = 218'$ $H_2 = 106.5'$ $H_3 = 15'$

$\gamma H_1 = 62.4(218) = 13603.2 \text{ lb/ft}^2$

$\gamma H_2 = 62.4(106.5) = 6645.6 \text{ lb/ft}^2$

$\gamma H_3 = 62.4(15) = 936 \text{ lb/ft}^2$

Project: | Computed: | Date: 8-2-16
Subject: | Checked: | Date:
Task: Overflow Section Uplift | Page: 1 | of: 3
Job #: | No:

$L = 321$ ft $\quad x = 28$ ft

Normal ⇒ $H_1 = 628 - 390 = 238$ ft

$K = 1 - E$ @ $E = 50\% \Rightarrow 0.5$

$H_2 = 417 - 390 = 27$ ft

$K = 0.5$

$H_4 = 429 - 390 = 39$ ft so $H_2 > H_4$

$0.05 H_1 = 11.9$ ft $> x$

$$H_3 = K(H_1 - H_4)\frac{(L - x)}{L} + H_4$$

$$H_3 = 0.5(238 - 39)\frac{(321 - 28)}{321} + 39 = 129.8 \text{ ft}$$

$\gamma H_1 = 14851.2 \quad \gamma H_2 = 1684.8 \quad \gamma H_3 = 8100.8$

@ x=0 y = 14861.2 @ x=28 y= 8199.4

$\frac{8100.8 - 14851.2}{28} = -241.1$ y= 14851.2 - 241.1x

@ x=28 y = 8199.4 @ x=390 y= 1684.8

$\frac{1684.8 - 8100.8}{321 - 28} = -21.9$ y= 8100.8 - 21.9x

Project:	Computed:	Date:
Subject:	Checked:	Date:
Task: PMF Uplift Non Overflow	Page: 3	of: 3
Job #:	No:	

713.4 ft $(713.4-410)\,62.4 = 18932.2$

410.0 ft $(425-410)\,62.4 = 936$

18932.2 @ -50   936 @ 235

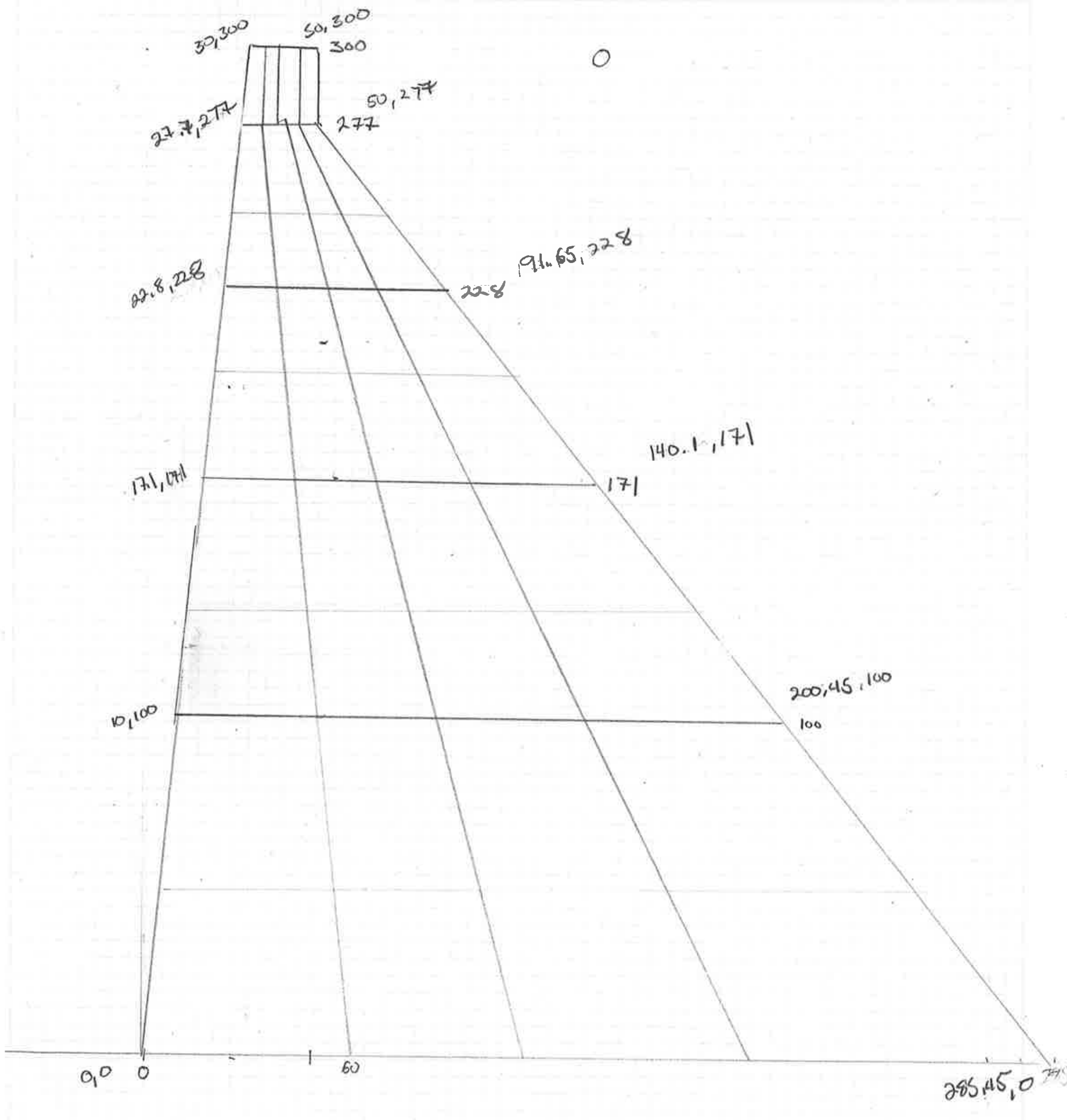
$$\frac{936-18932.2}{285} = \frac{-17996.2}{285} = -63.1$$

$$-63.1(-50) + x = 18932.2$$

$$3155 + x = 18932.2$$

$$x = 15777$$

Project:	Chehalis	Computed:	STA	Date:	4/11/16
Subject:	Coordinates for EAGD-SLIDE	Checked:		Date:	
Task:		Page:	1	of:	1
Job #:	10026522	No:	B-5812 CD-D		



Attachment C
RMR Calculations for Site Characterization Report

Deformation Modulus (E_m) is derived from RMR according to equations 14 and 15 with a coeffecient of regression (r^2) of 0.8 according to "Evaluation of the deformation modulus of rock masses using RMR. Comparison with dilatometer tests" by Jose M. Galera, M. Alvarez, and Z.T.

$$E_m(GPa) = 0.0876 \cdot RMR \quad \text{for } RMR \leq 50 \qquad (14)$$

$$E_m(GPa) = 0.0876 \cdot RMR + 1.056(RMR - 50) + 0.015(RMR - 50)^2 \quad \text{for } RMR > 50 \qquad (15)$$

1 Gpa = 145,037.7 psi

Hole ID	Run #	From ft	To ft	Lithology	RQD %	BASIC RMR	Deformation Modulus Gpa	Deformation Modulus psi
BH-1		29	35.3	Basalt	3	**12**	1.0512	1.52E+05
BH-1	1	35.3	40.3	Basalt	20	**67**	28.1562	4.08E+06
BH-1	2	40.3	45.5	Basalt	20	**68**	29.8248	4.33E+06
BH-1	3	45.5	50.5	Basalt	20	**69**	31.5234	4.57E+06
BH-1	4	50.5	55.5	Basalt	17	**60**	17.316	2.51E+06
BH-1	5	55.5	60.6	Basalt	20	**62**	20.2632	2.94E+06
BH-1	6	60.6	65.4	Basalt	13	**57**	13.1202	1.90E+06
BH-1	7	65.4	70.6	Basalt	20	**65**	24.909	3.61E+06
BH-1	8	70.6	75.5	Basalt	20	**65**	24.909	3.61E+06
BH-1	9	75.5	80.5	Basalt	20	**75**	42.345	6.14E+06
BH-1	10	80.5	85.4	Basalt	20	**61**	18.7746	2.72E+06
BH-1	11	85.4	90.5	Basalt	20	**62**	20.2632	2.94E+06
BH-1	12	90.5	95.3	Basalt	20	**62**	20.2632	2.94E+06
BH-1	13	95.3	100.4	Basalt	20	**63**	21.7818	3.16E+06
BH-1	14	100.4	105.5	Siltstone	20	**63**	21.7818	3.16E+06
BH-1	15	105.5	110.5	Siltstone	20	**61**	18.7746	2.72E+06
BH-1	16	110.5	115.4	Basalt	20	**68**	29.8248	4.33E+06
BH-1	17	115.4	120.5	Basalt	20	**68**	29.8248	4.33E+06
BH-1	18	120.5	125.1	Basalt	20	**75**	42.345	6.14E+06
BH-1	19	125.1	130.5	Siltstone	17	**55**	10.473	1.52E+06
BH-1	20	130.5	135.4	Siltstone	20	**63**	21.7818	3.16E+06
BH-1	21	135.4	140.4	Siltstone	20	**62**	20.2632	2.94E+06
BH-2	1	30	34.3	Basalt	21	**37**	3.2412	4.70E+05
BH-2	2	34.3	35.5	Basalt	58	**55**	10.473	1.52E+06
BH-2	3	35.5	40.5	Basalt	58	**55**	10.473	1.52E+06
BH-2	4	40.5	45.5	Basalt	94	**68**	29.8248	4.33E+06
BH-2	5	45.5	50.5	Basalt	100	**67**	28.1562	4.08E+06
BH-2	6	50.5	55.5	Basalt	80	**62**	20.2632	2.94E+06
BH-2	7	55.5	60.8	Basalt	89	**61**	18.7746	2.72E+06
BH-2	8	60.8	65.9	Basalt	100	**72**	36.7992	5.34E+06
BH-2	9	65.9	70.8	Basalt	100	**67**	28.1562	4.08E+06
BH-2	10	70.8	75.8	Basalt	100	**68**	29.8248	4.33E+06
BH-2	11	75.8	80.8	Basalt	88	**57**	13.1202	1.90E+06
BH-2	12	80.8	83.8	Siltstone	63	**47**	4.1172	5.97E+05
BH-2	13	83.8	85.8	Siltstone	65	**47**	4.1172	5.97E+05
BH-2	14	85.8	90.8	Siltstone	36	**39**	3.4164	4.96E+05
BH-2	15	90.8	95.9	Siltstone	90	**59**	15.8874	2.30E+06
BH-2	16	95.9	100.8	Siltstone	65	**52**	6.7272	9.76E+05
BH-2	17	100.8	105.8	Siltstone	58	**51**	5.5386	8.03E+05
BH-2	18	105.8	110.7	Siltstone	71	**51**	5.5386	8.03E+05
BH-2	19	110.7	116	Siltstone	96	**54**	9.1944	1.33E+06
BH-2	20	116	121	Siltstone	62	**50**	4.38	6.35E+05
BH-2	21	121	123.8	Siltstone	57	**48**	4.2048	6.10E+05
BH-2	22	123.8	126	Siltstone	100	**55**	10.473	1.52E+06
BH-2	23	126	131.1	Siltstone	84	**57**	13.1202	1.90E+06
BH-2	24	131.1	136.3	Basalt	90	**61**	18.7746	2.72E+06
BH-2	25	136.3	141.3	Siltstone	72	**51**	5.5386	8.03E+05
BH-2	26	141.3	146.3	Basalt	100	**64**	23.3304	3.38E+06
BH-2	27	146.3	151.3	Basalt	90	**72**	36.7992	5.34E+06
BH-2	28	151.3	156.3	Basalt	100	**73**	38.6178	5.60E+06
BH-2	29	156.3	161.2	Basalt	100	**78**	48.1608	6.99E+06
BH-2	30	161.2	166.2	Basalt	100	**81**	54.2466	7.87E+06
BH-2	31	166.2	171.2	Basalt	100	**73**	38.6178	5.60E+06
BH-2	32	171.2	176	Basalt	100	**66**	26.5176	3.85E+06
BH-2	33	176	181	Basalt	96	**75**	42.345	6.14E+06
BH-2	34	181	186	Basalt	100	**63**	21.7818	3.16E+06
BH-2	35	186	190.9	Basalt	100	**65**	24.909	3.61E+06
BH-2	36	190.9	196	Basalt	100	**67**	28.1562	4.08E+06
BH-2	37	196	200.9	Basalt	100	**69**	31.5234	4.57E+06
BH-2	38	200.9	206	Basalt	100	**67**	28.1562	4.08E+06
BH-2	39	206	211.1	Basalt	94	**62**	20.2632	2.94E+06
BH-2	40	211.1	212.6	Siltstone	80	**58**	14.4888	2.10E+06
BH-2	41	212.6	216.1	Siltstone	91	**57**	13.1202	1.90E+06
BH-2	42	216.1	221	Siltstone	63	**51**	5.5386	8.03E+05
BH-2	43	221	226	Siltstone	72	**48**	4.2048	6.10E+05
BH-2	44	226	231	Siltstone	68	**55**	10.473	1.52E+06
BH-2	45	231	236	Siltstone	60	**53**	7.9458	1.15E+06
BH-2	46	236	237	Siltstone	100	**55**	10.473	1.52E+06
BH-2	47	237	241	Siltstone	78	**55**	10.473	1.52E+06
BH-3	1	35	40	Basalt	95	**59**	15.8874	2.30E+06
BH-3	2	40	43.3	Basalt	100	**64**	23.3304	3.38E+06
BH-3	3	43.3	45	Basalt	100	**62**	20.2632	2.94E+06
BH-3	4	45	50	Basalt	100	**63**	21.7818	3.16E+06
BH-3	5	50	55	Basalt	100	**63**	21.7818	3.16E+06
BH-3	6	55	60	Basalt	100	**66**	26.5176	3.85E+06
BH-3	7	60	65	Basalt	100	**59**	15.8874	2.30E+06
BH-3	8	65	70	Basalt	100	**62**	20.2632	2.94E+06
BH-3	9	70	75	Basalt	100	**64**	23.3304	3.38E+06
BH-3	10	75	80	Basalt	100	**58**	14.4888	2.10E+06
BH-3	11	80	85	Basalt	100	**65**	24.909	3.61E+06
BH-3	12	85	90	Basalt	72	**53**	7.9458	1.15E+06
BH-3	13	90	95	Basalt	100	**62**	20.2632	2.94E+06
BH-3	14	95	96.4	Basalt	64	**51**	5.5386	8.03E+05
BH-3	15	96.4	100	Basalt	100	**64**	23.3304	3.38E+06
BH-3	16	100	105	Basalt	100	**62**	20.2632	2.94E+06
BH-3	17	105	110	Basalt	90	**58**	14.4888	2.10E+06
BH-3	18	110	115	Basalt	88	**56**	11.7816	1.71E+06
BH-3	19	115	120	Basalt	92	**65**	24.909	3.61E+06
BH-3	20	120	125	Basalt	100	**65**	24.909	3.61E+06
BH-3	21	125	130	Basalt	74	**56**	11.7816	1.71E+06

Summary			
Lithology	Average RMR	Average $E_{m\,(psi)}$	% of total
Basalt	62	3.15E+06	70%
Siltstone/claystone	52	1.39E+06	30%
Weighted Avg.	58.70563	2.61E+06	100%

BH-3	22	130	135	Siltstone	90	**60**	17.316	2.51E+06
BH-3	23	135	140	Siltstone	100	**58**	14.4888	2.10E+06
BH-3	24	140	145	Siltstone	100	**56**	11.7816	1.71E+06
BH-3	25	145	150	Siltstone	100	**58**	14.4888	2.10E+06
BH-4	1	65	70	Basalt	0	**30**	2.628	3.81E+05
BH-4	2	70	75	Basalt	56	**45**	3.942	5.72E+05
BH-4	3	75	80	Basalt	32	**40**	3.504	5.08E+05
BH-4	4	80	85	Basalt	0	**35**	3.066	4.45E+05
BH-4	5	85	90	Basalt	28	**43**	3.7668	5.46E+05
BH-4	6	90	95	Basalt	16	**30**	2.628	3.81E+05
BH-4	7	95	100	Siltstone	19	**34**	2.9784	4.32E+05
BH-4	8	100	105	Siltstone	44	**39**	3.4164	4.96E+05
BH-4	9	105	110	Siltstone	38	**36**	3.1536	4.57E+05
BH-4	10	110	115	Siltstone	68	**47**	4.1172	5.97E+05
BH-4	11	115	120	Siltstone	48	**36**	3.1536	4.57E+05
BH-5	1	50.4	55	Siltstone	63	**42**	3.6792	5.34E+05
BH-5	2	55	60	Siltstone	48	**38**	3.3288	4.83E+05
BH-5	3	60	65	Siltstone	94	**56**	11.7816	1.71E+06
BH-5	4	65	70	Siltstone	80	**45**	3.942	5.72E+05
BH-5	5	70	75	Basalt	100	**61**	18.7746	2.72E+06
BH-5	6	75	80	Basalt	100	**70**	33.252	4.82E+06
BH-5	7	80	85	Basalt	100	**68**	29.8248	4.33E+06
BH-5	8	85	90	Basalt	96	**65**	24.909	3.61E+06
BH-5	9	90	95	Basalt	62	**54**	9.1944	1.33E+06
BH-5	10	95	100	Basalt	100	**74**	40.4664	5.87E+06
BH-5	11	100	105	Basalt	96	**69**	31.5234	4.57E+06
BH-5	12	105	110	Basalt	94	**64**	23.3304	3.38E+06
BH-5	13	110	115	Basalt	94	**65**	24.909	3.61E+06
BH-5	14	115	120	Basalt	96	**67**	28.1562	4.08E+06
BH-5	15	120	125	Basalt	95	**62**	20.2632	2.94E+06
BH-5	16	125	130	Basalt	96	**69**	31.5234	4.57E+06
BH-5	17	130	135	Basalt	100	**67**	28.1562	4.08E+06
BH-5	18	135	140	Basalt	100	**69**	31.5234	4.57E+06
BH-5	19	140	145	Basalt	100	**67**	28.1562	4.08E+06
BH-5	20	145	150	Siltstone	100	**63**	21.7818	3.16E+06
BH-5	21	150	155	Siltstone	98	**55**	10.473	1.52E+06
BH-5	22	155	157.5	Siltstone	76	**45**	3.942	5.72E+05
BH-5	23	157.5	160	Siltstone	48	**38**	3.3288	4.83E+05
BH-5	24	160	165	Siltstone	70	**41**	3.5916	5.21E+05
BH-5	25	165	170	Siltstone	72	**46**	4.0296	5.84E+05
BH-5	26	170	175	Siltstone	96	**56**	11.7816	1.71E+06
BH-5	27	175	180	Siltstone	100	**51**	5.5386	8.03E+05
BH-5	28	180	185	Basalt	86	**55**	10.473	1.52E+06
BH-5	29	185	190	Basalt	96	**65**	24.909	3.61E+06
BH-5	30	190	195	Basalt	98	**70**	33.252	4.82E+06
BH-5	31	195	200	Basalt	100	**72**	36.7992	5.34E+06
BH-5	32	200	205	Basalt	100	**70**	33.252	4.82E+06
BH-5	33	205	210	Basalt	100	**63**	21.7818	3.16E+06
BH-5	34	210	215	Basalt	94	**60**	17.316	2.51E+06
BH-5	35	215	220	Basalt	100	**70**	33.252	4.82E+06
BH-5	36	220	225	Basalt	100	**68**	29.8248	4.33E+06
BH-5	37	225	230	Basalt	100	**68**	29.8248	4.33E+06
BH-5	38	230	235	Basalt	100	**73**	38.6178	5.60E+06
BH-5	39	235	240	Basalt	100	**66**	26.5176	3.85E+06
BH-5	40	240	245	Basalt	100	**70**	33.252	4.82E+06
BH-5	41	245	250	Basalt	100	**66**	26.5176	3.85E+06
BH-6	1	40.3	45	Siltstone	93	**58**	14.4888	2.10E+06
BH-6	2	45	50	Siltstone	90	**58**	14.4888	2.10E+06
BH-6	3	50	55	Siltstone	100	**58**	14.4888	2.10E+06
BH-6	4	55	60	Siltstone	90	**56**	11.7816	1.71E+06
BH-6	5	60	65	Basalt	96	**67**	28.1562	4.08E+06
BH-6	6	65	69.7	Basalt	87	**53**	7.9458	1.15E+06
BH-6	7	69.7	75	Basalt	50	**49**	4.2924	6.23E+05
BH-6	8	75	80	Basalt	85	**53**	7.9458	1.15E+06
BH-6	9	80	85	Basalt	92	**54**	9.1944	1.33E+06
BH-6	10	85	90	Basalt	82	**51**	5.5386	8.03E+05
BH-6	11	90	95	Basalt	96	**54**	9.1944	1.33E+06
BH-6	12	95	100	Basalt	100	**57**	13.1202	1.90E+06
BH-6	13	100	105	Basalt	96	**58**	14.4888	2.10E+06
BH-6	14	105	110	Basalt	90	**56**	11.7816	1.71E+06
BH-6	15	110	115	Basalt	100	**58**	14.4888	2.10E+06
BH-6	16	115	120	Basalt	100	**60**	17.316	2.51E+06
BH-6	17	120	125	Basalt	100	**56**	11.7816	1.71E+06
BH-6	18	125	130	Basalt	96	**60**	17.316	2.51E+06
BH-6	19	130	135	Basalt	100	**56**	11.7816	1.71E+06
BH-6	20	135	140	Basalt	86	**57**	13.1202	1.90E+06
BH-6	21	140	145	Basalt	100	**58**	14.4888	2.10E+06
BH-6	22	145	150	Basalt	90	**56**	11.7816	1.71E+06
BH-6	23	150	154.6	Basalt	91	**59**	15.8874	2.30E+06
BH-6	24	154.6	160	Basalt	90	**61**	18.7746	2.72E+06
BH-6	25	160	165	Basalt	100	**63**	21.7818	3.16E+06
BH-6	26	165	170.2	Basalt	100	**63**	21.7818	3.16E+06
BH-6	27	170.2	175	Basalt	92	**61**	18.7746	2.72E+06
BH-6	28	175	180	Siltstone	84	**53**	7.9458	1.15E+06
BH-6	29	180	185	Siltstone	91	**56**	11.7816	1.71E+06
BH-6	30	185	190	Siltstone	90	**54**	9.1944	1.33E+06
BH-6	31	190	195	Basalt	94	**63**	21.7818	3.16E+06
BH-6	32	195	200	Basalt	98	**63**	21.7818	3.16E+06
BH-6	33	200	205	Basalt	96	**58**	14.4888	2.10E+06
BH-6	34	205	210	Basalt	93	**58**	14.4888	2.10E+06
BH-6	35	210	215	Basalt	100	**65**	24.909	3.61E+06
BH-6	36	215	220	Basalt	90	**58**	14.4888	2.10E+06
BH-6	37	220	225	Basalt	52	**43**	3.7668	5.46E+05
BH-6	38	225	230	Basalt	84	**58**	14.4888	2.10E+06
BH-6	39	230	235	Basalt	92	**61**	18.7746	2.72E+06
BH-6	40	235	240	Basalt	98	**63**	21.7818	3.16E+06
BH-6	41	240	245	Basalt	92	**63**	21.7818	3.16E+06
BH-6	42	245	250	Basalt	100	**63**	21.7818	3.16E+06

BH-6	43	250	255	Basalt	100	**61**	18.7746	2.72E+06
BH-6	44	255	260	Basalt	100	**63**	21.7818	3.16E+06
BH-6	45	260	265	Siltstone	68	**44**	3.8544	5.59E+05
BH-6	46	265	270	Siltstone	46	**39**	3.4164	4.96E+05
BH-6	47	270	275	Siltstone	100	**55**	10.473	1.52E+06
BH-6	48	275	280	Siltstone	98	**56**	11.7816	1.71E+06
BH-6	49	280	285	Siltstone	92	**54**	9.1944	1.33E+06
BH-6	50	285	290	Siltstone	82	**51**	5.5386	8.03E+05
BH-6	51	290	295	Siltstone	94	**54**	9.1944	1.33E+06
BH-6	52	295	300	Siltstone	100	**61**	18.7746	2.72E+06
BH-6	53	300	305	Siltstone	100	**56**	11.7816	1.71E+06
BH-6	54	305	310	Siltstone	98	**56**	11.7816	1.71E+06
BH-6	55	310	315	Siltstone	90	**56**	11.7816	1.71E+06
BH-6	56	315	319.35	Siltstone	100	**58**	14.4888	2.10E+06
BH-6	57	319.35	320	Siltstone	100	**56**	11.7816	1.71E+06
BH-6	58	320	325	Basalt	98	**63**	21.7818	3.16E+06
BH-6	59	325	330	Basalt	100	**63**	21.7818	3.16E+06
BH-6	60	330	335	Basalt	100	**70**	33.252	4.82E+06
BH-6	61	335	340	Basalt	100	**68**	29.8248	4.33E+06
BH-6	62	340	345	Basalt	98	**68**	29.8248	4.33E+06
BH-6	63	345	350	Basalt	100	**68**	29.8248	4.33E+06
					Average:	**59**	**17.821**	**2.58E+06**

TABLE: Joint Coordinates			SORT Z FROM LOW TO HIGH								g (ft/sec2)	32.2	Reservoir-Base EL	427.8	Reservoir depth		
Joint	CoordSys	CoordType	XorR	Y	Z	SpecialJt	GlobalX	GlobalY	GlobalZ	GUID	gama (pcf)	62.4	Pool El, ft	628	200.2 ft		
Text	Text	Text	ft	ft	ft	Yes/No	ft	ft	ft	Text	Node above pool	Y,ft	Y, ft	bottom element	top element	.5*(top+bot)	mass
3240	GLOBAL	Cartesian	-50	0	410	Yes					1	218	200.2	0	1	0.5	169.7
3241	GLOBAL	Cartesian	-50	0	411	Yes					1	217	200.2	1	5.6	3.3	1120.2
3258	GLOBAL	Cartesian	-49.4167	0	416.6	Yes					1	211.4	200.2	5.6	5.6	5.6	1901.0
3264	GLOBAL	Cartesian	-48.8333	0	422.2	Yes					1	205.8	200.2	5.6	5.6	5.6	1901.0
3265	GLOBAL	Cartesian	-48.25	0	427.8	Yes					1	200.2	200.2	5.6	4.9846	5.2923	1796.6
3270	GLOBAL	Cartesian	-47.7452	0	432.7846	Yes					1	195.2154	195.2154	4.9846	4.9846	4.9846	1670.9
3275	GLOBAL	Cartesian	-47.2404	0	437.7692	Yes					1	190.2308	190.2308	4.9846	4.9846	4.9846	1649.5
3276	GLOBAL	Cartesian	-46.7356	0	442.7538	Yes					1	185.2462	185.2462	4.9846	4.9847	4.98465	1627.7
3277	GLOBAL	Cartesian	-46.2308	0	447.7385	Yes					1	180.2615	180.2615	4.9847	4.9846	4.98465	1605.7
3280	GLOBAL	Cartesian	-45.726	0	452.7231	Yes					1	175.2769	175.2769	4.9846	4.9846	4.9846	1583.3
3287	GLOBAL	Cartesian	-45.2212	0	457.7077	Yes					1	170.2923	170.2923	4.9846	4.9846	4.9846	1560.6
3314	GLOBAL	Cartesian	-44.7163	0	462.6923	Yes					1	165.3077	165.3077	4.9846	4.9846	4.9846	1537.6
3317	GLOBAL	Cartesian	-44.2115	0	467.6769	Yes					1	160.3231	160.3231	4.9846	4.9846	4.9846	1514.2
3320	GLOBAL	Cartesian	-43.7067	0	472.6615	Yes					1	155.3385	155.3385	4.9846	4.9847	4.98465	1490.5
3324	GLOBAL	Cartesian	-43.2019	0	477.6462	Yes					1	150.3538	150.3538	4.9847	4.9846	4.98465	1466.4
3327	GLOBAL	Cartesian	-42.6971	0	482.6308	Yes					1	145.3692	145.3692	4.9846	4.9846	4.9846	1441.9
3331	GLOBAL	Cartesian	-42.1923	0	487.6154	Yes					1	140.3846	140.3846	4.9846	4.9846	4.9846	1417.0
3334	GLOBAL	Cartesian	-41.6875	0	492.6	Yes					1	135.4	135.4	4.9846	4.9846	4.9846	1391.6
3341	GLOBAL	Cartesian	-41.1827	0	497.5846	Yes					1	130.4154	130.4154	4.9846	4.9846	4.9846	1365.7
3345	GLOBAL	Cartesian	-40.6779	0	502.5692	Yes					1	125.4308	125.4308	4.9846	4.9846	4.9846	1339.4
3351	GLOBAL	Cartesian	-40.1731	0	507.5538	Yes					1	120.4462	120.4462	4.9846	4.9847	4.98465	1312.5
3383	GLOBAL	Cartesian	-39.6683	0	512.5385	Yes					1	115.4615	115.4615	4.9847	4.9846	4.98465	1285.1
3385	GLOBAL	Cartesian	-39.1635	0	517.5231	Yes					1	110.4769	110.4769	4.9846	4.9846	4.9846	1257.0
3392	GLOBAL	Cartesian	-38.6587	0	522.5077	Yes					1	105.4923	105.4923	4.9846	4.9846	4.9846	1228.3
3395	GLOBAL	Cartesian	-38.1538	0	527.4923	Yes					1	100.5077	100.5077	4.9846	4.9846	4.9846	1198.9
3399	GLOBAL	Cartesian	-37.649	0	532.4769	Yes					1	95.5231	95.5231	4.9846	4.9846	4.9846	1168.8
3402	GLOBAL	Cartesian	-37.1442	0	537.4615	Yes					1	90.5385	90.5385	4.9846	4.9847	4.98465	1137.9
3407	GLOBAL	Cartesian	-36.6394	0	542.4462	Yes					1	85.5538	85.5538	4.9847	4.9846	4.98465	1106.2
3414	GLOBAL	Cartesian	-36.1346	0	547.4308	Yes					1	80.5692	80.5692	4.9846	4.9846	4.9846	1073.5
3419	GLOBAL	Cartesian	-35.6298	0	552.4154	Yes					1	75.5846	75.5846	4.9846	4.9846	4.9846	1039.7
3427	GLOBAL	Cartesian	-35.125	0	557.4	Yes					1	70.6	70.6	4.9846	4.9846	4.9846	1004.9
3457	GLOBAL	Cartesian	-34.6202	0	562.3846	Yes					1	65.6154	65.6154	4.9846	4.9846	4.9846	968.7
3465	GLOBAL	Cartesian	-34.1154	0	567.3692	Yes					1	60.6308	60.6308	4.9846	4.9846	4.9846	931.2
3471	GLOBAL	Cartesian	-33.6106	0	572.3538	Yes					1	55.6462	55.6462	4.9846	4.9847	4.98465	892.1
3474	GLOBAL	Cartesian	-33.1058	0	577.3385	Yes					1	50.6615	50.6615	4.9847	4.9846	4.98465	851.2
3478	GLOBAL	Cartesian	-32.601	0	582.3231	Yes					1	45.6769	45.6769	4.9846	4.9846	4.9846	808.3
3483	GLOBAL	Cartesian	-32.0962	0	587.3077	Yes					1	40.6923	40.6923	4.9846	4.9846	4.9846	762.9
3487	GLOBAL	Cartesian	-31.5913	0	592.2923	Yes					1	35.7077	35.7077	4.9846	4.9846	4.9846	714.6
3493	GLOBAL	Cartesian	-31.0865	0	597.2769	Yes					1	30.7231	30.7231	4.9846	4.9846	4.9846	662.9
3498	GLOBAL	Cartesian	-30.5817	0	602.2615	Yes					1	25.7385	25.7385	4.9846	4.9847	4.98465	606.7
3510	GLOBAL	Cartesian	-30.0769	0	607.2462	Yes					1	20.7538	20.7538	4.9847	4.9846	4.98465	544.8
3544	GLOBAL	Cartesian	-29.5721	0	612.2308	Yes					1	15.7692	15.7692	4.9846	4.9846	4.9846	474.9
3552	GLOBAL	Cartesian	-29.0673	0	617.2154	Yes					1	10.7846	10.7846	4.9846	4.9846	4.9846	392.7
3556	GLOBAL	Cartesian	-28.5625	0	622.2	Yes					1	5.8	5.8	4.9846	4.9846	4.9846	288.0
3564	GLOBAL	Cartesian	-28.0577	0	627.1846	Yes					1	0.8154	0.8154	4.9846	4.9846	4.9846	108.0
3569	GLOBAL	Cartesian	-27.5529	0	632.1692	Yes					0	0	0	4.9846	4.9846	4.9846	0.0
3573	GLOBAL	Cartesian	-27.0481	0	637.1538	Yes					0	0	0	4.9846	4.9847	4.98465	0.0
3578	GLOBAL	Cartesian	-26.5433	0	642.1385	Yes					0	0	0	4.9847	4.9846	4.98465	0.0
3584	GLOBAL	Cartesian	-26.0385	0	647.1231	Yes					0	0	0	4.9846	4.9846	4.9846	0.0
3589	GLOBAL	Cartesian	-25.5337	0	652.1077	Yes					0	0	0	4.9846	4.9846	4.9846	0.0
3605	GLOBAL	Cartesian	-25.0288	0	657.0923	Yes					0	0	0	4.9846	4.9846	4.9846	0.0
3642	GLOBAL	Cartesian	-24.524	0	662.0769	Yes					0	0	0	4.9846	4.9846	4.9846	0.0
3649	GLOBAL	Cartesian	-24.0192	0	667.0615	Yes					0	0	0	4.9846	4.9847	4.98465	0.0
3655	GLOBAL	Cartesian	-23.5144	0	672.0462	Yes					0	0	0	4.9847	4.9846	4.98465	0.0
3660	GLOBAL	Cartesian	-23.0096	0	677.0308	Yes					0	0	0	4.9846	4.9846	4.9846	0.0
3665	GLOBAL	Cartesian	-22.5048	0	682.0154	Yes					0	0	0	4.9846	4.9846	4.9846	0.0
3671	GLOBAL	Cartesian	-22	0	687	Yes					0	0	0	4.9846	4.48	4.7323	0.0
3678	GLOBAL	Cartesian	-21.52	0	691.48	Yes					0	0	0	4.48	4.48	4.48	0.0
3683	GLOBAL	Cartesian	-21.04	0	695.96	Yes					0	0	0	4.48	4.48	4.48	0.0
3690	GLOBAL	Cartesian	-20.56	0	700.44	Yes					0	0	0	4.48	4.48	4.48	0.0
3698	GLOBAL	Cartesian	-20.08	0	704.92	Yes					0	0	0	4.48	4.48	4.48	0.0
3742	GLOBAL	Cartesian	-19.6	0	709.4	Yes					0	0	0	4.48	4	4.24	0.0
3743	GLOBAL	Cartesian	-19.6	0	713.4	Yes										SUM	51370.5
															Check sum Westergaard		45307.9 OK

Appendix E
RCC Materials Sourcing

Chehalis Basin Strategy

Appendix E

RCC Materials Sourcing

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

Prepared for: State of Washington Recreation and Conservation Office and Chehalis Basin Work Group

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Appendix A Summary RCC Test Standards
Appendix B Excerpt from the Site Characterization Report

ACRONYMS AND ABBREVIATIONS

AAR	alkali-aggregate reaction
ACI	American Concrete Institute
AEA	air-entraining admixtures
ASCE	American Society of Civil Engineers
ASR	alkali-silica reaction
ASTM	American Society for Testing and Materials
CRBG	Columbia River Basalt Group
EM	Engineering Manual
FHWA	Federal Highway Administration
GERCC	grout enriched roller-compacted concrete
ICOLD	International Commission on Large Dams
kg/m3	Kilograms per cubic meter
LA	Los Angeles
lbs/cy3	pounds per cubic yard
mm	millimeter(s)
MSA	maximum size aggregate
NMSA	nominal maximum size of aggregate
PCA	Portland Cement Association
pcf	pounds per cubic foot
psi	pounds per square inch
RCC	roller-compacted concrete
SSD	saturated surface dry
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
WRA	water-reducing admixtures
WSDOT	Washington State Department of Transportation

1 INTRODUCTION

The Chehalis Dam Project considers two roller-compacted concrete (RCC) dam alternatives across the Chehalis River as flood mitigation. Both RCC dam options require mineral aggregate products for both RCC and conventional concretes, as well as cement and fly ash or other pozzolans. Generally, the American Society for Testing and Materials' (ASTM) C33 quality requirements guide the use of rock for riprap and rock and sand for RCC, concrete, and drain and filter materials. This report evaluates potential sources for each of these products, considering three important factors: quantity, quality, and economics. Similarly, sources for cement and pozzolan are considered.

Although ASTM C33 provides common quality standards, similar standards are adopted by state and federal transportation departments and frequently guide the development and use of commercially supplied aggregate materials for concrete, road base, and asphalt paving. Consequently, commercial supply of aggregate, concrete, and paving reveals much about the regional quality and availability of materials, including the potential to develop new or noncommercial sources. RCC for dams may consist of lower quality materials because the design often calls for a massive gravity structure that relies on conventional concrete or enriched facing elements to provide surface durability and water barrier, not needed within the dam body. Common quality indicators include tested properties such as Los Angeles (LA) abrasion, specific gravity, absorption, and the rock's mineral composition. Physical and chemical durability, strength, and acceptable alkali reactivity limits are the characteristics evaluated during testing. The mix design ultimately considers much more than compressive strength including workability, initial set and strength maturity, seepage design, lift cohesion, and, potentially, tensile strength.

A medium-cementitious RCC mix (i.e., 200-250 pounds per cubic yard (lbs/cy, 50 percent fly ash replacement) is anticipated, and all of the required products – considering quality, quantity, and commercially acceptable pricing – are regionally available. Extensive regional basalt deposits provide commercial supply, and closer and possibly more economical basalt sources potentially could be developed or expanded to meet the RCC and riprap needs for the project. Filter sands, concrete sand, and to a lesser degree, drain rock and concrete aggregate are available predominantly from more distant commercial sources that rely on alluvial and river sand and gravels. The alluvial deposits are generally farther away from the project, and new alluvial sources are unlikely to be developed for the Chehalis Dam project. Mix design development and a mix design program would be useful in evaluating the more favorable quarry locations as well as lower-quality, but less costly, basalts for potential use. In the case of cement and pozzolan, cement is produced in the Pacific Northwest, and imported products are also competitive. Fly ash, a by-product of coal-fired power generation, is regionally available, but socio-political pressures, together with high seasonal regional hydropower production, make future supplies somewhat unpredictable and risky. Implementing a mix design program as design progresses will be important to evaluate the wet and hardened properties and to support related design choices.

2 REFERENCES AND STANDARDS

There are a number of industry-developed guidelines for RCC materials that should be used when developing mix design programs and specification requirements. These guidelines shall serve as the overall basis for HDR mix design programs. They include the documents listed below. Note that the reference documents outlined below are not a comprehensive list of references, and others should be identified and consulted based on the specific needs of each individual project.

1. International Commission on Large Dams (ICOLD) Bulletin 126 Update, Roller Compacted Concrete Dams, 2014 to present (original bulletin published 2003).
2. ICOLD Bulletin 165, Selection of Materials for Concrete in Dam, November 2013.
3. ACI 207.5R, Roller Compacted Mass Concrete, American Concrete Institute, August 2011.
4. U.S. Army Corps of Engineers (USACE), Roller Compacted Concrete, Engineering Manual (EM) 1110-2-2006, 15 January 2000.
5. USACE, Standard Practice for Concrete for Civil Works Structures, EM 1110-2-2000, 1 February 1994.
6. U.S. Department of Interior, Bureau of Reclamation, Roller-Compacted Concrete, Design and Construction Considerations for Hydraulic Structures, 2005.
7. Portland Cement Association (PCA), Design Manual for Small RCC Gravity Dams, 2003.
8. PCA, Design Manual for RCC Spillways and Overtopping Protection, EB218, 2002.
9. PCA, Diagnosis and Control of Alkali-Aggregate Reactions in Concrete, IS413.OIT, 1997.
10. American Society of Civil Engineers (ASCE), Roller-Compacted Concrete, Technical Engineering and Design Guides as Adopted for the U.S. Army Corps of Engineers, No. 5, 1994.

The testing standards commonly used for RCC aggregate source evaluation and mix design studies are shown in Table A-1 in 0. These testing standards should form the basis of a progressive aggregate source evaluation and mix design studies completed over the course of planning and final design work. Other testing may be indicated or performed for new potential quarry sites or when unusual concerns related to aggregate are indicated by source geology. Specific project or site circumstances may warrant considering variances from these standards but only when evaluated through a thoughtful laboratory mix design program.

3 RCC MIX DESIGN OBJECTIVES

The primary objective of the mix design program performed during design is to identify and demonstrate that suitable aggregate, cementitious materials, water, and other additive materials are available to meet design objectives. Important material property and source considerations include:

- Compression and tensile strength of parent material and lift joints
- Durability of aggregates
- Permeability of cured RCC material
- Availability of aggregates, sands, cement, and admixtures
- Economic viability of material sources

An additional objective is to evaluate construction requirements that would be required to produce a consistent and economical mix that meets quality requirements. Construction considerations include:

- Specification requirements to address heat generation
- Specification and joint design requirements to address shrinkage
- Specification and design considerations to mitigate for creep during the life of the structure
- Specification requirements for aggregate quality and gradation
- Cementitious material quality and content, water content and control during construction as it pertains to target workability
- Additional design considerations necessary to produce a consistent material that is uniform and has well-bonded lift surfaces
- Mix sensitivity to water content

For Chehalis Dam, based on experience, an understanding of design requirements, and concerns related to project costs, the following overall mix design objectives form the basis of establishing an appraisal-level mix design and ultimately guiding a mix design program:

1. <u>Category of mix design</u>: Medium cementitious RCC depending on seepage control strategy.
2. <u>Seepage control strategy</u>: Both "separate" (upstream facing system) and "uniform" material with a mix consistency to allow for the use of grout enriched roller-compacted concrete (GERCC) procedures for the upstream and downstream facing systems.
3. <u>Unit Weight</u>: 142 to 150 pounds per cubic foot (pcf)
4. <u>Strength</u>: 1,000 to 1,700 pounds per square inch (psi) unconfined compressive strength at 28 days (2,100 to 3,600 psi at 1 year). Final mix design proportions will be based on lab test results,

design analysis results, facing systems requirements, constructability considerations, and costs. Table 3-1

5. and **Error! Reference source not found.** provide example cementitious content versus strength gains. Depending upon final aggregate selected for the initial mix design program, the target cementitious contents may be increased by 10-15 percent to compensate for potentially lower-strength basalts that exhibit lower specific gravities and acceptable but relatively high LA abrasions for basalt.
6. Durability: Moderate to severe freeze-thaw durability will be needed for exterior surfaces of the dam that are continually exposed to moisture and free water. Some erosion or water impact durability in the spillway and stilling basin may be required if conventional concrete facing is not utilized. If design leads to conventional concrete facing systems, thereby eliminating environmental exposure, freeze-thaw durability and erosion may not be factors.
7. Other: To account for temperature and other conditions at time of placement, and to increase time to initial set for more favorable untreated lift cohesion, an ASTM Type D super plasticizer and set retarder additive should be included as part of mix design studies.

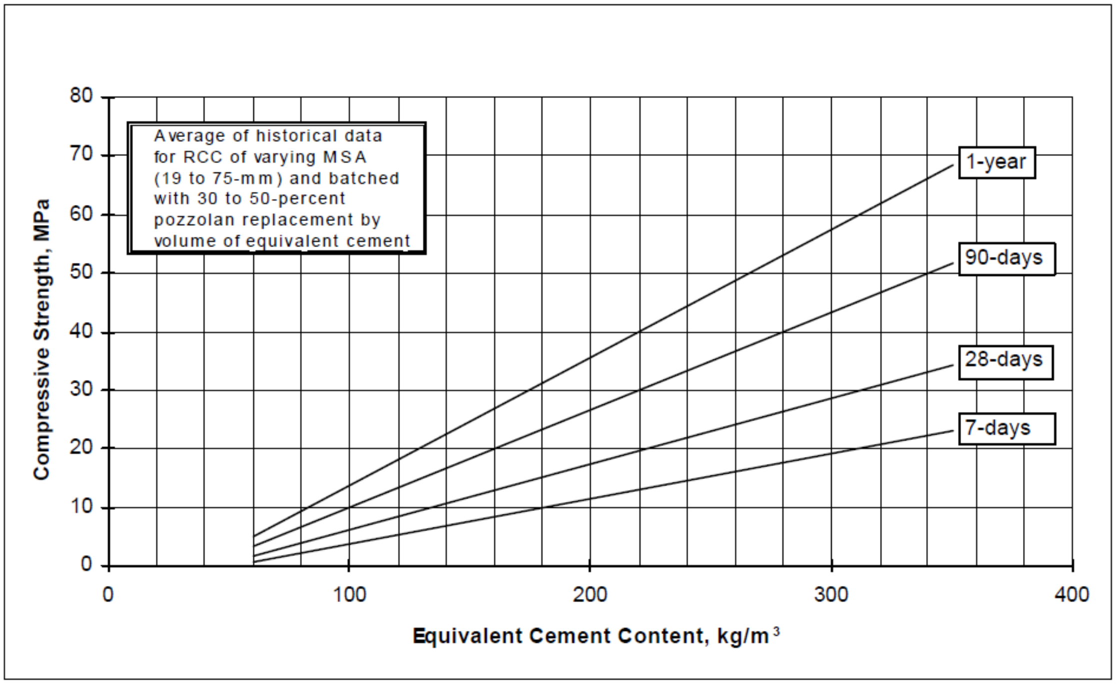
Table 3-1
Example – Cementitious Content vs. Compressive Strengths

TOTAL CEMENTITIOUS CONTENT		ESTIMATED COMPRESSIVE STRENGTH (PSI)	
KG/M^3	LBS/YD^3	28-DAY	1 YEAR
104	175	1015	2176
125	210	1378	2828
148	250	1668	3626

Note: kg/m^3 = kilograms per cubic meter; lbs/yd^3 = pounds per cubic yard
Developed from EM 1110-2-2006 (see **Error! Reference source not found.** below)

Figure 3-1

Equivalent Cement Content versus Compressive Strength; Average Historical Data for RCC Batched with Pozzolan (From EM 1110-2-2006)



4 RCC MIX MATERIAL CONSIDERATIONS

Investigation and selection of materials for RCC is a vital part of an RCC dam design process. The recommendations in USACE EM 1110-2-2000 and other references should be used as appropriate. Some specific considerations to address RCC best practices for materials selection are summarized in the following sections.

4.1 Aggregate

Depending on the size of the dam, local geology and related commercial product availability, aggregates for RCC production can be obtained from a variety of sources. Potential sources could include onsite quarries or gravel deposits (when suitable materials are available), nearby commercial sources (with hauling and local stockpiling of materials), or a combination of both.

High quality aggregate provides many potential benefits to the overall properties of the RCC such as:

- Low susceptibility to alkali-aggregate reaction (AAR)
- High concrete strength and modulus resulting from strong and high aggregate modulus
- Maintains optimal grading supporting high density, high modulus, reduced cement content
- Improved resistance to freeze-thaw breakdown, reduced concrete permeability, and improved water content requirements and workability due to low porosity and permeability of aggregates
- Reduced shrinkage due to low shrinkage aggregate
- Reduced volume of RCC required to achieve stability due to high specific gravity of aggregates
- Increased strength and improved workability due to favorable aggregate particle shapes

Aggregate characteristics that may adversely affect the properties of RCC include:

- Weak particles
- Flat and elongated particles
- Dimensionally unstable material
- Deleterious particles and minerals such as mica
- Unsound particles such as clay and organic material
- High and variable water absorption
- Amorphous silica and other compounds that might deleteriously react with cement or its hydration products
- Water soluble minerals such as salt and gypsum

Typically, a well-graded, high quality aggregate material meeting requirements of ASTM C33 is used and can yield maximum density when compacted in 12- to 14-inch loose lifts with a 10,000-pound, self-propelled, smooth vibratory drum roller. In some cases, a degree of aggregate quality relaxation may be an appropriate balance of mix quality and economic objectives.

Although current international guidelines suggest maximum size aggregate (MSA) in the range of 40 to 60 millimeters (mm) (1.6 to 2.4 inches), experience suggests that the MSA should typically be limited to 25 to 37 mm (1.0 to 1.5 inches), to reduce segregation during transport and placement. This might result in some overall modulus reduction; however, such a limit improves tensile and lift joint strength characteristics. Mix designs should also consider common commercial products when commercial aggregates are the likely economical choice.

In addition to the MSA requirements outlined above, the sand content should be relatively well-graded and range from 40 to 60 percent of the total aggregate grading by weight. The fines content (minus 200 sieve) should be non-plastic and about 4 to 8 percent of the grading by weight. Fines are an important component of the overall "paste" content of a mix. When sufficient "non-plastic" fines are not available from quarry or aggregate operations, supplemental fines for "paste" should be considered by using additional pozzolan (fly ash or slag). Use of coarse aggregate that is crushed and more than 50 percent of the total grading can help tensile strength, density, compaction, and stability under a vibratory roller. Although natural sand may help workability, it is not necessary to specify natural rather than crushed or manufactured sand.

A well-designed highway base coarse is typically ideal when aggregate is: 1) modified to the above requirements; 2) hard, durable, and chemically inert; and 3) generally spherical or cubic shaped. Such an aggregate will easily compact to high density, maximizing the strength contribution from the aggregate. With the appropriate paste (20 to 23 percent) and moisture content, the RCC mix will have suitable paste mobility without significantly reducing density and strength. As previously noted, this type of aggregate grading typically results in higher concrete modulus and reduced cementitious material content requirements. Example gradations for aggregate used in RCC is shown on the upper two curves on Figure 4-1 (U.S. Bureau of Reclamation [USBR] 2005) and Figure 4-2 (PCA 2003). For larger dams, some enhancements to gradation requirements may be considered consistent with industry best practices. This may include establishing gradation requirements for multiple stockpiles depending upon the nominal maximum size of aggregate (NMSA). Multiple stockpiles may be required or beneficial to allow RCC plant feed flexibility, to provide mixed gradation control, and to minimize larger aggregate segregation. Crushing to get roughly spherical or cubic shapes often requires secondary and tertiary crushing as well as use of crushing units, such as impact crushers, that are less susceptible to producing flat and elongated particles for a given rock type.

Figure 4-1
Average Gradation for Various Projects (from USBR 2005)

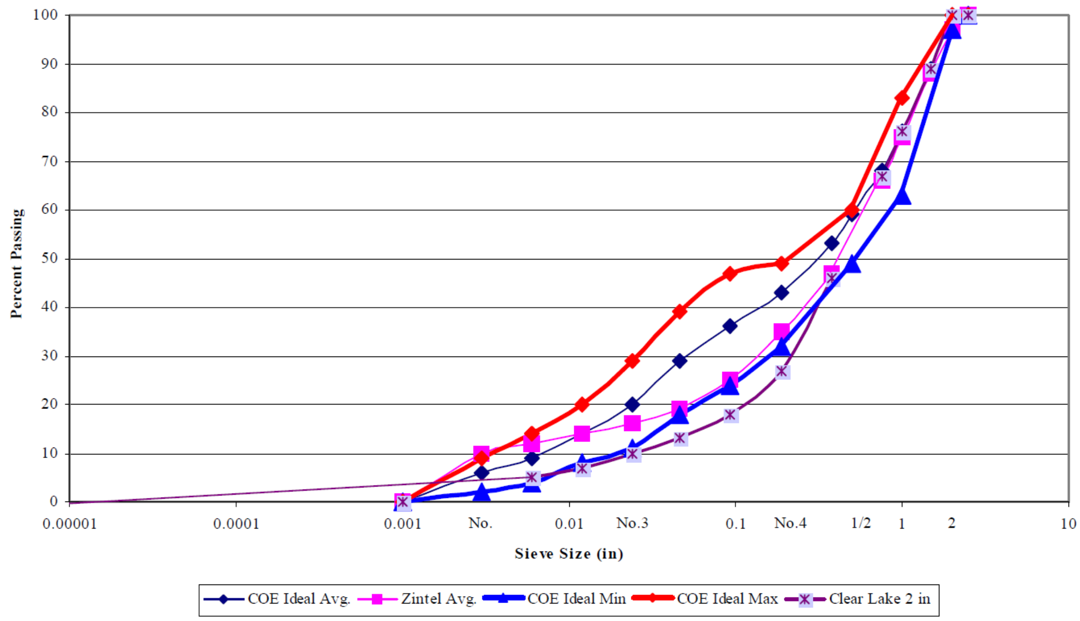
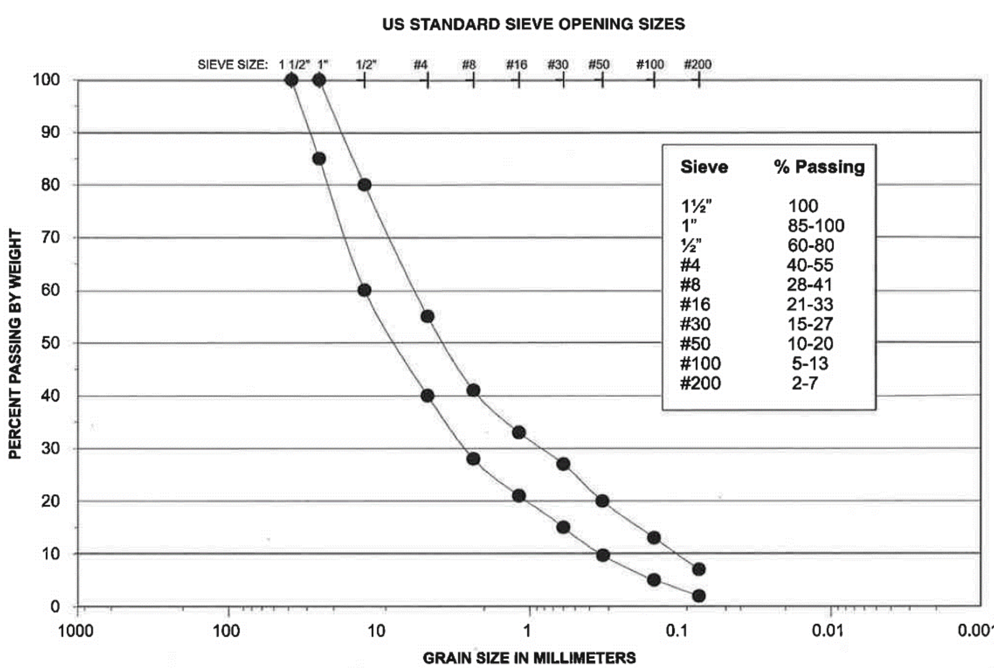
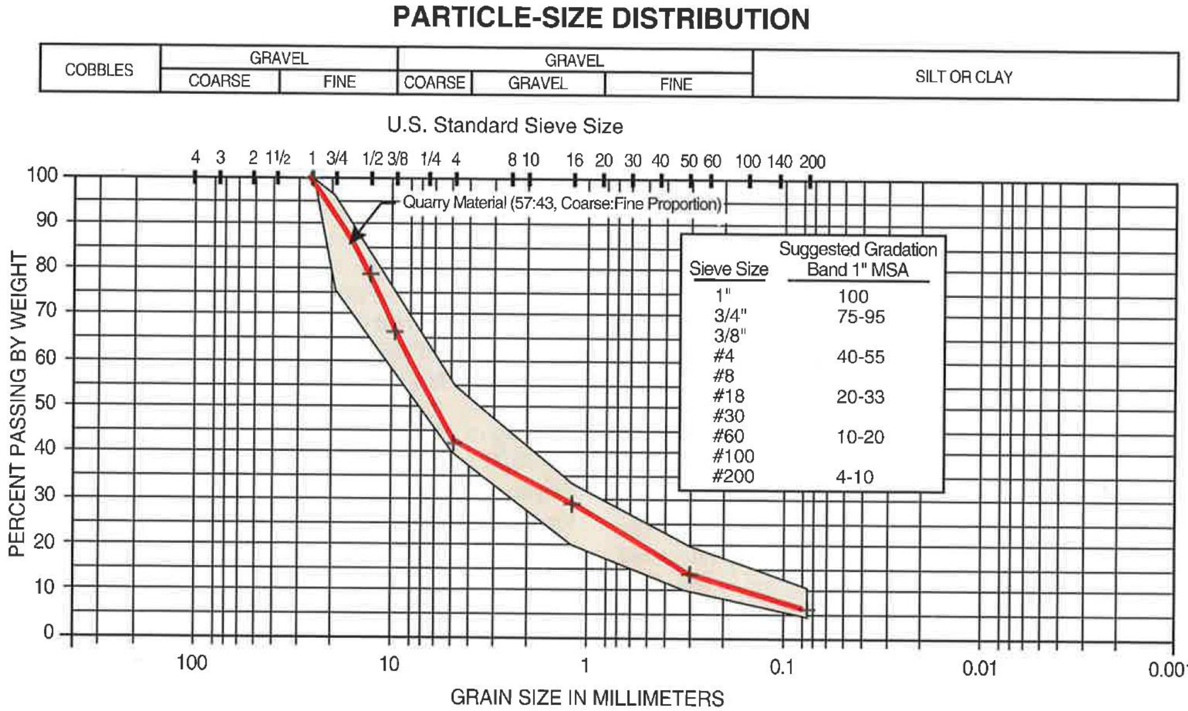


Figure 4-2
Typical Aggregate Gradation Band for a Small RCC Dam (from PCA 2003)



A typical grading curve for aggregate used on smaller projects with an MSA of 1 inch is shown on Figure 4-3-3 (PCA, 2002).

Figure 4-3
RCC Aggregate Design Gradation Bands (from PCA 2002)



Preliminary mix design targets and the mix design program depend on a good understanding of potential aggregate and material sourcing. Evaluation should consider site-based deposits, in-proximity active and inactive quarries or pits, and commercial sources. Understanding site and regional geology may provide valuable insight when considering noncommercial options. Often, commercial concrete and asphalt supply relies on aggregate of a quality similar to RCC. Consequently, determining what sources supply, or have supplied, commercial concrete operations and public asphalt paving projects can lead to information about specific sources, types of sources, and factors that might affect availability and economics. In more remote areas, commercial supply potential might not be clear or evident and it might be harder to determine the nature of locally available material quality and availability. Inspecting older, existing concrete infrastructure can provide insight into regional rock alkali reactivity susceptibility. Surface or outcrop sampling, test pitting, or even more comprehensive drilling and test excavation and test blasting programs should be considered where that information will be of high value to the project design. For example, if commercial supply seems practical, but site aggregates would provide significant economic or other benefits, it might be prudent to perform testing that informs whether site materials are available and of sufficient quality.

As design progresses, the mix design program should be developed to guide or verify the designed mix properties and provide a sound basis for a more reliable cost estimate. The program also will provide information to support specification development including measurement and payment, responsibility assignment, and contractor mix verification requirements. Project specifics will determine the nature and extent of design-level aggregate source and aggregate quality investigation and characterization. In addition to laboratory testing, evaluation components might include surface and test pit sampling, drilling, test excavations, test blasting, test crushing, and test placements.

4.2 Cement

Alternative sources of cement meeting the requirements of ASTM C150 Type I or Type II that can be obtained in reasonable proximity to the Chehalis Dam site should be evaluated. Type II Portland cement is more commonly used with RCC because of its low heat generation characteristics at early ages and its longer set times. Type II cement has an optional requirement for low-alkali content that could be important for aggregate sources that have AAR concerns. Heat generation concerns are less significant when a pozzolan is used in the mix, opening the door to the use of Type I cement. The regional evaluation of alternative cement and pozzolan sources should be conducted to make a final selection of cement for use in the laboratory mix design program. Type II is the preferred choice, or even a Type II with more aggressive heat generation limitations, if economically available.

4.3 Pozzolans

Pozzolans are a broad class of siliceous or aluminous materials which react chemically with calcium hydroxide to form compounds possessing cementitious properties. Commonly used pozzoloans include flyash or slag. Although a variety of pozzolans can be considered, the most commonly used is a Class F fly ash. Class F fly ash contributes to lower heat generation at early ages, may be used to replace cement and reduce costs, acts as a mineral filler to improve workability, and helps to delay final set. Class F fly ash normally resists both alkali-silica reaction (ASR) and sulfate attack. Other possible sources of pozzolan, including Class C fly ash, ground slag, or other reasonably available materials should be evaluated if a suitable Class F source is not located. Pozzolans should meet the requirements of ASTM C618.

Cement, Class F fly ash, and slag are currently available in the region with regional and national suppliers including CalPortland, Lafarge, Ash Grove, and Lehigh. Canadian (i.e. Genesee, Alberta) fly ash is also available, through regional rail terminals. In addition to regional cement mills, CalPortland, for example, reportedly supplies imported Asian cement from its parent, Tokyo-based Taiheiyo Cement Corporation. Recent and further coal-fired plant generation reductions are anticipated that will make fly ash a tight and increasingly unpredictable commodity. Additionally, strong regional hydropower generation during higher flow months seasonally reduces coal-fired power generation and suspends or limits fly ash availability. Design and mix design program planning and risk-mitigating strategies should consider alternative pozzolans as well as lower fly ash replacement mixes.

4.4 Cement and Fly Ash Sources

A preliminary assessment of cement and fly ash sources for the Chehalis Dam project is included in Table 4-11:

Table 4-1
Cement and Fly Ash Sources

SOURCE	LOCATION (DISTANCE)	PRODUCT(S)
CalPortland	Genesee, Alberta (900 mi)	Fly Ash
CalPortland	Portland, OR (terminal – 110 mi)	Cement/Fly Ash
CalPortland	Seattle, WA (terminal – 110 mi)	Cement/Fly Ash
Lafarge	Seattle, WA (110 mi)	Cement
Ash Grove	Durkee, OR (430 mi), and rail	Cement
Lehigh	Redding, California (530 mi), and rail	Cement
Western Pozzolan	Reno, NV (650 mi)	Fly Ash
Nevada Cement Co	Fernley, NV (680 mi)	Cement

4.5 Water

Water used for RCC production must be free from objectionable quantities of silt, organic matter, salts, and other impurities. Specifications should limit the soluble sulfate content to less than 3,000 parts per million. Aggregate and plant wash water is not acceptable. Any ice used in the mix water to control temperature should be produced from water meeting general mix water requirements (USBR 2005). It should be noted, however, that the low added water content of RCC practically limits the effectiveness of using ice to control the placing temperature of the RCC; other strategies for mix temperature control will likely be required. Chehalis River water is presumed to be available and of adequate quality.

Available water supplies should be assessed using the criteria outlined in the USACE EM 1110-2-2000 (also ASTM C94 and ASTM C1602). After a preferred source of water is identified for a project, it should be appropriately incorporated into the mix design test program because RCC performance, setting times, and strength development can vary significantly based on water source.

For the Chehalis Dam mix design program, potable water can be used for testing; however, comparative bedding mortar samples should be prepared with both potable water and the Chehalis River water to identify any water quality concerns. If concerns are identified, additional mix design studies and evaluations will be based on use of only site water.

4.6 Other Additives

Admixtures for RCC are typical of conventional concrete and should conform to ASTM specifications including C494. Although the USBR and USACE claim some success in the use of air-entraining admixtures (AEAs) (ASTM C260), other industry guidance typically focuses on the use of chemical water-reducing admixtures (WRAs) and set retarders to achieve mix design objectives related to strength, durability, and permeability. ASTM C494 classifies several types of WRA and set-controlling admixtures. Type A (water reducing) and Type D (water reducing and retarding) admixtures have been successfully used in mass RCC mixtures. The Type D additive set-retarding characteristics can be enhanced when used with Class F pozzolans. Retarders can be especially beneficial in hot weather for improving lift joint integrity.

The dosage rate of WRAs may depend on the cement to pozzolan ratio, mixture workability requirements, and aggregate grading. Mixtures using high pozzolan contents may experience prolonged delay (up to 36 hours) in setting when combined with low concrete temperatures and Type B (set-retarding) or Type D WRAs. Dosages of WRAs can be several times as much as recommended for conventionally placed concrete because of the drier consistency of RCC. Excessive dosages can result in minimal improvement or detrimental impact on short-term and long-term performance. Dosage ranges should be based on laboratory test results where the effects of varying dosages are evaluated. Admixtures in some cases may not be as effective with less workable RCC with Vebe times over 20 to 30 seconds.

The mix design program should include specific details on source and dosage of admixtures to be used in set time testing and process for determining dosage used for the cylinder preparation. Additionally, the regional availability of admixtures in the large quantities required should be evaluated.

5 RCC MIX MATERIAL ASSESSMENT

5.1 Regional Geologic Overview

The region around the Chehalis Dam project site has a limited number of existing quarries, located within basalt deposits. The predominant regional basalt unit is the Columbia River Basalt Group (CRBG). The CRBG is a Miocene aged basalt that is deposited over an extensive area in Oregon and Washington. The CRBG is a group of basalt flows that were deposited subaerially (surficial flows). The upper portion of each individual flow can be highly vesicular and weathered, but most of each flow is composed of strong basalt with close columnar jointing. The project site and its immediate environs is outside the CRBG depositional extents, but significant deposits of CRBG are located north and east of the project site.

The project site is located on the northern limb of the Willapa Hills anticline and is underlain predominantly by Eocene aged Crescent Formation basalt. These basalts were deposited in a marine environment and therefore have more deleterious interbedded material such as claystone and siltstones and are variable in quality and lateral extent. Previously identified potential dam site quarry locations are located in these deposits. Overall, deposits of CRBG are most likely to provide a more adequate aggregate material but are located farther from the project site, likely at commercial quarries.

5.2 RCC Aggregate

Potential regional aggregate sources as well as potential sources on or near the Chehalis Dam project site were initially identified in previous site characterization evaluations. Quarried basalt makes up the vast majority of the regional sources, with alluvial and river aggregate and sand commercially available within 30 miles of the potential dam site.

A desktop appraisal (HDR, Combined Dam and Fish Passage Alternatives Technical Memorandum, October 2014) and geological surveys were initially performed and included some limited aggregate quality testing for Alkali-silica reaction. These investigations have been followed by drilling, seismic refraction investigations, and additional aggregate quality testing (HDR and Shannon and Wilson, Phase 1 Site Characterization Technical Memorandum, August 2015). Additionally, the September 2016 Site Characterization Report (HDR) provides laboratory testing results of samples taken from three quarries investigated near the dam site. The results of these combined studies have preliminarily identified five candidate sources for the project RCC aggregate supply. Figure 5-1-1 shows the potential Chehalis Dam site and three potential local quarry locations including an inactive permitted Weyerhaeuser quarry (Rock Creek), and identifies two additional regional commercial basalt quarries that could be potential aggregate sources.

A data summary of identified quarries in close proximity to the Chehalis Dam site is presented in 5-1. The project size and aggregate quantity demand may dictate that more than one source is engaged or developed to produce sufficient quality aggregate at an acceptable rate. Commercial suppliers will be cognizant of a major project's impact to their ongoing operation and their reserves, which will factor into commercial aggregate supply viability and pricing.

Site-based or proximal development of an existing inactive or new quarry (quarries Q1-Q3) appears feasible and potentially favorable. Test data suggests some potential absorption and possible durability concerns with Q1 or Q2. These quarries should be cautiously advanced along with other options (Q3 and commercial quarries). Although preliminary investigations seem favorable, significant assumptions must be made as a result of the limited investigation at the concept level.

Table 5-1 provides comparative data, which leads to Q3 as the primary basis and Q1 as the alternative quarry for concept-level cost and design purposes. Cores showed good recovery and acceptable LA abrasions, and imply massive deposits. Some of the aggregates produced in Q1 and Q2 could be poor to marginal as a result of variable and low strengths, unusually high absorptions, relatively low specific gravities, and the relatively high-for-basalt LA abrasions test results. Appendix B contains a table excerpt from the Site Characterization Report (HDR 2016) that includes some aggregate quality test results along with the Washington State Department of Transportation (WSDOT) and the Federal Highway Administration (FHWA) aggregate criteria. The maximum allowable absorption typically allowed by WSDOT/FHWA is 3 percent. If the tests are truly indicative of the sources, sufficient strengths might require an increase in cementitious material. In addition, some of the aggregates may be susceptible to handling breakdown, and high and variable water demand will challenge admixture effectiveness and mix consistency. The dam design may allow use of some poor to marginal aggregate, either in a mixed blend or as a mix within lower stress zones of the dam. Additional investigation and testing, along with inclusion within the mix design program, are clearly warranted.

Figure 5-1

Potential RCC Aggregate Sources in Proximity of the Chehalis Dam Site

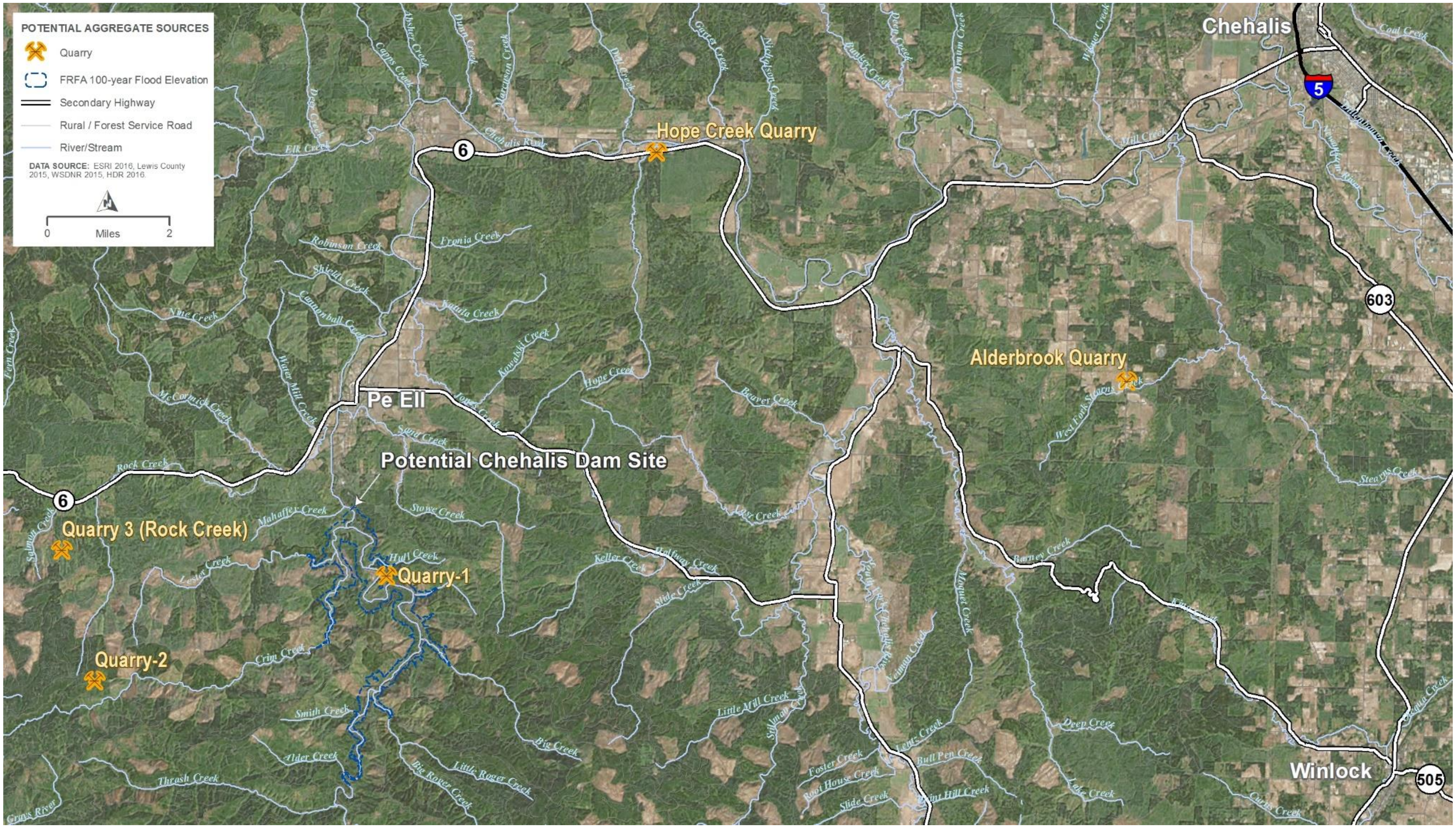


Table 5-1
Quarry Overview

QUARRY NAME		HOPE CREEK	ALDERBROOK	Q1	Q2	Q3 (ROCK CREEK)
LOCATION	LATITUDE	46.632	46.581	46.53	46.504	46.533
	LONGITUDE	-123.198	-123.034	-123.287	-123.383	-123.397
	DRIVING DISTANCE FROM SITE (APPROX.)	11 mi	24 mi	2 mi	7 mi	8 mi
	DRIVE TIME FROM SITE (APPROX.)	15 min	36 min	8 min	20 min	15 min
LABORATORY RESULTS	PETROGRAPHY	Altered Basalt	Altered Basalt	Altered Basalt	Altered Basalt	Altered Basalt
	RANGE OF LA ABRASION (%)	17.9	16.5	26.8 - 27.5	24.1 - 24.8	18.9
	RANGE OF ABSORPTION	6.75	2.71	4.69 - 8.26	3.72 - 4.04	1.37
	RANGE OF COMPRESSIVE STRENGTH (PSI)	NA	NA	1,834 – 5,237	9,655 – 13,210	NA
	RANGE OF BULK SPECIFIC GRAVITY SSD[1] (%)	2.71	2.71	2.49 – 2.65	2.69 – 2.71	2.78
	RANGE OF ASR[2] – 16 DAY	0.215	0.212	0.076 – 0.124	0.034 – 0.036	0.011 – 0.035
	ASR[2] – 1 YEAR	0.024	0.022	NA	NA	0.028
PHASE TESTED		1	1	2	2	1 and 2
TEST BASIS		1 grab sample	1 grab sample	3 core samples from 38-50', 84-95' and 127-140' depths	2 core samples from 15-27' and 45-55' depths	1 grab sample during each phase
QUANTITY CONFIDENCE		No Opinion	No Opinion	Moderate	High	High
ESTIMATED COST RANGE (BID BASIS)		$17 - $29 /ton	$17 - $27 /ton	$13 - $26 /ton	$14 - $24 /ton	$15 - $26 /ton
STATUS		Abandoned	Active	New Quarry	New Quarry	Inactive
QUALITATIVE RANKING		5	4	2	3	1

Notes:
SSD: Saturated Surface Dry
ASR: Alkali Silica Reactivity

5.3 Mix Design Testing Program

Considering the strong potential to use site-developed aggregates, and to guide design and specification development, a mix design program assuming site-developed aggregate sources is warranted. A staged aggregate source evaluation and mix design program should be considered, beginning in the preliminary design stage. A mix design and testing program should be completed using well-graded basalt aggregate and assessing three independent variables: 1) strength as a function of cementitious material content, 2) fly ash replacement percentage, and 3) use of basalt aggregate of varying quality. Because the national coal-fired power industry is changing and with it the potential supply of fly ash, consideration should be given to evaluating lower fly ash replacement percentages, requiring other design means to address thermal and crack control issues.

It is important to consider a mix design program during the preliminary design phase to begin the process of addressing construction and cost risk factors and to provide guidance in developing the construction contract to identify the preferred requirements for project construction.

Overall, the aggregate evaluation and primary mix testing program would include to following initial activities:

1. Further drilling and testing of the source rock in Quarries 1, 2, and 3 should be completed for additional evaluation of potential aggregate properties including LA abrasion and other possible adverse indicators of low specific gravity, high absorption, and variable low strength.
2. Estimates of the available quantities of aggregate available for Quarries 1, 2, and 3, considering practical limits for overburden and objectionable materials within the deposit.
3. Establish a range of mix proportions to be considered in preliminary mix designs for testing and evaluation, and for consideration in updates to estimated project costs.
4. Identify to what degree higher absorption, lighter and lower-strength basalt aggregates are included in the test program. Once test results are available, they would be evaluated to identify if such materials are acceptable and, if so, to what degree.

After the potential quarry source characteristics are better understood and considered in a mix design program, the results may provide flexibility for contractors which could result in improved bidding conditions. Given enough time and specification flexibility, contractors might explore and develop even closer proximity sources within the reservoir limits.

Preliminary design efforts should explore what measures need to be in place to allow and facilitate the permitting and development of the identified Q1 through Q3 quarries or alternative sources within the reservoir area.

Construction cost estimates assume use of Quarry Q3. Further site investigation and subsurface testing is required to generate greater confidence in use of Quarry Q1 and potentially other excavated materials from the dam site and/or reservoir basin. Aggregate testing, as well as preliminary mix design evaluation, would be performed during preliminary design with rock obtained from existing stockpiles at the Q3 site and through a laboratory crush of existing and foundation characterization core samples from Q1 and Q2.

The benefit and need for a design-level test crush and test section will be reviewed as the material evaluation, including the RCC mix design program, advances. Such a program may be advisable during final design should a mix design laboratory testing program indicate some important issues that need to be resolved prior to bidding and initiation of construction.

Attachment A
Summary RCC Test Standards

SUMMARY TEST STANDARDS

Table A-1

Summary of Testing References and Standards - RCC Aggregate and Mix Design Development

NO.	TEST	REF/STANDARD	PRIMARY	SECONDARY	COMMENTS
AGGREGATE AND AGGREGATE QUALITY					
1	Aggregate Quality	ASTM C33	X		
2	Sampling of Aggregates	ASTM D75, ASTM D3665	X		
3	Petrographic examination	ASTM C295	X	X	This test may not be required for commercial aggregate sources with proven performance and no AAR. Include estimate of percent crushed aggregate in coarse aggregate particles. Should be performed for onsite quarries to verify mineralogy.
4	Gradation	ASTM C136	X		
5	Minus No. 200 Sieve	ASTM C117	X		
6	Specific Gravity and Absorption	ASTM C127 ASTM C128	X		
7	Light-weight Particles	ASTM C123		X	This test should be performed if there are concerns related to low unit weights observed in lab tests or other history of low-weight materials from other uses.
8	Resistance to Degradation – small size coarse aggregate by abrasion	ASTM C131	X		For materials < 1½-inch size
9	Resistance to Degradation – large size coarse aggregates by abrasion	ASTM C535		X	For materials > ¾-inch size (not required when max particle size less than 1½ inches).
10	Atterberg Limits	ASTM D4318	X		
11	Sand Equivalent	ASTM D2419		X	
12	Fineness Modulus	ASTM C136		X	
13	Water Absorption Coarse and Fine Materials	ASTM C566, ASTM C127	X		C566 only used for aggregate > 2-inch size.

NO.	TEST	REF/STANDARD	PRIMARY	SECONDARY	COMMENTS
14	Clay lumps and friable particles	ASTM C142		**X**	Testing should be performed if concerns are identified without testing or history with quarry source.
15	Moisture/Density	ASTM D1557	**X**		
16	Flat and Elongated Particles	ASTM D4791	**X**		
17	Sodium and Magnesium Sulfate Soundness	ASTM C88		**X**	Perform testing if indicated by petrographic examination. Petrographic, freeze-thaw, ASR, and LA Abrasions are best guides, and this test should only be used if requested by client.
18	Alkali-Silica Reaction, Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar Bar Method)	ASTM C1260, ASTM C1293 ASTM C227 ASTM C1567	**X**	**X**	Perform testing if indicated by petrographic examination. Required if potential for AAR is possible based on other primary tests or reported AAR concerns.
19	Alkali – Carbonate Rock reaction	ASTM C586, ASTM C1105		**X**	Perform testing if indicated by petrographic examination.
20	Chert	ASTM C123		**X**	Perform testing if indicated by petrographic examination.
21	Organic Impurities	ASTM C40		**X**	Perform testing if indicated by petrographic examination.
22	Coal and Lignite	ASTM C123		**X**	Perform testing if indicated by petrographic examination.
23	Bulk Unit Weight or Density	ASTM C29		**X**	
MIX PROPORTIONING AND PROPERTIES					
24	Moisture/Density	ASTM D1557	**X**		
25	Vebe Consistency	ASTM C1170	**X**		Both Vebe and moisture density methods to be used in mix design.
26	Test Cylinders in Molds Using a Vibrating Hammer (Hilty Hammer or Equivalent)	ASTM C1435		**X**	May be required if target Vebe time exceeds 30 seconds.
27	Test Cylinders in Molds Using a Vibrating Table	ASTM C1176	**X**		
28	RCC Mix Design for Dam	ACI 207.5R	**X**		

NO.	TEST	REF/STANDARD	PRIMARY	SECONDARY	COMMENTS
29	Reducing Aggregate Samples to Testing Size	ASTM C702	**X**		
30	Sampling Aggregate	ASTM D75	**X**		
31	Freeze/Thaw Durability	ASTM C666, Procedure A		**X**	Some judgment required. RCC samples typically perform poorly with this test without air entrainment. May be a required test in locations where a structure may be critically saturated and subjected to freezing and thawing cycles.
32	Abrasion/erosion resistance	ASTM C1138		**X**	
33	Compressive Strength of Concrete Cylinders	ASTM C39	**X**		
34	Submerged Unit Weight of Cylinders	ASTM C642	**X**		
35	Set Time	ASTM C191	**X**		
36	Portland Cement	ASTM C150		**X**	
37	Fly Ash	ASTM C618		**X**	
38	Slag	ASTM C989		**X**	
39	Water	ASTM C94 ASTM C1602	**X**		
OTHER FIELD AND LABORATORY TESTING METHODS					
40	In-place Density and Water Content by Nuclear Gauge	ASTM D6938	**X**		
41	Sand Cone Density Test	ASTM D1556		**X**	
42	Portland Cement	ASTM C150		**X**	
43	Fly Ash and Natural Pozzolans	ASTM C618, and C311		**X**	
44	Chemical Admixtures for Concrete	ASTM C494		**X**	
45	Drilled Cores	ASTM C42		**X**	
46	Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar Bar Method0	ASTM C1567		**X**	
47	See Note 1				

Notes:
Additional test procedures for thermal properties, shear strength, tensile strength, etc. to be added based on project need.

Attachment B
Excerpt from the Site Characterization Report

EXCERPT FROM THE SITE CHARACTERIZATION REPORT

Table B- 1
Aggregate Source 2015 Test Results

QUARRY NAME	HAUL DISTANCE[1] (MILES)	SAMPLE DEPTH (FT)		UCS (PSI)	ABSORPTION	BULK SPECIFIC GRAVITY (SSD)	LA ABRASION	ASR (16-DAY)
		FROM	TO					
Quarry 1	2	38	50	5,763	6.46	2.60	27.1	0.08
		84	95	1,834	4.69	2.65	26.8	0.076
		127	140	5,237	8.26	2.49	27.5	0.124
Quarry 2	7	15	27	13,2102	4.04	2.69	24.8	0.034
		45	55	9,6553	3.72	2.71	24.1	0.036
Quarry 3 (Rock Creek)	8.3	NA	NA	NA	1.37	2.74	18.9	0.035
WSDOT4/FHWA5 Criteria	--	--	--		3 max.	2.55 min.	35 max.	See Note 6

Notes:

1. Haul distance in miles measured with Google Earth
2. UCS sample depth was 6.0 to 6.8 ft
3. UCS sample depth was 71.1 to 72.2 ft
4. WSDOT = Washington State Department of Transportation
5. FHWA = Federal Highway Administration
6. For ASR (16-day test), a test value of 0 to 0.10 is innocuous, 0.11 to 0.20 is acceptable if supplemental testing confirms expansion is not due to ASR, and greater than 0.20 requires additional testing.
5. SSD = saturated surface dry

Source: HDR 2016

Appendix F
Opinion of Probable Dam Construction Cost, Schedule and Constructability

Chehalis Basin Strategy

Appendix F
Opinion of Probable Dam Construction Cost, Schedule and Constructability

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

Prepared for: State of Washington Recreation and Conservation Office and Chehalis Basin Work Group

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Attachment B	Summary Table of Estimate Refinements Leading to the Final Conceptual Design OPC
Attachment C	Aggregate Sourcing Information Along with the Development of RCC Unit Pricing
Attachment D	RCC Placement Summary

ACRONYMS AND ABBREVIATIONS

AACE	Association for the Advancement of Cost Engineering
cy	cubic yard(s)
FRFA	flood retention and flow augmentation
FRO	flood retention only
GERCC	grout enriched roller-compacted concrete
M	million
OPC	opinion of probable cost
RCC	roller-compacted concrete
WBS	work breakdown structure

1 INTRODUCTION AND SUMMARY

This report provides an opinion of probable cost (OPC) for the Chehalis Basin Strategy concept-level dam designs. An OPC is presented for two primary project alternatives:

1. A new roller compacted concrete (RCC) dam providing flood retention only (FRO); dam and spillway crest elevations of 651.0 and 628.0 feet, respectively.
2. A new RCC dam providing flood retention and flow augmentation (FRFA); dam and spillway crest elevations of 710.0 and 687.0 feet, respectively.

The cost opinions presented in this document reflect updates to the costs most recently provided in an August 2016 memorandum based on the dam configurations presented in the Conceptual Design Report (HDR, 2016). Costs are offered in 2016 dollars.

The document *Guidelines for Construction Cost Estimating for Dam Engineers and Owners* (USSD, 2012) provides a description of varying cost estimating "levels" for dam projects. Levels provide an indication as to the degree of uncertainty associated with an estimate. Significant effort has been expended on evaluating RCC materials availability, design, and construction considerations. Accordingly, the RCC portion of the dam project has a higher degree of certainty than other portions of the project. The estimate completed for the RCC portion of work is consistent with a "reconnaissance-level" OPC. This type of estimate is generally in compliance with an Association for the Advancement of Cost Engineering (AACE) Class 3 estimate. The non-RCC components (such as clearing and grubbing, excavating, diversion tunnel, earthwork, piping, concrete, utility, and other site civil work) of the estimate are generally consistent with a "feasibility-level" OPC. This type of estimate is generally in compliance with an AACE Class 4 estimate.

A work breakdown structure (WBS) has been developed to guide the estimate. Quantities reflect a combination of applied judgment and take-offs generated from the concept-level drawings. Unit pricing similarly reflects applied judgment, and in a few select areas more developed unit pricing draws from productivity and resource-based estimate experience. The estimate does not reflect full project-specific productivity and resource-based estimates. Recognizing the early stage of design, the estimate offers low and high ranges developed by work item, allowing judgment-level adjustments to quantities and unit pricing to establish line-item ranges. Additionally, a minor allowance is included for unlisted work, and factors are applied to consider design and construction contingencies and noncontract costs to identify the possible range of full project costs. Additional project background and reference information can be found in the Conceptual Combined Dam and Fish Passage Design Report and its Appendix E (HDR, 2017). The costs also include factors for contract contingencies (design and procurement contingencies), construction contingencies (post award change and dispute resolution factor) and non-contract costs (project management, planning, design, permitting and construction management). The costs for integral fish passage facilities are addressed in a separate report and are not included in the total costs presented in this report.

1.1 Alternative Descriptions

The dam cross-section for each alternative is the same, with a 20-foot non-overflow crest width, 0.1H:1V battered upstream slope, and a 0.85H:1V downstream slope, each containing an internal 200-foot ungated ogee spillway crest and associated flip bucket energy dissipation structure. River outlet works and sluicing flood control are routed through similar intake and outlet works structures, discharging into a stilling basin adjacent to the downstream right-bank fish passage facilities. River diversion, a major construction feature and cost component, is conceptually planned through a 20-foot modified-horseshoe-shaped right abutment tunnel. Upstream and downstream cofferdams divert water through the diversion tunnel and provide limited protection for the construction work. The upstream cofferdam currently has a crest elevation of 465 feet, sufficient to pass approximately a 2.8-year return period flow in the river with 3 feet of freeboard.

The primary differences between the FRO and FRFA designs are the dam height, a related right abutment wing dam, and outlet works structures. Near the top of the FRO design, the right abutment hillside crests. A wing dam consisting of a composite earthfill/rockfill embankment curves back upstream from the main dam axis alignment to form the upper-right abutment of the FRFA dam configuration. The right abutment FRFA wing embankment transitions from the RCC after it turns toward the upstream abutment tie-in. Other differences between the alternatives are less significant and are not key factors in the cost estimates. It is important to note, however, that as the design develops, foundation and dam seepage design may vary between the FRFA, with a permanent storage pool, and the FRO, which is not intended to store water. For example, the FRO design may allow lower RCC mix design strengths or reduced or eliminated foundation grouting compared with the FRFA. A few of these differences are recognized in the development of the two estimates’ low and high ranges.

1.2 Cost Opinion Summary

Table 1-1 provides cost opinion and range information for FRO and FRFA alternatives, with a few items to note.

Table 1-1
Summary of OPC for Chehalis FRO and FRFA Dam Configurations

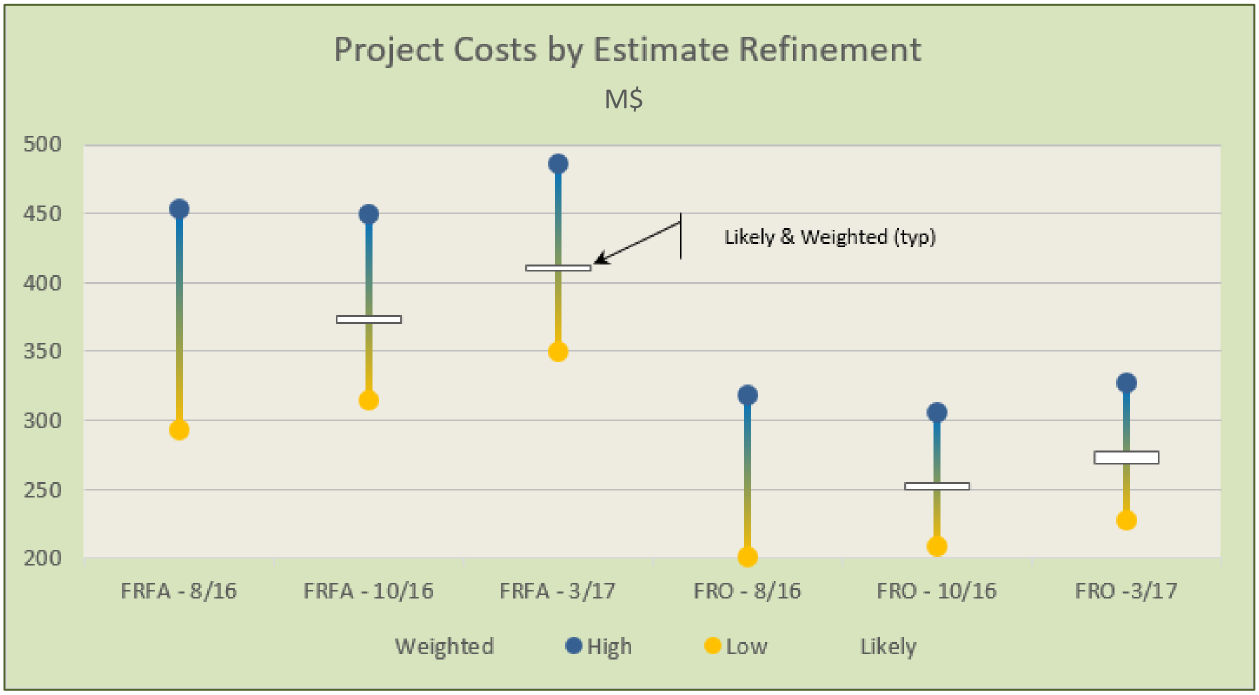
Summary Information	FRO	FRFA
Costs		
Low End Project Cost	222,000,000	342,000,000
Likely Project Cost	271,000,000	404,000,000
High End Project Cost	320,000,000	476,000,000
Project Cost Range from Likely	82% - 118%	85% - 118%
Total Weighted Project Cost	263,000,000	399,000,000
Select Cost Estimate Information		
Driving RCC quantity	870,000 cy	1,475,000 cy
RCC Unit Bid - Likely (FRO GERCC, FRFA U/S conv.)	$ 91.00	$ 96.00
RCC Unit Bid Range	$ 74.00 - $ 107.00	$ 81.00 - $ 111.00
RCC Scope - as % of Contractor Bid	44%	53%
Main Dam (including outlet works) - as % of Bid	79%	81%

First, note that a "likely" estimate is provided along with low- and high-range estimates for each dam configuration. These ranges have been established on a work-breakdown, line-item basis. Second, the scope of the RCC materials, and the total of the costs for the main dam components, are the dominant cost components of each alternative. The main dam percentage of the total cost includes the construction diversion tunnel and coffer dams; foundation excavation and treatments; and construction of the dam, including the facing systems, internal drainage and gallery system, and crest treatments. The main dam percentage does not include the primary hydraulic structures associated with the flood control and water supply outlet works, or the spillway. Because the RCC materials and main dam components of the project are such a significant portion of the total costs for the project, the aggregate and RCC unit prices have been developed from a more detailed breakdown of materials and work tasks that build up to composite unit prices. Third, the river diversion requirement will be very similar if not the same for each alternative. Hence, this component is driving the percentage of the main dam total costs higher for the smaller project.

1.3 Cost Estimate Refinement

As design development and site characterization have progressed, tracking of the project costs and cost changes have been maintained. A cost estimate range for each option was most recently developed and first communicated in an August 2016 memo. Subsequently, in October, the August estimate ranges were refined and updated to include an estimate judged by experience to represent the "likely" costs to provide better understanding of the cost ranges presented in that document. The October estimate was communicated as late as December 2016, recognizing that further refinements and better project cost definition was in progress to best capture the finalized conceptual design. Those refinements have recently been completed and are offered in this document and the corresponding final Conceptual Design Report. Figure 1-1 shows the evolution of the OPC estimates communicated over the course of the last several months.

Figure 1-1
Summary of Estimate Refinement



A detailed table of the cost estimate changes is provided in Attachment B. The primary design configuration and estimate changes that have resulted in the increased range and "likely" estimates include: 1) increasing excavation and corresponding RCC quantities following completion of our excavation definition; and 2) a more detailed work breakdown structure; and 3) a more detailed pricing of RCC components. The likely & weighted cost value, first established for the 10/16 estimate, has increased in the current estimate; however these cost increases have been offset by a corresponding reduction in the design contingency from 20 percent to 12.5 percent to better reflect the pricing approach and confidence associated with the RCC and associated aggregate and materials production. The likely & weighted cost value for the FRFA and FRO alternatives still falls within the original range of uncertainty provided in the 8/16 estimates.

2 COST ESTIMATING APPROACH

2.1 Line-Item Cost Development

Cost estimates were developed for each alternative as they progressed through concept-level design. Attachment A contains the estimate summaries for the FRO and FRFA options, with some annotation describing recent evolution of the estimates. The Attachment A tables include a work breakdown containing about 40 tasks within broader categories including; mobilization, diversion and dewatering, lands and easements, and the main dam. Depending on the line item, unit and lump sum costs have generally been applied by experience and judgment, with consideration of the Chehalis project specifics, emphasizing the RCC and related unit cost development. Although not incorporated into resource- or productivity-level estimates, area wage rates were considered, as was typical seasonal weather information. Other qualitative factors included work quantities, economies of scale, potential sequencing, and construction method impacts.

2.2 Range Development

Included in Attachment A are some notes and considerations that inform how the quantities or prices have been established. Also, for each line item, low- and high-range cost estimates have been developed. Working from a default low and high of 80 percent and 120 percent of "likely," respectively, some items' ranges were modified when appropriate to reflect a basis for a different range. The line-item low and high costs are added to form total low and high project cost boundaries and a range of expected project costs. The expected accuracy range for an AACE Class 4 estimate is described as -15 percent to -30 percent on the low side and +20 percent to +50 percent on the high side. For a Class 3 estimate, the low and high expected accuracy range is -10 percent to -20 percent and +10 percent to +30 percent, respectively. For comparison, the FRO and FRFA low estimates are 18 percent and 15 percent below the "likely" estimates, respectively. Similarly, the high estimate for each is about 18 percent above the "likely" estimates.

2.3 Contingencies and other Factors

Along with line-item pricing, Attachment A also shows each estimate's total "bid" price followed by a few "below-the-line" factors, which add up to a total estimated project cost. Inclusion of the factors described below is necessary for identification of realistic total project costs.

- **Design and procurement contingencies**: This contingency represents costs that arise as the design changes and evolves, adding scope to the work being priced. An example might be an early estimate that purposefully excludes foundation consolidation grouting in the early stage that is deemed necessary during later planning stages. Estimates in this document assume 12.5

percent of the base construction costs for this contingency. This contingency has decreased as design understanding and line-item pricing have been refined.

- **Construction contingency**: This contingency considers the costs a project is likely to encounter post-award as a result of necessary changes to work quantities, changes in scope, change orders, low-grade dispute resolution, etc. as a result of differing site conditions or changes in risk management and mitigation strategies. This report assumes a fairly standard 10 percent for this factor. As design develops and procurement strategies are considered, it is important to recognize that key cost and risk drivers for both alternatives include diversion and significant foundation construction. Both demand a strong design and planning effort to best manage this contingency. Typically, this contingency could either increase or, more likely, decrease depending on an evolving understanding of anticipated post-award site risks and conditions.
- **Noncontract costs**: Noncontract costs typically include costs that are outside of the construction contract(s); for example, permitting, site characterization, environmental mitigation, studies, preparation of engineering design documents, legal requirements, construction management during construction, etc. The subtotal costs prior to this factor being applied represent the costs of construction – first the bid and then the final quantities, changes (change orders) during construction, and low-grade dispute management costs. The “noncontract” costs should not be considered a contingency but the real costs of preparing for and monitoring construction. Each estimate contains a 25 percent factor for noncontract costs.

2.4 Diversion and Dewatering

Diversion and dewatering requirements will be very similar for the FRO and FRFA alternatives. Consequently, approximately $15 million (M) dollars, within a range of $13M-$19M, has been apportioned to the diversion and dewatering line items. The lower-right abutment diversion tunnel is the dominant feature, but also included are the routing and isolating cofferdams, foundation dewatering, conversion of the diversion tunnel after construction, and, finally, a diversion risk contingency related to low-impact risk that contractors might be inclined to include in their estimates. Ultimately, the diversion costs will depend on the degree of contractor design and responsibility established in the contract and contractors’ risk perception and tolerance. Unlikely large or low-frequency flood potential often is not contemplated in diversion design, and those direct and delay cost risks should be considered outside the scope of the diversion estimate, and, therefore, wholly or partially addressed by the construction contingency.

2.5 Lands and Easements

Perhaps overlapping with the below-the-line noncontract costs, costs have been included for property and flood easements, and basin road abandonment or relocation. As design moves forward, these costs should be better defined and separated into dam construction contract and noncontract costs.

2.6 Foundation Preparation

Beyond common and rock excavation to expose fresh and acceptable foundation rock and to accommodate the spillway and outlet works structures, the cost estimate includes dental concrete and a foundation grout curtain. The work quantities and prices established for excavation and dental concrete and the corresponding contingencies don't directly address unanticipated foundation anomaly treatment. Additionally, there is uncertainty associated with the limited number of borings on the right abutment due to steep terrain, as well as uncertainty associated with interpolation between bore holes. Further estimate refinement might consider a specific cost allocation for special foundation treatments; for example, dental excavation; shaping block construction; and shear zone treatments requiring dental excavation, dental concrete placement, and surface slush grouting. For the FRFA, a double grout curtain with secondary and tertiary holes has been anticipated and, conservatively, includes a foundation consolidation or blanket grouting program. For the FRO, which is not intended for storage purposes, a single grout curtain with secondary and tertiary holes without a foundation consolidation program has been included in the cost estimate.

2.7 Aggregate and RCC

Attachment C contains the quantity and cost development for the RCC aggregate and the corresponding total RCC unit cost development. Further information on the material sourcing can be found in Appendix E of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017). Aggregate unit costs and cost ranges were developed for each of three potential basalt quarries on or near the dam site, along with pricing from two commercial basalt quarries. A moderate to conservative look at the site-based pricing supports the costs included in the composite RCC unit costs. The composite RCC costs include the aggregate, cement, fly ash, RCC plant and delivery setup/ commissioning, RCC test section, mixing, delivering, placing, mix cooling, face systems, gallery, and drainage features. Many design choices are represented in those components – and therefore the final unit pricing remains conceptual until a more specific design emerges during the next (preliminary) phase of design. For example, the FRO unit cost has been developed considering grout-enriched RCC (GERCC) for both the upstream (seepage control) and downstream dam faces. A more conservative conventional concrete upstream face is currently included in the FRFA storage option. For comparison purposes, when adjusted to 2016 dollars using the U.S. Bureau of Reclamation's Construction Cost Trends, the Olivenhain Dam's RCC composite unit cost for about 1.4M cubic yards (cy) of RCC with GERCC faces was about $87/cy ($/cubic yard). Similarly, for the fairly recent 0.6M cy RCC San Vicente Dam buttress and dam raise, the equivalent 2016 RCC bid costs were approximately $82/cy. The FRO and FRFA unit prices are $91/cy within a range of $74-$107/cy, and $96/cy within a range of $81-$111/cy, respectively. A summary of the RCC placement analysis is presented in Attachment D.

2.8 Hydraulic Structures

Preliminary concepts include a combined low-level sluicing and integrated nominal river outlet works with stilling basin; a multilevel intake tower and trash rack; and an ungated (uncontrolled) crest spillway with flip bucket to river bed plunge pool spillway. Rough estimates of concrete and steel and mechanical components have been priced pending further design development of the various structures. Approximately 30,000 cy and 45,000 cy have been included for the reinforced and nonreinforced concretes for the FRO and FRFA structures, respectively. A related $23M and $21M have been included for the steel, gates, and valves for the FRO and FRFA, respectively. As design moves forward, it will be important to define the relationships and space shared between the dam base, spillway plunge pool, diversion discharge, outlet works discharge, and fish passage integration.

3 COST ESTIMATE CONSIDERATIONS

3.1 Schedule

The FRO and FRFA configurations will need 3- to 4-year and 4- to 5-year construction windows, respectively, for completion. Construction durations will depend to a degree on contract packaging. For example, the option to package a phase 1 diversion and site-work contract prior to a phase 2 main dam contract will result in a specific construction duration window for each alternative dam type. This scenario would make tunneling the primary construction discipline in phase 1 and the dam the predominant feature in phase 2. Advantages for this type of approach might include the following: Tunnel specialty contractors might be attracted as the primes for phase 1 and RCC contractors for phase 2; an early site work contract could facilitate a large-scale quarry evaluation and some initial crushing for access development, potentially lessening risk related to site-developed RCC aggregate; and expensive RCC plant and delivery and related standby costs associated with a tunneling delay could be avoided. Disadvantages of a two-phased approach might include a missed opportunity for contractor innovation related to diversion; an increase in the owner's risk burden for successful diversion performance; and a missed opportunity for schedule compression that might be possible under a single contract. The construction schedule is discussed in greater detail in the Conceptual Combined Dam and Fish Design Report.

Construction sequence and duration contribute to construction costs in a few notable, and sometimes indirect, ways. First, the construction infrastructure – or project indirect or overhead expense – is time-dependent, and the longer it takes to complete construction, the higher the cost for these expenses that are not directly related to work quantities. For example, project management staff, water supply, office establishment, and underallocated equipment are costs that are independent of the work scope but are a function of duration. Second, longer projects often demand longer capital and equipment resource commitment and therefore demand greater financial stability. Sureties, for example, look less favorably on longer duration work when agreeing to commit bond credit. Third, contractors may be more risk-tolerant if they have the opportunity to manage that risk over a longer construction period of time. Additional factors such as whether liquidated damages are included, schedule flexibility to absorb unexpected occurrences and balance of risk in the contract conditions contribute to the competitive outlook of a project, which can affect overall pricing as well.

3.2 Construction Considerations

Careful consideration of project unit costs and quantities are important for judgment-based, early cost opinions. In addition, early consideration of constructability, construction phasing, and construction risk are important to understanding cost drivers and potential variability in pricing. The site characterization and material testing that has been completed to date is extensive considering the early stage of project development. This characterization and material testing work has increased our understanding of the foundation and material properties for construction thereby reducing some of the most significant project cost uncertainties. As design progresses, targeted effort in the following areas will improve cost confidence (reducing the uncertainty bands) as well as provide important input for ongoing design development:

- Define project cost, schedule, and risk drivers, to include: 1) river hydrology and diversion; 2) foundation excavation and treatment approaches and uncertainties; 3) effective utilization of site-based RCC aggregates; and 4) effective and integrated layout of downstream structures – plunge pool, fish passage, river outlet works stilling basin, access, and diversion tunnel discharge portal.
- Develop basin hydrology and risk management requirements for construction. Specifically, the designs will need to consider nominal stream flows as well as higher-frequency flood flows that will occur on a seasonal basis. This permits an understanding of the timing and probable durations of low-flow periods necessary for key diversion activities such as tunnel portal development, cofferdam construction, and diversion phase transitions.
- Develop a detailed construction phasing framework emphasizing diversion construction, early low-level hydraulic structure construction, and foundation construction. Determine schedule sensitivities to sequence options, diversion approach, weather and hydrology, and other potential – including environmental – constraints.
- Make preliminary design selections and develop early details for the hydraulic structures.
- Refine concepts for site use including contactor access and staging.
- Perform additional site characterization and laboratory testing work to include an early mix design development program and the parameters for site-based RCC aggregate development.
- Critically evaluate diversion alternatives including impacts to sequencing and contractual design responsibility and risk allocation.
- Evaluate potential project delivery methods, including traditional as well as alternative, and specifically value-based procurement and contracting formats. An early understanding of options, along with advantages and disadvantages, gives great insight to risk management strategies.
- Evaluate RCC and dam component features and design options, such as thermal and joint design, facing systems, and drainage systems.

4 CONCLUSIONS

Concept-level opinion of probable costs calculates a likely $271M with a range of $222M to $320M for the FRO alternative dam configuration, and a likely $404M with a range of $342M to $476M for the FRFA dam configuration. These OPCs reflect judgment-level quantity and unit prices, with aggregate and RCC unit prices based on a more thorough work breakdown and detailed component pricing. Fish passage costs are not included in the dam cost estimates presented in this report. Cost drivers for the project are closely tied to schedule and project risks, including river hydrology, diversion design, and risk exposure; RCC dam component design option selections (e.g. facing systems); integration of the spillway, sluiceway, river outlet works, and downstream fish passage structures; and construction work packaging and sequence.

The opinion of probable project cost is based on information available at the time of the writing of this report, and on the basis of the engineer’s experience and qualifications, and represents their judgment as an experienced and qualified professional engineer. However, since the engineer has no control over the cost of labor, materials, equipment or services furnished by others, or over the contractor(s') methods of determining prices, or over competitive bidding or market conditions, the engineer does not guarantee that proposals, bids or actual project or construction cost will not vary from opinions of probable cost the engineer prepares.

5 REFERENCES

HDR, 2017. Conceptual Combined Dam and Fish Passage Design Report. Bellevue, Washington.

USSD (United States Society on Dams), 2012. *Guidelines for Construction Cost Estimating for Dam Engineers and Owners*

Attachment A
Judgment-level Cost Opinion Worksheets for the FRO and FRFA Estimates

HDR

Judgment-Level Cost Opinion
Pricing/Work Breakdown Summary

Project: ***Chehalis Dam***
Alternative: ***FRO*** ***Pre-Dwg; 0.1 : 0.85:1:***

Pricing - contractor cost basis 1 or bid basis 2: 2

Range Driver - 1 = %, 2 = Q & $, 3 = Combination:	3		
Default Low ⇨	80%	Default High ⇨	120%

Weighting ⇨	20% low	75% likely	5% high	20% low	75% likely	5% high
$201M - Jul-16	Low End	$221,599,130		Low End	$267,026,341	
	Likely	$270,651,653		Likely	$326,134,495	
$319M - Jul-18	High End	$320,357,759		High End	$386,030,216	
	Weighted	$263,326,454		Weighted	$317,307,651	

Base or Likely Cost Case

Work Item	Description	Quantity	Unit	Unit Price [1]	Total $	Estimate Notes & Considerations
	Phase 1 - Site Development, Diversion Constrruction,				$0	
0	**Mobilization**					
	Mobilization	1	LS	$3,500,000.00	$3,500,000	Contractor mob bid; balance of project overhead in below-the-line factors
1	**Clearing & Grubbing**				$0	
1.01	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	25	Acre	$30,000.00	$750,000	
1.02	Reservoir Clearing to 100-yr Flood Stage	756	Acre	$6,000.00	$4,536,000	Potentially in Phase 2 or possibly Phase 3 contract
2	**Temporary Access & Staging**				$0	
2.01	Construction Surveying & Layout	25	Acre	$10,000.00	$250,000	Under temporary access & staging; i.e. temporary works only, predominant surveys and layout in unallocated contractor project overhead expense (already in the unit pricing)
2.02	Pioneer/Access Roads (e.g. dam site, abutments, quarry site, etc.)	2.0	Mile	$700,000.00	$1,400,000	**Changes for final: increase access road development by adding 0.5 mile from 1.5 to 2 miles.** dependent upon aggregate sourcing, staging locations, contractor approach. Reference G-4_ETZ_21Sept2016 - JCA markups 01.pdf for site, non-quarry access concepts, totaling about 13,500lf. Say 50% new and full access development, 20% construction & track access only, 30% improved existing. Consider quarry acces costs in aggregate price range
2.03	Material Laydown Area Prep (minor excavation, grading, surfacing, drainage	18	Acre	$25,000.00	$450,000	1 acre at 5' avg cut to 5' average fill = 4000cy cut to fill; @ $6/cy cut to fill = $24,200/ac; 1ac surfacing at 6" & 30% surfaced = 430ton, @ 10/tn = $4.5k/ac
2.04	Temporary construction site access security control facilities (e.g. fencing, gates, etc.)	2,000	LF	$20.00	$40,000	predominant security expense in unallocated contractor project overhead expense
3	**Diversion & Dewatering**				$0	
3.01	Diversion Tunnel 20 ft modified horseshoe	1,500	LF	$8,000.00	$12,000,000	**Changes for final: increase length of tunnel to better reflect final drawing alignment.** increase high end for variability in linnig limits, portaling, tunnel plug adit construction, vent construction, etc.
3.02	Conventional Concrete Non-Reinforced Mass Concrete (100' plug following construction)	1,200	CY	$600.00	$720,000	low end 30'plug but include mechanical.
3.03	Coffer Dams (2) - Fill cells u/s and d/s + toe slopes	14,000	CY	$40.00	$560,000	check Q's with new crest heights, say 8,000 cy RCC @ 70 + 6,000 cy Rockfill @ 15. = 650KHigh end if pushed to 480 and rockfill - say 45kcy = $675K.
3.04	Foundation Excavation - seepage key (assume 20'wide x 150' long x 4' deep	450	CY	$8.00	$3,600	Cofferdam key allowance
3.05	Foundation Dewatering - assume several dewatering pump systems operating selectively 24/7 over 6 month foundation construction period	270	Day	$2,800.00	$756,000	**Changes for final: increase foundation exposure from 6 to 9 months.** 2nd contract may add unwaterring and time for dewatering for RCC foundation
3.06	Coffer Dams - Other assume 25' high x 150 top length, 35' base length, cell construction (e.g. sheet pile, steel, other fabricated metal items)	7,000	SF	$30	$210,000	may include isolation of portal structures, tailwater structures, peripheral dewatering stages
3.07	Coffer Dams - Risk contingency for overtopping	1	LS	$750,000.00	$750,000	contemplates partial or threshold-bound contractor responsibility, risk apportioned cost of event recovery, rework, delay
4	**Lands and Easements**				$0	
4.01	Reservoir Extents Fee Title	750	Acre	$4,400	$3,300,000	Best to be considered in non-contract costs. Perhaps cost conservatively overlaps with non-contract cost factor below.
4.02	Reservoir Extents/Flood Easement	55	Acre	$4,400	$242,000	Best to be considered in non-contract costs. Perhaps cost conservatively overlaps with non-contract cost factor below.
4.03	Reservoir orphaned access roadway reconnection allowance (to WeyCo?)	4.5	Mile	$1,000,000	$4,500,000	Unit price potentially higher for permanent versus constuction roads. Line item also perhaps better considered under non-contract cost factor.
	Phase 2 - Main Dam					
5	**Main Dam Structure**				$0	
5.01	Excavation - Foundation General	460,000	CY	$6.50	$2,990,000	
5.02	Excavation - Foundation Rock	110,000	CY	$25.00	$2,750,000	Some rock will be structural exc in fresh rock, most will be foundation footprint, getting to good rock below the rock contact; i.e potentially a high degree ripable.
5.03	Roller Compacted Concrete - Composite Scope	870,000	CY	$91.00	$79,170,000	**Changes for final: revised quantity to reflect QTO after CDR drawings, adjusted unit prices to reflect only GERCC; expanded RCC unit cost development composite workbreakdown, and revisited RCC unit pricing.** RCC unit pricing includes aggregate, cemen-fly ash, lift bedding, abutment bedding, dam joints, and full GERCC for both upstream and downstream faces. Conventional concrete spillway face included elsewhere.
5.04	Fill - Foundation Backfill	315,000	CY	$5.50	$1,732,500	**Changes for final: none.**
5.05	Conventional Concrete Reinforced (miscellaneous)	500	CY	$850.00	$425,000	Refine quantities along with all strucures next phase.
5.06	Conventional Concrete Non-Reinforced Mass Concrete (bedding, abutment contact, cover over sluice conduits, assume nominal contact layer)	12,000	CY	$400.00	$4,800,000	Refine quantities along with all strucures next phase.
5.07	Concrete - Dam and Crest Spillway	6,500	CY	$750.00	$4,875,000	**Changes for final: None. Consider this item only as upper spillway. Use a lower low end considering potential for less spillway quantity for FRO.** No facing should be included if flip bucket chute face is elsewhere. Leave in for ogee, spillway approach walls, piers.
5.08	Foundation Treatment - Grout Curtain Drilling	23,000	LF	$45.00	$1,035,000	**Changes for final: adjust quantity from 22,500 to 23,000lf, and slight increase to cement for grouting.** revisited pricing; 1200lf @ 10', plus 50% secondary, plus 25% tertiary @ 70' deep = 14,700lf; plus say 170,000 sf @ 400sf/ hole @ 20' deep = 8,500lf = 23,200 lf;
5.09	Foundation Treatment - Grout Curtain Cement	16,000	Sack	$40.00	$640,000	**Changes for final: increase sacks to 0.7 sack per lf. Lower range considered for both drilling and cement for grouting operations based on limited exposure of structure under stored water service conditions.**
5.1	Flood Regulating Conduit Control Structures - Reinforced Concrete	5,000	CY	$800.00	$4,000,000	Assume 2' thick around perimeter of sluices & air shafts. Refine quantities along with all strucures next phase; include inside and downstream of dam; include control building on crest or at downstream, depending on final concept drawings
5.11	Flood Regulating Conduit Control Gates - Fab and Construct	200,000	LB	$15.00	$3,000,000	Assume 2 @ 30 tons, 1 @ 40 tons
5.12	Bulkhead gates	570,000	LB	$15.00	$8,550,000	Assume 2 @ 25 tons, 1 @ 35 tons and 4 @ 50 tons
5.13	Hoists, cylinders, machinery	300,000	LB	$15.00	$4,500,000	
5.14	Reservoir drain valve in tunnel plug (assume 4x4' knife valve)	1	Each	$200,000.00	$200,000	
5.15	Unused	0	Each	$0.00	$0	
5.16	Unused	0	Each	$0.00	$0	
5.17	Unused	0	CY	$0.00	$0	

Range Development

Work Item	Driven by Percent: Low End % (def=80%)	High End % (def=120%)	Low % Total $	High % Total $	Driven by Q & Unit $: Low End Q	Low End Unit $	Low End Total $	High End Q	High End Unit $	High End Total $	Driven by Combo: Low End Tot $	High End Tot $
Phase 1			$0	$0			info?			info?	$0	$0
0			$0	$0			info?			info?	$0	$0
Mobilization	100%	140%	$3,500,000	$4,900,000			info?			info?	$3,500,000	$4,900,000
1			$0	$0			info?			info?	$0	$0
1.01			$600,000	$900,000	18	$30,000.00	$540,000	25	$25,000.00	$625,000	$540,000	$625,000
1.02			$3,628,800	$5,443,200		$5,000.00	$3,780,000		$7,500.00	$5,670,000	$3,780,000	$5,670,000
2			$0	$0			info?			info?	$0	$0
2.01	100%	150%	$250,000	$375,000			info?			info?	$250,000	$375,000
2.02			$1,120,000	$1,680,000	1.5	$750,000.00	$1,125,000	2.25	$800,000.00	$1,800,000	$1,125,000	$1,800,000
2.03			$360,000	$540,000	13	$30,000.00	$390,000	22		$550,000	$390,000	$550,000
2.04			$32,000	$48,000			info?			info?	$32,000	$48,000
3			$0	$0			info?			info?	$0	$0
3.01	90%	125%	$10,800,000	$15,000,000			info?			info?	$10,800,000	$15,000,000
3.02			$576,000	$864,000		$450.00	$540,000		$650.00	$780,000	$540,000	$780,000
3.03			$448,000	$672,000			info?			info?	$448,000	$672,000
3.04		300%	$2,880	$10,800			info?			info?	$2,880	$10,800
3.05		150%	$604,800	$1,134,000			info?			info?	$604,800	$1,134,000
3.06			$168,000	$252,000			info?			info?	$168,000	$252,000
3.07			$600,000	$900,000			info?			info?	$600,000	$900,000
4			$0	$0			info?			info?	$0	$0
4.01	100%	100%	$3,300,000	$3,300,000			info?			info?	$3,300,000	$3,300,000
4.02	100%	100%	$242,000	$242,000			info?			info?	$242,000	$242,000
4.03		110%	$3,600,000	$4,950,000			info?			info?	$3,600,000	$4,950,000
Phase 2											$0	$0
5			$0	$0			info?			info?	$0	$0
5.01		110%	$2,392,000	$3,289,000		$5.50	$2,530,000		$7.50	$3,450,000	$2,530,000	$3,450,000
5.02		110%	$2,200,000	$3,025,000		$25.00	$2,750,000		$30.00	$3,300,000	$2,750,000	$3,300,000
5.03			$63,336,000	$95,004,000		$74.00	$64,380,000		$107.00	$93,090,000	$64,380,000	$93,090,000
5.04			$1,386,000	$2,079,000			info?			info?	$1,386,000	$2,079,000
5.05			$340,000	$510,000			info?			info?	$340,000	$510,000
5.06			$3,840,000	$5,760,000			info?			info?	$3,840,000	$5,760,000
5.07	60%	110%	$2,925,000	$5,362,500			info?			info?	$2,925,000	$5,362,500
5.08	70%	110%	$724,500	$1,138,500			info?			info?	$724,500	$1,138,500
5.09	70%	110%	$448,000	$704,000			info?			info?	$448,000	$704,000
5.1			$3,200,000	$4,800,000			info?			info?	$3,200,000	$4,800,000
5.11			$2,400,000	$3,600,000			info?			info?	$2,400,000	$3,600,000
5.12			$6,840,000	$10,260,000			info?			info?	$6,840,000	$10,260,000
5.13			$3,600,000	$5,400,000			info?			info?	$3,600,000	$5,400,000
5.14	50%	100%	$100,000	$200,000			info?			info?	$100,000	$200,000
5.15			$0	$0			info?			info?	$0	$0
5.16			$0	$0			info?			info?	$0	$0
5.17			$0	$0			info?			info?	$0	$0

Judgment-Level Cost Opinion
Pricing/Work Breakdown Summary

Project: ***Chehalis Dam***
Alternative: ***FRO*** ***Pre-Dwg; 0.1 : 0.85:1:***

Pricing - contractor cost basis 1 or bid basis 2: **2**

Range Driver - 1 = %, 2 = Q & $, 3 = Combination:	**3**		
Default Low ⇨	**80%**	**Default High ⇨**	**120%**

Weighting ⇨	20% low	75% likely	5% high	20% low	75% likely	5% high
$201M - Jul-16	**Low End**	**$221,599,130**		**Low End**	**$267,026,341**	
	Likely	**$270,651,653**		**Likely**	**$326,134,495**	
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	Weighted	**$263,326,454**		**Weighted**	**$317,307,651**	

		Base or Likely Cost Case					Range Development: Driven by Percent				Range Development: Driven by Q & Unit $						Range Development: Driven by Combo	
Work Item	**Description**	**Quantity**	**Unit**	**Unit Price** [1]	**Total $**	**Estimate Notes & Considerations**	**Low End % (def=80%)**	**High End % (def=120%)**	**Low % Total $**	**High % Total $**	**Low End Q**	**Low End Unit $**	**Low End Total $**	**High End Q**	**High End Unit $**	**High End Total $**	**Low End Tot $**	**High End Tot $**
5.18	Unused	0	CY	$0.00	$0				$0	$0			info?			info?	$0	$0
5.19	Trashrack steel framing	1,134,000	LB	$6.50	$7,371,000	Assumes 250 ft high, 10 members 3' dia x 4.5'deep, steel columns.			$5,896,800	$8,845,200			info?			info?	$5,896,800	$8,845,200
5.2	Trashrack - concete side wall and decking	3,200	CY	$850.00	$2,720,000				$2,176,000	$3,264,000			info?			info?	$2,176,000	$3,264,000
6	**Spillway Flip Bucket**				$0				$0	$0			info?			info?	$0	$0
6.01	Conventional Concrete Reinforced (assume 3' surface)	6,350	CY	$700.00	$4,445,000				$3,556,000	$5,334,000			info?			info?	$3,556,000	$5,334,000
6.02	Non-Conventional Concrete Non-Reinforced (could be RCC grout enriched, assume 30ft wedge under reinforced concrete surface)	9,750	CY	$225.00	$2,193,750	Unit price - accomodates higher RCC placement and utilization of some mass conventional concrete. Lower range due to strong potential for this volume to be less for FRO	75%	100%	$1,645,313	$2,193,750			info?			info?	$1,645,313	$2,193,750
7	**Sluice Stilling Basin**				$0				$0	$0			info?			info?	$0	$0
7.01	Excavation - Foundation General	20,000	CY	$8.00	$160,000				$128,000	$192,000			info?			info?	$128,000	$192,000
7.02	Excavation - Foundation Rock	10,000	CY	$30.00	$300,000				$240,000	$360,000			info?			info?	$240,000	$360,000
7.03	Fill - Foundation Backfill	18,000	CY	$9.00	$162,000				$129,600	$194,400			info?			info?	$129,600	$194,400
7.04	Conventional Concrete Reinforced (assume 3' thick surface)	4,900	CY	$800.00	$3,920,000				$3,136,000	$4,704,000			info?			info?	$3,136,000	$4,704,000
7.05	Conventional Concrete Non-Reinforced (assume 4.5'thick bedding under floor only)	2,000	CY	$400.00	$800,000	Refine quantities after drawings are complete; must schedule after dam is up; include control building on crest;			$640,000	$960,000			info?			info?	$640,000	$960,000
8	**Wing Dam Structure**				$0				$0	$0			info?			info?	$0	$0
8.01	Unused	0	ls	$0.00	$0				$0	$0			info?			info?	$0	$0
8.02	Unused	0	ls	$0.00	$0				$0	$0			info?			info?	$0	$0
8.03	Unused	0	ls	$0.00	$0				$0	$0			info?			info?	$0	$0
8.04	Unused	0	ls	$0.00	$0				$0	$0			info?			info?	$0	$0
	Composite & Unlisted Work																	
55	Fish passage structure - costs not included	1	ls	$0	$0	Costs independently assessed in report			$0	$0			info?			info?	$0	$0
56	Unlisted Work	1	ls	$3,500,000	$3,500,000		85%	115%	$2,975,000	$4,025,000			info?			info?	$2,975,000	$4,025,000
57					$0				$0	$0			info?			info?	$0	$0
58					$0				$0	$0			info?			info?	$0	$0
59					$0				$0	$0			info?			info?	$0	$0
60					$0				$0	$0			info?			info?	$0	$0
61					$0				$0	$0			info?			info?	$0	$0
62					$0				$0	$0			info?			info?	$0	$0
63					$0				$0	$0			info?			info?	$0	$0
64					$0				$0	$0			info?			info?	$0	$0
65					$0				$0	$0			info?			info?	$0	$0
					$0				$0	$0			info?			info?	$0	$0
Subtotal without mobilization & general expense					**$178,206,850**				**$144,086,693**	**$212,415,350**			**$76,035,000**			**$109,265,000**	**$145,908,893**	**$210,935,150**

Mobilization & project indirect expense		0%		$0	unallocated project indirect or jobsite overhead assumed in unit pricing
Contractor Cost				**$178,206,850**	Note 1: Unit prices as noted in header, either reflect a bid price basis (no factor application of corporate OH & profit), or a contractor cost basis requiring a corporate OH & profit to get to a bid total
Contractor Margin - corporate overhead & profit		~~0%~~	Bid Basis	$0	Note 2: NA - not applicable to project; NE - not evident in estimate; NI - noted but not itemized in estimate
Contractor Bid - before design/procurement contingencies				**$178,206,850**	
Contract Contingencies - design and procurement contingencies		12.5%		$22,275,856	⇦ RCC estimate dominance, work breakdown thoroughness, and work understanding support a design contingency lower than typical (i.e. 20%) at this early design level
Contract Cost - contractor bid with design & procurement contingencies				**$200,482,706**	
Construction Contingency: post-award change & dispute factor		10%		$20,048,271	
Non-Contract Costs: PM, planning, design, CM ...		25%		$50,120,677	⇦ permitting, site characterization, CM during construction,etc.
Total Project Cost - before escalation				**$270,651,653**	**Compares to $201M low bound, and $319M high bound July 2016**
Escalation - annual %; from; to	3.5%	1-Oct-16	1-Mar-22	$55,482,842	⇦ Presume NTP - early 2019, say 6 years construction = 2.5 + 3 years = 5.5 years
Total Project Cost - including escalation			**5.4 yr**	**$326,134,495**	**<< 183% above total w/o mobilization**

Judgment-Level Cost Opinion
Pricing/Work Breakdown Summary

Project: ***Chehalis Dam***
Alternative: ***FRFA*** ***Pre-Dwg; 0.1 : 0.85:1:***

Pricing - contractor cost basis 1 or bid basis 2: 2

Range Driver - 1 = %, 2 = Q & $, 3 = Combination: 3
Default Low ⇨ 80% **Default High ⇨** 120%

Weighting ⇨	**20% low**	**70% likely**	**10% high**	**20% low**	**70% likely**	**10% high**
$293M - Jul-16	**Low End**	**$342,440,309**		**Low End**	**$424,700,495**	
	Likely	**$404,440,113**		**Likely**	**$501,593,742**	
$454M - Jul-18	**High End**	**$475,512,272**		**High End**	**$589,738,684**	
	Weighted	**$399,147,368**		**Weighted**	**$495,029,587**	

		Base or Likely Cost Case					**Range Development: Driven by Percent**				**Driven by Q & Unit $**						**Driven by Combo**	
Work Item	**Description**	**Quantity**	**Unit**	**Unit Price [1]**	**Total $**	**Estimate Notes & Considerations**	**Low End % (def=80%)**	**High End % (def=120%)**	**Low % Total $**	**High % Total $**	**Low End Q**	**Low End Unit $**	**Low End Total $**	**High End Q**	**High End Unit $**	**High End Total $**	**Low End Tot $**	**High End Tot $**
	Phase 1 - Site Development, Diversion Constrruction,				$0				$0	$0			info?			info?	$0	$0
0	**Mobilization**								$0	$0			info?			info?	$0	$0
	Mobilization	1	LS	$5,000,000.00	$5,000,000	Contractor mob bid; balance of project overhead in below-the-line factors	100%	140%	$5,000,000	$7,000,000			info?			info?	$5,000,000	$7,000,000
1	**Clearing & Grubbing**				$0				$0	$0			info?			info?	$0	$0
1.01	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	30	Acre	$30,000.00	$900,000				$720,000	$1,080,000	25	$30,000.00	$750,000	35	$25,000.00	$875,000	$750,000	$875,000
1.02	Reservoir Clearing to 100-yr Flood Stage	1,206	Acre	$6,000.00	$7,236,000	Potentially in Phase 2 or possibly Phase 3 contract			$5,788,800	$8,683,200		$5,000.00	$6,030,000		$7,500.00	$9,045,000	$6,030,000	$9,045,000
2	**Temporary Access & Staging**				$0				$0	$0			info?			info?	$0	$0
2.01	Construction Surveying & Layout	35	Acre	$10,000.00	$350,000	Under temporary access & staging; i.e. temporary works only, predominant surveys and layout in unallocated contractor project overhead expense (already in the unit pricing)	100%	150%	$350,000	$525,000			info?			info?	$350,000	$525,000
2.02	Pioneer/Access Roads (e.g. dam site, abutments, quarry site, etc.)	3	Mile	$700,000.00	$2,100,000	**Changes for final: increase access road development by adding 1 mile, from 2 to 3.** dependent upon aggregate sourcing, staging locations, contractor approach. Reference Chehalis_All_Figs_2016-10-19.pdf drawing G-3, for site, non-quarry access concepts, totaling about 10,000lf of new access, say 5000lf of upgraded access. Say 50% new and full access development, 20% construction & track access only, 30% improved existing. Consider quarry acces costs in aggregate price range			$1,680,000	$2,520,000	2.5	$750,000.00	$1,875,000	3.5	$800,000.00	$2,800,000	$1,875,000	$2,800,000
2.03	Material Laydown Area Prep (minor excavation, grading, surfacing, drainage	20	Acre	$25,000.00	$500,000	1 acre at 5' avg cut to 5' average fill = 4000cy cut to fill; @ $6/cy cut to fill = $24,200/ac; 1ac surfacing at 6" & 30% surfaced = 430ton, @ 10/tn = $4.5k/ac			$400,000	$600,000	15	$30,000.00	$450,000	25		$625,000	$450,000	$625,000
2.04	Temporary construction site access security control facilities (e.g. fencing, gates, etc.)	2,200	LF	$20.00	$44,000	predominant security expense in unallocated contractor project overhead expense			$35,200	$52,800			info?			info?	$35,200	$52,800
3	**Diversion & Dewatering**				$0				$0	$0			info?			info?	$0	$0
3.01	Diversion Tunnel 20 ft modified horseshoe	1,500	LF	$8,000.00	$12,000,000	**Changes for final: increase length of tunnel to better reflect final drawing alignment.** increase high end for variability in linnig limits, portaling, tunnel plug adit construction, vent construction, etc.	90%	125%	$10,800,000	$15,000,000			info?			info?	$10,800,000	$15,000,000
3.02	Conventional Concrete Non-Reinforced Mass Concrete (100' plug following construction)	1,200	CY	$600.00	$720,000	low end 30'plug but include mechanical.			$576,000	$864,000			info?		$650.00	$780,000	$576,000	$780,000
3.03	Coffer Dams (2) - Fill cells u/s and d/s + toe slopes	14,000	CY	$40.00	$560,000	check Q's with new crest heights, say 8,000 cy RCC @ 70 + 6,000 cy Rockfill @ 15. = 650KHigh end if pushed to 480 and rockfill - say 45kcy = $675K.			$448,000	$672,000			info?			info?	$448,000	$672,000
3.04	Foundation Excavation - seepage key (assume 20'wide x 150' long x 4' deep	450	CY	$8.00	$3,600	Cofferdam key allowance		300%	$2,880	$10,800			info?			info?	$2,880	$10,800
3.05	Foundation Dewatering - assume several dewatering pump systems operating selectively 24/7 over 12 month foundation construction exposure	360	Day	$2,800.00	$1,008,000	**Changes for final: increase foundation exposure from 6 to 12 months.** 2nd contract may add unwaterring and time for dewatering for RCC foundation		150%	$806,400	$1,512,000			info?			info?	$806,400	$1,512,000
3.06	Coffer Dams - Other assume 25' high x 150 top length, 35' base length, cell construction (e.g. sheet pile, steel, other fabricated metal items)	7,000	SF	$30	$210,000	may include isolation of portal structures, tailwater structures, peripheral dewatering stages			$168,000	$252,000			info?			info?	$168,000	$252,000
3.07	Coffer Dams - Risk contingency for overtopping	1	LS	$1,000,000.00	$1,000,000	contemplates partial or threshold-bound contractor responsibility, risk apportioned cost of event recovery, rework, delay			$800,000	$1,200,000			info?			info?	$800,000	$1,200,000
4	**Lands and Easements**				$0				$0	$0			info?			info?	$0	$0
4.01	Reservoir Extents Fee Title	1,200	Acre	$4,400	$5,280,000	Best to be considered in non-contract costs. Perhaps cost conservatively overlaps with non-contract cost factor below.	100%	100%	$5,280,000	$5,280,000			info?			info?	$5,280,000	$5,280,000
4.02	Reservoir Extents/Flood Easement	110	Acre	$4,400	$484,000	Best to be considered in non-contract costs. Perhaps cost conservatively overlaps with non-contract cost factor below.	100%	100%	$484,000	$484,000			info?			info?	$484,000	$484,000
4.03	Reservoir orphaned access roadway reconnection allowance (to WeyCo?)	5	Mile	$1,000,000	$5,000,000	Unit price potentially higher for permanent versus constuction roads. Line item also perhaps better considered under non-contract cost factor.		110%	$4,000,000	$5,500,000			info?			info?	$4,000,000	$5,500,000
	Phase 2 - Main Dam																$0	$0
5	**Main Dam Structure**				$0				$0	$0			info?			info?	$0	$0
5.01	Excavation - Foundation General	710,000	CY	$6.50	$4,615,000	**Changes for final: revised quantities.** Reference FRFA S-1 annotated from Chehalis_All_Figs_2016-10-19.pdf, also this worksheet FRFA Exc Guess tab.			$3,692,000	$5,538,000		$5.50	$3,905,000		$7.50	$5,325,000	$3,905,000	$5,325,000
5.02	Excavation - Foundation Rock	210,000	CY	$27.00	$5,670,000	**Changes for final: revised quantities.** Reference FRFA S-1 annotated from Chehalis_All_Figs_2016-10-19.pdf, also this worksheet FRFA Exc Guess tab. Some rock will be structural exc in fresh rock, most will be foundation footprint, getting to good rock below the rock contact; i.e potentially a high degree ripable.			$4,536,000	$6,804,000		$25.00	$5,250,000		$30.00	$6,300,000	$5,250,000	$6,300,000
5.03	Roller Compacted Concrete - Composite Scope	1,475,000	CY	$96.00	$141,600,000	**Changes for final: revised quantities. Expanded RCC unit cost development work breakdown, revisited unit pricing, and increased unit pricing to reflect upstream conventional face and downstream GERCC.** RCC unit pricing includes aggregate, cement-fly ash, lift bedding, abutment bedding, dam joints, and **2.5' upstream conventional face and downstream GERCC.** Conventional concrete spillway face - included elsewhere.			$113,280,000	$169,920,000		$81.00	$119,475,000		$111.00	$163,725,000	$119,475,000	$163,725,000
5.04	Fill - Foundation Backfill	375,000	CY	$5.50	$2,062,500	**Changes for final: revised quantities.**			$1,650,000	$2,475,000			info?			info?	$1,650,000	$2,475,000
5.05	Conventional Concrete Reinforced (miscellaneous)	750	CY	$850.00	$637,500	Refine quantities along with all strucures next phase.			$510,000	$765,000			info?			info?	$510,000	$765,000
5.06	Conventional Concrete Non-Reinforced Mass Concrete (bedding, abutment contact, cover over sluice conduits, assume nominal contact layer)	15,000	CY	$400.00	$6,000,000	Refine quantities along with all strucures next phase.			$4,800,000	$7,200,000			info?			info?	$4,800,000	$7,200,000
5.07	Concrete - Dam and Crest Spillway	6,500	CY	$750.00	$4,875,000	**Changes for final: None. Consider this item only as upper spillway.** No facing should be included if flip bucket chute face is elsewhere. Leave in for ogee, spillway approach walls, piers.		120%	$3,900,000	$5,850,000			info?			info?	$3,900,000	$5,850,000
5.08	Foundation Treatment - Grout Curtain Drilling	50,000	LF	$45.00	$2,250,000	revisited pricing; 1700lf @ 10', plus 50% secondary, plus 25% tertiary @ 80' deep = 298 holes @ 90' = 26,820lf; if consolidation grouting - add 220,000 sf @ 400sf/ hole @ 20' deep = 11,000lf = 37,820 lf. If double curtain plus 25% extra = 383 holes @ 90' = 34,470lf, plus 11k consolidation grouting = 45,470lf. Use 50k lf. Depth: 300' at 35'+300' at 85'+500' at 140' + 200' at 130' + 400' at 55' = 154,000 / 1700' = 90'.			$1,800,000	$2,700,000			info?			info?	$1,800,000	$2,700,000
5.09	Foundation Treatment - Grout Curtain Cement	35,000	Sack	$40.00	$1,400,000	**Changes for final: revised quantity, increased unit price.** Assume 0.7 bag per lf			$1,120,000	$1,680,000			info?			info?	$1,120,000	$1,680,000
5.1	Flood Regulating Conduit Control Structures - Reinforced Concrete	5,800	CY	$800.00	$4,640,000	Assume 2' thick around perimeter of sluices & air shafts. Refine quantities along with all strucures next phase; include inside and downstream of dam; include control building on crest or at downstream, depending on final concept drawings			$3,712,000	$5,568,000			info?			info?	$3,712,000	$5,568,000
5.11	Flood Regulating Conduit Control Gates - Fab and Construct	120,000	LB	$15.00	$1,800,000	Assume 2 @ 30 tons.			$1,440,000	$2,160,000			info?			info?	$1,440,000	$2,160,000
5.12	Bulkhead gates	300,000	LB	$15.00	$4,500,000	Assume 2 @ 25 tons, and 2 @ 50 tons			$3,600,000	$5,400,000			info?			info?	$3,600,000	$5,400,000
5.13	Hoists, cylinders, machinery	200,000	LB	$15.00	$3,000,000				$2,400,000	$3,600,000			info?			info?	$2,400,000	$3,600,000
5.14	Reservoir drain valve in tunnel plug (assume 4x4' knife valve)	1	Each	$200,000.00	$200,000		50%	100%	$100,000	$200,000			info?			info?	$100,000	$200,000
5.15	WQ Regulating Outlets w/ hollow cone valves (4 - 4'dia)	4	Each	$375,000.00	$1,500,000				$1,200,000	$1,800,000			info?			info?	$1,200,000	$1,800,000

Judgment-Level Cost Opinion
Pricing/Work Breakdown Summary

Project: ***Chehalis Dam***
Alternative: ***FRFA Pre-Dwg; 0.1 : 0.85:1:***

Pricing - contractor cost basis 1 or bid basis 2: **2**

Range Driver - 1 = %, 2 = Q & $, 3 = Combination: **3**
Default Low ⇨ **80%** **Default High ⇨** **120%**

	Weighting ⇨	20% low	70% likely	10% high	20% low	70% likely	10% high
$293M - Jul-16		**Low End**	**$342,440,309**		**Low End**	**$424,700,495**	
		Likely	**$404,440,113**		**Likely**	**$501,593,742**	
$454M - Jul-18		**High End**	**$475,512,272**		**High End**	**$589,738,684**	
		Weighted	**$399,147,368**		**Weighted**	**$495,029,587**	

		Base or Likely Cost Case					**Range Development: Driven by Percent**				**Driven by Q & Unit $**						**Driven by Combo**	
Work Item	**Description**	**Quantity**	**Unit**	**Unit Price [1]**	**Total $**	**Estimate Notes & Considerations**	**Low End % (def=80%)**	**High End % (def=120%)**	**Low % Total $**	**High % Total $**	**Low End Q**	**Low End Unit $**	**Low End Total $**	**High End Q**	**High End Unit $**	**High End Total $**	**Low End Tot $**	**High End Tot $**
5.16	WQ Regulating Outlet w/ hollow cone valves (1 - 7'dia)	1	Each	$1,100,000.00	$1,100,000				$880,000	$1,320,000			info?			info?	$880,000	$1,320,000
5.17	WQ Intake Tower - Conventional Concrete Reinforced (assume 20% of 300' high x 40'x40' 1/4 round section)	2,800	CY	$750.00	$2,100,000				$1,680,000	$2,520,000			info?			info?	$1,680,000	$2,520,000
5.18	WQ Intake Tower - Conventional Concrete Non-Reinforced (assume 80% of 300' high x 40'x40' 1/4 round section)	11,200	CY	$400.00	$4,480,000	Refine quantities along with all strucures next phase.	100%		$4,480,000	$5,376,000			info?			info?	$4,480,000	$5,376,000
5.19	Trashrack steel framing	1,360,000	LB	$6.50	$8,840,000	Assume 300 ft high, 10 members 3' dia x 4.5'deep, steel column			$7,072,000	$10,608,000			info?			info?	$7,072,000	$10,608,000
5.2	Trashrack - concrete side wall and decking	2,000	CY	$850.00	$1,700,000				$1,360,000	$2,040,000			info?			info?	$1,360,000	$2,040,000
6	**Spillway Flip Bucket**				$0				$0	$0			info?			info?	$0	$0
6.01	Conventional Concrete Reinforced (assume 3' thick surface)	7,800	CY	$700.00	$5,460,000				$4,368,000	$6,552,000			info?			info?	$4,368,000	$6,552,000
6.02	Non-Conventional Concrete Non-Reinforced (could be RCC grout enriched, assume 30ft wedge under reinforced concrete surface)	9,750	CY	$225.00	$2,193,750	unit price - accomodates higher RCC placement and utilization of some mass conventional concrete	85%		$1,864,688	$2,632,500			info?			info?	$1,864,688	$2,632,500
7	**Sluice Stilling Basin**				$0				$0	$0			info?			info?	$0	$0
7.01	Excavation - Foundation General	20,000	CY	$8.00	$160,000	Refine all excavation and backfill quantities next phase			$128,000	$192,000			info?			info?	$128,000	$192,000
7.02	Excavation - Foundation Rock	10,000	CY	$30.00	$300,000				$240,000	$360,000			info?			info?	$240,000	$360,000
7.03	Fill - Foundation Backfill	18,000	CY	$9.00	$162,000				$129,600	$194,400			info?			info?	$129,600	$194,400
7.04	Conventional Concrete Reinforced (assume 3' thick surface)	4,900	CY	$800.00	$3,920,000				$3,136,000	$4,704,000			info?			info?	$3,136,000	$4,704,000
7.05	Conventional Concrete Non-Reinforced (assume 4.5'thick bedding under floor only)	2,000	CY	$400.00	$800,000				$640,000	$960,000			info?			info?	$640,000	$960,000
8	**Wing Dam Structure**				$0				$0	$0			info?			info?	$0	$0
8.01	Excavation - Foundation General (assume footprint 270' @ widest x 10 ft deep)	33,333	CY	$6.50	$216,667				$173,333	$260,000			info?			info?	$173,333	$260,000
8.02	Excavation Cutoff Trench - Foundation Rock (assume trench 30 ft wide x 20 ft deep)	13,333	CY	$30.00	$400,000				$320,000	$480,000			info?			info?	$320,000	$480,000
8.03	Fill - Wingdam Embankment	120,000	CY	$15.00	$1,800,000		90%		$1,620,000	$2,160,000			info?			info?	$1,620,000	$2,160,000
8.04	Fill - Wingdam Riprap Facing (assume 5' blanket U/S and D/S)	8,000	CY	$65.00	$520,000				$416,000	$624,000			info?			info?	$416,000	$624,000
	Composite & Unlisted Work																	
55	Fish passage structure - costs not included	1	ls	$0	$0	Costs independently assessed in report			$0	$0			info?			info?	$0	$0
56	Unlisted Work	1	ls	$5,000,000	$5,000,000		85%	115%	$4,250,000	$5,750,000			info?			info?	$4,250,000	$5,750,000
57					$0				$0	$0			info?			info?	$0	$0
58					$0				$0	$0			info?			info?	$0	$0
59					$0				$0	$0			info?			info?	$0	$0
60					$0				$0	$0			info?			info?	$0	$0
61					$0				$0	$0			info?			info?	$0	$0
62					$0				$0	$0			info?			info?	$0	$0
63					$0				$0	$0			info?			info?	$0	$0
64					$0				$0	$0			info?			info?	$0	$0
65					$0				$0	$0			info?			info?	$0	$0
					$0				$0	$0			info?			info?	$0	$0
Subtotal without mobilization & general expense					**$266,298,017**		**$217,836,901**	**$319,628,700**	**$217,836,901**	**$319,628,700**			**$137,735,000**			**$189,475,000**	**$225,475,101**	**$313,094,500**

Mobilization & project indirect expense		0%		$0	unallocated project indirect or jobsite overhead assumed in unit pricing
Contractor Cost				**$266,298,017**	Note 1: Unit prices as noted in header, either reflect a bid price basis (no factor application of corporate OH & profit), or a contractor cost basis requiring a corporate OH & profit to get to a bid total
Contractor Margin - corporate overhead & profit		~~0%~~	Bid Basis	$0	Note 2: NA - not applicable to project; NE - not evident in estimate; NI - noted but not itemized in estimate
Contractor Bid - before design/procurement contingencies				**$266,298,017**	
Contract Contingencies - design and procurement contingencies		12.5%		$33,287,252	⇦ RCC estimate dominance, work breakdown thoroughness, and work understanding support a design contingency lower than typical (i.e. 20%) at this early design level
Contract Cost - contracator bid with design & procurement contingencies				**$299,585,269**	
Construction Contingency: post-award change & dispute factor		10%		$29,958,527	
Non-Contract Costs: PM, planning, design, CM ...		25%		$74,896,317	⇦ permitting, site characterization, CM during construction,etc.
Total Project Cost - before escalation				**$404,440,113**	**Compares to $293M low bound, and $454M high bound July 2016**
Escalation - annual %; from; to	3.5%	1-Oct-16	1-Jan-23	$97,153,629	⇦ Presume NTP - early 2019, say 7 years construction = 2.5 + 3.75 years = 6.25 years
Total Project Cost - including escalation			**6.3 yr**	**$501,593,742**	**<< 188% above total w/o mobilization**

Attachment B
Summary Table of Estimate Refinements Leading to the Final Conceptual Design OPC

Chehalis Judgment-Level Cost Opinion

Summary of Final vs. Draft (Oct-2016) - FRFA and FRO Alternatives

Item #	Adjustment ($)	Estimate Refinement Rationale
FRFA Adjustments		
2.02	700,000	Considering November 2016 site visit and 2016 site development planning, **increase access road** development by adding 1 mile, from 2 to 3.
3.01	640,000	Considering final preliminary design planning, **increase diversion tunnel length** to better reflect final drawing alignment. (Diversion tunnel was pushed further into the bank to generate more conservative clearance between the anticipated foundation grout curtain and the tunnel alignment.)
3.05	504,000	**Increase dewater to better reflect** foundation exposure duration.
5.01	390,910	Revised **(increased) excavation quantities** to better reflect finalized preliminary drawing layout
5.02	198,936	Revised **(increased) excavation quantities** to better reflect finalized preliminary drawing layout
5.03	12,865,000	Revised **(increased) RCC quantity** to better reflect finalized preliminary drawing layout. 155k cy @ $83/cy, which was the original unit price)
5.03	19,175,000	**Increased RCC unit price** on 1,475,000cy by $16/cy; refined unit pricing to reflect cast-in-place upstream face, GERCC downstream face; added RCC test section, cooling, and plant and delivery system erection and commissioning; refined RCC mix, deliver and place unit pricing; increased RCC aggregate pricing to better reflect site quarry development uncertainty.
5.04	-428,054	**Revised (decreased) backfill quantities** to better reflect finalized preliminary drawing layout
5.09	245,000	**Revised (increased) cement for grouting quantity** to better reflect the drilling quantity @ 0.7 bag/lf; and **increased the unit price of cement** for grouting.
Various	215,307	Minor adjustments (i.e. rounded quantities for unchanged estimate items)
Subtotal	**34,506,099**	**Subtotal line-item cost adjustments**
	-13,071,131	Reduced design and procurement contingencies from 20% to 12.5%; primarily reflecting the thorough WBS and expanded RCC WBS & understanding.
Subtotal	21,434,968	Net cost additions to the "likely" estimate; before construction contingencies and non-contract cost factor
	35%	Construction contingency and non-contract cost factor to arrive at total adjustments before escalation (unchanged)
	7,502,239	
Total	**28,937,206**	**Total cost adjustments to likely estimate, before escalation**
	28,000,000	Rounded comparison from summary
FRO Adjustments		
2.02	350,000	Considering November 2016 site visit and 2016 site development planning, **increase access road** development by adding 0.5 mile, from 1.5 to 2.
3.01	640,000	Considering final preliminary design planning, **increase diversion tunnel length** to better reflect final drawing alignment. (Diversion tunnel was pushed further into the bank to generate more conservative clearance between the anticipated foundation grout curtain and the tunnel alignment.)
3.05	252,000	**Increase dewater** to better reflect foundation exposure duration.
5.03	10,000,000	Revised **(increased) RCC quantity** to better reflect finalized preliminary drawing layout. 125k cy @ $80/cy, which was the original unit price)
5.03	9,438,720	**Increased RCC unit price** on 870,000cy by $14/cy; refined unit pricing to reflect GERCC both upstream and downstream faces; added RCC test section, cooling, and plant and delivery system erection and commissioning; increased RCC mix, deliver and place unit pricing; increased RCC aggregate pricing to better reflect site quarry development uncertainty.
5.08	22,500	**Adjust (increase) grout drilling quantities** to reflect a single curtain; 50% secondary, 25% tertiary; **increase unit price**
5.09	115,000	**Revised (increased) cement for grouting quantity** to better reflect the drilling quantity @ 0.7 bag/lf; and **increased the unit price of cement** for grouting.
5.14	200,000	Added tunnel plug reservoir drain valve overlooked in prior estimate
Various	82,148	Minor adjustments (i.e. rounded quantities for unchanged estimate items)
Subtotal	**21,100,368**	**Subtotal line-item cost adjustments**
	-9,145,440	Reduced design and procurement contingencies from 20% to 12.5%; primarily reflecting the thorough WBS and expanded RCC WBS & understanding.
Subtotal	11,954,928	Net cost additions to the "likely" estimate; before construction contingencies and non-contract cost factor
	35%	Construction contingency and non-contract cost factor to arrive at total adjustments before escalation (unchanged)
	4,184,225	
Total	**16,139,153**	**Total cost adjustments to likely estimate, before escalation**
	16,000,000	Rounded comparison from summary

Attachment C
Aggregate Sourcing Information Along with the Development of RCC Unit Pricing

Chehalis Dam - Constructability, Schedule and Cost Support
Aggregate Sourcing Options & Cost Range Development

Select Quantity Drivers	from tab "SP> Agg Demand Q3 RC"
RCC	1,475,000 cy
RCC aggregate - supplied or stockpiled	2,635,000 tn
All aggregates - supplied or stockpiled	2,690,000 tn
Quarry Feed - if fully site produced	3,090,000 tn
Quarry drill & shoot volume	1,820,000 cy
Quarry Area @ 40' benches + 20%	18 ac
Base Stripping Volume @ 40'	590,000 cy

Essence of Analysis

Analysis considers variety in source type, rock type, location, delivery costs, commercial supply costs, and site-produced cost approach. If design is flexible enough to utilize or allow a few of the alternatives, it is not necessary or appropriate to presume the lowest of the low and the highest of the high as a recommended range.

Range of "direct" unit cost	Low	$11.48 /tn	Average of all	$15.97 /tn	High	$21.43 /tn
Range with "bid" unit cost	Low	$14.92 /tn	Average of all	$20.76 /tn	High	$27.86 /tn
Safe Site-approach "direct" - Option 3	Low	$12.84 /tn	Likely	$15.89 /tn	High	$18.96 /tn
Safe Site-approach "bid" - Option 3	Low	$16.69 /tn	Likely	$20.65 /tn	High	$24.65 /tn
Recommended to Use in Est - "Direct"	**Low**	**$12.00 /tn**	**Likely**	**$14.50 /tn**	**High**	**$17.50 /tn**
Recommended to Use in Est - "Bid"	**Low**	**$15.60 /tn**	**Likely**	**$18.85 /tn**	**High**	**$22.75 /tn**

Source Option ID		**1**			**2**			**3**			**4**			**5**		
Siource Option Name	**Formula & Starting Costs Column**	**Commercial Basalt - Alderbrook**			**Commercial Basalt - Hope Creek**			**"Quarry 1" Basalt**		1	**"Quarry 3" Basalt Rock Creek**		3	**"Quarry 2" Basalt**		2
Material Type		**Basalt**			**Basalt**			**Basalt**			**Basalt**			**Basalt**		
Source Type		**Commercial**			**Commercial**			**Developed**			**Developed**			**Developed**		
Representative Pit Name(s)		**Alderbrook Qaurry**			**Hope Creek Quarry**			**Quarry 1: 2-mile, Right Reservoir Rim**			**Rock Creek, Quarry 3: 9-mi**			**Quarry 2: 7-mi**		
Rock Quality (good rock durability)		**good; LA 16.5, sg 2.71, abs 2.7, ASR marg. @ 0.212**			**marginal; LA 17.9, sg 2.71, abs 6.75, ASR marg. @ 0.215**			**poor-marginal; LA 27.1, sg. 2.60, abs. 6.5; ASR 0.08**			**good; LA 18.9, sg 2.73, abs 1.08, ASR marg. @ 0.011**			**good; LA 24.5, sg 2.2.7, abs 3.9, ASR marg. @ 0.035**		
Source Quality (evident consistency)		**undetermined; inquiry needed**			**undetermined; obsorption suspect, inquiry needed**			**single boring, limited outcrop, thick overburden; more investigation required**			**undetermined; inquiry needed**			**single boring, limited outcrop, more investigation required**		
Evident Quantity		**undetermined; inquiry needed**			**undetermined; favorable expandabilty on map; inquiry needed**			**single boring, limited outcrop, thick overburned; more investigation required**			**undetermined; favorable expandabilty on map; inquiry needed; assumed - no basis - 20' overburden depth**			**undetermined; favorable expandabilty on map; inquiry needed; assumed - 10' overburden depth, siesmic line**		
Quantity/Quality Risk Rating		**moderate to low risk**			**moderate to high risk**			**moderate to high risk**			**low to moderate risk**			**TBD**		
Unit Cost Development (not applicable grayed, entry or override light shade)																
		Low End	Likely	High End	Low End	Likely	High End	Low End	Likely	High End	Low End	Likely	High End	Low End	Likely	High End
Haul Development																
Site quarry mileage to site	10 mi	[redacted]						2.0 mi	2.3 mi	2.5 mi	9 mi	9 mi	9 mi	8 mi	8 mi	8 mi
Commercial Mileage to site	20 mi	25 mi	25 mi	25 mi	11 mi	11 mi	11 mi	~~20 mi~~	~~20 mi~~	~~20 mi~~	~~26 mi~~	~~39 mi~~	~~42 mi~~	~~20 mi~~	~~20 mi~~	~~20 mi~~
1-way drive time (approx Googlemaps)	35 min	33 min	38 min	40 min	20 min	20 min	25 min	~~22 min~~	~~35 min~~	~~47 min~~	~~40 min~~	~~55 min~~	~~60 min~~	~~35 min~~	~~35 min~~	~~35 min~~
Load & unload time	5 min	5 min	8 min	10 min	5 min	8 min	10 min	~~5 min~~	~~5 min~~	~~5 min~~	~~5 min~~	~~5 min~~	~~5 min~~	~~5 min~~	~~5 min~~	~~5 min~~
Traffic allowance (each way)	5 min	5 min	5 min	10 min	0 min	5 min	10 min	~~5 min~~	~~5 min~~	~~5 min~~	~~5 min~~	~~10 min~~	~~15 min~~	~~0 min~~	~~5 min~~	~~10 min~~
Calculated cycle time	80 min	76 min	94 min	110 min	45 min	58 min	80 min	~~59 min~~	~~85 min~~	~~109 min~~	~~95 min~~	~~135 min~~	~~155 min~~	~~75 min~~	~~85 min~~	~~95 min~~
Truck capacity	25 tn	35 tn	35 tn	25 tn	35 tn	35 tn	25 tn	~~25 tn~~	~~25 tn~~	~~25 tn~~	~~35 tn~~	~~35 tn~~	~~35 tn~~	~~35 tn~~	~~35 tn~~	~~25 tn~~
Truck hourly rate	$110.00 /hr	$100.00 /hr	$110.00 /hr	$115.00 /hr	$100.00 /hr	$110.00 /hr	$115.00 /hr	~~$110.00 /hr~~	~~$110.00 /hr~~	~~$110.00 /hr~~	~~$105.00 /hr~~	~~$110.00 /hr~~	~~$110.00 /hr~~	~~$105.00 /hr~~	~~$105.00 /hr~~	~~$95.00 /hr~~
Rail Mileage to Site	NA	NA	NA	NA	NA	NA	NA	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~
Quantity Basis - overriden underlined																
Quarry Area	18 ac	[redacted]						19 ac	18 ac	22 ac	17 ac	19 ac	21 ac	16 ac	18 ac	21 ac
Quarry feed	3,090,000 tn							3,090,000 tn	3,090,000 tn	3,350,000 tn	3,090,000 tn	3,090,000 tn	3,090,000 tn	3,090,000 tn	3,090,000 tn	3,350,000 tn
Furnish Commercial Aggregate	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	~~2,690,000 tn~~	~~2,690,000 tn~~	~~2,690,000 tn~~	~~2,690,000 tn~~	~~2,690,000 tn~~	~~2,690,000 tn~~	~~320,000 tn~~	~~320,000 tn~~	~~2,690,000 tn~~
Produce Site Aggregate	2,690,000 tn	[redacted]						2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn	2,690,000 tn
Overburden Volume	590,000 cy							1,250,000 cy	590,000 cy	1,450,000 cy	580,000 cy	600,000 cy	700,000 cy	280,000 cy	300,000 cy	350,000 cy
Topsoil Volume - 9" over area	22,000 cy							23,000 cy	22,000 cy	27,000 cy	21,000 cy	23,000 cy	26,000 cy	20,000 cy	22,000 cy	26,000 cy
Cost factors - overriden underlined																
Royalty - $/tn (assumed in land use agg.)	$0.75 /tn	[redacted]						$0.00 /tn	$0.00 /tn	$0.00 /tn	$0.00 /tn	$0.00 /tn	$0.00 /tn	$0.00 /tn	$0.00 /tn	$0.00 /tn
Contractor investigations - ls	$70,000							$50,000	$150,000	$250,000	$50,000	$75,000.00 /tn	$150,000	$50,000	$100,000.00 /tn	$200,000
Access development - ls	$150,000							$150,000	$200,000	$300,000	$50,000	$100,000	$150,000	$300,000	$350,000	$500,000
Clearing - $/ac	$4,000.00 /ac							$3,000.00 /ac	$4,000.00 /ac	$5,000.00 /ac	$3,000.00 /ac	$4,000.00 /ac	$5,000.00 /ac	$3,000.00 /ac	$4,000.00 /ac	$5,000.00 /ac
Exc topsoil & overburden - $/cy	$4.00 /cy							$3.00 /cy	$4.00 /cy	$5.00 /cy	$3.00 /cy	$4.00 /cy	$5.00 /cy	$3.00 /cy	$4.00 /cy	$5.00 /cy
Drill & blast - $/tn	$2.00 /tn							$1.50 /tn	$2.00 /tn	$2.25 /tn	$1.50 /tn	$2.00 /tn	$2.25 /tn	$1.50 /tn	$2.00 /tn	$2.25 /tn
Extract/load/haul - $/tn	$2.00 /tn							$1.75 /tn	$2.00 /tn	$2.50 /tn	$1.75 /tn	$2.00 /tn	$2.50 /tn	$1.75 /tn	$2.00 /tn	$2.50 /tn
Crushing & process - $/tn	$4.50 /tn							$3.50 /tn	$4.50 /tn	$5.00 /tn	$3.50 /tn	$4.50 /tn	$5.00 /tn	$3.50 /tn	$4.50 /tn	$5.00 /tn
Stockpile - $/tn	$1.40 /tn	$1.00 /tn	$1.50 /tn	$2.00 /tn	$1.40 /tn	$1.50 /tn	$2.00 /tn	$1.20 /tn	$1.40 /tn	$2.00 /tn	$1.20 /tn	$1.40 /tn	$2.00 /tn	$1.20 /tn	$1.40 /tn	$2.00 /tn
Site truck haul - $/tm	$0.60 /tm	[redacted]						$0.50 /tm	$0.65 /tm	$0.80 /tm	$0.35 /tm	$0.40 /tm	$0.45 /tm	$0.45 /tm	$0.50 /tm	$0.65 /tm
Place topsoil - $/cy	$4.00 /cy							$3.00 /cy	$4.00 /cy	$4.50 /cy	$3.00 /cy	$4.00 /cy	$4.50 /cy	$3.00 /cy	$4.00 /cy	$4.50 /cy
Dress & reclamation - $/ac	$4,000.00 /ac							$1,500.00 /ac	$4,000.00 /ac	$6,000.00 /ac	$1,500.00 /ac	$4,000.00 /ac	$6,000.00 /ac	$1,500.00 /ac	$4,000.00 /ac	$6,000.00 /ac
Dewater allowance - ls	$50,000							$0	$0	$100,000	$0	$0	$100,000	$0	$0	$100,000
Mob/demob - ls	$150,000							$0	$200,000	$350,000	$0	$200,000	$350,000	$0	$200,000	$350,000
Commercial supply - $/tn	$10.00 /tn	$9.00 /tn	$10.00 /tn	$11.00 /tn	$9.00 /tn	$10.00 /tn	$11.00 /tn	~~$10.00 /tn~~	~~$10.00 /tn~~	~~$10.00 /tn~~	~~$10.00 /tn~~	~~$10.00 /tn~~	~~$12.00 /tn~~	~~$9.00 /tn~~	~~$10.00 /tn~~	~~$11.50 /tn~~
Commercial truck haul - $/tm	$0.23 /tm	$0.14 /tm	$0.20 /tm	$0.34 /tm	$0.19 /tm	$0.28 /tm	$0.56 /tm	~~$0.22 /tm~~	~~$0.31 /tm~~	~~$0.40 /tm~~	~~$0.18 /tm~~	~~$0.18 /tm~~	~~$0.19 /tm~~	~~$0.19 /tm~~	~~$0.21 /tm~~	~~$0.30 /tm~~
Commercial rail haul - $/tm	NA	NA	NA	NA	NA	NA	NA	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~	~~NA~~
Costs																
Royalty	$2,017,500	[redacted]						$0	$0	$0	$0	$0	$0	$0	$0	$0
Contractor investigations	$70,000							$50,000	$150,000	$250,000	$50,000	$75,000	$150,000	$50,000	$100,000	$200,000
Access development	$150,000							$150,000	$200,000	$300,000	$50,000	$100,000	$150,000	$300,000	$350,000	$500,000
Clearing	$72,000							$57,000	$72,000	$110,000	$51,000	$76,000	$105,000	$48,000	$72,000	$105,000
Exc topsoil & overburden	$2,360,000							$3,750,000	$2,360,000	$7,250,000	$1,740,000	$2,400,000	$3,500,000	$840,000	$1,200,000	$1,750,000
Drill & blast	$6,180,000							$4,635,000	$6,180,000	$7,537,500	$4,635,000	$6,180,000	$6,952,500	$4,635,000	$6,180,000	$7,537,500
Extract/load/haul	$6,180,000							$5,407,500	$6,180,000	$8,375,000	$5,407,500	$6,180,000	$7,725,000	$5,407,500	$6,180,000	$8,375,000
Crushing & process	$13,905,000							$10,815,000	$13,905,000	$16,750,000	$10,815,000	$13,905,000	$15,450,000	$10,815,000	$13,905,000	$16,750,000
Stockpile	$3,766,000							$3,228,000	$3,766,000	$5,380,000	$3,228,000	$3,766,000	$5,380,000	$3,228,000	$3,766,000	$5,380,000
Site truck haul	$16,140,000							$2,690,000	$4,021,550	$5,380,000	$8,473,500	$9,684,000	$10,894,500	$9,684,000	$10,760,000	$13,988,000
Place topsoil	$88,000							$69,000	$88,000	$121,500	$63,000	$92,000	$117,000	$60,000	$88,000	$117,000
Dress & reclamation	$72,000							$28,500	$72,000	$132,000	$25,500	$76,000	$126,000	$24,000	$72,000	$126,000
Dewater allowance	$50,000							$0	$0	$100,000	$0	$0	$100,000	$0	$0	$100,000
Mob/demob	$150,000							$0	$200,000	$350,000	$0	$200,000	$350,000	$0	$200,000	$350,000
Commercial supply	$26,900,000	$24,210,000	$26,900,000	$29,590,000	$24,210,000	$26,900,000	$29,590,000	~~$26,900,000~~	~~$26,900,000~~	~~$26,900,000~~	~~$26,900,000~~	~~$26,900,000~~	~~$32,280,000~~	~~$2,880,000~~	~~$3,200,000~~	~~$30,935,000~~
Commercial truck haul	$12,374,000	$9,735,238	$13,245,048	$22,685,667	$5,764,286	$8,172,476	$16,498,667	~~$11,638,733~~	~~$16,767,667~~	~~$21,502,067~~	~~$12,777,500~~	~~$19,022,143~~	~~$21,840,238~~	~~$1,200,000~~	~~$1,360,000~~	~~$16,184,833~~
Commercial rail haul																
Direct Total Cost - Site	$51,200,500	[redacted]						$30,880,000	$37,194,550	$52,036,000	$34,538,500	$42,734,000	$51,000,000	$35,091,500	$42,873,000	$55,278,500
Direct Unit Cost - Site	$19.03							$11.48	$13.83	$19.34	$12.84	$15.89	$18.96	$13.05	$15.94	$20.55
Direct Total Cost - Commercial (Truck)	$43,040,000	$36,635,238	$44,180,048	$57,655,667	$33,740,286	$39,107,476	$51,468,667	~~$41,766,733~~	~~$47,433,667~~	~~$53,782,067~~	~~$42,905,500~~	~~$49,688,143~~	~~$59,500,238~~	~~$7,308,000~~	~~$8,326,000~~	~~$52,499,833~~
Direct Unit Cost - Commercial (Truck)	$16.00	$13.62	$16.42	$21.43	$12.54	$14.54	$19.13	~~$15.53~~	~~$17.63~~	~~$19.99~~	~~$15.95~~	~~$18.47~~	~~$22.12~~	~~$22.84~~	~~$26.02~~	~~$19.52~~
Direct Total Cost - Commercial (Rail)																
Direct Unit Cost - Commercial (Rail)																
Contractor Unallocated Indirect plus OH & P	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%
"Bid" Unit Costs (note: accent indicates out of "recommended" low to high range		$17.70 /tn	$21.35 /tn	**$27.86 /tn**	$16.31 /tn	$18.90 /tn	**$24.87 /tn**	**$14.92 /tn**	$17.98 /tn	**$25.15 /tn**	$16.69 /tn	$20.65 /tn	**$24.65 /tn**	$16.96 /tn	$20.72 /tn	**$26.71 /tn**

Aggregate Use and Demand - Quarry 1 Basalt

Crushing Operations

Assumptions:		Effective Productions:	Single Op	All Ops Product	All Ops - Feed
Primary feed rate (say 36x48 @ 6"-8")	650 tph				
Primary's in operation	1	Hourly	550	550 tph	650 tph
% waste through eliminations or reject at primary	15%	Shift	4,400	4,400 tps	5,200 tps
% loss secondary processing or recomb % mismatch	0%	Daily	8,800	8,800 tpd	10,400 tpd
Hours per shift	10	Weekly	39,600	39,600 tpw	46,800 tpw
Productive (crushing) hours per shift	8.0	Monthly	139,000	139,000 tpm	164,000 tpm
Shifts per day	2	Annual	1,668,000	1,668,000 tpy	1,968,000 tpy
Worked shifts per week	9	Annualized/Mo	130,000	130,000 tpm	153,000 tpm
Weeks per month (10 weeks down per year)	3.50				
Months per year	12				
Stockpile Loss - all products avg	3%				

Concrete/RCC Operations

Assumptions for stand-alone plant operations:		Productions and Demand	cy	tons - agg product
RCC Mixer Capacity (say 6 cy @ 70sec cy)	309	Hourly - theoretical	1,234 cy/hr	2,204 tons /
Mixers in plant assembly	4	Hourly - plant max	864 cy/hr	1,543 tons /
Combined mixer efficiency within plant ops	70%	Max Shift	6,912 cy/sh	12,345 tons /
Hours per shift	10	Probable Max per Day	12,096 cy/dy	21,604 tons /
Productive (batching) hours per shift	8.0	Probable Max Week	60,480 cy/wk	108,018 tons /
Net effective shifts per day	1.75	Probable Max Month	211,680 cy/mo	378,065 tons /
Net effective days per week	5	Daily b4 weather average @ 60%	7,258 cy/dy	12,962 tons /
Net effective weeks per month	3.50	Non-weather days per year:	237 dy/yr	
Mix design aggregate tons per cy	1.70	Days required	203 dy	or 10 mo
RCC & conc batch waste %	2%	Use for average RCC/mo	130,000 cy/mo	232,183 tons /
Agg used for batching - tn/cy	1.73			
Agg needed including stockpile loss - tn/cy	1.79 tpy ➔		2,634,380	tn of RCC agg product needed
Annual production capacity: 2,058 hr	1,778 k cy	10 mo	2,685,880	tn site-produced agg product needed

Note: Considers 1st to last lift productivity with various production control; RCC, forming, gallery, joints, split placements, etc.

Aggregate Demand:

Initial RCC Aggregate Stockpile		1,200,000	tons ➔	9 months
Conc/RCC Placement	1,475,000 cy			
Months of Placement	11.3 ➔	130,000 cy/mo		

RCC month	RCC Placed	Agg Used	Agg SP and Produced	Aggregate on Hand	Depletion per month
0	0	0	1,200,000	1,200,000	
1	130,000	232,183	1,330,000	1,097,817	-102,183
2	260,000	464,365	1,460,000	995,635	-102,183
3	390,000	696,548	1,590,000	893,452	-102,183
4	520,000	928,730	1,720,000	791,270	-102,183
5	650,000	1,160,913	1,850,000	689,087	-102,183
6	780,000	1,393,096	1,980,000	586,904	-102,183
7	910,000	1,625,278	2,110,000	484,722	-102,183
8	1,040,000	1,857,461	2,240,000	382,539	-102,183
9	1,170,000	2,089,643	2,370,000	280,357	-102,183
10	1,300,000	2,321,826	2,500,000	178,174	-102,183
11	1,430,000	2,554,009	2,630,000	75,991	-102,183
12	1,475,000	2,634,380	2,634,380	0	-75,991
13	1,475,000	2,634,380	2,634,380	0	0
14	1,475,000	2,634,380	2,634,380	0	0
15	1,475,000	2,634,380	2,634,380	0	0
16	1,475,000	2,634,380	2,634,380	0	0
17	1,475,000	2,634,380	2,634,380	0	0
18	1,475,000	2,634,380	2,634,380	0	0
19	1,475,000	2,634,380	2,634,380	0	0
20	1,475,000	2,634,380	2,634,380	0	0
21	1,475,000	2,634,380	2,634,380	0	0
22	1,475,000	2,634,380	2,634,380	0	0
23	1,475,000	2,634,380	2,634,380		
24	1,475,000	2,634,380	2,634,380		
25	1,475,000	2,634,380	2,634,380		
26	1,475,000	2,634,380	2,634,380		
27	1,475,000	2,634,380	2,634,380		
28	1,475,000	2,634,380	2,634,380		
29	1,475,000	2,634,380	2,634,380	0	0
30	1,475,000	2,634,380	2,634,380	0	0
31	1,475,000	2,634,380	2,634,380	0	0
32	1,475,000	2,634,380	2,634,380	0	0
33	1,475,000	2,634,380	2,634,380	0	0
34	1,475,000	2,634,380	2,634,380	0	0
35	1,475,000	2,634,380	2,634,380	0	0
36	1,475,000	2,634,380	2,634,380	0	0
37	1,475,000	2,634,380	2,634,380	0	0
38	1,475,000	2,634,380	2,634,380	0	0
39	1,475,000	2,634,380	2,634,380	0	0
40	1,475,000	2,634,380	2,634,380	0	0

Note: driven off of annualized monthly product; early peak RCC placement might necessitate higher agg production or larger initial stockpiling

Rough Stockpile sizing:

Assume 3 equal stockpiles — 400,000 tons per stockpile
at 40' h, 30° slopes, 1.4 tn/cy; 1 pile; with 75' — 525 ' w — 550 ' l
between long piles and 50' on perimeter… ≈ 1,825 ' w — 650 ' l — 28 acres

Rough Quarry Sizing:	
Site-produced RCC aggregate needed	2,634,380 tn
Site-produced concrete aggregates needed	tn
Other crushed/processed products	51,500 tn
Total process feed	3,088,761 tn
Quarry volume @ 1.7 ton product / cy shot	1,816,918 cy
Presumed quarry effective depth - ft	80
Raw quarry area - before access - sf	613,210
Quarry area increase for access/benching	40%
Quarry area to develop - sf	858,494
- acres	19.7
- if square - ftxft	927
Presumed stripping depth - ft	40
Overburden / stripping volume - cy	1,272,000
Quarry volume feed plus overburden - cy	3,088,918

Driving Concrete Quantities

RCC Total[1]	1,475,000	cy	3,400 tn/cy	⇦ site-produced
Assumed Conv Conc Total	104,650	cy	3,200 tn/cy	⇦ commercial
Other Agg Products Total	50,000	tn		

1 - from RCC Dam Q-s & Placement Plan - R06.xls; dwgs - Chehalis_All_Figs_2016-10-19.pdf

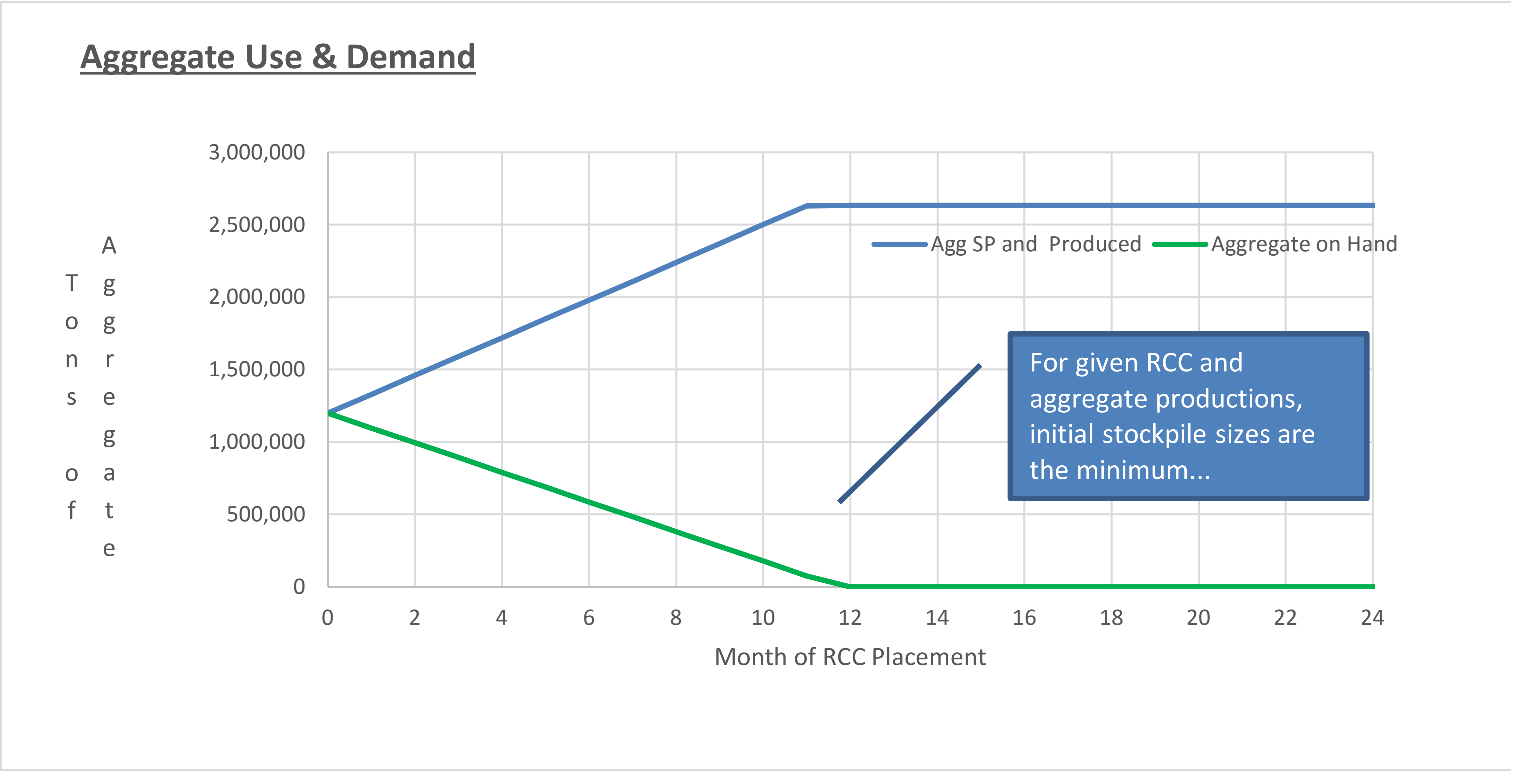
Conventional Concrete Summary:	104,650
3.2 Plug	1,200
5.5 miscellaneous	750
5.6 un-reinforced mass; bedding, abut's, sluice cover	51,400
5.10 outlet and outlet shaft	5,800
5.17 Intake tower	14,000
5.18 Intake trashrack walls	2,000
6.1-2 flip bucket	17,500
7.4-5 Stilling basin	7,000
Unlisted	5,000

Summary for Report

RCC	1,475,000 cy	11 months	134,091 cy/mo
Aggregate Product - tons	2,686,000 tn	21 months	130,000 tpm
Overburden: depth, cy	40 lf	1,272,000 cy	
Quarry Dimensions	20 ac	927 lf x lf	80 lf rock depth

RCC weather impact

Month	Lost Days	Prod Shfts
Jan	11	34
Feb	11	28
Mar	12	32
Apr	9	36
May	4	48
Jun	1	52
Jul	1	52
Aug	1	54
Sep	6	42
Oct	11	34
Nov	12	30
Dec	12	32
Totals	91	474
Effective shifts / week ⇨		9.1



Aggregate Use and Demand - Quarry 2 Basalt

Crushing Operations

Assumptions:		Effective Productions:	Single Op	All Ops Product	All Ops - Feed
Primary feed rate (say 36x48 @ 6"-8")	650 tph				
Primary's in operation	1	Hourly	550	550 tph	650 tph
% waste through eliminations or reject at primary	15%	Shift	4,400	4,400 tps	5,200 tps
% loss secondary processing or recomb % mismatch	0%	Daily	8,800	8,800 tpd	10,400 tpd
Hours per shift	10	Weekly	39,600	39,600 tpw	46,800 tpw
Productive (crushing) hours per shift	8.0	Monthly	139,000	139,000 tpm	164,000 tpm
Shifts per day	2	Annual	1,668,000	1,668,000 tpy	1,968,000 tpy
Worked shifts per week	9	Annualized/Mo	130,000	130,000 tpm	153,000 tpm
Weeks per month (10 weeks down per year)	3.50				
Months per year	12				
Stockpile Loss - all products avg	3%				

Concrete/RCC Operations

Assumptions for stand-alone plant operations:		Productions and Demand	cy	tons - agg product
RCC Mixer Capacity (say 6 cy @ 70sec cy)	309	Hourly - theoretical	1,234 cy/hr	2,204 tons /
Mixers in plant assembly	4	Hourly - plant max	864 cy/hr	1,543 tons /
Combined mixer efficiency within plant ops	70%	Max Shift	6,912 cy/sh	12,345 tons /
Hours per shift	10	Probable Max per Day	12,096 cy/dy	21,604 tons /
Productive (batching) hours per shift	8.0	Probable Max Week	60,480 cy/wk	108,018 tons /
Net effective shifts per day	1.75	Probable Max Month	211,680 cy/mo	378,065 tons /
Net effective days per week	5	Daily b4 weather average @ 60%	7,258 cy/dy	12,962 tons /
Net effective weeks per month	3.50	Non-weather days per year:	237 dy/yr	
Mix design aggregate tons per cy	1.70	Days required	203 dy	or 10 mo
RCC & conc batch waste %	2%	Use for average RCC/mo	130,000 cy/mo	232,183 tons /
Agg used for batching - tn/cy	1.73			
Agg needed including stockpile loss - tn/cy	1.79 tpcy ➔		2,634,380	tn of RCC agg product needed
Annual production capacity: 2,058 hr	1,778 k cy	10 mo	2,685,880	tn site-produced agg product needed

Considers 1st to last lift productivity with various production control; RCC, forming, gallery, joints, split placements, etc.

Aggregate Demand:

Initial RCC Aggregate Stockpile 1,200,000 tons ➔ 9 months
Conc/RCC Placement 1,475,000 cy
Months of Placement 11.3 ➔ 130,000 cy/mo

RCC month	RCC Placed	Agg Used	Agg SP and Produced	Aggregate on Hand	Depletion per month
0	0	0	1,200,000	1,200,000	
1	130,000	232,183	1,330,000	1,097,817	-102,183
2	260,000	464,365	1,460,000	995,635	-102,183
3	390,000	696,548	1,590,000	893,452	-102,183
4	520,000	928,730	1,720,000	791,270	-102,183
5	650,000	1,160,913	1,850,000	689,087	-102,183
6	780,000	1,393,096	1,980,000	586,904	-102,183
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10	1,300,000	2,321,826	2,500,000	178,174	-102,183
11	1,430,000	2,554,009	2,630,000	75,991	-102,183
12	1,475,000	2,634,380	2,634,380	0	-75,991
13	1,475,000	2,634,380	2,634,380	0	0
14	1,475,000	2,634,380	2,634,380	0	0
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22	1,475,000	2,634,380	2,634,380	0	0
23	1,475,000	2,634,380	2,634,380		0
24	1,475,000	2,634,380	2,634,380		0
25	1,475,000	2,634,380	2,634,380		0
26	1,475,000	2,634,380	2,634,380		0
27	1,475,000	2,634,380	2,634,380		0
28	1,475,000	2,634,380	2,634,380		0
29	1,475,000	2,634,380	2,634,380		0
30	1,475,000	2,634,380	2,634,380	0	0
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32	1,475,000	2,634,380	2,634,380	0	0
33	1,475,000	2,634,380	2,634,380	0	0
34	1,475,000	2,634,380	2,634,380	0	0
35	1,475,000	2,634,380	2,634,380	0	0
36	1,475,000	2,634,380	2,634,380	0	0
37	1,475,000	2,634,380	2,634,380	0	0
38	1,475,000	2,634,380	2,634,380	0	0
39	1,475,000	2,634,380	2,634,380	0	0
40	1,475,000	2,634,380	2,634,380	0	0

driven off of annualized monthly product; early peak RCC placement might necessitate higher agg production or larger initial stockpiling

Rough Stockpile sizing:

Assume 3 equal stockpiles	400,000 tons per stockpile		
at 40' h, 30° slopes, 1.4 tn/cy; 1 pile; with 75'	525 ' w	550 ' l	
between long piles and 50' on perimeter... ≈	1,825 ' w	650 ' l	28 acres

Rough Quarry Sizing:	
Site-produced RCC aggregate needed	2,634,380 tn
Site-produced concrete aggregates needed	tn
Other crushed/processed products	51,500 tn
Total process feed	3,088,761 tn
Quarry volume @ 1.7 ton product / cy shot	1,816,918 cy
Presumed quarry effective depth - ft	80
Raw quarry area - before access - sf	613,210
Quarry area increase for access/benching	25%
Quarry area to develop - sf	766,512
- acres	17.6
- if square - ftxft	876
Presumed stripping depth - ft	10
Overburden / stripping volume - cy	284,000
Quarry volume feed plus overburden - cy	2,100,918

Driving Concrete Quantities

RCC Total[1]	1,475,000	cy	3,400 tn/cy ⇦ site-produced
Assumed Conv Conc Total	104,650	cy	3,200 tn/cy ⇦ commercial
Other Agg Products Total	50,000	tn	

1 - from RCC Dam Q-s & Placement Plan - R06.xls; dwgs - Chehalis_All_Figs_2016-10-19.pdf

Conventional Concrete Summary:	104,650
3.2 Plug	1,200
5.5 miscellaneous	750
5.6 un-reinforced mass; bedding, abut's, sluice cover	51,400
5.10 outlet and outlet shaft	5,800
5.17 Intake tower	14,000
5.18 Intake trashrack walls	2,000
6.1-2 flip bucket	17,500
7.4-5 Stilling basin	7,000
Unlisted	5,000

Summary for Report

RCC	1,475,000 cy	11 months	134,091 cy/mo
Aggregate Product - tons	2,686,000 tn	21 months	130,000 tpm
Overburden: depth, cy	10 lf	284,000 cy	
Quarry Dimensions	18 ac	876 lf x lf	80 lf rock depth

RCC weather impact

Month	Lost Days	Prod Shfts
Jan	11	34
Feb	11	28
Mar	12	32
Apr	9	36
May	4	48
Jun	1	52
Jul	1	52
Aug	1	54
Sep	6	42
Oct	11	34
Nov	12	30
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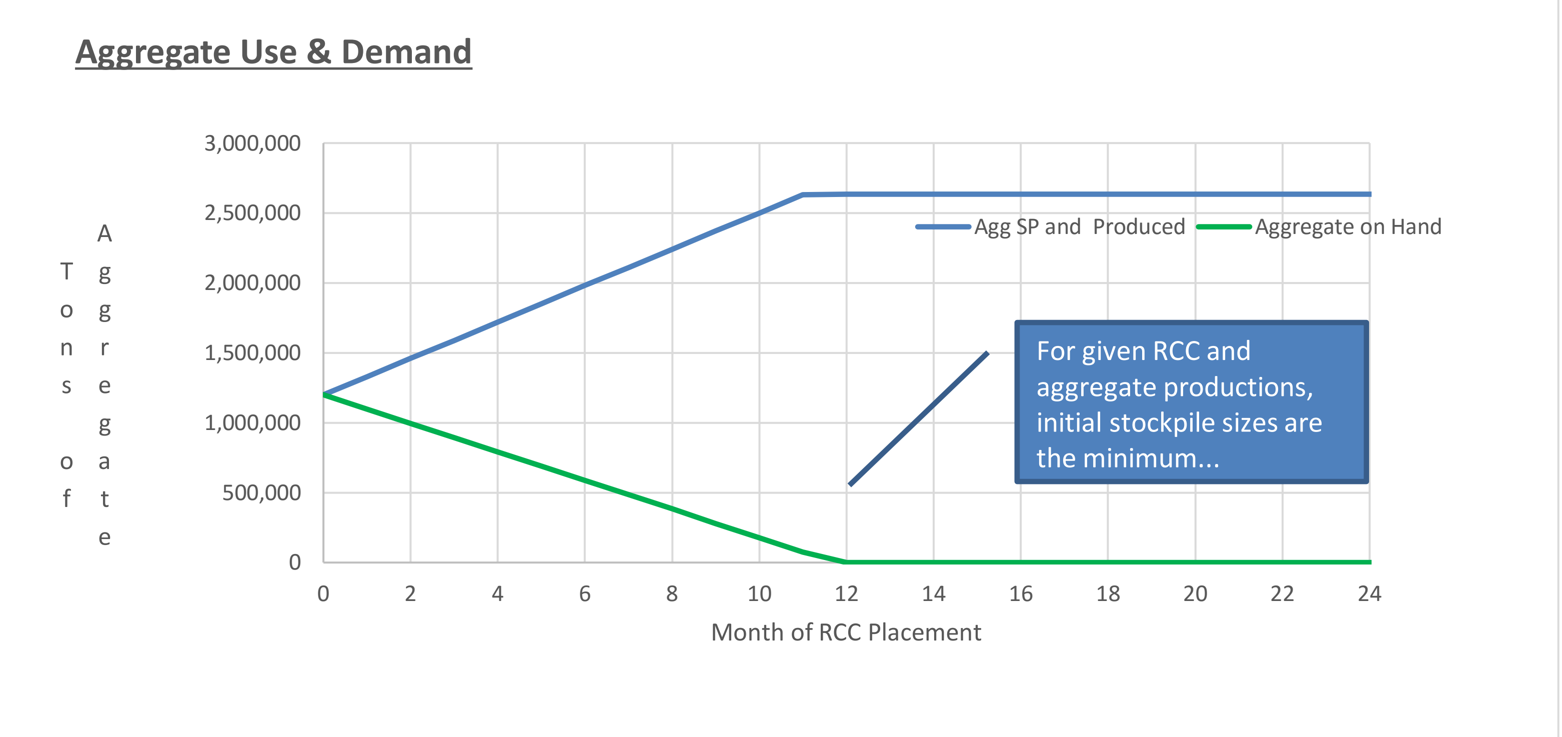
Totals	91	474
Effective shifts / week ⇨		9.1



HDR

Aggregate Use and Demand - Quarry 3 Rock Creek Basalt

Crushing Operations

Assumptions:		Effective Productions:	Single Op	All Ops Product	All Ops - Feed
Primary feed rate (say 36x48 @ 6"-8")	650 tph				
Primary's in operation	1	Hourly	550	550 tph	650 tph
% waste through eliminations or reject at primary	15%	Shift	4,400	4,400 tps	5,200 tps
% loss secondary processing or recomb % mismatch	0%	Daily	8,800	8,800 tpd	10,400 tpd
Hours per shift	10	Weekly	39,600	39,600 tpw	46,800 tpw
Productive (crushing) hours per shift	8.0	Monthly	139,000	139,000 tpm	164,000 tpm
Shifts per day	2	Annual	1,668,000	1,668,000 tpy	1,968,000 tpy
Worked shifts per week	9	Annualized/Mo	130,000	130,000 tpm	153,000 tpm
Weeks per month (10 weeks down per year)	3.50				
Months per year	12				
Stockpile Loss - all products avg	3%				

Concrete/RCC Operations

Assumptions for stand-alone plant operations:			Productions and Demand	cy	tons - agg product
RCC Mixer Capacity (say 6 cy @ 70sec cy)		309	Hourly - theoretical	1,234 cy/hr	2,204 tons /
Mixers in plant assembly		4	Hourly - plant max	864 cy/hr	1,543 tons /
Combined mixer efficiency within plant ops		70%	Max Shift	6,912 cy/sh	12,345 tons /
Hours per shift		10	Probable Max per Day	12,096 cy/dy	21,604 tons /
Productive (batching) hours per shift		8.0	Probable Max Week	60,480 cy/wk	108,018 tons /
Net effective shifts per day		1.75	Probable Max Month	211,680 cy/mo	378,065 tons /
Net effective days per week		5	Daily b4 weather average @ 60%	7,258 cy/dy	12,962 tons /
Net effective weeks per month		3.50	Non-weather days per year:	237 dy/yr	
Mix design aggregate tons per cy		1.70	Days required	203 dy	or 10 mo
RCC & conc batch waste %		2%	Use for average RCC/mo	130,000 cy/mo	232,183 tons /
Agg used for batching - tn/cy		1.73			
Agg needed including stockpile loss - tn/cy		1.79	tpcy ➔	2,634,380	tn of RCC agg product needed
Annual production capacity:	2,058 hr	1,778 k cy	10 mo	2,685,880	tn site-produced agg product needed

Note: Considers 1st to last lift productivity with various production control; RCC, forming, gallery, joints, split placements, etc.

Aggregate Demand:

Initial RCC Aggregate Stockpile 1,200,000 tons ➔ 9 months

Conc/RCC Placement 1,475,000 cy

Months of Placement 11.3 ➔ 130,000 cy/mo

RCC month	RCC Placed	Agg Used	Agg SP and Produced	Aggregate on Hand	Depletion per month
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Note: driven off of annualized monthly product; early peak RCC placement might necessitate higher agg production or larger initial stockpiling

Rough Stockpile sizing:

Assume 3 equal stockpiles	400,000	tons per stockpile	
at 40' h, 30° slopes, 1.4 tn/cy; 1 pile; with 75'	525 ' w	550 ' l	
between long piles and 50' on perimeter… ≈	1,825 ' w	650 ' l	28 acres

Driving Concrete Quantities

RCC Total[1]	1,475,000	cy	3,400 tn/cy ⇦ site-produced
Assumed Conv Conc Total	104,650	cy	3,200 tn/cy ⇦ commercial
Other Agg Products Total	50,000	tn	

1 - from RCC Dam Q-s & Placement Plan - R06.xls; dwgs - Chehalis_All_Figs_2016-10-19.pdf

Conventional Concrete Summary:	104,650
3.2 Plug	1,200
5.5 miscellaneous	750
5.6 un-reinforced mass; bedding, abut's, sluice cover	51,400
5.10 outlet and outlet shaft	5,800
5.17 Intake tower	14,000
5.18 Intake trashrack walls	2,000
6.1-2 flip bucket	17,500
7.4-5 Stilling basin	7,000
Unlisted	5,000

Rough Quarry Sizing:

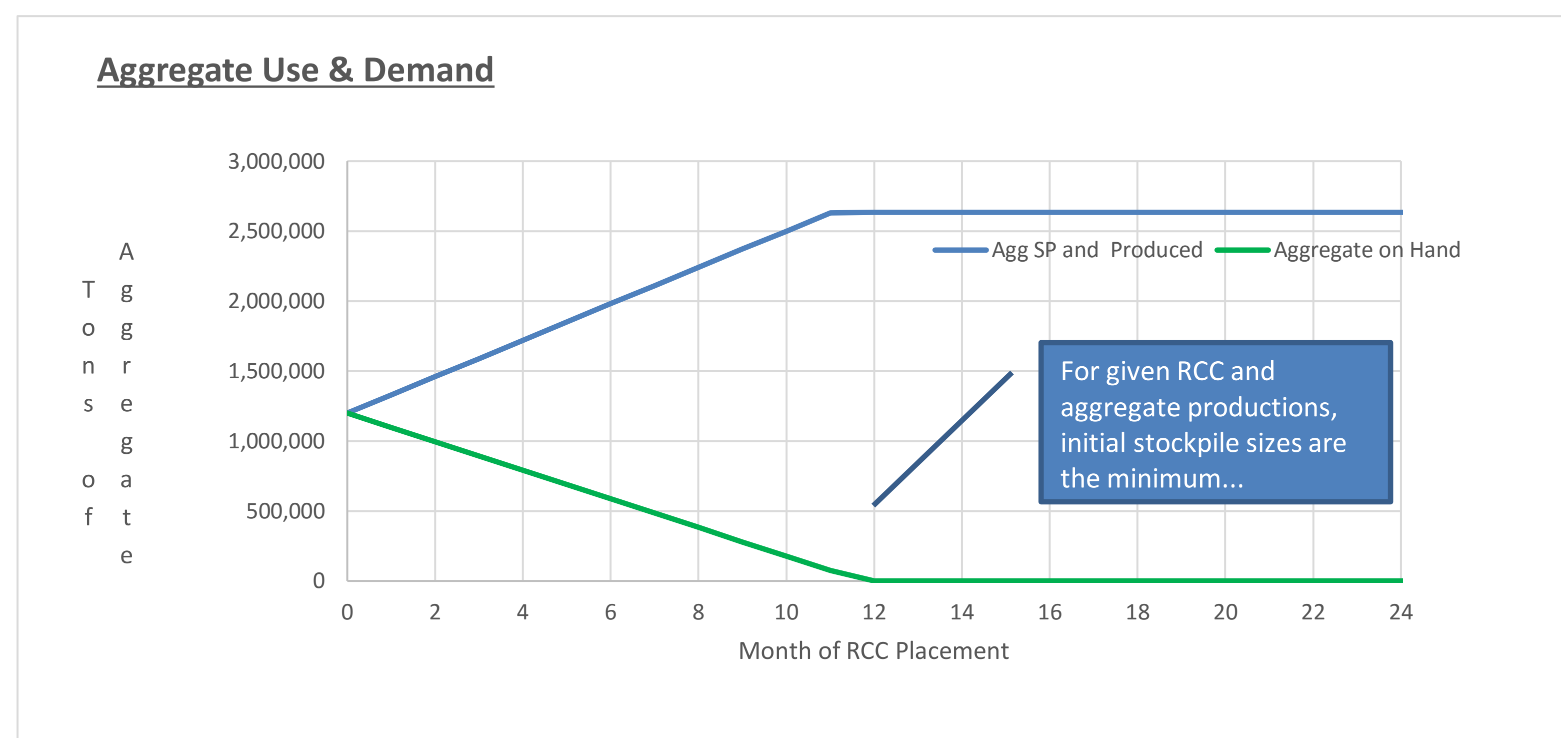
Rough Quarry Sizing:	
Site-produced RCC aggregate needed	2,634,380 tn
Site-produced concrete aggregates needed	tn
Other crushed/processed products	51,500 tn
Total process feed	3,088,761 tn
Quarry volume @ 1.7 ton product / cy shot	1,816,918 cy
Presumed quarry effective depth - ft	80
Raw quarry area - before access - sf	613,210
Quarry area increase for access/benching	30%
Quarry area to develop - sf	797,173
- acres	18.3
- if square - ftxft	893
Presumed stripping depth - ft	20
Overburden / stripping volume - cy	590,000
Quarry volume feed plus overburden - cy	2,406,918

Summary for Report

RCC	1,475,000 cy	11 months	134,091 cy/mo
Aggregate Product - tons	2,686,000 tn	21 months	130,000 tpm
Overburden: depth, cy	20 lf	590,000 cy	
Quarry Dimensions	18 ac	893 lf x lf	80 lf rock depth

RCC weather impact

Month	Lost Days	Prod Shfts
Jan	11	34
Feb	11	28
Mar	12	32
Apr	9	36
May	4	48
Jun	1	52
Jul	1	52
Aug	1	54
Sep	6	42
Oct	11	34
Nov	12	30
Dec	12	32
Totals	91	474
	Effective shifts / week ⇨	9.1



Chehalis Dam - Constructability, Schedule and Cost Support

RCC - Jugdment Level Cost Breakdown

Driving RCC Quantity 870,000 cy *(driven off of lift chart; RCC Dam Q-s & Placement Plan - R07.xls; dwgs - Chehalis_All_Figs_2016-10-19.pdf, checked this sheet ")*

RCC Unit Cost Development by Component

Cost Component	Qty Determination	Ref Qty	Priced Qty	Unit	Low	Likely	High	Likely Total	RCC Unit Contribution
Aggregate	3,550 #/cy w/1.05%		1,621,463	ton	12.00	14.50	17.50	$ 23,511,206	$ 27.02 /cy
Cement & FA	285 #/cy w/1.03%		127,694	ton	110.00	130.00	140.00	$ 16,600,253	$ 19.08 /cy
Other materials	RCC		870,000	cy	0.00	0.50	1.25	$ 435,000	$ 0.50 /cy
RCC test section	ls		1	ls	125,000	250,000	300,000	$ 250,000	$ 0.29 /cy
RCC plant & delivery mob-setup-demob	ls		1	ls	200,000	350,000	450,000	$ 350,000	$ 0.40 /cy
Mix	RCC		870,000	cy	5.50	6.00	6.50	$ 5,220,000	$ 6.00 /cy
Mix Cooling	250,000 cy @ 5-10 def F cooling		200,000	cy	0.00	3.00	5.00	$ 600,000	$ 0.69 /cy
Deliver	RCC		870,000	cy	4.00	4.50	5.50	$ 3,915,000	$ 4.50 /cy
Place	RCC		870,000	cy	4.00	4.50	5.50	$ 3,915,000	$ 4.50 /cy
Dam Joints	75 ft on center		313,200	sf	3.00	3.50	4.00	$ 1,096,200	$ 1.26 /cy
Bedding	12.5% of RCC @ 1/2in		4,531	cy	275.00	300.00	325.00	$ 1,359,375	$ 1.56 /cy
Facing forming (slope u/s, vert d/s)	full u/s & d/s from "RCC - Lift Chart"		449,611	sf	3.00	3.75	5.00	$ 1,686,041	$ 1.94 /cy
Facing options:									
Base - GERCC (all faces)	0.75 gal/sf of all faces	41,631 cy	1,667	cy	450.00	700.00	800.00	$ 1,167,214	$ 1.34 /cy
Option 1 - u/s conventional (@1/2 facing)	2.5 ft thick - on 1/2 formwork		20,815	cy	250.00	290.00	320.00	$ 6,036,444	$ 6.94 /cy
Option 1 - d/s GERCC face	0.75 gal/sf of 1/2 formwork		834	cy	650.00	700.00	800.00	$ 583,607	$ 0.67 /cy
Option 2 - Facing (u/s & d/s conventional)	2.5 ft thick - on all formwork		41,631	cy	250.00	290.00	320.00	$ 12,072,888	$ 13.88 /cy
GERCC - abutment contact	0.75 gal/sf of projected abut x 150%	13,778 cy	1,380	cy	650.00	700.00	800.00	$ 965,732	$ 1.11 /cy
Gallery	1330' + 2 adits @ 100'		1,530	lf	500.00	1000.00	1200.00	$ 1,530,000	$ 1.76 /cy
Other drain features	RCC		870,000	cy	0.25	0.75	1.25	$ 652,500	$ 0.75 /cy
Total with Base Facing (GERCC all faces)					$ 51,738,048	$ 63,253,522	$ 74,649,953		
					$ 59.47 /cy	$ 72.71 /cy	$ 85.80 /cy		$ 72.71 /cy
with contractor unallocated indirect expense (15%) and overhead & profit 15%)			25%		**$ 64,700,000**	**$ 79,100,000**	**$ 93,300,000**		
Total RCC - inclusive bid					**$74.37 /cy**	**$90.92 /cy**	**$107.24 /cy**		
Cementitious materials - bid			on 123,975 tn		**$141.63 /tn**	**$167.38 /tn**	**$180.25 /tn**		
Aggregate materials - bid			on 1,544,250 tn		**$15.75 /tn**	**$19.03 /tn**	**$22.97 /tn**		
Facing elements - form & place - bid			on 870,000 cy		**$2.41 /cy**	**$3.28 /cy**	**$4.12 /cy**		
Mix - Deliver - Place - Other - bid			on 870,000 cy		**$23.82 /cy**	**$30.01 /cy**	**$36.67 /cy**		
RCC - w/out cementitious - bid			on 870,000 cy		**$54.19 /cy**	**$67.07 /cy**	**$81.56 /cy**		
Check					**$64,700,000**	**$79,100,000**	**$93,300,000**		
Total with full conventional facings (Option 2)					$ 61,395,358	$ 74,159,195	$ 86,637,801		
					$70.57 /cy	$85.24 /cy	$99.58 /cy		$85.24 /cy
with contractor unallocated indirect expense (15%) and overhead & profit 15%)			25%		**$ 76,700,000**	**$ 92,700,000**	**$ 108,300,000**		
Contractor Bid Unit Prices					**$88.16 /cy**	**$106.55 /cy**	**$124.48 /cy**		
Contractor Bid Unit Prices (Option 1)					**$81.51 /cy**	**$98.72 /cy**	**$115.87 /cy**		

Comments:

For FRO, no planned or long-term storage; low range - 1) no cooling, 2) no admixture or other material, 3) reduced GERCC application or utilization, 4) potentially reduced gallery and drain features.

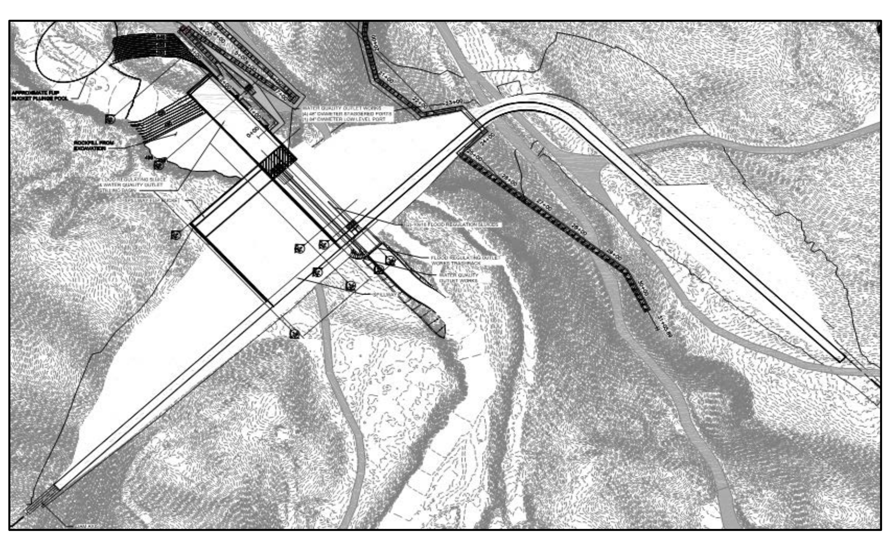
8-9 month window, potentially more flexibility to avoid hot weather; say 2 months @ 100kcy / mo = 200k cy

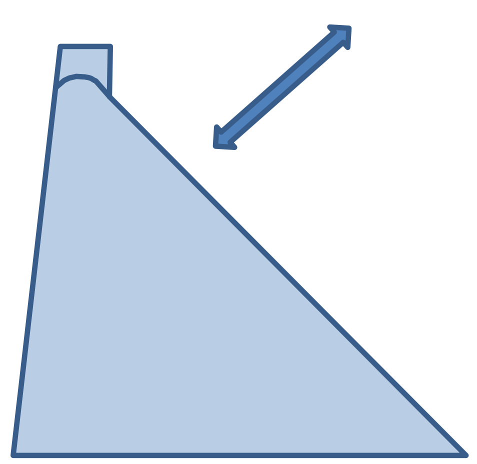
248,000 sf abutment contact x 1.5' thick

Use Base - all faces GERCC, as basis for estimate; FRO facing and water retention not as critical as with FRFA

Cost Development Qualifications

1) Cost judgement contemplates 2016 costs
2) The unit costs and cost ranges reflect industry experience and judgment, not developed estimates
3) The quantities reflect rough estimates of preliminary drawings, and anticipated design details
4) The low and high ranges intend to reflect design development and choices that might affect both the quantity and nature of the work, within the context of the given dam size. For example, while the low cement and pozzolan price may not be as low as shown, the quantity might be less.





Quantity Development - supporting parameters and numbers

	Max	NA	NA
Approx crest length	1,400		
Crest width	20.0		
Crest elev	651.0		
U/S work point elev	651.0		
Foundation elev	399.0		
U/S slope	0.1		
D/S slope	0.85		
Joint spacing	75		
Approx vertical profile sf	238,215		
Structure H	252	0	0
H of D/S chimney	23.5	#DIV/0!	#DIV/0!
Elev of D/S chimney break	627.5	#DIV/0!	#DIV/0!
Base width	239.4	0.0	0.0
Area - max	30,400 sf	#DIV/0!	#DIV/0!
Volume / lf of dam	1,126 cy	#DIV/0!	#DIV/0!
U/S face vertical length	253.3	0	0
U/S face length as % of H	100.5%	#DIV/0!	#DIV/0!
D/S face length	323	#DIV/0!	#DIV/0!
D/S face length as % of H	128.3%	#DIV/0!	#DIV/0!
full joint sf / cy of dam	0.36 sf/cy	#DIV/0!	#DIV/0!
Approx RCC Check	775 lf	30,400 full sf	880,000 cy

from Chehalis_All_Figs_2016-10-19.pdf; compare to lift length sum "RCC Dam Q-s & Placement Plan - R07.xls"

HDR

Chehalis Dam - Constructability, Schedule and Cost Support

RCC - Jugdment Level Cost Breakdown

Driving RCC Quantity *1,475,000 cy* *(driven off of lift chart; RCC Dam Q-s & Placement Plan - R07.xls; dwgs - Chehalis_All_Figs_2016-10-19.pdf, checked this sheet ")*

RCC Unit Cost Development by Component

Cost Component	Qty Determination	Ref Qty	Priced Qty	Unit	Low	Likely	High	Likely Total	RCC Unit Contribution
Aggregate	3,550 #/cy w/1.05%		2,749,031	ton	12.00	14.50	17.50	$ 39,860,953	$ 27.02 /cy
Cement & FA	285 #/cy w/1.03%		216,493	ton	110.00	130.00	140.00	$ 28,144,106	$ 19.08 /cy
Other materials	RCC		1,475,000	cy	0.75	1.00	1.25	$ 1,475,000	$ 1.00 /cy
RCC test section	ls		1	ls	125,000	250,000	300,000	$ 250,000	$ 0.17 /cy
RCC plant & delivery mob-setup-demob	ls		1	ls	300,000	400,000	500,000	$ 400,000	$ 0.27 /cy
Mix	RCC		1,475,000	cy	5.00	5.50	6.00	$ 8,112,500	$ 5.50 /cy
Mix Cooling	250,000 cy @ 5-10 def F cooling		390,000	cy	2.50	3.50	4.50	$ 1,365,000	$ 0.93 /cy
Deliver	RCC		1,475,000	cy	4.00	5.00	5.50	$ 7,375,000	$ 5.00 /cy
Place	RCC		1,475,000	cy	4.00	4.50	5.00	$ 6,637,500	$ 4.50 /cy
Dam Joints	75 ft on center		531,000	sf	2.75	3.00	3.50	$ 1,593,000	$ 1.08 /cy
Bedding	12.5% of RCC @ 1/2in		7,682	cy	275.00	300.00	325.00	$ 2,304,688	$ 1.56 /cy
Facing forming (slope u/s, vert d/s)	full u/s & d/s from "RCC - Lift Chart"		650,211	sf	3.00	3.75	5.00	$ 2,438,291	$ 1.65 /cy
Facing options:									
Base - GERCC (all faces)	0.75 gal/sf of all faces	60,205 cy	2,411	cy	650.00	700.00	800.00	$ 1,687,983	$ 1.14 /cy
Option 1 - u/s conventional (@1/2 facing)	2.5 ft thick - on 1/2 formwork		30,102	cy	250.00	290.00	320.00	$ 8,729,685	$ 5.92 /cy
Option 1 - d/s GERCC face	0.75 gal/sf of 1/2 formwork		1,206	cy	650.00	700.00	800.00	$ 843,991	$ 0.57 /cy
Option 2 - Facing (u/s & d/s conventional)	2.5 ft thick - on all formwork		60,205	cy	250.00	290.00	320.00	$ 17,459,369	$ 11.84 /cy
GERCC - abutment contact	0.75 gal/sf of projected abut x 150%	17,778 cy	1,780	cy	650.00	700.00	800.00	$ 1,246,106	$ 0.84 /cy
Gallery	1,600' + 2 adits @ 50'+ 1 @ 180'		1,880	lf	800.00	900.00	1100.00	$ 1,692,000	$ 1.15 /cy
Other drain features	RCC		1,475,000	cy	0.50	0.75	1.25	$ 1,106,250	$ 0.75 /cy
Total with Base Facing (GERCC all faces)					$ 88,973,393	$ 105,688,377	$ 122,024,628		
					$ 60.32 /cy	$ 71.65 /cy	$ 82.73 /cy		$ 71.65 /cy
with contractor unallocated indirect expense (15%) and overhead & profit 15%)			25%		**$ 111,200,000**	**$ 132,100,000**	**$ 152,500,000**		
Total RCC - inclusive bid					**$75.39 /cy**	**$89.56 /cy**	**$103.39 /cy**		
Cementitious materials - bid			on 210,188 tn		**$141.63 /tn**	**$167.38 /tn**	**$180.25 /tn**		
Aggregate materials - bid			on 2,618,125 tn		**$15.75 /tn**	**$19.03 /tn**	**$22.97 /tn**		
Facing elements - form & place - bid			on 1,475,000 cy		**$2.39 /cy**	**$2.80 /cy**	**$3.51 /cy**		
Mix - Deliver - Place - Other - bid			on 1,475,000 cy		**$24.87 /cy**	**$29.13 /cy**	**$33.42 /cy**		
RCC - w/out cementitious - bid			on 1,475,000 cy		**$55.21 /cy**	**$65.71 /cy**	**$77.70 /cy**		
Check					**$111,200,000**	**$132,100,000**	**$152,500,000**		
Total with full conventional facings (Option 2)					$ 102,457,161	$ 121,459,763	$ 139,361,016		
					$69.46 /cy	$82.35 /cy	$94.48 /cy		$82.35 /cy
with contractor unallocated indirect expense (15%) and overhead & profit 15%)			25%		**$ 128,100,000**	**$ 151,800,000**	**$ 174,200,000**		
Contractor Bid Unit Prices					**$86.85 /cy**	**$102.92 /cy**	**$118.10 /cy**		
Contractor Bid Unit Prices (Option 1)					**$81.11 /cy**	**$96.25 /cy**	**$110.76 /cy**		

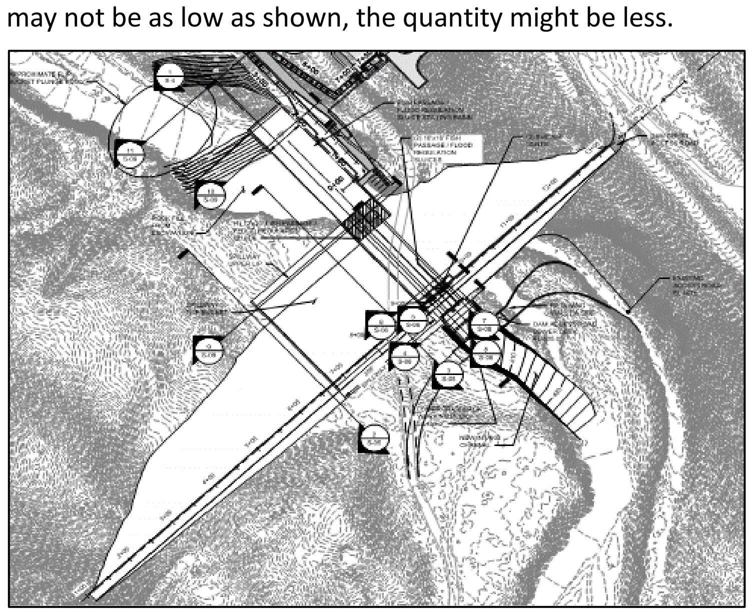
Comment: 3 months @ 130kcy / mo = 390k cy

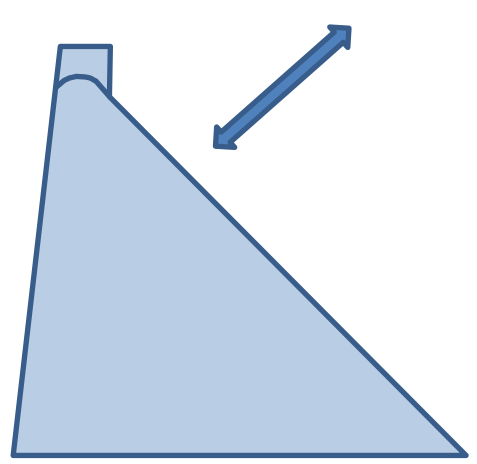
Comment: 320,000 sf abutment contact x 1.5' thick

Note: Use Option 1) u/s conventional & d/s GERCC, as basis for estimate

Cost Development Qualifications

1) Cost judgement contemplates 2016 costs
2) The unit costs and cost ranges reflect industry experience and judgment, not developed estimates
3) The quantities reflect rough estimates of preliminary drawings, and anticipated design details
4) The low and high ranges intend to reflect design development and choices that might affect both the quantity and nature of the work, within the context of the given dam size. For example, while the low cement and pozzolan price may not be as low as shown, the quantity might be less.



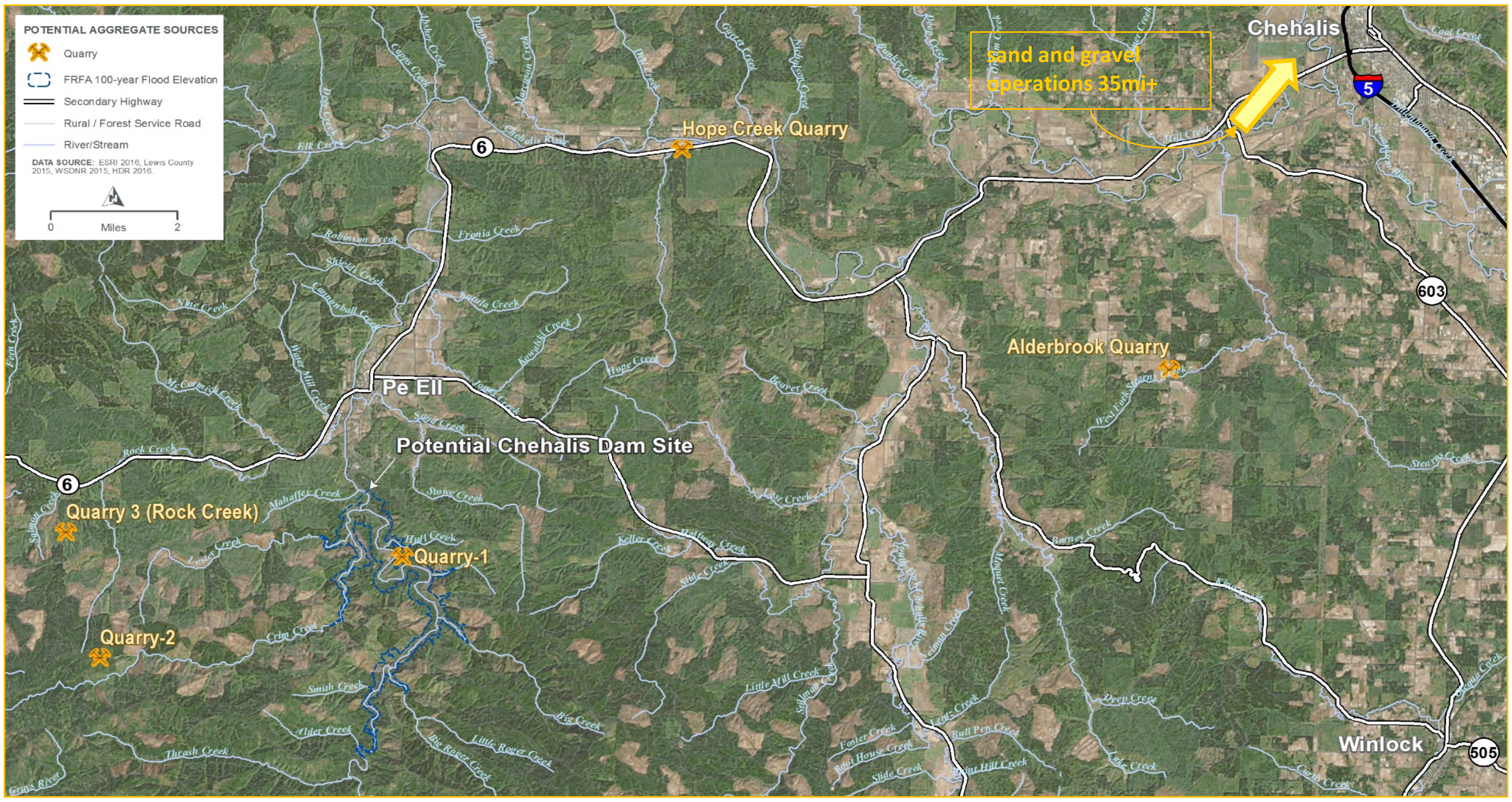


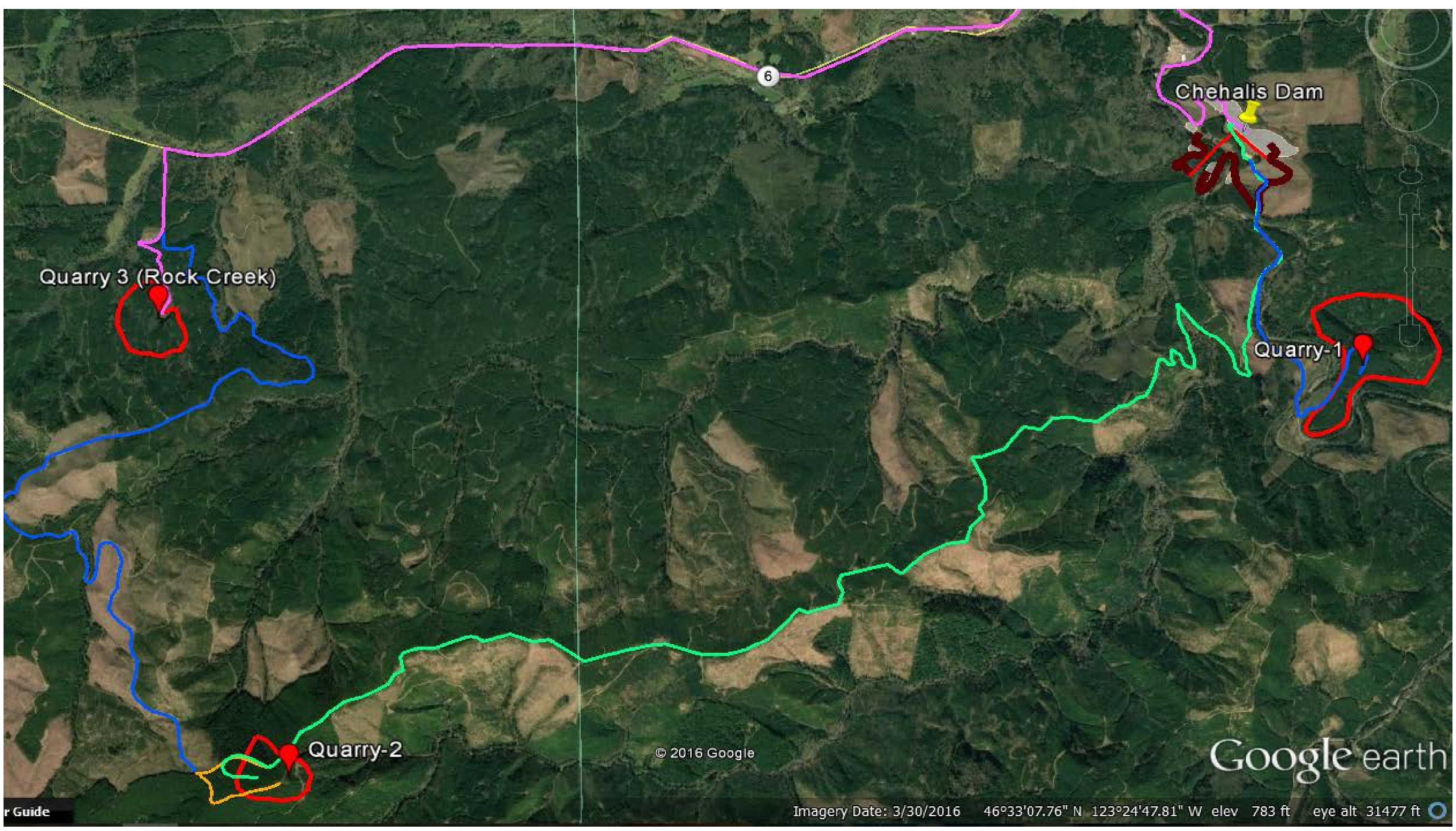
Quantity Development - supporting parameters and numbers

	Max	NA	NA
Approx crest length	1,650		
Crest width	20.0		
Crest elev	710.0		
U/S work point elev	710.0		
Foundation elev	399.0		
U/S slope	0.1		
D/S slope	0.85		
Joint spacing	75		
Approx vertical profile sf	335,537		
Structure H	311	0	0
H of D/S chimney	23.5	#DIV/0!	#DIV/0!
Elev of D/S chimney break	686.5	#DIV/0!	#DIV/0!
Base width	295.5	0.0	0.0
Area - max	46,178 sf	#DIV/0!	#DIV/0!
Volume / lf of dam	1,710 cy	#DIV/0!	#DIV/0!
U/S face vertical length	312.6	0	0
U/S face length as % of H	100.5%	#DIV/0!	#DIV/0!
D/S face length	401	#DIV/0!	#DIV/0!
D/S face length as % of H	128.9%	#DIV/0!	#DIV/0!
full joint sf / cy of dam	0.36 sf/cy	#DIV/0!	#DIV/0!
Approx RCC Check	890 lf	46,178 full sf	1,530,000 cy

Comment: **from Chehalis_All_Figs_2016-10-19.pdf; compare to lift length sum "RCC Dam Q-s & Placement Plan - R07.xls"**

Scoggins Dam - Constructability, Schedule and Cost Support
Select Aggregate and Quarry Locations





Attachment D
RCC Placement Summary

Chehalis Dam
Concept-Level RCC Placement Analysis

Attachment D; 1 page

FRFA and FRO Dam Placement & Production Summary

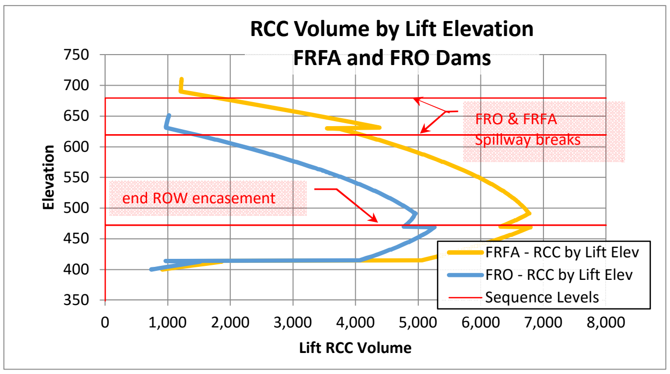
Development Assumptions

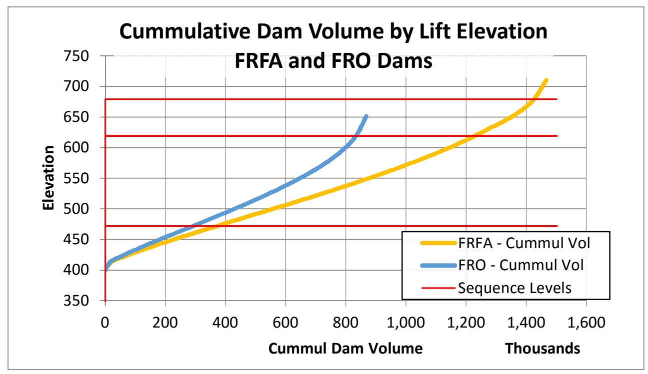
Dam volume developed horizontally on 1 foot increments; Profiles and cross sections from "Chehalis_All_Figs_2016-10-19.pdf" drawings, slightly preceding CDR set.
Cross section assumptions; 3" abutment contact; GERCC facing elements upstream and downstream; RCC for GERCC is in dam quantity.
Setup and downtime for major delivery reconfigurations and other no-work periods not included in placement months.
Sustainable plant and delivery system capacity: 750 cy/hr 16 hr/dy 12,000 cy/dy Vertical placement zones drive production at 15% to 95% of capacity.
Dam placement noteable breaks: 399 start; 407 thru 442 & 472 (top of sluice, ROW encasements); 619 FRO spillway break; 627 FRFA wing start; 679 FRFA spillway break.
Judgment production reduction factors A (95%) thru G (15%); considering start up; lift size; lift length; amount of form/face; gallery, split placements; narrow top lifts.
Placement duration expanded to reflect annual average of 19, rather than 26 or 30 available placement days per month.
For feasibilitiy level analysis no RCC toe area or apron treatments have been considered. The flip bucket foundation prism is incorporated, vertical to el 469.

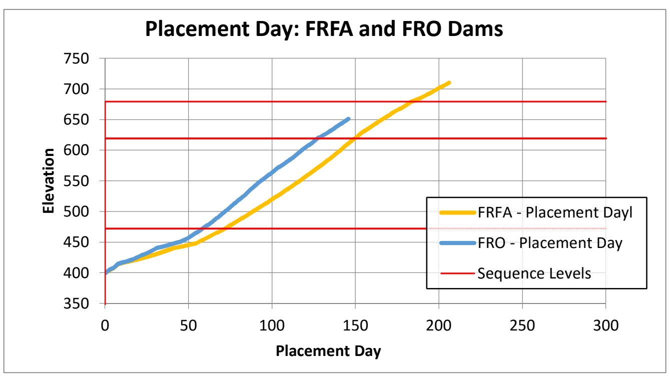
Key Observations

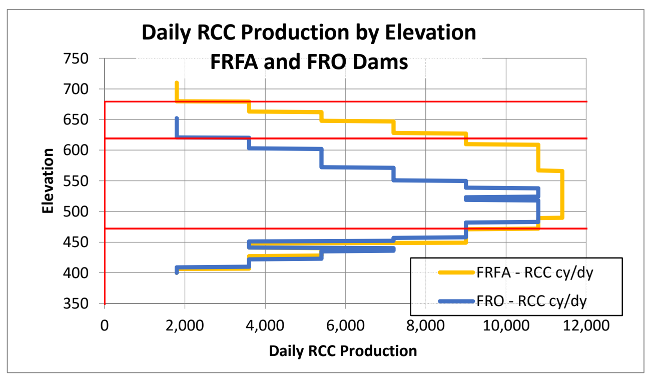
For the FRO dam, about 40% of the lifts are less than 3,500 cy with the max being 5,300 cy. For the FRFA dam, about half the lifts are smaller than 5,500 cy, non larger than 7,000cy.
The larger lifts reside FRFA elevations 415-620, and FRO 415-590, with plant capacity always greater than a single lift volume plant/delivery capacity generally controls production.
Controlling factors outside of these zones: split placements, gallery and drain drilling interference, narrowing lifts; giving way to intense formwork and GERCC placements.
FRFA faces and narrowing lifts will begin controling production above, i.e. elev. 600; 1200' L x 100' w lifts, approx 80% of the volume is complete but only 50% the facing formwork.
FRO faces and narrowing lifts will begin controling production above, i.e. elev. 560; 1080' L x 85' w lifts, approx 80% of the volume is complete but only 53% the facing formwork.
Large dam productions have demonstrated highly variable monthly production, with peaks often 150% to approaching 300% of monthly average.

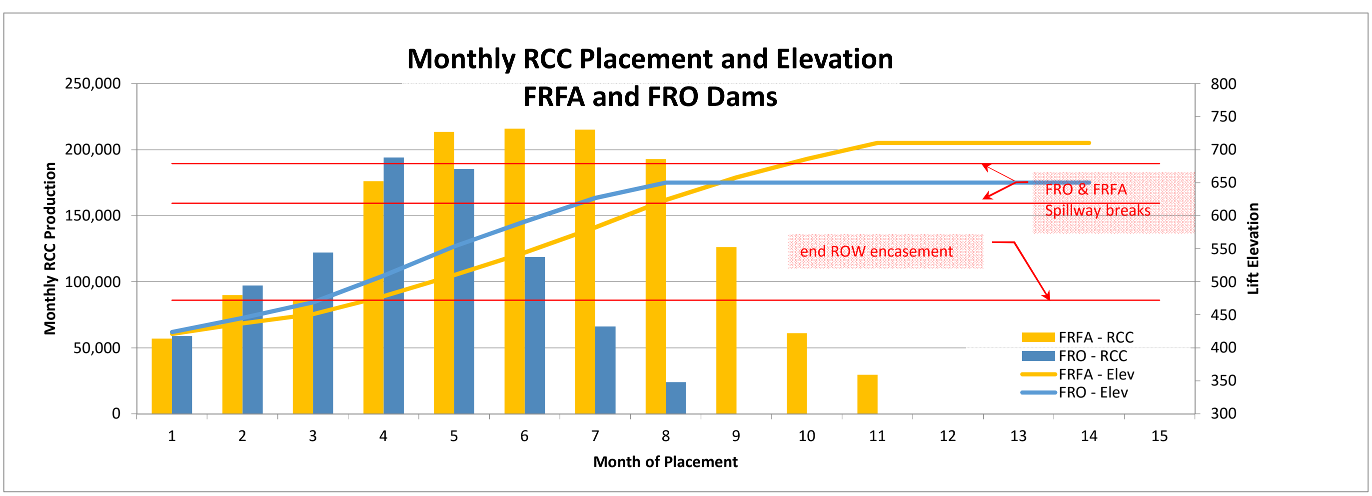
As developed FRFA:	216,000 cy/mo peak	133,000 cy/mo avg	⇦ for comparison: Olivenhain - 1.4 Mcy 308' high @ 294 kcy/mo peak vs 159kcy/mo avg; Upper
As developed FRO:	194,000 cy/mo peak	108,000 cy/mo avg	Stillwater - 1.5 Mcy 299' high @ 267 kcy/mo peak vs 164 kcy/mo avg;











Appendix G
Fish Passage Alternative Concept Design

Chehalis Basin Strategy

Appendix G

— Fish Passage Alternative Concept Design —

Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

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Attachment E Floating Surface Collector Technical Memorandum

ACRONYMS AND ABBREVIATIONS LIST

AACE	American Association of Cost Engineering
AWS	auxiliary water supply
CBS	Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project
cfs	cubic feet per second
CHTR	collect, handle, transfer, and release
CMU	concrete masonry unit
EDT	Ecosystem Diagnosis and Treatment
FGS	fish guidance system
Flood Authority	Chehalis River Basin Flood Authority
fps	feet per second
FRFA	Flood Retention Flow Augmentation
FRO	Flood Retention Only
FSC	Floating Surface Collector
FTE	full time equivalent
GPM	gallons per minute
HEC-RAS	Hydrologic Engineering Center - River Analysis System
LWM	large woody material
NMFS	National Marine Fisheries Service (a.k.a – NOAA Fisheries)
NOAA	National Oceanic and Atmospheric Administration
O&M	operations and maintenance
OFM	Office of Financial Management
OPCC	Opinion of Probable Construction Cost
PEIS	Programmatic Environmental Impact Statement
PLC	Programmable Logic Controller

PMF	Probable Maximum Flood
RCW	Revised Code of Washington
SR	State Route
TM	Technical Memorandum
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
VFD	Variable frequency drive
WDFW	Washington Department of Fish and Wildlife
Work Group	Chehalis Basin Workgroup
WDOE	Washington Department of Ecology
WSDOT	Washington State Department of Transportation
WSE	Watershed Science and Engineering
WSEL	water surface elevation

EXECUTIVE SUMMARY

The Chehalis Basin (basin) has historically been prone to flooding. The economic damages of the 2007 flood alone were estimated at more than $900 million, with one-third of that damage coming from disruption and damage to the transportation system, including Interstate 5 (I-5), other state highways, and rail lines. In 2011, the Washington State Legislature required the Office of Financial Management to prepare a report on alternative flood damage reduction projects and – in coordination with tribal governments, local governments, and state and federal agencies — to recommend priority flood hazard mitigation projects for continued feasibility assessment and design work. In response to this recommendation, The Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project was formed to evaluate the feasibility of mitigating flood hazards within the basin while exploring opportunities to enhance ecological conditions, aquatic habitat, and the abundance of fish in the basin. Along with several other options, considered either independently or in combination, a floodwater retention concept consisting of two potential dam alternatives on the upper Chehalis River was advanced through the conceptual design phase. The initial layout, technical feasibility, and Opinion of Probable Construction Cost of alternate dam and fish passage configurations at the proposed dam site were documented in a Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR, 2014a).

Introduction

A fish passage design study team comprising engineers and biologists from HDR Engineering, Inc. and Anchor QEA has been tasked to evaluate and further develop the fish passage alternatives documented in HDR, 2014a. The study includes refinement of design criteria, further concept level design development, performance assessment, and evaluation of costs for potential fish passage facilities that could accommodate passage of upstream and downstream migrating fish species, should such a flood damage reduction structure be built. These activities were performed in collaboration with members of the Flood Damage Reduction Technical committee and in concert with numerous other physical and biological studies being performed by others to evaluate potential flood damage reduction and aquatic species enhancement strategies.

This report presents refined fish passage design criteria, updated fish facility concepts, and anticipated fish passage performance for two potential flood damage reduction structure alternatives being developed by the dam design study team: the Flood Retention Only (FRO) and Flood Retention Flow Augmentation (FRFA) structural alternatives. This information is intended to be used by the Chehalis Basin Strategy Work Group to inform decisions regarding the integration and performance of potential fish passage technologies with flood damage reduction structural alternatives being developed by the dam design study team.

Refinement of Design Criteria

The fish passage design team and members of the Chehalis Basin Strategy – Flood Damage Reduction Technical Committee carried out several fish passage subcommittee meetings throughout development of this study. Of primary importance at these meetings were the sharing of new information and the discussion, interpretation, and formulation of design criteria that would guide the refinement of potential fish passage concepts.

Concurrently WDFW led an extensive field sampling program to collect data and better understand the phenology, abundance, habitat requirements, distribution, and migration patterns of fish present within the Chehalis River and more specifically within the inundation limits of the proposed dam inundation area. Using new and available historic data, WDFW has assisted the fish passage design team with biological fish criteria development in collaboration with other participating technical committee stakeholders. These criteria and their importance included:

- Fish occurrence and distribution: Informs the selection of species and life stages targeted for fish passage design.
- Fish migration timing: Informs the understanding of seasonality, anticipated hydrologic conditions, and duration of periods when target fish species may be expected to migrate upstream and/or downstream of the dam location.
- Fish abundance: Informs the estimation of the annual number of fish that require passage as well as the peak daily rate of migration that influences facility size and operation requirements.

Fish Passage Alternatives

Potentially viable fish passage alternatives evaluated in the previous biennium (HDR, 2014a) that appeared to have the highest likelihood of meeting the project objectives were advanced as part of this study effort. Alternatives considered and developed to a concept level of design include:

- A series of fish passage conduits for the FRO dam alternative that would allow both upstream and downstream volitional passage of all target species and life stages during run-of-river dam operations.
- A collect, handle, transfer, and release facility (trap and transport) for the FRO dam that would provide upstream passage of all target species and life stages during flood operations.
- A technical fish ladder for the FRFA dam alternative that would allow upstream volitional passage for adult salmonids.
- A collect, handle, transfer, and release facility (trap and transport) for the FRFA dam that would provide upstream passage of all target species and lifestages.
- A floating surface collector for the FRFA dam alternative that would provide downstream passage of post-spawn adult steelhead, juvenile salmonids, and resident fish species.

- A fixed multi-port collector and pressurized bypass for the FRFA dam alternative that would provide downstream passage of post-spawn adult steelhead, juvenile salmonids, and resident fish species.

The FRO and FRFA dams are required to provide provisions for both upstream and downstream fish and lamprey passage. While each fish passage alternative has been developed individually, upstream and downstream alternatives are paired with one another to provide a system of fish passage facilities that can accommodate project goals, given a proposed impoundment structure. Only one passage combination was developed for the FRO dam, by decision of the Fish Passage Subcommittee. The FRO passage combination provides upstream and downstream passage primarily through the FRO conduits. Upstream passage is provided via the CTHR facility during flood operations and associated impoundment periods. No downstream passage facilities are provided during impoundment periods and downstream passage is delayed until the latter portions of the reservoir drawdown period. Two upstream and two downstream fish passage alternatives are provided for the FRFA dam, resulting in four possible passage combinations as follows:

- Upstream Fish Passage: CHTR Facility
  Downstream Fish Passage: Fixed Multi-Port Collector
- Upstream Fish Passage: CHTR Facility
  Downstream Fish Passage: Floating Surface Collector
- Upstream Fish Passage: Technical Fish Ladder
  Downstream Fish Passage: Fixed Multi-Port Collector
- Upstream Fish Passage: Technical Fish Ladder
  Downstream Fish Passage: Floating Surface Collector

Comparison of Alternatives

The refined alternatives and combinations of alternatives were evaluated against factors that served to distinguish each alternative from one another and gage how well each alternative met the project objectives and priority values voiced by participants of the Flood Reduction Technical Committee. For this study, four factors were selected as follows: anticipated performance and survival, reliability, construction costs, operation and maintenance (O&M) costs. The intent of anticipated performance and survival was to inform other commensurate studies of the potential influence that alternative implementation may have on existing fish populations or future recovery efforts. Construction costs and O&M costs were selected over total lifecycle costs due to the anticipated availability and source of capital construction funds versus annual operating expenses.

The FRO Conduits are anticipated to have a greater level of performance and total survival for every species and life stage with the least associated capital and O&M costs. However, it was required that a CHTR facility be integrated into the project to provide upstream fish passage when fish passage is not possible through the conduits - during flood control operations. Downstream passage for the FRO dam is only provided via the conduits when flood control operations are not occurring. During flood control operations, fish are accumulated in the inundation area and are not allowed to move downstream until normal, run-of-river operation resume. Upstream passage performance for the FRO conduits is anticipated to range from 59% to 96% accounting for the full range of target species and life stages. Downstream passage performance is anticipated to range from 74% to 95% for the full range of target species and life stages. When flood operations are in progress, the CHTR facility is anticipated to have upstream passage performance of 54% for resident and juvenile salmonid fish species and up to 91% for adult salmonids. During this phase of evaluation, upstream lamprey passage is not accommodated during CHTR operations and therefore, would be allowed to pass upstream through the FRO conduits after run-of-river operations resumed.

Upstream and downstream fish passage combinations were also evaluated and compared for the FRFA dam alternative. Overall, a combination including a CHTR facility for upstream passage and a Floating Surface Collector (FSC) for downstream passage appeared to provide the greatest level of performance and reliability, given the anticipated level of anticipated construction and O&M costs. The CHTR facility is identical to the CHTR facility evaluated for the FRO dam alternative and would therefore perform similarly. The FSC is anticipated to perform at a 64% total performance and survival rate for downstream migrating juvenile salmonids and resident fish species. Performance is anticipated to reduce to approximately 59% and 45% for downstream migrating adult residents and post-spawn adult steelhead respectively. As with the CHTR facility, there were uncertainties associated with the downstream passage of lamprey through the FSC alternative.

Construction and O&M Costs

The estimated construction cost for integration of the fish passage conduits in the FRO dam is integral to dam construction and is therefore not evaluated in this document. Construction costs for the CHTR facility required to provide upstream fish passage during FRO flood operations is estimated to range between $13.8 and $27.6 million. The estimated O&M cost for fish passage at the FRO dam is estimated to be $20,000 annually. This low annual O&M cost is attributed to the episodic level of operation anticipated only when flood operations occur (estimated to be once in every 5 to 7 years).

Combined construction costs for the upstream and downstream fish passage combination including the CHTR facility and FSC is estimated to range from $75.8 to $151.5 million. The estimated O&M cost for both facilities is anticipated to be on the order of $1,233,000. The higher O&M cost for fish passage is attributed to the high level of effort and resources required for facilities that would operate almost year-round.

Next Steps

Data and analysis from multiple reports and studies associated with the Chehalis Basin Strategy, including the Draft Programmatic Environmental Impact Statement (PEIS), the Draft Combined Dam and Fish Passage Conceptual Design Report, and this report, has been provided to the Fish Passage Technical Subcommittee and their parent organizations; the Flood Damage Reduction Committee and Chehalis Basin Strategy Work Group. Upon resolution of any remaining questions, information needs, concerns, and/or comments, a preferred alternative or combination of preferred alternatives can be selected for the FRO and FRFA dam options.

The Fish Passage Technical Subcommittee recognized in previous subcommittee discussions that some efficiencies may be gained by moving forward with a fish passage alternative prior to selection by Washington State if an alternative is likely to be selected. The subcommittee identified one such alternative – the CHTR facility. The CHTR facility is common to both the FRO and FRFA dam options, has a much higher level of total survival, is believed to provide more reliable passage with less uncertainty, and requires less capital construction cost than other upstream fish passage options. For these reasons, the Fish Passage Technical Subcommittee has chosen to proceed with development of the portions of the CHTR facility that would be common to both the FRO and FRFA dam options. Advancing this concept forward will also serve to maintain communication among participants of the Fish Passage Subcommittee and help resolve ongoing discussions relative to target species and important biological goals for the project. Results of concept development for the CHTR facility will be provided in future fish passage study documentation.

1 INTRODUCTION

The fish passage design study team comprising engineers and biologists from HDR Engineering, Inc. and Anchor QEA have been tasked to further develop and evaluate fish passage previously identified concepts for a potential flood damage reduction structure located near Pe Ell, Washington as part of the Chehalis Basin Flood Strategy Project. The study includes refinement of design criteria, further concept level design development, performance assessment, and evaluation of costs for potential fish passage facilities that could accommodate passage of upstream and downstream migrating fish species, should such a flood damage reduction structure be built. These activities were performed in collaboration with members of the Flood Damage Reduction Technical committee and in concert with numerous other physical and biological studies being performed by others to evaluate potential flood damage reduction and aquatic species enhancement strategies.

This report presents refined design criteria, updated fish facility concepts, and anticipated fish passage performance for two potential flood damage reduction structure alternatives being developed by the dam design study team: the Flood Retention Only (FRO) and Flood Retention Flow Augmentation (FRFA) structural alternatives. Detailed information relative to each structural alternative can be found in the Combined Dam and Fish Passage Concept Design (HDR, 2016b).

1.1 Project Background

The Chehalis Basin (basin) has historically been prone to flooding. The economic damages of the 2007 flood alone were estimated at more than $900 million, with one-third of that damage coming from disruption and damage to the transportation system, including Interstate 5 (I-5), other state highways, and rail lines. Many different flood hazard mitigation projects and approaches have been proposed and studied in response to the major floods in the Chehalis Basin. After the 2007 flood, the Chehalis River Basin Flood Authority (Flood Authority) was created to focus on developing flood hazard mitigation measures throughout the basin and to identify and implement flood damage reduction projects. The Flood Authority has been studying water retention in the upper Chehalis River Basin along with smaller flood hazard mitigation projects in the lower portion of the basin.

In 2011, the Washington State Legislature required the Office of Financial Management (OFM) to prepare a report on alternative flood damage reduction projects and – in coordination with tribal governments, local governments, and state and federal agencies — to recommend priority flood hazard mitigation projects for continued feasibility assessment and design work. In response to the legislative direction, the Ruckelshaus Center published the Chehalis Basin Flood Hazard Mitigation Alternatives Report in December 2012. That report compiled existing information on the potential flood hazard mitigation projects that seemed of most interest to basin leaders and decision makers at that time.

Potential flood hazard mitigation benefits, adverse impacts, costs, and implementation issues were summarized for each project to the degree that such information was available. Along with that effort, the Chehalis Basin Work Group (Work Group), composed of Chehalis Basin leaders, recommended to then Washington Governor Christine Gregoire a series of actions that, taken together, would represent a significant investment to reduce flood damage, enhance natural floodplain function and fisheries, and put basin leaders on firm footing to make critical decisions about large-scale projects. The Work Group recognized that habitat loss in the basin has contributed to a reduction in native fish populations and set the goal to develop a basin wide strategy to integrate flood damage reduction and environmental enhancement.

The Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project (CBS Project) is evaluating the feasibility of mitigating flood hazards within the basin while exploring opportunities to enhance ecological conditions, aquatic habitat, and the abundance of fish in the basin. The scope of the Project has included studying alternative water retention structures (dams), options for protecting I-5 and other floodplain at-risk facilities and structures with or without a dam, and other small flood reduction projects throughout the basin. As this project has proceeded, viable options to accomplish project objectives have narrowed as a result of analyses and evaluations conducted by the project team. Along with several other options, considered either independently or in combination, the floodwater retention concept consisting of two potential dam alternatives on the upper Chehalis River has been advanced through the conceptual design phase.

The initial layout, technical feasibility, and Opinion of Probable Construction Cost (OPCC) of multiple alternate dam and fish passage configurations at the proposed dam site were documented in HDR's 2014 Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR, 2014a). The memorandum also provided recommendations on additional site characterization and engineering evaluations that would be required to reduce design uncertainty, refine estimated project costs, and support selection of a preferred alternative. An initial site investigation program was completed in early 2015. Results were documented by HDR and Shannon & Wilson in a Phase 1 Site Characterization Technical Memorandum (TM), dated August of 2015. Additional site characterization (Phase 2) and engineering evaluations were completed in 2015 and 2016 by Shannon & Wilson. Additional hydrologic and hydraulic evaluations have been performed by Watershed Science and Engineering (WSE).

1.2 Proposed Dam Location and Size

The proposed dam site is located south of State Route (SR)-6 in Lewis County, Washington, on the main stem of the Chehalis River, about 1 mile south of Pe Ell (the southwest corner of Section 3, Township 12N, Range 5W). This site was selected from several alternative locations identified and evaluated in previous studies (S&W, 2009a; 2009b). The design storage volumes and corresponding estimated water storage elevations for the Flood Retention Only (FRO) and Flood Retention Flow Augmentation (FRFA) configurations presented in this document are summarized in Table 1-1. See also Appendix A of the

Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017) for the proposed dam location and schematics of the FRO and FRFA dam alternatives. The storage volumes and corresponding dam heights and inundation areas are subject to change as climate change and operation studies advance through the planning process. Further evaluation of alternate dam sites is not included in the current study.

Figure 1-1 shows the inundation limits for the FRO and FRFA dam alternatives. Figure 1-2 and Figure 1-3 show renderings of the FRO dam. Figure 1-4 and Figure 1-5 show renderings of the FRFA dam.

Table 1-1
Summary of Dam Characteristics

CHARACTERISTIC	FRO DAM	FRFA DAM
Structural Height	254	313
Crest Length	1,550	2,390
Crest Elevation	651	710
Spillway Length	208	208
Spillway Crest Elevation	628	687
Primary Conduit/Outlet Size	12x20	(4) 4-ft dia, (1) 7-ft dia
Primary Conduit/Outlet Invert Elevation	408	530, 560, 590, 620; 500
Secondary Conduit/Sluice Size	10x16	10x16
Secondary Conduit/Sluice Invert Elevation	411	420
Minimum Estimated Reservoir WSEL (drought)	-	520
Normal Operating Reservoir WSEL Range	-	588 - 628
Water Storage Volume	-	65,000 ac-ft
Flood Storage Volume	65,000 ac-ft	65,000 ac-ft
Maximum Total Storage Volume	65,000 ac-ft	130,000 ac-ft
Probable Maximum Flood (PMF) Reservoir WSEL	650	709
Maximum Spillway Capacity (PMF Flow)	75,000 cfs	75,000 cfs

Notes:
1. All values are in feet unless otherwise noted.
2. Maximum flood storage volumes and elevations are to spillway crest and do not include flood routing capacity between the design flood (100-year event) and the Probable Maximum Flood (PMF).
3. Maximum Spillway Capacity is equivalent to the PMF.

Figure 1-1
Inundation limits for the FRO and FRFA dam alternatives.

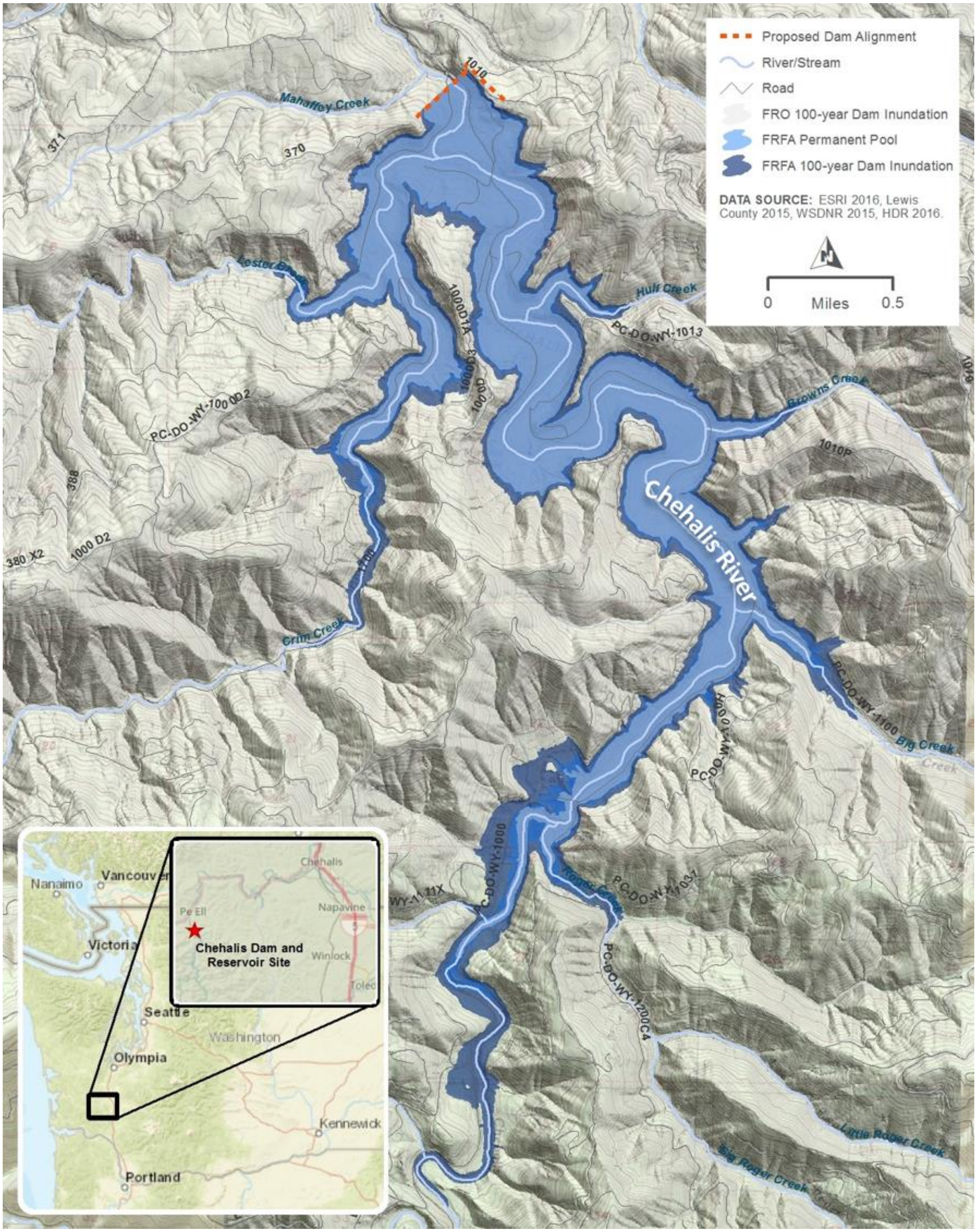


Figure 1-2
FRO Dam Rendering, Looking Upstream

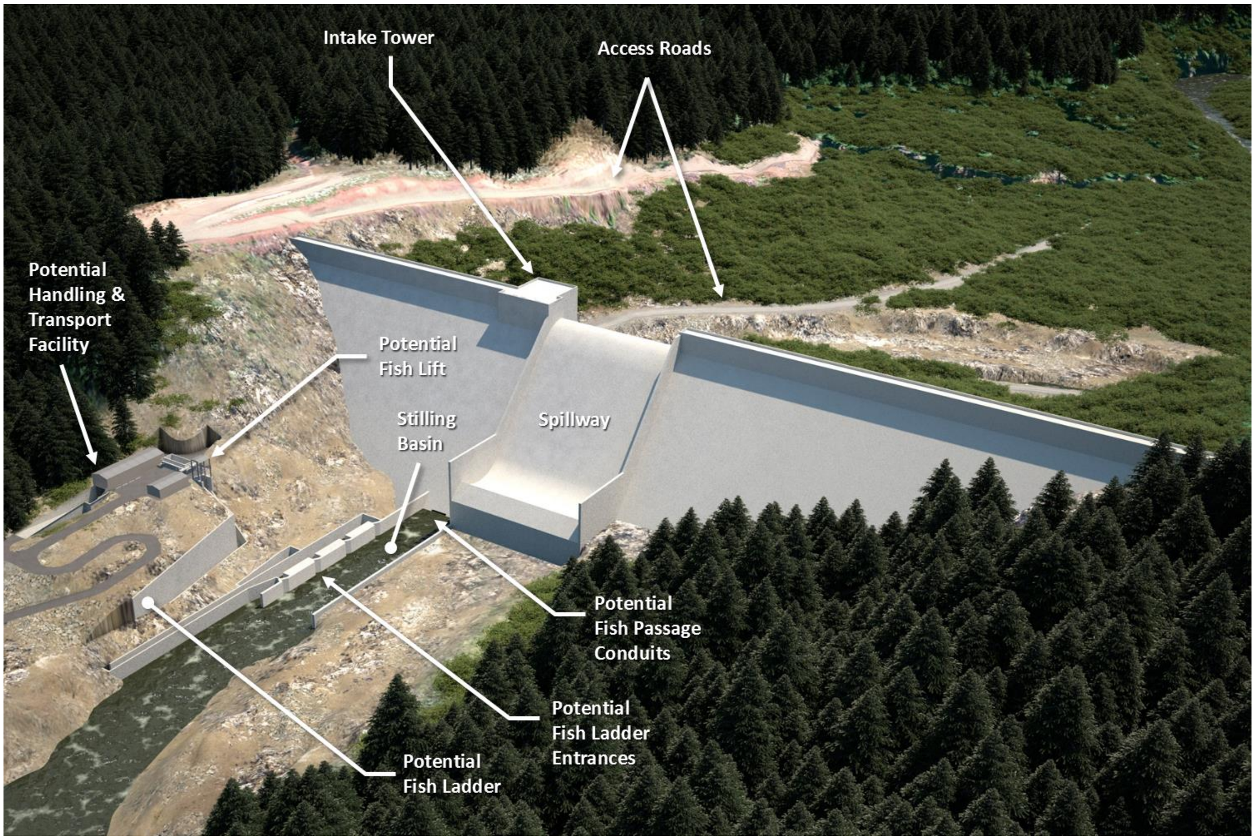


Figure 1-3
FRO Dam Rendering, Looking Downstream

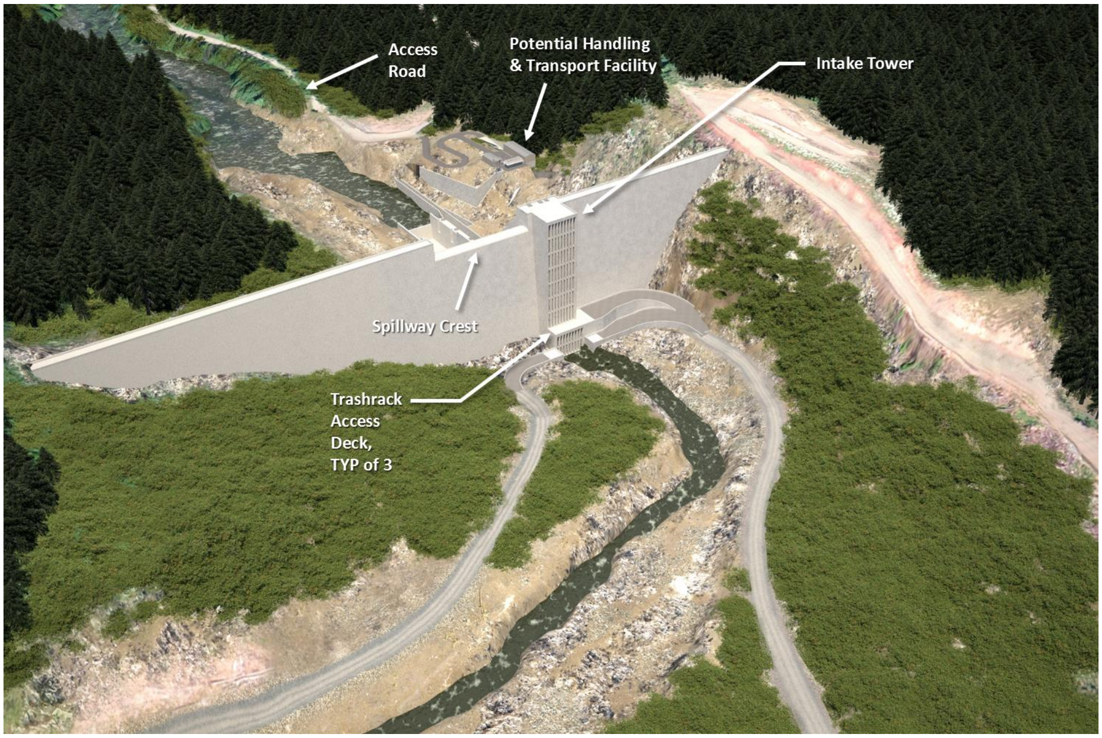


Figure 1-4
FRFA Dam Rendering

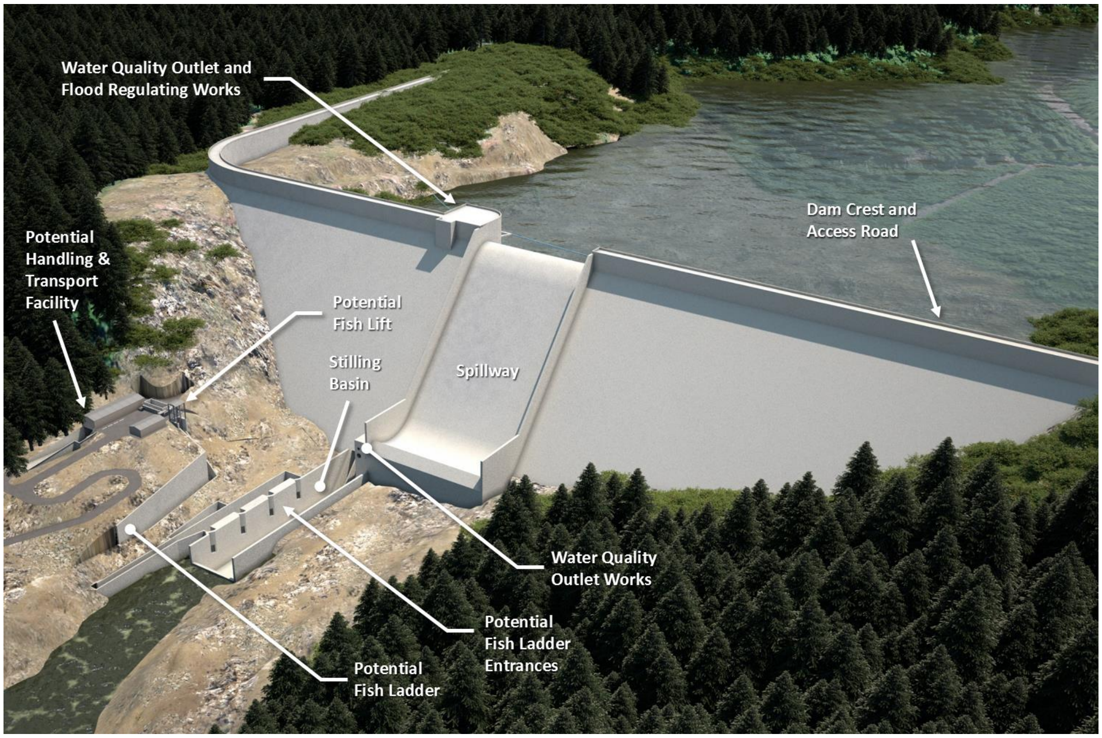
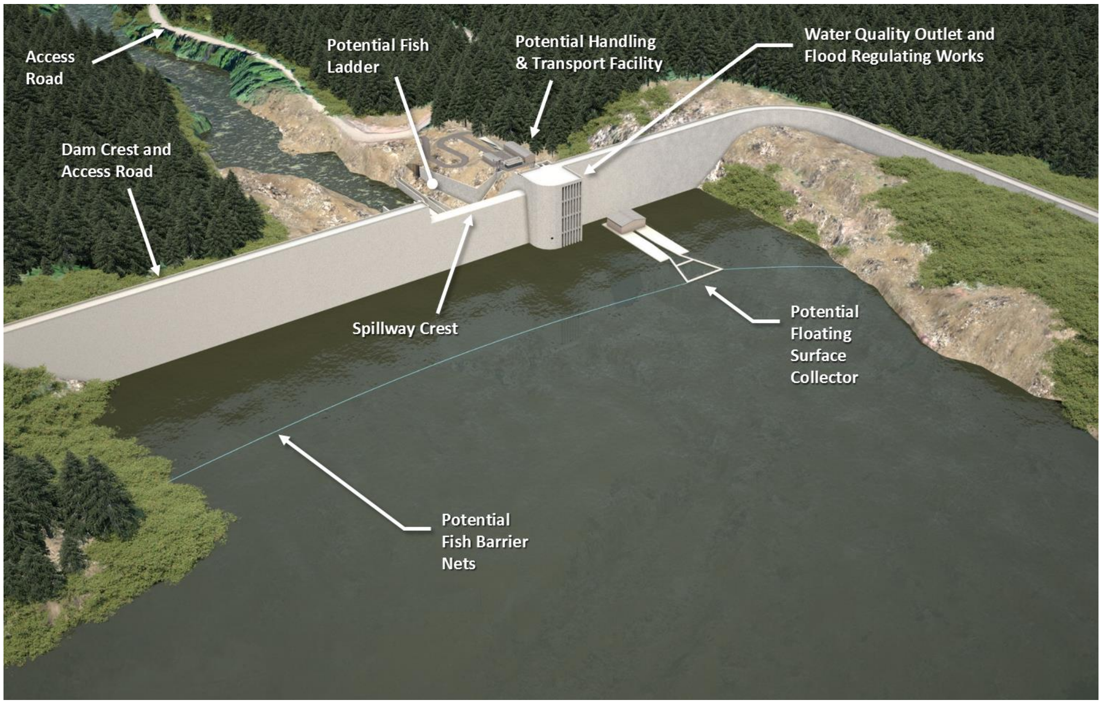


Figure 1-5
FRFA Dam Rendering, Looking Downstream



1.3 Previous Reports

Previous reports and technical memoranda listed below form the basis for the updated conceptual fish passage design options presented in this technical memorandum:

- Interim Fish Passage Design Criteria Technical Memorandum, October 2013 – The Interim Fish Passage Design Criteria Technical Memorandum evaluated biological and technical aspects of fish passage facility alternative development.
- Draft Dam Design Technical Memorandum, March 2014 – The objectives of the Dam Design Technical Memorandum (TM) were to identify any fatal flaws that would limit or preclude construction of a water retention structure at the proposed location on the main stem of the Chehalis River, and to develop technically feasible options for a flood retention or a multipurpose dam at that site.
- Fish Passage Design Technical Memorandum, May 2014 – The Fish Passage Design TM evaluated potential fish passage technologies, established design criteria, and developed options for upstream and downstream passage of adult and juvenile fish that could be integrated with feasible water retention (dam) structures.
- Combined Dam and Fish Passage Alternatives Technical Memorandum, September 2014 – The Combined Dam and Fish Passage Alternatives TM built upon the findings of the Dam Design TM and the Fish Passage Design TM to combine selected dam design options with selected fish

passage options to describe four integrated alternatives that can be compared in terms of function, constructability, and capital and operations and maintenance costs.

- Interim Dam Design Criteria Technical Memorandum, draft version December 2015 – The Interim Dam Design Criteria Technical Memorandum combined dam and fish passage criteria to facilitate discussion. The fish passage criteria continued to develop in collaboration with representatives from various resources agencies.

1.4 Study Purpose and Intent

The integration of fish passage systems in flood damage reduction structure design is required by the State of Washington and defined in Revised Code of Washington (RCW) 77.57.030, *Fishways required in dams, obstructions – penalties, remedies for failure.* RCW 77.57.030 requires that dam owners provide safe and timely fish passage for all fish species and fish life stages present in an affected area. Fish passage facility conceptual design has progressed simultaneously with dam design efforts throughout the CBS Project. The purpose of this document is to summarize the results and conclusions of fish passage facility concept development performed in 2015-16, which built on the preceding design development activities presented in HDR, 2014d and e. This information is intended to be used by the Work Group to inform decisions regarding the integration and performance of potential fish passage technologies with flood damage reduction structural alternatives being developed by the dam design study team.

1.5 Scope of this Document

The scope of this document includes presenting and summarizing the results and conclusions of the following activities:

- Site investigations conducted to verify assumed upstream and downstream passage facility locations;
- Collaboration with members of the Flood Damage Reduction Technical Committee to refine and verify biological and technical fish passage design criteria;
- Assessment of updated proposed dam operations and develop revised flow and stage duration statistics throughout anticipated migration periods of target fish species;
- One-dimensional hydraulic modeling to evaluate existing and proposed hydraulic conditions on a reach type scale;
- Hydraulic calculations for the purposes of sizing and configuring fish passage facility elements;
- Confirmation of the general orientation, location, and size of fish passage facilities and verification that the overall fish passage facility strategy addresses the seasonal performance requirements for both upstream and downstream passage;
- Verification and refinement of selected upstream and downstream fish passage facility descriptions, layouts, and operational parameters;

- Verification and updating of anticipated performance expectations for upstream and downstream fish passage facilities in collaboration with the Flood Damage Reduction Technical Committee.
- Refinement of passage facility elements and refinement favorable conditions for fish attraction, collection, or passage based on the results of the above listed activities.
- Evaluation of sediment and debris transport through FRO alternative tunnels and development of conclusions regarding potential impact to fish passage; and
- Development of a draft report documenting the refined design considerations/criteria, selection of facility types to be further developed, and documentation of defined performance expectations.

Activities which have been completed to a conceptual level and are continuing to be further developed at the time of document completion are listed below.

- Perform advanced hydraulic modeling using computational fluid dynamics where needed to assess complex hydraulic interaction with proposed dam structural configurations and confirm hydraulic control features of the dam structure.

1.6 Project Team

The following personnel were involved in the various evaluations required to complete the updated conceptual designs:

Project Manager:	Beth Peterson, P.E.
Technical Manager and Lead Civil Eng:	Keith Moen, P.E.
Lead Dam Engineer:	Keith A. Ferguson, P.E.
Lead Geotechnical Engineer:	Dan Osmun
Geologists/Geotechnical Engineers:	Andrew Little
	John Charlton
Lead Hydraulic Engineer:	Ed Zapel, P.E.
Lead Fish Passage Designer:	Michael Garello, P.E.
Constructability and Cost Estimating:	Jeffrey Allen
Project Support:	Gokhan Inci, PhD Geotechnical Eng.
	Matthew Prociv, P.E., Fish Passage Design
	Shaun Bevan, P.E., Fish Passage Design
	Taylor Hoffman, Water Resources EIT
	John Ferguson, Fish Passage Biology
	John Hess, Materials Engineering
	Anna Mallonee, Engineering Intern

Additional staff provided for drawings, document production, and quality control.

2 FISH PASSAGE CRITERIA

The biological and technical fish passage criteria used in previous CBS Project studies (i.e. HDR, 2014a and HDR, 2014c) were refined using updated site specific information, additional research of available literature, and focused collaboration with participants of the Flood Damage Reduction Technical Committee. The results or this effort are described in the following section. The refined criteria are carried forward to inform the conceptual design of potential fish passage alternatives described later in Section 4 of this document.

2.1 Collaboration with Technical Committees

The fish passage design team and members of the Chehalis Basin Strategy – Flood Damage Reduction Technical Committee coordinated and carried out several fish passage subcommittee meetings throughout development of this study. These meetings became forums for information transfer, detailed discussion, and decision making relative to biological and technical aspects of fish passage facility alternative development. Of primary importance were the discussion, interpretation, and formulation of design criteria. Participants attending these meetings included representatives from the following organizations:

- Washington Department of Fish and Wildlife (WDFW)
- U.S. Fish and Wildlife Service (USFWS)
- National Marine Fisheries Service (NMFS)
- Washington State Department of Ecology (WDOE)
- Quinault Indian Tribe
- State of Washington Consultant Study Team

Meeting dates, agenda, and notes resulting from these meetings are included in Attachment A of this document and form a basis for criteria refinement and identification of key assumptions necessary to continue the engineering development of potential fish passage facilities.

2.2 Biological Design Criteria

As part of the Chehalis Project, WDFW has led an extensive field sampling program to collect data and better understand the phenology, abundance, habitat requirements, distribution, and migration patterns of fish present within the Chehalis River and more specifically within the inundation limits of the reservoir. Using new and available historic data, WDFW has assisted the fish passage design team with biological fish criteria development in collaboration with other participating technical committee stakeholders. The three primary biological design criteria that have the most influence on facility type, size, and configuration are the following:

- Fish occurrence and distribution: Informs the selection of species and life stages targeted for fish passage design.
- Fish migration timing: Informs the understanding of seasonality, anticipated hydrologic conditions, and duration of periods when target fish species may be expected to migrate upstream and/or downstream of the dam location.
- Fish abundance: Informs the estimation of the annual number of fish that require passage as well as the peak daily rate of migration that influences facility size and operation requirements.

2.2.1 Fish Occurrence and Distribution

The selection of fish species and life stages for fish passage design was derived from field-specific data obtained by WDFW in 2015 and 2016 in addition to readily available historical documentation developed for the Chehalis Basin. In general, the State of Washington interprets its regulatory authority (RCW 77.57.030) to require provision for passage of all fish and fish life stages believed to be present in the system. For the purposes of fish passage alternative development, anadromous and fluvial species known to be present within the influence of the dam, in the inundation area of the associated reservoir, and upstream of the reservoir were selected for both upstream and downstream passage. These primary species and their known swimming and leaping abilities were used to influence fish passage technology selection and development of specific technical design criteria. Species known to occur downstream of the dam project area were selected for consideration but did not directly influence the development of specific technical design criteria.

The life histories and specific life stages of each target species were also considered relative to their known occurrence, distribution, and movement through the project area. Life stages of specific species were selected if they have been observed moving – or are believed to move – through the project area (either upstream or downstream).

Target fish species and their respective life stages which were selected for the purposes of design development in this study are presented in Table 2-1.

Table 2-1
Target Fish Species and Life Stages Selected for Design

SPECIES	UPSTREAM	DOWNSTREAM
Spring Chinook	Adult, Juvenile	Juvenile
Fall Chinook	Adult, Juvenile	Juvenile
Coho	Adult, Juvenile	Juvenile
Winter Steelhead	Adult, Juvenile	Adult, Juvenile
Coastal Cutthroat	Adult, Juvenile	Adult, Juvenile
Pacific Lamprey	Adult	Ammocoetes, Macropthalmia
Western Brook Lamprey	Adult	Ammocoetes, Macropthalmia

Bull trout are believed to be present downstream of the proposed dam location and remain a species of consideration throughout alternative development and concept design. Of the species and life stages targeted for upstream passage, juvenile salmonids (steelhead and salmon), juvenile cutthroat trout, and lamprey exhibit the most variable life history, are the weakest swimmers, and represent the most difficult organisms requiring passage. Therefore, technical design criteria used to target the passage requirements of these other species and life stages were believed to also accommodate the requirements of bull trout.

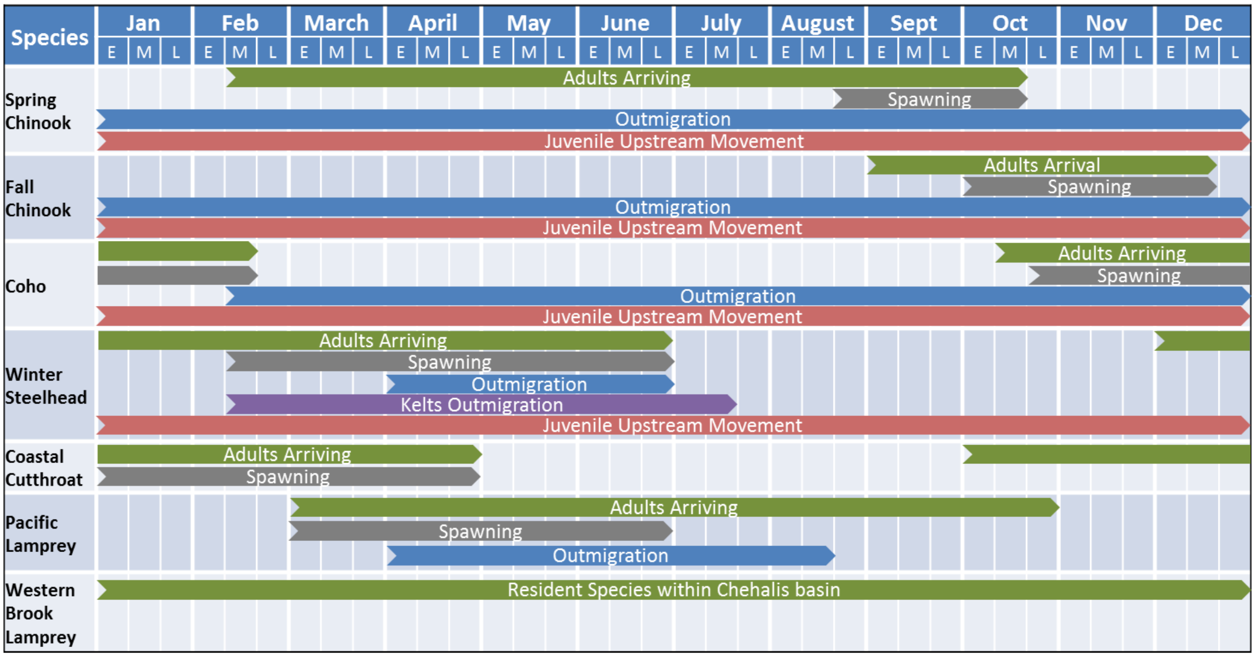
Lamprey passage technologies are relatively new, and few facilities exist in the western United States that target lamprey for passage or collection and transport above dams. Where applicable, readily available best practices, lessons learned from experimental facilities on the Columbia River, and interviews with researchers who specialize in the understanding of lamprey behavior and navigational capabilities were used to inform lamprey passage facility requirements and anticipated performance.

2.2.2 Fish Migration Timing

Each target species is known to have unique migration behavior and is believed to pass upstream or downstream through the project area at specific times of the year for specific durations. The migration timing and duration influence the design and operation of proposed fish passage facilities by defining physical, operational, and environmental conditions expected to occur while passage facilitation is required. The migration timing and duration for each selected fish species and life stage were discussed at fish passage subcommittee meetings as new information was collected in the field. The resulting conclusions, which were used in fish passage alternative design development, are shown in Figure 2-1.

The selected values provide a conservative summary of upstream migration, spawning, and outmigration periods suitable to inform robust fish passage designs. Aquatic target species' actual migration and spawning periods are far more complicated and nuanced. For the purposes of alternative development and preparation of conceptual designs, these nuances are not anticipated to be controlling factors in the design and comparison of the fish passage options.

Figure 2-1
Anticipated Migration Periods of the Targeted Species and Life Stages (Periodicity)



Notes: E=Early, M=Middle, and L-Late

2.2.3 Fish Abundance

Fish abundance was evaluated by WDFW and discussed during fish passage subcommittee meetings. Abundance was described in terms of peak annual and peak daily rates of migration. The peak daily rate of migration for both upstream and downstream migrating fish influences the size of many components to fish passage alternatives. The following paragraphs summarize the conclusions from two references developed by WDFW (WDFW, 2016a and 2016b). These results were consulted for the purposes of design development during this study.

2.2.3.1 Upstream Migration

Upstream migration rates were estimated based upon two factors: 1) historic data relative to adult spawner survey results and escapement records, and 2) proposed annual peak goals after project implementation and potential habitat restoration. The peak rate of annual migration for adult salmonids moving upstream was as follows:

- Spring Chinook: 1,350/yr
- Fall Chinook: 3,900/yr
- Coho: 12,900/yr
- Steelhead: 5,630/yr

The peak annual rates for adult upstream migrating Pacific lamprey, cutthroat trout, resident fish, and juvenile salmonids have not been estimated. Although these species are an important influence on the overall design of each fish passage alternative, their peak rate of migration is currently unknown and is not anticipated to significantly influence facility size to the extent of adult salmonids.

Given the total number of anticipated adult upstream migrants, WDFW proposes the use of NMFS guidance to derive peak daily values from peak annual estimates. Given the information presented in the literature, peak daily count can be estimated as 10 percent of the maximum annual run (Bates, 1992). To be conservative, NMFS suggests an estimate of 20 percent of the peak annual count based on Bell (1991). If 20 percent of the peak annual count is used, and the peak hourly is calculated as being 10 percent of the daily run, then the peak hourly count is approximately 2 percent of the annual run. Using this methodology a combined peak daily count of 4,800 adult salmonids or a peak hourly count of 475 adult salmonids is used for design purposes.

2.2.3.2 Downstream Migration

Table 2-2 summarizes the total abundance numbers recommended for reference in designing downstream passage alternatives for juvenile salmon and steelhead. These values represent the total number of fish expected to be produced upstream of the location selected for the dam. These numbers do not reflect habitat degradation resulting from the inundation pool, which will restrict spawning and rearing habitat available for all species, but especially Chinook salmon (Ashcraft et al., 2016).

Table 2-2
Predicted Abundance of Juvenile Salmon and Steelhead that will Migrate Downstream from Freshwater Habitat above River Mile 108 of the Chehalis River

SPECIES	LIFE STAGE	MIGRATION PERIOD	MAXIMUM ABUNDANCE
Coho Salmon	Fall Parr	September – December	340,000
	Spring Smolt	March – June	17,000
Steelhead Trout	Fall Parr	September – December	97,000
	Spring Smolt	March – June	14,500
Chinook Salmon	Subyearling (Fry)	January – April	229,000
	Subyearling (Parr/Smolt)	May – August	114,500
	Yearling	March - June	11,000
Other Species	Data unavailable to support conclusions regarding downstream migration.		

For spring smolts, freshwater capacity and migration timing were used to predict total daily arrivals between January and August using two example migration curves originating from other river systems. Timing curve 1 represented a free flowing river (Coweeman River), whereas timing curve 2 represented a dammed river where smolts rear in cooler stream temperatures and navigate a reservoir during their downstream migration (Cowlitz River). The expected daily numbers (mean and maximum values) of downstream migrants were similar between the two migration timing curves when all species were included. However, when only coho salmon and steelhead trout were included, mean and maximum values were higher under timing curve 1 than timing curve 2. This difference is relevant because of the uncertainties associated with continued production of Chinook salmon above river mile 108 were a dam to be constructed. The difference between the two scenarios results from the smolts of coho salmon and steelhead trout having a more protracted migration timing under timing curve 2 than timing curve 1.

For fall migrants, timing curves were not available, and daily numbers were approximated based on available information (see WDFW, 2016a and 2016b). Estimates of daily numbers of fall migrants were based upon the maximum daily values derived for spring smolts of coho salmon and steelhead trout increased by a multiplier of 17.0. The resulting maximum daily abundance selected for design purposes is therefore 55,500 smolt as indicated in Table 2-3.

Table 2-3
Predicted Daily Numbers of Juvenile Salmon and Steelhead Originating from Freshwater Habitat Upstream of River Mile 108 in the Chehalis River

DAILY METRIC	SPRING SMOLTS (JAN – AUG)		SPRING SMOLTS (JAN – AUG) COHO AND STEELHEAD ONLY		FALL SMOLTS (SEP – DEC) COHO AND STEELHEAD ONLY	
	Daily Abundance (Timing 1)	Daily Abundance (Timing 2)	Daily Abundance (Timing 1)	Daily Abundance (Timing 2)	Daily Abundance (Timing 1)	Daily Abundance (Timing 2)
Mean	1,919	1,882	203	82	3,451	1,394
Maximum	11,013	10,935	3,265	668	55,505	11,356

2.3 Technical Design Criteria

This section identifies specific technical design criteria and references specific sources of design criteria and guidance relevant to the development of fish passage designs. NMFS fisheries design criteria are broken into two categories – criteria and guidelines. Criteria are specific standards for fish passage design that require an approved variance from the governing state or federal agency in order to deviate from the established criteria. Guidelines provide a range of values or, in some instances, specific values that the designer should seek to achieve but that can be adjusted without agency approval in light of project-specific conditions. Site-specific biological and physical rationale for deviating from an agency-established criterion is required. Values different from established guidelines should support better performance or solve site-specific issues. The reasons for using values different from established guidelines may be requested by governing agencies during development of the design. The list of criteria provided in Section 2.3 is not intended to be an all-inclusive list of criteria for design but is used to guide alternative formulation and concept development.

The following documents provide the guidelines that are used during concept design. If two or more agencies provide differing guidance on a design criterion, the most conservative guidance for fish passage and protection will be followed.

- National Marine Fisheries Service (NMFS). 2011. Anadromous Salmonid Passage Facility Design.
- United States Fish and Wildlife Service (USFWS). 2010. Best Management Practices to Minimize Adverse Effects to Pacific Lamprey.
- Washington Department of Fish and Wildlife (WDFW). 2000. Draft Fish Protection Screen Guidelines for Washington State.
- Washington Department of Fish and Wildlife (WDFW). 2000. Draft Fishway Guidelines for Washington State.
- Washington Department of Fish and Wildlife (WDFW). 2013. Water Crossing Design Guidelines.

2.3.1 Passage Tunnels

During the 2014 study, the criteria for the fish passage tunnels in the Flood Retention Only (FRO) alternative were based on Washington State Department of Fish and Wildlife's 2013 *Water Crossing Design Guidelines* (WDFW, 2013). The WDFW document suggests that a minimum hydraulic design target of 0.8 feet of water depth and a maximum flow velocity of 2 feet per second be used for water crossing structures with lengths of approximately 200 feet. However, in consultation with members of the Fish Passage Subcommittee in 2015 and 2016, it was determined that the natural flow characteristics in this reach of the river were more restrictive to passage than WDFW's guidelines. It was agreed that the hydraulic conditions in the natural channel upstream and downstream of the passage tunnels would negate the passage benefit of designing the tunnels to WDFW's guidelines. Therefore, the Subcommittee concluded that the proposed flow velocity and depth through the tunnels mimic the flow velocity and depth occurring naturally through the existing river reach at the location of the dam. This premise influenced the overall approach towards designing and evaluating performance of upstream and downstream passage through the conduits.

2.3.2 Fishway Criteria

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: the fishway entrance, fish ladder, and fishway exit. Table 2-4, Table 2-5, and Table 2-6 below list the fishway entrance, fish ladder, and fishway exit criteria, respectively.

Table 2-4
Fishway Entrance Criteria

CRITERION	VALUE	REFERENCE
Location	Easily located by fish	NMFS 2011, WDFW 2009
Width	4 feet, minimum	NMFS 2011
Depth	6 feet, minimum	NMFS 2011
Head Differential, adults	1 – 1.5 feet	NMFS 2011, WDFW 2009
Head Differential, juveniles	0.13 inches	NMFS 2011
Attraction Flow	5% – 10% of the maximum of the 5% exceedance flows for the migration period of each species	NMFS 2011
AWS Energy Dissipation Factor	16 ft-lbs/sec/ft^3	NMFS 2011
AWS Diffuser Velocity, vertical	1 fps, maximum	NMFS 2011
AWS Diffuser Velocity, horiz.	0.5 ft/s, maximum	NMFS 2011
AWS Diffuser Bar Spacing	1.75 mm, maximum (juvenile criteria)	NMFS 2011
Fish Burst Speed	27 fps, maximum	Bell 1991, pg. 6.3 (Steelhead)
Fish Burst Duration	10 seconds, maximum	Bell 1991, p.g 6.2
Depth Required for Jumping	2 feet, minimum	USFS Handbook 2090.21, Adult Salmonid Migration Blockage Table (adapted)

Note:
*AWS – Auxiliary Water System (supplemental attraction flow)

Table 2-5
Fish Ladder Criteria

CRITERION	VALUE	REFERENCE
Head differential, juveniles	0.7 feet, maximum	NMFS 2011
Head differential, adults	1.0 feet	NMFS 2011
Energy Dissipation Factor	2 ft-lbs/sec/ft^3 (juvenile criterion)	NMFS 2011
Turning Pool	Radius corners	NMFS 2011, WDFW 2009
Pool Width	6 feet, minimum	NMFS 2011
Pool Length	8 feet, minimum	NMFS 2011
Pool Depth	5 feet, minimum	NMFS 2011
Baffle Orifice Dimensions	18 inches high x 15 inches wide	WDFW 2009
Freeboard	3 feet, minimum	NMFS 2011, WDFW 2009

Table 2-6
Fishway Exit Criteria

CRITERION	VALUE	REFERENCE
Head differential	0.25 to 1.0 feet	NMFS 2011
Length	2x fish ladder pool length	NMFS 2011
Location	Along the shoreline Downstream current < 4 ft/s Minimize fallback	NMFS 2011, WDFW 2009
Coarse Trashrack – Velocity	1.5 fps, maximum	NMFS 2011
Coarse Trashrack – Water Depth	Equal to fish ladder exit pool depth	NMFS 2011
Coarse Trashrack – Bar Spacing	10 inches, minimum	NMFS 2011
Coarse Trashrack – Support Bar Spacing	24 inches, minimum	NMFS 2011
Coarse Trashrack – Slope	1 horizontal: 5 vertical	NMFS 2011

2.3.3 Lamprey Passage

Upstream and downstream passage of lamprey is an important consideration for each potential fish passage alternative. As requested by participating resource agencies and tribal entities, incorporation of the best available science relating to the passage of lamprey was considered throughout conceptual design. As mentioned previously in Section 2.2.1, best practices, lessons learned from experimental facilities on the Columbia River, and interviews with researchers who specialize in the understanding of lamprey behavior and navigational capabilities were used to inform lamprey passage facility requirements. Key facility requirements related to the passage of lamprey are summarized in Table 2-7. The following resources outline a number of experimental facilities and best practices focusing on passing lamprey upstream which were used to form a basis of design for lamprey passage technologies and measures:

- Best Management Practices to Minimize Adverse Effects to Pacific Lamprey (USFWS, 2010)
- Adult Pacific Lamprey Passage: Data Synthesis and Fishway Improvement Prioritization Tools (Keefer et al., 2012)
- Pacific Lamprey and NRCS: Conservation, Management and Guidelines for Instream and Riparian Activities (USDA, 2011).
- Pacific Lamprey Protection Guidelines (USDA, 2010)
- Lamprey Passage in the Willamette Basin: Considerations, Challenges, and Examples (USFWS, 2011)
- Adult Pacific Lamprey: Known passage challenges and opportunities for improvement (Keefer et al., 2014)
- Evaluation of Adult Pacific Lamprey Fish Passage at Snake River Dams (Stevens et al., 2015)

Table 2-7
Lamprey Upstream Passage Criteria

CRITERION	VALUE	REFERENCE
Flow Velocity (max.)	4 to 6 fps	USDA 2010
Wall Finish	Smooth	USDA 2010
Corner Geometry	Rounded	USDA 2010

2.3.4 Trashracks

Trashracks are commonly used to exclude large debris from entering fish passage facilities. They are commonly used at fishway exits and entrances. Table 2-8 below lists the design criteria for trashracks.

Table 2-8
Trashrack Criteria

CRITERION	VALUE	REFERENCE
Velocity	1.5 ft/s, maximum	NMFS 2011
Water Depth	Equal to fish ladder exit pool depth	NMFS 2011
Bar Spacing	10 inches, minimum	NMFS 2011
Support Bar Spacing	24 inches, minimum	NMFS 2011
Slope	1 horizontal: 5 vertical	NMFS 2011

2.3.5 Fish Screen and Bypass Criteria

The downstream passage system consists of five major components:

- Fish screens (Table 2-4) to protect juvenile fish from entrainment or impingement. These are often located adjacent to the bypass channel (Table 2-9).
- A bypass channel (Table 2-10). The bypass channel conveys the fish and is often located adjacent to the fish screens.
- A bypass entrance (Table 2-11), located at the end of the fish screens.
- A bypass conduit (Table 2-12), which conveys fish from the bypass entrance to a point of release downstream (bypass exit).
- A bypass exit (Table 2-13), located at the end of the bypass conduit.

Table 2-9
Fish Screen Criteria

CRITERION	VALUE	REFERENCE
Approach Velocity, Va	0.4 fps	NMFS 2011
Sweeping Velocity, Vs	Vs > Va and ΔVs ≥ 0	NMFS 2011, WDFW 2009
Screen Orientation (river)	Parallel to flow	NMFS 2011, WDFW 2009
Screen Orientation (reservoir)	As required to maximize fish attraction	NMFS 2011, WDFW 2009
Screen Type	Wedgewire or profile bar	n/a
Screen Opening	1.75mm	NMFS 2011
Screen Open area	27% minimum	NMFS 2011, WDFW 2009
Screen Cleaning	Automatic	n/a
Head Differential to start automated screen cleaning	0.1 feet	NMFS 2011

Table 2-10
Bypass Channel Criteria

CRITERION	VALUE	REFERENCE
Acceleration, Ac	0.2 fps/ft > Ac > 0	NMFS 2011
Bypass Entrance Location	At downstream end of screens	NMFS 2011
Capture Velocity, Vc	Vc ≥ 8 fps	Stakeholder input

Table 2-11
Bypass Entrance Criteria

CRITERION	VALUE	REFERENCE
Flow Control	Independent & at bypass entrance	NMFS 2011
Velocity, Ve	Ve > 110% of bypass channel velocity	NMFS 2011, WDFW 2009
Capture Velocity, Vc	Vc ≥ 8 fps	Stakeholder input
Width	18 inches, minimum	NMFS 2011, WDFW 2009
Depth of Water Over Weir	1 foot min	NMFS 2011

Table 2-12
Bypass Conduit Criteria

CRITERION	VALUE	REFERENCE
Flow	Approx. 5% of screened flow	NMFS 2011
Flow Type	Open channel	NMFS 2011, WDFW 2009
Water Depth	40% of channel diameter or width and 9 inches minimum	NMFS 2011 and WDFW 2009, respectively
Velocity, Goal	6 fps to 12 fps	NMFS 2011
Velocity, Minimum	2 fps	NMFS 2011
Velocity, Maximum	30 fps	WDFW 2009
Material	Smooth interior surfaces, walls, joints	NMFS 2011, WDFW 2009
Closure Valves	None allowed within conduit	NMFS 2011, WDFW 2009
Hydraulic Jumps	None allowed within conduit	NMFS 2011, WDFW 2009

Table 2-13
Bypass Exit Criteria

CRITERIA	VALUE	REFERENCE
Velocity	25 fps, maximum	NMFS 2011, WDFW 2009
Location	• Strong downstream current • Sufficient depth to avoid fish injury • Minimize adult attraction	NMFS 2011, WDFW 2009

2.3.6 Fish Trapping and Holding

In upstream and downstream trap and transport facilities such as a CHTR or Trap and Transport facility there are multiple factors that impact the health and safety of fish. Some such factors include how long fish can safely be held and how densely they can be contained. The criteria for fish trapping and holding were developed in consultation with the Fish Passage Subcommittee and are provided in Table 2-14 and Table 2-15 below.

Table 2-14
Fish Trapping and Holding Criteria

CRITERIA	VALUE	REFERENCE
Holding duration – holding gallery	24 hours, maximum	NOAA Fisheries 2011
Holding duration – hopper and transport tank	24 hours, maximum 1/2 hour, maximum during peak run rates	NOAA Fisheries 2011
Temperature	50°F	NOAA Fisheries 2011
Dissolved oxygen	6 to 7 parts per million	NOAA Fisheries 2011
Water supply, holding, fry	0.0075 gallons per minute (gpm) per fish	Piper et al. 1982
Water supply, holding, smolts	0.13 gpm per fish	Piper et al. 1982
Water supply, holding, adults	0.67 gpm per fish	NOAA Fisheries 2011
Adult jump provisions	Required	NOAA Fisheries 2011
Segregation of fish	Capability required	Not applicable
General	Decrease poundage of fish held by 5% for every degree over 50ºF	

Table 2-15
Fish Size and Holding Volume Criteria

SPECIES	AVERAGE ASSUMED WEIGHT/FISH (POUNDS)	LONG-TERM HOLDING: FLOW/FISH (GPM)	HOLDING VOLUME (CF/POUNDS)
Spring-run Chinook salmon	23	1	0.25
Fall-run Chinook salmon	23	1	0.25
Coho salmon	9.5	0.5	0.25
Winter-run steelhead	9	2.0	0.25
Summer-run steelhead	8	2.0	0.25
Coastal cutthroat trout	1	Unknown	0.25
Lamprey		Unknown	
Resident species		Unknown	

Note:
*Fry size based on Chinook. Fry Holding Volume per NMFS 2011. Adult Holding Volume per Bell 1991, pg. 33.1 and assumes 30 minutes to 1 hour holding time. Adult fish sizes per Bell 1991, pgs. 5.8-5.31.

2.3.7 Fish Passage Design Flows

Fish passage design flow criteria influence several factors associated with fish passage facility size and complexity. NMFS and WDFW provide guidelines for the selection of high and low flows to be used in the design of fish passage facilities. Guidelines presented by NMFS and WDFW are based on exceedance calculations of mean daily flows but can be modified to suit site-specific requirements. The exceedance flows statistically represent the flow equaled or exceeded during certain percentages of the time when migrating fish may be present. The established guidelines are used to set instream flow depths, flow velocities, debris and bedload conditions, fish attraction requirements, tailwater fluctuations, and numerous other factors that a facility might experience while target fish species are migrating.

NMFS (2011) requires 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. WDFW (2000b) suggests a 10 percent exceedance flow be used as a high design flow. NMFS (2011) requires a low fish passage design flow equal to the mean daily stream flow that is exceeded 95 percent of the time during periods when migrating fish are typically present. WDFW recommends that a low flow be established based upon site-specific conditions. 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.

Mean daily flows at the proposed dam site were estimated by Watershed Science & Engineering (WSE). WSE used a precipitation-weighted basin area ratio to relate the effective watershed area occurring above the proposed dam site to the effective watershed area occurring upstream of the USGS gage 12020000 near Doty. Mean daily flows from USGS gage 12020000 near Doty were reduced using this ratio in order to estimate mean daily flows at the proposed dam site for water years 1940 through 2012. An exceedance analysis was then performed on the estimated flows at the proposed dam site. Annual flow exceedance flows are summarized in Table 2-16.

The 5 percent 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 (Table 2-17). The lowest 95 percent exceedance flow and the largest 5 percent exceedance determined the fish passage design flow range that both FRO and FRFA 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. Although future dam operations for the FRFA dam configuration may limit the low flows to a much higher value, this lower, more conservative value was used for conceptual design purposes in both dam configurations.

Table 2-16
Annual Flow Exceedance at the Proposed Dam Site

PERCENT OF TIME EXCEEDED	FLOW (CFS)
99%	15
95%	19
90%	24
80%	37
75%	48
50%	171
25%	437
10%	960
5%	1,447
1%	2,957

Table 2-17
Flow Exceedance during Fish Migration Periods at the Proposed Dam Site

FISH SPECIES AND MIGRATION	95 PERCENT EXCEEDANCE (CFS) (MIN DESIGN FLOW)	5 PERCENT EXCEEDANCE (CFS) (MAX DESIGN FLOW)
Spring Chinook	18	882
Fall Chinook	**16**	1,592
Coho	36	**2,197**
Winter Steelhead	63	1,724
Coastal Cutthroat	34	1,908
Pacific Lamprey	17	737
Western Brook Lamprey	19	1,447

2.3.8 Range of Reservoir Fluctuation

Anticipated reservoir pool fluctuation for the FRFA dam configuration is a significant factor in determining the type, size, and complexity of upstream and downstream fish passage facilities. Upstream fish passage technologies may require safe release or exit of fish to the reservoir pool. Downstream fish passage technologies in the reservoir either float or include multiple inlets to maintain a hydraulic connection with the reservoir surface. Each type of fish passage technology must accommodate some form of continuous hydraulic connection throughout the anticipated range of pool elevations. As the pool fluctuations become larger, the facilities become larger and more complex. In many cases, certain fish passage technologies can be dismissed because they are unable to accommodate large pool fluctuations.

Historic river flows from October 1, 1988, through January 1, 2016, were input into a reservoir operations simulation model (Anchor, 2016) to estimate the FRFA reservoir elevations. The first water year (1989) was omitted from the results summarized below because it included filling of the reservoir and therefore did not represent a typical operating scenario.

Modeled FRFA reservoir elevations from water year 1990 through 2015 are presented in Figure 2-2 and Figure 2-3. Figure 2-2 provides a timeline of reservoir elevations showing long term trends. Figure 2-3 overlays each water year to demonstrate the seasonality of reservoir operations.

The conservation pool of the FRFA reservoir is expected to normally fluctuate between water surface elevation (WSEL) 588 to 628 (40 feet) over the course of a year and is regulated to seasonally enhance water quality and instream flow downstream of the FRFA dam structure. Flood events may bring the reservoir pool higher than WSEL 628; potentially as high as WSEL 709 in a PMF event. Extreme drought or reservoir drawdown conditions may bring the reservoir pool as low as WSEL 520. Therefore, the normal operational range of the reservoir during which fish passage must be provided is anticipated to be 40 feet, while periodic extremes could cause the reservoir to fluctuate up to 189 feet. Fish passage outside the normal operational range is not required as such fluctuations will be extreme and periodic.

Figure 2-2

Long Term Trends of FRFA Reservoir Elevation. Results from FRFA Reservoir Operations Modeling

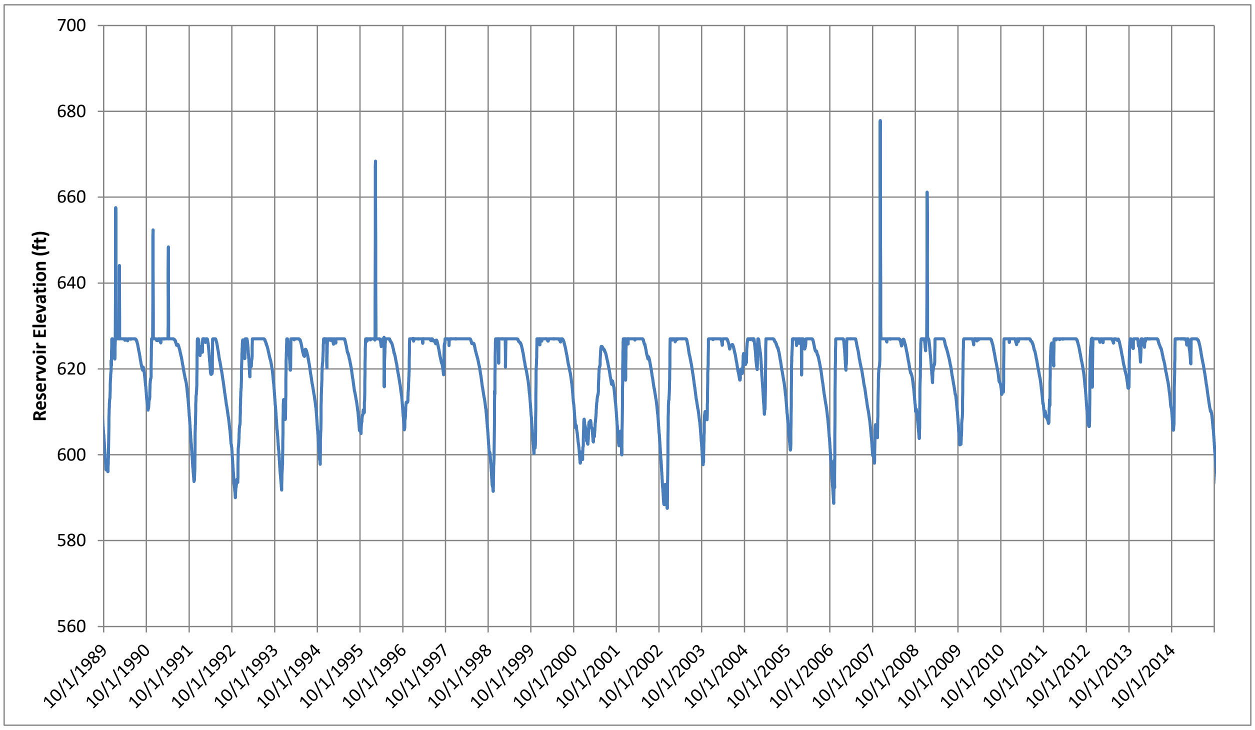
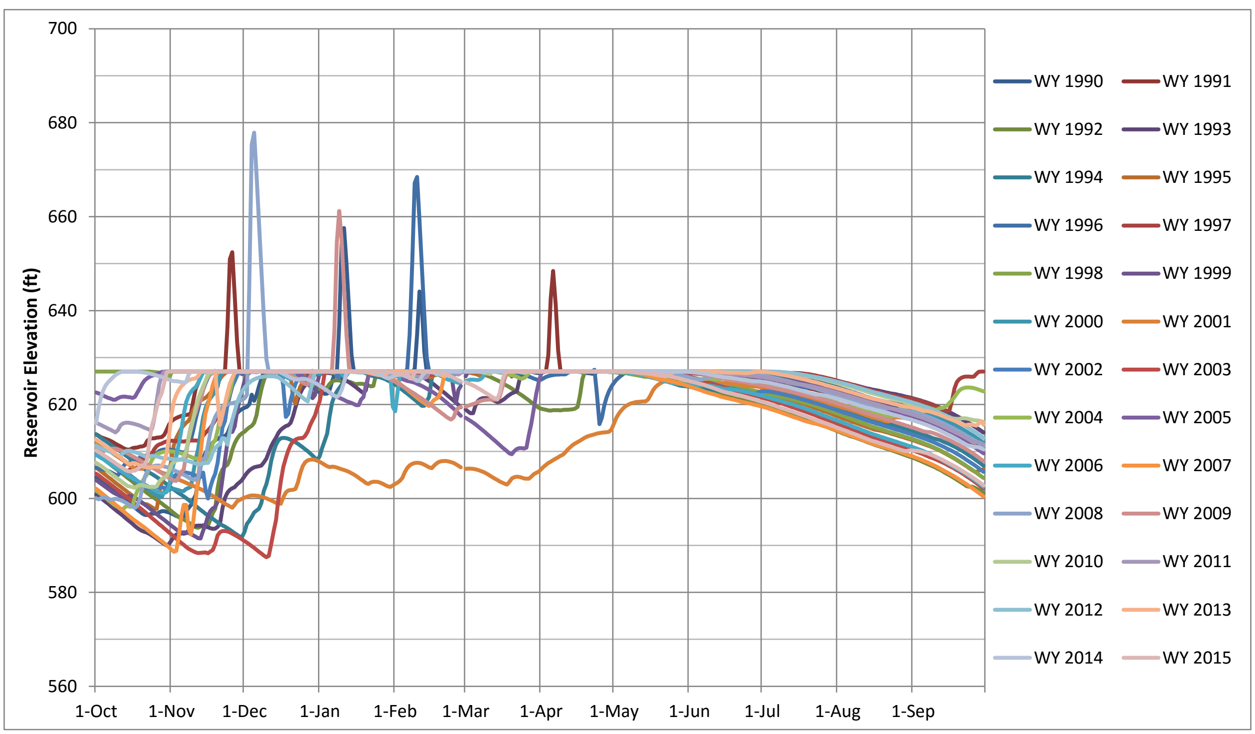


Figure 2-3

Seasonal Trends of FRFA Reservoir Elevation. Results from FRFA Reservoir Operations Modeling



3 DEVELOPMENT OF ALTERNATIVE EVALUATION FACTORS

3.1 Previous Evaluation Factors

In 2014 HDR considered multiple potential options to provide fish passage at the Chehalis Dam and evaluated these options with respect to 12 evaluation factors (HDR, 2014d):

- Attraction and access to fish passage facility
- Collection and fish passage effectiveness
- Volitional passage system
- Fish migrating/passing through reservoir
- Passage of native non-target species and multiple life stages
- Potential for fish passage evaluation or biological monitoring
- Adaptability of collection and passage
- Reliability of collection and passage
- Construction complexity and methods
- Durability of facility
- Permitting complexity

These evaluation factors were assigned a weight by the Fish Passage Subcommittee in accordance with their significance in relation to one another.

These 12 factors were developed to address the specific goal of that phase: To identify the fish passage options that can mitigate the potential impacts of such a flood retention structure and to select which options "will move forward into the next phase of alternative evaluation" (2014 Fish Passage Report). The subsequent 2014 *Combined Dam and Fish Passage Alternatives Technical Memorandum* (2014 Technical Memorandum) identified and reviewed many alternatives and recommended several alternatives for further refinement and development in the current phase.

3.2 Current Alternative Evaluation Factors

The refined alternatives presented in this report must be evaluated against factors that will help to further reduce the number of upstream and downstream passage alternatives to one or two, each, that best meet the project objectives and will be further developed in the next phase of the project. To do so, the alternatives are evaluated against a set of factors appropriate to this level of development. These factors examine potential influence of the alternatives on fish populations, their risk of under- or non-performance, and their construction and annual operations and maintenance costs: anticipated performance and survival, reliability, construction costs, O&M costs. The following subsections describe in more detail these evaluation factors for the refined alternatives.

3.2.1 Anticipated Fish Passage Performance and Survival

Each alternative is evaluated against its potential influence on the populations of each of the identified target species and life stages. The influence on each species and life stage is evaluated by investigating the passage performance and fish survival estimates for each alternative. Fish passage performance and fish survival are multiplied together to provide a total survival value. These terms were defined in the October 7, 2016 *Technical Memorandum: Rationale for Development of Performance and Survival Estimates for Anticipated Fish Passage Facilities,* and are repeated below:

- Total Survival: The total estimated percentage of fish that successfully navigate and survive the proposed fish passage facility and contribute to upstream and/or downstream life histories being considered in the EDT population response modeling.
- Performance: The proportion of fish that are anticipated to successfully navigate the fish passage facility.
- Survival : The proportion of fish that are not harmed or perish while attempting to navigate the fish passage facility.

A similar performance factor was used in evaluation of alternatives in the previous phase of development. The previous performance values were subjective, based on a multi-factor rating and scoring system, rather than on data directly obtained from the operation of similar facilities. The values developed for the Anticipated Fish Passage Performance and Survival evaluation factor in this report are based on available performance and survival data (where available) from existing facilities and lessons learned derived through years of operation by operations staff, facility biologists, and agency and consulting engineers and biologists. Further discussion of this evaluation factor and related results is provided in Attachment C: Fish Passage Performance Rationale.

3.2.2 Reliability

The reliability of the facility reflects the potential of the facility to continuously perform at peak efficiency. Peak efficiency is reflected in the performance and survival defined in Section 3.2.1. The reliability of an alternative is reduced when it under-performs or is temporarily out of service. Each alternative is evaluated based on its ability to fully meet performance objectives despite sediment and debris loads, equipment or system failures, or operator errors by providing operational redundancy and flexibility, especially during extreme or emergency conditions. Reliability will be assessed using the following metrics:

- Level of certainty that fish passage survival objectives will be met throughout the full range of environmental conditions.
- Level of certainty that fish passage performance objectives will be met throughout the full range of normal river flow and reservoir operating conditions.
- Potential for interrupted operation, such as loss of water supply throughout adverse environmental and mechanical conditions.
- Simplicity of components, including:
  - Skill level required to operate and maintain system components, such as certifications, manufacturer training, etc.
  - Level of effort and frequency required for facility maintenance, such as screen cleaning.
  - Ability to maintain the facility without dewatering or in-stream access.
- Ability to keep target species alive during emergency and unforeseen conditions.

For example:

Low – Supplying water to fish passage facilities by pumping from the tailwater is considered less reliable because it relies on multiple factors to keep operating, including mechanical components, outside electricity, emergency generators, and proper maintenance by trained technicians. Fouling of intake screens with debris is also considered less reliable because the screens can quickly become occluded, cutting off water supply, if automated cleaning systems cease to function or cannot keep up with the debris load. For sorting and handling, the use of multiple systems with complex, automated components would also be considered less reliable as their proper function is dependent on frequent maintenance by highly skilled technicians. Multiple features required to operate the facility that are considered less reliable, taken together, suggest that a low level of reliability for the alternative is appropriate.

Moderate - Supplying water to fish passage facilities using gravity is considered more reliable because it is composed of simpler components and relies less on mechanical and electrical equipment, which could fail, interrupting normal operations or eliminating the ability to supply water in emergency conditions. The gravity water supply, however, still relies on a screened intake which is subject to fowling. Intake screens are considered less reliable as described above, in the low reliability example. The sorting and handling portion of the facility uses both simple, manual systems and complex, automated systems. The simple, manual systems are considered more reliable because they require less maintenance and do not depend on electricity to operate. The complex, automated systems are considered less reliable for the reasons stated above in the low reliability example. The mix of more reliable and less reliable features, taken together, suggest that a moderate level of reliability for the alternative is appropriate.

High – Supplying the water to fish passage using gravity is considered more reliable because it is requires few components, the components it does rely on are simple, emergency power is supplied by an emergency generator, and the components can be manually operated in the case of complete power loss. The water supplied to the facility passes through a coarse trashrack directly into the system. This intake is considered more reliable as the coarse trashrack has less potential for occlusion than a fish screen and trashrack cleaning is provided by an automated system with manual backup. Finally, fish passage through the facility is considered more reliable as fish are able to move a short distance, volitionally through the facility. The facility does not require sorting and handling. It is unlikely fish will tire before fully passing through the facility. Multiple features required to operate the facility that are considered more reliable, taken together, suggest that a high level of reliability for the alternative is appropriate.

3.2.3 Cost Estimates

Proposed alternatives are also evaluated based on cost. Estimates of cost are largely based on unit price estimates developed from experience on other similar projects. Cost is broken into two categories: construction and operation and maintenance. Each category is further explained in the following subsections.

In consideration of the following subsections, Construction and Operations and Maintenance, the potential for landslides must be kept in mind. The potential for reservoir landslides could affect future costs from a capital cost perspective, as well as a long-term operation and maintenance cost perspective. This document does not explore the potential costs due to landslides in detail. If, during future development of the alternatives, it is found that these landslides require stabilization or mitigation the construction cost of the affected alternatives could substantially increase. Conversely, future development could indicate that no treatment is required and the landslides could simply be acknowledged as an ongoing operations and maintenance cost associated with the dam. In such a case, the construction cost would not need to increase due to landslide stabilization or mitigation.

3.2.3.1 Construction

Construction cost includes the capital cost and the implementation cost. The capital cost is the fixed, one-time expense, for construction of the proposed alternative. The implementation cost includes items such as engineering design, permitting, administration, and construction inspection and monitoring. An Opinion of Probable Construction Cost (OPCC) for each alternative is used to evaluate each alternative.

The methodology used in development of the construction cost opinion is a parametric estimate based the cost opinion prepared in conjunction with the technical memorandum *Combined Dam and Fish Passage Alternatives* dated October 2014. In July 2016, the 2014 cost opinion was updated based on an appraisal of the conceptual design figures and other background documents. The updated costs were compared to facilities of similar size and scope that were constructed in the last 10 years or are currently under construction. The opinion of cost for each alternative was adjusted based on this comparison and provided to the Flood Damage Reduction Technical Committee as an interim cost opinion in July 2016. The cost opinion has been updated based on further development of the alternatives since July 2016.

Based on the level of project definition and budgetary material pricing this is a rough, order-of-magnitude cost opinion. It is not comparable with a cost opinion performed to AACE guidelines. The margin for error for this cost opinion is not defined but is likely greater than L: -25% H: +50%. Based on the site conditions, limited site access, and associated difficulties, this cost opinion includes a 30% contingency.

3.2.3.2 Operation and Maintenance

Operation and maintenance costs are the costs that are incurred continuously over the life of the project. Operational costs are costs associated with items such as staffing required to keep the facilities functioning, power costs, regular debris cleaning, fish handling and transport, and periodic inspection. Maintenance costs are the costs associated with keeping system components functioning and actions that allow system components to achieve their optimal useful life, such as painting, lubrication of moving parts, repair of damage, replacement of broken or non-functional parts, and periodic inspection. The annual level of effort required to operate and maintain the facilities for each alternative was estimated using full-time equivalents for operations, maintenance, and technician positions. The annual salary and benefits for each position were developed through consideration of actual salaries for full-time employees at similar facilities. Additional non-labor costs, such as electricity usage, were estimated through consideration of actual costs for non-labor items at similar facilities.

4 DESCRIPTION OF ALTERNATIVES

4.1 Alternative Formulations

In 2014 HDR issued the *Fish Passage Design Technical Memorandum* in support of the *Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species* (2014d). The 2014 Fish Passage Report "summarize(d) key fish passage design considerations and (evaluated) initial fish passage options for the Chehalis Basin Strategy program". The options from this report were evaluated in the 2014 *Combined Dam and Fish Passage Alternatives Technical Memorandum* produced by HDR (2014 Technical Memorandum). In the 2014 Technical Memorandum, the fish passage options that were shown to have the highest likelihood of meeting the project objectives were advanced to the current phase of alternative evaluation. HDR and Anchor QEA, with input from the Fish Passage Technical Committee, have refined and developed the options advanced from 2014 throughout this phase (2015 and 2016). Key refinements made to fish passage alternatives as part of this effort are as follows:

1. The FRFA – Upstream Fish Passage Experimental Fishway and Exit Tower alternative was eliminated from further consideration. Updated flood control operations resulted in a lower seasonal range if anticipated reservoir fluctuations and therefore a more conventional linear type of ladder exit was selected for further development which has a much lower level of structural complexity, material volume, and cost.

2. The FRFA – Downstream Fish Passage Combination Collection Facilities alternative was eliminated from further consideration. The Fish Passage Technical Subcommittee agreed that the complex combination of site-specific reservoir characteristics and lack of proven track record led to a high level of uncertainty with regard to fish passage performance. Further discussion of this alternative led to the introduction of the Fixed Multi-Port Collector in its place.

3. The FRFA – Downstream Fish Passage Fixed Multi-Port Collector was added as a downstream fish passage option. This option had not yet been developed until this most recent phase of work. Although never constructed as proposed, the cost estimate for this option is largely based on the fixed multi-port collection system and helical bypass currently under construction by the USBR at Cle Elum Dam, Washington. The estimated cost was assumed to be about 30% less than the Cle Elum outlet based upon the relative size and complexity of construction.

4. Upon refinement of the Technical Fishway and elimination of the Experimental Exit Tower option (described above), the fish ladder exit for this alternative is more complex than previously assumed to accommodate the range of anticipated reservoir fluctuations. The cost estimate has been updated to reflect this complexity.

5. The Fish Passage Subcommittee made several decisions relating to the Forebay Collector that changed the original assumptions regarding the operation of the facility. The changes in operation changed the size of the facility. These decisions have also provided additional direction, allowing the design to be developed further. These changes resulted in an increase in the estimated construction and operation and maintenance costs of the Forebay Collector. Some of the specific changes include:

- Nearly doubling the attraction flow of the floating surface collector facility up to a potential attraction flow of 1,000 cfs.
- The fish guidance and lead nets that are about 2.5 times longer than previously considered to accommodate a different collection barge position and guidance approach angle.
- Several supporting structure and facilities have been added to make the fish passage and handling facilities reflect the latest practice and experience.
- The collection barge mooring and anchorage components were expanded to accommodate a normal operational range of 40 vertical feet and an emergency range of 189 feet for flood control events.

The refined alternatives are presented in this Section of this report. The alternatives in this report, the 2014 options from which they originated, and the species and life stages they are intended to focus on are summarized in Table 4-1 below.

Table 4-1
Design Species, Lifestage, and Movement Direction Design Focus by Alternative

ALTERNATIVE	ASSOCIATED OPTION FROM 2014 FISH PASSAGE REPORT	UPSTREAM OR DOWNSTREAM PASSAGE	ADULT SALMON & STEELHEAD	RESIDENT FISH	JUVENILE MIGRATING SALMON/ STEELHEAD	LAMPREY
FRO						
Fish Passage Conduits	Run of River Tunnels	Upstream, Downstream	x	x	x	x
CHTR	CHTR	Upstream	x			
FRFA						
Technical Fish Ladder	Technical Fishway & Exit Structure	Upstream	x	x	x	x
CHTR	CHTR	Upstream	x	x	x	x
Floating Surface Collector	Floating Forebay Collector	Downstream	x*	x	x	
Multi-Port Collector	Fixed Collection Facility	Downstream	x*	x	x	

Note:
*Post-spawn adult steelhead only

4.2 Fish Passage Alternatives for the Flood Retention Only Dam (FRO)

One set of fish passage alternatives was carried forward in the design process to accommodate both upstream and downstream fish passage for the FRO dam option. Fish passage is provided primarily through the integration of three multi-use conduits occurring through the base of the dam. The conduits are to be used in combination with flood control operations. When flood events occur, fish passage is no longer accommodated through the conduits and a separate Collect, Handle, Transport, and Release facility (CHTR) is provided until flood operations cease. A description of the proposed fish passage alternatives for the FRO dam option is provided in the following paragraphs.

4.2.1 FRO Conduits

The FRO conduits provide a route for adult salmon and steelhead, resident fish, and lamprey to volitionally pass upstream and downstream of the FRO dam. The FRO conduits consist of two 10-foot-wide by 16-foot-high and one 12-foot-wide by 20-foot-high (Figure 4-2) rectangular tunnels through the base of the FRO dam. All three conduits are designed to mimic the hydraulic and sediment conveyance characteristics of the natural river channel at the dam location. During normal flow, the river is conveyed in an open-channel hydraulic condition through the conduits. A bed of natural substrate forms on the floor of the conduits under these conditions. During high flow periods, water is impounded behind (upstream of) the dam, the conduit gates are mostly closed to significantly reduce flow downstream, and fish passage through the conduits is stopped by the resulting hydraulic conditions. When the conduits are impassible, upstream fish passage is provided via the CHTR facility (see Section 4.2.2).

4.2.1.1 Design Elements

The FRO conduits consist of two primary design elements:

- The 10’x16’ conduits
- The 12’x20’ conduit

Figure 4-1
FRO Conduits Plan View

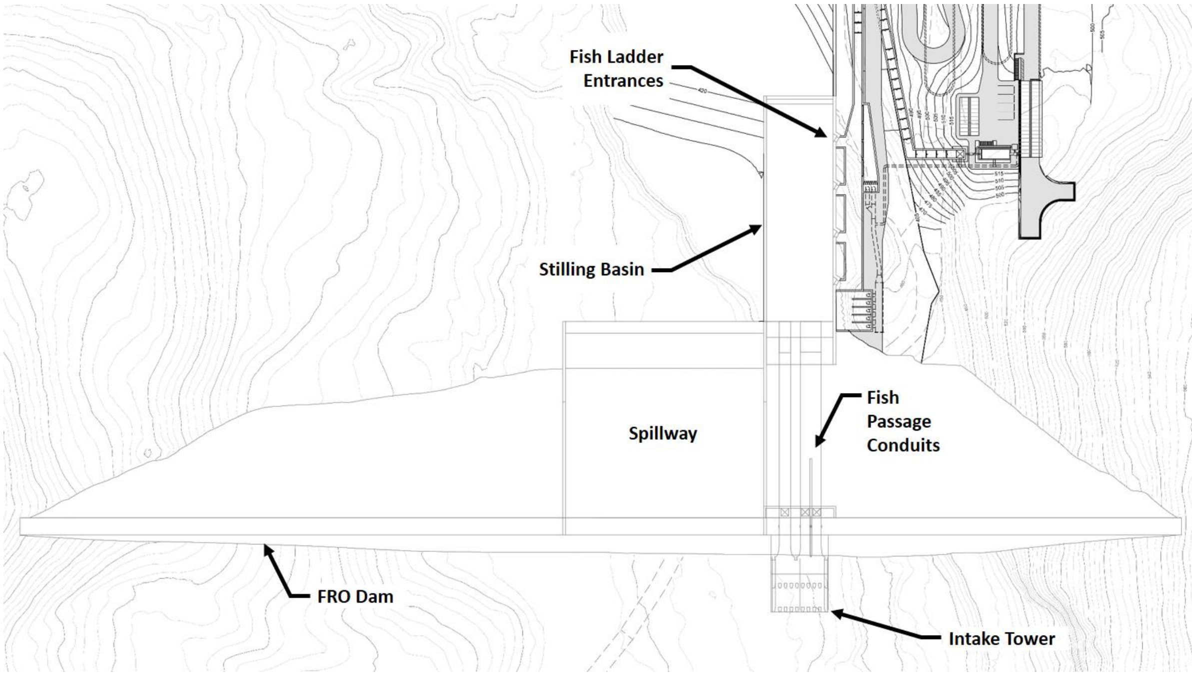
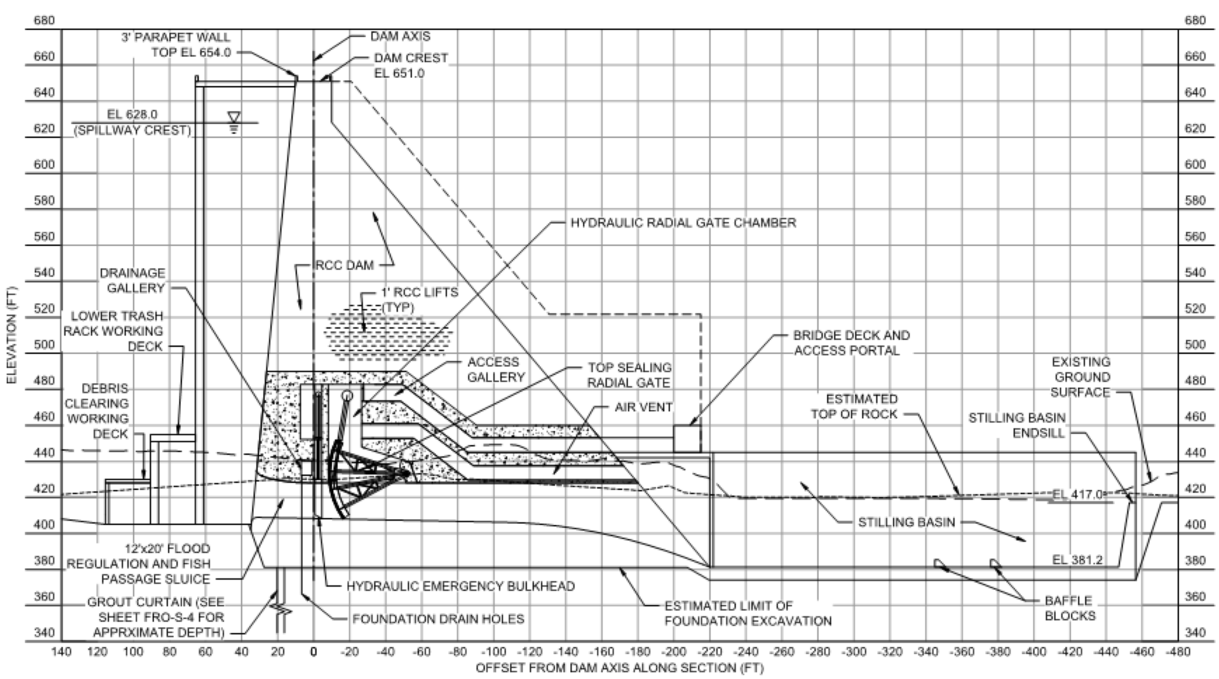


Figure 4-2
FRO Conduit Profile



4.2.1.1.1 The 10'x16' Conduits

Two 10-foot-wide by 16-foot-high by 100-foot-long concrete conduits are cast in place at the base of the FRO dam. The conduits are rectangular with enlarged, radiused ceilings and sidewalls at the entrances at the upstream face of the dam where the conduits are open to the reservoir (Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017), Figures FRO-S-3 and FRO-S-7). Both conduits have an approximate invert elevation of 411.0 at the centerline of the dam. Each conduit is slightly sloped toward the stilling basin downstream. Each conduit contains a radial gate capable of shutting off flow through the conduits.

The conduits are located side-by-side and are separated by wall (Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017) , Figure FRO-S-3). At the downstream end of the conduits, the separating wall ends and a single conduit approximately 22 feet wide by 130 feet long continues to the downstream face of the dam where the conduit is open to the stilling basin. As the wider, single conduit approaches the stilling basin the ceiling maintains its slope while the floor curves smoothly down to the elevation of the stilling basin just prior to the downstream face of the dam.

4.2.1.1.2 The 12'x20' Conduit

One 12-foot-wide by 20-foot-high by 230-foot-long concrete conduit is cast in place at the base of the FRO dam. The conduit is rectangular with an enlarged, radiused ceiling and sidewalls at the entrances at the upstream face of the dam where the conduit is open to the reservoir (Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017), Figures FRO-S-3 and FRO-S-7). The conduit has an approximate invert elevation of 408.0 at the centerline of the dam. Similar to the 10'x16' conduits, this conduit is slightly sloped toward the stilling basin downstream. The conduit contains a radial gate capable of shutting off flow through the conduits.

The conduit is located adjacent to and southwest of the 10'x16' conduits. At the downstream end, the conduit is open to the stilling basin. As the conduit approaches the stilling basin the ceiling maintains its slope while the floor curves smoothly down to the elevation of the stilling basin just prior to the downstream face of the dam.

4.2.1.2 Theory of Operation

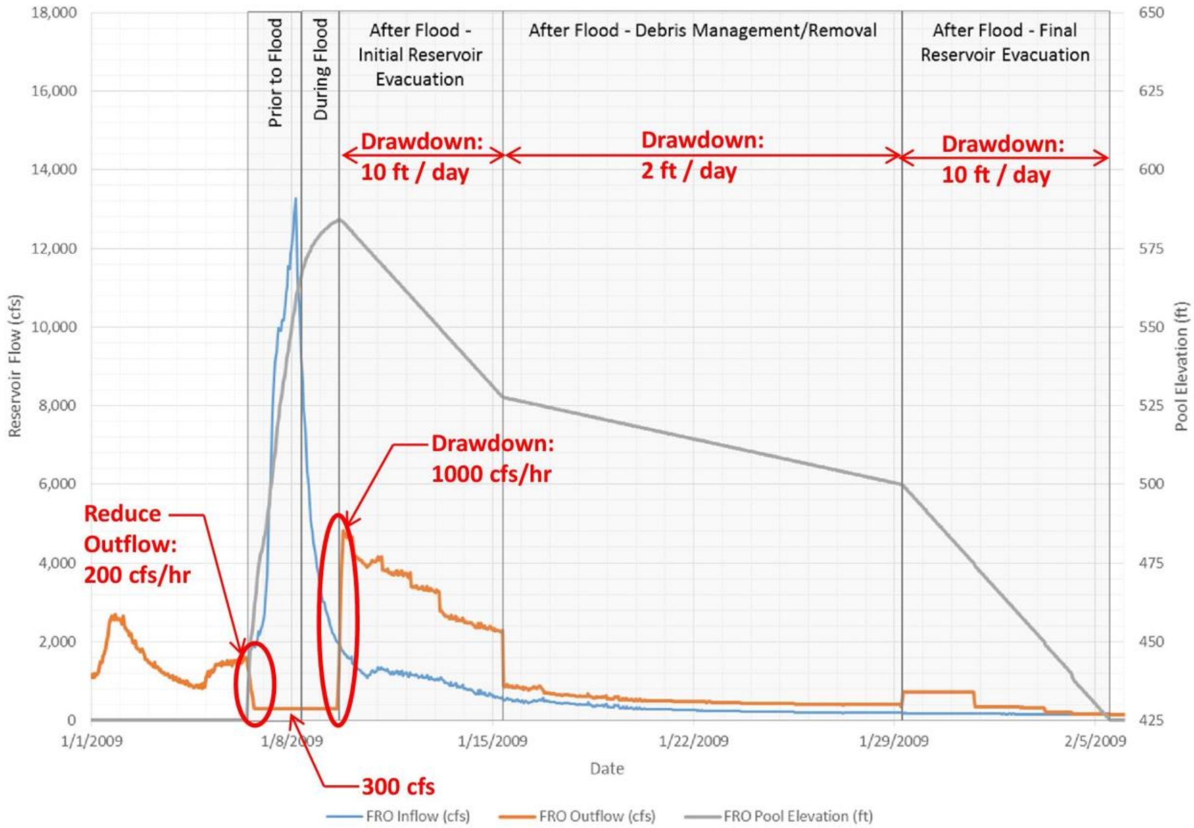
The FRO conduits are intended to provide year-round, safe, volitional upstream and downstream passage for migrating adult salmon and steelhead, resident fish, and lamprey for the full range of flow conditions up through the high fish passage design flow as required by NMFS criteria. The high fish passage design flow is 2,200 cfs. Hydraulic modeling results indicate the conduits provide depths and velocities similar to those of the natural river reach throughout the majority of the fish passage design flow range.

The Chehalis River channel at the proposed location of the FRO dam is a natural rectangular channel incised in hard rock up to about a 4,000 cfs river flow. Above 4,000 cfs the river rises above the incised rock channel and begins to widen across an existing terrace. The narrow and deep rectangular rock channel creates natural river velocities that are well in excess of the 2-feet-per-second fish passage velocity suggested by NMFS for fish passage design. However, it was agreed among the stakeholders that mimicking the natural hydraulic conditions was the most appropriate approach for the design of the conduits, in part because the incised rock channel remains upstream and downstream of the dam after the dam is constructed.

Hydraulic analysis of the conduits and the reaches of natural stream channel above and below the proposed dam shows a typical water depth of 6 feet in the two 10-foot-wide by 16-foot-high conduits and a water depth of 9 feet in the 12-foot-wide by 20-foot-high conduit. The conduits are anticipated to replicate the natural river depths and velocities exhibited by the natural channel through which fish will pass whether the dam is in place or not up throughout the range of targeted fish passage flows. Sediment transport analysis of the conduits shows that the invert of the conduits will be bedded with natural sediment during normal operations. During higher flow events above the anticipated fish migration flows and during flood retention scenarios, the sediment is anticipated to flush out of the conduits. Material will naturally begin to mobilize out of the conduits as discharges rise above 4,000 cfs or by higher flow velocity occurring under the radial flood control gates as the gates are closed. A more detailed hydraulic assessment of fish passage and hydraulics through the conduits is presented in Attachment B.

Flood regulation operation would be initiated whenever the downstream river flow at the Grand Mound gage site is forecast to increase above 38,000 cfs within 48 hours. Flood regulation operations commence by gradually closing the radial gates to reduce the outflow to the minimum value of 300 cfs over the specified downstream ramping rate period. Once the flood has passed, reservoir evacuation operations will begin. Figure 4-3 below illustrates how the FRO dam would have been operated had it been in place during the January 2009 flood event. The reservoir is drawn down as quickly as possible while continuing to minimize flood risk downstream and protect the fluvial ecosystem. However, drawdown of the reservoir slows upon reaching WSEL 528, to allow time for floating flood debris to be collected and floating to a sorting area where it can be readily removed after the reservoir recedes. The operations model run by Anchor QEA indicates that flood events will occur on average once every 5 to 7 years (Anchor, 2016). Fish passage through the conduits may be delayed during flood operations anywhere from just over two weeks to nearly 60 days.

Figure 4-3
FRO Dam Hourly Flows and Elevations for the January 2009 Flood Event



4.2.1.3 Maintenance

Maintenance of the FRO conduits will be required. Maintenance activities can be broken into regular, annual, and infrequent periods.

Regular maintenance for the FRO conduits refers to maintenance that is required regularly following each impoundment event and as needed after substantial freshets that could occur on a more frequent basis. While operating as run-of-river (no impoundment) no regular maintenance is anticipated. Impoundment events, and thus more regular maintenance, may occur a few times annually. Regular maintenance includes:

- Debris removal from the upstream trashracks. Debris removal from trashrack of screen structure in operation. Three landings for debris removal operations are shown in below.
- Observation of the FRO conduits for debris inside the conduits, damage to the conduits, or evidence to damage to the radial gates.
- Inspection of the trashracks for damage.

Figure 4-4
FRO Dam, Taken Upstream, Looking Downstream

Regular cycles of submergence and exposure as well as sluicing of sediment during the drawdown of impoundments puts additional wear on the equipment, metal work, and concrete walls and floors in the FRO conduits. It is expected maintenance tasks that may ordinarily be undertaken every few years on an exposed structure and not subject to regular sluicing will need to occur annually on the conduits. Annual maintenance tasks may include:

- Repaint steel structures, including trashrack and radial gates.
- Inspect concrete floor and sidewalls of the conduits. Repair as necessary.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Service gate operators.
- Substantial repair concrete floors and sidewalls of the conduits. This includes dental removal of the existing concrete floors and pouring new concrete floors.
- Repair radial gates.
- Replace gate operators.

4.2.1.4 Anticipated Fish Passage Performance and Survival

Table 4-2 shows the anticipated fish passage performance and survival for the FRO conduits. A discussion of the rationale behind performance and survival value is provided in Attachment C of this document.

Table 4-2
FRO Conduits Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	95%	99%	**94%**
Fall Chinook	95%	99%	**94%**
Coho	95%	99%	**94%**
Winter Steelhead	97%	99%	**96%**
Coastal Cutthroat	93%	99%	**92%**
Pacific Lamprey	97%	99%	**96%**
Western Brook Lamprey	97%	99%	**96%**
JUVENILE UPSTREAM			
Spring Chinook	65%	99%	**64%**
Fall Chinook	65%	99%	**64%**
Coho	65%	99%	**64%**
Winter Steelhead	80%	99%	**79%**
Coastal Cutthroat	65%	99%	**64%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
ADULT DOWNSTREAM			
Spring Chinook	-	-	**-**
Fall Chinook	-	-	**-**
Coho	-	-	**-**
Winter Steelhead	98%	75%	**74%**
Coastal Cutthroat	98%	80%	**78%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
JUVENILE DOWNSTREAM			
Spring Chinook	100%	85%	**85%**
Fall Chinook	100%	85%	**85%**
Coho	100%	85%	**85%**
Winter Steelhead	100%	95%	**95%**
Coastal Cutthroat	100%	85%	**85%**
Pacific Lamprey	100%	95%	**95%**
Western Brook Lamprey	100%	95%	**95%**

There are no known existing conduits similar to the proposed FRO conduits that could assist in predicting fish passage performance and survival in the FRO conduits. The likely surrogate for a technology of this nature is fish passage through culverts, which has been studied extensively. In addition, some studies at Mud Mountain Dam provide information on the success of out-migrant fish through similar conduits. Performance and survival through the FRO conduits is based on the success of out-migrants at Mud Mountain Dam and the performance of fish passage through culverts, but are adjusted based on conditions that are unique to the FRO dam on the Chehalis River. The following is a summary of the information collected regarding the potential performance of conduits similar to the FRO conduits:

- In general, there are few examples of conduits through dams that are configured for the purpose of fish passage. No known conduits of this nature have been identified in a similar situation for the purposes of upstream passage. The likely surrogate for a technology of this nature would be fish passage through culverts, which has been studied in detail over the past several decades. Culvert fish passage information exists with regard to design rationale, guidelines, and velocity targets. Available information suggests that passage through long conduits of this nature can be successful when velocity and depth criteria are met.
- Design guidelines are readily available for adult salmonid upstream passage. Guidelines and swim capabilities for juvenile upstream passage can be derived from the literature, but formal design guidelines are not available.
- Mud Mountain Dam is an example of successful routing out-migrant fish downstream through a similar type of conduit. Available information suggests that the performance levels and survival for outmigrating juveniles is high as long as velocity criteria are met and the conduit is kept clear of debris and free from sharp edges protruding into the water column. Mud Mountain Dam is on the White River in Washington State.
- A review of literature characterizing the results of barotrauma studies suggests that out-migrants can be passed through partially open radial gates or open valves with a high level of survival when fish are being passed downstream at 1 atmosphere or approximately 34 feet of static water depth. Survival was documented to incrementally decrease as depth and/or pressure increased or as valve openings decreased.

The anticipated performance and survival values in Table 4-2 have been adjusted to account for a number of site-specific factors, including:

- It is assumed that the performance and survival values are provided for periods of time when the fish passage conduits are open. The CHTR facility would be operated when flood retention gates are closed. Performance and survival would then default to those values provided for CHTR during periods of flood retention.

- Passage performance is largely a function of the engineering design and capability to provide adequate depths and velocities.
- Roughness elements are planned in the larger center conduit, which would provide a corridor of water velocity suitable for juvenile upstream migration more often.
- Larger adult salmonids were given a higher level of performance and survival for upstream migration than juveniles, given that hydraulic criteria for juvenile fish would be met less often.
- Juvenile salmonids were also given a high level of performance for outmigration. Survival for outmigrating juveniles was lowered slightly in light of the potential interaction with the upstream trashrack. It was assumed that if debris loading occurs, juvenile fish would be more susceptible to being swept into a debris-laden trashrack, which may cause more injury or mortality.
- Inlet and outlet conditions are anticipated to impact juvenile survival during downstream migration through hydraulics, predation, and other factors. Therefore, weaker swimming fish such as cutthroat trout, Chinook, and coho have slightly lower survival rates than those of steelhead.
  - Outmigrating post-spawn adult steelhead are less energetic and possibly more susceptible to injury during downstream migration and are also given a slightly decreased survival.
  - Juvenile winter steelhead are less dependent upon the hydraulic fringe. They generally exhibit a larger size and better swimming ability, which makes them more capable of ascending the conduit.
  - Juvenile steelhead are more capable of handling the varied hydraulic conditions in the conduit as well as other factors at the inlet and outlet. Predation is less of a factor for them than for other species.

4.2.1.5 Reliability

The FRO conduits are expected to have a high level of reliability due to their low potential for mechanical failure and relative simplicity of operation, and because the fish remain in the river channel. The FRO dam operates primarily as a run-of-river dam with the conduits serving as the pass-through channel. Except during infrequent ponding events the gates are open and the conduits remain unobstructed by mechanical equipment. Trashracks at the upstream end of the conduits collect debris that could otherwise occlude the tunnels. Access decks to the upper and lower trashracks allow for simple and frequent cleaning to minimize occlusion of the inlets and fish injury due to debris. The infrequent use of the gates and their primary position out of the flow significantly reduces the likelihood of mechanical failure. In addition, since the gates also serve as the primary shutoff mechanism for flow, they are critical to dam safety and are more likely to be well maintained and inspected. The simplicity of operation and low potential for mechanical failure provide a high level of certainty that the FRO conduits will perform as intended throughout the range of environmental conditions.

The conduits are also the only alternative in which fish are not taken off-channel during normal operating (passage) conditions. The lack of handling and man-made off-channel passage systems significantly reduces the potential for under-performance caused by delay, injury, and mortality and non-performance caused by a complete shutdown of the passage facilities due to environmentally or mechanically adverse conditions, such as debris clogging or a cut-off or reduction in water supply, respectively. Additionally, all the water in the river passes through the conduits during normal operation and at ponding events, giving a very high level of certainty that the water supply for fish passage will be maintained through the range of environmental, and most mechanical, conditions.

Reliability of the FRO conduits is somewhat reduced by ponding events that occur several times a year, on average. During ponding events flow through the FRO conduits is substantially reduced and managed for flood attenuation in such a way as to preclude fish passage. Fish passage may be precluded for several days to several weeks through the conduits. If coupled with the FRO CHTR alternative, upstream passage is maintained through flood attenuation events. However, no avenue for downstream passage is provided during these periods.

4.2.1.6 Cost Summary

The FRO conduits are the least expensive of all the fish passage alternatives. The FRO conduits serve a dual purpose – flood control and fish passage. Without fish passage, the FRO conduits are still required for operation of the FRO dam. As such, the costs of constructing and operating the conduits are included as part of the FRO dam and are not broken out separately. Small modifications to the design of the conduits to accommodate fish passage provide substantial savings to the overall cost for FRO dam and fish passage construction.

4.2.2 Collection, Handling, Transport, & Release Facility (CHTR)

The primary means of upstream and downstream passage at the FRO dam is via the conduits. When water is impounded behind the FRO dam during high flow events, the conduits are closed, and fish passage is provided via Capture, Handling, Transport, and Release Facility (CHTR), as described in Section 4.2.1.2. Resident and juvenile fish and lamprey have varied life histories that can accommodate infrequent interruptions in upstream and downstream passage of moderate duration. The CHTR is a fish passage alternative intended to collect migrating adult salmon and steelhead moving upstream and safely transport them upstream of the FRO dam. Although the facility is not designed specifically for upstream migrating juvenile salmon and steelhead, its design does not exclude them. Upstream migration of juvenile species through trap and transport facilities has been documented and is expected. If juvenile species do enter the CHTR facility, means of holding and transport are provided.

- The CHTR facility is not anticipated to operate for most of the year when implemented in conjunction with the FRO dam configuration. When flood control scenarios require its operation, it is a staffed facility that is operated 24 hours a day until flood operations cease and passage through the conduits resumes. The CHTR for the FRO dam is similar to the CHTR facility for the FRFA dam, as described in Section4.3.2, with a few exceptions. The following subsections focus on describing the differences between the FRO CHTR and the FRFA CHTR facilities.

4.2.2.1 Design Elements

The CHTR associated with the FRO dam is shown in Figure 4-5 below and in detail in Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017), Figures FRO-C-1 through FRO-C-4. The FRO CHTR is similar to that proposed for the FRFA dam presented in Section 4.4.2. However, differences in the operation, conditions, and fish passage alternatives associated with the FRO dam result in some differences in the design elements described for the FRFA dam CHTR:

- The FRO dam CHTR has a longer fish ladder entrance pool than the FRFA configuration. The fish ladder entrance is configured optimally in consideration of access, fish attraction, and constructability constraints. In general, the ladder entrance is in the same location for both the FRO and FRFA dam CHTR alternatives. However, the lower height of the FRO dam sets the stilling basin further upstream and the fish ladder entrance pool is elongated to conform to these stilling pool dimensions.
- Lamprey passage is not required in the FRO CHTR alternative, so facilities for lamprey passage are not included. Lamprey passage is accommodated through the FRO conduits during normal operational scenarios when no pool is present upstream of the dam structure. Lamprey provisions are provided for the CHTR facility in conjunction with the FRFA dam configuration.
- The FRFA dam configuration includes technical provisions for the collection and transport or bypass of downstream migrating fish from the reservoir, whereas migrating fish passing the FRO structure are passed downstream through open conduits. Therefore, provisions for receiving, evaluating, and passing downstream migrating fish are unique only to the FRFA CHTR facility and are not provided in the FRO CHTR facility.
- Numerous components of the FRO CHTR facility (e.g. sorting, holding, workup, and transfer facilities, as well as the electrical/mechanical and storage buildings) will be vacant and unmonitored for long durations. As such, the FRO CHTR facilities are provided with additional security measures befitting their mostly vacant status.

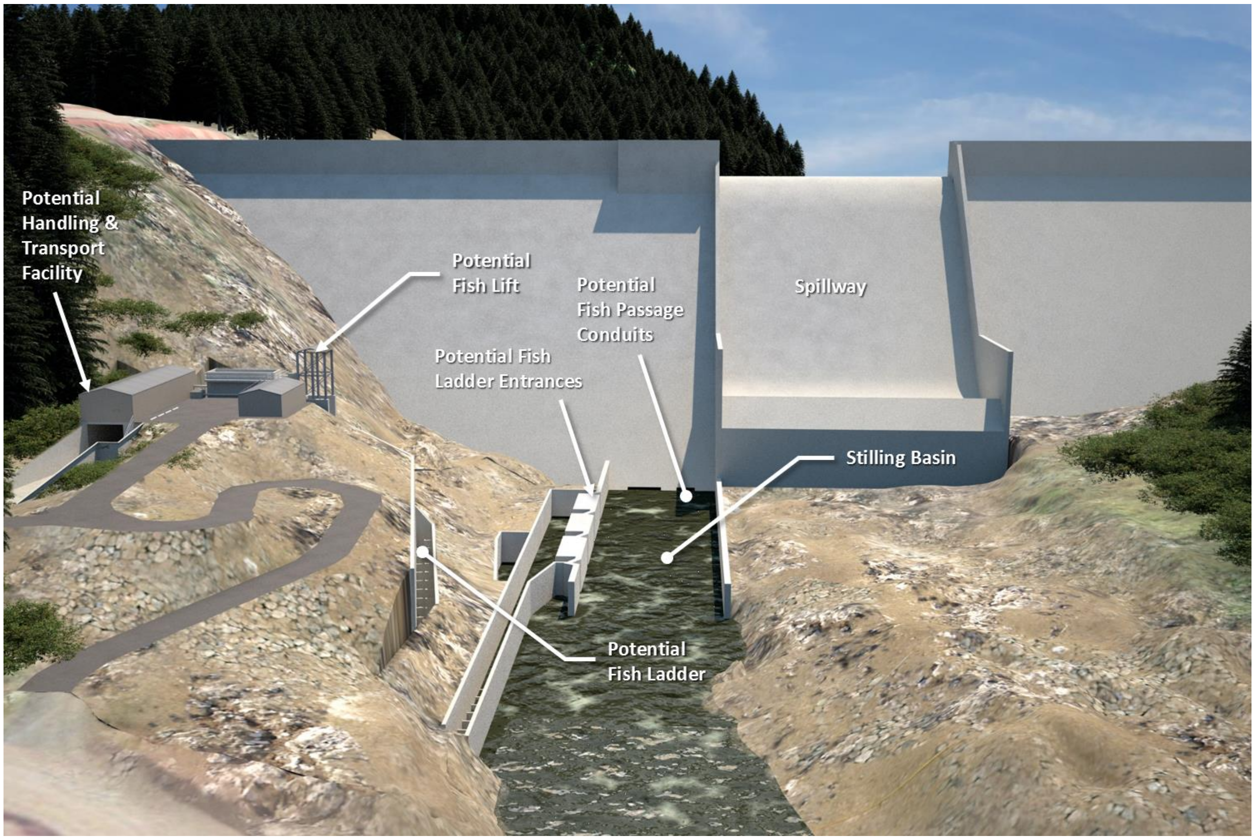
4.2.2.2 Theory of Operation

The CHTR associated with the FRO dam operates the same way as the CHTR proposed for the FRFA dam. However, differences in the operation, conditions, and fish passage alternatives associated with the FRO dam result in some differences in the theory of operation described for the FRFA dam CHTR:

- Upstream and downstream fish passage is provided through the conduits for the majority of the year. When water is impounded behind the FRO dam during high flow events, the conduits are closed, and fish passage for adult salmonids is provided via the CHTR facility throughout flood scenario operations. While in operation, the FRO CHTR facility is staffed daily to perform upstream transport activities until passage through the conduits resumes. As flood operations cease and the flood pool recedes, upstream and downstream passage of all species and life-stages resumes through the conduits and the FRO CHTR facility performs shut-down procedures.

- Upstream and downstream lamprey passage is provided via the conduits when they are open. The infrequent interruptions in lamprey passage are expected to have minimal adverse impact on the populations. Therefore, facilities for lamprey passage are not provided in the FRO CHTR alternative.
- Downstream passage of juvenile salmon and steelhead is provided via the conduits when they are open. The infrequent interruptions in juvenile salmon and steelhead downstream passage are expected to have minimal adverse impact on the populations. Therefore, alternate facilities for juvenile salmon and steelhead downstream passage are not required or provided.
- Upstream passage of juvenile salmon and steelhead is provided via the conduits when they are open. The infrequent interruptions in juvenile passage are expected to have minimal adverse impact on the populations. Therefore, alternative upstream passage of juvenile fish passage is not required in the FRO CHTR alternative. However, upstream migration of juvenile species through trap and transport facilities has been documented and is expected to occur at some level during FRO CHTR operations. Although the CHTR is not specifically designed for upstream passage of juveniles, juveniles may pass through the facility and is expected to occur to some degree. The same holding, sorting, and transport facilities for juveniles that are included in FRFA CHTR are included in the FRO CHTR, with the exception of facilitating the offloading of juvenile transport tanks from the floating surface collector (FSC).

Figure 4-5
FRO Dam, Taken Downstream, Looking Upstream, During Normal Operation of the Conduits



4.2.2.3 Maintenance

Maintenance activities can be separated into regular, annual, and infrequent maintenance.

Regular maintenance of the CHTR during the course of a year will not be required as the facilities will primarily lie dormant. Regular maintenance will only be required during operation of the CHTR. As this facility will only be operated for short to moderate durations once every few years, regular maintenance also includes maintenance items associated with startup and shutdown of the CHTR. Some regular maintenance items may also occur outside of regular operation as a result of annual inspections, to keep the facility ready for operation on short notice. Regular maintenance will include:

Startup

- Inspect and operate motors, gates, gate actuators, and other equipment. Clean and lubricate as necessary. Ensure equipment is in good working order before beginning fish passage operations.
- Clean all tanks, holding galleries, work-up tables, flumes, and other holding and sorting facilities.
- Inspect electrical equipment.

- Test all lighting and replace bulbs as necessary.
- Inspect all doors and fence gates. Lubricate as necessary to ensure they are in good working order.
- Clean debris from access roads and walkways.

During Operation

- Debris removal within the fish ladder and fish lift areas. Supply water is screened so debris is expected to come primarily from falling debris, such as leaves.

Shutdown

- Service manual and motorized gate operators.
- Inspect, service, and replace if necessary, all gates, crowders, and other fish handling and sorting equipment pumps.
- Inspect fish screens and trashracks for damage.
- Shutdown and depower all equipment not requiring power during dormant periods.
- Move all loose equipment, parts, tools, and other items necessary for operation into the Storage Building.
- Set lighting and security systems as required during vacancy.
- Secure Storage Building, Electrical/Mechanical Building, and the site as required during vacancy.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Service trapping and lift systems, including repaint steel parts and service motors.
- Repaint buildings and steel pipes and structures.
- Replace trash rakes.
- Replace gate operators, trapping system components, lift system components, crowder motors, and other fish handling and sorting equipment.
- Replace Bypass Pipe(s) between Bypass Hoppers and the dam penetration as necessary due to damage from exposure, debris, or flood events.
- Replace in-line energy dissipation valve and other shutoff and control valves as necessary.

A summary of the anticipated level of effort for operation and maintenance is provided in the table below. Level of effort for the FRO CHTR has been adjusted to reflect operation of the FRO CHTR facility once every several years.

Table 4-3
Operation and Maintenance Level of Effort for the FRO CHTR Facility

ALTERNATIVE	AVERAGE ANNUAL DURATION	PERSONNEL REQUIREMENTS (FTE = FULL TIME EMPLOYEE)	SPECIAL CONSIDERATIONS
FRO – CHTR Facility	0.3 months	Operations Staff - 0.03 FTE Biological Staff - 0.02 FTE Maintenance Staff - 0.04 FTE Total = 0.09 FTE	• Operation of CHTR facility daily during operational periods. • Operation of transport vehicle required daily during CHTR operation. • Annual inspections and maintenance required to ensure operable during infrequent ponding events.

4.2.2.4 *Anticipated Fish Passage Performance and Survival*

The anticipated fish passage performance and survival for the FRO CHTR fish passage alternative during its periodic operational periods, based on the performance of other CHTR facilities and adjusted based on site-specific conditions is shown in Table 4-4. A discussion of the rationale behind performance and survival value is provided in Attachment C of this document. There are numerous examples of trap and transport facilities in the Pacific Northwest that collect and transport adult anadromous salmonids with high levels of performance and with very low levels of injury or direct mortality. The FRO CHTR facility is anticipated to perform similarly to the FRFA CHTR facility for adult upstream migration of adult steelhead and salmon. Given that provisions for lamprey and juvenile upstream migrating fish are not provided for these short durations, passage performance is reduced from that shown for the FRFA CHTR facility.

Table 4-4
FRO Capture, Handling, Transport, and Release (CHTR), Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	93%	98%	**91%**
Fall Chinook	93%	98%	**91%**
Coho	93%	98%	**91%**
Winter Steelhead	93%	98%	**91%**
Coastal Cutthroat	55%	98%	**54%**
Lamprey	0%	90%	**0%**
JUVENILE UPSTREAM			
Spring Chinook	55%	90%	**50%**
Fall Chinook	55%	90%	**50%**
Coho	55%	90%	**50%**
Winter Steelhead	60%	90%	**54%**
Coastal Cutthroat	50%	90%	**45%**

Some of the site-specific conditions accounted for include:

- Modern adult collection facilities are typically designed for the collection of adult upstream migrating salmonids. As such, the FRO CHTR adheres to the design guidelines for adult salmonid passage. Provisions for juvenile, resident, and lamprey passage are not provided for by the FRO CHTR alternative. Therefore, it is anticipated that juvenile and resident passage will be reduced and lamprey passage will cease until the FRO flood operations cease and the conduits reopen.
- There is a higher level of confidence that a CHTR facility will perform well for upstream migrating adult Chinook, coho, and steelhead. Like facilities had performance and survival values around 90 percent and 98 percent, respectively.
- Reduced performance and survival values were provided for cutthroat trout compared with adult Chinook, coho, and steelhead. The reduced performance value is most substantially attributed to the general focus of other facilities on adult salmonids and the lack of data on cutthroat trout collection.

4.2.2.5 Reliability

The CHTR for the FRO dam provides a moderate level of reliability. While a high level of skill is not required to operate a CHTR facility and such facilities have a long history of successful operation, many aspects of the facility reduce the certainty that the facility will perform optimally under all conditions. The water supply pumps, single-lift hopper, fish sorting gates, and truck transfer mechanisms do not require special certifications to operate and can be operated successfully with minimal on-the-job training. Many of the trap and transport facilities elsewhere in the country operate using the same mechanical systems and have had consistent success. However, the same facilities also succumb to mechanical and operational failings that reduce their reliability.

Mechanical systems require regular maintenance and can fail or perform sub-optimally with little notice, reducing reliability. For instance, the water supply for the CHTR is pumped from the tailwater to the various tanks, flumes, and hose bibs to maintain flow and fish health. As with any pumped system, there is the possibility for occlusion of the intake screens, pump failure, or piping failure. Some redundancy is provided in the electrical system for the pumps via an emergency generator. Similar mechanical failures can occur in valves, weirs, gates, and other water and fish transfer connections throughout the handling and transport systems. Failures in any one of these components will likely not prevent the CHTR from passing fish they may reduce the performance.

CHTR facilities, like all trap and transport facilities, are subject to operational mistakes that have the potential to reduce the reliability of the CHTR facility. However, complete shutdown of passage is unlikely.

4.2.2.6 Cost Summary

The estimated construction cost of the FRO CHTR is the same as that of the FRFA CHTR (Section 4.3.1.5) since they serve the same function. A summary of the estimated construction costs, included the estimated upper and lower bounds, is provided in Table 4-5 below. A detailed breakdown of construction costs can be found in Attachment D.

CHTR facilities are fully manned facilities that require more operations personnel and man hours to operate than other upstream passage facilities. However, the short duration and episodic operation of the CHTR significantly reduces the operation and maintenance cost of this upstream passage facility compared to its FRFA counterpart. A summary of the estimated O&M cost is provided in Table 4-5 below. A detailed breakdown of O&M costs can be found in Attachment D.

Table 4-5
FRO CHTR Estimated Construction and O&M Costs

FISH PASSAGE OPTION	CONSTRUCTION COSTS			O&M COSTS
	LOWER BOUND COST ($ MILLION)	MIDDLE BOUND COST ($ MILLION)	UPPER BOUND COST ($ MILLION)	($)
FRO – Upstream Fish Passage: CHTR Facility	$13.8	$18.4	$27.6	$20,000

4.3 Flood Retention, Flood Augmentation Dam (FRFA)

Two downstream and two upstream fish passage alternatives were further refined and evaluated for the FRFA dam alternative, based upon the specific physical configuration and operational parameters anticipated for the FRFA dam alternative. The FRFA fish passage alternatives included the following:

- Upstream Fish Passage – Technical Fish Ladder
- Upstream Fish Passage – CHTR Facility

- Downstream Fish Passage – Floating Surface Collector
- Downstream Fish Passage – Fixed Multi-Port Collector

A brief summary of the primary alternative intent, components, and performance characteristics are provided in the following subsections.

4.3.1 Technical Fish Ladder

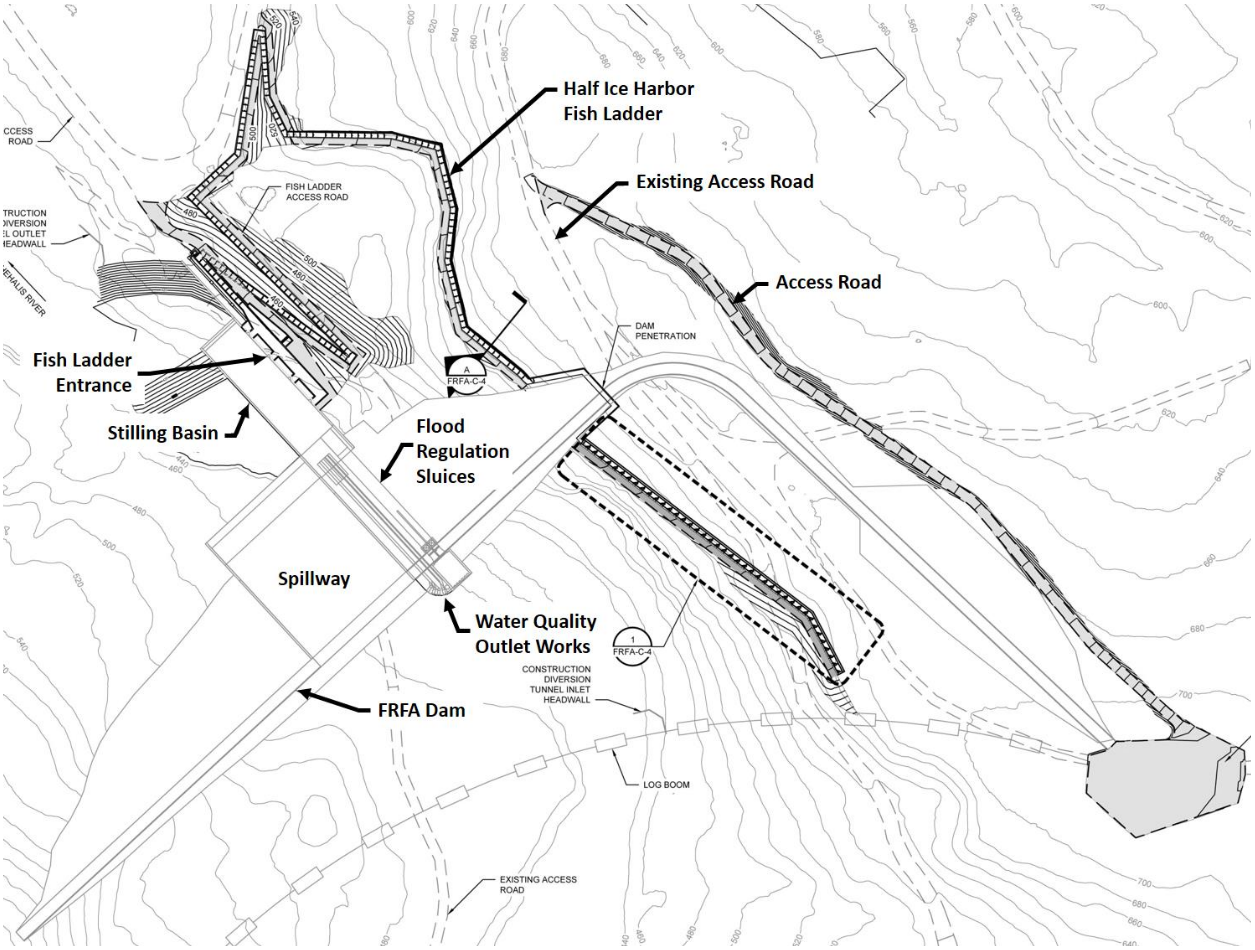
The Technical Fish Ladder alternative is intended to provide a route for adult and juvenile salmon and steelhead, resident fish, and lamprey to volitionally pass upstream. The technical fish ladder consists of a fish ladder entrance at the stilling basin, a 2,900-foot-long half-ice harbor fish ladder, and a 41-gate fish ladder exit into the reservoir. Water is supplied to the fish ladder through one of the fish ladder exit gates. Additional attraction water is provided to the fish ladder entrance via a pipeline connected to the water quality outlet pipes. The headwater and tailwater can vary throughout the range of fish passage flows from WSEL 588 to 628 and WSEL 419.5 to 422.8, respectively. Figure 4-6 through Figure 4-9 provide general information about the Technical Fish Ladder. Figures FRFA-C-2 through FRFA-C-4 in Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017) show this alternative in more detail.

4.3.1.1 Design Elements

The Technical Fish Ladder, shown in Figure 4-6, consists of three primary design elements:

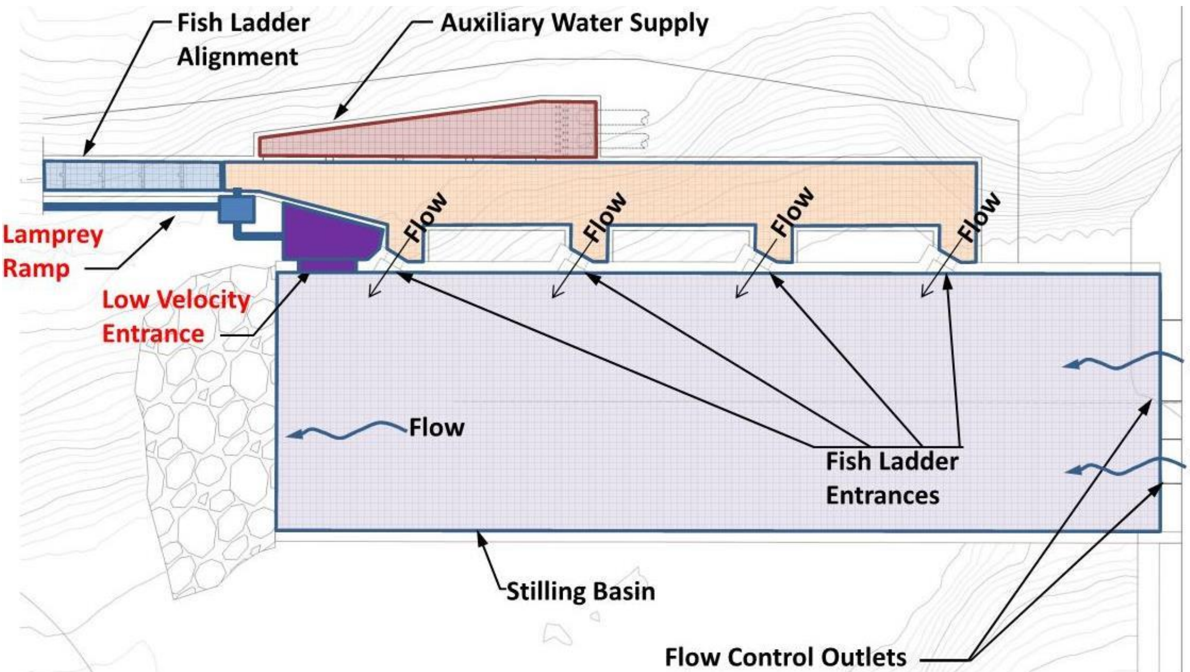
- Fish Ladder Entrance
- Half-Ice Harbor Fish Ladder
- Fish Ladder Exit

Figure 4-6
Technical Fish Ladder Plan View



4.3.1.1.1 Fish Ladder Entrance

Figure 4-7
Technical Fish Ladder - Fish Ladder Entrance



The Fish Ladder Entrance is a concrete structure to the right of the stilling basin that consists of four fish ladder entrance gates, one low velocity entrance gate, a lamprey ramp, a lamprey collection and transport facility, an entrance pool, and an auxiliary water upwell and diffusion chamber. The entrance structure is about 205 feet long by 28 feet wide by 26 feet high. Four short channels branch off of the entrance pool to the west where they tie into the east wall of the stilling basin. The stilling basin wall has a 6 feet wide by 10 feet tall rectangular penetration at the end of each of the channels. Each penetration contains a fish ladder entrance gate capable of covering the entire penetration or leaving it completely unobstructed. The low velocity entrance is located adjacent to the northern-most entrance gate, at the downstream end of the stilling basin. It consists of a 3.5 foot wide by 10 foot high penetration through the stilling basin wall with a gate that is also capable of blocking the full opening or leaving it fully open. A 28 feet long by 17 feet wide by 26 feet high pool connects the low velocity entrance to the fish ladder entrance pool. The low velocity entrance pool is open to the fish ladder entrance pool via a penetration in the west fish ladder entrance wall of the same size. An identical gate also serves at this penetration.

A 1 foot wide, Bonneville-style, steel flume penetrates the north wall of the low velocity entrance pool. The flume extends northeast to a small lamprey resting box. Two additional 1 foot wide flumes penetrate the lamprey resting box; one to the east and one to the north. The eastern flume penetrates the fish ladder entrance wall. The northern flume runs parallel to the fish ladder and terminates at the lamprey collection tank adjacent to the first turn in the fish ladder. The northern lamprey flume and collection tank are located at grade.

Water is supplied to the fish ladder entrance pool by the fish ladder and the auxiliary water upwell and diffuser chamber. The fish ladder connects to the entrance pool on the north wall. The auxiliary water chamber is located adjacent to the fish ladder connection, on the east wall of the entrance pool. Where the chamber is adjacent to the entrance pool, four sets of 17 feet long by 4 feet high diffuser screens are set into the wall. The auxiliary water chamber is trapezoidal shaped and the same height as the fish ladder entrance pool. It is 91 feet long and tapers from 5 feet at the downstream (north) end to 15 feet wide at the upstream (south) end, adjacent to the upwell. The upwell is about 15 feet square and about 40 feet high. While the top of the walls stay at elevation 440 throughout entire Fish Ladder Entrance, including the auxiliary water chamber, the floor in the upwell drops an additional 14 feet below the floor in the entrance pool and the auxiliary water diffuser area.

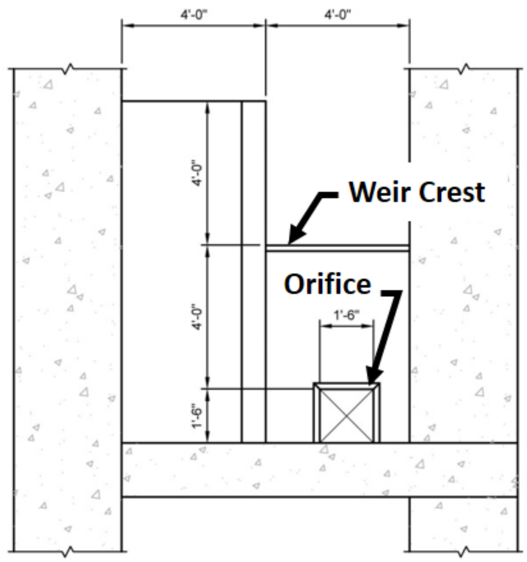
The Fish Ladder Entrance is cut into hard rock on the right river bank. The adjacent stilling basin is cut deeper into the rock by over 30 feet, however, it is likely the deeper cut for the stilling basin walls will be nearly vertical. In the southwest corner the Fish Ladder Entrance will require about 15 feet of concrete fill as the rock bank drops at nearly a 1:1 slope in this area. The rock slope rises quickly to the northeast, leaving most of the Fish Ladder Entrance to be founded on rock cut. Outside of the southwest corner, about half of Fish Ladder Entrance is cut into the rock up to 50 feet deep. The remaining structure is cut 60 or more feet into rock.

In order to access the Fish Ladder Entrance, an access road must also be cut into the hillside. The access road is located primarily on the east side of the structure and wraps around to the southeast corner. It is cut 30 or more feet into the soil, just above the rock line. A retaining wall is constructed on the east side of the road for the full height of the cut.

4.3.1.1.2 Half-Ice Harbor Fish Ladder

The Half-Ice Harbor Fish Ladder is over 2,200 feet long, extending from the fish ladder entrance to the fish ladder exit. It consists primarily of 8-foot-wide by 10-foot-long pools. Baffles are located at the upstream and downstream end of each pool. The baffles are 4 feet wide by 5 feet 6 inches tall. The baffle rises to the full height of the adjacent wall for the remaining 4 feet of width. At the floor, in the center of short section of the baffle is an 18 inch square orifice. A typical section of the baffles is shown in Figure 4-8 below. The floor slopes at a 1 foot vertical drop across each pool, for a rough slope of 9.2%.

Figure 4-8
Technical Fish Ladder Typical Baffle Section



Turning pools or resting pools are located about every 10 standard pools. Turning pools are generally twice as wide as a standard pool and the upstream weir is located adjacent to the downstream weir with a short wall separating them. Turning pools allow the normally linear fish ladder to change horizontal direction. Turns are usually close to 180 degrees but may be as little as 10 degrees. Resting pools are located in long linear sections of the fish ladder. They are the same width as a standard pool and have the same 1 foot change in floor slope over their length but the pool length is twice as long as a standard pool.

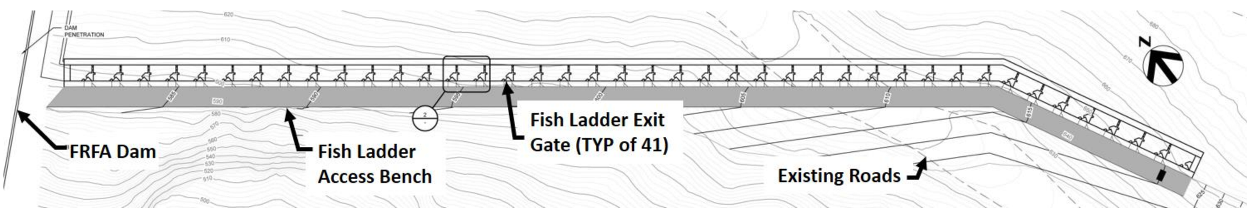
The Half-Ice Harbor section of the Technical Fish Ladder is a slab-on-grade structure. As such, horizontal location follows the grade of the hillside as it ascends to the reservoir. The Fish Ladder is cut into the hillside. Where possible the wall of the Fish Ladder also serves as a retaining wall for the hill above. Where combining these functions is not possible, separate retaining walls are constructed. The retaining walls may be 20 feet high or more in some locations. The Fish Ladder is cut into the hillside to allow sufficient space for a 10-foot wide road to also be benched into the hill adjacent to the Fish Ladder on the downhill side.

A transport channel penetrates the dam at a floor elevation of 583. To accommodate the slab and walls of the transport channel, the opening in the dam is approximately 10 feet wide by 12 feet high. An emergency shutoff gate is located in the transport channel at the upstream dam face. The height of the penetration locates the penetration and Fish Ladder Exit east of the Floating Surface Collector alternative, however, the locations of the Technical Fish Ladder and the Multi-Port Collector are located in the same space. The design of both the Technical Fish Ladder and the Multi-Port Collector would need to be adjusted if both alternatives were selected.

4.3.1.1.3 Fish Ladder Exit

The Fish Ladder Exit begins where the Fish Ladder penetrates the upstream face of the dam. The Fish Ladder exit consists of 41 half-ice harbor type pools. Each pool is 8 feet wide by 14 feet long by 10 feet high. Unlike most fish ladder exits, this fish ladder exit has a concrete ceiling the runs the full length of Fish Ladder Exit, from the penetration through the dam to the upstream end. The Fish Ladder floor has a continuous slope, rising 1 foot per pool, beginning at the downstream pool at WSEL 583 and rising to WSEL 623 at the upstream pool. An 18 inch wide channel is oriented 45 degrees to the pool wall at the upstream end of each pool, adjacent to the baffle. The channel leads to a 4 foot wide by 5 foot high gated opening in the wall of the Fish Ladder Exit. Each gate is mounted to the outside face of the west (reservoir side) Fish Ladder wall. Submersible gate operators are mounted to the top of the wall. The Fish Ladder Exit is shown in Figure 4-9.

Figure 4-9
Technical Fish Ladder Exit Plan View



Similar to the Half-Ice Harbor Fish Ladder section, the Fish Ladder Exit is a slab-on-grade structure benched into the hillside with a 16 foot wide gravel bench in the grade on the west side. A jib crane on the top of the dam and a stair tower provide person and equipment access to the gravel bench, respectively. There is no road access to the Fish Ladder Exit. The Fish Ladder Exit is founded on rock. A separate retaining wall supports the soil portion of the hillslope above the Fish Ladder Exit. The Fish Ladder Exit is constructed against the retaining wall where required.

4.3.1.2 Theory of Operation

The technical fish ladder is intended to provide year-round, safe, volitional upstream passage for migrating adult and juvenile salmon and steelhead, resident fish, and lamprey for the full range of reservoir pool elevations during normal operation. Although adult salmon and steelhead will only pass upstream during certain periods of the year, operation of the technical fish ladder year-round is intended to also accommodate resident fish, juvenile salmon and steelhead, and lamprey that currently traverse this reach of the Chehalis and may wish to move upstream at any time. Juvenile salmon and steelhead are not precluded from moving upstream through the fish ladder, though the length of the fish ladder may make this passage facility difficult for juveniles to fully traverse.

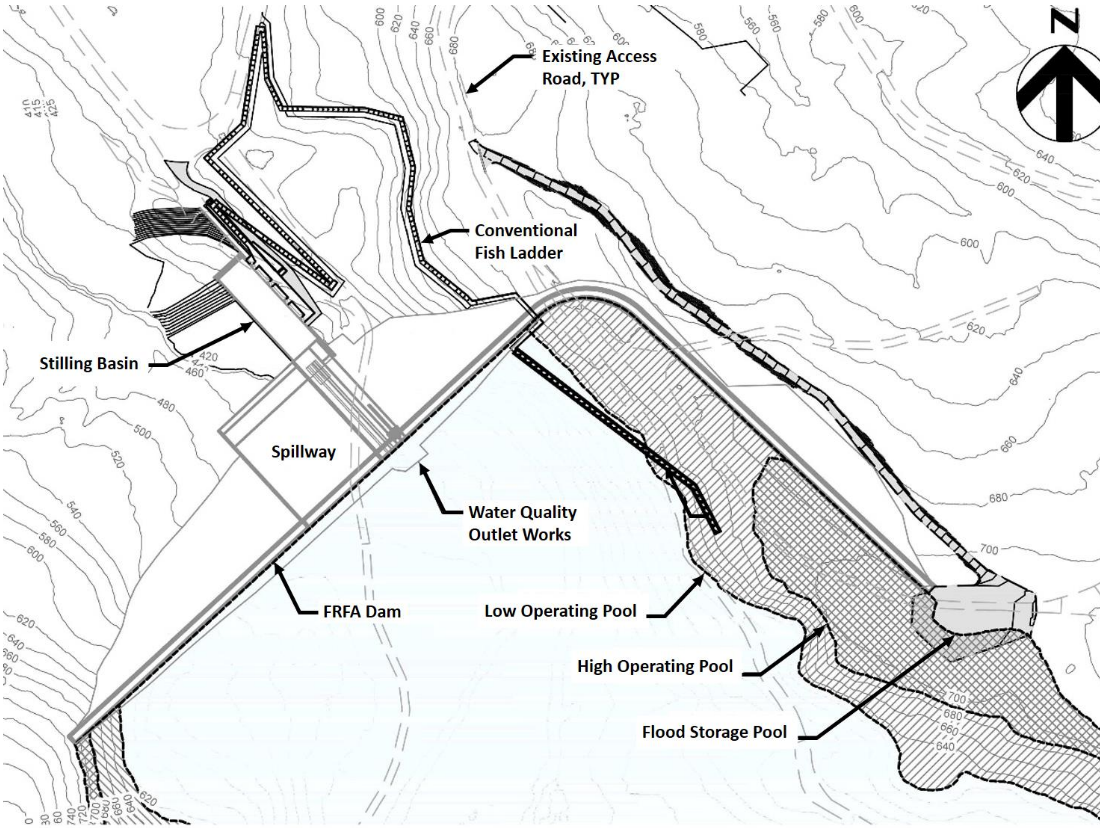
The technical fish ladder meets the NMFS design criteria for passing adult salmonids, including 1- foot drops across the fish ladder baffles, attraction flows at the entrance greater than 10 percent of the 5 percent exceedance flow (250 cfs), and a hydraulic drop at the primary entrance gate of 1 to 1.5 feet. In addition to meeting NMFS criteria at the fish ladder entrance, the low-velocity entrance has been incorporated to accommodate juvenile fish and lamprey. Juvenile fish pass over two weirs at the entrance, each with 6-inch hydraulic drops across them as suggested by the literature, to enter the fish ladder. Similarly, lamprey entering the low-velocity entrance are provided with flat, smooth surfaces to attach to as they make their way up a separate flume to a collection tank. The collection tank is regularly transported upstream in a trap-and-transport-type operation.

Auxiliary water is provided to the fish ladder entrance from the reservoir water quality control works through a large-diameter conduit. An in-line energy dissipation valve reduces much of the pressure in the auxiliary water supply pipe. The remaining energy is dissipated in the deep upwell at the fish ladder entrance. The energy dissipation valve is an automated valve that will adjust to meet the desired flow based on the reservoir elevation. Diffuser baffles for the auxiliary water supply will be hydraulically tested at startup and set once. No additional adjustment of the diffuser baffles should be necessary.

Turning pools and resting pools are located throughout the fish ladder to provide fish places to rest on their 210-foot climb to the reservoir.

Approximately 30 cfs is supplied to the fish ladder via the fish ladder exit gates. Of the 41 fish ladder exit gates, only one gate operates at a time to maintain a continuous hydraulic connection with the reservoir at the reservoir surface. The gates operate automatically based on the reservoir elevation. Each gate accommodates 1 foot of reservoir operation. As the reservoir fluctuates during its normal operation from WSEL 628 to 588, as shown in Figure 4-10, the fish ladder exit gates open and close to maintain 30 cfs in the fish ladder. Since each gate can only accommodate about 1 foot of water surface fluctuation, the gates automatically track the reservoir and switch on and off as necessary. When operation transitions from one gate to another, the high gate will slowly close while the lower gate slowly opens.

Figure 4-10
Technical Fish Ladder and FRFA Reservoir Inundation Map



A concrete ceiling runs the full length of the fish ladder exit to minimize the opportunity for debris to enter the fish ladder when portions of or all of the fish ladder exit is submerged. The ceiling also limits the height of the structure, minimizing expensive additional structural support features. Air management systems are included in the ceiling to prevent entrapment of air and floatation forces when the facility is submerged.

When the reservoir water elevation rises above the normal operating maximum, the emergency shutoff gate at the dam penetration closes automatically. The emergency shutoff gate stops flow down the fish ladder. During such events, the fish ladder will drain through the orifices back to the tailwater. The fish ladder will remain dry until the emergency shutoff gate is reopened. Closing and opening of the shutoff gate will occur slowly to allow fish in the ladder to escape downstream and to prevent surging in the fish ladder. However, in the event of a catastrophic event such as an earthquake or runaway (uncontrolled discharge) in the fish ladder downstream, the emergency shutoff gate will close quickly to minimize the risk to the dam.

4.3.1.3 Maintenance

Maintenance activities will be conducted using the stair tower access and jib crane. Where heavy equipment is needed for maintenance, access, equipment, and supplies will be provided via boat or barge. Maintenance activities can be separated into regular, annual, and infrequent periods.

Regular maintenance occurs throughout the normal operation of the reservoir. It includes:

- Inspection and debris removal inside the fish ladder, especially at the fish ladder exit following submergence, both at flood events and for the lower pools during normal operation. Material will be of size small enough to fit through the trashracks at the exit gates. Minimal debris removal is expected
- Debris removal from trashracks at the exit gates.
- Inspection and debris removal from the auxiliary water diffuser screens.
- Inspection of mechanical and electrical systems following submergence of the fish ladder exit, both at flood events and for the lower pools normal operation

Many features of the Technical Fish Ladder do not require maintenance on a regular basis but do require annual inspection and/or maintenance to keep them in good working order and prolong their functional life. Beyond the regular needs for annual inspection, regular cycles of submergence and exposure throughout the normal operating period put additional wear on all the equipment and metal work in the Fish Ladder Exit. It is expected maintenance tasks that may ordinarily be undertaken every few years on the Fish Ladder and the Fish Ladder Entrance will need to occur annually on the Fish Ladder Exit. Annual maintenance tasks may include:

- Repaint steel structures at the Fish Ladder Exit.
- Inspect, operate, and service the emergency shutoff gate the dam penetration and the Fish Ladder Entrance Gates.
- Service manual and motorized gate operators.
- Inspect and service the in-line energy dissipation valve and Fish Ladder Exit gates.
- Drain fish ladder and inspect all fish ladder pools, weirs, and orifices. Remove debris. Repair damaged concrete.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Repaint steel structures at the Half-Ice Harbor Fish Ladder and the Fish Ladder Entrance.
- Inspect and repair access grating and supports as necessary.
- Replace gate operators as necessary.
- Repair or replace Fish Ladder Entrance and Exit gates that may have been damaged during flood events.

- Maintain gravel bench at the Fish Ladder Exit.
- Maintain access road.

A summary of the anticipated level of effort for operation and maintenance is provided in the table below.

Table 4-6
Operation and Maintenance Level of Effort for the Technical Fish Ladder

ALTERNATIVE	AVERAGE ANNUAL DURATION	PERSONNEL REQUIREMENTS (FTE = FULL TIME EMPLOYEE)	SPECIAL CONSIDERATIONS
FRFA – Upstream CHTR Facility	12 months	Operations Staff – 1.58 FTE Biological Staff – 1.26 FTE Maintenance Staff – 0.89 FTE Total = 3.73 FTE	• Operation of each facility daily during operation. • Operation of transport vehicle required daily during operation.

4.3.1.4 Anticipated Fish Passage Performance and Survival

The following is the anticipated fish passage performance and survival for the Technical Fish Ladder alternative. A discussion of the rationale behind performance and survival value is provided in Attachment C of this document.

Table 4-7
Technical Fish Ladder Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	80%	99%	**79%**
Fall Chinook	80%	99%	**79%**
Coho	80%	99%	**79%**
Winter Steelhead	80%	99%	**79%**
Coastal Cutthroat	70%	99%	**69%**
Pacific Lamprey*	60%	90%	**54%**
Western Brook Lamprey*	60%	90%	**54%**
JUVENILE UPSTREAM			
Spring Chinook	0%	50%	**0%**
Fall Chinook	0%	50%	**0%**
Coho	0%	50%	**0%**
Winter Steelhead	0%	50%	**0%**
Coastal Cutthroat	0%	50%	**0%**

Note:
*Lamprey are passed through a lamprey ramp, collection hopper, and transport to the head of reservoir.

The anticipated fish passage performance and survival is based on the performance of other volitional fish ladders and adjusted based on conditions that are unique to the alternative proposed for the FRFA dam on the Chehalis River. There are limited examples of volitional passage fish ladders in the Pacific Northwest designed for hydraulic heads greater than 150 feet, or volitional passage fish ladders that accommodate reservoir fluctuations at the ladder exit of 30-40 feet. The following is a summary of the performance of similar fish ladders:

- Mid-height ladders up to 150 ft tall on the Columbia River show a high level of passage performance for adult anadromous salmonids and bull trout.
- The North Fork Fish Ladder on the Clackamas River in Oregon is an example still in operation that extends over 2 miles and ascends a height of 240 feet. The North Fork ladder passes from 95 to 100 percent of adult Chinook and steelhead. Bull trout have also shown a high level of success but are not the current focus of monitoring efforts, so less data are available. The 2-mile- long fish ladder at North Fork performs at levels higher than other examples of its kind for the following reasons:
  - The ladder entrance accommodates up to 280 cfs of attraction flow through multiple entrances using an auxiliary water supply (AWS) originating from the Faraday Diversion Dam
  - The water temperature and water quality composition originating from the AWS are identical to those of the river that are triggering fish movement
  - The reach between North Fork Dam and Faraday Diversion Dam is not subject to high levels of thermal gain
  - North Fork Reservoir exhibits limited thermal stratification as a result of its narrow configuration and hydropower operations; therefore, the water entering the ladder exit is similar to that of the river
  - Cool water is injected into the ladder at two additional downstream locations, which further improves fish attraction during portions of the year

A number of site-specific factors are accounted for in the performance estimate in Table 4-7 including:

- The ladder performance and survival values include fish entering the ladder entrance, release into the reservoir, and passage through the reservoir. The performance and survival documented at similar facilities did not always include these pathways.
- Reservoir transit introduces some level of uncertainty and will require site-specific verification.
- Loss of migration cues at the reservoir through flow modification may influence the timing of upstream migrants.
- The expected performance of the ladder is at or above 80 percent, with survival values of 99 percent for adult Chinook, steelhead, and coho. The value of 80 percent was selected to accommodate uncertainty of reservoir transit.
- Slightly lower performance is anticipated for cutthroat trout, given potential issues with attraction into the ladder entrance as well as the expenditure of energy and motivation required

by smaller fish. Cutthroat trout are expected to perform similarly to bull trout, which have been documented to ascend the North Fork Ladder in as few as 10 to 12 hours. The lower performance value of 70 percent was selected to accommodate uncertainty of reservoir transit.

- Little is known about the motivation of juvenile fish to ascend a high ladder such as this. Given this uncertainty and the likelihood that the ladder would be optimized for adult salmonids, low performance and survival values of 0 percent and 50 percent, respectively, were selected.
- We have assumed that provisions to improve lamprey passage would be incorporated into the ladder entrance, baffles, floor, and walls, and a separate trap and transport facility for lamprey would be provided. However, given the lack of data available relative to lamprey passage in high dam facilities, values reflecting expected low performance (60 percent) and high survival (90 percent) were selected.

4.3.1.5 Reliability

The Technical Fish Ladder is expected to have a moderate level of reliability due to a variety of factors. As described above, technical fish ladders have a long history of documented high performance passing fish. However, very few technical fish ladders exist for passages over 150 vertical feet. The few ladders that do exist have mixed results except the North Fork Ladder, described in Section 4.2.2.4, which has very high performance. Many of the lessons learned from the North Fork Ladder can be applied here, but the mixed results of other high head fish ladders suggest a moderate level of reliability should be anticipated.

Several aspects of the Technical Fish Ladder alternative also indicate a greater level reliability may be warranted. One such positive influence on reliability is the supply of water via gravity. The water flowing through the fish ladder enters through the fish ladder exit gates. The only mechanical system the supply water relies on is the automated exit gates. However, in the event of a gate failure, the gates can still be moved manually to ensure a steady supply of water. This operation also improves reliability by ensuring adequate water is supplied to the fish ladder to allow fish to safely exit the ladder in the event of an emergency gate failure. Similarly, the simplicity of the mechanical systems requires a low level of effort to keep the ladder in working order and a relatively low level of skill and training for staff to operate, improving reliability.

Although several aspects indicate a greater level of reliability, there are other aspects of the Technical Fish Ladder alternative that suggest a lower level of reliability. For instance, while the mechanical systems require a low level of effort, the automated gate systems at the fish ladder entrance and exit require a moderate level of skill to maintain. The automation software will be somewhat complex to adjust if improvements need to be made. Similarly, the mechanical components that convert the software signals into physical action are complicated pieces of equipment. This equipment requires special training and knowledge to maintain. If the individual with the special knowledge and training is not readily available the performance of the fish ladder may be negatively impacted until the situation is remedied. The potential for such a situation also reduces the reliability of this alternative.

The history of performance of and lessons learned from other fish ladders at high head passages, taken alone, indicates a moderate level of reliability may be anticipated. However, multiple other aspects of the Technical Fish Ladder alternative, such as complex exit gate automation and a gravity water supply, respectively, suggest both lesser and greater levels of reliability by themselves. When all these factors are considered together, a moderate reliability level for the Technical Fish Ladder alternative appears warranted.

4.3.1.6 Cost Summary

The Technical Fish Ladder has the highest construction cost of all the upstream fish passage alternatives. The height and length of this fish ladder, as well as the large reservoir fluctuation, make the Technical Fish Ladder more expensive than other technical fish ladders. The foundation costs associated this alternative are also much greater than many other technical fish ladders. Excavation on steep slopes and slope stabilization add a substantively to the overall cost. In addition, most other technical fish ladders are designed to operate with reservoir fluctuations of 10 feet or less; with a few in the 15- to 20-foot fluctuation range. The much larger fluctuation in this reservoir requires over twice as many exit gates as other fish ladders.

A summary of the estimated construction costs, included the estimated upper and lower bounds, is provided in Table 4-8 below. A detailed breakdown of construction costs can be found in Attachment D.

Conversely, the Technical Fish Ladder has the lowest operation and maintenance cost. The Technical Fish Ladder is simple to operate. Flow to the fish ladder is supplied by gravity via automated fish ladder exit gates. Relatively little debris is expected to enter the fish ladder due to the log boom in the reservoir upstream. The only mechanical systems associated with the fish ladder are the automated exit gates so the annual maintenance is relatively low compared with other alternatives. A summary of the estimated O&M cost is provided in Table 4-8 below. A detailed breakdown of O&M costs can be found in Attachment D.

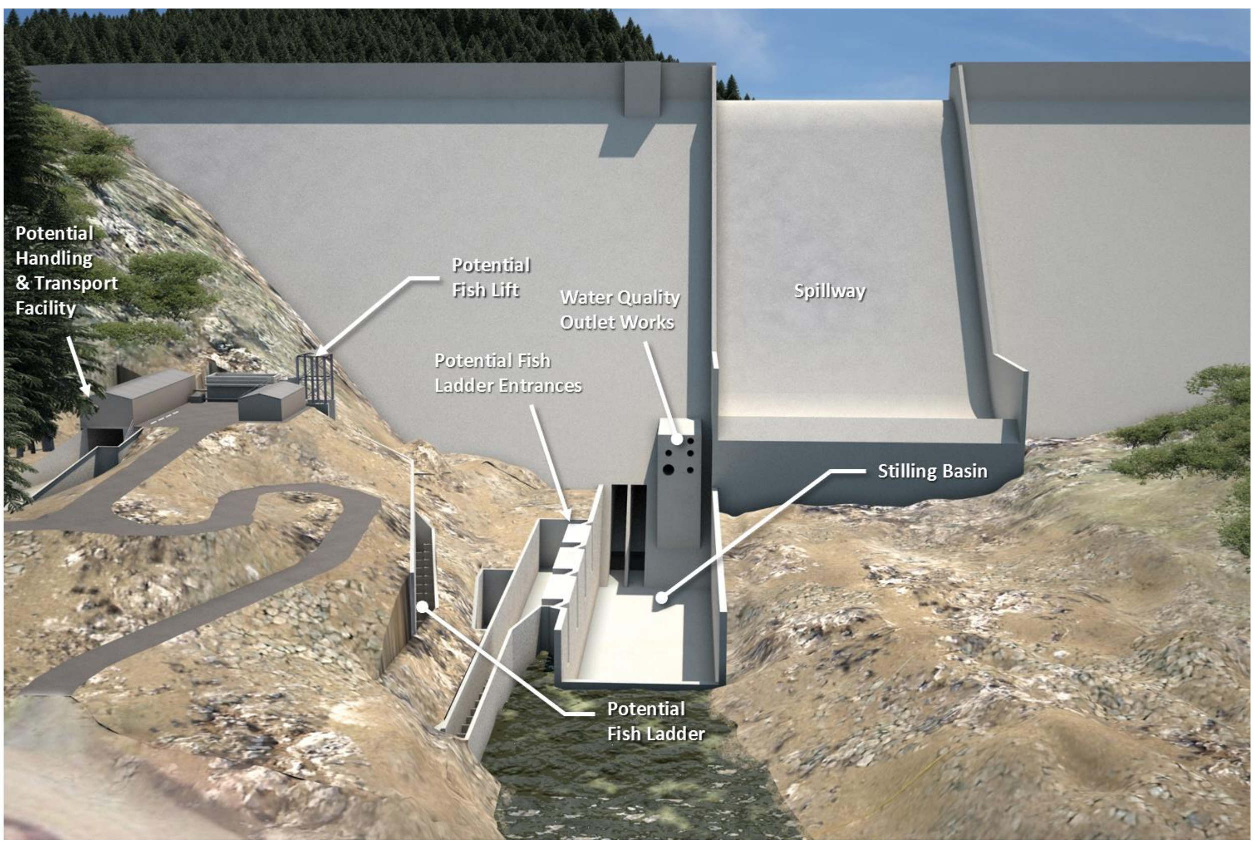
Table 4-8
FRFA Technical Fish Ladder Estimated Construction and O&M Costs

FISH PASSAGE OPTION	CONSTRUCTION COSTS			O&M COSTS
	LOWER BOUND COST ($ MILLION)	MIDDLE BOUND COST ($ MILLION)	UPPER BOUND COST ($ MILLION)	($)
FRFA – Upstream Fish Passage: Technical Fish Ladder	$49.1	$65.4	$98.1	$198,000

4.3.2 Collection, Handling, Transport, & Release Facility (CHTR)

The CHTR for the FRFA dam is a fish passage alternative intended to collect migrating adult salmon and steelhead, juvenile salmon and steelhead, resident fish, and lamprey moving upstream and safely transport them upstream of the FRFA dam. The CHTR consists of a short fish ladder, a fish lift, holding galleries, sorting stations, and transportation. Fish enter the CHTR through the stilling basin, where they are attracted to the flow from the fish ladder entrance. Flow from the fish ladder guides them through the entrance pool, up the fish ladder, and into a fish trap and holding area. Inside the holding area they are crowded into a hopper, which is lifted to a flume. After they are released in the flume, the fish are sorted by size through bar grating and diverted to holding galleries. When released from the holding galleries, the fish move to sorting stations where they can be manually sorted and examined or simply passed through to transport tanks. Fish are transferred from the transport tanks via water-to-water transfer directly to transport trucks. The trucks are driven above the dam to predetermined release sites in the reservoir or on Chehalis River or one of its tributaries. The FRFA CHTR is a staffed facility that is operated year-round. A rendering of the stilling basin and CHTR is provided in the figure below.

Figure 4-11
RFA Dam, Taken Downstream, Looking Upstream, During Normal Operation



4.3.2.1 Design Elements

The CHTR consists of six design elements:

- Fish Ladder
- Fish Lift
- Holding Galleries
- Sorting Stations
- Transportation
- Mechanical / Electrical and Storage Buildings

4.3.2.1.1 Fish Ladder

The Fish Ladder consists of a fish ladder entrance at the stilling basin and a 400-foot-long half-ice harbor fish ladder to the Fish Lift. Water is supplied to the fish ladder at the Fish Lift. Additional attraction water is provided to the fish ladder entrance via a pipeline connected to the Water Quality Outlet pipes. While the water surface elevation in the Fish Lift remains constant, the tailwater can vary throughout the range of fish passage flows, from WSEL 419.5 to 422.8.

The Fish Ladder Entrance is identical to that described in Section 4.3.1.1.1 Fish Ladder Entrance.

The Half-Ice Harbor Fish Ladder is about 400 feet long, extending from the Fish Ladder Entrance to the Fish Lift. The pools are identical to the Technical Fish Ladder except for the width, length, and floor slope. Baffles are located at the upstream and downstream end of each pool. The baffles are 4 feet wide by 5 feet 6 inches tall. The baffle rises to the full height of the adjacent wall for the remaining 4 feet of width. At the floor, in the center of short section of the baffle is an 18 inch square orifice. The floor slopes at a 0.7 foot vertical drop across each pool, which meets the NMFS criteria for juvenile salmonids and is also comparable to the swimming capabilities of resident fish like cutthroat and bull trout. The overall fish ladder has a rough slope of 6.5%. Each pool has a width of 8 feet and a length of 10 feet.

There are two turning pools and one resting pool in the fish ladder. The first turning pool is the 10th pool upstream from the fish ladder entrance pool and turns the fish ladder about 170 degrees. The resting pool is the 20th pool. Finally, the second turning pool is the 30th pool, 4 pools downstream from the Fish Lift, and turns the fish ladder 100 degrees. The turning pools and resting pools are both twice as long a standard pool.

The fish ladder is a slab-on-grade structure cut into the hillside. In the lower portion of the fish ladder the structure is benched into rock. The fish ladder continues to be founded on rock for its full length but the depth of the rock cut lessens as the fish ladder ascends. A substantial depth of soil remains above the rock line, therefore retaining walls are required along the full length of the fish ladder. The retaining walls may be 30 feet high or more in some locations. The access road to the fish ladder entrance also provides access to roughly half of the fish ladder pools. The remaining pools are accessible via the grating over every pool. A separate access road is not provided for these remaining pools. The fish lift at the end of the fish ladder is accessible from the handling and transport facility parking lot.

4.3.2.1.2 Fish Lift

The Fish Lift consists of a trapping mechanism, hopper, and lift system at the upstream end of the fish ladder. The trapping mechanism and hopper sit inside a concrete sump, into which the hopper is lowered during trapping. Side clearances between the hopper and sump sidewalls do not exceed 1 inch, thereby minimizing fish access below the hopper. Flexible side seals are used to ensure that fish do not pass below the hopper. The sump is 10 feet wide by 10 feet long. The concrete floor of the sump is 4 feet below the fish ladder floor and roughly 50 feet below existing grade. An 18 inch diameter pipe supplies 30 cfs to a diffuser system in the walls of the Fish Lift. Water is diffused through screens at the bottom of all three walls of the Fish Lift. The screens extend from the bottom of the sump and above the adjacent floor of the fish ladder. Water to the Fish Lift is supplied via a tee off of the auxiliary water supply pipeline, downstream of the in-line energy dissipation valve.

The trapping mechanism is a vee-trap built into the hopper, which allows fish to volitionally enter, but not exit, the hopper. All components exposed to fish have welds and sharp edges ground smooth to the touch, and other features as required to minimize injuries. The vee-trap allows for temporary closure to avoid conflict with hopper lifting and loading operations. The vee-trap does not allow fish entry into unsafe areas such as behind or under the hopper.

A full-sized hopper is located inside the Fish Lift walls and rests in the sump. The maximum water volume in the hopper is greater than 0.15 cubic feet per pound of fish at the maximum fish loading density. This provides the hopper with a sufficient volume of water for fish safety. Hopper freeboard, the distance from the water surface in the hopper to the top of the hopper bucket, is greater than the water depth within the hopper, to reduce risk of fish jumping out during lifting operations. Fail-safe measures are provided to prevent entry of fish into the holding pool area to be occupied by the hopper before the hopper is lowered into position. The hopper interior is smooth, and designed to safeguard fish.

The lift system consists of a crane, motor, and cables mounted to the top of a steel tower over the hopper. Cables attach to the structural supports at the top of the hopper. The motor lifts all the fish trapped in the hopper by the trapping mechanism over 80 feet vertically to a flume that leads to the handling and transport facility. The same water supply that feeds the Fish Lift and Fish Ladder also supplies the flume. The flume is sloped down toward the holding galleries. Inside the flume, metallic graders separate the fish and send them down separate flumes to the adult and juvenile holding galleries.

4.3.2.1.3 Holding Galleries

Three holding galleries are provided for adult fish, juvenile fish, and lamprey. The adult and juvenile holding galleries are located at the handling and transport facility. The lamprey holding gallery is located adjacent to the first turning pool as described in Section 4.3.1.1.1 Fish Ladder Entrance. The flume from the top of the fish lift carries adult and juvenile fish to their respective holding galleries.

The adult holding gallery is an elevated concrete structure downstream of the Fish Lift, flume, and grader. It is sized to provide a minimum volume of 0.75 cubic feet per pound of fish based on the trap capacity, with water temperatures less than 50° F, and dissolved oxygen between 6 to 7 parts per million. The adult holding gallery has a separate water supply and drain system. Adult Holding Gallery water supply capacity is 2 gallons per minute per adult fish for the predetermined adult salmon trap holding capacity.

The adult holding gallery includes provisions to minimize adult jumping which may result in injury or mortality. High freeboard on holding pool walls (5 feet or more) and sprinklers above the holding pool water surface reduce the ability of fish to detect movement above the holding gallery will be used to minimize jumping.

The adult holding gallery has a motorized crowder to encourage fish to move towards the exit of the holding gallery over the false weir and into the sorting facility. The crowders have a maximum clear bar spacing of 7/8 inch. The side gap tolerances do not exceed 1 inch. The holding gallery also has side and bottom seals sufficient to allow crowder movement without binding, and to prevent fish movement behind the crowder panel.

A false weir is included at the upstream side of the adult holding gallery. It provides vertical flow and is used in conjunction with a bifurcation gate and distribution flumes that route fish to a specific areas of the sorting facility.

- The juvenile holding gallery will be sized to provide a minimum volume of 0.75 cubic feet per pound of fish based on trap capacity, with water temperatures less than 50° F, and dissolved oxygen between 6 to 7 parts per million. The juvenile holding gallery has a separate water supply and drain system. The water supply capacity is 2 gallons per minute per adult fish for the predetermined adult salmon trap holding capacity. If the Floating Surface Collector alternative, described in Section 4.3.3, is selected with the CHTR alternative, the juvenile holding gallery will also accommodate receiving juvenile fish from FSC transport truck tanks via water-to-water transfer.

The trap capacity of the juvenile holding gallery is determined by the maximum daily fish return, or by the number of fish expected to be trapped before the trap catch is transported. The poundage of fish is determined by the weight of an average fish targeted for trapping, times the maximum number of fish.

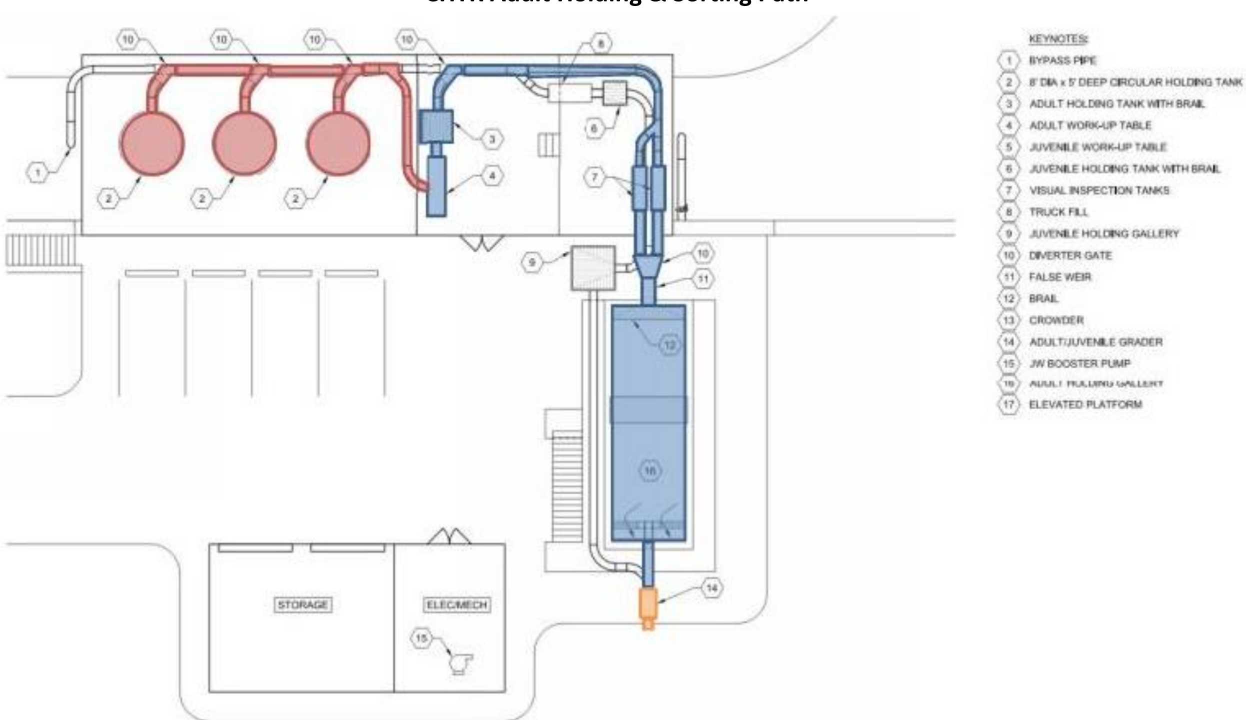
The juvenile holding gallery includes provisions that minimize jumping which may result in injury or mortality. Netting over the pool strong enough to prevent juveniles from breaking through the mesh fabric and sprinklers above the holding pool water surface to reduce the ability of fish to detect movement above the holding gallery are used to minimize jumping.

The juvenile holding gallery has a floor brail to encourage fish to move towards the exit of the holding gallery and into the sorting facility. The floor brail is composed of sufficiently sized screen material (based on life stage and species present), to preclude injury or mortality of non-target species. Side gap openings do not exceed 1 inch with seals included to cover all gaps. The floor brail panel is kept in the lowest position until flow passes over the flow egress weir. It is manually operated to allow movement of the brail at 2 feet/minute (upward and downward) to minimize the stress on the fish induced by crowded them between the floor brail and lock flow egress weir.

The lamprey holding area provides the continuous flow and depth down the flume to the low velocity entrance at the Fish Ladder Entrance. Lamprey are hand crowded and manually transitioned to the hopper. The hopper is moved to a transport truck to be driven upstream of the dam and released.

4.3.2.1.4 Sorting Stations

Figure 4-12
CHTR Adult Holding & Sorting Path

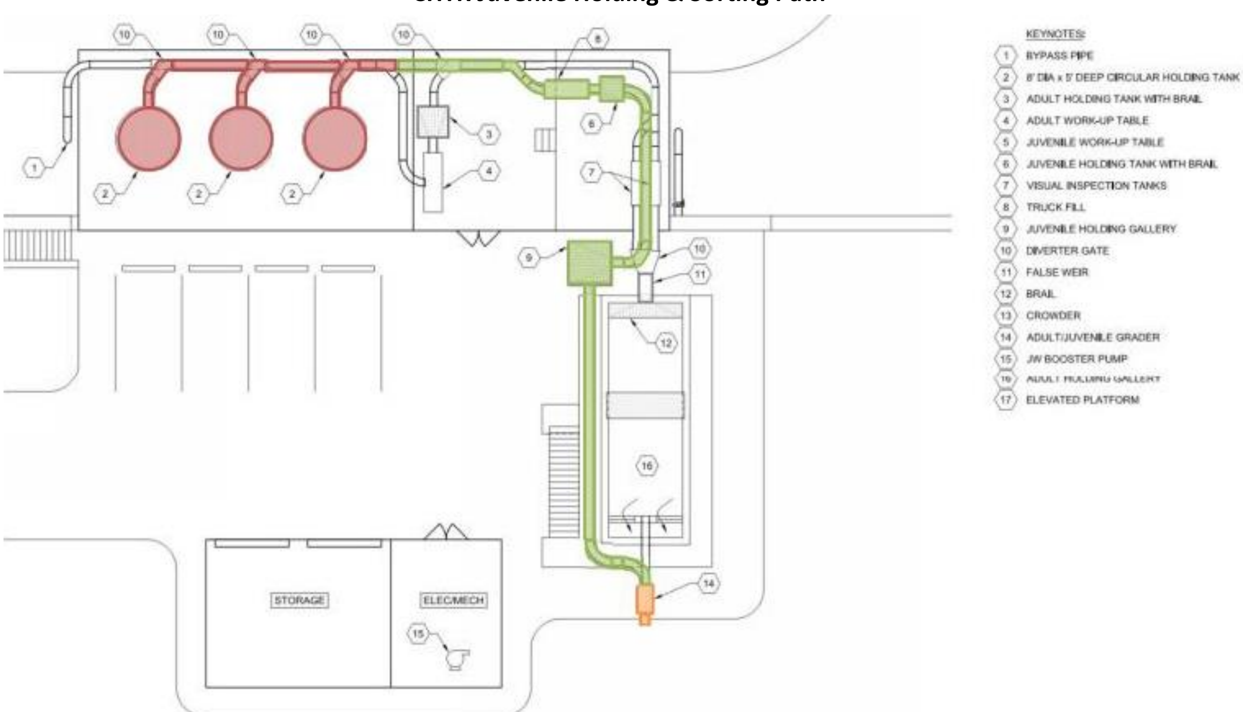


From the adult holding gallery fish are crowded and pass over the false weir. They are then directed to one of two visual inspection tanks by means of an automated diverter gate. The operator determines where the fish should be held and directs the fish back into the adult primary transport flume. Fish are then sorted by actuated panel gates, within the adult primary transport flume, to any of three holding tanks, bypass, or adult anesthesia tank and work-up station.

Anesthetized fish are routed to one of the three circular holding tanks to allow monitoring of fish to ensure full recovery from the anesthetic effect prior to transport. Hydraulic conditions within the recovery tank insure that in partially or fully anesthetized fish are not impinged on an outflow grating or any other hazardous area.

The distribution flume is used when fish are routed to anesthetic tanks, recovery tanks, pre-transport holding tanks, and direct to transport holding tanks. The flumes have smooth joints, sides, and bottom with no abrupt vertical or horizontal bends. Surfaces of the flumes are continuously wetted. Horizontal and vertical radius of curvature is at least 5 times the flume width, minimizing the risk of fish strike injuries. The minimum inside diameter of the distribution flume is 15 inches.

Figure 4-13
CHTR Juvenile Holding & Sorting Path



From the juvenile holding gallery fish are brailed into the juvenile primary transport flume. The juvenile fish are directed to the anesthesia tank and juvenile work-up station. From the juvenile work-up station the operator directs the fish back into the adult primary transport flume. Fish are then sorted by actuated panel gates, within the primary transport flume, to any of three holding tanks.

Anesthetized fish are routed to one of the three circular holding tanks to allow monitoring of fish to ensure full recovery from the anesthetic effect prior to transport. Hydraulic conditions within the recovery tank insure that in partially or fully anesthetized fish are not impinged on an outflow grating or any other hazardous area.

The distribution flume is used when fish are routed to anesthetic tanks, recovery tanks, pre-transport holding tanks, and direct to transport holding tanks. The flumes have smooth joints, sides, and bottom with no abrupt vertical or horizontal bends. Surfaces of the flumes are continuously wetted. Horizontal and vertical radius of curvature is at least 5 times the flume width, minimizing the risk of fish strike injuries. The minimum inside diameter of the juvenile distribution flume is 10-12 inches.

4.3.2.1.5 Transportation

Three circular holding tanks are supported on an elevated platform. Retaining walls, support columns for the elevated platform, and grading of access roads allow for fish transport trucks to drive directly beneath the circular tanks. Fish held in the circular holding tanks are transferred to transport vehicles by direct water-to-water transfer. Water-to-water transfer is achieved through a vertical collar fixed to the bottom of each circular tank. The collar allows for some vertical and horizontal adjustment such that the collar can be locked to the receiving port on the top of the transport truck. The vertical collars and transport truck tanks are designed to minimize fish handling stress.

Truck transport tanks arrive to receive fish with full tanks of water. When the vertical collars from the circular tanks are attached to the truck tanks water surface control is transferred to the truck transport tank so that water and fish do not plunge abruptly from the holding tank into the fish transport tank during loading. The fish egress opening from the circular holding tank into the transport tank has a minimum horizontal cross-sectional area of 3 square feet, and has a smooth transition that minimizes the potential for fish injury. An oxygen gas supply is provided to each truck transport tank to ensure proper dissolved oxygen levels are maintained in the tank during transport. Transport vehicle can be segregated into three independent vessels to accommodate multiple species, life stages, or desired release points.

Trucks transport fish to designated upstream and downstream points of release. Potential release points have not currently been sited and will be developed as this alternative is developed further.

4.3.2.1.6 Mechanical / Electrical and Storage Buildings

Prefabricated, concrete masonry unit (CMU), or buildings of similar construction are located adjacent to the handling and sorting facility to house mechanical and electrical equipment and provide storage for equipment and materials associated with the CHTR. The buildings are secured facilities with outdoor lighting to reduce the risk of vandalism or theft. The Mechanical / Electrical Building houses the transformer, distribution panel, circuit breakers, programmable logic controller (PLC), alarms, and other related items necessary for the performance of the CHTR.

4.3.2.2 Theory of Operation

The CHTR is a fish passage alternative intended to collect migrating adult salmon and steelhead, juvenile salmon and steelhead, resident fish, and lamprey moving upstream and safely transport them upstream of the FRFA dam. While adult salmon and steelhead will only pass upstream during certain periods of the year, operation of the CHTR year-round is intended to accommodate resident fish, lamprey, and juvenile salmon and steelhead that currently traverse this reach of the Chehalis River and may wish to move upstream at any time.

The CHTR fish ladder meets the NMFS design criteria for passing juvenile salmonids, including 0.7-foot drops across the fish ladder baffles and attraction flows at the entrance greater than 10 percent of the 5 percent exceedance flow (250 cfs). Provisions for attraction and collection of lamprey, juvenile salmonids, and resident fish are provided at the ladder entrance similar to the FRFA technical fish ladder entrance described in Section 4.3.1, Technical Fish Ladder. Similarly, lamprey entering the low velocity entrance are provided with flat, smooth surfaces to attach to as they make their way up a separate flume to a collection tank. The collection tank is regularly transported upstream in a trap-and-transport-type operation.

Auxiliary water is provided to the fish ladder entrance from the reservoir Water Quality Control Works. An in-line energy dissipation valve reduces the pressure in the auxiliary water supply pipe. The remaining energy is dissipated in the deep upwell at the fish ladder entrance. The energy dissipation valve is an automated valve that will adjust to meet the desired flow by burning more or less energy based on the reservoir elevation. Diffuser baffles for the auxiliary water supply will be hydraulically tested at startup and set once. No additional adjustment of the diffuser baffles should be necessary.

A resting pool and two turning pools are evenly spaced throughout the fish ladder to provide fish areas to recover on their 40-foot climb to the fish lift.

About 30 cfs is supplied to the fish ladder via the fish lift. Water to the fish lift comes from the reservoir via a pipeline that branches off the auxiliary water supply pipeline. Prior to entering the fish lift, a pipe branches off the fish lift supply pipe to provide water to the handling and sorting facilities. The remaining water passes into a stilling chamber around the fish lift sump, where the water is stilled and diffused through wall screens into the hopper area and travels down the fish ladder.

Fish and lamprey pass up the fish ladder through the fish ladder entrance gates and low-velocity entrance. Lamprey are attracted to the low velocity and smooth surfaces of the lamprey flume, which they follow to a holding tank adjacent to the first turn in the fish ladder. The holding tank is removed and manually transported upstream, where lamprey are safely released above the dam. Adult and juvenile fish continue up the fish ladder to a hopper where they are trapped and held. The hopper trap capacity is determined by the maximum daily fish return and by the number of fish expected to be trapped before the trap catch is transported. The poundage of fish is determined by the weight of an average fish targeted for trapping, multiplied by the maximum number of fish.

The motorized lift mechanism is triggered by a user set frequency and raises the fish over 80 feet to an elevated flume at the upstream end of the handling and transport facility. A mechanical trip automatically opens a door on the hopper and safely releases fish into the wetted flume. Inside the flume, fish travel by gravity to a section of smooth bar grating in the floor where adults are separated from juveniles. The adults and juveniles travel in dedicated flumes to separate holding galleries.

From the adult holding gallery, adult fish are crowded and pass over the false weir. They are then directed to one of two visual inspection tanks by means of an automated diverter gate. The operator determines where the fish should be held and directs the fish back into the adult primary transport flume. Fish are then sorted by actuated panel gates, within the adult primary transport flume, to any of three holding tanks, bypass, or adult anesthesia tank and work-up station.

From the juvenile holding gallery, fish are brailed into the juvenile primary transport flume. The juvenile fish are directed to the anesthesia tank and juvenile work-up station. From the juvenile work-up station the operator directs the fish back into the adult primary transport flume. Fish are then sorted by actuated panel gates, within the primary transport flume, to any of three holding tanks.

Anesthetized adult and juvenile fish are routed to separate circular holding tanks to allow monitoring of fish to ensure full recovery from the anesthetic effect prior to transport.

Transport trucks carrying tanks specially made to transport fish drive to a dedicated loading area beneath the circular tanks. Fisheries personnel attach a vertical collar that extends from the bottom of the circular tank to the tank on the transport truck. Tanks on the transport trucks are filled full with water prior to connection to the collar. When connections are made, the collar is also filled with water to facilitate water-to-water transfer of the fish. A horizontal gate in the floor of the tank is opened hydraulically, connecting the circular tank to the tank on the transport truck. Water in the transport truck tank is slowly drained through juvenile criteria screens in the tank wall. This water is drained directly onto the pavement of the loading area where it is carried via a floor drain back to the river. As water is drained in the truck tank, the water elevation in the circular tank slowly recedes. The slope in the floor of the circular tank ensures that all the fish exit the circular tank through the floor gate and into the truck tank. After the water level is drawn down to the top of the transport tank, a gate in the floor of the circular tank is closed, the empty vertical collar is detached from the truck tank, and the door to the truck tank is sealed.

Figure 4-14
Water-to-Water Transfer from Holding Tank to Transport Truck, Courtesy of Puget Sound Energy

4.3.2.3 Maintenance

Maintenance activities can be broken into regular, annual, and infrequent periods.

Regular maintenance occurs throughout operation within the fish passage design flow range. It includes:

- Debris removal within the fish ladder and fish lift areas. Supply water is screened so debris is expected to come primarily from falling debris, such as leaves.
- Cleaning of tanks, holding galleries, work-up tables, flumes, and other holding and sorting facilities to ensure fish health.

Many features of the CHTR do not require maintenance on a regular basis but do require annual inspection and/or maintenance to keep them in good working order and prolong their functional life. Annual maintenance tasks may include:

- Service manual and motorized gate operators.
- Service trapping and lift systems, including repaint steel parts and service motors.
- Inspect, service, and replace if necessary, all gates, crowders, and other fish handling and sorting equipment pumps.
- Inspect fish screens and trashracks for damage.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Repaint buildings and steel pipes and structures.
- Replace trash rakes.
- Replace gate operators, trapping system components, lift system components, crowder motors, and other fish handling and sorting equipment.
- Replace Bypass Pipe(s) between Bypass Hoppers and the dam penetration as necessary due to damage from exposure, debris, or flood events.
- Replace in-line energy dissipation valve and other shutoff and control valves as necessary.

A summary of the anticipated level of effort for operation and maintenance is provided in the table below.

Table 4-9
Operation and Maintenance Level of Effort for the FRFA CHTR

ALTERNATIVE	AVERAGE ANNUAL DURATION	PERSONNEL REQUIREMENTS (FTE = FULL TIME EMPLOYEE)	SPECIAL CONSIDERATIONS
FRFA – Upstream Technical Fish Ladder	12 months	Operations Staff – 0.90 FTE Biological Staff – 0.76 FTE Maintenance Staff – 0.31 FTE Total = 1.97 FTE	• Operation of facility daily during operation. • Daily inspections required to maintain optimum hydraulic settings and perform debris removal.

4.3.2.4 Anticipated Fish Passage Performance and Survival

The following is the anticipated fish passage performance and survival for the Technical Fish Ladder alternative. A discussion of the rationale behind performance and survival value is provided in Attachment C of this document.

Table 4-10
Capture, Handling, Transport, and Release (CHTR), Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT UPSTREAM			
Spring Chinook	93%	98%	**91%**
Fall Chinook	93%	98%	**91%**
Coho	93%	98%	**91%**
Winter Steelhead	93%	98%	**91%**
Coastal Cutthroat	88%	98%	**86%**
Pacific Lamprey	60%	90%	**54%**
Western Brook Lamprey	60%	90%	**54%**
JUVENILE UPSTREAM			
Spring Chinook	60%	90%	**54%**
Fall Chinook	60%	90%	**54%**
Coho	60%	90%	**54%**
Winter Steelhead	65%	90%	**58.5%**
Coastal Cutthroat	60%	90%	**54%**

The anticipated fish passage performance and survival is based on the performance of other CHTR facilities and is adjusted based on conditions that are unique to the alternative proposed for the FRFA dam on the Chehalis River. There are numerous examples of trap and transport facilities in the Pacific Northwest that collect and transport adult anadromous salmonids with high levels of performance and with very low levels of injury or direct mortality. Following are a few examples that were used as a basis of comparison:

- Merwin Dam Adult Collection Facility – Lewis River, Washington State
- North Fork Adult Sorting Facility – North Fork Clackamas River, Oregon State
- Lower Baker Adult Collection Facility – Baker River, Washington State
- Cougar Dam Adult Collection Facility – South Fork McKenzie River, Oregon State
- Cowlitz Adult Collection Facility – Cowlitz River, Washington State
- White River Diversion Dam Adult Collection Facility – White River, Washington State
- Minto Adult Collection Facility – North Santiam River, Oregon State
- Foster Fish Collection Facility – South Santiam River, Oregon State

A number of factors specific to the FRFA dam, this location, and the Chehalis River adjust the expected performance and survival for this alternative compared with other similar facilities. Some of these factors include:

- Modern adult collection facilities are typically designed for the collection of adult upstream migrating salmonids. Juvenile collection during upstream migration has historically been incidental, and, therefore, limited data exists.
- There is a higher level of confidence that a CHTR facility will perform well for upstream migrating adult Chinook, coho, and steelhead. Like facilities had performance and survival values around 90 percent and 98 percent, respectively.
- Reduced performance and survival values were provided for cutthroat trout compared with adult Chinook, coho, and steelhead. The reduced performance value is most substantially attributed to the general focus of other facilities on adult salmonids and the lack of data on cutthroat trout collection.
- Upstream migrating juvenile fish were given lower performance and survival values than those for adults. These values reflect the uncertainty of attracting fish into a ladder entrance, predation, and motivation to ascend the ladder into a holding gallery. Additional provisions could be engineered into a facility of this nature to improve juvenile fish collection and safe transfer. These include multiple low-head entrances, lower head differential between pools, and segregation zones in holding galleries to decrease predation. Such provisions should be explored by the fish passage subcommittee during preliminary design. However, changes made to improve juvenile fish collection might adversely affect adult fish collection and should be evaluated accordingly.
- The reduced size, motivation, and swimming capability for juvenile cutthroat trout resulted in lower juvenile upstream passage performance and survival values compared with those associated with adult upstream passage.
- Since lamprey passage through such facilities has not historically been a focus, limited data is available. Additional research is needed. One source of data may be ongoing efforts on the Mid-Columbia and Yakima Rivers, but this has not been investigated. This information can be used to inform revisions to the table as data becomes available. Given the high level of uncertainty associated with collection of lamprey in a trap and transport situation, lower performance and survival values were provided.

4.3.2.5 Reliability

The CHTR for the FRFA dam provides a moderate level of reliability. Reliability for the FRFA CHTR is nearly identical to that of the FRO CHTR discussed in Section 4.2.2.5 with one exception. It is expected that reliability for the FRFA CHTR will be slightly higher than the FRO CHTR as water for the FRFA CHTR will be supplied by gravity. A gravity water supply greatly reduces the mechanical and operational uncertainty associated with pumped supplies. However, the improvement in reliability due to a gravity water supply has a relatively minor impact on the overall reliability. As such, the reliability for the FRFA CHTR is also moderate.

4.3.2.6 Cost Summary

The CHTR has the lowest construction cost of all the upstream fish passage alternatives. The short height and length of the fish ladder, the minimal work on steep slopes, as well as the lack of necessity for physical structures for upstream release, more than offset the cost of sorting, holding, and transport facilities. The CHTR requires significantly more personnel and man-hours to operate and maintain. When the operation and maintenance cost is amortized, the total life-cycle cost of the CTHR is still substantially less expensive than the Technical Fish Ladders.

A summary of the estimated construction costs, included the estimated upper and lower bounds, is provided in Table 4-11 below. A detailed breakdown of construction costs can be found in Attachment D.

The CHTR has the highest operation and maintenance cost of the upstream passage alternatives. The CHTR is a fully manned facility. It requires a year-round, full-time staff to operate. Staffing is doubled 9 months of the year, when the majority of upstream passage is occurring. Additional temporary staff are brought in several times a year for short periods (less than 1 week) to conduct annual inspections and assist in larger maintenance efforts. Flow to the fish ladder and holding and transfer facilities is supplied by gravity via the dam regulating outlets. Regular maintenance of the fish ladder should be minimal as the supply water is fully screened and relatively little debris is expected to enter the fish ladder. The CHTR has many mechanical systems, such as fish crowders, oxygenation systems, and fish transfer equipment, so the annual maintenance is fairly high compared with other alternatives. A summary of the estimated O&M cost is provided in Table 4-11 below. A detailed breakdown of O&M costs can be found in Attachment D.

Table 4-11
FRFA CHTR Estimated Construction and O&M Costs

FISH PASSAGE OPTION	CONSTRUCTION COSTS LOWER BOUND COST ($ MILLION)	MIDDLE BOUND COST ($ MILLION)	UPPER BOUND COST ($ MILLION)	O&M COSTS ($)
FRFA – Upstream Fish Passage: CHTR Facility	$13.8	$18.4	$27.6	$375,000

4.3.3 Floating Surface Collector

The Floating Surface Collector alternative is intended to collect and safely pass outmigrating adult steelhead, juvenile salmonids, and juvenile and adult resident fish from the FRFA reservoir to a designated release point downstream of the FRFA dam structure. Attraction flows of 500 to 1,000 cfs are generated at the entrance to a large floating collection barge, which creates a positive flow-net that reaches out into the reservoir. Fish entering this "zone of influence" are triggered to move downstream toward the artificial reservoir outlet. Fish are guided from the reservoir into the entrance of the collection barge with the assistance of guidance and lead net systems. Fish entering the floating collection barge are moved via transport vehicle downstream of the dam and released into the Chehalis River. The floating surface collector system would operate year-round when reservoir elevations are within the anticipated normal operational range of WSEL 588 to 628.

4.3.3.1 Design Elements

In general, the Floating Surface Collector alternative consists of six primary design elements:

- Floating Collection Barge
- Net Transition Structure
- Mooring and Anchorage Systems
- Guidance and Lead Nets
- Fish Sorting and Holding System
- Fish Transport System

A summary of each primary design element for the Floating Surface Collector alternative is discussed further in the following paragraphs. An overall summary of the primary design elements and their respective configuration is illustrated in Figure 4-15. Figure 4-16 provides a more detailed illustration of the Net Transition Structure and Collection Barge elements.

Figure 4-15
Primary Design Elements of the FRFA Floating Surface Collector Alternative

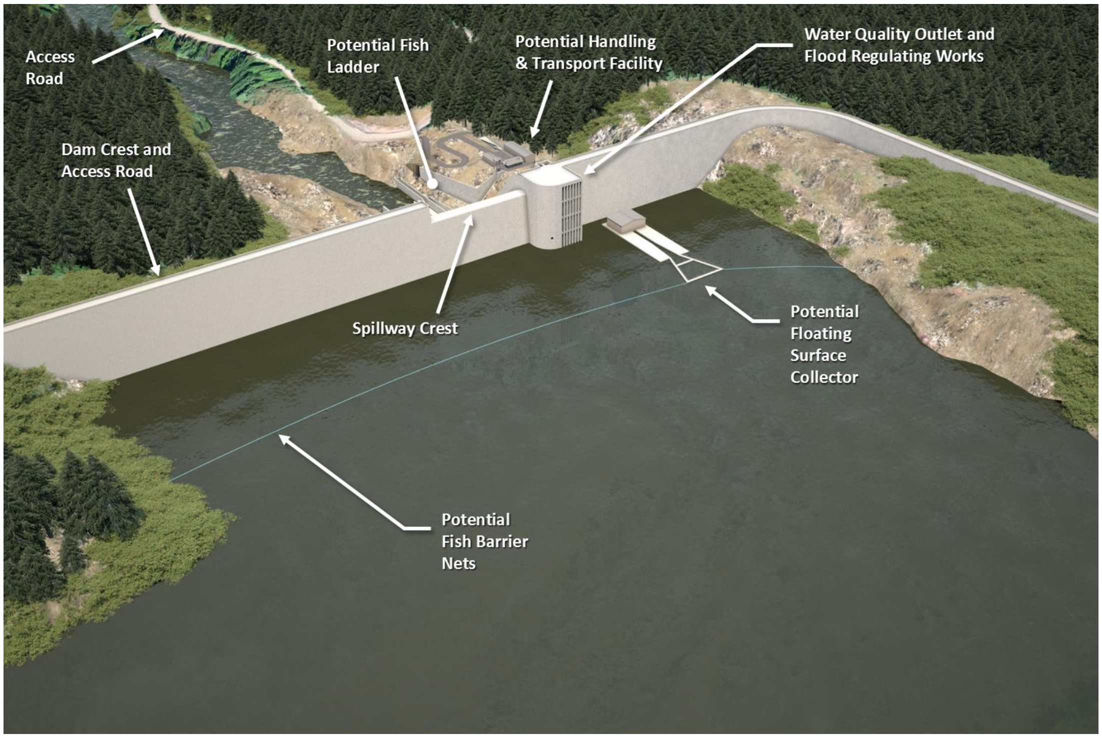
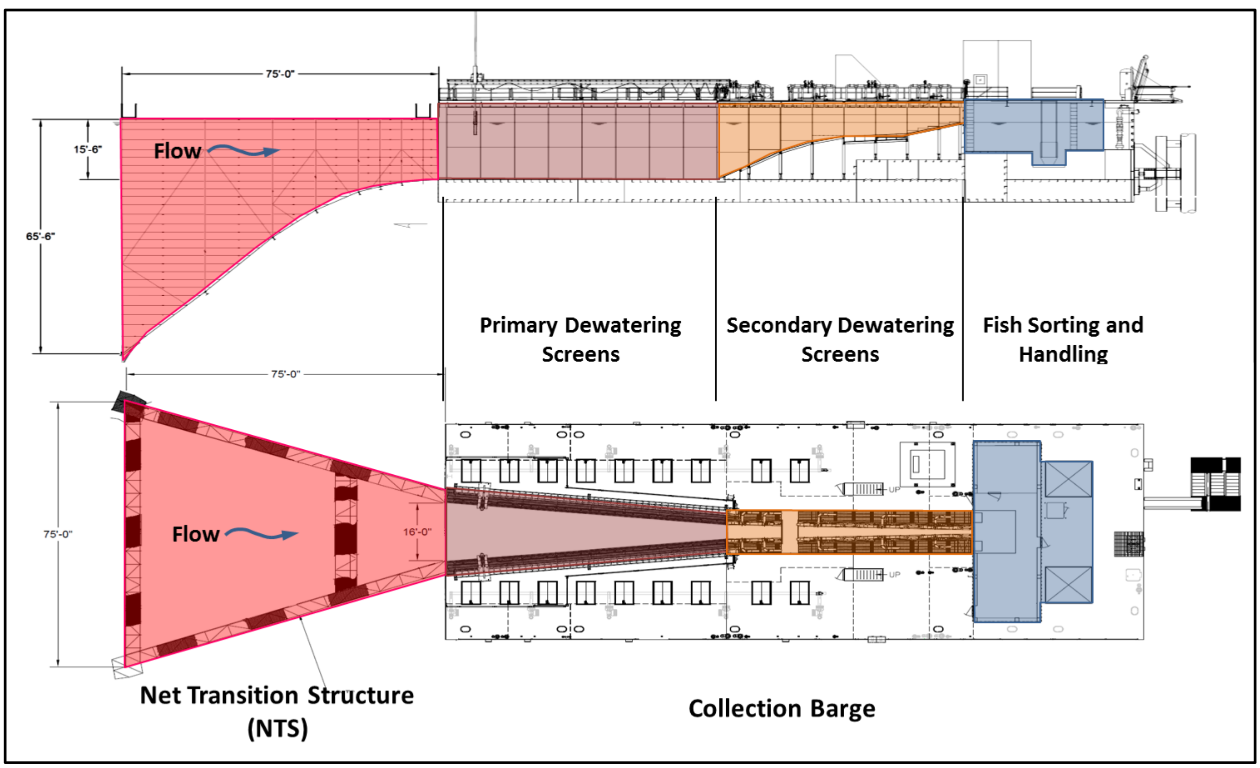


Figure 4-16
Floating Surface Collector - Net Transitions Structure and Collection Barge



4.3.3.1.1 Collection Barge

The collection barge is the most complex operational component of the Floating Surface Collector fish passage alternative. The collection barge is comprised of a 156-foot long by 60-foot wide floatation module sized to accommodate numerous onboard structural, mechanical, hydraulic, electric, and instrumentation systems necessary to accomplish the objective of fish collection. A brief summary of the potential onboard systems is provided in the following list of bullets:

- Floatation – With a total weight of almost 2,000,000 pounds, the collection barge is designed with a substantial system of ballast tanks aligned along its belly. Ballast tanks are operated automatically using a computer controlled system of monitoring, instrumentation, and flow control devices that add or remove ballast water from each ballast tank individually. Adjustments are made to ballast tanks to provide an even keel and to maintain a user desired draft that optimizes fish attraction flow and velocity into the inlet of the net transition structure and primary dewatering screens.
- Attraction Flow Pumps – Eighteen low-head, high-volume submersible pumps are required to generate target attraction flows of 500 to 1,000 cfs. Each attraction pump is located in a separate chamber connected to a joint hydraulic plenum that runs along the length of the dewatering screens. In general, an array of variable frequency drives (VFDs) and direct on-off controlled pumps provide user flexibility with respect to flow and flow distribution along the length of the dewatering screens.
- Dewatering Screens – Vertical flat-plate screen panels, vertical traveling screens, or a combination of potential screening technologies are used to remove water from the collection flume while safely bypassing the fish being collected. Each screening system will possess 1.75 mm slots and be designed to accommodate target approach velocities of 0.4 ft/s at an attraction flow of 500 cfs with a 15 to 20% factor of safety. The current conceptual design requires that the proposed primary screening bay is 66 feet in length with a maximum depth of 15.5 feet at the furthest upstream extent. The secondary screening bay is 50 feet in length with a depth of 7 feet at the upstream extent and depth of approximately 1-foot at the most downstream extent. As part of the dewatering screen system, automated mechanical screen cleaning equipment operate at regular intervals to keep the screen face free of debris.
- Porosity Control – A secondary set of adjustable porosity control panels are provided on the backside of the dewatering screens. The porosity control panels can be independently adjusted from the operations deck to facilitate optimum flow velocity acceleration and flow distribution along the length of the primary and secondary dewatering screen bays.
- Fish Sorting and Handling Facilities – At the most downstream extent of the secondary dewatering screen bay, approximately 6 cfs of flow and any fish that have been collected pass into the rear of the collection barge and enter a series of sorting and holding facilities. The fish

sorting and holding facilities conceptualized for this alternative are discussed further in the following sections.

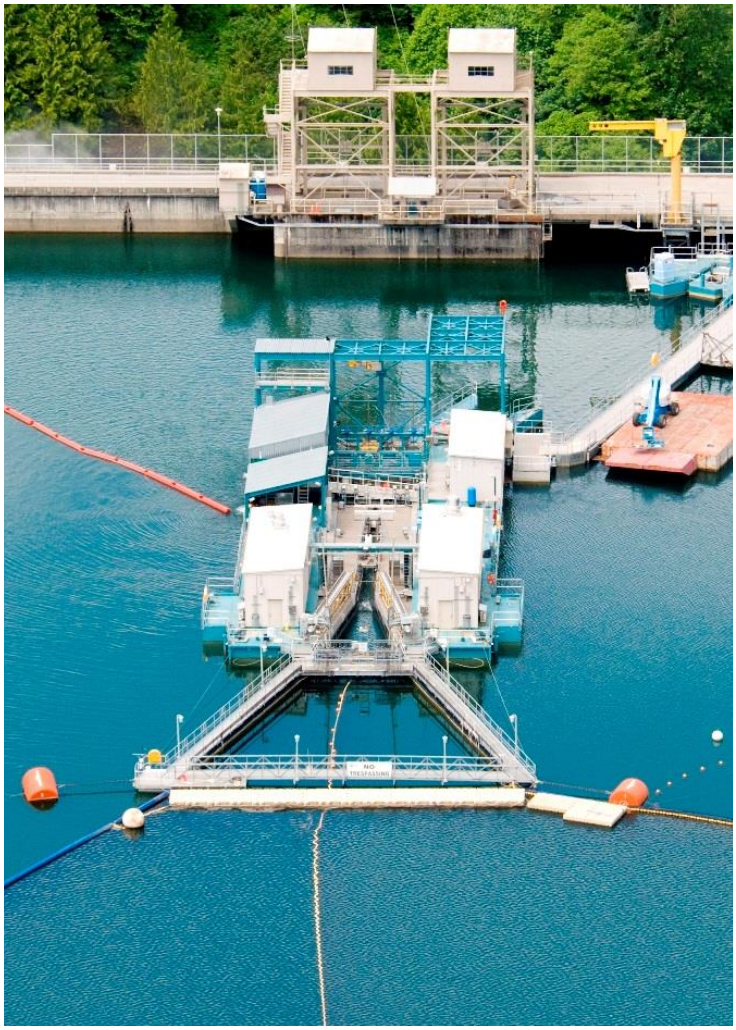
- Water Management and Removal – As water and fish are drawn into the collector, dewatering screens, porosity control panels, and attraction flow pumps are used to incrementally decrease the amount of water handled in the collection flume while safely guiding fish further into the collector towards the fish sorting and holding area. Water that is drawn through the dewatering screens bypasses the rest of the collection flume and is conveyed out through the attraction pumps. Water that remains in the collection flume is continuously conveyed downstream to the sorting and holding area. Eventually, the water is removed from the rear of the holding area and is discharged to the reservoir using another array of leveling pumps.
- Shore-Based Electrical Supply – The array of attraction pumps and on-board mechanical systems require a substantial level of electrical power to operate. It is anticipated for this concept that electrical power will be brought to the dam structure via transmission lines and shore-based service power will be routed to the primary electrical panel located on the collection barge. Facilities similar in concept are known to create approximately 1 megawatt of instantaneous electrical demand at an attraction flow of 1,000 cfs and a demand of 0.5 megawatts at an attraction flow of 500 cfs.
- Access, Shelters, and Safety – The deck of the collection barge is composed of numerous travel surfaces, shelters, stairs, and safety railing to accommodate safe, comfortable, and efficient access to all operational components of the collection barge.

Figure 4-17
Swift FSC Collection Barge, Dewatering Screens, Screen Cleaning Systems, and Shelter (Courtesy of PacifiCorp)

4.3.3.1.2 Net Transition Structure

The net transition structure (NTS) is composed of a system of large, fabricated, impermeable panels attached to a steel frame. The panels are positioned at the upstream face of the collection barge and are configured to create a gradual and uniform increase in flow velocity from the reservoir to the inlet of the primary dewatering screens. The net transition structure transitions over its 75 foot length from 75 feet wide and roughly 65 feet tall upstream to 16 feet wide and roughly 15 feet tall downstream. Floatation tanks on the top of the NTS on the port and starboard sides keep the structure afloat. Guide nets extend outward from the upstream edges of the NTS. A lead net extends outward into the reservoir from the middle of the upstream edge of the NTS. An example photograph of the Upper Baker NTS is shown in Figure 4-18.

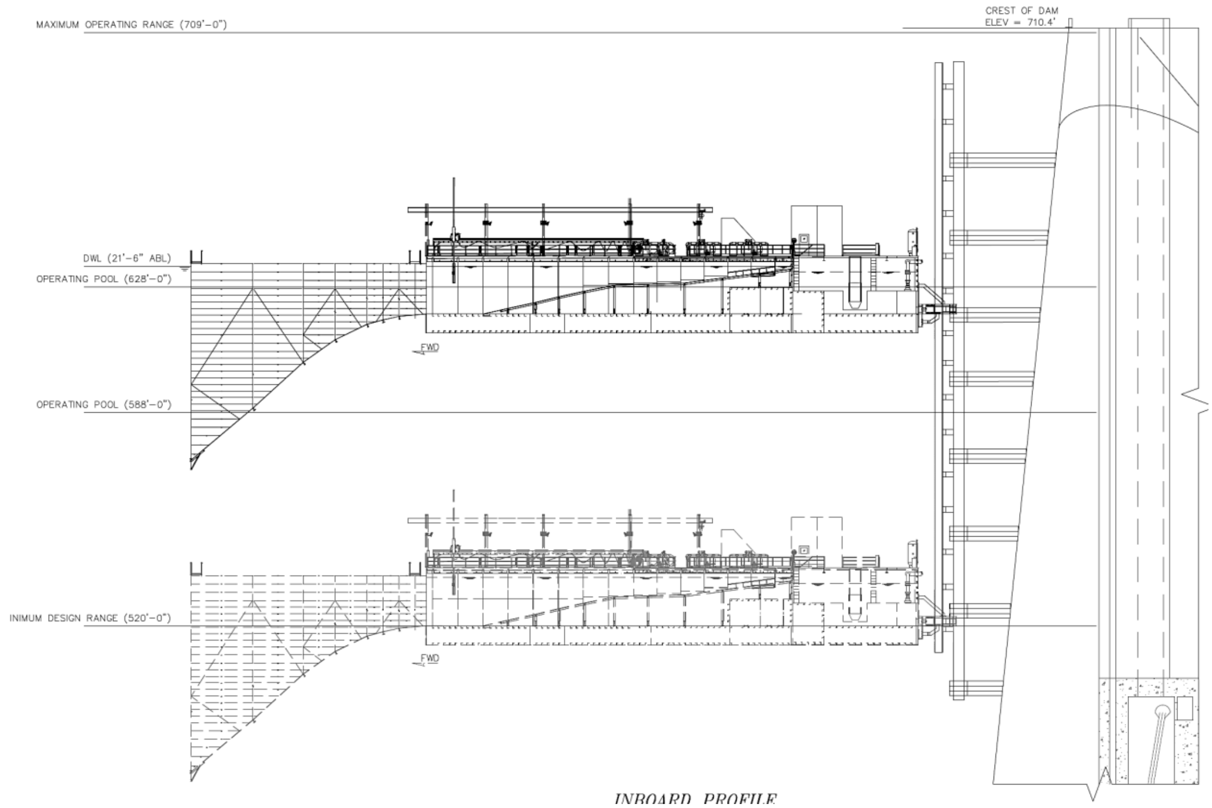
Figure 4-18
Upper Baker FSC Net Transition Structure in Foreground (Courtesy of PSE)



4.3.3.1.3 Mooring and Anchorage Systems

The collection barge is moored in a fixed horizontal position using a system of three vertical guide rails. Two guide rails located at the rear of the collection barge are structurally anchored to the upstream face of the FRFA dam, while one guide rail is located near the port bow of the collection barge and is independently supported by a system of piles. The FSC and piles are shown in relation to the dam in figures FRFA-C-9 and FRFA-C-10 in Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017) and described in more detail in the Floating Surface Collector Technical Memorandum in Attachment E. The guide rails extend vertically approximately 200 feet allowing the collection barge to rise and fall with the total anticipated reservoir fluctuation of 189 feet. The NTS is anchored to the upstream face of the collection barge. The NTS rises and falls with the reservoir in unison with the collection barge. Figure 4-19 provides an illustration of the guide rail system mounted to the face of the dam as well as the potential vertical positioning of the collection barge.

Figure 4-19
Floating Surface Collector Mooring System Mounted to Face of Dam FRFA Dam Structure



4.3.3.1.4 Guidance and Lead Nets

Guidance nets extend upstream from the corners of the NTS horizontally to the shore and vertically to the reservoir bottom. A lead net extends from the center of the NTS at its upstream end horizontally upstream several hundred feet and vertically to the bottom of the reservoir. Guidance and lead nets accomplish two objectives: 1) create a positive physical barrier that excludes fish from traveling downstream of the collection barge; and 2) create a smooth transitional pathway from the natural shoreline to the net transition structure. Guidance nets are anchored to both vertical edges of the net transition structure and extend outward at an angle to the natural shoreline. A total net length of just over 2,000 feet may be required but the final shore anchorage location is still under evaluation. The guidance nets extend vertically from the reservoir surface to the floor of the reservoir and are suspended by a linear array of floatation booms and weights. Floatation is provided at the reservoir surface as well as at approximately half depth of the water column. The top half of the net is weighted and neatly folds down upon the mid-level array of floatation booms as the reservoir elevation rises and falls. Ballast water within the main floatation support system can be adjusted so that the top half of the net can be lowered below the water surface during emergency flood operations to protect the net system from damage and to allow debris to pass over the spillway.

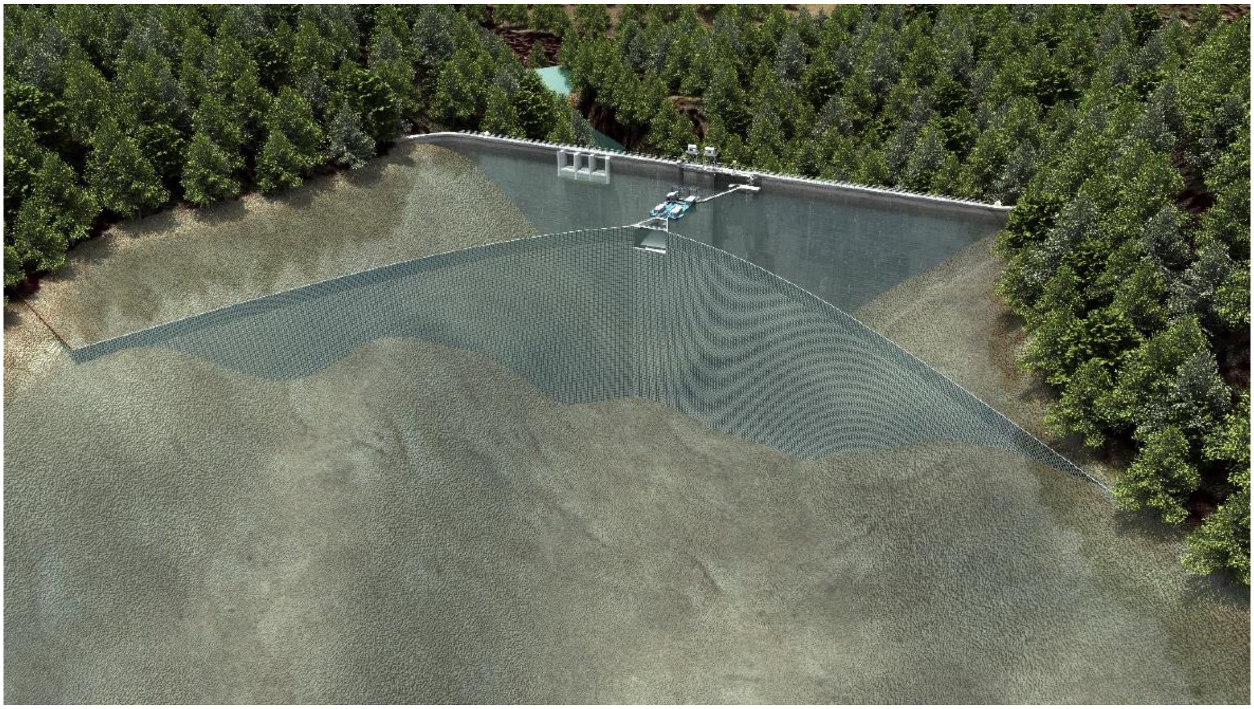
A single lead net originates in the center of the net transition structure and extents straight upstream, splitting the net transition structure down the middle. The lead net extends upstream up to 300 feet with the purpose of intercepting fish that may be traveling across the reservoir in cross-currents and leading them deeper within the zone of influence and eventually into the entrance of the collection barge.

Both net systems are to be constructed of Dyneema netting with varied opening sizes. In general, the top netting panels typically exhibit circular openings with a diameter of 3/32nds of an inch. The lower netting panels can exhibit larger openings in excess of 1/4th of an inch. In the past, many of the nets were constructed with nylon derivatives and they performed poorly. Recently, existing facilities have been replacing their net systems using the Dyneema or Spectra product as they are stronger and more durable which ultimately reduces maintenance and replacement costs over the life of the project.

Anchorage for each netting element must be considered more carefully during the next phase of design development. At this stage of development the intent is to have multiple anchorage points along the shoreline to accommodate flexibility and adjustability during future operations.

An illustration showing the guidance nets for the Lower Baker Floating Surface Collector is provided as Figure 4-20.

Figure 4-20
Example 3D Rendering of Full Guidance Nets on the Lower Baker FSC (Courtesy of Pacific Netting Products)



4.3.3.1.5 Fish Handling Systems

Water and fish that pass through the collection flume are conveyed into the rear of the collection barge where a series of sorting and holding facilities reside. Initially, fish are conveyed down a grading flume which separates received fish into three separate size classes: fry, smolt, and adult sized fish. Fry sized fish less than 80 mm move through narrow grader openings and are conveyed to a separate fry holding pool. Smolt-sized fish from 80 to 160 mm pass through a wider set of grader bars and are conveyed to a second holding pool. Larger fish above 160 mm pass over the remainder of the flume and are conveyed to a third holding pool. Combined, all holding areas are sized to accommodate the anticipated peak rate of migration: 55,000 juvenile outmigrants per day. Here, fish can be held for a maximum of 24-hours prior to being moved downstream.

Figure 4-21
Example Grading Flume and Holding Pools Present on the Cushman FSC

4.3.3.1.6 Transport Systems

When desired, fish present in the fish holding area can be transported downstream via a truck and transport operation. Technicians begin the process by operating mechanical crowders which encourage the two larger size classes of fish into one of two separate hoppers. The hoppers are raised out of the collection barge using an overhead gantry crane mounted on the dam and placed on the bed of a transfer vehicle. Fry are hand carried up in a separate fry box and placed in the same transport vehicle. All fish are then transported downstream to monitoring and evaluation facilities downstream and are eventually released back into the Chehalis River downstream of the FRFA dam structure where they can continue their migration downstream. Examples of a fish hopper and the process of raising the hopper vertically to a transport vehicle are illustrated in Figure 4-22 and Figure 4-22.

Figure 4-22
Example Fish Hopper at Cushman Adult Collection Facility (Courtesy of Tacoma Power)

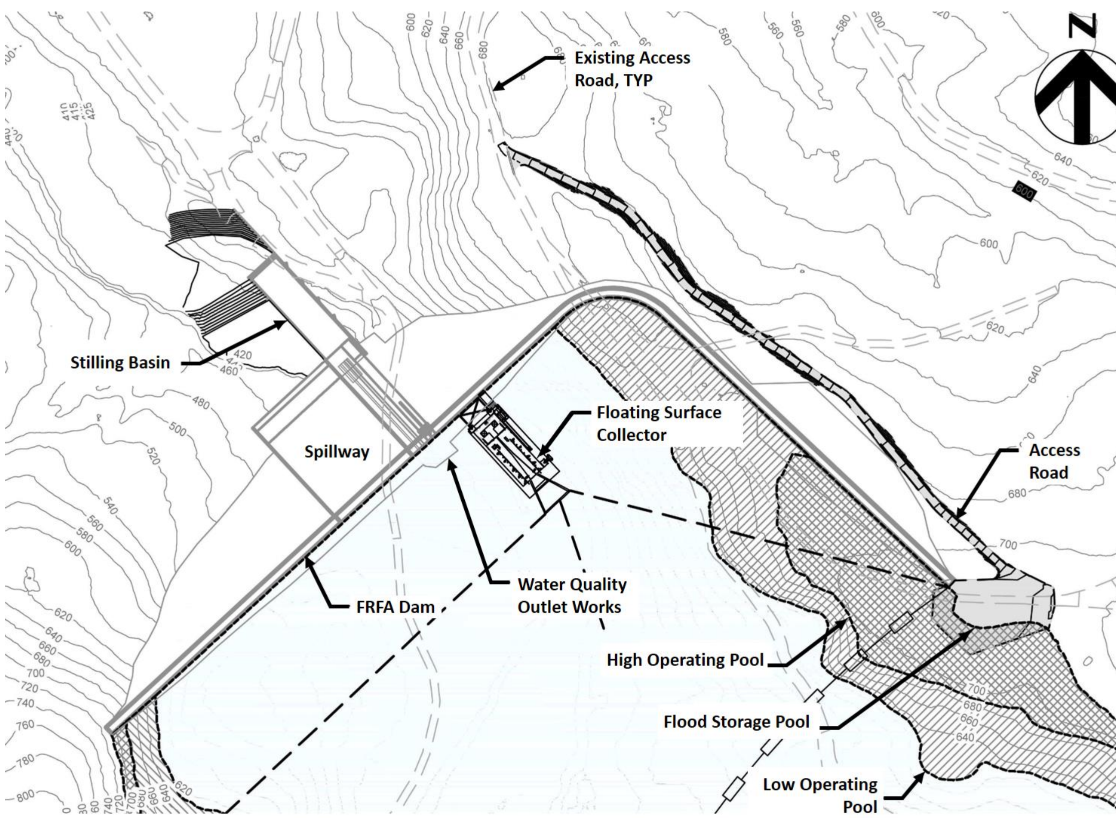
Figure 4-23
Example process of Lifting Hopper to Transport Vehicle at Swift FSC (Courtesy of PacifiCorp)

4.3.3.2 Theory of Operation

The floating surface collector is intended to provide safe passage for downstream migrating juvenile salmonids, adult steelhead, and resident fish year-round, within the normal anticipated operational range of reservoir pool elevations. During operations outside of typical reservoir elevation levels, the floating surface collector must be able to accommodate the full range of potential reservoir elevations without damage. The floating surface collector is also intended to provide resident fish the ability for downstream migration throughout the year.

The floating surface collector will operate normally within a 40-foot range of reservoir elevations with the capability of safely accommodating a potential fluctuation of up to 189 feet during episodic flood or drought operations. The conservation pool of the FRFA reservoir is expected to fluctuate normally between WSEL 588 to 628 (40 feet) over the course of a year, as shown in Figure 4-24, and is regulated to seasonally enhance water quality and instream flow downstream of the FRFA dam structure. Flood events may bring the reservoir pool higher than WSEL 628; potentially as high as WSEL 709 in a PMF event. Extreme drought or reservoir drawdown conditions may bring the reservoir pool as low as WSEL 520. During flood or extreme drought events, the floating surface collector will be placed in a nonoperational state but will rise or lower vertically to the high and low elevation limits safely on the guide rails and without damage to its components. It is expected that significant debris removal and inspection of the facility will be required following flood events prior to startup.

Figure 4-24
FSC and FRFA Reservoir Inundation Map



The fish screens located on the collection barge are designed to be in conformance with NMFS juvenile screening criteria up to an anticipated attraction flow of 500 cfs (inclusive of a factor of safety). The onboard low-head attraction pumps have capability to increase the attraction flow up to a maximum of 1,000 cfs. Approach velocity increases proportionally at the higher attraction flow. Other like facilities are known to operate at these higher attraction flows, outside of the standardized approach velocities, for several months out of the year to improve collection efficiency during the peak rate of outmigration. Typically, these periods of higher attraction flows correspond with migration periods when larger smolts with greater swimming capability are anticipated to occur in the reservoir. For the purposes of estimating costs and level of effort for the facility, it is assumed that the facility is operated for 10 months a year with an attraction flow of 500 cfs and two months of each year with an attraction flow of up to 1,000 cfs. This is consistent with how other facilities are operated in the Pacific Northwest, such as Upper and Lower Baker.

Full-depth guidance nets are placed upstream of the net transition structure and expand outward to points on the left and right banks of the reservoir. The guidance nets are attached to the upstream end of the net transition structure at its port and starboard corners. The tops of the nets are designed to rise and fall with the reservoir to ensure fish guidance is maintained for the targeted normal reservoir fluctuation. At extreme flood stages, the linear array of floats that support the top of the nets are allowed to fill with water and the tops of the nets submerge, allowing for debris passage over the spillway. Additional systems of floats and weights attached to the nets lower in the water column allow the guidance nets to fold at designated locations to maintain a full exclusion barrier throughout the storage pool range without binding or other damage to the nets. However, because of the proximity of the net systems to the spillway, it is likely the nets will be laden with debris and possibly damaged during extreme events. As with the collection barge itself, the nets will require debris removal and careful inspection by divers prior to startup after large flood events.

The attraction flow pumps draw in flow to the inlet of the net transition structure, creating a positive flow net toward the collection barge and along the edges of the fish guidance nets. Outmigrating fish sense the positive movement of water and are behaviorally triggered to move toward the simulated outlet of the reservoir. As the fish move toward the inlet to the collection barge, the gradually varying changes in geometry of the net transition cause fish to experience a gradual and uniform increase in flow velocity. As water is drawn into the collection barge, dewatering screens begin to uniformly remove the water that passes down the gradually narrowing collection channel. This process continues until the majority of the water has passed through the primary dewatering screens and approximately 105 cfs is moving at a targeted capture velocity of 8 ft/s. Any fish present within the water column are considered captured at this point and continue downstream through a short deceleration flume where the flow is reduced to 6 cfs and a velocity of approximately 6 fps. Here, fish pass through a grader flume that separates them into three specific size classes and conveys them into separate holding pools at the rear of the collection barge (for example: fry less than 80 millimeters (mm); smolt, juveniles, and residents up to 160 mm; and fish larger than 160 mm, which may be post-spawn adult steelhead or larger cutthroat trout or bull trout). From the holding pools, technicians operate mechanical crowders that encourage the two larger size classes of fish into one of two hoppers. The hoppers are raised out of the collection barge using an overhead gantry crane and placed on the bed of a transfer vehicle. Fry are hand carried up in a separate fry box and placed in the same transport vehicle. All fish are then transported downstream to monitoring and evaluation facilities, and are eventually released back into the Chehalis River downstream of the FRFA dam structure, where they can continue their migration downstream.

4.3.3.3 Maintenance

The Floating Surface Collector is comprised of numerous complex mechanical, structural, and floatation systems that must all work in tandem to produce adequate levels of performance. Maintenance activities can be broken into regular, annual, and infrequent periods.

Regular more frequent maintenance is anticipated to occur throughout the normal operation of the facility. It can be expected to include:

- Weekly and sometimes daily removal of debris inside the screen structure and fish handling systems. Debris material will consist of buoyant and semi-buoyant material small enough to fit through the primary trashracks which reside in front of the net transition structure.
- Weekly and sometimes daily removal of debris from the primary trashracks.
- Inspection and adjustment of mechanical screen cleaning equipment.
- Inspection and adjustment of ballast, floatation, and hydraulic control systems.
- Typical vehicle maintenance activities for hopper transfer trucks and facility support boats.

It is expected that several maintenance tasks will be required on an annual basis on the Floating Surface Collector. Annual maintenance tasks may include:

- Inspect, service, and replace if necessary all pumping equipment associated with attraction flow, ballasting systems, and service water.
- Service screen cleaning system.
- Inspect and service ramp gates and hoppers.
- Inspect fish screens and trashracks for damage.
- Inspect, modify, repair, and/or adjust porosity control systems behind the primary and secondary dewatering screens.

Infrequent maintenance primarily refers to maintenance that is conducted on an as needed basis. It may be several years between such maintenance activities or as components unexpectedly fail while in use. Infrequent maintenance activities may include:

- Repaint steel structures including gantry equipment, access tower, guide rails, mooring towers, screen cleaning systems, and ramp gates and hoppers.

- Replace pumping equipment associated with attraction flow, ballasting systems, and service water as necessary.
- Replace or repair trash rakes.
- Replace gate operators.
- Replace instrumentation and control equipment associated with all ballast, hydraulic, and mechanical monitoring systems.
- Replace attraction water pumps as necessary due to damage or normal wear.
- Repair or provide replacement parts for hopper transfer trucks and facility support boats.

A summary of the anticipated level of effort for operation and maintenance is provided in the table below.

Table 4-12
Operation and Maintenance Level of Effort for the Floating Surface Collector

ALTERNATIVE	AVERAGE ANNUAL DURATION	PERSONNEL REQUIREMENTS (FTE = FULL TIME EMPLOYEE)	SPECIAL CONSIDERATIONS
FRFA – Downstream Floating Surface Collector	12 months	Operations Staff – 1.58 FTE Biological Staff – 1.26 FTE Maintenance Staff – 0.59 FTE Total = 3.43 FTE	• Operation of facility daily during operation. • Daily inspections required to maintain optimum hydraulic settings and perform debris removal. • Operation of transport vehicle required daily during operation. Creates substantial power demand of up to 1-megawatt during operation.

4.3.3.4 *Anticipated Fish Passage Performance and Survival*

The following table summarizes the anticipated fish passage performance and survival for the Floating Surface Collector alternative. A discussion of the rationale behind performance and survival value is provided in Attachment C of this document.

Table 4-13
Floating Surface Collector Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT DOWNSTREAM			
Spring Chinook	-	-	**-**
Fall Chinook	-	-	**-**
Coho	-	-	**-**
Winter Steelhead	50%	80%	**40%**
Coastal Cutthroat	65%	80%	**52%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
JUVENILE DOWNSTREAM			
Spring Chinook	65%	98%	**64%**
Fall Chinook	65%	98%	**64%**
Coho	65%	98%	**64%**
Winter Steelhead	65%	98%	**64%**
Coastal Cutthroat	65%	98%	**64%**
Pacific Lamprey	3%	10%	**0.3%**
Western Brook Lamprey	3%	10%	**0.3%**

The values in Table 4-13 are based on the performance of other floating fish collection facilities currently in operation at other locations, taking into account the unique conditions at the proposed FRFA dam. Existing facilities used to inform selection of performance and survival values are listed in Table 4-14. Table 4-14 lists some key factors influencing the performance and survival, however, the primary influences on the performance and survival of floating surface collectors is still being studied. The Fish Passage Technical Committee considered and discussed many other factors specific to each site in developing the estimated performance and survival for this alternative.

Table 4-14
Summary of FSCs Currently in Operation Used to Inform Design and Performance

FACILITY	OWNER - LOCATION	VERTICAL FLUCTUATION	ATTRACTION FLOW	FISH TRANSPORT	1ST YEAR OF OPERATION
Upper Baker	PSE - Baker River, WA	30 to 60	500/1,000	Trap and transport	2008
Lower Baker	PSE - Baker River, WA	30 to 60	500/1,000	Trap and transport	2013
Swift	PacifiCorp -Lewis River, WA	100	500/750	Trap and transport	2012
North Fork	PGE - Clackamas River, WA	2 to 10	600/1,000	7-mile long bypass conduit	2015

FACILITY	OWNER - LOCATION	VERTICAL FLUCTUATION	ATTRACTION FLOW	FISH TRANSPORT	1ST YEAR OF OPERATION
Cushman	Tacoma Power - Skokomish River, WA	20	250	Trap and transport	2015

The following are assumptions made regarding the performance and survival of fish passage through a Floating Surface Collector at the proposed FRFA dam on the Chehalis River:

- The selected performance value is intended to accommodate fish passage from the head of the reservoir to the point of release downstream of the dam.
- It is assumed that collection performance of downstream migrating juvenile salmonids that have entered the zone of influence can be achieved in excess of 90 percent with the implementation of adequate guidance nets, lead nets, and attraction flow (e.g. - Upper Baker, Lower Baker, and North Fork FSCs).
- Swift and Cushman facilities listed above have reported combined reservoir transit and collection values on the order of 19 to 26 percent as a result of reservoir temperature, stratification, circulation, and fish transit issues. Many of these concerns are addressed through site-specific solutions integrated into the facility after the first several years of operation.
- The success of reservoir transit at this location is speculative, and site-specific studies are likely to be required after implementation to determine actual performance. All values were reduced from the expected facility performance to account for this uncertainty.
- Like facilities of this nature typically experience survival rates meeting or exceeding 98 percent for juvenile salmonids and resident fish.
- Juvenile lamprey passage performance and survival values are anticipated to be low given the uncertainty and lack of data available for lamprey and use of this type of fish passage technology. It is speculated that juvenile lamprey (ammocoetes and macropthalmia) will move to the head of reservoir. Although values may be higher than 30 percent in practice, the selected value was kept low to be conservative and represent an appropriate level of uncertainty.

4.3.3.5 Reliability

The Floating Surface Collector (FSC) provides a moderate level of reliability. Many components of the FSC, such as the water supply/attraction pumps, gates, valves, hopper, transport truck connections, and so on, are nearly identical to that of the FRO CHTR discussed in Section 4.2.2.5. However, the FSC has several additional systems and operational aspects that influence its reliability as well. Two primary aspects are reservoir transit and FSC flotation.

One of the primary aspects of FSC operation that impacts anticipated reliability is the success of reservoir transit. It is well documented at multiple other FSCs that, once fish enter the net transition structure, their survival is consistently high (see Attachment C). However, the success of getting fish from the head of the reservoir into the facility is highly variable. Studies have shown that reservoir transit success is dependent on multiple factors, including water temperature, thermal stratification, reservoir circulation patterns, and predation. The uncertainty associated with reservoir transit reduces the reliability of the FSC alternative but not so substantially as to lower the anticipated reliability to a low rating.

Another driver of reliability for the FSC is the floatation of the structure. Effective movement and capture of fish within the FSC is highly dependent on the stability and control of the floatation of the structure. Differences in the draft of the FSC from the required values and variations in the draft between corners can create suboptimal hydraulic conditions in the capture channel, resulting in fish rejection and escapement and reduction in overall performance. Appropriate design of floatation and stability and correctly anticipating environmental conditions such as wave magnitude, duration, and frequency, wind conditions, reservoir currents, and momentum of attraction flow pump discharges, are critical to ensuring proper the hydraulic operating conditions can be met. The uncertainty associated with being able to meet these conditions also adversely impacts reliability.

Floating surface collectors are more complex trap and transport facilities than fixed collectors. However, the basic function of the facility, collecting downstream migrants, is the same, indicating a moderate level of reliability. Impacts to reliability of the alternative due to the uncertainty of performance in reservoir transit and meeting hydraulic operating conditions reduce the level of reliability from that of the traditional CHTR facility described in Section 4.2.2.5. However, these impacts are not significant enough to offset the well-documented reliability of the technology at other dams and lower the general anticipated reliability for the FSC alternative.

4.3.3.6 Cost Summary

There are five full-scale facilities similar to the Floating Surface Collector conceptualized for the Chehalis River. Construction costs for existing facilities are reported to range from $24 million (Cushman) up to $60 million (Swift). Construction costs include facility fabrication and initial deployment, however they do not represent total implementation costs, which are reportedly in excess of $100 million for facilities like Upper Baker, Lower Baker, and Swift. A breakdown of implementation costs is provided in Attachment D.

Costs for a Floating Surface Collector at the Chehalis River are anticipated to be similar to that of other facilities currently in operation. The collection barge, guide nets, lead net, mechanical, fish transport systems and operational parameters are similar in nature to other facilities built within the last decade. Information and knowledge from the construction and implementation of these other existing facilities enhances the level of confidence of the anticipated costs for the FSC alternative. However, the mooring and anchoring systems for this facility are unique, having to accommodate approximately 189 feet of reservoir fluctuation – only comparable Swift at 100 feet. Therefore, some additional variances and contingencies must be accommodated for this individual design element to adequately capture anticipated costs.

A summary of the estimated construction costs, included the estimated upper and lower bounds, is provided in Table 4-15 below. A detailed breakdown of construction costs can be found in Attachment D.

The Floating Surface Collector (FSC) has the highest operation and maintenance cost of the all the passage alternatives. The FSC is a fully manned facility. It requires a year-round, full-time staff to operate. Staffing is doubled 6 months of the year, when the majority of downstream passage is occurring. Additional temporary staff are brought in several times a year for short periods (less than 1 week) to conduct annual inspections and assist in larger maintenance efforts. Attraction flow through the FSC and flow for the holding and transfer facilities is supplied by a system of on-board pumps. The pump system runs continuously, year-round. Roughly 60% of the annual O&M cost is for the purchase of power. The FSC has many mechanical systems, including attraction pumps, screen cleaning systems, circulation pumps for life support systems, fish crowders, oxygenation systems, fish transfer equipment, and automation software and hardware so the annual maintenance is fairly high compared with other alternatives. A summary of the estimated O&M cost is provided in Table 4-15 below. A detailed breakdown of O&M costs can be found in Attachment D.

Table 4-15
FRFA FSC Estimated Construction and O&M Costs

FISH PASSAGE OPTION	CONSTRUCTION COSTS			O&M COSTS
	LOWER BOUND COST ($ MILLION)	MIDDLE BOUND COST ($ MILLION)	UPPER BOUND COST ($ MILLION)	($)
FRFA – Downstream Fish Passage: Floating Surface Collector	$62.0	$82.6	$123.9	$858,000

4.3.4 Fixed Multi-Port Fish Collector with Fish Bypass Conduit

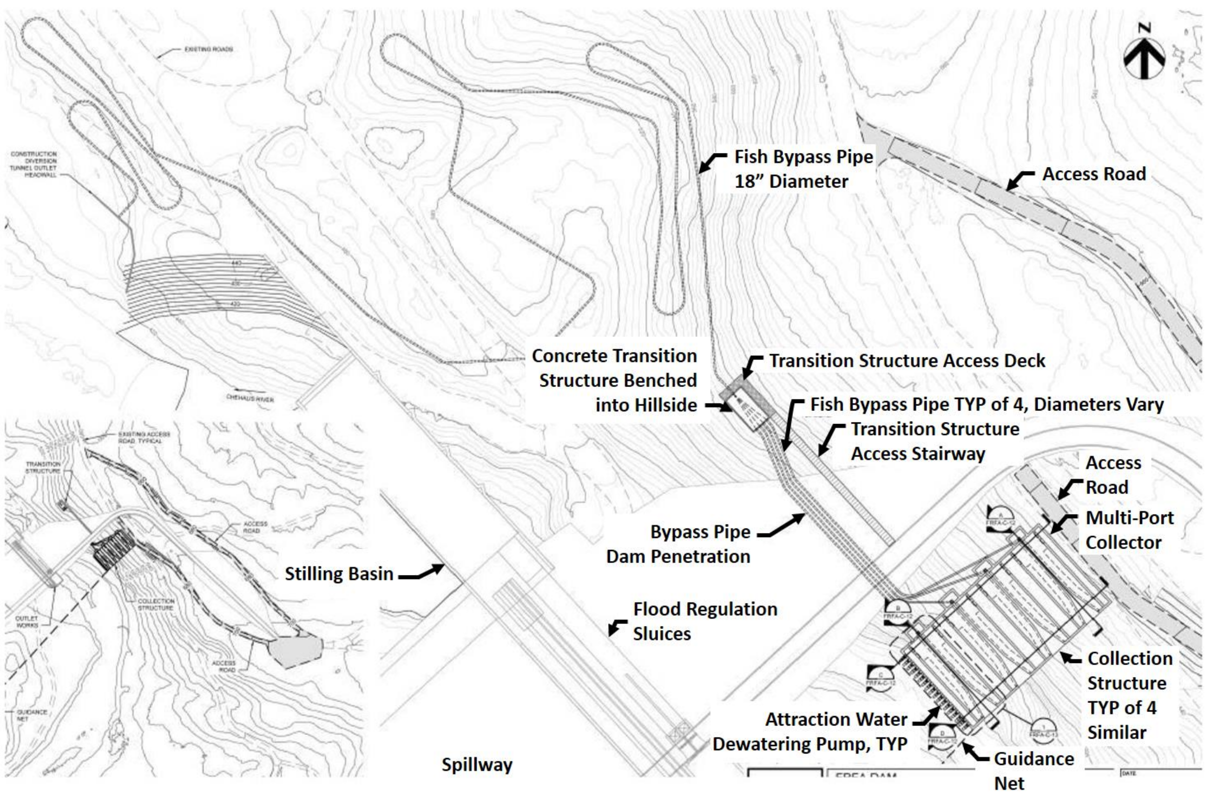
The fixed multi-port fish collector with fish bypass conduit (Multi-Port Collector) alternative is intended to collect downstream juvenile migrants, resident fish, and post-spawn adult steelhead and pass them safely downstream of the FRFA dam. The multi-port collector consists of four sets of vee-type dewatering screens, each staggered 10 feet vertically to provide downstream collection throughout the full range of normal reservoir fluctuation, from WSEL 628 to WSEL 588. Low-head, high-flow pumps pull water through the fish screens into a pump plenum common to all four sets of screens. Water drawn into the screens creates attraction for aquatic species moving downstream. A small amount of water remains in the channel at the end of the screens and carries species over a weir and into a bypass conduit. The bypass conduit carries species under pressurized flow downstream and through the dam at velocities faster than they can escape. On the downstream side of the dam, the bypass conduit transitions to open channel flow and enters a consolidation structure where the individual bypass pipes from each fish screen transition to a single pressurized conduit that carries the species downstream. The downstream moving species are returned safely to the Chehalis River downstream of the dam via an outfall structure. The outfall structure discharges into a deep, swift-moving pool in the river channel, where the aquatic species can continue their migration or volitional movement.

4.3.4.1 Design Elements

The Multi-Port Collector consists of several design elements, including:

- Fixed inlets with dewatering screens,
- A bypass penetration through the dam,
- A bypass consolidation structure,
- A bypass conduit from the consolidation structure to the river, and
- A bypass outfall.

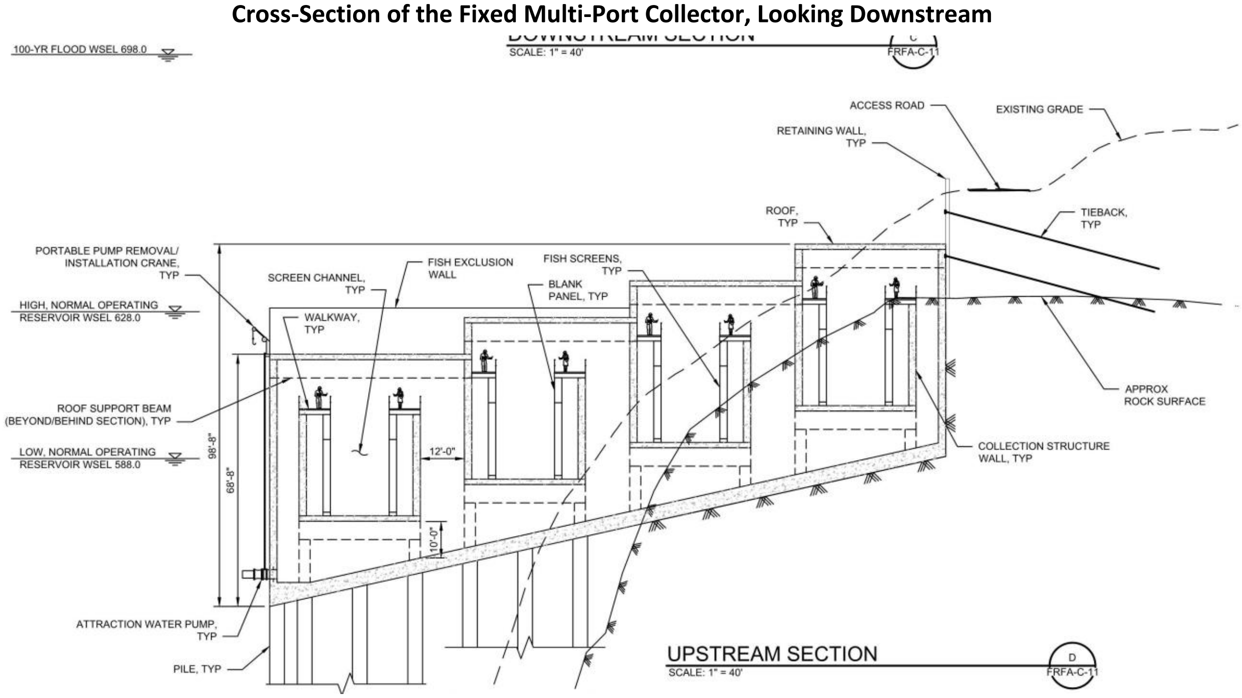
Figure 4-25
Fixed Multi-Port Collector Plan



4.3.4.1.1 Fixed Inlets with Dewatering Screens

The Multi-Port Collector is a single concrete structure that contains four fixed inlets with dewatering screens that are vertically staggered and discharge to a common pump plenum (Figure 4-26). The structure is 114 feet long by 185 feet wide by 96 feet tall. It is oriented such that the length of the structure is perpendicular to the dam. The structure is benched into the hillside such that roughly half of it is founded directly on rock. Due to the steep slope of the hillside, the left side of the structure must be supported on piles. Some of the piles will be over 100 feet in length. A retaining wall, such as a soldier pile wall with tiebacks will likely be required to retain the non-rock portion of the benched hillside. The retaining wall may be more than 60 feet tall. Further excavation of the hillside upstream of the structure will be required to allow aquatic species to access to the upper inlets. Similarly, aquatic species will be excluded from the Water Quality Outlet Works and Flood Regulation Sluices by fine-mesh barrier nets located just upstream of these intakes and extended the full depth of the reservoir.

Figure 4-26
Cross-Section of the Fixed Multi-Port Collector, Looking Downstream



A thick foundation slab will likely be required to support the weight of all four screens and accommodate the differing foundation conditions. The pump plenum is sloped to the left, maintaining a minimum 10 foot depth in the plenum between the plenum floor and the bottom of the screen channel slab. Dewatered flow from the screens is discharged out the left side of the plenum via 18 low-head submersible pumps.

Each screen structure is supported on columns founded on the plenum slab. Multiple deep concrete beams support the thin screen-structure slab. 20-foot-tall fish screens line either side of the screen channels. As flow moves downstream from the inlet, toward the dam, the screen channel narrows from 16 feet wide to 2 feet wide. The floor of the screen channel begins to rise about halfway downstream from the inlet. The channel floor transitions to a movable ramp that changes slope to accommodate the normal fluctuation in the reservoir. A 30-inch by 30-inch hopper is attached to the downstream end of the traveling ramp. The hopper extends 11 feet below the top of the ramp before beginning a square-to-round transition. A vertical steel pipe is fixed to the bottom of the transition and slides within a receiving pipe. The receiving pipe is fixed and continues into a 15-foot-radius sweep that brings the pipe back to an elevation necessary to accommodate the dam penetration. Plans and sections of the screen structures are provided in figures FRFA-C-11 through FRFA-C-13 in Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017) .

4.3.4.1.2 Bypass Penetration through the Dam

The bypass pipes from each screen structure are routed downstream to penetrate the dam at a common invert elevation and as close to each other as safe and practical. Downstream of the dam, each of the pipes continues to a consolidation structure. Each of the four bypass pipes between the multi-port collector and the consolidation structure is a different size to accommodate the physical and hydraulic situations of each screen structure. Additional detail showing the bypass penetration through the dam is shown in figure FRFA-C-12 in Appendix A of the Conceptual Combined Dam and Fish Passage Design Report (HDR, 2017).

4.3.4.1.3 Bypass Consolidation Structure

The four bypass pipes turn toward the hillside on 15-foot radii immediately downstream of where they exit the dam. The four pipes turn back downstream and rise a few feet where they enter the Consolidation Structure.

The Consolidation Structure is 20-foot-wide by 40-foot-long by about 10-foot-tall open concrete structure (see Figure 4-25). It is benched into the hillside and founded partially on rock and partially on piles, similar to the Multi-Port Collector. Four bypass pipes penetrate the upstream wall; the wall closest to the dam. Each pipe discharges to its own transition channel where the water changes from pressurized to open channel flow. Each of the four channels converge smoothly down the length of the structure. A single outlet is provided at the downstream end of the structure where a single 18-inch diameter bypass conduit is attached. The four upstream and one downstream bypass pipes are buried and penetrate the structure while buried.

4.3.4.1.4 Bypass Conduit

The 18-inch diameter bypass conduit is a steel pipe thimble as it passes through the Consolidation Structure. Downstream of the Consolidation Structure the pipe is flanged to what will likely be an HDPE pipe. Over 2,900 feet of pipe is buried in the hillside downstream of the Consolidation Structure. The pipe switches back on the way down the hill in order to maintain the appropriate hydraulic pressure. Upon reaching the Chehalis River the pipe transitions to an outfall structure at about elevation 425.

4.3.4.1.5 Bypass Outfall

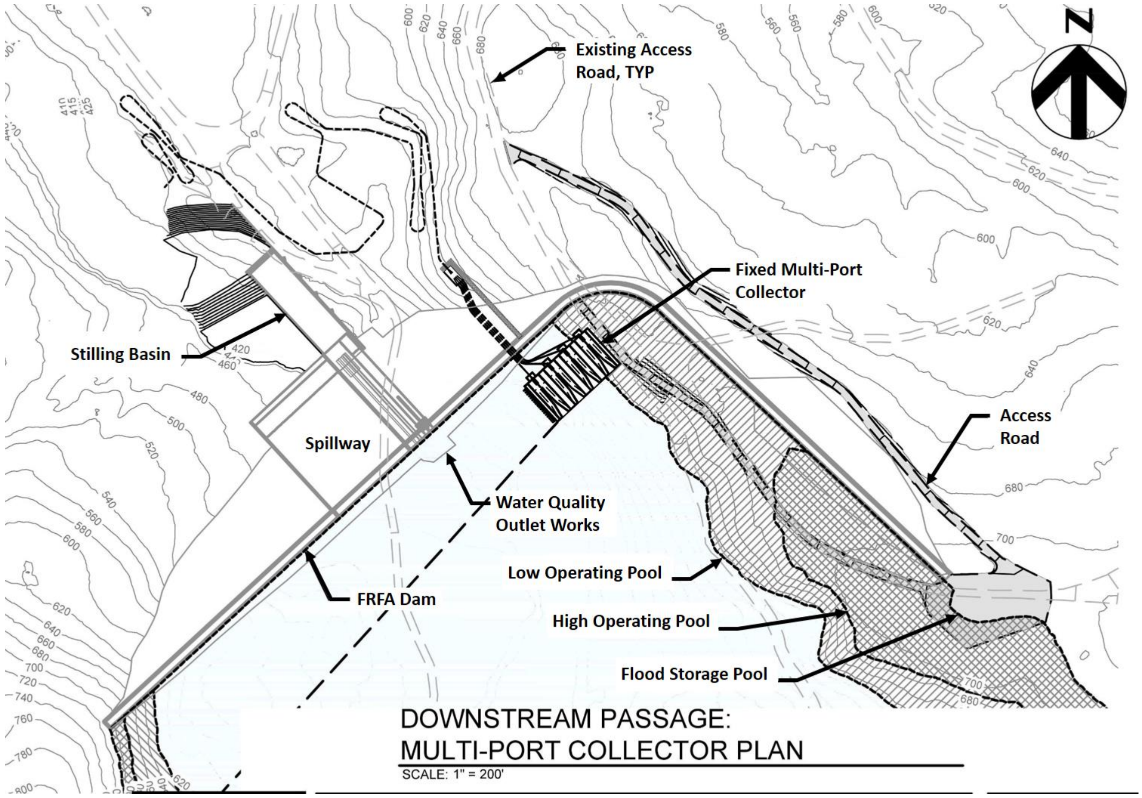
The bypass outfall structure is located on the banks of the Chehalis River downstream of the stilling basin (see Figure 4-25). It is a concrete structure about 30 feet in length. The bypass conduit penetrates the structure on the side opposite the river. Inside the structure the channel immediately transitions from round at the pipe penetration to rectangular over a length of about 20 feet. The rectangular channel is about 30-inches wide. The channel continues about another 10 feet where the flow leaves the end of the structure and plunges into a deep, swift portion of the Chehalis River.

4.3.4.2 Operation

The Multi-Port Collector is intended to provide safe passage for downstream migrating juvenile salmonids, adult steelhead, and resident fish year-round, through the full range of reservoir storage pool elevations. Although juvenile salmonids and adult steelhead will only pass downstream during certain periods of the year, resident fish currently traverse this reach of the Chehalis River regularly. Resident fish may move upstream and downstream multiple times throughout the year. The Multi-Port Collector is intended to provide resident fish as close to the same opportunities to migrate downstream as possible. Other alternatives address upstream migration of resident fish.

The Multi-Port Collector is intended to provide downstream passage during normal reservoir operation only. The reservoir fluctuates between WSEL 588 to 628 normally over the course of a year, as shown in Figure 4-26 and Figure 4-27, to enhance water quality downstream. Flood events may bring the reservoir pool higher than WSEL 628; potentially as high as WSEL 709 in a PMF event. During flood events, the Multi-Port Collector may be completely submerged. Concrete lids over each screen structure are intended to reduce the amount of debris that could clog or damage facility components. Goosenecks, small ports in the ceiling and/or walls, or other devices are included to prevent entrapment of air and floatation forces when the facility is submerged. It is expected that debris removal and inspection of the facility will be required following flood events.

Figure 4-27
Fixed Multi-Port Collector and FRFA Reservoir Inundation Map



A full-depth barrier net is placed upstream of, and outside the hydraulic zone of influence of, the water quality outlet works and flood regulation sluice intakes. The barrier net is attached to the left, upstream corner of the multi-port collector and extends to the left bank of the reservoir (see Figure 4-27). The design and operation of the net are similar to those described in the FSC alternative. The top of the net will rise and fall with the reservoir to ensure fish guidance is maintained for the targeted normal reservoir fluctuation. At extreme flood stages, the net will be submerged and will require debris removal and careful inspection by divers prior to startup after flood events. A thin wall above the lower collection structures excludes fish from passing downstream of the multi-port collector within the reservoir.

The fish screens are designed to NMFS juvenile criteria. Approach velocity criteria are met, with a factor of safety, when the facility is operated at its normal intended attraction flow of 500 cfs. The low-head submersible pumps have capability to increase the attraction flow to 1,000 cfs. Approach velocity increases proportionally at the higher attraction flow.

The attraction flow pumps are capable of drawing water from any of the four screen structures within the multi-port collector via the pump plenum. Only one screen structure operates at a time. Each screen structure operates over a 10-foot reservoir range. The highest screen structure operates from reservoir WSEL 628 to 618 while the bottom screen structure operates from reservoir WSEL 598 to 588. Each screen structure is capable of overlapping the next-higher screen structure's operation by 2 feet to ensure that downstream passage is continuous when transitioning from the operation of one screen structure to another. For example, the bottom screen structure may operate up to reservoir WSEL 600 while the next-higher screen structure is brought online for operation at the low end of its operating range (reservoir WSEL 598 to 608). Screen structures are never operated in a submerged condition. The water surface in the reservoir is always well below the access walkway above the fish screens.

When the multi-port collector begins operation, the attraction water pumps are off. The dewatering gates on the downstream side of the screen structure being started are fully opened. Then, the attraction water pumps are slowly started, bringing the attraction flow up to the desired amount. Transitioning from operation of one screen structure to another is similar, except the attraction water pumps continue running while one set of dewatering screens is slowly opened and the other set is slowly closed. Attraction flow is drawn in through the inlet of the screen structure being operated. As flow passes down the screen channel, water is drawn out through the fish screens and flow in the channel accelerates within NMFS criteria until approximately 25 cfs remaining in the channel passes over the end of the traveling ramp and into the bypass hopper.

Inside the hopper, the water surface is kept high to reduce the plunge depth off the end of the traveling ramp. The hopper is designed such that once flow falls into the hopper it is carried immediately and smoothly straight down into the vertical pipe at the bottom of the hopper and becomes pressurized flow. The pipe attached to the hopper slides within a larger pipe as the hopper elevation varies with the reservoir. A series of gaskets between the sliding pipe attached to the hopper and the larger stationary pipe seal the gap between the pipes and keep it mostly water tight. After the sliding pipe, the water continues down vertically, entering a 15-foot-radius vertical sweep to bring the pipe back up to the elevation needed to penetrate the dam. Pressurized flow in the pipe continues until the water is brought back to open channel flow at the consolidation structure on the downstream side of the dam. The bypass pipes from each of the four screen structures will operate at roughly the same flow rate, but each of the pipes is of a different diameter in order to achieve the same open-channel water surface elevation in the consolidation structure. The uniform water surface elevation in the consolidation structure allows the bypass flow for any normal operating reservoir elevation to be channeled to a single 18-inch-diameter bypass pipe for the remainder of the return to the river.

At the downstream end of the consolidation structure, the flow from any of the four screen structures is returned to pressurized pipe flow. Flow in this pipe may range from 15 to 25 fps. The bypass pipe leaves the consolidation structure and is immediately buried in the hillside. It continues for about 2,900 feet at a constant slope down to the river. The pipe length and slope are determined by the hydraulics to ensure the bypass pipe remains pressurized but does not experience a pressure more than 1 atmosphere above ambient air pressure.

The bypass outfall is located at the edge of the river, at the downstream end of the bypass pipe. The outfall structure transitions the bypass from a pipe to a rectangular shape and the water changes from pressurized flow to open-channel flow. Once achieving open-channel flow, the water flows a short distance before penetrating the construction diversion outlet headwall and plunging into a deep, swift portion of the Chehalis River. Plunge velocities will be no greater than 25 fps, the maximum allowed by NMFS. Above about a 2-year flood event, the bypass outfall channel will be submerged.

4.3.4.3 Maintenance

Regular maintenance of the Multi-Port Collector will be required. Maintenance activities can be broken into regular, annual, and infrequent periods.

Regular maintenance occurs throughout the normal operation of the reservoir. It includes:

- Debris removal inside screen structures following submergence, both following flood events and for the lower screen structures during normal operation. Material will be of size small enough to fit through the trashracks.
- Debris removal from trashrack of screen structure in operation.
- Inspection of mechanical and electrical systems following submergence, both flood events and for the lower screen structures during normal operation

Regular cycles of submergence and exposure throughout the normal operating period puts additional wear on all the equipment and metal work in the Multi-Port Collector. It is expected maintenance tasks that may ordinarily be undertaken every few years on an exposed structure will need to occur annually on the Multi-Port Collector. Annual maintenance tasks may include:

- Repaint steel structures.
- Repaint Bypass Pipes between Bypass Hoppers and the dam penetration depending on material selected.
- Service manual and motorized gate operators.
- Service screen cleaning system, including repaint steel parts and service motors.
- Inspect, service, and replace if necessary, all attraction water pumps.
- Inspect and service ramp gates and hoppers including repaint steel parts and service motors.
- Inspect fish screens and trashracks for damage.

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include:

- Replace attraction water pumps.
- Replace trash rakes.
- Replace gate operators.

- Repair or replace Bypass Outfall damaged by flood event.
- Replace Bypass Pipe(s) between Bypass Hoppers and the dam penetration as necessary due to damage from exposure, debris, or flood events.
- Repaint Bypass Pipes between Bypass Hoppers and the dam penetration as necessary and depending on pipe material selected.
- Replace attraction water pumps as necessary due to damage or normal wear.

A summary of the anticipated level of effort for operation and maintenance is provided in the table below.

Table 4-16
Operation and Maintenance Level of Effort for the Fixed Multi-Port Collector

ALTERNATIVE	AVERAGE ANNUAL DURATION	PERSONNEL REQUIREMENTS (FTE = FULL TIME EMPLOYEE)	SPECIAL CONSIDERATIONS
FRFA – Downstream Fixed Multi-Port Collector	12 months	Operations Staff – 0.9 FTE Biological Staff – 0.76 FTE Maintenance Staff – 0.31 FTE Total = 1.97 FTE	• Operation of facility daily during operation. • Daily inspections required to maintain optimum hydraulic settings and perform debris removal.

4.3.4.4 Anticipated Fish Passage Performance and Survival

The following is the anticipated fish passage performance and survival for the Multi-Port Collector alternative. A discussion of the rationale behind performance and survival value is provided in Attachment C of this document.

Table 4-17
Fixed Multi-Port Collector Anticipated Performance and Survival

TARGET SPECIES	PERFORMANCE	SURVIVAL	TOTAL SURVIVAL
ADULT DOWNSTREAM			
Spring Chinook	-	-	**-**
Fall Chinook	-	-	**-**
Coho	-	-	**-**
Winter Steelhead	50%	90%	**45%**
Coastal Cutthroat	65%	90%	**59%**
Pacific Lamprey	-	-	**-**
Western Brook Lamprey	-	-	**-**
JUVENILE DOWNSTREAM			
Spring Chinook	65%	99%	**64%**
Fall Chinook	65%	99%	**64%**
Coho	65%	99%	**64%**
Winter Steelhead	65%	99%	**64%**
Coastal Cutthroat	65%	99%	**64%**
Pacific Lamprey	3%	20%	**0.6%**
Western Brook Lamprey	3%	20%	**0.6%**

The anticipated fish passage performance and survival is based on the performance of other fixed fish collection facilities and adjusted based on assumptions related to the conditions that are unique to a proposed FRFA dam on the Chehalis River. There are a number of fixed collector bypass facilities that can be used to inform selection of performance and survival values but are not identical to the proposed alternative. Examples are:

- River Mill Fixed Collector and bypass
- Pelton-Round Butte Fixed Collector
- Soda Springs Bypass Facility
- Cowlitz Falls Fixed Collector and bypass

The Cle Elum Dam Multi-Port Fixed Collection Facility with helical bypass is currently under construction. Physical modeling results are available through the U.S. Bureau of Reclamation but only provide information on the design of the helical bypass and do not provide insight into the performance of the collection ports.

The following are assumptions made regarding the performance and survival through Multi-Port Collector at the proposed FRFA dam on the Chehalis River:

- The selected performance value is intended to accommodate fish passage from the head of the reservoir to the point of release downstream of the dam.

- It is assumed that with adequate configuration in the dam face and sufficient attraction flow, each individual collection port should perform as well as the fixed and floating surface collectors currently in operation with performance values in excess of 0.9, similar to the River Mill and Soda Springs facilities.
- The success of reservoir transit is speculative, and site-specific studies are likely to be required after implementation has occurred to determine performance. All values were therefore reduced from the expected facility performance to account for this uncertainty in the reservoir.
- Survival values were slightly increased from those given for floating surface collectors to account for use of a passive bypass system.
- Lamprey passage performance and survival values are expected to be low, given the uncertainty and lack of data available for this type of technology.

4.3.4.5 Reliability

The Multi-Port Collector has a moderate level of reliability. The reliability of the Multi-Port Collector is similar to that of the Floating Surface Collector in the use of multiple low-head pumps to attract fish in the reservoir to the collector and the requirement of fish to transit the reservoir to the collector located at the surface of the reservoir. The Multi-Port Collector also has its own unique mechanical and operational situations that affect its reliability. One such aspect is the volitional transport of fish downstream. Volitional transport via pressure pipes eliminates uncertainty associated with fish handling but increases the likelihood that debris passing through the collector occludes the pressure pipe, reducing performance via increased mortality or injury or by ceasing downstream transport completely until the blockage is removed by maintenance staff. The complete submergence of the dewatering screen structures also increases the potential for damage of mechanical equipment. Equipment that undergoes cycles of regular submergence and exposure shortens equipment life. The reliability of this alternative is reduced by the potential for critical equipment such as screens cleaners or hoppers to fail without warning during fish passage periods.

Converse to the added uncertainty in the mechanical and passage systems, the history of successful, consistent downstream passage via vee-screen structures with automated clearing systems suggests the system will perform well under a range of environmental conditions and mechanical situations. Even the volitional pressure pipe for downstream passage is now used in at least two locations – Soda Springs Dam on the North Umpqua River, Oregon and Green Peter Dam on the Middle Santiam River, Oregon. While these pressure pipes are both currently undergoing biologic testing, early results indicate a high level of performance. Considering the adverse and positive influences on reliability for this alternative, it is anticipated that the Multi-Port Collector will have a moderate level of reliability.

4.3.4.6 Cost Summary

The Multi-Port Collector is the most expensive of all the fish passage alternatives from a capital constructability perspective. Construction of the Collector in its required location make the foundation costs associated this alternative cost much higher than the other alternatives currently considered. Excavation on steep slopes, pile foundations, and slope stabilization add a substantively to the overall cost. In addition, it is physically impractical to employ a single, tall set of fish screens to collect fish over the large fluctuation in reservoir elevation during normal operation. Most other downstream V-screen type fish collection systems that have been constructed address fluctuations of 10 feet or less; with only one in the 20 to 30 foot fluctuation range (Pelton Round-Butte, Deschutes River, OR as one example). The large fluctuation in this reservoir requires multiple fixed V-screen dewatering bays in order to remain within this practical operational range. As much of the construction costs lies with the equipment, metal work, and concrete associated a screen structure, the necessity for multiple screen structures also multiplies the total construction cost.

A summary of the estimated construction costs, included the estimated upper and lower bounds, is provided in Table 4-18 below. A detailed breakdown of construction costs can be found in Attachment D.

The FRFA Multi-Port Collector has a lower operation and maintenance cost than the Floating Surface Collector. The Multi-Port Collector is a fixed, downstream fish passage facility that is mostly automated. This downstream collector is much simpler in complexity and operation than the FSC, therefore it requires only about half the year-round maintenance and fisheries technician hours to oversee its operation and keep it functional. An additional half-time fisheries technician is brought in for 9 months of the year, corresponding to fish migration periods. Additional temporary staff are brought in several times a year for short periods (less than 1 week) to conduct annual inspections and assist in larger maintenance efforts. These larger maintenance activities are expected to require less effort than those for the FSC as this facility has fewer and simpler mechanical systems and components. Attraction flow through the Multi-Port Collector is supplied by a system of low-head, high-volume pumps. The pump system runs continuously, year-round. Nearly 70% of the annual O&M cost is for the purchase of power. The Multi-Port Collector has several mechanical systems, including attraction pumps, screen cleaning systems, hopper and traveling ramp, and automation software and hardware. The annual maintenance is lower than that for the FSC as the systems are simpler but still more complex than those needed for the CHTR and Technical Fish Ladder. A summary of the estimated O&M cost is provided in Table 4-18 below. A detailed breakdown of O&M costs can be found in Attachment D.

Table 4-18
FRFA Fixed Multi-Port Collector Estimated Construction and O&M Costs

	CONSTRUCTION COSTS			O&M COSTS
FISH PASSAGE OPTION	LOWER BOUND COST ($ MILLION)	MIDDLE BOUND COST ($ MILLION)	UPPER BOUND COST ($ MILLION)	($)
FRFA – Downstream Fish Passage: Fixed Multi-Port Collector	$80.0	$106.6	$159.9	$764,000

5 COMPARISON OF ALTERNATIVES

5.1 Comparison Approach

The following section compares the success of each of the alternatives at achieving the evaluation factors. No assessment has been made regarding the relative importance of each evaluation factor to the others in the overall assessment of the fish passage alternatives and therefore the comparisons presented herein are unweighted. Quantitative values are associated with the Cost Estimate and Fish Passage Performance evaluation factors via estimated construction and O&M costs and total survival, respectively. While these values are quantitative, no attempt is made to use the values in a decision or comparison model. In addition, the evaluation of the reliability of each alternative produces entirely qualitative results. The comparison of the alternatives identifies differentiating features and allows for the general ranking of alternatives against one another. A visual comparison of the alternatives against the evaluation factors is provided in Figure 5-1 below. Results of the comparison of alternatives are intended to inform decision makers in their selection of alternatives to move forward into the next stage of development.

The fish passage alternatives for the FRO dam – the conduits and the CHTR facility – are not compared against each other as the fish passage alternatives are for the FRFA dam. Early in the process of developing fish passage alternatives, the Fish Passage Technical Committee decided that fish passage at the FRO dam should be provided via the conduits. As operation of the FRO dam was further developed the committee determined that a CHTR was required to provide upstream passage during ponding events. Together the conduits and CHTR provide upstream passage throughout the fish passage period (year-round) and downstream passage during non-ponding periods. As such, the fish passage alternatives for the FRO dam option are not compared herein. However, the alternatives' consistency with the evaluation factors are included in Figure 5-1 below to inform decision makers in their selection of a dam option to move forward into the next stage of development. The upstream and downstream fish passage alternatives for the FRFA dam are compared against each other in traditional fashion.

5.2 Summary and Comparison of Anticipated Fish Passage Performance and Survival

The following section compares the anticipated performance and survival of the proposed fish passage alternatives. The anticipated fish passage performance and survival are consolidated to the representative parameter of total survival. As stated in Section 3.2.1, the performance of the fish passage facility multiplied by the estimated survival through the facility equals the total survival, or the estimated percentage of fish that successfully navigate and survive the proposed fish passage facility.

Figure 5-1 summarizes the anticipated total survival for the upstream and downstream passage facilities for both the FRO and FRFA dam alternatives.

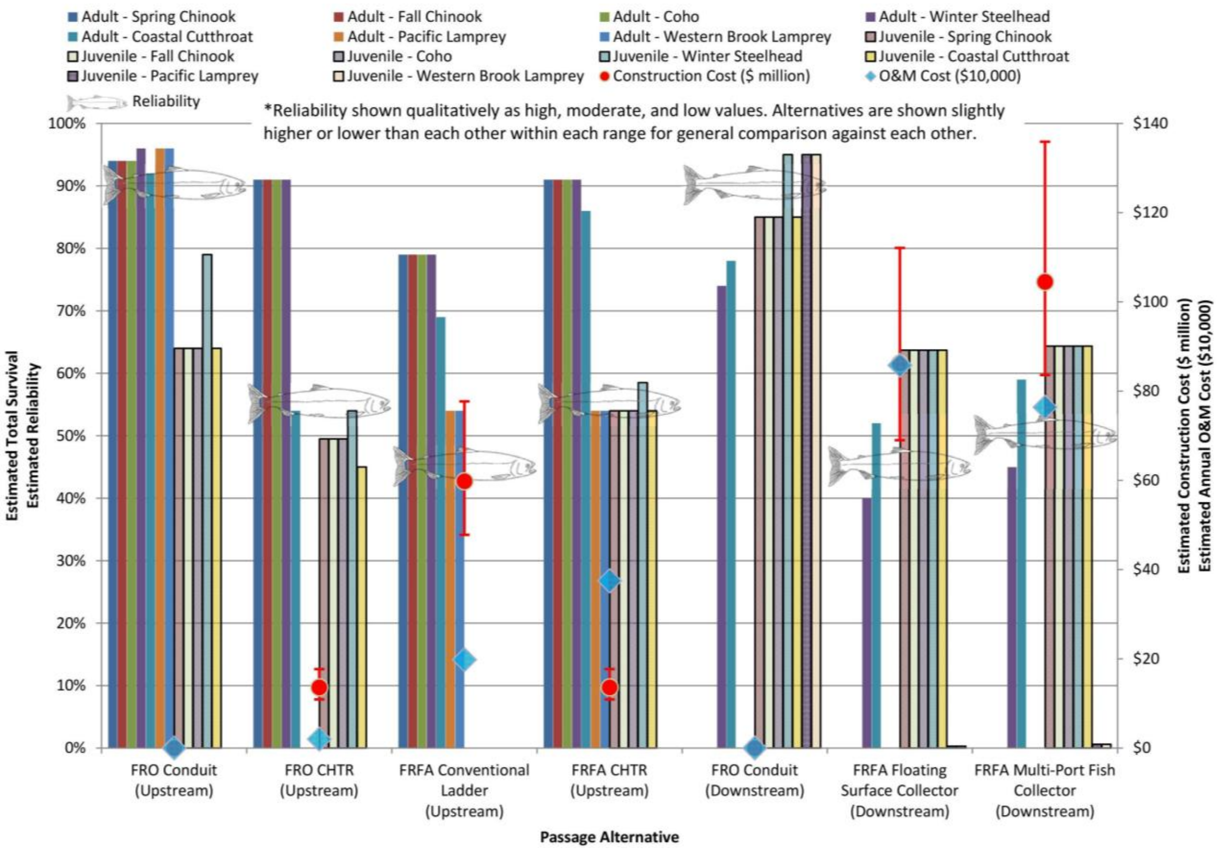
Only one avenue for upstream and downstream passage is provided for the FRO dam – via the conduits – with upstream passage augmented by a CHTR facility for upstream passage during dam ponding events. The FRO Conduits are anticipated to have better total survival for every species and life stage compared to the CHTR facility for upstream passage. However, there is CHTR will only be in operation a fraction of the year, corresponding to when fish passage is not possible through the conduits during ponding events. Downstream passage for the FRO dam is only provided via the conduits. During a ponding event, species are not anticipated to move downstream and are delayed until normal, run-of-river operation resumes.

There are two alternatives each for upstream and downstream fish passage for the FRFA dam. Upstream and downstream passage is independent of each other so it is only appropriate to make comparisons between alternatives separately for upstream and downstream alternatives. Upstream fish passage is a comparison of a volitional passage alternative (Technical Fish Ladder) versus a non-volitional alternative (CHTR). The CHTR is expected to have better total survival for all species and life stages than the Technical Fish Ladder. In fact, the Technical Fish Ladder is anticipated to exclude all upstream moving juveniles due to its length and height. Juveniles have been observed passing upstream through technical fish ladders, indicating a non-zero performance may be appropriate. However, the Technical Fish Ladder alternative is expected to require too much energy for juveniles to be able to transit the entire fish ladder to the reservoir. Therefore, while the survival for juveniles is expected to be moderate (50%), performance is expected to be zero (0%) as no juveniles are anticipated to be able to traverse the full length and pass into the reservoir.

Similar to the upstream passage alternatives, the downstream fish passage is a comparison of a volitional alternative (Multi-Port Collector) versus a non-volitional alternative (FSC). The Multi-Port Collector is expected to have nearly the same total survival as the Floating Surface Collector (FSC) for juveniles. However, the Multi-Port Collector is expected to have noticeably better total survival for adult Winter Steelhead and Cutthroat passing downstream. While the performance for each species and life stage is the same for the Multi-Port Collector and FSC, the Multi-Port Collector has better survival. Better survival for the Multi-Port Collector reflects the volitional nature of the passage system and the lack of fish handling.

In comparing alternatives between dam options, the FRO conduits have much better total survival than all of the upstream and downstream passage alternatives for the FRFA dam.

Figure 5-1
Comparison of Evaluation Factor Results for Each Fish Passage Alternative



5.3 Summary and Comparison of Reliability

Section 5.2 discusses the estimated total survival for fish passing each alternative under normal operating conditions, performing optimally. However, there are risks, inherent and often unique, to each alternative that the passage facility may not be able to perform at normal operating conditions at all times and that the facility may not perform optimally. While many of these risks can be identified, they often cannot be quantified, as they can for the other evaluation factors. As such, reliability is evaluated qualitatively. The anticipated reliability for each alternative is shown in Figure 5-1.

Fish passage for the FRO dam is provided via the conduits and a CHTR facility that is only used for upstream passage during ponding events. The FRO conduits have a high level of reliability as they are rarely used and are regularly maintained to preserve dam safety. The CHTR for the FRO dam has a moderate level of reliability. The CHTR's reliability is lower than the conduits primarily because it requires manual handling and transport of fish. However, there are numerous trap and transport facilities in operation and the technology has a long history of successfully passing fish upstream, so a low reliability rating is not warranted either.

As in the comparison of performance and survival, it is appropriate to compare the upstream and downstream fish passage alternatives for the FRFA dam separately. Both the Technical Fish Ladder and the CHTR have a moderate level of reliability, but the CHTR is anticipated to have somewhat better reliability than the Technical Fish Ladder. Technical fish ladders under 150 feet in height differential have a long history of reliable performance. However, at more than 200 feet of differential, the Technical Fish Ladder alternative has a higher level of uncertainty associated with biological performance, reducing its anticipated reliability.

Similar to the upstream fish passage alternatives, the downstream fish passage alternatives for the FRFA dam are both anticipated to have a moderate level of reliability. The Floating Surface Collector (FSC) has a slightly lower level of reliability than the Multi-Port Collector. The floating nature of the FSC, the difficulties in achieving optimal hydraulic conditions due to trim and draft control, and the non-volitional (manual) nature of the passage introduce additional uncertainty that is otherwise not present with the Multi-Port Collector alternative.

In comparing alternatives between dam options, the FRO conduits have the greatest anticipated reliability compared to the other upstream and downstream passage alternatives for the FRFA dam.

5.4 Summary and Comparison of Cost Estimate

Construction and annual operations and maintenance (O&M) costs are fundamental considerations in comparing fish passage alternatives. A summary of the estimated construction and O&M costs of each fish passage option are provided in Table 5-1 and compared graphically in Figure 5-1. Construction cost estimates have an expected accuracy range of -25% and +50%. The estimated construction cost for upstream and downstream fish passage at the FRO dam is estimated to range between $13.8 and $27.6 million. The estimated O&M cost for fish passage at the FRO dam is estimated to be $20,000 annually. Estimated construction costs for upstream and downstream fish passage at the FRFA dam range from $13.8 to $98.1 million and from $62.0 to $159.9 million, respectively. Estimated, annual O&M costs for upstream and downstream fish passage at the FRFA dam range from $198,000 to $375,000 and from $764,000 to $858,000, respectively.

Table 5-1
Fish Facilities Construction and O&M Costs

FISH PASSAGE OPTION	CONSTRUCTION COSTS		O&M COSTS	
	LOWER BOUND COST ($ MILLION)	MIDDLE BOUND COST ($ MILLION)	UPPER BOUND COST ($ MILLION)	($)
FRO – Fish Passage Conduits	Integral to dam construction.			
FRO – Upstream Fish Passage: CHTR Facility	$13.8	$18.4	$27.6	$20,000
FRFA – Upstream Fish Passage: CHTR Facility	$13.8	$18.4	$27.6	$375,000
FRFA – Upstream Fish Passage: Technical Fish Ladder	$49.1	$65.4	$98.1	$198,000
FRFA – Downstream Fish Passage: Fixed Multi-Port Collector	$80.0	$106.6	$159.9	$764,000
FRFA – Downstream Fish Passage: Floating Surface Collector	$62.0	$82.6	$123.9	$858,000

6 DISCUSSION OF COMBINATIONS OF ALTERNATIVES

The FRO and FRFA dams are required to provide provisions for upstream and downstream fish and lamprey passage. While each fish passage alternative has been developed individually, upstream and downstream alternatives must be paired with one another to provide a fully passable impoundment structure. Only one passage combination was developed for the FRO dam, by decision of the Fish Passage Technical Committee. The FRO passage combination provides upstream and downstream passage primarily through the FRO conduits. Upstream passage is provided via the CTHR facility during impoundment periods. No downstream passage facilities are provided during impoundment periods so downstream passage is delayed during these events. Two upstream and downstream fish passage alternatives are provided for the FRFA dam, resulting in four possible passage combinations:

- Upstream Fish Passage: CHTR Facility
  Downstream Fish Passage: Fixed Multi-Port Collector
- Upstream Fish Passage: CHTR Facility
  Downstream Fish Passage: Floating Surface Collector
- Upstream Fish Passage: Technical Fish Ladder
  Downstream Fish Passage: Fixed Multi-Port Collector
- Upstream Fish Passage: Technical Fish Ladder
  Downstream Fish Passage: Floating Surface Collector

6.1 Discussion of Fish Passage

The upstream and downstream total survival for each passage combination is commensurate with the total survival for each alternative in the passage combination. Each passage combination's general ability to pass each life stage and target species can be generally grouped into three categories – not provided, negligible, and fractional. Alternatives that do not include provisions for passage of a species and life stage can be considered as not providing passage for that species and life stage. Some alternatives provide such a low level of total survival for a species and life stage that passage can be considered negligible for that species and life stage. For the purposes of this report, total survival below 10% is considered negligible. Total survival greater than 10% is considered fractional. Fractional passage also indicates that, while passage is provided, fewer fish or lamprey will pass upstream or downstream of the passage structure than would pass in the natural environment. Where passage is not provided or is negligible, the species and life stage in question is considered as not passing the dam. Table 6-1 is based on Figure 5-1 and summarizes the species and life stage groups passing the dams for each passage combination.

Table 6-1
Species and Life-Stage Groups Passing FRO and FRFA Dams by Fish Passage Alternative Combination

FISH PASSAGE ALTERNATIVE COMBINATION	UPSTREAM PASSAGE			DOWNSTREAM PASSAGE		
	ADULT FISH	ADULT LAMPREY	JUVENILE FISH	JUVENILE FISH	JUVENILE LAMPREY	ADULT FISH
FRO Dam Fish Passage Conduits with CHTR Facility	X	X	X	X	X	X
FRFA Dam Upstream Fish Passage: CHTR Facility Downstream Fish Passage: Fixed Multi-Port Collector	X	X	X	X		X
FRFA Dam Upstream Fish Passage: CHTR Facility Downstream Fish Passage: Floating Surface Collector	X	X	X	X		X
FRFA Dam Upstream Fish Passage: Technical Fish Ladder Downstream Fish Passage: Fixed Multi-Port Collector	X	X		X		X
FRFA Dam Upstream Fish Passage: Technical Fish Ladder Downstream Fish Passage: Floating Surface Collector	X	X		X		X

Table 6-1 indicates that passage combinations with the Technical Fish Ladder alternative do not pass juvenile fish upstream. It also indicates that juvenile lamprey only pass downstream via the FRO conduits. The exclusion of juvenile passage upstream in the Technical Fish Ladder combination is not anticipated to exclude fish species entirely over time from the watershed upstream of the dam. Juvenile salmonids and trout have been shown to move upstream and downstream throughout their fresh water rearing cycle. However, if these fish do not have the opportunity to pass upstream the impact on the overall population is not anticipated to result in the loss of upstream habitat. Adults will continue to move upstream and spawn, repopulating the upstream reaches. Many of the juveniles will rear upstream until they are ready to migrate to the ocean, without coming close enough to the dam to fall downstream. Those that do fall downstream are before migrating are still able to rear in the downstream reaches until they are also ready to migrate to the ocean. Similarly, adult lamprey will move upstream and populate the reaches above the dam with juveniles. The juveniles may rear upstream of the dam without every passing downstream of the dam prior to migration. Those that do can continue to rear in reaches downstream of the dam.

6.2 Discussion of Cost Estimates

There are two components to the single avenue for upstream and downstream fish passage facilities at the FRO dam: the conduits and a CHTR facility. The conduits are integral to the FRO dam construction and therefore do not have an additional construction cost. The CHTR facility for the FRO dam is estimated to cost between $13.8 and $27.6 million.

There are two upstream and two downstream fish passage alternatives for the FRFA dam. The FRFA dam must have one upstream and one downstream fish passage facility, allowing for four combinations of facilities. The estimated construction costs and annual operation and maintenance costs for each of the combinations of alternatives are provided in Table 6-2. As shown in the table, annual O&M cost is inversely correlated to estimated construction cost.

Table 6-2
Cost Estimates for Combinations of Upstream and Downstream Fish Passage Alternatives

	CONSTRUCTION COSTS			O&M COSTS
FISH PASSAGE ALTERNATIVE COMBINATION	LOWER BOUND COST ($ MILLION)	MIDDLE BOUND COST ($ MILLION)	UPPER BOUND COST ($ MILLION)	($)
FRO Dam Fish Passage Conduits with CHTR Facility	$13.8	$18.4	$27.6	$20,000
FRFA Dam Upstream Fish Passage: CHTR Facility Downstream Fish Passage: Fixed Multi-Port Collector	$93.8	$125.0	$187.5	$1,139,000
FRFA Dam Upstream Fish Passage: CHTR Facility Downstream Fish Passage: Floating Surface Collector	$75.8	$101.0	$151.5	$1,233,000
FRFA Dam Upstream Fish Passage: Technical Fish Ladder Downstream Fish Passage: Fixed Multi-Port Collector	$129.1	$172.0	$258.0	$962,000
FRFA Dam Upstream Fish Passage: Technical Fish Ladder Downstream Fish Passage: Floating Surface Collector	$111.1	$148.0	$222.0	$1,056,000

6.3 Discussion of FRFA Fish Passage Alternative Combinations

The four fish passage alternative combinations can be classified as shown in the following table:

Table 6-3
FRFA Fish Passage Alternative Combination Types

FISH PASSAGE ALTERNATIVE COMBINATION	VOLITIONAL	NON-VOLITIONAL	MIXED
FRFA Dam Upstream Fish Passage: CHTR Facility Downstream Fish Passage: Fixed Multi-Port Collector			X
FRFA Dam Upstream Fish Passage: CHTR Facility Downstream Fish Passage: Floating Surface Collector		X	
FRFA Dam Upstream Fish Passage: Technical Fish Ladder Downstream Fish Passage: Fixed Multi-Port Collector	X		
FRFA Dam Upstream Fish Passage: Technical Fish Ladder Downstream Fish Passage: Floating Surface Collector			X

The fully volitional passage combination eliminates any handling of fish during normal operation, giving it greater reliability than the other combinations in this aspect of operation. However, the improvement in reliability in upstream passage is offset by the detriments to passage caused by the length of the ladder. The fully volitional combination also carries the greatest capital cost but the least annual operation and maintenance cost. As noted above, this combination excludes the upstream passage of juvenile fish.

The trap-and-transport passage combination is non-volitional. Fish are handled and moved upstream and downstream by human actions, reducing the reliability of this combination. The CHTR/FSC passage combination requires the least cost to construct but carries the greatest annual operation and maintenance cost. Combining the handling and transport facilities for upstream (CHTR) and downstream (FSC) passage may result in a savings in the capital cost. Potential savings in capital cost was not explored as part of this document. Similar savings in annual O&M costs are not expected as both facilities are expected to pass fish upstream and downstream year-round. The non-volitional alternative combination also provides the most flexibility to accommodate future changes to flood attenuation operation. This combination can accommodate the widest range of changes to flood attenuation operations, such as increases in the reservoir elevation, reservoir fluctuation, and flow.

The mixed passage combinations – the Technical Fish Ladder with the FSC and the CHTR with the Multi-Port Collector – do not provide unique advantages as the volitional and non-volitional combinations do. The Technical Fish Ladder/FSC and CHTR/Multi-Port combinations have the lowest and highest combined reliability of the four combinations, respectively. However the differences in reliability are only marginal as all four upstream and downstream alternatives are classified as moderate in reliability. Similarly, both combinations have capital and O&M costs that are neither the greatest nor the lowest among the four combinations. One clear differentiator between the mixed combinations is that the Technical Fish Ladder/FSC does not have juvenile upstream fish passage, whereas the CHTR/Multi-Port does.

6.4 Discussion of FRO versus FRFA Dam Fish Passage

FRO fish passage combination performs better for all the evaluation factors – performance, reliability, and cost – than any of the FRFA fish passage alternative combinations. The FRO conduits provide a high level of reliability. None of the other fish passage alternatives are expected to have this level of reliability. Similarly, the FRO conduits have much higher total survival rates than any of the other upstream and downstream alternatives. Both the reliability and total survival for the FRO conduits are reflective of combined FRO passage alternatives as the CHTR is expected to be used infrequently and for relatively short durations. The exclusion of upstream passage for adult lamprey when the CHTR is in operation is not anticipated to result in the exclusion of the species upstream of the dam. The FRO conduits are integral to the safe operation of the dam so the capital and O&M costs are included in the dam costs and not considered as a cost for fish passage. The capital and O&M costs of the FRO CHTR facility are the lowest of all the alternatives considered, save only the conduits.

While the total survival is used as an evaluation factor in the comparison of the fish passage alternatives, it is not reflective of post-project population abundance when comparing fish passage alternative combinations between a dam options. Total survival may not be reflective of post-project population abundance when comparing alternatives between dam options because reservoir storage, inundation area, and dam operation has a much greater impact to post-project population abundance than the effectiveness of the fish passage facilities. A comparison of post-project population abundance may be found in the Ecosystem Diagnosis and Treatment (EDT) modeling results. The EDT modeling results are summarized in the Programmatic Environmental Impact Statement (PEIS).

7 NEXT STEPS

The following paragraphs provide a summary of the next steps that are necessary to move forward with further development of fish passage provisions at a future Chehalis Dam.

7.1 Selection of Preferred Fish Passage Alternatives by the Fish Passage Technical Subcommittee

Data and analysis from multiple reports and studies associated with the Chehalis Basin Strategy, including the Draft Programmatic Environmental Impact Statement (PEIS), the Draft Combined Dam and Fish Passage Conceptual Design Report, and this report, have been provided to the Fish Passage Technical Subcommittee and their parent organizations. Upon resolution of any remaining questions, information needs, concerns, and/or comments, the Fish Passage Technical Subcommittee can select preferred fish passage alternatives for the FRO and FRFA dam options. Recommendations of preferred alternatives by the Fish Passage Technical Subcommittee will then be passed to Washington State. Selection of a single upstream and a single downstream fish passage alternative and a single dam option by the State of Washington is expected to occur in the next biennium.

The Fish Passage Technical Subcommittee recognized in previous subcommittee discussions that some efficiencies may be gained by moving forward with a fish passage alternative prior to selection by Washington State if an alternative is highly likely to be selected. The subcommittee identified one such alternative – the CHTR. A CHTR facility is common to both the FRO and FRFA dam options, has a much higher level of total survival, is far less costly than other upstream fish passage options, and is moderately reliable. For these reasons, the Fish Passage Technical Subcommittee has chosen to proceed with development of the portions of the CHTR facility that would be common to both the FRO and FRFA dam options.

7.2 Selection of a Preferred Dam Option by Washington State

Similar to the selection of preferred fish passage alternatives, data and analysis from multiple reports and studies associated with the Chehalis Basin Strategy have been provided stakeholders. This data and analyses, as well as input from stakeholders, is being provided to Washington State to inform their selection of a single dam option. Selection of a final dam option is necessary in order to determine which upstream and downstream fish passage alternatives will be considered. As noted above, the Fish Passage Technical Subcommittee expects the CHTR will be selected for upstream passage regardless of dam option. However, selection of a dam option is necessary in order to efficiently move forward with a downstream FRFA alternative or with the FRO conduits. Until such time as a dam option is selected, further development of a downstream FRFA alternative or FRO conduits cannot move forward. Selection of a single dam option by the State of Washington is expected to occur in the next biennium.

7.3 Development of Selected Fish Passage Alternatives

Development of selected fish passage alternatives is expected to proceed along two separate timelines in the near future. As discussed in Section 7.1, the Fish Passage Technical Subcommittee has identified the CHTR as an upstream fish passage technology that will likely be selected regardless of which dam option moves forward. While Washington State is considering selection of a single dam option and single upstream and downstream fish passage alternatives, the Fish Passage Technical Subcommittee will continue with their monthly meetings to facilitate ongoing further development of a CHTR facility to a preliminary design level. If selection of a CHTR facility is confirmed by Washington State, development of the design to a preliminary design level is anticipated to be completed in the next biennium.

While a CHTR facility is being further developed, development of the other upstream and downstream fish passage alternatives will remain idle until Washington State makes their selections. However, the Fish Passage Technical Subcommittee will continue to discuss and set design criteria that are common to all upstream and downstream alternatives, such as anticipated population abundance for adult cutthroat trout. Upon selection of upstream and downstream fish passage alternatives by Washington State, the Fish Passage Technical Subcommittee will likely incorporate development of these alternatives into their meetings and the selected alternatives are anticipated to be developed to a preliminary design level in the next biennium.

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Attachment A
Fish Passage Technical Subcommittee Meeting Notes

HDR

Meeting Minutes

Project:	**Chehalis Basin Strategy**
Subject:	Fish Passage Workshop
Date:	Wednesday, January 20, 2016
Location:	HDR Olympic Office
Attendees:	Matt Prociv – HDR Shaun Bevin – HDR Mike Garello – HDR Larry Swenson – Anchor John Ferguson – Anchor Keith Kirkendall – NOAA Justin Allegro – WDFW Don Ponder – WDFW Jessica Hausman – WA DOE Mark Mobbs – Quinault Tribe Jeff Brown – NOAA Carol Cloen – WDFW Jeff Fisher – NOAA

- Mike Garello (HDR) and John Ferguson (Anchor) start meeting introducing what previous analysis was done.
  - They also state that the analysis will be re-done as the alternatives are refined and more information is known.
  - State they need to come to consensus on design criteria by February 15, 2016.
    - State if consensus on design criteria is delayed past this date it will impact subsequent documents and deadlines.
  - Ferguson (Anchor) states there are 3 goals the consultant team would like to accomplish today:
    - Explain what are the consultants scoped to do,
    - Come to consensus on as much design criteria as possible,
    - Identify steps to get the remaining missing information in time to meet the February 15th deadline.
- Justin Allegro (WDFW) asked that the Programmatic Environmental Impact Statement (PEIS) be tied into the fish passage design criteria.
  - John Ferguson (Anchor) says we have not been very involved with the EIS and asked that WDFW provide input on what they would like to be included because WDFW has been more involved
  - WDFW says they will follow up with the Anchor/HDR team next week (last week of January) to help provide input on how to incorporate EIS. Jessica (DOE) says she will follow up with Justin (WDFW) next week as well.
- Mark (Quinault tribe) says he would like to look at what would happen if flood occurred during period of smolt migration
- Mike (HDR) discusses work conducted over the previous Biennium and the deliverables and presentations conducted during the Biennium.
  - Flood Retention Only (FRO) dam option:
    - Mostly run of river.
    - Retains about 5-10 year flood events and above.
    - The consultant team is currently reworking the operations plan for the FRO.

        - Fish passage tunnels would meet fish passage design criteria up to 2,000 cfs, which is the calculated high fish passage design flow. As the flow in the fish passage tunnels rise above 2,000 cfs, fish would still be pass the tunnels but passage would diminish. When the river flow reaches a pre-determined flood level (set in the operations plan) the fish passage tunnels would be shut off and a base flow would be provided to the river below the dam through the regulating outlet (RO).
        - The fish passage tunnels provide adult passage at 2 fps.
        - 2,000 cfs high passage design flow is based on Coho migration period.
        - WDFW asked how will CHTR be operated if only operated every 5 years or will it be operated above 2000 cfs each year. Mike (HDR) responds that when the fish passage tunnels are shut down (about every 5 years), the shut down would last about 25 days. A Capture, Handling, Transport, and Return (CHTR) facility would be operated during this period for continued passage.
    - Mike (HDR) describes how multiple options for passage were discussed in previous reports, presentations, and webinars, as well as in the Water Retention Technical Committee meetings and Policy Workshops.
        - Previous documents are public and posted online. Contact Ferguson (Anchor) or Garello (HDR) for the specific website address.
- Carol (WDFW) suggests including specific lamprey criteria or state specifically in the report that lamprey passage was not included in the design.
- Mark (Quinault) states that lost habitat and reproduction for amphibians for the FRO and FRFA needs to be addressed in the documents. Several people note that this is being addressed in other documents and should not be included in the fish passage documents.
- Mike Garello (HDR) – fish passage team will do 2D and 3D computer models to assess performance of fish passage tunnels on FRO this year.
- Shaun Bevin (HDR) provides approximate (annual) flow information requested by WDFW and NOAA representatives:
    - 2yr flood ≈ 6,500 cfs
    - 10yr flood ≈ 13,000 cfs
    - 1% exceedence ≈ 2,900 cfs
    - 5% exceedence ≈ 1,400 cfs
    - Low flow (summer) ≈ 14 cfs
- Mike (HDR) discusses the FRFA dam option:
    - FRFA = Flood Retention, Flow Augmentation
    - Upstream (US) passage options are a fish ladder with an experimental exit (due to ≈ 100 foot fluctuation in the reservoir) or a CHTR.
        - For comparison, the North Fork fish ladder is 240 feet tall and 2 miles long. From what he has heard it passes fish efficiently. Bull trout have about an 8 to 12 hour transit time.
    - Downstream (DS) passage options are a floating surface collector (FSC) or a head-of-reservoir collector and/or an in-stream collector(s)
- Mike (HDR) shows the FRO and FRFA dam options with fish passage options on the screen.

- Mike (HDR) explains that when evacuating the reservoir in the FRO dam option the RO will be used and the fish tunnels will remain closed until the reservoir has reached the level where the tunnels will be opened to provide sediment management.
- Mike (HDR) explains that the FRO dam option has remained in consideration because everyone wanted to continue to look at a run-of-river situation even though it will have debris, sediment, and passage challenges similar to Mud Mountain Dam.
- Jeff Fischer (NOAA) stated the FRO tunnels (as shown on the drawing presented on the screen) are located at the thalweg of the river so we would likely lose fish moving at the edges (margins) of the river. He suggests locating tunnels at the edge of river as well as the thalweg so we don't lose life histories. Discussion ensues and concluded in the need for the design team to consider horizantal and vertical spacing of tunnel inlets.
- Discussion of juveniles passing downstream during a flood with the FRO:
  - Mike (HDR) states the RO is unscreened so it will discharge juveniles downstream. However, juveniles stay in the top 50-60 feet of the reservoir so they likely won't pass downstream until the reservoir gets down to 50-60 feet deep.
    - Larry (Anchor) states that this might cause issues for fish due to the pressure differential – going from 50-60 feet of pressure to atmospheric almost instantly. He also notes that this is similar to what was done on the Columbia for years where spillgates would be cracked open and juveniles would pass under the gates with about 30-40 feet of head exiting instantly to atmospheric.
  - Mike (HDR) also states that each time the reservoir drops it cuts a new channel through the reservoir bottom resulting in a high sediment load through the fish tunnels and downstream for about 2 weeks. This sediment load may also delay fish passage through the tunnels. How do we address this?
- Justin Allegro (WDFW) – draft PEIS will be out in October and feedback provided in November.
  - Governor will select an alternative in December 2016.
  - Final will be out May/June 2017
  - WDFW will not have a lot of the fish data/analysis done for the draft but it will be in the final. They are in the process of trying to figure out how to put a place holder in the draft for future WDFW information.
  - WDFW reiterated that they would like to get as much of the Fish Passage Concept Report into the draft PEIS as possible.
- Ferguson (Anchor) states that fish abundance numbers set the sizes of the facility components such as fish ladder pools, hoppers, tanks, etc. These numbers are needed for the design criteria (Feb 15) in order to adequately compare the passage alternatives.
- Jeff Fisher (NOAA) - wants to aim high for population abundance, do not want to go the Buckley route and have the facilities not able to handle # of fish.
- Juvenile upstream passage:
  - Jeff Fisher (NOAA) states juvenile upstream passage is required. Juvenile passage has been documented in this reach. WDFW agrees – says, in general, "new facilities will need to pass all species all the time."
  - **SUMMARY: The committee agrees that upstream juvenile upstream passage for both FRO and FRFA is required.**

- Jeff (NOAA) – says water gets hot here, and we know fish move around to find cold water. States "how can we cut this off"
  - Committee agrees there will be significant impacts and compromises for passing juveniles as well as adults upstream (i.e. – 600 pool ladder instead of a 300 pool ladder because smaller steps are needed for juveniles)
  - NOAA & WDFW both state that fish ladders and CHTRs do not collect and pass upstream juvenile migrants very well
  - NOAA states that upstream passage of juveniles is not traditionally for high head dams
  - The Umatilla Tribe is currently recording fish passing through their fish ladder looking for juveniles passing upstream
  - Committee agrees that the effectiveness of upstream juvenile passage on the FRFA option will likely be very low
- Mike Garello (HDR) – asked for Juvenile migration timing from WDFW
  - Justin Allegro (WDFW) – asked for migration timing figure from previous reports and shown on the screen to be sent around to group
  - **Migration timing to be finalized by February 5th.**
  - Justin Allegro (WDFW) said he will provide migration timing for upstream juveniles but until then it can be assumed that it is year-round (salmon only)
- Mark (Quinault Tribe) – says he doesn't think Bull Trout occur in Chehalis this high up.
  - Jeff Fisher (NOAA) – doesn't think Bull Trout are at the project location either but will check with USFWS as they provided this migration timing originally
- Ferguson (Anchor) stated that changes to the Chehalis River flow by the FRFA would be mostly attenuated just downstream of the twin cities.
  - Larry (Anchor) asked how the FRO and FRFA would affect existing migration patterns
    - Ferguson (Anchor) said migration patterns for the FRO would likely not be affected much because the flow regime of the river is only being changed during flood events
- Fish Passage Abundance Discussion
  - WDFW asked if migrating fish "clump" (show up on the same day)
    - Ferguson (Anchor) said he did not know; there is no data.
  - John Ferguson (Anchor) – Coho and Fall Chinook are the vast majority of the salmon in the system at the project site. From EDT maximum coho was 8,400 fish for year, average was about half that. Stealhead 808 maximum, fall Chinook 9000, spring Chinook 600.
  - Jeff Fisher (NOAA) – thinks there was study that stated we are at roughly 10-25% of historic run size.
  - Discussion that we could apply factor of about 5 to John's numbers to come up with fish abundance size but NOAA, WDFW, and Quinault Tribe will go back and confirm.
    - Jeff Fisher (NOAA) and Justin (WDFW) will discuss with Mark (Quinault Tribe) to come up with abundance criteria.
    - Peak annual and peak day numbers will be provided for all target species, juveniles and adults.

- Jeff Fisher (NOAA) says February 15th may not be reasonable to get all this data, but will have meeting with WDFW in the next week or so and get back to design team to determine if gathering all this information is feasible in this time frame.
  - Mike Garello (HDR) – points out we need to consider downstream migration abundance
    - WDFW says smolt trap should inform this decision
- Mike Garello (HDR) – asked for CHTR facility would we release directly to reservoir or to specific tributaries
  - Jeff Brown (NOAA) asks are the fish in the creeks that directly feed the reservoir genetically separate?
  - WDFW – Guessing that we would want to transport right to the head of reservoir. But we don't know right now.
  - Consensus is that we decide later but agree it will be needed to determine the operation and maintenance costs for the Concept Report.
- Fish Passage Performance Criteria/Targets for Design
  - Juvenile Downstream Passage
    - Keith (NOAA) - 75% is typical minimum for total from entering reservoir to return downstream of the dam. Here it is not possible to assess the loss through the reservoir because the dam hasn't been constructed yet.
    - Keith (NOAA) 75% is a pure MINIMUM. Typically aim higher.
    - Keith (NOAA) suggests 95% collection efficiency and 98% handling efficiency.
    - Consensus is to set target of 95% efficiency through reservoir to downstream of the dam (reservoir, collection and handling combined) but that it may not be possible to get there.
  - Adult Upstream Passage
    - Consensus is for 95% efficiency.
  - Mark (Quinault Tribe) - Need to look at Lamprey as we move forward. We need abundance numbers and/or efficiency and handling requirements.
    - John Ferguson (Anchor) – will do some outreach to see if USFW has handling guidelines
- Jeff Fisher (NOAA) asks what is being done to reduce the chance of juvenile predation at the downstream outlets.
- NOAA suggests consultants look at a helix for downstream passage similar to Cle Elum Dam.
- Next meeting will be February 12th, same time/same place – 10am to 1pm in HDR's Olympia office.
  - John (Anchor) will send a meeting invitation and agenda.

Project:	Computed:	Date: 1/20/16
Subject:	Checked:	Date:
Task:	Page:	of:
Job #:	No:	

CHEHALIS FISH PASSAGE WORKSHOP

SIGN-IN SHEET

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Jeff Fisher	NOAA	jeff.fisher@noaa.gov

Meeting Minutes

Project:	**Chehalis Basin Strategy**
Subject:	Fish Passage Subgroup Workshop
Date:	Friday, February 12, 2016
Location:	HDR Olympia Office
Attendees:	Shaun Bevan – HDR Mike Garello – HDR Larry Swenson – Anchor (via phone) John Ferguson – Anchor Chip McConnaha – ICF (via phone) Justin Allegro – WDFW (via phone) Don Ponder – WDFW Jim Pacheco – WA DOE Jessica Hausman – WA DOE Mark Mobbs – Quinault Tribe Jeff Brown – NOAA Jeff Fisher – NOAA (via phone)

1) Mike Garello (HDR) and John Ferguson (Anchor) started the meeting by checking in on three action items from previous meetings: periodicity timing, fish abundance and EDT modeling:
   a) Periodicity Timing
      i) Justin Allegro (WDFW) – Mara has provided feedback on periodicity chart and sent an email with comments out to the group during the meeting. The emailed comments included:
         (1) Spring Chinook adult arrival - Extend arrival to mid-October. This is based on timing of movements identified in the radio telemetry study conducted by USGS/WDFW/Chehalis Tribe.
         (2) Spring Chinook and Fall Chinook outmigration - Extend from January to December. There are at least three freshwater life histories – fry migrant, subyearling smolt migrant, yearling smolt migration. In multiple watersheds in western Washington, Chinook are observed to outmigrate at all months of the year with the majority of the outmigration occurring between January and July.
         (3) Coho salmon arrival - Arrival should continue through January. This is based on arrival timing of coho salmon in the Chehalis basin in general (early and late fall pulses) and the spawn timing observed by WDFW in the Upper Chehalis subbasin.
         (4) Coho salmon outmigration - Outmigration should be extended through December. This is based on general understanding of downstream movements of juvenile coho in the fall months associated with freshets. This is also based on radio telemetry study conducted in 2013 in the Upper Chehalis by USGS.
         (5) Winter Steelhead spawning - Spawning should start at the beginning of February. This is based on WDFW spawner surveys conducted in the Upper Chehalis 2014, 2015 spawn years.
         (6) Pacific lamprey arrival and spawning - Arrival = March – October, Spawning = March – June. Adult Pacific lamprey arrive in freshwater between the months of March and October and then spawn the following spring (March – June). I checked with Wydoski & Whitney and talked with Marie to confirm.
      ii) Chip McConnaha (ICF) asked if the life history provided by WDFW should be included in the EDT model.

(1) Attendees agreed that it is appropriate to use the updated life periodicity table for EDT model updates. Attendees also agreed that it periodicity may need to be adjusted in the future to better reflect entire Chehalis basin, instead of dam site specific periodicity.

(2) ACTION ITEM: HDR to update periodicity table using WDFW comments before Tuesday (Feb 16)

(3) Mark Mobbs (Quinault) had a question on juvenile migration timing

(a) Previously agreement that juvenile upstream migration will be shown on periodicity chart to occur year round. The chart had not been updated at this time, but will be the week after the meeting.

(4) Justin Allegro (WDFW) asked about incorporating text on why migration timings were modified. Mike Garello indicated that the consultants will incorporate this discussion into the conceptual Design Report due in June.

iii) Discussion on Bull Trout

(1) John Ferguson (Anchor) & Mark Mobbs (Quinault) – bull trout to be considered into the periodicity table and consideration given in the conceptual design report.

(a) Mike Garello (HDR) agrees and states we will keep bull trout in periodicity chart, with the footnote that we don't expect them to be located here.

(2) Justin Allegro (WDFW) & Jeff Fisher (NOAA) – USFWS has not provided input on bull trout yet. Justin asked for suggestion on how to get USFWS input.

(a) John Ferguson (Anchor) suggests Justin forward updated periodicity chart to USFW and ask them if they have any final input.

b) Fish Abundance

i) Justin Allegro (WDFW) had not had an opportunity to work on a proposal for others to review.

(1) ACTION ITEM - John Ferguson (Anchor) asked if draft proposal can be completed by February 19th, Justin Allegro (WDFW) indicated he would meet that deadline.

c) EDT modeling

i) John Ferguson (Anchor) asked question of how we model the design alternatives in the EDT model. John started the discussion that the group would need to agree on passage effectiveness/survival for each alternatives upstream and downstream passage facility.

(1) Jeff Brown (NOAA) suggests modeling multiple degrees of passage effectiveness, similar to a sensitivity analysis.

(a) John Ferguson (Anchor) states it is okay to run a few preliminary effectiveness values, before we pick a final effectiveness. Asked Chip McConnaha to come up with list of inputs that he needs for model.

(i) Chip McConnaha (ICF) – primary inputs are periodicity, and survival rate. We can give survival rate a flat rate or seasonal rate. Survival rates need to take into account **capture efficiency** and **survival** of those once captured.

1. FRO – previously assumed very small or no impact to survival rate.
2. FRFA – previously assumed seasonally shaped adult passage, no juvenile passage.

(2) Jim Pacheco (DOE) suggests that the results should be tested and incorporate known data from existing facilities.

HDR

(3) Jeff Brown (NOAA) states a lot of the survival and effectiveness numbers are there for other facilities for adults upstream and juvenile downstream. However, juvenile upstream passage effectiveness data may not exist and would likely be qualitative/informed guess.
(4) John Ferguson (Anchor) developed and discussed approach using spreadsheet analysis and cooperative discussion among the subcommittee. The table will have efficiency and survival, product of the two will be the number provided to Chip McConnaha (ICF) for EDT modeling.
  (a) Will need to discuss during the process if we want to add seasonal variation.
  (b) Justin Allegro (WDFW) would like the seasonality to be revisited to make sure it is accommodated for appropriate life stages.
  (c) The values within the table should be reliant upon information available from existing facilities when possible.
  (d) Example table provided below:

Fish Species	**FRO**			**FRFA**		
	Tunnels	**CHTR**	**CHTR**	**Ladder**	**FSC**	**Head Res**
Coho						
Adult U/S						
Adult D/S						
Juv U/S						
Juv D/S						

  (a) ACTION ITEM: Attendees agreed to meet in March to cooperate on populating passage efficiency/survival table. HDR and Anchor will prepare draft tables for review by others prior to meeting.

2) Design Criteria Discussion
  a) Mike Garello (HDR) brought up the discussion on two design criteria topics: juvenile upstream design criteria and juvenile downstream design criteria.
    i) Juvenile upstream passage design criteria
      (1) Mike Garello (HDR) asked if we assume 0.7 ft ladder hydraulic differential, would it be challenged
        (a) Jeff Brown (NOAA) – says the state may request 0.5 ft. NMFS criteria is 0.7 ft drop for fish < 80 mm.
          (i) Suggests we use 0.5 ft for jumps to CHTR.
            1. Group agrees that most trap and haul facilities are not designed/monitor for juveniles.
          (ii) Suggests we may want to consider 1 ft for full height ladder for adult energy expenditure purposes.
            1. The reason for this is that we would accept the fact that any full height ladder will have low juvenile passage performance, and therefore it should be optimized for adults.
        (b) ACTION ITEM: Mike Garello (HDR) to touch base with PGE to see if their fish ladder passes juvenile fish and what sort of effectiveness.
        (c) ACTION ITEM: John Ferguson (Anchor) to look up pit tag lengths of juvenile in the Chehalis.
      (2) Mike Garello (HDR) asked about tunnel velocity criteria for juveniles, Jeff Fisher (NOAA) previously proposed 0.3 ft/s. Mike asked for a literature search to determine criteria.

(a) **ACTION ITEM: Mike Garello (HDR) to email Jeff Fisher (NOAA) and Jeff Brown (NOAA) to discuss velocity criteria for juvenile upstream migration.**
(b) Shaun Bevan (HDR) asked a question of how much cross-section area needs to meet juvenile and adult velocity criteria.
  (i) Jeff Brown (NOAA) does not need velocity criteria met for entire tunnel cross section.
  (ii) Mike Garello (HDR) suggests using Thompson criteria of 20% of area connected contiguously along tunnel. Attendees were okay with this proposed criterion.
(c) Jeff Brown (NOAA) stated that upstream transport channel minimum velocity is 1.5 ft/s, which competes with our juvenile upstream passage criteria of 0.3 ft/s.
(d) Jeff Brown (NOAA) is curious to whether the tunnels will have sediment in the bottom for the FRO option. Jeff states preference is to have roughness.
  (i) Shaun Bevan (HDR) says the current thought is that they will have sediment up to the fish passage flow, at which point the tunnel sediment would scour out and pass sediment.
(e) Jeff Brown (NOAA) proposed idea of adding fish passage elements to a few tunnels on margins. Use the center tunnels for passing flow, sediment, and debris.

ii) Juvenile downstream passage design criteria
  (1) Head of Reservoir
    (a) Max design flow approximately 1,600 to 2,000 cfs. Inefficient at flows higher than that as juveniles will bypass the screens into the reservoir. There are very few if any similar facilities of this nature in existence. Soda Springs and Pelton Round Butte collector are similar buy exist near dams in the reservoir.
    (b) Mark Mobbs (Quinault) asked what happens with fish that pass to the reservoir. How will fish exit the reservoir? Will coho benefit from reservoir rearing?
    (c) Jeff Brown (NOAA) asks how fish passed into reservoir at high flow will affect performance and efficiency numbers. We will need to evaluate percent of fish that may pass during high flows and compare to expected 75% reservoir passage of FSC.
  (2) Floating Surface Collector
    (a) Attendees verified that 75% efficiency from head of reservoir to release point downstream and 95% at collector zone of influence to release point downstream.
    (b) Attendees discussed attraction flow. Upper Baker, Lower Baker, Swift, and North Fork collectors are all set up for 500/1000 cfs. Typically operate at 500 cfs.
      (i) Attendees agreed that FSC be sized for 500 cfs to be comparable to existing facilities
    (c) Barrier/guide nets would be required and would likely be replaced/repaired every 3 or so years.
  (3) Mike Garello (HDR) – Asked for literature of passing fish through pressure differential, with gates, and what are the
    (a) Jeff Brown (NOAA) has potential study for fish survival through gates that are partially closed.
    (b) John Ferguson (Anchor) said USACE likely has data for Columbia River submerged gate fish survival to atmospheric pressure.

3) Workplan and Schedule Update
  a) Mike Garello (HDR) – Gave overview of workplan and timeline

i) Updated design criteria agreed to by Feb 15. No later than March 1.
ii) EDT modeling parameters by end of March
iii) Final Design Criteria Report in May
iv) Conceptual Design Report in June
v) Final Conceptual Design Report in September

4) Next Meetings
   a) March 4, 10am – Conference call to present preliminary fish passage efficiency prepared by Anchor and HDR
   b) March 9, 1pm to 4pm – Meet in HDR Olympia office to collaborate on fish passage efficiency of each alternative

Chehalis Fish Passage Workshop 2/12/2016

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✓ Jessica Hausman	ECY	jessica.hausman@ecy.wa.gov
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✓ John Ferguson	AQEA	✓ jferguson@anchorqea.com
✓ LARRY SWENSON	AQEA	
✓ CHIP MCONNAH	ICF	
✓ JEFF FISHER	NOAA	
✓ JUSTIN ALLEGRO	WDFW	

Meeting Minutes

Project:	**Chehalis Basin Strategy**
Subject:	Fish Passage Subgroup Workshop
Date:	Friday, April 15, 2016
Location:	HDR Olympia Office
Attendees:	Shaun Bevan – HDR Mike Garello – HDR Matt Prociv – HDR John Ferguson – Anchor Larry Swanson – Anchor (via phone) Bob Montgomery – Anchor Mark Mobbs – Quinault Indian Nation Mara Zimmerman – WDFW (via phone) Dan Rodding – WDFW (via phone) Justin Allegro – WDFW Jessica Hausman – WA DOE Jeff Fisher – NOAA Jeff Brown – NOAA

1) Reviewed previous action items:
   a) Periodicity/Migration Timing
      i) Mara Zimmerman (WDFW) previously provided comments have been incorporated
      ii) John Ferguson (Anchor) suggested moving Spring Chinook spawning from last week of August to middle of October.
      iii) John Ferguson (Anchor) – Fall Chinook spawning timing first week in October to middle of December
      iv) John Ferguson (Anchor) – Coho spawning last week of October through mid February
      v) John Ferguson (Anchor) – Winter Steelhead middle of February to first week in June
      vi) Mara Zimmerman (WDFW) suggested moving adult arrival to end of spawning for Chinook, coho, and winter steelhead
      vii) Mara Zimmerman (WDFW) questioned why summer steelhead were on the list, they are not known to be at this project location. Mara says there is no wild population, only hatchery population moving to hatchery.
         (1) Jeff Fisher (NOAA) agrees, he doesn't know of any wild population here.
         (2) Mark Mobbs (Quinault) doesn't know of any evidence of historic population at project.
      viii) Dan Rodding (WDFW) states that kelts can not migrate prior to spawning, so group agreed to move kelt outmigration from Mid February through Mid July.
      ix) Jeff Fisher (NOAA) has made several attempts to get input from USFWS on Bull Trout
         (1) Has not received any feedback
         (2) Mark Mobbs (Quinault) suggested with potential future listing of Bull Trout we should make more of an effort to get this finalized.
         (3) Action Item: Mike Garello (HDR) and John Ferguson (Anchor) to send periodicity to USFWS to give them another chance to comment on cutthroat, bull trout, and lamprey.
   b) Population Abundance (presented by Justin Allegro – WDFW)
      i) Justin Allegro went over a few methods with Mara Zimmerman
         (1) Upstream
            (a) Presented three methods for determining future projected peak annual runs.
            (b) Proposed peak daily has yet to be determined.

HDR

(i) John Ferguson (Anchor) stated 1%-2% for maximum peak hourly rate, from Kozmo Bates (1992). This document also suggests 10% for peak daily rate.

(2) Downstream

(a) WDFW is having more difficulty coming up with method, but Mara Zimmerman has a vision on how she would like to determine values

(i) Mara Zimmerman (WDFW) is thinking of using mean smolt estimates from traps across the state and coming up with correlation of watershed to smolts.

(ii) Mike Garello (HDR) suggested using the adults and redds to come up with projected smolt outmigrant numbers. The Group preferred Mike's method if possible.

2) Fish Passage Facility Performance and Survival Estimate

a) Mike Garello (HDR) reviewed last biennium method of developing performance/survival estimate.

i) It was scored through rating and scoring system

b) This Biennium

i) Consultant team revised performance and survival estimates to better inform EDT modeling. Reviewed with Jeff Brown (NOAA). The values are based on:

(1) 1st: Actual performance and survival numbers from operating projects.

(2) 2nd: Estimated performance and survival at similar operating projects.

(3) 3rd: No data is available or there are no operating projects similar to facility. Performance and survival estimates are based on professional judgment.

ii) Jeff Fisher (NOAA) asked for the performance and survival estimate to be narrated with write-up, or notes in the table.

iii) Action Item: Jeff Fisher (NOAA) needs two weeks to review, will coordinate with Jeff Brown

(1) Want to have background/references fill out prior to NMFS review.

(2) HDR to send out by 29th of April.

(3) Comments due 13th of May.

(4) Dan Rodding (WDFW) will review as well.

iv) Dan Rodding (WDFW) would like consultants to discuss range of values from other facilities and then the value we assumed to make sure consultants are not being overly optimistic.

v) Jeff Brown (NOAA) asked how we incorporate CHTR into FRO, because it will be operated very infrequently.

(1) Group was not yet certain how it will be implemented, but will consider it moving forward.

vi) Mike Garello (HDR) discussed FRO tunnel downstream survival numbers

(1) Consultant lowered survival for downstream due to debris rack, potential debris, and predation.

vii) Jeff Brown (NOAA) suggested that Lamprey CHTR occurs at Bonneville and John Ferguson says they are transplanted all over. Therefore, suggest increasing performance.

viii) Group agreed to increase Lamprey performance for ladder, with the assumption they would be trapped within the ladder to transport them.

ix) Discussion on FRO monitoring and evaluation question. Do we need to monitoring and evaluation facility for FRO tunnels?

(1) Group agrees that we don't currently have a good basis for changing values for monitoring and evaluation.

3) Dam operations modeling update by Bob Montgomery (Anchor)

i) Look at how far downstream temperature benefits are seen

ii) Modeling does predict lake turnover (mixing)
iii) Haven't looked climate change/perpetual drought
(1) Climate change for this basin: Same annual precipitation, but arriving more in winter, less in summer. Also more precipitation via storm events (more intense precipitation for short events).
iv) Modeling does account for (model) rain-on-snow events

b) FRFA Operations
i) Bob Montgomery (Anchor) reviewed temperature effects on weighted usable area of habitat and the tools used to determine most effective range of operation.
(1) Found temperature is bigger benefit than flow.
(2) With this reduced flow release, the results show far less variation in the conservation pool elevation. All years had less than 40 ft of fluctuation, most had less than 35 ft. The only time forebay elevations where outside this range was during flood operations every few years for short periods of time.
(a) Proposed flood operations has not changed since last biennium
(b) Trigger for retention is defined by "major flood" as defined by NOAA
(i) Trigger is 38,800 cfs at Grand Mount (~7 year event)
(ii) About a 15% chance of operation above conservation pool in any given year

c) FRO Operations
i) Same trigger for initiating flood operations as FRFA
(1) Will be different drawdown operations due to slides and debris management
(2) For January 2009 event, Fro would have pool for approximately 1 month
(3) This biennium we are including a longer closure for debris management, will update analysis on closure duration once modeling is finalized.

4) FRO Fish Passage Tunnel refinement and design progress
a) Mike Garello (HDR) summarized thought process on how we would get fish out of FRO reservoir and maximize survival.
i) Lower RO outlet being removed, was the primary flood control facility for FRO.
(1) Jeff Brown (NOAA) concerned that we will not have control on how the sediment is deposited in the tunnels, and we have discussed that it will be difficult to meet velocity criteria even with set geometry.
ii) HDR plans to present more on design refinement at next meetings.

5) Head of Reservoir Discussion
a) Discussion: Large uncertainty in how often fish will pass into reservoir, do we ever get fish out of the reservoir, how do we handle the tributaries.
i) Jeff Brown (NOAA) votes to move down the path of using a passive surface collector and eliminating the head of reservoir collector. John Ferguson (Anchor) and Mark Mobbs (Quinault), and Jessica Hausman (WA DOE) all agree.

6) Next meeting:
a) May 20th 10am- 1pm @ HDR Olympia office.

FISH PASSAGE TECHNICAL COMMITTEE MEETING NOTES

Date: June 9, 2016
Time: 10:00 am to 1:00 pm
Location: HDR Olympia Office
Participants: Mike Garello (HDR), Shaun Bevin (HDR), Matt Prociv (HDR), Ed Zapel (HDR), Anna Mallonee (HDR), John Ferguson (Anchor QEA), Justin Allegro (WDFW), Don Ponder (WDFW via phone), Mara Zimmerman (WDFW via phone), Carol Cloen (WDFW), Jeff Fisher (NMFS), Jeff Brown (NMFS), Mark Mobbs (Quinault Tribe), Larry Swenson (Anchor QEA via phone), Jessica Hausman (WDOE)

Conference Call Information: 866-583-7984, 74660139#; Leader-Pin: 532659#

Meeting Objectives

1. Updates on action items already in progress
2. Discussion on refinement of fish passage conduits
3. Discussion on refinement of fish ladder concept
4. Discussion on and selection of floating surface collector design criteria

Discussion Topics

1. Updates on action items already in progress
   A. Performance and survival values
      i. HDR and AnchorQEA sent out working version of performance & survival memorandum and updated spreadsheet via e-mail to the subcommittee on June 6, 2016 for review. Mike presented a brief overview of memo to meeting participants.
      ii. Survival = Fish not harmed
      iii. Are values for Steelhead yearling and sub-yearlings?
      iv. Send questions & comments to Mike G. Two weeks to get comments back, June 24th
      v. Justin – Asked for disclaimer indicating permitting numbers may be different than this even though used in EDT
      vi. Mara – suggested we look at Cowlitz Falls on trap & haul
      vii. Ed – White River & Sunset Falls too
      viii. Mark – Add river names to project names, to help reader better determine location of facilities
      ix. Justin – prefers adding specific values received via phone. Using for each REF facility

      x. Jeff Fisher – will obtain performance testing reports and provide them to the consultant team for review and incorporation. Consultants will synthesize the data when received.

xi. Operates (fixed port collector) one gate @ a time so performance analogous to fixed collector

B. Population abundance

i. Juvenile numbers per calculations but need to be done and agreed on by agencies and tribes.

a. Population abundance is needed to size and provide cost estimate for FSC & CHTR fish passage alternatives.

b. WDFW agreed to provide Juvenile abundance by June 24th

ii. Matt – identified that Cutthroat abundance has not been provided yet

a. Mara– cutthroat seen in river scape above dam

b. WDFW to provide Cutthroat abundance – adult and juvenile. Not much, if any, data available to develop abundance numbers, but need to cover because present and soon to be listed

c. No due date set, but needed soon to inform design; not likely to significantly influence design given potential for small numbers

d. Mark – Cutthroat found in upper watershed. Use ha

iii. Mark and Jeff Fisher to talk about contacting USFWS to get them involved in this committee

2. Refinement of fish passage conduits

A. Preliminary hydraulic characteristics

i. Mike G. presented brief overview of FRO dam operations

ii. Is there juvenile stranding risk within the reservoir at drawdown?

a. Justin – 10ft/day drawdown sounds too FAST

b. HDR to review and get answer back to committee

iii. Average 15% chance of occurring (Flood control, gates shut, trap and haul)

a. About 7-year reoccurrence; can be three times in five years, one time in 10 years; about seven years on average

iv. 38,000 cfs and Grand Mound Gate predicted within 48 hours triggers flood operating @ FRO

a. Matt says 50% variability @ FRO in flow; Ed in Storm centering between FRO dam and Grand Mound gage locations

v. Ed explains FRO conduit operations

a. Water always backwatered in FRO conduits in normal operation to mimic existing conditions

vi. Mark – is FRO operation increasing duration of turbidity verses existing?

a. Ed – unknown. Very hard to model. Should be looked into.

vii. Carol – Q verses velocity plot looks like FRO is reducing river flow. Suggest modifying presentation of data/graphs so clearer to public in PEIS.

viii. Ed– conduits mimic existing gorge. Gorge and conduits are slack water @ low flow because downstream is hydraulic control, existing and proposed (stilling basin).

    a. Next graphic= overlay velocity, existing and proposed on plan view of river through upstream and downstream of dam.

ix. Ed – Goal to match conduit function to existing conditions.

    a. Jeff Brown – goal of culvert design always to match existing conditions rather than set velocities, exc. (one size fits all = NO). This will also apply to setting/informing juvenile criteria – if juveniles excluded naturally at times, would be reasonable to allow conduits to exclude at same times.

x. Don – will have some sediment & size depositing as existing?

    a. Ed – starting sediment transport model now. It should tell us. But if match existing hydraulics then would expect sediment transport & depositions to act the same.

xi. FRO does pond during normal (non-flood) operations. Just not during fish passage flows. (Q> fish passage flow)

xii. Group came to agreement to proceed with conduit design approach to mimic existing conditions rather than achieve 2 fps criteria.

B. Refinement of volitional passage requirements

    i. Jeff Brown ok with information presented in slide presentation

    ii. 65mm length for PARR selected because jump height criteria cutoff in published criteria

    iii. Mara – Confirmed that the design is based around 65mm PARR because that is what we have data for not necessarily because it represents the fish present in the river.

    iv. Do we use criteria on this slide or design to mimic existing/natural conditions?

        a. Consensus is to compare existing/natural versus proposed conditions

        b. 2D/3D hydraulic modeling will still be conducted for the FRO dam option to evaluate existing versus proposed conditions. Juvenile criteria is still necessary for FRFA because the fish ladder and CHTR are not intended to mimic the natural stream system as the FRO is.

        c. Look at percentage of time in different flow regimes for existing and proposed and compare.

    v. Jeff Brown – put sediment in conduits during construction of dam so not wait for event to fill conduit with sediment.

3. Refinement of fish ladder concept (not discussed)

4. Floating surface collector design criteria

    A. Matt P. provided the attendees with a brief overview of the conept level design elements and initial layout of facility components.

    B. Jeff Fisher – Would like to see flexibility in guide net configuration so that anchor points can be moved to accommodate unforeseen conditions or to improve effectiveness. This will decrease deployment costs over the life of the project.

C. Tracing through Res: at other FSC facilities have hydropower outflow in addition to FSC flow; here will only have a few hundred CFS outflow.
    i. Larry – Narrow Res & outflow = better FSC performance
D. Need to look at fully document, updated performance percent, low outflow and attractions flow
E. M & E will likely be required for FSC, so we should plan on it. Agencies currently don't have requirement on whether it occurs on land or on FSC.
    i. Applies to center facilities too
F. Mike to send presentation to teams
G. John to send date when Chip needs final performance/survival table

5. Next Meeting
    A. July 8th – 10 am to 1pm, HDR Olympia Office
6. Adjourn at approximately 1:00 PM

FISH PASSAGE TECHNICAL COMMITTEE MEETING MINUTES

Date: July 8, 2016
Time: 10:00 am to 1:00 pm
Location: HDR Olympia Office
Participants: Mike Garello (HDR), Shaun Bevan (HDR), Matt Prociv (HDR), Ed Zapel (HDR), Anna Mallonee (HDR), John Ferguson (Anchor QEA), Don Ponder (WDFW), Mara Zimmerman (WDFW), Jeff Brown (NMFS), Mark Mobbs (Quinault Tribe), Larry Swenson (Anchor QEA), Jessica Hausman (WDOE)

Meeting Minutes

1. Updates on action items already in progress
   A. Population abundance
      a. Cutthroat trout abundance - Still needs to be identified
      c. John Ferguson - has also put together downstream migrant numbers.
         i. Methodology - Used the previous spawner estimate provided by WDFW, published values of average eggs per female, and published estimates of egg to smolt survival (Bradford–1995, Scheuerell et al. – 2006) to develop an estimate of smolt outmigrant abundance for Chinook and coho salmon.
         iii. John's preliminary estimate is 500,000 to 1 million smolts annually
      d. Mara Zimmerman -
         i. Methodology - Used a mixed methodology of spawner numbers for chinook and basin size (habitat) for coho.
         ii. Mara's preliminary estimate is about 800,000 annually. This is within the range John's method estimated.
         iii. Mara also applied smolt outmigration timing from similar basins to get daily numbers and timing.
            1. Mike Garello reiterated that developing abundance numbers independently, based on two different methodologies serves to help reassure that the numbers are reasonable and defensible.
      e. **Decision:** Engineering should move forward assuming a Baker size FSC. FSC sizing will be updated after the August meeting when Mara and John have finalized the values.
      e. **<u>ACTION ITEM:</u> Mara and John to put together joint memo finalizing abundance numbers. Draft document by next meeting in August. John and Mara will coordinate via phone after meeting.**
   B. Performance and survival tables
      a. Only received comments from Carol Cloen (WDFW), moving forward.
      b. Jeff Fisher still needs to provide annual performance reports for similar facilities from other basins.
      c. John Ferguson – There was a question posed as to why juvenile upstream performance for FRO tunnel is 60%. We are now focusing on designing tunnels to mimic existing hydraulics, so this value may increase.
   C. Fish Passage Conduit refinements and hydraulic modeling

a. Ed Zapel - HDR has put together sediment models for the existing and proposed conditions. 2-dimensional hydraulic modeling of the conduits and river upstream and downstream of the FRO dam will begin shortly.

2. Reservoir drawdown
   - A. Existing reservoir characteristics
     - a. Matt Prociv provided a review of work to estimate slopes that exist within the reservoir footprint.
   - B. Review of current draw-down strategy
     - a. Matt Prociv provided a review of FRO drawdown operations
   - C. Proposed approach to address fish stranding during draw-down operations
     - a. Matt Prociv proposed we grade drainage channels in areas identified as potential stranding locations. Will coordinate with Bob Montgomery to develop additional detail regarding this possible mitigation.
3. Refinement of fish ladder concept
   - A. Overall concept
     - a. Mike Garello presented overview of fish ladder layouts
   - B. Entrance options
     - a. Discussion on having entrance at velocity barrier versus in stilling basin
       - i. Velocity Barrier - the canyon downstream is approximately 75 feet wide and doesn't have the width to meet standard fish barrier design criteria to effectively exclude fish at high flows. Matt Prociv said he was able to design a velocity barrier that met the goals of a velocity barrier at high flows fairly well at a width of 150 feet. He notes that this widens the river considerably at this location and the resulting sediment deposition at the toe of the apron would likely make the barrier ineffective or block fish access to the fish ladder entrance.
       - ii. Stilling basin - Jeff Brown suggested slanting the stilling basin sill to concentrate flow toward ladder entrance side of the stilling basin during low flow.
         1. Don Ponder noted that the design of the stilling basin needs to be well thought through and discussed with the fisheries agencies if the ladder entrance is to be co-located with the stilling basin.
       - iii. **Decision:** The group agreed to move forward with locating the fish ladder entrance in the stilling basin.
     - b. Discussion regarding water temperature in the fish ladder
       - i. Temperature control of the water in the fish ladder and AWS at low flow in the FRFA dam option is required and needs to be developed. For example, if cold water is used in the ladder, then, when fish reach the ladder exit they could reject entering the reservoir because of the much warmer temperature at the top of the reservoir.
       - ii. Jeff Brown suggests juvenile passage may not be needed for the FRO CHTR facility.
   - C. Exit options
     - a. Mike Garello - presented 3 exit options.
       - i. Linear exit with 40 gates,
         1. Similar to Soda Springs (Umpqua) and North Fork (Clackamas) projects

ii. On face of dam with three stacked sets of exit gates (40 gates total) and transport channels connecting each set, and
iii. Fish lock at top of ladder to forebay.
1. **Decision:** This option was removed from further consideration because it does not allow fish to pass volitionally. This alternative is to remain a fully volitional option.

b. Guide Nets
i. Mike Garello provided an overview of proposed block net and center guide net.
ii. Jeff Brown asked question on how adults get past the nets.
0. Mike Garello stated that the current thought is to provide a passage way with nets from the fish ladder exits to the barrier/guidance net.

D. Jeff Brown - asked who will be making the decision for which fish passage facility is carried forward for each dam?
a. **Decision:** The group will provide recommendations for each dam alternative that we pass along.
b. The group agreed that operation and maintenance of facilities may need to be weighted more heavily to reflect the uncertainty and high variability of operational quality (directly related to performance and survival) tied or O&M funding.

4. Floating surface collector design criteria
A. Updated design concept and criteria for FSC
a. Attraction Flow
i. Mike Garello presented initial data HDR has collected on FSC facilities and their reservoir characteristics in an attempt to inform the selection of an attraction flow value.
ii. Jeff Brown would like to tease out the data for what is happening within reservoir separately from what happens right in-front of collector.
iii. **Decision:** The group agreed for now that we should move forward with meeting NMFS/state screening criteria at 500 cfs attraction flow and have the ability to pump 1000 cfs with screening criteria being exceeded. As additional information is collected and synthesized by HDR regarding other FSC facilities and their reservoir characteristics, the design attraction flow value(s) may be revisited.
iv. Jeff Brown and Ed Zapel suggested that a physical model of the reservoir is needed in the work done in the next biennium in order to discover potential fatal issues with the preferred alternatives before proceeding to design development. The rest of the group agreed. Mark Mobbs offered to communicate the need for this model being in the next budget through QIN channels.

5. Adjourn
A. Next meeting will be August 26 @ 10am in HDR Olympia
B. Mike Garello to send out PowerPoint and meeting notes
C. Mike Garello to send out next meeting invite

FISH PASSAGE TECHNICAL COMMITTEE MEETING MINUTES

Date: August 26, 2016
Time: 10:00 am to 1:00 pm
Location: HDR Olympia Office
Participants: Mike Garello (HDR), Matt Prociv (HDR), John Ferguson (Anchor QEA), Carol Cloen (WDFW), Justin Allegro (WDFW), Jeff Brown (NMFS), Mark Mobbs (Quinault Tribe), Jessica Hausman (WDOE)
Phone: Ed Zapel (HDR), Bob Montgomery (Andioi), Don Ponder (WDFW), Mara Zimmerman (WDFW)

Meeting Minutes

1. Bob – Describes the process for FRO reservoir flood fill and drawdown. When the gates are closed, the river downstream of the dam still sees 300 cfs from the conduits plus tributary flow downstream of the dam. Drawdown is 10 ft/day - limited to this due to slope stability in reservoir. During debris removal 2 ft/day drawdown as needed to remove debris. Longer times required for more debris, larger flood events. Debris removed at landing upstream. Debris removal duration is about 2 wks; larger for larger flood events.
   A. Justin – During the transition, is the 2 in/day criteria met downstream? Referring to vertical line on outflow hydrograph.
      a. Bob – Good observation. 10 ft/day exceeds river ramping rates of 2 in/hr written for hydro facilities. This can be addressed
2. Bob – Summarized the reservoir stranding report, including the literature review and application to Chehalis. Slopes <5% are a potential concern. Stranding independent of drawdown when ramping 7 to 16 inches per hour. Slope has greater influence. Possible mitigation – Address fish stranding after a flood by performing fish rescue, change ramping based upon fish presence, and/or draw down at night when fish are more active.
   A. Ed – What about physical modification of the reservoir as possible mitigation for stranding?
      Bob – May not be sustainable given reservoir sedimentation.
      Ed – It has reasonable support in the literature.
      Bob – Could be addressed as an option but would require adaptive management.
   B. Justin – How will it change sedimentation?
      Bob – Sediment transport modeling suggests about 1 ft of accumulation. Narrow canyon confines areas of concern to small areas. May not be a significant concern.
      John – Reiterated Justin's approach: adaptive management.
      Justin – Possible need for subcommittee to inform operations.
      Carol – Have lamprey ammocoetes been considered? They may likely move into sediments.
      Bob/John – Will make additional efforts to identify literature regarding ammocoetes if it exists.
3. Don – Transition zones between flows informed by natural hydrograph transitions.
4. Bob – Discusses seasonal reservoir refill:

A. Proposing to reduce flow and increase quality (decrease water temperature). Optimization exercise to confirm actual operations. During refill (arbitrary modeling exercise) at first storms. In = out after conservation pool is full. May be key to allow early pulses for fish, then refine at a later date. There is some flexibility.
B. Jeff – Optimize for fish as long as there is assurance that reservoir will fill.
C. Mark – First storms coincide with spawners. Flow mods may impact spawning success.
D. Carol/John – Seasonal cues for salmon and other species. Likely operations would be addressing this issue in next biennium.
E. Bob – Lots of flexibility possible.
F. Mark – Lamprey would be reduced.

5. **<u>ACTION ITEM</u>: John Ferguson will send out a draft memo on reservoir stranding.**
   A. Summary of memo conclusions may need to be added into the text of the PEIS.
6. Current FRFA refill approach is to refill 40 feet of operating reservoir swing 1st with fall floods.
   A. Concern with spawning cues for Chinook (mostly fall) and Coho but also other cues for other aquatic species.
   B. Also concern with inundation of redds in reservoir if refill is delayed.
      a. Need to get into specific details to address this so it makes the most sense to delay conversation on this until next biennium.
3. Bob leaves call, Mark joins.
4. No update of cutthroat trout abundance
   B. Keep as target species
   C. Low abundance but no numbers; should not substantially impact passage design/cost.
   D. Periodicity for cutthroat is year round.
2. **<u>ACTION ITEM</u>: Formal memo from Justin on upstream and downstream population abundance to formalize/document numbers for design.**
3. **<u>ACTION ITEM</u>: HDR to send FRO conduit design draft memo to full group.**
   A. Update of memo sent to WDFW prior to FRO conduit design update with WDFW last week.
4. **<u>ACTION ITEM</u>: Jeff Brown to get swift performance data and reports where possible from Michelle Day.**
5. Put juvenile upstream passage efficiency for CHTRs at 60% because fish will separate at the handling facility and will be moved separately. Injury for this grating/bar rack system is about 1%.
   A. This is for fish motivated to move upstream.
   B. Group agrees CHTR juvenile upstream performance is shown as same percent as the FRO conduits. This requires closer look. Put Conduits at 65% and CHTR at 55% for now to illustrate that the conduits have better performance.
6. HDR will put separate Lamprey trap and transport facilities on the full length ladder. Doing so should increase performance from 40% to 60%.
7. **<u>ACTION ITEM</u>: HDR will revise the performance table and send it to the team.**
8. The group agrees no juvenile or Lamprey passage with CHTR facility for FRO is required.
9. Adjourn
   A. Next meeting will be September 30th at 10:30 AM to 1:30 in HDR Olympia. (Meeting was cancelled.)
   B. Mike Garello to send out PowerPoint and meeting notes
   C. Mike Garello to send out next meeting invite

Attachment B
Conduit Hydraulics Evaluation

TECHNICAL MEMORANDUM

Date: June 30, 2017
To: Fish Passage Technical Subcommittee
From: Fish Passage Design Study Team
Re: Assessment of FRO Concept Fish Passage Conduit Hydraulics and Anticipated Performance

Introduction

The fish passage design team has been tasked with assessing fish passage performance of the flood retention only (FRO) structure proposed as part of the Chehalis Basin Flood Strategy. The assessment reviews the optimum conduit number, sizes, and inverts, and quantifies the ability of the conduits to match existing conditions and criteria for successful fish passage. Methodology and assumptions used in the assessment and results of the assessment are discussed in this document. Conclusions and recommendations are presented to inform subsequent fish passage design refinements and next steps.

Document Objectives

The following objectives are met in this memorandum:

- Identify the criteria that define successful conduit passage;
- Estimate the depths and velocities for existing and proposed conditions in the project reach;
- Summarize how sediment accumulations in the conduits can influence variations in estimated depths and velocities;
- Compare the similarity of proposed conditions with existing conditions and evaluate the ability of proposed conditions to meet fish passage criteria;
- Assess the impact of the proposed project on upstream and downstream fish passage; and
- Summarize conclusions and additional refinements that could be considered in the next biennium to improve fish passage through the proposed conduits.

Document Purpose

The purpose of this document is to inform the Flood Damage Reduction Subcommittee regarding the anticipated performance of the FRO fish passage conduits. Additionally, this document includes recommendations for the Fish Passage and Dam Design Team regarding next steps for making refinements and improving fish passage performance.

Scope of Work

The following work elements were completed to inform the content of this memorandum:

- A 1-Dimensional (1-D) Hydrologic Engineering Center River Analysis System (HEC-RAS) model of project reach and a variety of potential conduit designs to determine highest performing conduit sizes and inverts;
- An unsteady sediment transport simulation of existing conditions in the channel to study the variation of depth and velocity when influenced by sediment accumulation and scour;
- Development of fish passage criteria and equivalent flow rates from the 1-D HEC-RAS and unsteady sediment transport simulation data;
- A 1-D HEC-RAS model of both existing river reach and chosen proposed conduit configurations;
- Comparison of existing and proposed conditions using velocity and flow data from the 1-D HEC-RAS steady state simulation; and
- Development of a spreadsheet model to determine the impact of hydraulic changes and flood retention operations on fish passage at the project location.

Methods

Determination of Fish Passage Criteria

The fish passage criteria used in this report were developed from guidance provided by agencies and during fish passage subcommittee meetings. Hydraulic design criteria provided by the Washington Department of Fish and Wildlife's (WDFW) 2013 *Water Crossing Design Guidelines* sets the maximum velocities that allow passage through certain structure designs and provides a numerical goal for water crossing structures to achieve. Criteria for adult species across water crossing structures with lengths of approximately 200 feet is 2 feet per second (fps); criteria for juvenile species for the same design specifications is 1.3 fps (HDR, 2016a). However, during discussions with the fish passage subcommittee, mimicking natural depth and velocities exhibited in the existing river channel was prioritized over meeting hydraulic design criteria in an attempt to accommodate a broader range of species and life stages. Comparison of proposed with existing hydraulic conditions in the reach, as a result, are assessed first in this technical memo. Proposed conditions are still assessed through the use of adult species hydraulic design criteria in order to understand the quality of proposed conditions if they do not match existing conditions in the reach.

In addition to the velocities discussed above, fish passage design flow rates were used to understand the performance of the FRO fish passage conduits. Flow rates were developed as part of the Fish Passage Concept Design Report (HDR, 2016a) and were based upon National Marine Fisheries Service (NMFS) and WDFW guidelines involving exceedance calculations and estimation of U.S. Geological Survey (USGS) flows at the proposed site. The highest 5 percent exceedance flow of species expected to pass through the site was calculated to be 2,200 cubic feet per second (cfs) and was used to determine when upstream fish passage would occur.

Flow rates equivalent to the 2 fps adult velocity criteria for existing and proposed conditions on the reach were developed to assess the performance of the FRO fish passage conduits. The flow rates corresponding to 2 fps in the existing and proposed reaches were determined through the use of a rating curve. The rating curves for existing conditions, proposed clear water conduits, and proposed conduits with sediment were developed from the 1-D HEC-RAS model data. Velocity and flow data for both existing and proposed conditions were taken from a cross section located at the downstream end of the proposed sluice. The data was plotted, and trend lines were fitted to the data. The flow rate in the reach for a velocity of 2 fps was calculated from the trend line equation. The resulting flow rate used to determine fish passage during existing conditions was 335.02 cfs, and the flow rate to determine fish passage during proposed conditions was 2,496.00 cfs for sluices and stilling basin free of sediment and 417.66 cfs for sluices containing sediment.

Determination of Initial Conduit Sizes and Inverts

The hydraulic analysis through the proposed fish passage conduits was conducted through a combination of hand and desktop conduit sizing exercises, with the one-dimensional modeling of hydraulics and sediment transport being conducted using HEC-RAS (HDR, 2016b). The HEC-RAS model extended several hundred yards upstream of the dam site and a similar distance downstream, and included more detailed topographic and bathymetric surface data collected as part of this conceptual design effort. The dam site is located astride a natural bedrock gorge, through which the Chehalis River has cut a deep channel that alternately fills and scours with sediment in response to various flow conditions. The conduit invert elevation, conduit widths, and overall conveyance were designed to roughly approximate a similar conveyance capacity as the natural gorge reach, such that natural morphology processes are largely preserved through the reach.

Early design stages included the modeling of multiple fish passage conduit options, namely the following configurations:

- Two 12-by-20-foot conduits, with invert elevations from 406 to 433 feet in 1-foot increments
- Two 14-by-24-foot conduits, with invert elevations from 406 to 433 feet in 1-foot increments
- Two 16-by-28-foot conduits, with invert elevations from 406 to 433 feet in 1-foot increments

A geometry file for each of these configurations was modeled in HEC-RAS with the conduits as open boxes and run with discharges ranging from 25 cfs to 2,000 cfs. Total discharge, water surface elevation, and velocity in the channel were plotted for this set of flows at each cross section in the proposed dam where head loss occurs (not including friction loss). However, these conduit alternatives did achieve velocities below the fish passage criteria, so more conduit layouts were designed, modeled, and evaluated using the HEC-RAS software.

To decrease these velocities through the channel, more conduits were included in the second iteration of design, and the invert elevations of these conduits were lowered from the originally proposed invert elevation of 433 feet, to submerge them below tailwater elevation and provide greater conveyance area.

Additional refinements were made after reviewing the current sluice configuration at Mud Mountain Dam, where a larger sluice with a lower elevation is used as a “workhorse” to pass the majority of the sediment, with the other two sluices able to pass higher discharges during flood events. This review is discussed further in Appendix B of the Combined Dam and Fish Passage Conceptual Design Report (HDR, 2017).

The refined conduit designs were modeled as both open-box conduits and closed culverts. These new configuration options were:

- One 12-by-20-foot conduit at elevation 408 feet and two 10-by-16-foot conduits at elevation 411 feet
- One 12-by-20-foot conduit at elevation 411 feet and two 10-by-16-foot conduits at elevation 414 feet
- One 12-by-20-foot conduit at elevation 408 feet and four 10-by-16-foot conduits at elevation 411 feet
- One 12-by-20-foot conduit at elevation 411 feet and four 10-by-16-foot conduits at elevation 414 feet

These four new configurations were run through HEC-RAS with various discharges from 25 cfs to 5,000 cfs at cross sections upstream, within, and downstream of the proposed dam structure. Existing channel conditions, obtained from Light Detection and Ranging (LiDAR) and field data, were used in the closed conduit modeling of the four configurations. A separate HEC-RAS model was made for the current channel conditions to act as an existing conditions baseline for comparative purposes.

Among the parameters presented by HEC-RAS after running the conduit configurations through the range of flows listed above, water surface elevation, velocity in the channel, and bed shear in the channel were plotted against existing channel parameters to view the performance of the options in terms of the existing reach. The comparison resulted in the selection of one of the conduit designs, discussed later in this report.

Influence of Sediment Accumulation and Scour on Depth and Velocity in Channel

Sediment transport processes in the existing channel were evaluated using the HEC-RAS sediment transport capability, applying the Meyer-Peter-Muller equation for gravel bed streams, and running the simulation for two consecutive representative hydrologic water years (HDR, 2016b). The simulation was run until a moderate equilibrium was reached in sediment throughout, with higher flows scouring the bed and lower flows tending to deposit in lower-velocity areas of the bed. When this equilibrium in sediment was reached, the 1-D HEC-RAS model was revised to incorporate the final sediment profiles into the existing baseline.

It is recognized that the accumulation and sluicing of sediment to and from the conduits will create a high level of variation in the depth and velocity values used to assess fish passage of the proposed conduits. In an effort to illustrate how this variation influences fish passage, and while considering the expectation of high flows to flush out sediment within the conduits, two versions of the proposed model were created: one that includes sediment in the conduits and one that does not. For both of the proposed models, cross sections upstream and downstream of the dam site were updated based on the revised existing model, but only one was adjusted manually to accurately characterize sediment deposition in the conduits. The model that was adjusted to reflect these estimates of change in bed profile and conveyance area contains sediment profiles informed by the updated, "post-event" existing channel geometry. The HEC-RAS models were then run again as steady-state models for the existing and proposed configurations, with flows from 25 to 4,000 cfs.

To establish the flows at which sediment would impact flow conditions and velocities, the bed mobility threshold was determined using Shields curve and known D_{50}, D_{84}, and D_{100} grain sizes of 15, 24, and 48 millimeters (mm), respectively. A description of the methods to determining the D_{50} grain size is included in Appendix B of the Combined Dam and Fish Passage Conceptual Design Report. Methods for establishing the D_{84} and D_{100} grain sizes are similar to this described method.

The Shields parameter was determined to be 0.06 from the Shields curve, and critical shear was calculated for the D_{50}, D_{84}, and D_{100} particle sizes using this value and the modified Shields equation. This modified equation takes into account the relationship among particles of different sizes, understanding that larger particles are likely to block smaller particles from movement (USDA). It is for this reason that the mobility threshold for a D_{84} particle size is used to predict sediment motion within the channel. It is assumed that when the D_{84} particle moves, so too will the particle of size D_{50}, as the D_{84} sediment particles are no longer blocking the path of the D_{50}. The modified Shields equation used in this analysis is as follows:

$$\tau_{ci} = 102.6 * {\tau^*}_{D_{50}} * D_i^{0.3} * D_{50}^{0.7}$$

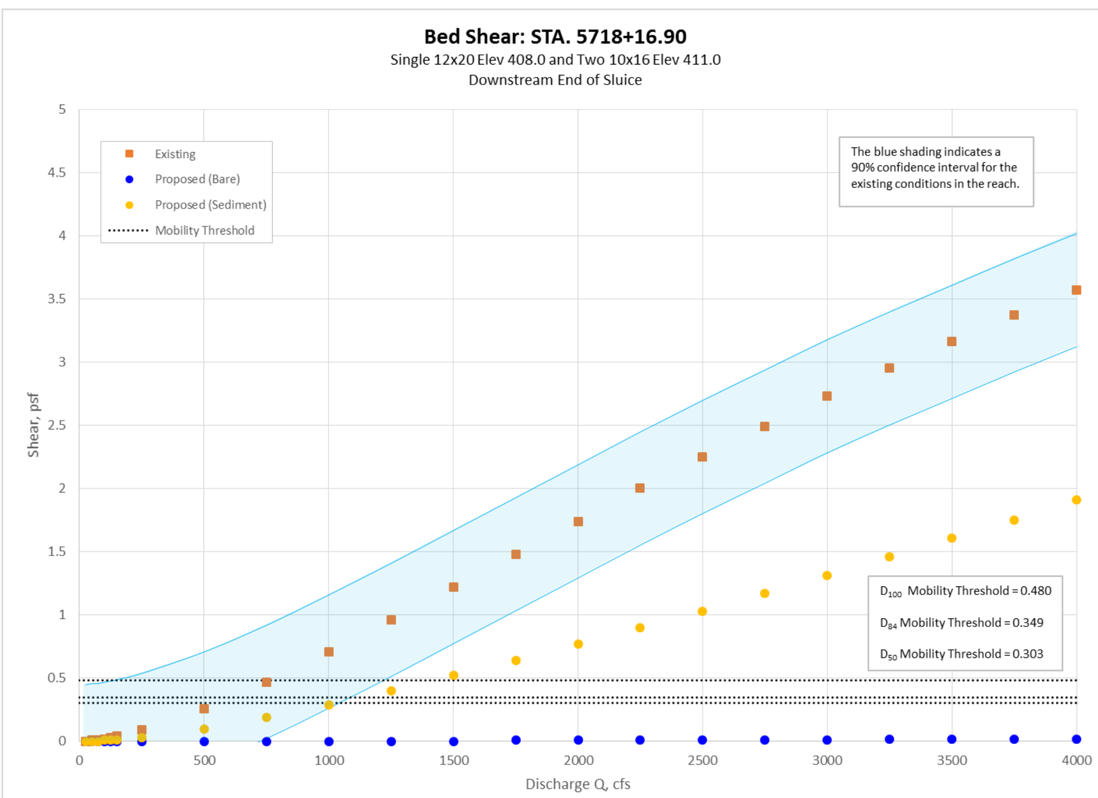
Where

τ_{ci} = Critical shear stress at which sediment particle of interest begins to move, psf
${\tau^*}_{D_{50}}$ = Dimensionless Shields parameter for the D_{50} particle size
D_i = Diameter of the particle size of interest, ft
D_{50} = Diameter of the median or 50th percentile particle size of the channel bed, ft

The critical shear values calculated from this equation (for the three sediment sizes mentioned previously) were compared with the estimated shear in the channel for all chosen flows (25-4,000 cfs) at each cross section. Velocity and corresponding flow conditions were determined at locations where the estimated bed shear exceeds the mobility threshold, causing particle motion.

Using data from the updated 1-D steady-state models that were created from the unsteady flow sediment transport model, existing conditions of the channel were compared with proposed conditions of the three-sluice option, much like was done in the preliminary 1-D HEC-RAS model. Estimated bed shears for both proposed models were compared on a single graphic, along with bed shear values for the existing channel reach. Confidence intervals of 90 percent for existing conditions were created in Microsoft Excel, and mobility lines were added to the plots to illustrate the limit at which sediment particles would remain stable, and when motion of particles of size D_{50}, D_{84}, or D_{100} would occur. A brief discussion of the creation of these confidence intervals can be found in the "Comparison of Proposed Conditions to Existing Median Velocity Distributions" section. Confidence intervals for the various parameters (bed shear, velocity, water depth) were created in the same fashion as those discussed in that section.

Figure 1
Bed Shear vs. Flow Plot



The bed shear comparison at cross section 5718+16.90 is included above, in Figure 1. The comparison of other parameters (velocity and water depth) between existing and proposed configurations is discussed later in this memo.

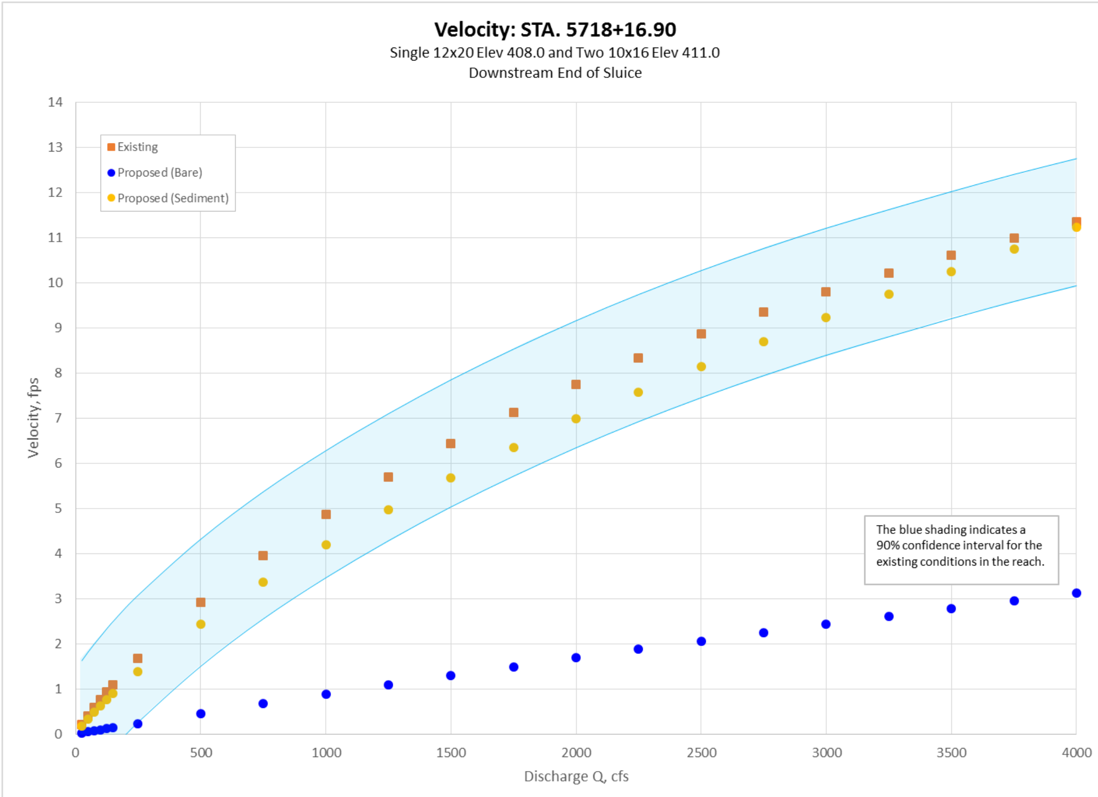
Comparison of Proposed Conditions to Existing Velocity Distributions

In order to assess the similarity of existing and proposed hydraulic conditions at the proposed dam site, confidence intervals from results of existing steady-state data were generated and compared with the proposed data sets. These confidence intervals for the existing reach were created using Excel's "CONFIDENCE.NORM" function, with a significance value of 0.10. This function produced confidence levels of 90 percent and allowed for simple comparison of the proposed versus existing channel reach. Confidence intervals were generated at seven cross sections upstream and downstream of the proposed dam. The cross sections selected were located upstream of the proposed dam site (5727+83.00, 5722+59.00), at the sluice mouth (5720+33.00), halfway down the splitter wall inside the sluice conduit (5719+53.00), at the downstream end of the sluice (5718+16.90), at the upstream end of the stilling basin (5717+20.90), and downstream of the proposed site (5713+54.00). Figures displaying the velocity versus flow plots are included with this technical memo in Attachment A.

At each cross section, after the creation of the existing channel confidence intervals, the data set for existing, proposed (both with and without sediment), and the upper and lower bounds of the 90 percent confidence intervals were plotted on a single graphic. This plot of existing and proposed conditions in the reach allowed for simple visual comparison at any particular site.

To illustrate the process of creating these graphics, the velocity comparison at cross section 5718+16.90 at the downstream end of the conduit is discussed here. The velocity and flow data in the existing and proposed reach was generated by the HEC-RAS 1-D steady-state model and was plotted for the cross section. Figure 2 below displays the data generated by the model. A 90 percent confidence interval was added to the plot for the existing data. This confidence interval is indicated with light-blue shading between two upper and lower bounds of the interval, shown as light-blue lines.

Figure 2
Velocity vs. Flow Plot Generated by HEC-RAS Steady-State Flow Data



Proposed condition steady-state simulation data was used to compare proposed hydraulic conditions with existing hydraulic conditions. Plots containing existing and proposed conditions as described above and shown in Figure 2 were compared at seven sites within the project reach. These plots were used to identify locations at which fish passage criteria were met by visually comparing existing and proposed data points, 90 percent confidence intervals, and a known criteria of 2 fps. Results of this visual comparison are summarized in Table 9 of the Results section.

Assessment of Proposed Conduits Fish Passage Performance

The ability of proposed conditions to meet hydraulic fish passage criteria was assessed over a continuous 28-year period using simulated FRO operations data. A spreadsheet model containing hydraulic and flood operations data for the reach during the 28-year period was developed by Anchor QEA. The hydraulic data contained in the spreadsheet model included hourly inflow and outflow at the proposed dam site. Existing flows in the reach were assumed to be equal to the inflows generated by the Anchor QEA model. Arguments were entered into the spreadsheet model to determine when existing or proposed flows were less than 2,200 cfs, when existing or proposed velocities were less than

2 fps, and when flow and velocity criteria were met simultaneously. Because the hydraulic data only contained flow rates, the flow rates equivalent to 2 fps discussed earlier in this report were used to determine whether conditions met the velocity criteria. The frequency at which existing and proposed conditions met criteria over the period of data was calculated on monthly and annual bases.

Impact of Flood Control Operations on Fish Passage Frequency

The effect of proposed flood operations on fish passage was assessed through the use of the aforementioned spreadsheet model and simulated flood operations data. Flood operations data developed by Anchor QEA in the spreadsheet model included hourly reservoir pool elevation and pool storage. Arguments were entered into the spreadsheet model to determine when proposed conditions would allow for fish passage. More specifically, upstream fish passage occurred with gates were open, and downstream passage occurred as the reservoir storage decreased and depths were below 1 atmosphere regardless of gate position. During a flood event, the proposed flood retention structure gates would close and temporarily store water in the reservoir upstream (Anchor QEA, 2014). Reservoir pool storage and flow rate data in the model was used to estimate when gates were open. If reservoir pool storage was equal to zero and flow rates were within typical ranges, gates were assumed to be open to allow upstream fish passage.

Reservoir pool elevation was also used to determine whether depths in the reservoir were less than 1 atm. If the reservoir pool elevation was less than the depth equivalent to 1 atm of pressure, downstream passage was considered possible. Flow rates were assumed to not be a limitation for downstream passage.

Cumulative Impact of Proposed FRO Structure on Fish Passage Frequency

The fish passage criteria and flood operations arguments used in the spreadsheet model and described in the above sections were combined to determine the cumulative net impact of the proposed project to fish passage. The combined arguments for existing and proposed conditions were compiled on monthly and yearly bases and plotted for comparison.

Results

Existing and Proposed Conduit Hydraulics

1-D Simulation Results

From the 1-D simulation, plotted results showed that implementation of a dam with the previously discussed fish passage conduit configurations on the Chehalis River would maintain fish passage conditions through the dam reach similar to the existing natural reach (HDR, 2017). Velocity and discharge plots demonstrated that the proposed conduit sizes and inverts would roughly maintain existing velocities in the channel for fish passage flows of up to 2,000 cfs. Also, the conduits were shown to not affect the water surface elevation through the channel, except through the dam itself and within the stilling basin. Bed shear plots showed no change in the reach above and below the dam. Though the parameters are similar to existing conditions, improvements could be made to better meet fish passage criteria throughout the project reach. This indicates that more design is necessary to refine the conduit configuration used in the FRO structure.

Comparing both the three- and five-conduit options with the existing channel conditions found that both options with a single 12-by-20-foot conduit at invert elevation 408 feet and either two or four 10-by-16-foot conduits at elevation 411 feet would meet both the fish passage criteria and the flood control outlet design discharge. The five-conduit option better meets the fish passage criteria, but it would additionally complicate the dam design and operations. For this reason, the three-conduit option was chosen for more in-depth evaluation. Although these configurations yielded results close to meeting the criteria, it is recognized that more work must be done to find the most appropriate solution. These fish passage conduits are continually being refined in order to better meet both fish passage and dam design criteria.

Unsteady Sediment Transport Simulation Results

Results of the unsteady sediment transport simulation were used to inform the conduit configuration selection and the sediment profile of the existing reach, and these can be found with this technical memo in Attachment A (HDR, 2017).

Estimated Depths and Velocities for Existing and Proposed Conditions

Depths and velocities in the project reach for both the existing and proposed channels were estimated using the 1-D HEC-RAS model as discussed in the methods section of this memo. Flows from 25 to 4,000 cfs resulted in various bed elevations, flow depths, and velocities throughout the reach. These parameters at cross section 5719+53.00 (halfway down the splitter wall) are shown below in Table 2. Tables 3 and 4 contain similar information for cross sections 5718+16.90 and 5717+20.90 (at the downstream end of the sluice conduits and in the upstream end of the stilling basin). Output data from this HEC-RAS model is included at the end of this memo.

Table 2
Raw Data for Existing and Proposed at 5719+53.00, Halfway Down Splitter Wall Inside Sluice

CROSS SECTION 5719+53.00: HALFWAY DOWN SPLITTER WALL INSIDE SLUICE									
FLOW (CFS)	MINIMUM BED ELEVATION (FT)			WATER DEPTH (FT)			VELOCITY (FPS)		
	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)
25	411.93	406.4	411.64	5.33	10.88	5.67	0.21	0.09	0.14
50	411.93	406.4	411.64	5.69	11.11	5.92	0.39	0.17	0.26
75	411.93	406.4	411.64	5.9	11.27	6.09	0.55	0.25	0.38
100	411.93	406.4	411.64	6.08	11.41	6.23	0.71	0.33	0.5
125	411.93	406.4	411.64	6.24	11.51	6.37	0.87	0.41	0.61
150	411.93	406.4	411.64	6.4	11.63	6.48	1.01	0.48	0.72
250	411.93	406.4	411.64	6.92	12.03	6.9	1.53	0.77	1.13
500	411.93	406.4	411.64	7.94	12.72	7.66	2.61	1.44	2.04
750	411.93	406.4	411.64	8.79	13.23	8.28	3.42	2.06	2.83
1000	411.93	406.4	411.64	9.54	13.69	8.79	4.05	2.65	3.56
1250	411.93	406.4	411.64	10.22	14.11	9.26	4.57	3.19	4.22
1500	411.93	406.4	411.64	10.84	14.5	9.71	5	3.71	4.83
1750	411.93	406.4	411.64	11.43	14.88	10.14	5.36	4.2	5.39
2000	411.93	406.4	411.64	11.99	15.25	10.54	5.67	4.67	5.93
2250	411.93	406.4	411.64	12.53	15.6	10.96	5.94	5.12	6.42
2500	411.93	406.4	411.64	13.05	15.94	11.34	6.17	5.55	6.88
2750	411.93	406.4	411.64	13.61	16.26	11.72	6.32	5.97	7.33
3000	411.93	406.4	411.64	14.1	16.57	12.09	6.44	6.38	7.75
3250	411.93	406.4	411.64	14.58	16.87	12.45	6.54	6.77	8.15
3500	411.93	406.4	411.64	15.03	17.15	12.8	6.63	7.16	8.54
3750	411.93	406.4	411.64	15.49	17.43	13.14	6.69	7.53	8.92
4000	411.93	406.4	411.64	15.93	17.69	13.48	6.74	7.9	9.27

Table 3

Raw Data for Existing and Proposed at 5718+16.90, At Downstream End of Sluice Conduits

CROSS SECTION 5718+16.90: AT DOWNSTREAM END OF SLUICE CONDUITS									
FLOW (CFS)	MINIMUM BED ELEVATION (FT)			WATER DEPTH (FT)			VELOCITY (FPS)		
	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)
25	411.62	387	413.2	5.64	30.28	4.11	0.22	0.02	0.18
50	411.62	387	413.2	5.99	30.51	4.36	0.41	0.05	0.34
75	411.62	387	413.2	6.2	30.67	4.53	0.59	0.07	0.49
100	411.62	387	413.2	6.38	30.81	4.66	0.76	0.1	0.63
125	411.62	387	413.2	6.54	30.91	4.8	0.93	0.12	0.77
150	411.62	387	413.2	6.68	31.03	4.91	1.09	0.14	0.9
250	411.62	387	413.2	7.18	31.43	5.31	1.67	0.23	1.38
500	411.62	387	413.2	8.1	32.14	6.02	2.91	0.46	2.44
750	411.62	387	413.2	8.81	32.67	6.57	3.96	0.67	3.36
1000	411.62	387	413.2	9.42	33.15	7.01	4.87	0.89	4.2
1250	411.62	387	413.2	9.96	33.6	7.4	5.69	1.09	4.97
1500	411.62	387	413.2	10.43	34.02	7.76	6.44	1.3	5.68
1750	411.62	387	413.2	10.86	34.44	8.1	7.12	1.49	6.35
2000	411.62	387	413.2	11.27	34.85	8.41	7.75	1.69	6.99
2250	411.62	387	413.2	11.66	35.24	8.73	8.33	1.88	7.58
2500	411.62	387	413.2	12.03	35.61	9.02	8.86	2.06	8.15
2750	411.62	387	413.2	12.39	35.97	9.29	9.35	2.25	8.7
3000	411.62	387	413.2	12.74	36.32	9.56	9.8	2.43	9.23
3250	411.62	387	413.2	13.07	36.67	9.81	10.21	2.61	9.74
3500	411.62	387	413.2	13.4	36.99	10.04	10.61	2.78	10.25
3750	411.62	387	413.2	13.71	37.32	10.26	10.99	2.95	10.74
4000	411.62	387	413.2	14.01	37.63	10.47	11.34	3.12	11.23

Table 4

Raw Data for Existing and Proposed at 5717+20.90, Upstream End of Stilling Basin

CROSS SECTION 5717+20.90: UPSTREAM END OF STILLING BASIN									
FLOW (CFS)	MINIMUM BED ELEVATION (FT)			WATER DEPTH (FT)			VELOCITY (FPS)		
	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)	EXISTING CHANNEL	PROPOSED (BARE)	PROPOSED (SEDIMENT)
25	415.14	387	415.6	2.11	30.28	1.71	0.49	0.01	0.21
50	415.14	387	415.6	2.45	30.51	1.96	0.78	0.02	0.36
75	415.14	387	415.6	2.64	30.67	2.13	1.05	0.03	0.5
100	415.14	387	415.6	2.81	30.81	2.26	1.29	0.05	0.63
125	415.14	387	415.6	2.95	30.91	2.39	1.5	0.06	0.75
150	415.14	387	415.6	3.08	31.03	2.5	1.7	0.07	0.86
250	415.14	387	415.6	3.5	31.44	2.9	2.37	0.11	1.23
500	415.14	387	415.6	4.25	32.14	3.62	3.66	0.22	1.97
750	415.14	387	415.6	4.8	32.68	4.18	4.67	0.33	2.56
1000	415.14	387	415.6	5.26	33.16	4.63	5.55	0.43	3.08
1250	415.14	387	415.6	5.65	33.61	5.06	6.33	0.53	3.53
1500	415.14	387	415.6	5.97	34.04	5.45	7.1	0.63	3.93
1750	415.14	387	415.6	6.24	34.46	5.83	7.83	0.73	4.29
2000	415.14	387	415.6	6.49	34.87	6.19	8.54	0.82	4.61
2250	415.14	387	415.6	6.71	35.27	6.56	9.21	0.91	4.9
2500	415.14	387	415.6	6.91	35.65	6.91	9.88	1	5.17
2750	415.14	387	415.6	7.09	36.02	7.25	10.53	1.09	5.42
3000	415.14	387	415.6	7.25	36.37	7.58	11.18	1.18	5.66
3250	415.14	387	415.6	7.4	36.72	7.9	11.82	1.26	5.88
3500	415.14	387	415.6	7.52	37.06	8.2	12.48	1.35	6.1
3750	415.14	387	415.6	7.62	37.39	8.5	13.16	1.43	6.3
4000	415.14	387	415.6	7.71	37.71	8.8	13.84	1.52	6.5

Impact of Sediment Accumulation

Estimated bed elevations from the HEC-RAS models for existing and proposed conditions show variability among water depths and velocities in the reach. Reviewing the values provided in Tables 2 through 4, located in the previous section of this memo, it becomes clear that sediment accumulation in the sluices and stilling basin causes a change in water depths and velocities at any particular cross section. Deposition of particles in a flow channel causes depth and overall conveyance area to decrease, with velocities simultaneously increasing to keep a relatively constant flow through the reach. Figure 4 overlays sediment profiles for the proposed conduits with and without sediment, showing clearly the difference in potential flow area simply from a profile view for the channel reach. Figure 5 shows a close-up of the same figure at the project location. Flows go from right to left in these profiles.

Figure 4
Comparison of Proposed Sediment Profiles – Channel Reach

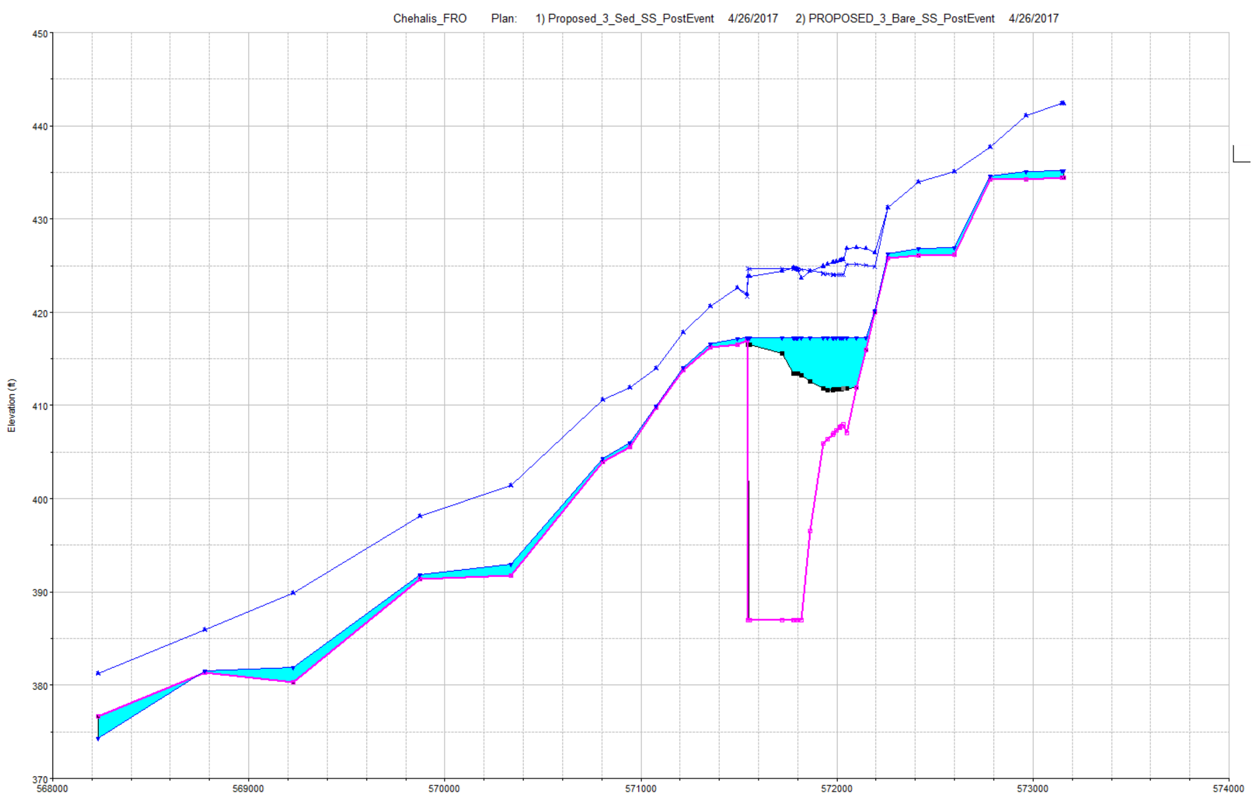
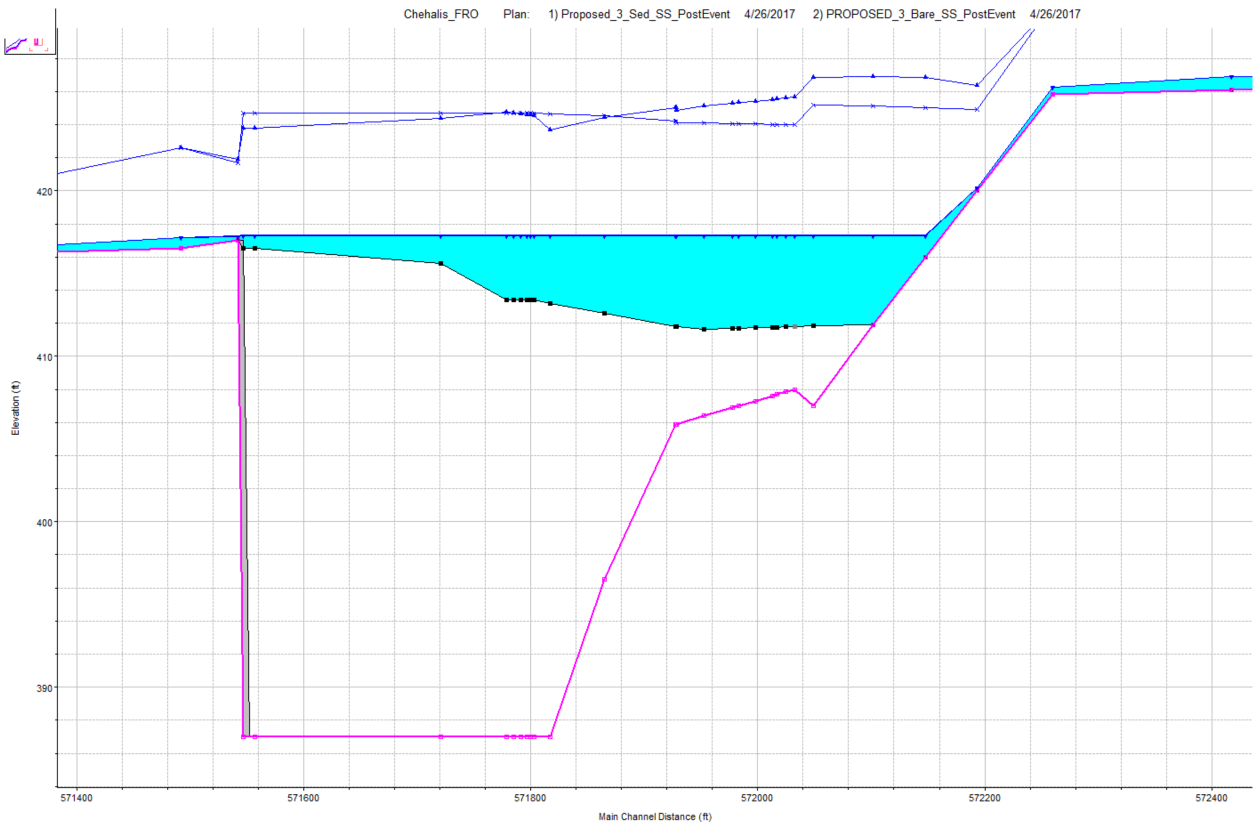


Figure 5
Comparison of Proposed Sediment Profiles – Project Detail



The blue shading in these figures is representative of the water surface elevation through the proposed model with sediment at a flow of 25 cfs. The blue lines indicate water surface elevations at a flow of 4,000 cfs. The model that does not contain sediment is outlined in a bright pink color, for comparative purposes. These profile plots indicate a difference in conveyance area, most easily seen by comparing the stilling basin minimum elevations. The sediment differential within the stilling basin is as much as 30 feet between these two proposed models, averaging around 25 feet of sediment deposition within this proposed structure. The potential increase in sediment of 25 feet is expected to decrease conveyance area greatly in the stilling basin and impact flow conditions through the reach.

The potential for such a large change in conveyance area, water depths, and velocities indicates the importance of sediment mobility in the project reach. To find when this sediment would become mobile and start to change these parameters, bed mobility thresholds were calculated for the D_{50}, D_{84}, and D_{100} size particles. They are included here, along with sediment sizes, in Table 5.

Table 5
Mobility Thresholds

SEDIMENT SIZE			MOBILITY THRESHOLD
D_{50}	15 mm	0.157 ft	0.480 psf
D_{84}	24 mm	0.079 ft	0.349 psf
D_{100}	48 mm	0.049 ft	0.303 psf

HEC-RAS-generated values for bed shear were compared with these mobility thresholds to find when particle motion at a particular site would occur. These values are included in Table 6, below. Tables 7 and 8 include velocity and flow values for the same cross sections for the proposed models.

Table 6
Mobility for Specified Existing Cross Sections

CROSS SECTION AND DESCRIPTION		D_{84} SEDIMENT MOVEMENT	
		VELOCITY (FPS)	FLOW (CFS)
5727+83.00	Upstream of proposed dam site	2.07	25
5722+59.00	Upstream of proposed dam site	2.69	25
5720+33.00	At sluice mouth, upstream	3.42	750
5719+53.00	Halfway down splitter wall	3.42	750
5718+16.90	Downstream end of sluice	3.96	750
5717+20.90	Upstream end of stilling basin	3.66	500
5713+54.00	Downstream of proposed dam site	2.00	25

For each cross section in the existing reach, the average velocity and flow at which particles of sediment of the D_{84} grain size would move are below 4 fps and 750 cfs.

Table 7
Mobility for Specified Proposed (Bare) Cross Sections

CROSS SECTION AND DESCRIPTION		D_{84} SEDIMENT MOVEMENT	
		VELOCITY (FPS)	FLOW (CFS)
5727+83.00	Upstream of proposed dam site	2.07	25
5722+59.00	Upstream of proposed dam site	2.66	25
5720+33.00	At sluice mouth, upstream	>8.85	>4,000
5719+53.00	Halfway down splitter wall	>7.9	>4,000
5718+16.90	Downstream end of sluice	>3.12	>4,000
5717+20.90	Upstream end of stilling basin	>1.52	>4,000
5713+54.00	Downstream of proposed dam site	2.00	25

Table 8
Mobility for Specified Proposed (Sediment) Cross Sections

CROSS SECTION AND DESCRIPTION		D_{84} SEDIMENT MOVEMENT	
		VELOCITY (FPS)	FLOW (CFS)
5727+83.00	Upstream of proposed dam site	2.07	25
5722+59.00	Upstream of proposed dam site	2.66	25
5720+33.00	At sluice mouth, upstream	4.83	1,500
5719+53.00	Halfway down splitter wall	4.83	1,500
5718+16.90	Downstream end of sluice	4.97	1,250
5717+20.90	Upstream end of stilling basin	4.9	2,250
5713+54.00	Downstream of proposed dam site	2.00	25

Bed mobility calculations for the proposed conduit and stilling basin configuration containing sediment show movement of particles of the D_{84} grain size (24 mm or 0.079 ft) at all cross sections for some flow at or below 2,500 cfs.

Comparison of Pre- and Post-Project Hydraulics

Although average proposed velocities do not always meet the fish passage criteria of 2 fps, these velocities are consistently matching or lower than those of the existing channel at the same cross section. Table 9, included below, shows at which of the seven cross sections the 2 fps criteria is met, at which cross sections the existing conditions are matched or improved upon, and at which cross sections proposed velocities are below the top 90 percent confidence interval of existing average velocities. This table indicates passage criteria met (or not met) for all flows specified in the HEC-RAS model (25 - 2,000 cfs), and for both proposed with and proposed without sediment.

Table 9
Velocity Criteria Assessment for Specified Cross Sections for Flows of 25 to 2,000 cfs

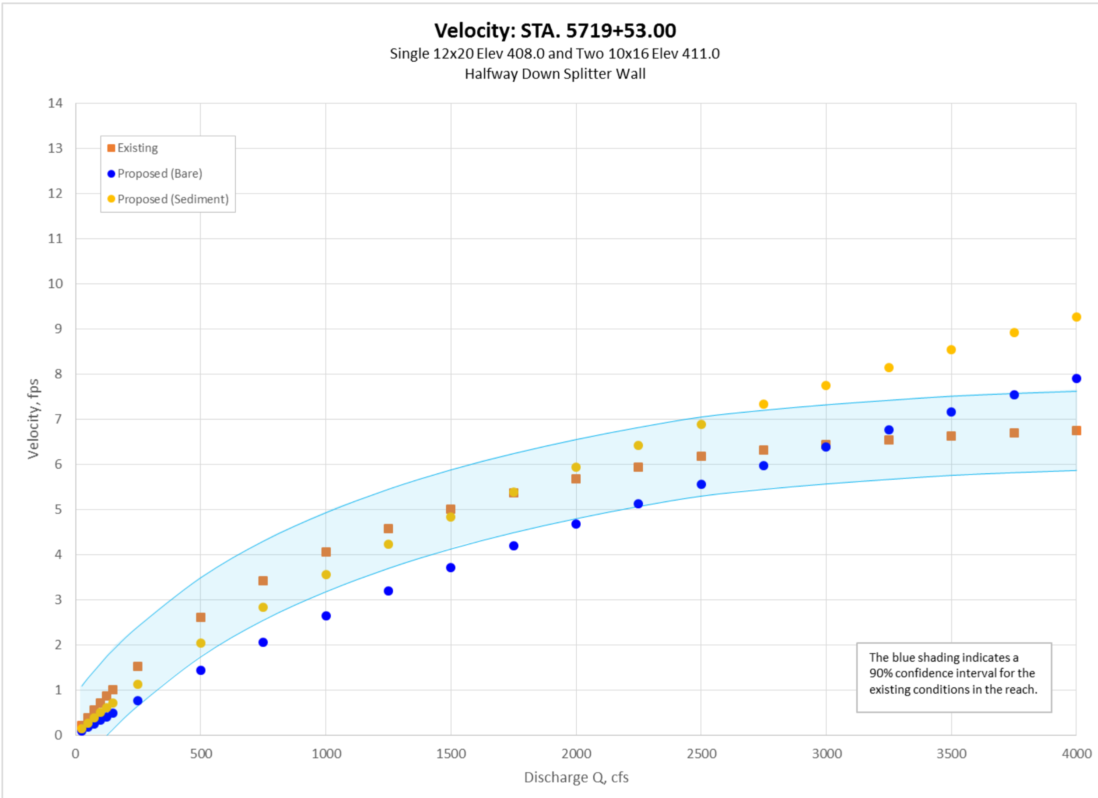
CROSS SECTION AND DESCRIPTION		AVERAGE VELOCITY LESS THAN OR EQUAL TO 2 FPS	AVERAGE VELOCITY LESS THAN OR SAME AS EXISTING	AVERAGE VELOCITY BELOW TOP 90% CONFIDENCE INTERVAL
5727+83.00	Upstream of proposed dam site	No	Yes	Yes
5722+59.00	Upstream of proposed dam site	No	Yes	Yes
5720+33.00	At sluice mouth, upstream	No	No	Yes
5719+53.00	Halfway down splitter wall	No	No	Yes
5718+16.90	Downstream end of sluice	No	Yes	Yes
5717+20.90	Upstream end of stilling basin	No	Yes	Yes
5713+54.00	Downstream of proposed dam site	No	Yes	Yes

Imitating depth and velocity conditions in the existing river channel was agreed to be the most important criteria for design of the proposed fish passage conduits. Currently, none of the locations within the existing project reach meet the WDFW maximum velocity of 2 fps for adult species for all chosen flows. At this stage in design, the proposed model that does not include sediment (bare) appears to meet the 2 fps criteria for all flows at only two of the selected cross sections: at the downstream end of the sluice and at the upstream end of the stilling basin. At all cross sections, however, both proposed models are estimated to either mimic the existing channel within 90% confidence intervals or improve existing fish passage conditions for flows of up to 2,000 cfs.

At only two locations within the channel do velocities through the proposed conduits exceed the existing velocities. This flow exceedance is estimated to occur at the sluice mouth and halfway down the splitter wall, and only at flows greater than 1,750 cfs. At a flow of 2,000 cfs, these velocities exceed current flow conditions only by about 0.26 fps. Although proposed velocities greater than existing conditions are expected for the proposed model containing sediment, they do not exceed the upper limit of the confidence interval until flows are upwards of fish passage requirements, at 2,750 cfs.

Figure 6, below, shows the proposed and existing velocities at cross section 5719+53.00, where the proposed (with sediment) velocity exceeds both existing velocities and the upper limit of the 90 percent confidence interval at high flows. The yellow marker is used to represent proposed with sediment data and the orange marker is used to represent existing conditions. This figure is included in an effort to show this flow difference in relation to the 90 percent confidence interval at this particular cross section.

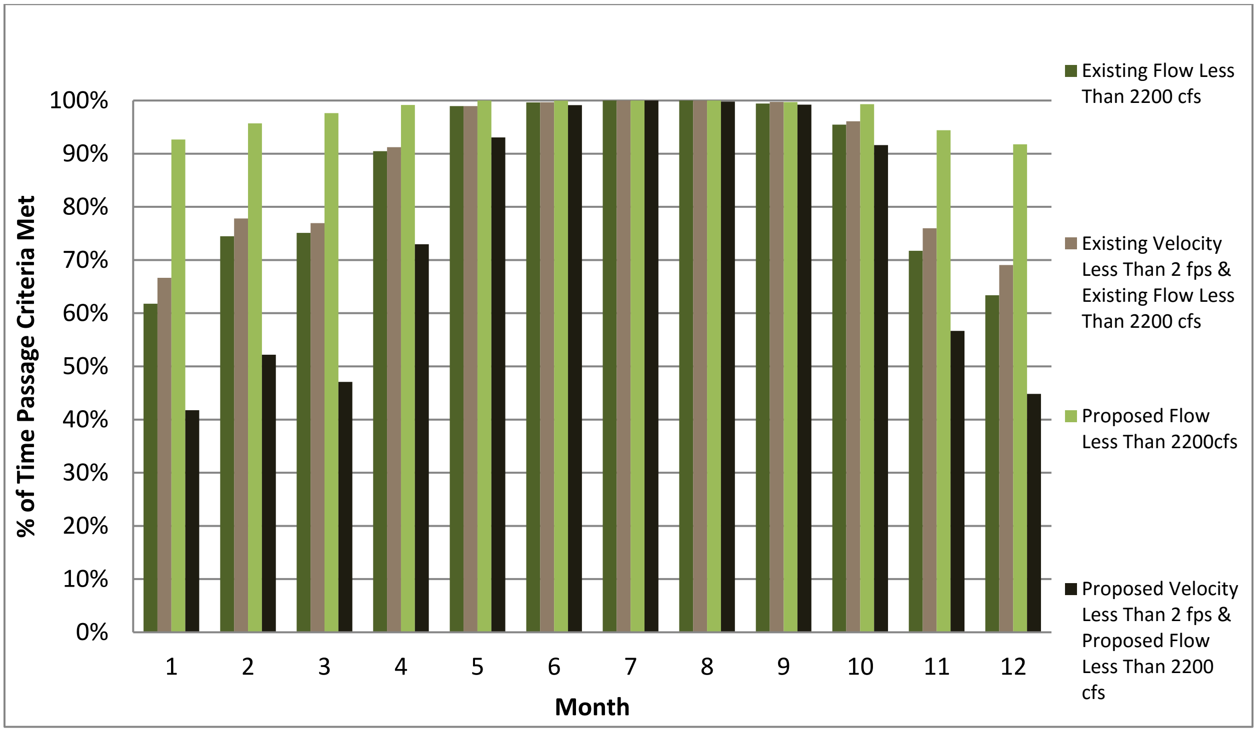
Figure 6
Velocity vs. Flow at Cross Section 5719+53.00



Proposed Conduits Fish Passage Performance

The simulated FRO hydraulic and flood operations data also indicates that the proposed hydraulic conditions in the reach meet the fish passage criteria at a frequency similar to or higher than existing conditions. Existing flows are less than the 2,200 cfs criteria approximately 86 percent of the time, and proposed flows are less than 2,200 cfs approximately 98 percent of the time. Of flows less than 2,200 cfs, existing velocities are less than 2 fps 88 percent of the time for existing and 75 percent of the time for proposed hydraulic conditions. The results of the fish passage criteria assessment suggest that the modeled hydraulic conditions and fish passage frequency at the reach are similar for existing and proposed or improve for proposed conditions. Figure 7 compares the frequency of upstream passage for existing and proposed conditions on a monthly basis for the fish passage criteria discussed.

Figure 7
Frequency of Upstream Fish Passage Criteria Met for Existing and Proposed Conditions



Impact of FRO Flood Operations on Upstream Fish Passage

There are two event types that impact fish passage directly: closure of the hydraulic control gates as a result of flood operations; and surcharge of the conduit inlets due to an increase of flow above 6,000 cfs and transition to inlet control. Results of the fish passage assessment indicate that flood operations would occur six times during five of the 24 years studied (two events in 1990, and one event each in 1991, 1996, 2007, and 2009). Duration of flood operations for the events modeled ranged from a minimum of 27 days to a maximum of 55 days. Average length of flood control events spanned approximately 34 days.

According to the results of the fish passage assessment, surcharge events occurred in 17 of the 28 years studied, and lasted between 0.4 to 5.4 days. Of those events, the time when flows were below 2,200 cfs ranged from less than one day to 3.3 days.

Tables 10, 11, and 12 present the average monthly and annual time fish passage would be limited as a result of flood control operations and surcharge events.

Table 10
Monthly Fish Passage Limitation Due to Flood Control Operations

MONTH	FLOOD RETENTION OPERATION		FLOOD RETENTION OPERATION AND FLOWS BELOW 2,200 CFS		SURCHARGE EVENTS		SURCHARGE EVENTS AND FLOW BELOW 2,200 CFS	
	HOURS	DAYS	HOURS	DAYS	HOURS	DAYS	HOURS	DAYS
January	1,268	52.8	1,105	46.0	59	2.5	0	0.0
February	1,278	53.3	1,121	46.7	62	2.6	0	0.0
March	357	14.9	357	14.9	18	0.8	0	0.0
April	645	26.9	585	24.4	115	4.8	80	3.3
May	0	0.0	0	0.0	0	0.0	0	0.0
June	0	0.0	0	0.0	0	0.0	0	0.0
July	0	0.0	0	0.0	0	0.0	0	0.0
August	0	0.0	0	0.0	0	0.0	0	0.0
September	0	0.0	0	0.0	11	0.5	0	0.0
October	0	0.0	0	0.0	2	0.1	0	0.0
November	181	7.5	131	5.5	220	9.2	71	3.0
December	1,211	50.5	1,093	45.5	136	5.7	0	0.0

Cumulative Impact of Proposed Project on Upstream Fish Passage

The cumulative impact of the project includes hydraulic changes and flood operations associated with the proposed flood retention structure. Flood control operations occurring during all flows in the reach are expected to occur one out of every five years, with potential durations of 27 to 55 days. This limitation of passage includes any naturally occurring events where flows in the reach are greater than 2,200 cfs and fish passage is not possible regardless of flood operations. Not taking into account the high flow events, the total time fish passage would not be available due to flood control operations is approximately 24 to 49 days during an event. Table 11 below presents these results on an annual basis. High fish passage velocities in the reach further reduce time during which fish passage is provided; for potentially 37 to 128 days per year, fish passage is not provided due to velocities greater than 2 fps. Combined with impacts due to flood control, fish passage is estimated to be inhibited from 40 to 131 days per year.

Table 11
Fish Passage Limitation Due to Flood Retention Operations

WATER YEAR	NUMBER OF EVENTS	FLOOD CONTROL OPERATION			FLOOD CONTROL OPERATION & FLOW BELOW 2,200 CFS		
		HOURS	DAYS	PERCENT	HOURS	DAYS	PERCENT
1988	0	0	0.0	0.00%	0	0.0	0.00%
1989	0	0	0.0	0.00%	0	0.0	0.00%
1990	2	2,000	83.3	22.83%	1,768	73.7	20.18%
1991	1	645	26.9	7.36%	585	24.4	6.68%
1992	0	0	0.0	0.00%	0	0.0	0.00%
1993	0	0	0.0	0.00%	0	0.0	0.00%
1994	0	0	0.0	0.00%	0	0.0	0.00%
1995	0	0	0.0	0.00%	0	0.0	0.00%
1996	1	767	32.0	8.73%	670	27.9	7.63%
1997	0	0	0.0	0.00%	0	0.0	0.00%
1998	0	0	0.0	0.00%	0	0.0	0.00%
1999	0	0	0.0	0.00%	0	0.0	0.00%
2000	0	0	0.0	0.00%	0	0.0	0.00%
2001	0	0	0.0	0.00%	0	0.0	0.00%
2002	0	0	0.0	0.00%	0	0.0	0.00%
2003	0	0	0.0	0.00%	0	0.0	0.00%
2004	0	0	0.0	0.00%	0	0.0	0.00%
2005	0	0	0.0	0.00%	0	0.0	0.00%
2006	0	0	0.0	0.00%	0	0.0	0.00%
2007	1*	719	30.0	8.21%	622	25.9	7.10%
2008	1*	88	3.7	1.00%	88	3.7	1.00%
2009	1	721	30.0	8.23%	659	27.5	7.52%
2010	0	0	0.0	0.00%	0	0.0	0.00%
2011	0	0	0.0	0.00%	0	0.0	0.00%
2012	0	0	0.0	0.00%	0	0.0	0.00%
2013	0	0	0.0	0.00%	0	0.0	0.00%
2014	0	0	0.0	0.00%	0	0.0	0.00%
2015	0	0	0.0	0.00%	0	0.0	0.00%
2016	0	0	0.0	0.00%	0	0.0	0.00%

Note:
*Flood operations began in December 2007 and extended into January 2008; the operation during this time is considered a single event.

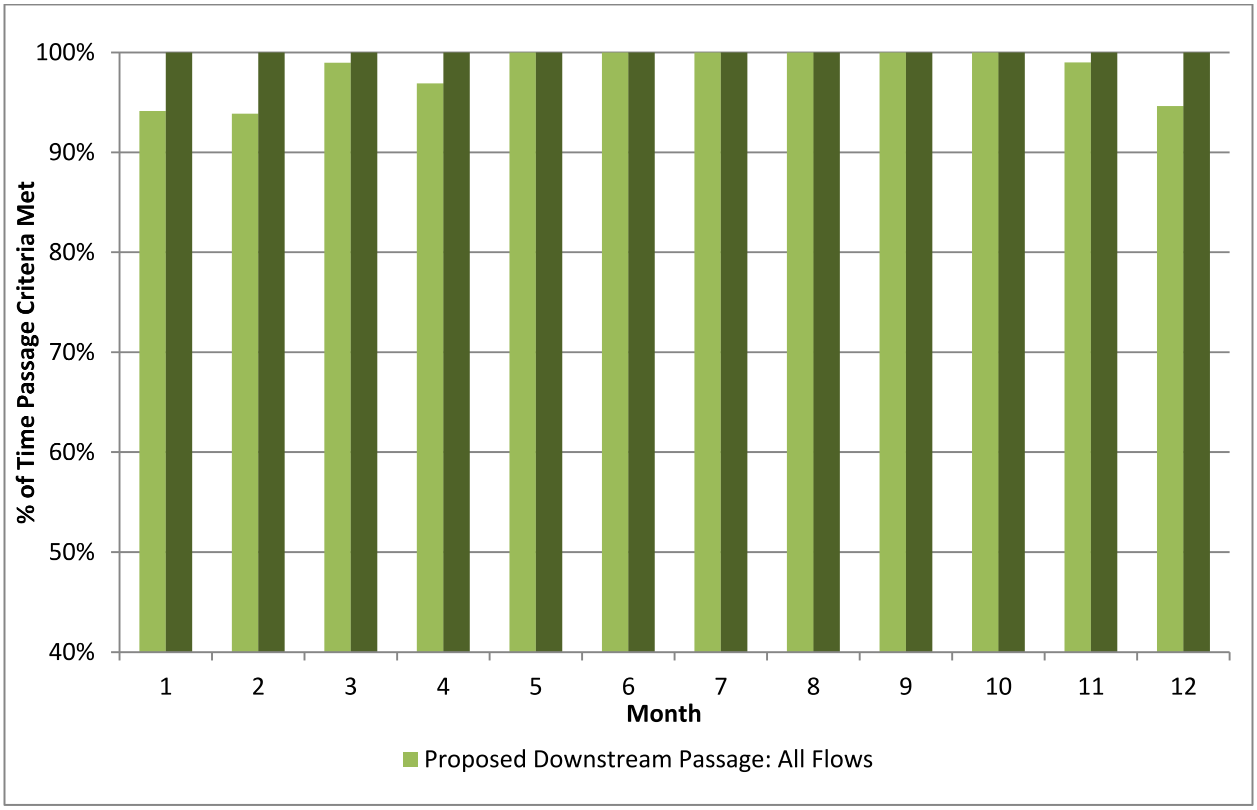
Table 12
Fish Passage Limitation Due to Surcharge Event

WATER YEAR	NUMBER OF EVENTS	SURCHARGE EVENT			SURCHARGE EVENT & FLOW BELOW 2200CFS		
		HOURS	DAYS	PERCENT	HOURS	DAYS	PERCENT
1988	0	0	0.0	0.00%	0	0.0	0.00%
1989	1	9	0.4	0.10%	0	0.0	0.00%
1990	1	1	0.0	0.01%	0	0.0	0.00%
1991	0	0	0.0	0.00%	0	0.0	0.00%
1992	0	0	0.0	0.00%	0	0.0	0.00%
1993	0	0	0.0	0.00%	0	0.0	0.00%
1994	3	37	1.5	0.42%	0	0.0	0.00%
1995	0	9	0.4	0.10%	0	0.0	0.00%
1996	2	130	5.4	1.48%	80	3.3	0.91%
1997	3	17	0.7	0.19%	0	0.0	0.00%
1998	2	31	1.3	0.35%	0	0.0	0.00%
1999	2	60	2.5	0.68%	0	0.0	0.00%
2000	0	0	0.0	0.00%	0	0.0	0.00%
2001	2	47	2.0	0.54%	0	0.0	0.00%
2002	1	23	1.0	0.26%	0	0.0	0.00%
2003	0	0	0.0	0.00%	0	0.0	0.00%
2004	0	0	0.0	0.00%	0	0.0	0.00%
2005	0	0	0.0	0.00%	0	0.0	0.00%
2006	3	74	3.1	0.84%	0	0.0	0.00%
2007	0	0	0.0	0.00%	0	0.0	0.00%
2008	0	0	0.0	0.00%	0	0.0	0.00%
2009	0	0	0.0	0.00%	0	0.0	0.00%
2010	1	2	0.1	0.02%	0	0.0	0.00%
2011	1	2	0.1	0.02%	0	0.0	0.00%
2012	1	105	4.4	1.20%	71	3.0	0.81%
2013	1	11	0.5	0.13%	0	0.0	0.00%
2014	2	11	0.5	0.13%	0	0.0	0.00%
2015	3	54	2.3	0.62%	0	0.0	0.00%
2016	0	0	0.0	0.00%	0	0.0	0.00%

Cumulative Impact of Proposed Project on Downstream Fish Passage

The results of the assessment demonstrated that downstream fish passage is minimally limited by the proposed FRO conduits. Surcharging behind the conduits reaches a depth of 1 atm and prohibits downstream passage on average 2 percent of the time, or for 7.3 days annually. Figure 11 below shows downstream fish passage frequency on a monthly basis.

Figure 8
Frequency of Downstream Passage Available for Existing and Proposed Conditions



Conclusions and Next Steps

The following conclusions were made based upon the results presented in this memo.

- In the HEC-RAS modeling, three-conduit design was shown to more effectively meet the overall cost, complexity, and operational requirements demanded of this project while maintaining the hydraulics of the natural channel to foster fish passage characteristics like those of this specific river reach.
- As indicated by the 1-D steady-state HEC-RAS results for the reach, proposed hydraulic conditions with or without sediment are expected to mimic existing conditions within 90 percent confidence intervals at all cross sections for flows up to 2,000 cfs. Additionally,

proposed velocities only exceed existing velocities in the channel at the sluice mouth and halfway down the splitter wall, and only for flows above 1,750 cfs.

- Proposed conditions meet fish passage flow and velocity criteria at improved or similar frequencies, according to simulated hydraulic and flood operations data. Flows in the proposed reach are more frequently below 2,200 cfs compared with flows in the existing reach; and for flows less than 2,200 cfs, proposed condition velocities were less than 2 fps approximately 75 percent of the time, as opposed to 88 percent of the time under existing conditions.
- Flood operations impact fish passage approximately one of every five years for a potential duration of 27 to 55 days (for all flows), and impact fish passage for a duration of 24 to 49 days when incoming flows are less than 2,200 cfs.
- Fish passage is limited more frequently by flow rates above 2 fps; high velocities occur 37 to 128 days per year. The combined impact to fish passage is estimated to be 40 to 131 days per year. As the FRO structure and fish passage conduit designs are refined, improvements can be made to increase the frequency that proposed conditions meet the fish passage velocity criteria.

Although this preliminary investigation has shown the conduits to have potential for successful fish passage, it is evident that more work must be done to make them viable options for the proposed dam structure. Steps to be taken in the next biennium to improve conditions for fish passage through these conduits include the following.

- Run an unsteady sediment transport simulation for the refined three-conduit design using HEC-RAS to better understand sediment movement within the proposed structure and conduits.
- Utilize a two-dimensional analysis tool to provide example velocity fields for two separate sediment conditions: with sediment and without sediment.
- Refine the proposed conduits to improve conditions within the conduits to better meet current conditions and/or fish passage criteria as described in the methods section of this memo.
- Explore the possibility of implementing a five-conduit design over the three-conduit design presented and discussed in this memo.

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Attachment C
Fish Passage Performance Rationale

TECHNICAL MEMORANDUM

Date: October 7, 2016

To: Fish Passage Technical Subcommittee

From: Fish Passage Design Study Team

Re: Rationale for Development of Performance and Survival Estimates for Anticipated Fish Passage Facilities

Introduction

Two flood damage reduction structures have been identified as potential alternatives for providing flood flow retention and peak flow attenuation in the Chehalis River watershed as part of the Chehalis Basin Strategy process. Implementation of such structures is anticipated to impede upstream and downstream migration of fish passage species and various life-stages of those species existing within the river. Studies in the previous biennium evaluated potential fish passage technologies and developed systems of fish passage facility alternatives to mitigate the potential impacts of such a flood retention structure.

In the current biennium, as part of the efforts to investigate the potential influence that proposed fish passage structures may have on fish populations, fish passage performance and fish survival estimates have been developed. The resulting values will be used as a basis of ongoing Ecosystem Diagnosis & Treatment (EDT) modeling and assessment of the effectiveness of potential fish passage facility alternatives.

In the last biennium the fish passage design team was tasked with estimating the performance and survival of each potential fish passage alternative. At that time, the values were developed by the consultant team based upon a multi-factor rating and scoring system. The evaluation factors included: how accessible the entrance to the fish passage facility were to fish; what is the anticipated attraction and collection efficiency into the facility; what is the anticipated passage performance after fish enter the system; and what is the anticipated survival for the facility as a whole. The resulting values were subjective, based on the rates and scores selected in the matrix rather than on data available directly in the field derived from the operation of like facilities.

In the current biennium, resource agencies participating in the fish passage subcommittee meetings requested that the performance and survival estimates be refined to include data (where available) from existing facilities and lessons learned derived through years of operation. The following document describes the methods, basis of development, and results of the refined performance and survival estimates.

Objective and Purpose

The goal of this activity is to refine the anticipated fish passage performance and survival estimates for the various fish passage facilities being proposed for the Chehalis Basin Strategy process. The resulting values will be used to inform ongoing EDT modeling and fish passage alternative assessments.

This document was originally prepared as a working version for review by the Fish Passage Subcommittee participants. Comments received from the subcommittee were incorporated into the document and redistributed to the subcommittee. This version of the document incorporates all comments received to date and is concurrent with the DRAFT of *Appendix E – Fish Passage Alternative Concept Design*.

Methods

Total survival values are developed for each target species and life stage to be directed through each of the potential fish passage facility alternatives. These values are recorded in Table 1 with the species and life stage along each row and each fish passage alternative along each column. The fish passage alternatives are divided by the two dam alternatives: Flood Retention Only (FRO) and Flood Retention Flow Augmentation (FRFA).

A Total Survival value is the product of the Fish Passage Performance and Fish Survival inputs where:

Total Survival = The total estimated percentage of fish that successfully navigate and survive the proposed fish passage facility and contribute to upstream and/or downstream life histories being considered in the EDT population response modeling.

Performance = The proportion of fish that are anticipated to successfully navigate the fish passage facility.

Survival = The proportion of fish that are not harmed or perish while attempting to navigate the fish passage facility.

Table 1

Fish Passage Facility Performance and Survival Estimate

Fish Species	FRO						FRFA											
	Fish Conduits			CHTR			CHTR			Ladder w Lamp T&T			FSC			Fixed Multi-Port		
	Perf	Surv	Total Surv	Perf	Surv	Total Surv	Perf	Surv	Total Surv	Perf	Surv	Total Surv	Perf	Surv	Total Surv	Perf	Surv	Total Surv
Spring Chinook																		
Adult U/S	0.95	0.99	**94%**	0.93	0.98	**91%**	0.93	0.98	**91%**	0.80	0.99	**79%**						
Juv U/S	0.65	0.99	**64%**	0.55	0.90	**49.5%**	0.60	0.90	**54.0%**	0.00	0.50	**0%**						
Juv D/S	1.00	0.85	**85%**										0.65	0.98	**64%**	0.65	0.99	**64%**
Fall Chinook																		
Adult U/S	0.95	0.99	**94%**	0.93	0.98	**91%**	0.93	0.98	**91%**	0.80	0.99	**79%**						
Juv U/S	0.65	0.99	**64%**	0.55	0.90	**49.5%**	0.60	0.90	**54.0%**	0.00	0.50	**0%**						
Juv D/S	1.00	0.85	**85%**										0.65	0.98	**64%**	0.65	0.99	**64%**
Coho																		
Adult U/S	0.95	0.99	**94%**	0.93	0.98	**91%**	0.93	0.98	**91%**	0.80	0.99	**79%**						
Juv U/S	0.65	0.99	**64%**	0.55	0.90	**49.5%**	0.60	0.90	**54.0%**	0.00	0.50	**0%**						
Juv D/S	1.00	0.85	**85%**										0.65	0.98	**64%**	0.65	0.99	**64%**
Winter Steelhead																		
Adult U/S	0.97	0.99	**96%**	0.93	0.98	**91%**	0.93	0.98	**91%**	0.80	0.99	**79%**						
Adult D/S	0.98	0.75	**74%**										0.50	0.80	**40%**	0.50	0.90	**45%**
Juv U/S	0.80	0.99	**79%**	0.60	0.90	**54.0%**	0.65	0.90	**58.5%**	0.00	0.50	**0%**						
Juv D/S	1.00	0.95	**95%**										0.65	0.98	**64%**	0.65	0.99	**64%**
Coastal Cutthroat																		
Adult U/S	0.93	0.99	**92%**	0.55	0.98	**54%**	0.88	0.98	**86%**	0.70	0.99	**69%**						
Adult D/S	0.98	0.80	**78%**										0.65	0.80	**52%**	0.65	0.9	**59%**
Juv U/S	0.65	0.99	**59%**	0.50	0.90	**45.0%**	0.60	0.90	**54.0%**	0.00	0.50	**0%**						
Juv D/S	1.00	0.85	**85%**										0.65	0.98	64%	0.65	0.99	**64%**
Pacific Lamprey																		
Adult U/S	0.97	0.99	**96%**	0.00	0.90	**0.0%**	0.60	0.90	**54.0%**	0.60	0.9	**54%**						
Juv D/S	1.00	0.95	**95%**										0.03	0.10	**0.3%**	0.03	0.20	**0.6%**
Western Brook Lamprey																		
Adult U/S	0.97	0.99	**96%**	0.00	0.90	**0.0%**	0.60	0.90	**54.0%**	0.60	0.9	**54%**						
Juv D/S	1.00	0.95	**95%**										0.03	0.10	**0.3%**	0.03	0.20	**0.6%**

Values for Fish Passage Performance and Survival are assigned based upon a three tier system relative to the level of information that is available for a given fish passage technology. The three tier system is listed in order of priority and described as follows:

1. Values are derived from available performance data and information originating from existing fish passage facilities similar in nature and application as the proposed fish passage alternative.
2. Values are derived from data and information originating from existing fish passage facilities with less similarity and/or application as the proposed fish passage alternative. Some uncertainty exists and the value is adjusted using professional judgment.
3. Data and information from existing fish passage facilities are not available. Values are derived primarily from professional judgment. A higher level of uncertainty exists so selected values are conservative, meaning less performance is anticipated.

All data sources, caveats, limitations, and uncertainties are briefly summarized in the Considerations section of this Technical Memorandum.

Definitions

The following abbreviations are used in Table 1 and are defined below:

FRO	The Flood Retention Only concrete dam alternative which operates in a run-of-river condition for the majority of its operation. Gates are closed and the Conduits are blocked periodically to attenuate flood events which are anticipated to cause property damage downstream.
Fish Conduits	Fish passage conduits which are the primary means of upstream and downstream fish passage when the FRO gates are open.
CHTR	Collect Handle Transfer Release fish passage alternative. Also referred to as “Trap and Transport.” Upstream migrating fish are collected downstream of the dam and transported to a release point in the river located upstream of the dam.
FRFA	The Flood Retention Flow Augmentation concrete dam alternative which is operated to maintain a conservation pool during normal flow conditions. A reserve volume of storage is kept available above the conservation pool to periodically attenuate flood events which are anticipated to cause property damage downstream. During normal operation, water control outlets can be used to optimize downstream instream flow temperatures and water quality.

Ladder with Lamp Trap &Transport	A conventional fish ladder configured to achieve volitional passage of target fish species from the FRFA dam stilling basin to the water surface in the reservoir. The ladder includes an auxiliary water supply at its entrance to improve passage effectiveness, as required by National Marine Fisheries Service criteria, and has a multi-outlet fish exit structure in the reservoir to accommodate a 40-foot reservoir fluctuation. The 40-foot reservoir fluctuation corresponds to the current anticipated normal operating range of conservation pool operating levels. A separate entrance off of the fish ladder entrance is provided for lamprey, directing them to a separate holding tank where they are trapped and transported upstream.
FSC	The Floating Surface Collector fish passage alternative which would be located in the reservoir, near the Water Quality Control Works, of the FRFA dam alternative. A floating surface collector floats in the reservoir and collects downstream migrants throughout the anticipated range of normal operating levels in the conservation pool. Collected migrants may be transported to a handling facility where they can be evaluated and released downstream or directly to a release point in the river downstream of the dam.
Head Res	The Head of Reservoir fish passage alternative consisted of one or more instream juvenile out-migrant collection facilities located at the head(s) of the reservoir where major tributaries met the flood pool. This alternative was superseded and replaced with a fixed multi-port outlet with passive bypass by decision of the Fish Passage Subcommittee. Values for this alternative are not provided in Table 1, as this alternative was dropped from evaluation.
Fixed Multi-Port	The Fixed Multi-Port Collector fish passage alternative is a passive out-migrant collection system located in the reservoir, near the upstream face of the FRFA dam. The Fixed Multi-Port Collector collects and routes fish through a passive, pressurized bypass pipe to a release location downstream of the stilling basin.

Considerations

The following bulleted list provides a brief summary of considerations generated in the development of the fish passage performance and survival scores. The list is separated first by dam alternative, then by fish passage alternative, and finally by fish species or life stage, as required.

Flood Retention Only Dam Alternative

Fish Passage Conduits

There are no known conduits similar in nature that are also used for fish passage that could assist in anticipating fish passage performance and survival to the FRO Conduits. The likely surrogate for a technology of this nature is fish passage through culverts which has been studied extensively. In addition, some studies at Mud Mountain Dam provide information on the success of out-migrant fish through similar conduits. Performance and survival through the FRO Conduits is based the success of out-migrants at Mud Mountain Dam and the performance of a similar technology – culverts – and is adjusted based on conditions that are unique to the FRO dam on the Chehalis River. The following is a summary the information collected regarding the potential performance of conduits similar to the FRO Conduits:

- In general, there are few examples of conduits through dams which are configured for the purpose of fish passage. No known conduits of this nature have been identified in a similar situation for the purposes of upstream passage. The likely surrogate for a technology of this nature would be fish passage through culverts which has been studied to a high level of detail over the past several decades. Available information for culvert fish passage exists with regard to design rationale, guidelines, and velocity targets. Available information suggests that passage through long conduits of this nature can be accommodated to a high degree of performance when velocity and depth criteria are met.
- Design guidelines are readily available for adult salmonid upstream passage. Guidelines and swim capabilities for juvenile upstream passage can be derived from the literature but formal design guidelines are not available.
- Mud Mountain Dam provides information on the success of routing out-migrant fish downstream through a similar type of conduit. Information available suggests that the performance levels and survival for out-migrating juveniles is high as long as velocity criteria are met and the conduit is kept clear of debris and free from sharp edges protruding into the water column. Mud Mountain Dam is located on the White River in Washington State.
- A review of literature characterizing the results of barotrauma studies suggests that out-migrants can be passed through partially open radial gates or open valves with a high level of survival when fish are being passed downstream at 1 atmosphere or approximately 34 feet of static water depth. Survival was documented to incrementally decrease as depth and/or pressure increased or as valve openings decreased.

A number of factors specific to the FRO dam, this location, and the Chehalis River adjust the expected performance and survival for this alternative from the information gathered for similar conduits. Some of these factors include:

- It is assumed that the performance and survival values are provided for periods of time when the fish passage conduits are open. The CHTR facility would be operated when flood retention gates are closed. Performance and survival would then default to those values provided for CHTR during periods of flood retention.
- Passage performance is largely a function of the engineering design and capability to provide adequate depths and velocities.
- Roughness elements are planned in the larger center conduit which would provide a corridor of water velocity suitable for juvenile upstream migration more often.
- Larger adult salmonids were given a higher level of performance and survival for upstream migration than juveniles given that hydraulic criteria for juvenile fish would be met less often.
- Juvenile salmonids were also given a high level of performance for out-migration. Survival for out-migrating juveniles was lowered slightly due to the potential interaction with the upstream trashrack. It was assumed that if debris loading occurs, juvenile fish would be more susceptible to being swept into a debris laden trashrack which may cause more injury or mortality.
- Inlet and outlet conditions are anticipated to impact juvenile survival during downstream migration through hydraulics, predation, and other factors. Therefore, weaker swimming fish such as cutthroat trout, Chinook, and coho have slightly lower survival rates than those of steelhead.
  - Out-migrating post-spawn adult steelhead are less energetic and possibly more susceptible to injury during downstream migration and are also given a slightly decreased survival.
  - Juvenile winter steelhead are less dependent upon the hydraulic fringe. They generally exhibit a larger size and better swimming ability which makes them more capable of ascending the conduit.
  - Juvenile steelhead are more capable of handling the varied hydraulic conditions in the conduit as well as other factors at the inlet and outlet. Predation less of a factor than for other species.

Capture, Handling, Transport, and Release

The anticipated fish passage performance and survival is based on the performance of other CHTR facilities and is adjusted based on conditions that are unique to the alternative proposed for the FRFA and FRO dams on the Chehalis River. There are numerous examples of trap and transport facilities in the Pacific Northwest that collect and transport adult anadromous salmonids with high levels of performance and with very low levels of injury or direct mortality. Following are a few examples which were used as a basis of comparison:

- Merwin Dam Adult Collection Facility –Lewis River, Washington State
- North Fork Adult Sorting Facility –North Fork Clackamas River, Oregon State
- Lower Baker Adult Collection Facility –Baker River, Washington State

- Cougar Dam Adult Collection Facility –South Fork McKenzie River, Oregon State
- Cowlitz Adult Collection Facility – Cowlitz River, Washington State
- White River Diversion Dam Adult Collection Facility – White River, Washington State
- Minto Adult Collection Facility – North Santiam River, Oregon State
- Foster Fish Collection Facility – South Santiam River, Oregon State

A number of factors specific to the FRFA and FRO dams, this location, and the Chehalis River adjust the expected performance and survival for this alternative compared to other similar facilities. Some of these factors include:

- Modern adult collection facilities are typically designed for the collection of adult upstream migrating salmonids. Juvenile collection during upstream migration has historically been incidental and therefore limited data exists.
- There is a higher level of confidence that a CHTR facility will perform well for upstream migrating adult Chinook, coho, and steelhead. Higher performance and survival values close to typical compliance standards for like facilities such as 90% and 98%, were provided.
- Reduced performance and survival values were provided for cutthroat trout compared to adult Chinook, coho, and steelhead. The reduced performance value is most substantially attributed to the general focus of other facilities on adult salmonids and the lack of data on cutthroat trout collection.
- Upstream migrating juvenile fish were given a lower performance and survival values than those for adults. These values accommodate the uncertainty of attracting fish into a ladder entrance, predation, and motivation to ascend the ladder into a holding gallery. Additional provisions could be engineered into a facility of this nature to improve juvenile fish collection and safe transfer such as multiple low-head entrances, lower head differential between pools, and segregation zones in holding galleries to decrease predation. Such provisions should be explored by the fish passage subcommittee during preliminary design. However, changes made to improve juvenile fish collection may adversely affect adult fish collection and should be evaluated accordingly.
- The reduced size, motivation, and swimming capability for juvenile cutthroat trout resulted in lower juvenile upstream passage performance and survival values compared to those associated with adult upstream passage.
- Since lamprey passage through such facilities has not historically been a focus, limited data are available. Additional research is needed. One source of data may be ongoing efforts on the Mid-Columbia and Yakima Rivers but these have not been investigated. This information can be used to inform revisions to the table as data becomes available. Given the high level of uncertainty associated with collection of lamprey in a trap and transport situation, lower performance and survival values were provided.

Flood Retention Flow Augmentation Dam Alternative

Collect, Handle, Transport, and Release (CHTR)

The anticipated fish passage performance and survival for the Collect, Handle, Transport, and Release (CHTR) alternative associated with the FRO dam is identical to the CHTR for the FRFA dam. The basis for the anticipated performance and survival, as well as the project-specific factors that adjust the expected performance and survival compared to other similar facilities are also identical to the CHTR for the FRFA dam.

Volitional Passage (Technical) Fish Ladder

The anticipated fish passage performance and survival is based on the performance of other volitional fish ladders and adjusted based on conditions that are unique to the alternative proposed for the FRFA dam on the Chehalis River. The following is a summary the information collected regarding the potential performance of fish ladders similar to the Volitional Passage (Conventional) Fish Ladder:

- There are limited examples of volitional passage fish ladders in the Pacific Northwest designed for hydraulic heads greater than 150 feet or volitional passage fish ladders that accommodate reservoir fluctuations at the ladder exit of 30-40 feet.
- Mid height ladders up to 150-ft tall on the Columbia River show a high level of passage performance for adult anadromous salmonids and bull trout.
- The North Fork Fish Ladder is an example still in operation that extends over 2 miles and ascends a height of 240 feet.
- The North Fork ladder passes from 95 to 100 percent of adult Chinook and steelhead. Bull trout have also shown a high level of success but are not the current focus of monitoring efforts so less data is available.
- The North Fork Fish Ladder is an example still in operation that extends over 2 miles and ascends a height of 240 feet. The North Fork ladder passes from 95 to 100 percent of adult Chinook and steelhead. Bull trout have also shown a high level of success but are not the current focus of monitoring efforts so less data is available. The 2-mile long fish ladder at North Fork performs at levels higher than other examples of its kind due to the following reasons:
    - The ladder entrance accommodates up to 280 cfs of attraction flow through multiple entrances using an AWS originating from the Faraday Diversion Dam,
    - The water temperature and water quality composition originating from the AWS are identical to the river,
    - The reach between North Form Dam and Faraday Diversion Dam is not subject to high levels of thermal gain,
    - North Fork Reservoir exhibits limited thermal stratification due to its narrow configuration and hydropower operations; therefore, the water entering the ladder exit is similar to that of the river to begin with, and

  - Cool water is injected into the ladder at two additional downstream locations which further improves fish attraction during portions of the year.

A number of factors specific to the FRFA dam, this location, and the Chehalis River adjust the expected performance and survival for this alternative from that of similar facilities. Some of these factors include:

- The ladder performance and survival values include fish entering the ladder entrance, release into the reservoir, and passage through the reservoir.
- Reservoir transit introduces some level of uncertainty and will require site specific verification.
- Loss of migration cues at the reservoir through flow modification may influence the timing of upstream migrants.
- The ladder is anticipated to perform at high performance values at or above 0.8 with survival values of 0.99 for adult Chinook, steelhead, and coho. The value of 0.8 was selected to accommodate uncertainty of reservoir transit.
- Slightly lower performance is anticipated for bull trout and cut throat trout given potential issues with attraction into the ladder entrance as well as the expenditure of energy and motivation required by smaller fish. However, pit tagged bull trout have been documented to ascend the North Fork Ladder in as few as 10 to 12 hours. The lower performance value of 0.7 was selected to accommodate uncertainty of reservoir transit.
- Little is known about the motivation of juvenile fish to ascend a high ladder such as this. Given this uncertainty and the likelihood that the ladder would be optimized for adult salmonids, a low performance and survival value of 0 and 0.5, respectively, were selected.
- It is assumed that provisions to improve lamprey passage would be incorporated into the ladder entrance, baffles, floor, and walls. However, given the lack of data available relative to lamprey passage in high dam facilities, low performance (0.1) and high survival (0.99) were selected.

Floating Surface Collector

The anticipated fish passage performance and survival is based on the performance of other floating fish collection facilities currently in operation at other locations and adjusted based on assumptions related to the conditions that are unique to a proposed FRFA dam on the Chehalis River. Existing facilities used to inform selection of performance and survival values similar to the proposed alternative are summarized in Table 2.

Table 2
Summary of FSCs Currently in Operation Used to Inform Design and Performance

FACILITY	OWNER - LOCATION	VERTICAL FLUCTUATION	ATTRACTION FLOW	FISH TRANSPORT	1ST YEAR OF OPERATION
Upper Baker	PSE - Baker River, WA	30 to 60	500/1,000	Trap and transport	2008
Lower Baker	PSE - Baker River, WA	30 to 60	500/1,000	Trap and transport	2013
Swift	PacifiCorp -Lewis River, WA	100	500/750	Trap and transport	2012
North Fork	PGE - Clackamas River, WA	2 to 10	600/1,000	7-mile long bypass conduit	2015
Cushman	Tacoma Power - Skokomish River, WA	20	250	Trap and transport	2015

The following are assumptions made regarding the performance and survival of fish passage through a Floating Surface Collector at the proposed FRFA dam on the Chehalis River:

- The selected performance value is intended to accommodate fish passage from the head of reservoir to the point of release downstream of the dam.
- It is assumed that collection performance after fish have entered the zone of influence can be achieved in excess of 0.9 with the implementation of adequate guidance nets, lead nets, and attraction flow (Upper Baker, Lower Baker, and North Fork FSCs).
- Swift and Cushman have reported values lower than 0.4 due to reservoir temperature, stratification, circulation, and fish transit issues.
- The success of reservoir transit at this location is speculative and will likely require site specific studies after implementation has occurred to determine actual performance. All values were therefore reduced from the expected facility performance to accommodate for this uncertainty in the proposed reservoir.
- Like facilities of this nature typically accommodate survival rates meeting or exceeding a value of 98%.
- Juvenile lamprey passage performance and survival values are anticipated to be low given the uncertainty and lack of data available for lamprey and use of this type of fish passage technology. Although values may be higher than 0.3% in practice, the selected value remained low to be conservative and represent an appropriate level of uncertainty.

Passive Multi-Port Outlet with Passive fish Bypass

The anticipated fish passage performance and survival is based on the performance of other fixed fish collection facilities and adjusted based on assumptions related to the conditions that are unique to a proposed FRFA dam on the Chehalis River. There are a number of fixed collectors bypass facilities that can be used to inform selection of performance and survival values but are not identical to the proposed alternative. Examples are:

- River Mill Fixed Collector and bypass
- Soda Springs Bypass Facility
- Cowlitz Falls Fixed Collector and bypass
- The Cle Elum Dam Multi-Port Fixed Collection Facility with helical bypass
    - The Cle Elum collector is currently under construction. Results from physical modeling results are available through Reclamation but only provide information on the design of the helical bypass and do not provide insight into the performance of the collection ports.

The following are assumptions made regarding the performance and survival through Multi-Port Collector at the proposed FRFA dam on the Chehalis River:

- The selected performance value is intended to accommodate fish passage from the head of reservoir to the point of release downstream of the dam.
- It is assumed that with adequate configuration in the dam face and sufficient attraction flow, each individual collection port should perform as well as the fixed and floating surface collectors currently in operation with performance values in excess of 0.9 similar to River Mill and Soda Springs facilities.
- The success of reservoir transit is speculative and will likely require site specific studies after implementation has occurred to determine performance. All values were therefore reduced from the expected facility performance to accommodate for this uncertainty in the reservoir.
- Survival values were slightly increased from those given for floating surface collectors to accommodate use of a passive bypass system.
- Lamprey passage performance and survival values are anticipated to be low given the uncertainty and lack of data available for this type of technology.

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Attachment D
Fish Passage Facility Cost Data

TECHNICAL MEMORANDUM

Date: February 22, 2017
To: Flood Damage Reduction Technical Committee
From: Fish Passage Design Study Team
Cc: Michael Garello (HDR)
Re: Chehalis Basin Dam Fish Passage Alternatives – Conceptual Design Construction Cost Opinion

Executive Summary

Hard and soft costs associated with construction of each of the five fish passage alternatives were developed and are stated herein. The fish passage alternatives include a capture, handle, transport, and release facility (CHTR) for the flood retention only dam, and a CHTR, conventional fish ladder, floating surface collector, and fixed multi-port collector for the flood retention flow augmentation dam. Total costs for each alternative are summarized in Table 1 below in present value (2016) dollars. The costs presented in this technical memorandum reflect order-of-magnitude cost opinions that were developed in 2016. The costs presented herein represent a Class V cost opinion, as defined by the American Association of Cost Engineers International with a margin of error of L: -25% H: +50%. A summary of this cost opinion is also presented in the Final version of *Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species, Combined Dam and Fish Passage Conceptual Design Report* as well as its appendix, *Appendix G – Fish Passage Alternative Concept Design*.

Table 1
Summary of Construction Costs Based on July 2016 Order-of-magnitude Cost Opinion

FISH PASSAGE ALTERNATIVE	LOWER BOUND COST ($ MILLION)	MIDDLE COST ($ MILLION)	UPPER BOUND COST ($ MILLION)
FRO DAM			
Upstream: FRO Conduits	Integral to dam construction		
Upstream: CHTR Facility	$13.8	$18.4	$27.6
FRFA DAM			
Upstream: CHTR Facility	$13.8	$18.4	$27.6
Upstream: Conventional Fish Ladder	$49.1	$65.4	$98.1
Downstream: Floating Surface Collector	$62.0	$82.6	$123.9
Downstream: Fixed, Multi-Port Collector	$80.0	$106.6	$159.9

Notes:
CHTR = capture, handle, transport, and release
FRFA = flood retention flow augmentation
FRO = flood retention only

Project Background

The Chehalis Basin has historically been prone to flooding. The economic damages of the 2007 flood alone were estimated at more than $900 million, with one-third of that damage coming from disruption and damage to the transportation system, including Interstate 5 (I-5), other state highways, and rail lines. Many different flood hazard mitigation projects and approaches have been proposed and studied in response to the major floods in the Chehalis Basin. After the 2007 flood, the Chehalis River Basin Flood Authority was created to focus on developing flood hazard mitigation measures throughout the basin and to identify and implement flood damage reduction projects. The Chehalis River Basin Flood Authority has been studying water retention in the upper Chehalis River Basin along with smaller flood hazard mitigation projects in the lower portion of the basin.

In 2011, the Washington State Legislature required the Office of Financial Management to prepare a report on alternative flood damage reduction projects and – in coordination with tribal governments, local governments, and state and federal agencies — to recommend priority flood hazard mitigation projects for continued feasibility assessment and design work. In response to the legislative direction, the Ruckelshaus Center published the Chehalis Basin Flood Hazard Mitigation Alternatives Report in December 2012. That report compiled existing information on the potential flood hazard mitigation projects that seemed of most interest to Basin leaders and decision makers at that time. Potential flood hazard mitigation benefits, adverse impacts, costs, and implementation issues were summarized for each project to the degree that such information was available. Along with that effort, the Chehalis Basin Work Group, composed of Chehalis Basin leaders, recommended to then Washington Governor Christine Gregoire a series of actions that, taken together, would represent a significant investment to reduce flood damage, enhance natural floodplain function and fisheries, and put Basin leaders on firm footing to make critical decisions about large- scale projects. The Chehalis Basin Work Group recognized that habitat loss in the Basin has contributed to a reduction in native fish populations and set the goal to develop a Basin-wide strategy to integrate flood damage reduction and environmental enhancement.

The Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project is evaluating the feasibility of mitigating flood hazards within the Basin while exploring opportunities to enhance ecological conditions, aquatic habitat, and the abundance of fish in the Basin. The scope of the project has included studying alternative water retention structures (dams), options for protecting I-5 with or without a dam, and other small flood reduction projects throughout the Chehalis Basin with or without a dam. Based upon further study, the viable options to accomplish the project objectives have narrowed. Those options include a water retention facility at the proposed Chehalis dam site.

The technical feasibility and estimated costs of alternate dam and fish passage configurations at the proposed dam site were documented in HDR's 2014 Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR 2014). The memorandum also provided recommendations on additional site characterization and engineering evaluations that would be required to reduce design uncertainty, refine estimated project costs, and support selection of a preferred alternative. Additional evaluations completed during 2015-2016 have narrowed the dam to two options – flood retention only (FRO) and flood retention flow augmentation (FRFA). State and federal agencies require upstream and downstream fish passage to be provided for both dam options. The dam options are described below. Fish passage alternatives are described in the Scope of Work section.

Flood Retention Only Dam

The FRO concrete dam alternative operates in a run-of-river condition for the majority of the year. Fish passage/flood regulation conduits are located at the thalweg of the river and allow flow to pass through the dam. The conduits are a fish passage and flood regulation system intended to provide a route for adult salmon and steelhead, resident fish, and lamprey to volitionally pass upstream and downstream of the FRO dam as well as allow flood events to be retained and flows downstream to be metered for flood control. During high flow periods, water is impounded behind (upstream of) the dam, the conduit gates are partially closed (throttled) to reduce flows in the river, and fish passage through the conduits is not anticipated. Flow through the conduits is reduced and controlled to attenuate flood events that are expected to cause property damage downstream. When the conduits are throttled, upstream fish passage is provided via the capture, handle, transport, and release (CHTR) facility.

Flood Retention Flow Augmentation Dam

The FRFA concrete dam alternative is operated to maintain a conservation pool during normal flow conditions. A reserve volume of storage is kept available above the conservation pool to periodically attenuate flood events that are expected to cause property damage downstream. During normal operation, the water quality control outlets draw water from different levels of the storage pool to optimize downstream instream flow temperatures and water quality. The storage pool precludes fish passage through conduits in the dam, similar to the FRO dam, so separate upstream and downstream fish passage facilities are required.

Scope of Work

This concept design cost exercise involves preparing budgetary costs associated with one upstream fish passage alternative for the FRO dam, two upstream fish passage alternatives for the FRFA dam, and two downstream fish passage alternatives for the FRFA dam. Short summaries of each of these alternatives are provided in the sections below:

FRO Dam – Upstream Alternative

Capture, Handling, Transport, and Release Facility

The primary means of upstream and downstream passage at the FRO dam is via the conduits. When water is impounded behind the FRO dam during high flow events, the conduits are closed, and fish passage is provided via a CHTR. The CHTR is a fish passage alternative intended to collect migrating adult salmon and steelhead moving upstream and safely transport them upstream of the FRO dam. Though not designed specifically for upstream migrating juvenile salmon and steelhead, the facility does not exclude them.

The CHTR consists of a short fish ladder, fish lift, holding galleries, sorting stations, and transportation. Fish enter the CHTR through the stilling basin, where they are attracted to the flow from the fish ladder entrance. Flow from the fish ladder guides them through the entrance pool, up the fish ladder, and into a fish trap and holding area. Inside the holding area they are crowded into a hopper, which is lifted to a flume. After they are released in the flume, the fish are sorted by size through bar grating and diverted to holding galleries. When released from the holding galleries, the fish move to sorting stations where they can be manually sorted and examined or simply passed through to transport tanks. Fish are transferred from the transport tanks directly to transport trucks. The trucks are driven above the dam to predetermined release sites in the reservoir or on Chehalis River or one of its tributaries. The CHTR is a staffed facility that is operated, on average, once every several years, during flood events.

FRFA Dam – Upstream Alternative

Capture, Handle, Transport, and Release Facility

The CHTR for the FRFA dam is identical to the facility for the FRO dam, except the juvenile holding gallery is capable of receiving juvenile fish transported downstream from the floating surface collector, and a separate ramp, holding facility, and transport hopper for lamprey are provided adjacent to the fish ladder. The CHTR is a staffed facility that is operated year-round.

Conventional Fish Ladder

The conventional fish ladder is a fish passage alternative intended to provide a route for adult salmon and steelhead to volitionally pass over the FRFA dam. The conventional fish ladder consists of a fish ladder entrance with auxiliary water supply at the stilling basin, a 2,900-foot-long half-ice harbor fish ladder, and a 41-gate fish ladder exit into the reservoir. Water is supplied to the fish ladder through one of the fish ladder exit gates. Auxiliary attraction water is provided to the fish ladder entrance via a pipeline connected to the water quality outlet pipes. A separate ramp, holding facility, and transport hopper for lamprey are provided adjacent to the fish ladder.

FRFA Dam – Downstream Alternative

Floating Surface Collector

The floating surface collector alternative is intended to float on the surface of the reservoir near the dam and collect and safely pass out-migrating adult steelhead, juvenile salmonids, and juvenile and adult resident fish from the FRFA reservoir to a designated release point downstream of the FRFA dam structure. Attraction flows of 500 to 1,000 cubic feet per second are generated at the entrance to a large floating collection barge, which creates a positive flow-net that reaches out into the reservoir. Fish entering this “zone of influence” are triggered to move downstream toward the artificial reservoir outlet. Fish are guided from the reservoir into the entrance of the collection barge with the assistance of guidance and lead net systems that span the full width and depth of the reservoir. Fish entering the floating collection barge are moved via transport vehicle downstream of the dam to a handling and sorting facility where they can be examined and evaluated. They can be transported from the handling and sorting facility, or directly from the floating surface collector, and released into the Chehalis River downstream of the dam. The floating surface collector system will operate year-round when reservoir elevations are within the anticipated normal operational range of water surface elevation (WSEL) 588 to 628.

Fixed, Multi-Port Collector

The fixed multi-port fish collector (multi-port collector) is a fish passage alternative intended to collect downstream juvenile migrants, resident fish, and post-spawn adult steelhead and pass them safely downstream of the FRFA dam. The multi-port collector consists of four sets of vee-type dewatering screens, each staggered 10 feet vertically to provide downstream collection throughout the full range of normal reservoir fluctuation, from WSEL 628 to WSEL 588. Low-head, high flow pumps pull 500 to 1,000 cubic feet per second of attraction water through the fish screens into a pump plenum common to all four sets of screens. Water drawn into the screens creates attraction for aquatic species moving downstream. A small amount of water remains in the channel at the end of the screens and carries species over a weir and into a bypass conduit. The bypass conduit carries species under pressurized flow downstream and through the dam at velocities faster than they can escape. On the downstream side of the dam, the bypass conduit transitions to open channel flow and enters a consolidation structure where the individual bypass pipes from each fish screen transition to a single pressurized conduit that carries the species downstream. The downstream moving species are returned safely to the Chehalis River downstream of the dam via an outfall structure. The outfall structure discharges into a deep, swift-moving pool in the river channel, where the aquatic species can continue their migration or volitional movement.

Project Schedule

A preliminary project schedule has been developed by the Dam Design and Fish Passage Design Study Teams. The schedule assumes design and permitting will take roughly 8 years. If final design were to start in the beginning of 2017, this would lead to completion at the end of 2024. Construction of the fish passage alternatives is assumed to occur concurrently with construction of the dam. Each fish passage alternative is expected to have a different construction schedule based on the facilities to be constructed, their location on the site, and the level of complexity. Construction of any of the alternatives is not expected to last less than 1 year or more than 2 years.

Method of Accomplishment

It is assumed that the selection of the general contractor will be through a Federal procurement process. It is assumed that construction of the dam and fish passage facilities will be performed by the same general contractor, but that the general contractor will choose separate staff, dedicated staff, or a subcontractor to lead construction of the fish passage facilities. The heavy/civil general contractor will self-perform the site work and earthwork, deep foundations and associated dewatering and concrete work. In accordance with the Federal procedures, subcontractors will be secured through a competitive bid process for the buried piping, trap and haul facility building, above-grade mechanical piping, fish screens and associated mechanical equipment, marine vessels, and all electrical and industrial/commercial equipment. Equipment specific to the trapping and hauling of fish will be procured through a competitive vendor solicitation process. The successful vendors will provide packages that will be complete for their respective scopes of work. The general contractor will operate with site office staff necessary to manage the work in conjunction with the necessary home office support.

The contractor will provide competent, suitably qualified personnel to survey and lay out the work and perform construction in accordance with the contract documents. The contractor shall at all times maintain good discipline and order at the site as required for the safety or protection of persons, property, and the environment. All work at the site shall be performed during regular working hours, 8 hours per day, Monday through Friday. This cost opinion does not include any shiftwork, overtime, or holiday/weekend work.

The contractor shall supervise, inspect, and direct the work competently, and will be solely responsible for the means, methods, techniques, sequences, and procedures of construction. The contractor will provide coordination and overall project oversight for the startup and commissioning of all mechanical and electrical systems. The contractor will hire a qualified, licensed naval architect, directly or through the subcontractor responsible for construction of naval vessels, to provide support during construction, in development of the launch plan, during launching of all vessels, and during weight and balancing of the vessels during startup.

Financial Data

The costs provided herein take into account price escalation over the course of these multiyear projects. All costs presented are net present value for 2016.

Cost Opinion Assumptions and Exclusions

Assumptions

The following assumptions were made in development of this cost opinion:

- Unit prices are for material and equipment, installed, including costs associated with the contractor's:
  - Overhead
  - Procurement
  - Profit

- Bond
- Insurance
- Business and Occupation Tax (B&O)
  - B&O tax is not an additional tax related to performing the work or obtaining materials, as is a sales tax. B&O tax is a necessary cost of doing business in the State. It is assumed the contractor already performs work in the State and the B&O tax would not be a unique requirement of these projects. The B&O tax is considered part of the contractor's overhead. It is broken out separately from the contractor's overhead herein for informational purposes.

- The projects will not be exempt from state and local sales tax.
- It is assumed that high-voltage power service will be extended from the existing power grid to supply facilities related to the dam. Power requirements for fish passage facilities are in excess of those included in the dam costs. Costs for the fish passage alternatives include costs associated with extending additional high-voltage power service from the existing power grid to power the fish passage facilities.

Exclusions

This cost opinion excludes the following:

- All permits, regulatory fees, environmental fees, or requirements and acquisition of such
- Any work related to hazardous materials or waste
- Any rock excavation beyond assumed estimate quantities
- Communications or security systems
- Costs associated with temporary or permanent utility service for construction power

Cost Opinion Methodology

The estimate methodology is primarily based on appraisal of the conceptual design documents produced by HDR. Costs from similar projects were utilized and indexed accordingly. RS Means costing software and historical data were used where applicable. Material quantity takeoffs were performed using Revit, Navisworks, and Bluebeam software. Costs for the fixed multi-port collector were developed via a parametric estimate considering projects of similar size and scope, then scaling the cost based on key, representative design factors. The Opinion of Probable Construction Cost was prepared using Microsoft Excel spreadsheet methods in general accordance with Association of Cost Engineers International guidelines.

Classification & Level of Confidence

Based on the level of project definition and budgetary material pricing this is a Class V estimate as determined by Association of Cost Engineers International guidelines. The margin for error for this estimate is L: -25% H: +50%. Based on the guidelines, this estimate would normally include a 25 percent contingency. However, based on the site conditions, limited site access, and associated difficulties, this estimate includes a 30 percent contingency.

References

HDR, Inc., 2014. C*ombined Dam and Fish Passage Alternatives Technical Memorandum.* Prepared for the State of Washington. October.

HDR, Inc., 2016. *Draft Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species, Fish Passage Alternatives, Figures FRO-C-1 through FRO-C-4 and FRFA-C-1 through FRFA-C-13.* Prepared for the Chehalis Basin Workgroup. December 14.

HDR, Inc., 2016. *Draft Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species, Appendix E – Fish Passage Alternative Concept Design Report.* Prepared for the Chehalis Basin Workgroup. December 14.

Attachment E
Floating Surface Collector Technical Memorandum

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Technical Memorandum

Date : 2016-Sep-13
AAA Ref : CFHDR007.001-TM003
Client Ref :

To : File
Cc :
From : Conor Shannon
Subject : CHEHALIS FLOATING FOREBAY COLLECTOR CONCEPT

REFERENCES

[a] CFHDR007.001-TM001
[b] CFHDR007.001-TM002
[c] Chehalis – HDR FSC info for AAA
[d] LightShipEst.xlsx (Dated 31 Aug 2016)
[e] Dwg CS-806-01 CONCEPT SKETCH MAIN DECK ARRANGEMENT
[f] Dwg CS-806-02 CONCEPT SKETCH LOWER DECK ARRANGEMENT
[g] Dwg CS-806-03 CONCEPT SKETCH INBOARD PROFILE
[h] Dwg CS-806-04 CONCEPT SKETCH MAIN DECK ARRANGEMENT WITH NTS
[i] Dwg CS-806-03 CONCEPT SKETCH INBOARD PROFILE WITH NTS
[j] Dwg CS-806-03 CONCEPT SKETCH INBOARD PROFILE WITH NTS AND DAM

This memorandum is an update to the concept design discussed in reference [a] and [b]. Markups from reference [c] have been incorporated into this memo. To date, complete design criteria and features for the new Chehalis River flood control dam forebay collector have not been established. The following assumptions from reference [b] remain:

- Attraction flow rate approximately 500 cfs with ability to increase to 1000 cfs.
- Fish sorting will be done on board; sampling will be done ashore.
- Fish will be off loaded via two hoppers.
- Design reservoir pool level change is 189 feet. Pool level change for normal operations is 40 feet.
- Collector moored to the dam.

The concept shown in references [e] through [j] combine the primary and secondary screen channel of the North Fork Reservoir collector with the sorting area of the Lake Cushman Reservoir collector. The North Fork collector has 900 cfs flow capacity, but does neither sorting or sampling on board. The Lake Cushman Reservoir collector sorts on board and off loads smolt, fry, and adults via two hoppers. This design continues to use Swift collector pile yokes based on a design pool level change of 189 feet.

Based on design requirement changes from reference [b], the new concept shortened the sorting area, reducing the overall length of the FSC from 164 feet to 156 feet. The molded depth was reduced from 26.5 feet to 24.5 feet. The Net Transition Structure (NTS) remains added forward of the fish channel entrance. A new ramp profile within the primary and secondary screen areas was identified as seen in reference [c]. The sorting area floor elevations were changed to 13.4 feet above baseline for the main floor and 7.4 feet above baseline for the hopper sump floors. A void was added to the aft six feet of the FSC, shortening the sorting area.

This concept has a sorting area that floods to an operating level of approximately 18.4 feet above baseline (~49% full) during normal operations. During the operating condition, the FSC maintains a design draft of 21.5 feet and zero trim and heel. As the sorting area floods, a series of grates direct the adults, fry, and smolt into different parts of the sorting area from where the adults, smolt and fry will be loaded into one of two hoppers (port and starboard) for offload.

The concept shown continues to have all sampling features removed from the main deck, including the pre-engineered building that housed sampling tanks and associated workstations. The transformer is in the weather forward and starboard of the sorting area on the main deck. The stairwell openings from the main deck to the port and starboard electrical rooms are located in watertight doghouses centered at ~frame 48 to prevent down flooding when the sorting area and trim tank are flooded. The pump room dimensions remain the same at 10 feet by 40 feet by 14.5 feet.

In order to determine an estimated lightship weight requirement, the following was accomplished:

- An arrangement of belly tanks, ballast tanks, trim tanks and voids were created in the geometry file (CHEHALISFSCg.GF1) to determine maximum ballast capacity of the current design. These tank and void boundaries were located according to key longitudinal and transverse structural members to facilitate construction. As the design progresses and weight estimates are refined, tank boundaries may have to be relocated.
- A Rhino 3d surface model was created with main structural boundaries and used to estimate structural weight. 35% was added to account for stiffeners and other structural members.
- Non Structural lightship weights (group 200-700) were taken from North Fork. For items in the primary and secondary screen areas, these weights should be fairly accurate. Weights in the sorting area will have more variation since this design is based on the Lake Cushman FSC aft of the screen areas.

Based on the above analysis the following observations were made:

- The estimated FSC lightship is approximately 134 kips deficient. This is true if all ballast tanks and void VD-75-0 are full, the trim tanks are at 50%, and the sorting area is at an operation level of 49%.
- The estimated lightship weight is approximately 13 feet forward of the where it needs to be. This means that the additional 134 kips cannot be added to the current hull to achieve the required moment. Ballast water would have to be relocated/rearranged aft.

Combined data from the multiple weight estimation resources can be seen in reference [d].

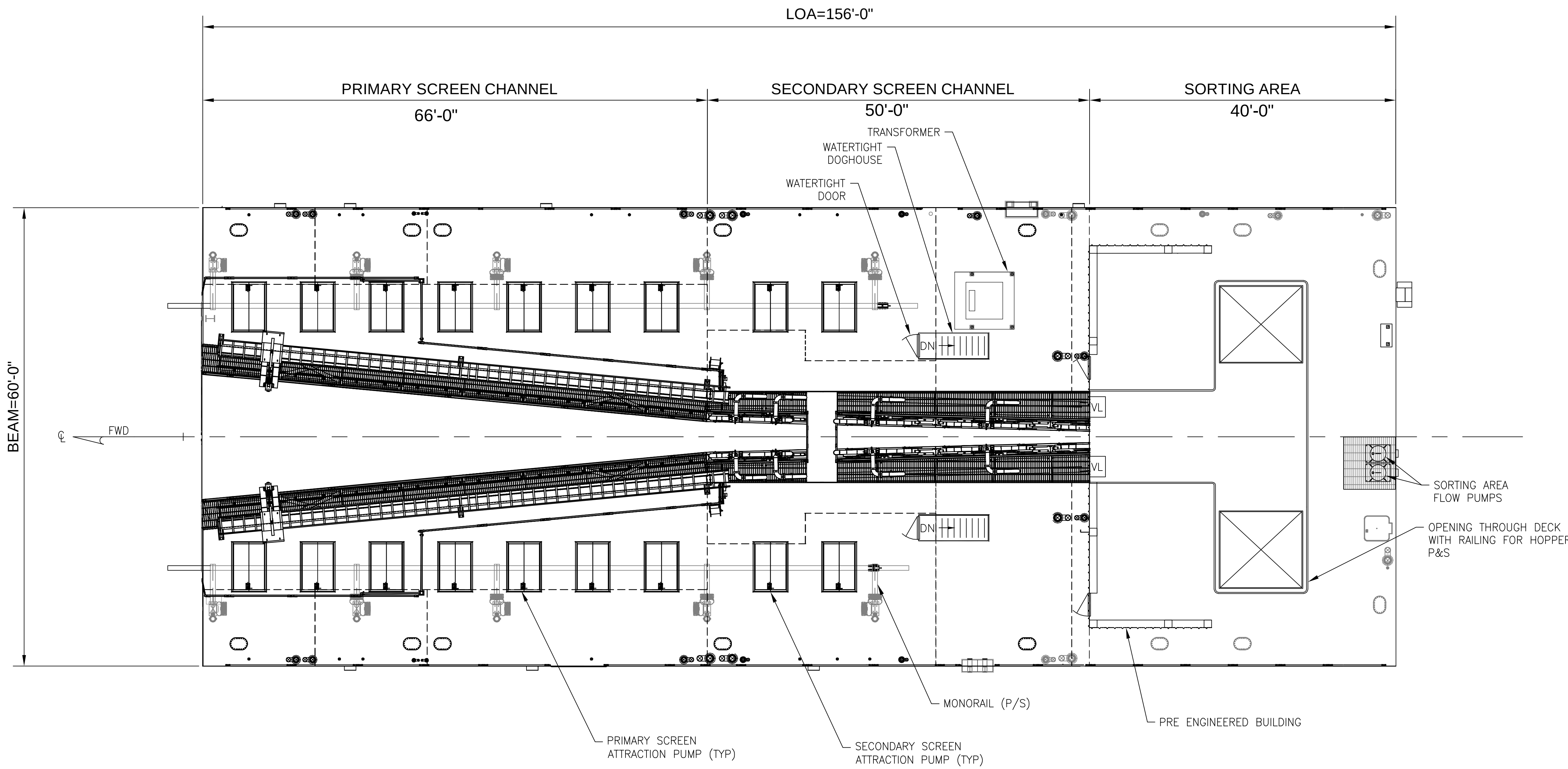
This concept, as shown with the previous, will not sink if the sorting area and adjacent trim tank are damaged. This assumes the following:

- Sorting area operating level is at 18.4 feet above baseline (~54% full)
- Down flooding points for the stairwells are less than 110 feet aft of the fish channel entrance

Such flooding could occur with attraction pumps off if the gate at the entrance to the sorting area closing the fish channel is open or doesn’t completely seal. (During operation with attraction pumps on, the fish channel water level is below the reservoir level. With attraction pumps off the fish channel water level will rise to the reservoir level.)

It is important to note that this survivability is based on the two bullets above only. If the operating level of the sorting area is reduced, the down flooding points must move further forward.

When the Sorting area is emptied from its operating level, the FSC takes on a forward trim resulting in a freeboard of 2.00 feet at the fore perpendicular. With the primary and secondary pumps off, the FSC is estimated to trim forward another six to twelve inches. This could result in a freeboard of as little as 12 inches at the fish channel entranc



PLAN
MAIN DECK
SCALE: 1/8"=1'-0"

PRELIMINARY
DATE: 09/02/2016

SCALE: 1/8" = 1'-0"

ART ANDERSON ASSOCIATES
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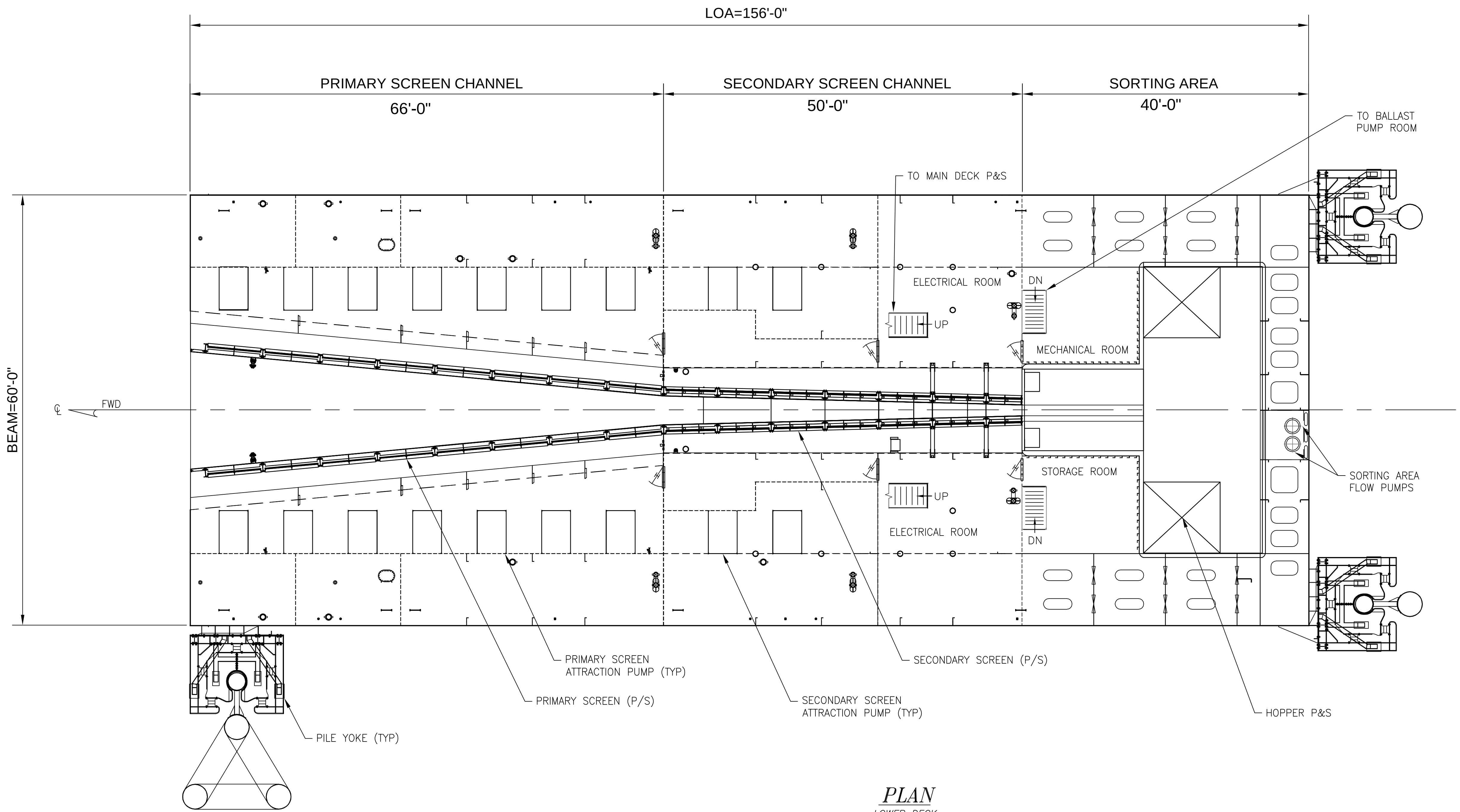
(360) 479-5600

CONTRACT NUMBER CFHDR007.001
TASK NUMBER 806

DSGN	E. SNYDER	07/12/2016
DWN	L. THOMAS	07/12/2016
CHK	J. CARLSON	07/12/2016
CHK	M. MCKAY	07/12/2016
OWNER		

CHEHALIS FLOATING FOREBAY COLLECTOR
CONCEPT SKETCH
MAIN DECK ARRANGEMENT

SIZE	CAGE CODE	DRAWING NUMBER	REV
D		CS-806-01	–
SCALE: AS SHOWN		CAD FILENAME CS-806-01.dwg	SHT 1 OF 6



PLAN
LOWER DECK
SCALE: 1/8"=1'-0"

PRELIMINARY
DATE: 09/02/2016

0 2' 4' 6' 8' 12' 16' 24'
SCALE: 1/8" = 1'-0"

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DSGN	E. SNYDER	07/12/2016
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CHEHALIS FLOATING FOREBAY COLLECTOR
CONCEPT SKETCH
LOWER DECK ARRANGEMENT

SIZE	CAGE CODE	DRAWING NUMBER	REV
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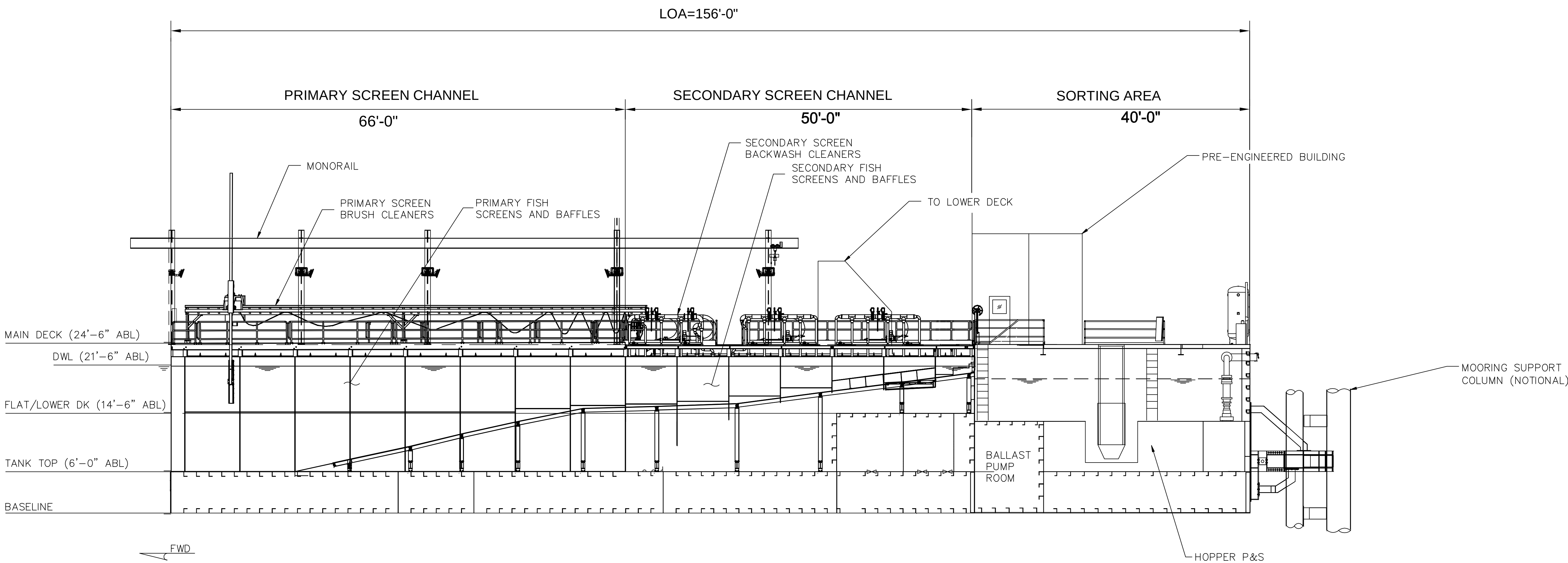
SCALE: AS SHOWN		CAD FILENAME CS-806-02.dwg	SHT 2 OF 6



INBOARD PROFILE

SCALE: 1/8"=1'-0"

PRELIMINARY

DATE: 09/02/2016

SCALE: 1/8" = 1'-0"

ART ANDERSON ASSOCIATES
202 PACIFIC AVE, BREMERTON, WA 98337
(360) 479-5600

CONTRACT NUMBER CFHDR007.001	
TASK NUMBER 806	
DSGN E. SNYDER 07/12/2016	
DWN A. WOHLSEN 07/12/2016	
CHK J. CARLSON 07/12/2016	
CHK M. MCKAY 07/12/2016	
OWNER	

CHEHALIS FLOATING FOREBAY COLLECTOR
CONCEPT SKETCH
INBOARD PROFILE

SIZE	CAGE CODE	DRAWING NUMBER	REV
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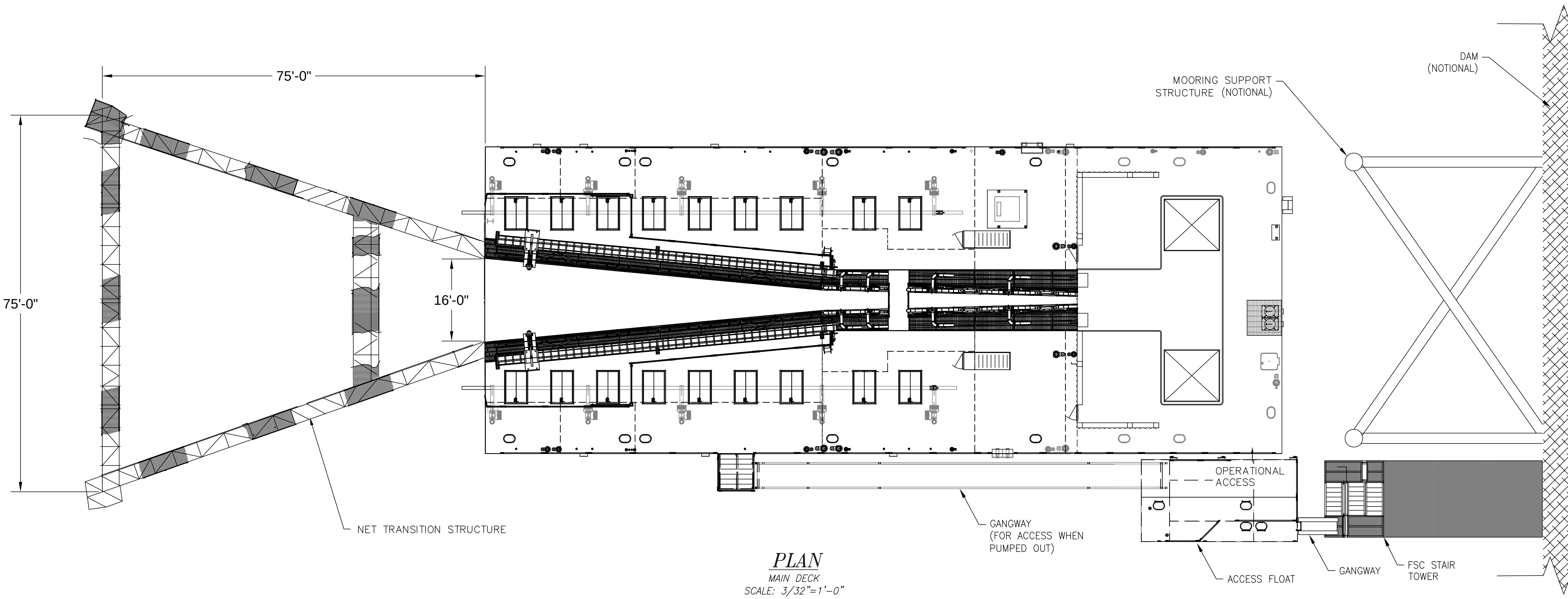
SCALE: AS SHOWN	CAD FILENAME CS-806-03.dwg	SHT 3 OF 6	



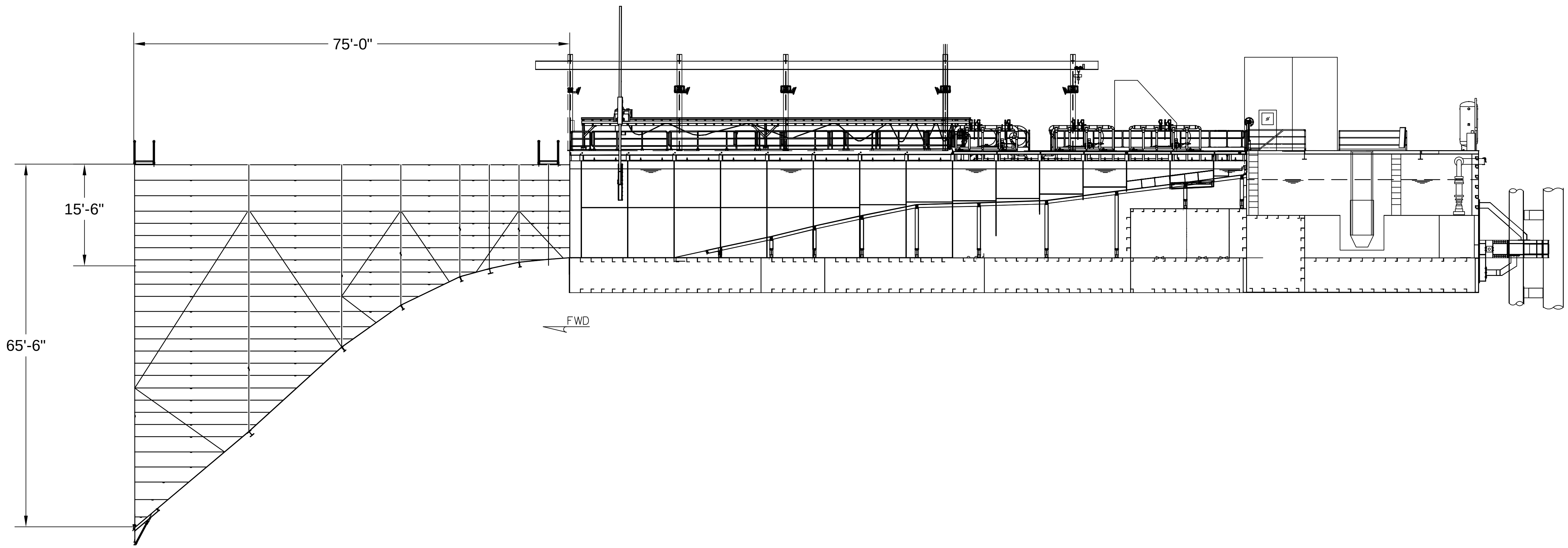
PLAN
MAIN DECK
SCALE: 3/32"=1'-0"

PRELIMINARY
DATE: 09/02/2016

0 2' 4' 6' 8' 10' 12' 16' 20' 24' 32'
SCALE: 3/32" = 1'-0"

ART ANDERSON ASSOCIATES
202 PACIFIC AVE, BREMERTON, WA 98337
(360) 479-5600

CONTRACT NUMBER CFHDR007.001 TASK NUMBER 806	CHEHALIS FLOATING FOREBAY COLLECTOR CONCEPT SKETCH MAIN DECK ARRANGEMENT W/ NTS
DSGN 07/12/2016 E. SNYDER	
DWN 07/12/2016 L. THOMAS	
CHK 07/12/2016 J. CARLSON	
CHK 07/12/2016 M. MCKAY	SIZE D · CAGE CODE · DRAWING NUMBER CS-806-04 · REV –
OWNER	SCALE: AS SHOWN · CAD FILENAME CS-806-01.dwg · SHT 4 OF 6



INBOARD PROFILE

SCALE: 3/32"=1'-0"

PRELIMINARY

DATE: 09/02/2016

0 2' 4' 6' 8' 10' 12' 16' 20' 24' 32'

SCALE: 3/32" = 1'-0"

ART ANDERSON
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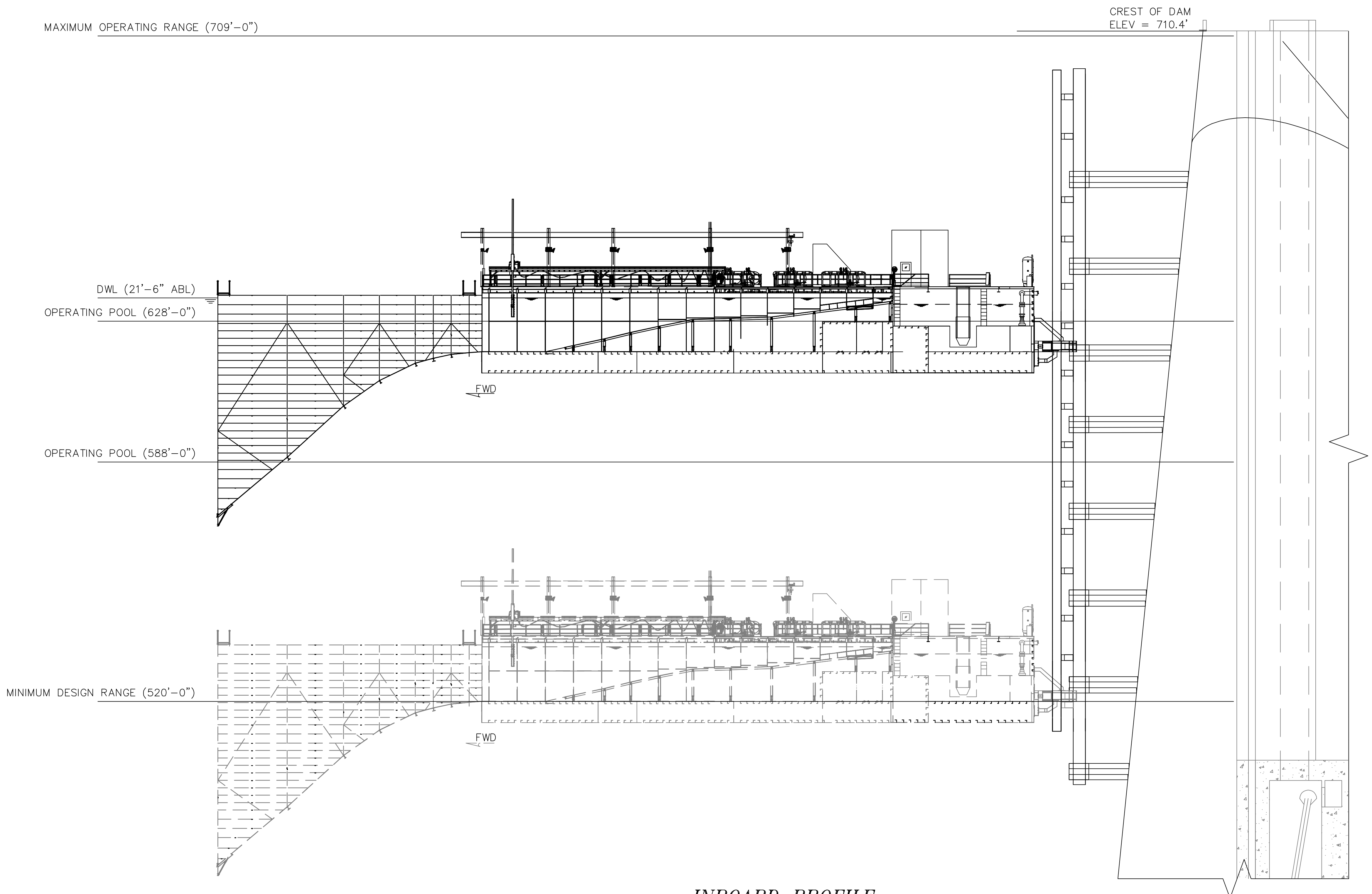
CONTRACT NUMBER CFHDR007.001 TASK NUMBER 806	CHEHALIS FLOATING FOREBAY COLLECTOR CONCEPT SKETCH INBOARD PROFILE W/ NTS
DSGN 07/12/2016 E. SNYDER	
DWN 07/12/2016 A. WOHLSEN	
CHK 07/12/2016 J. CARLSON	
CHK 07/12/2016 M. MCKAY	SIZE D · CAGE CODE · DRAWING NUMBER CS-806-05 · REV –
OWNER	SCALE: AS SHOWN · CAD FILENAME CS-806-03.dwg · SHT 5 OF 6



INBOARD PROFILE
SCALE: 1/16"=1'-0"

PRELIMINARY
DATE: 09/02/2016

0 2' 4' 6' 8' 10' 12' 16' 20' 24' 32'
SCALE: 3/32" = 1'-0"

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CONTRACT NUMBER CFHDR007.001 TASK NUMBER 806	CHEHALIS FLOATING FOREBAY COLLECTOR CONCEPT SKETCH INBOARD PROFILE W/ NTS AND DAM		
DSGN 07/12/2016 E. SNYDER			
DWN 07/12/2016 A. WOHLSEN			
CHK 07/12/2016 J. CARLSON			
CHK 07/12/2016 M. MCKAY	SIZE D	CAGE CODE	DRAWING NUMBER CS-806-06 / REV –
OWNER	SCALE: AS SHOWN	CAD FILENAME CS-806-03.dwg	SHT 6 OF 6

A.1.b. Chehalis Basin Strategy: Fish Passage CHTR Preliminary Design Report. February 2018.

Chehalis Basin Strategy

—— Fish Passage: CHTR Preliminary Design Report ——

Reducing Flood Damage and
Restoring Aquatic Species Habitat

February 2018

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ACRONYMS AND ABBREVIATIONS LIST

AWS	auxiliary water supply
cfs	cubic feet per second
CHTR	collect, handle, transfer, and release
fps	feet per second
FRFA	flood retention flow augmentation
FRO	flood retention only
FRO-X	flood retention only – expandable
NOAA Fisheries	National Oceanic and Atmospheric Administration Fisheries
O&M	operations and maintenance
RCW	Revised Code of Washington
WDFW	Washington Department of Fish and Wildlife

EXECUTIVE SUMMARY

> **Document Development**
>
> This document is reflective of the design of the collect, handle, transfer, and release fish passage facility to a preliminary design level.

The state of Washington retained a team of consultants to study and develop fish passage alternatives for proposed flood retention structural alternatives as part of the Chehalis Basin Strategy. The consultant team, composed of engineers and biologists from Anchor QEA, LLC, and HDR Engineering, Inc., were tasked with advancing the preliminary design of a collect, handle, transfer, and release (CHTR) facility for the purposes of collecting and transporting upstream migrating fish above a potential flood damage reduction structure (dam) near Pe Ell, Washington if one were to be implemented.

The fish passage study and design team facilitated meetings with the Fish Passage Technical Subcommittee of the Flood Damage Reduction Technical Committee, with the intent of refining design criteria, obtaining feedback on design modifications, and maintaining agency concurrence on the preliminary design. The Fish Passage Technical Subcommittee is composed of state, federal, and tribal participants, including the National Oceanic and Atmospheric Administration Fisheries, U.S. Fish and Wildlife Service, Washington Department of Fish and Wildlife, and the Quinault Indian Nation. The study included refinement of design criteria, preliminary level design development of a CHTR fish passage facility, and evaluation of costs for potential fish passage facilities that could accommodate passage of upstream migrating fish species, should a run-of-river-type dam be built. These activities were performed in collaboration with members of the Flood Damage Reduction Technical Committee, and in concert with numerous other physical and biological studies being performed as part of the Chehalis Basin Strategy to evaluate potential flood damage reduction and aquatic species habitat restoration strategies.

This report presents refined design criteria and updated design concepts for the CHTR facility fish passage alternative associated with the flood retention only (FRO) and Flood Retention Only – Expandable (FRO-X; combined flood retention flow augmentation and FRO functions) dam alternatives. In their initial operational phases, each of these flood retention concepts would operate in a run-of-river condition where fish passage would be provided through an array of fish passage conduits through the dam. The potential CHTR facility would provide safe and timely upstream fish passage during flow events that trigger flood retention operations. Detailed information relative to the FRO dam alternative can be found in the *Combined Dam and Fish Passage Concept Design Report* (HDR 2016). Detailed design information relative to the FRO-X structural alternative will be provided in the design development report for the FRO-X structural alternative currently under development.

1 INTRODUCTION

1.1 Background

1.1.1 Need

The purpose of the Chehalis Basin Strategy is to evaluate the feasibility of mitigating flood hazards within the Chehalis Basin while exploring opportunities to enhance ecological conditions, aquatic habitat, and the abundance of fish. The scope of the Chehalis Basin Strategy has included studying water retention alternatives (dams), options for protecting Interstate 5 and other floodplain at-risk facilities and structures with or without a dam, and other small flood damage reduction projects throughout the Chehalis Basin (Figure 1-1). Along with several other options, considered either independently or in combination, the water retention concept, consisting of a dam on the upper Chehalis River including juvenile and adult fish passage facilities, was advanced through the conceptual design phase. Results from the work completed in the conceptual design phase were summarized in the *Combined Dam and Fish Passage Design Report* (HDR 2016). This report summarizes the further development of one of the fish passage alternatives, collect, handle, transfer, and release (CHTR) facility, which is common to the flood retention only (FRO), Flood Retention Only – Expandable (FRO-X; dam with combined flood retention flow augmentation [FRFA] and FRO functions), and FRFA dam alternatives studied.

1.1.2 Proposed Dam

The proposed dam site was selected from several alternative locations identified and evaluated in previous studies (Shannon & Wilson 2009a, 2009b). The design storage volumes and corresponding estimated water storage elevations for the FRO and FRO-X dam configurations presented in this document are summarized in Table 1-1. The storage volumes and corresponding dam heights and inundation areas are subject to change as climate change and operation studies advance through the planning process.

Table 1-1
Summary of Dam Storage Volumes and Maximum Water Surface Elevations

CONFIGURATION	CONSERVATION POOL[1] VOLUME (ACRE FEET)	FLOOD STORAGE VOLUME (ACRE FEET)	MAXIMUM CONSERVATION POOL[1] ELEVATION (FEET)	MAXIMUM FLOOD STORAGE[2] ELEVATION (FEET)
FRO/FRO-X	0	65,000	-	628
FRO-X operated as FRFA	to be determined	65,000	to be determined	to be determined

1. Conservation pool is the pool of water stored in the reservoir all year to seasonally enhance water quality and instream flow downstream of the dam.
2. Maximum flood storage volumes and elevations are to spillway crest and do not include flood routing capacity between the design flood (100-year event) and the Probable Maximum Flood.

Figure 1-1
Chehalis Basin

Dam Alternative

The CHTR fish passage facility is designed to provide fish passage during impoundment events for the run-of-river-type dam alternatives currently under consideration: FRO dam and a FRO-X dam. The FRO-X dam is under development and would be operated as a run-of-river dam, but would be capable of being operated as a FRFA dam in the future.

The proposed dam site is south of State Route 6 in Lewis County, Washington, on the mainstem Chehalis River, about 1 mile south of Pe Ell (the southwest corner of Section 3, Township 12N, Range 5W). Figure 1-2 shows a rendering of the FRO dam. Figure 1-3 shows an example of the approximate flood storage inundation area (65,000 acre-feet of flood storage at the spillway crest) for the FRO configuration. A map of the approximate flood storage inundation area for the FRO-X dam alternative is not yet available.

Figure 1-2
Flood Retention Only Dam Rendering

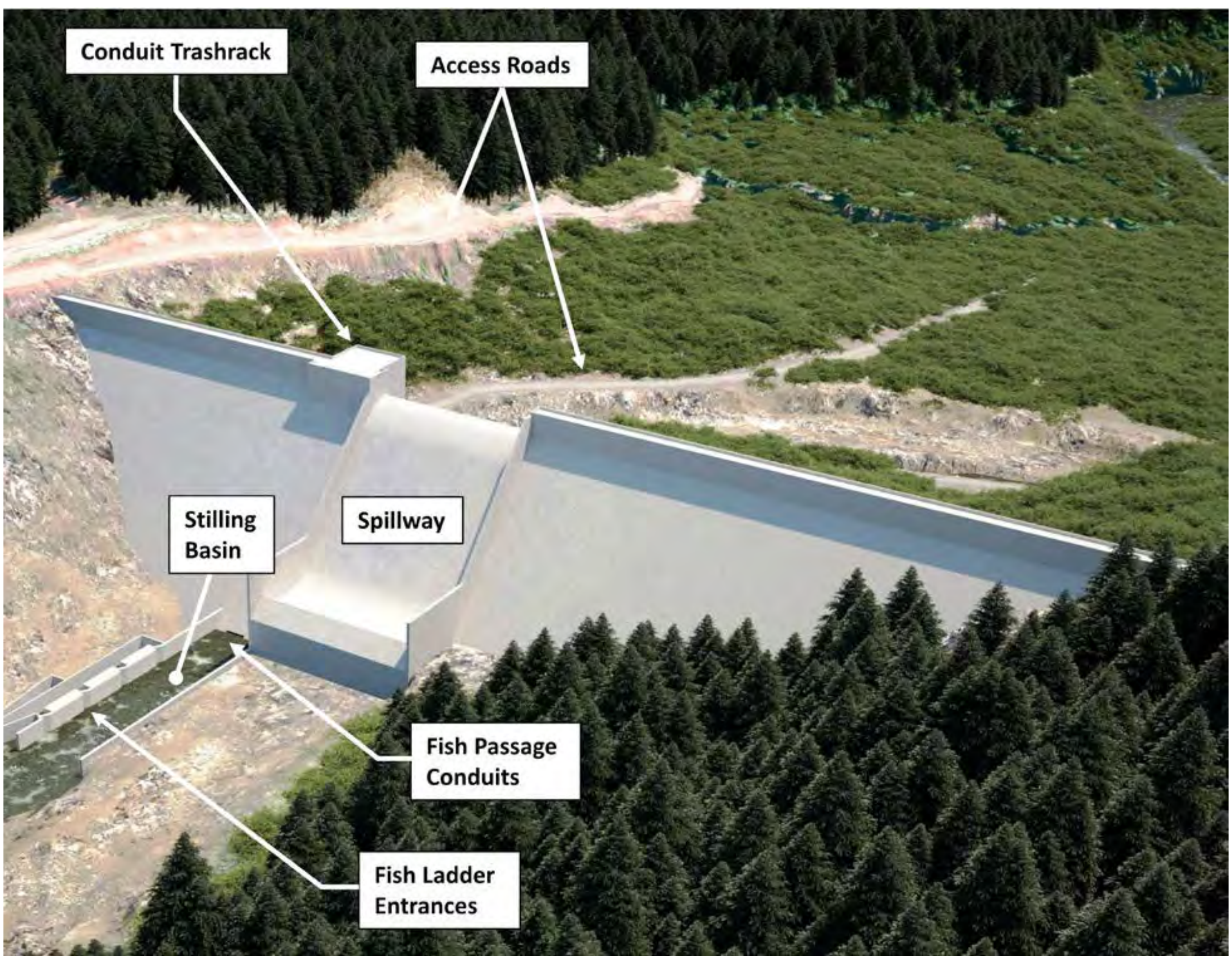
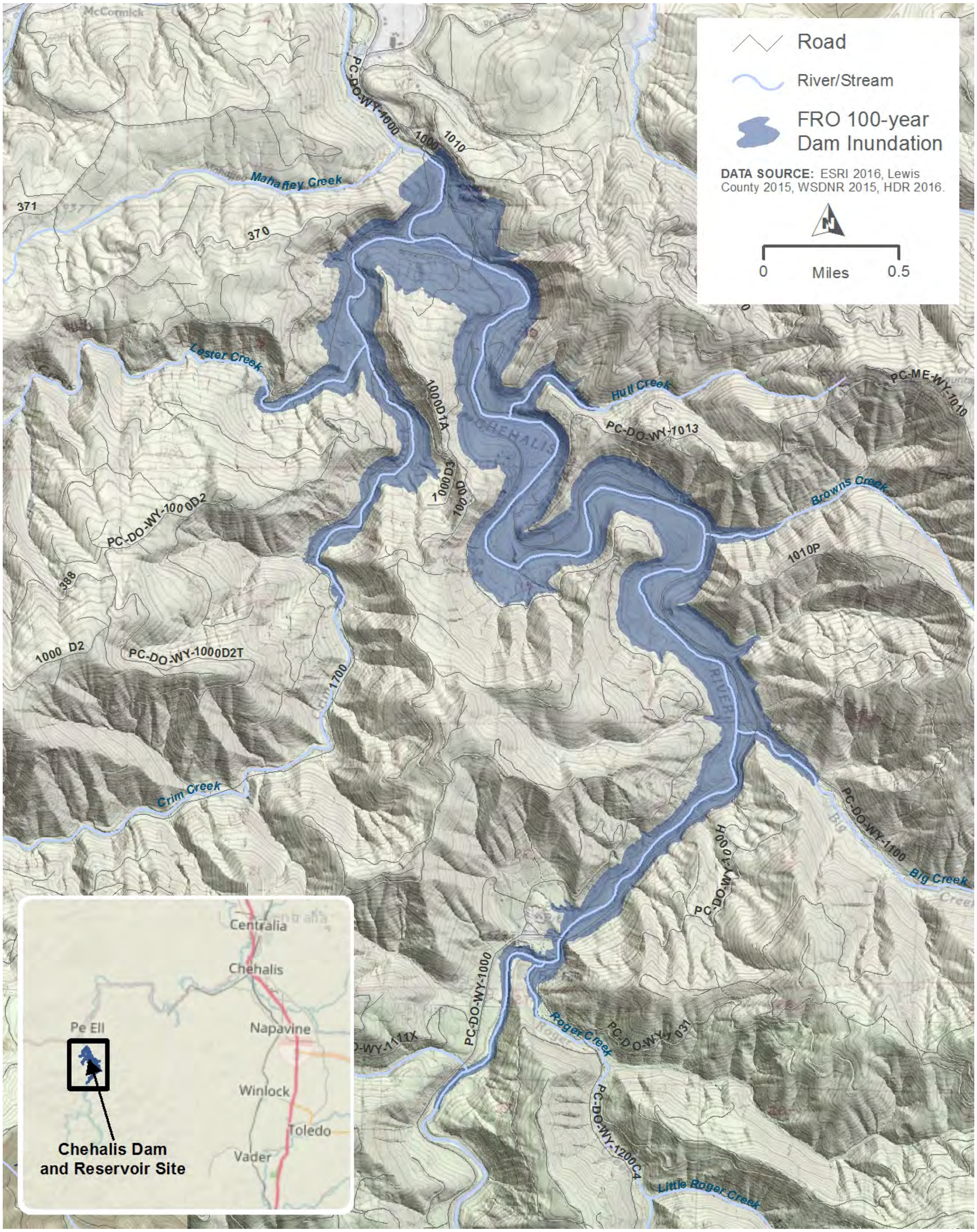


Figure 1-3

Flood Retention Only Flood Storage Inundation Area (65,000 Acre-Feet of Flood Storage)



1.1.3 Previous Alternative Analyses

Development of fish passage alternatives is an integral component of the dam design. An initial evaluation of potential fish passage technologies that could be integrated into design of each dam alternative was performed and documented in the *Fish Passage Design Technical Memorandum* (HDR 2014a). The *Combined Dam and Fish Passage Alternatives Technical Memorandum* (HDR 2014b), built upon the findings of the previous report (HDR 2014a) to combine selected dam design options with selected fish passage options to describe four integrated alternatives that could be compared in terms of function, constructability, and capital and operations and maintenance (O&M) costs. These integrated fish passage alternatives were further developed, evaluated, and compared in Appendix G of the *Combined Dam and Fish Passage Conceptual Design Report* (HDR 2017). That report also summarized the additional fish passage and dam design work accomplished during the 2015 to 2017 biennium.

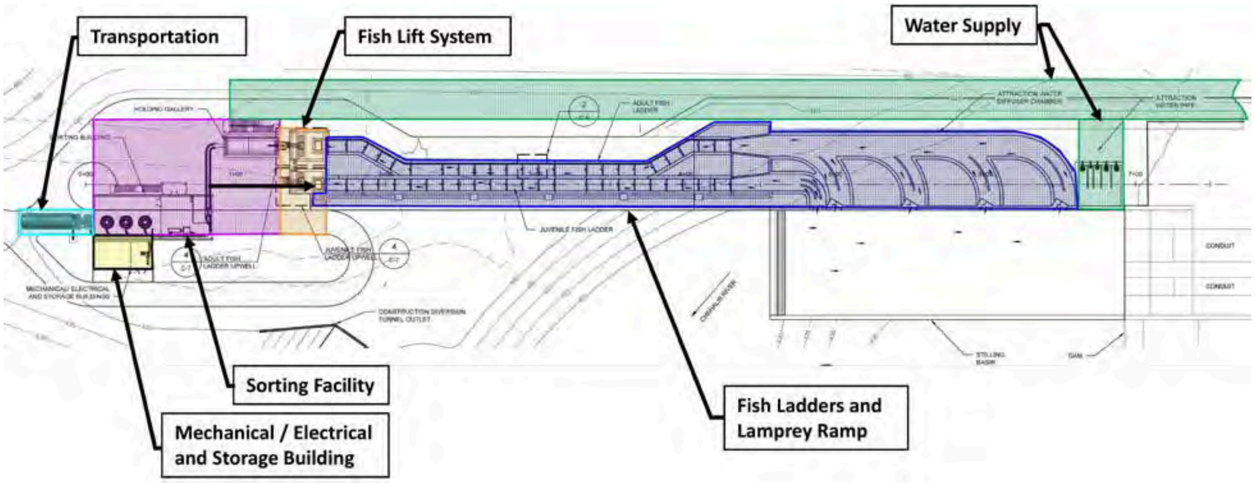
In early 2017, the Chehalis Basin Strategy managers recognized that additional information on the feasibility of designing the CHTR fish passage alternative was needed to inform the future development of a project-level environmental impact statement. Shortly following a decision to proceed from conceptual into a preliminary design effort for the CHTR associated with the FRO dam, development of a FRO-X dam alternative was initiated. The FRO-X dam alternative could be operated initially as a run-of-river facility, similar to the FRO, but would have the capability of operating as a FRFA dam in the future, if needed. The CHTR fish passage facility presented in this report was developed to be compatible with the FRO alternative or FRO-X dam alternative when it is operated as a run-of-river impoundment structure.

1.1.4 Collect, Handle, Transfer, and Release Alternative

The primary means of upstream and downstream passage at the FRO dam or FRO-X dam would be via a series of three to five conduits that extend through the base of dam and provide an open-channel flow condition for river flows less than 4,500 cubic feet per second (cfs). When water is impounded behind the FRO dam or run-of-river FRO-X dam during floods, the conduits would be closed, and fish passage provided via a CHTR facility. The CHTR facility is intended to provide fish passage to upstream-moving adult salmon, steelhead, resident fish, and lamprey as well as juvenile salmon and steelhead. The facility is designed to safely collect these species and life stages and transport them upstream of the dam using specialized vehicles.

The CHTR facility is anticipated to operate approximately only once every 7 years on average. When flood control scenarios require its operation, it would be a staffed facility that is operated 24 hours a day until flood operations cease and passage through the conduits resumes. A concept layout of the CHTR facility is provided in Figure 1-4, and more detailed drawings are provided in Appendix B of this document. For more details associated with the operational strategy of the FRO dam alternative, refer to the *Combined Dam and Fish Passage Conceptual Design Report* (HDR 2017) and the *Draft Operations Plan for Flood Retention Facilities* (Anchor QEA 2016).

Figure 1-4
Collect, Handle, Transfer, and Release Facility General Layout



Note: Sorting Facility refers to the sorting building, holding gallery, and surrounding area.

1.1.5 Collaboration with Technical Subcommittee

The fish passage design team and members of the Chehalis Basin Strategy Flood Damage Reduction Technical Committee coordinated and carried out several Fish Passage Technical Subcommittee meetings throughout development of the preliminary design of the CHTR. Of primary importance at these meetings were the discussion, interpretation, and formulation of design elements and criteria that would be carried forward to final design of the CHTR fish passage facility for a run-of-river dam alternative. Participants attending these meetings included representatives from the following organizations:

- National Oceanic and Atmospheric Administration Fisheries (NOAA Fisheries)
- Quinault Indian Nation
- State of Washington consultant study team
- U.S. Fish and Wildlife Service
- Washington Department of Fish and Wildlife (WDFW)
- Washington Department of Ecology

Meeting notes are included in Appendix A, and form the basis for criteria refinement, identification of key assumptions, and design decisions necessary to continue the engineering development of the CHTR fish passage facilities.

1.2 Purpose and Scope

Washington State's regulatory authority, defined in Revised Code of Washington (RCW) 77.57.030, requires that dam owners provide safe and timely fish passage for all fish species and fish life stages present in an affected area. Fish passage for the FRO and FRO-X dam alternatives would be provided via conduits through the dams. Fish passage through these conduits is addressed in separate reports, including Appendix G of the *Combined Dam and Fish Passage Conceptual Design Report* (HDR 2017). During periods when these dam alternatives are impounding water, fish passage would be provided via a CHTR facility. Preliminary design of the CHTR facility took place following the conceptual dam design efforts as part of the Chehalis Basin Strategy. This report documents the design criteria, main design elements, preliminary costs data, and preliminary design of the CHTR fish passage facility for FRO or FRO-X dam alternatives conducted in 2017. Preliminary design development for the CHTR, documented herein, was built on preceding design development activities (HDR 2016). This information will be used in future design phases to inform the final design of the CHTR passage facility.

The following activities were performed during preparation of this document:

- Collaboration with members of the Flood Damage Reduction Technical Committee (specifically, with the Fish Passage Technical Subcommittee) to refine and verify biological and technical fish passage design criteria
- Assessment of updated proposed dam operations and flow and stage duration statistics throughout anticipated migration periods of target fish species
- Performance of various hydraulic calculations for the purposes of sizing and configuring fish passage facility elements
- Confirmation of general orientation, location, and sizing of fish passage facilities and verification that the overall fish passage facility strategy addresses the performance requirements for upstream passage of target fish species during flood impoundment events
- Verification and refinement of fish passage facility design element descriptions, layouts, and operational parameters
- Verification and update of anticipated performance expectations for the CHTR fish passage facility in collaboration with the Fish Passage Technical Subcommittee
- Use of results gathered from the above activities to refine passage facility elements and refine favorable conditions for fish attraction, collection, or passage
- Development of a report documenting the refined design criteria, main design elements, preliminary costs data, and preliminary design

1.3 Project Team

The following consultant team personnel were involved in the various evaluations required to complete the updated conceptual designs:

Project Manager:	Beth Peterson, P.E.
Technical Manager and Lead Civil Engineer:	Keith Moen, P.E.
Lead Dam Engineer:	Keith A. Ferguson, P.E.
Lead Geotechnical Engineer:	Dan Osmun
Geological Engineers:	Andrew Little John Charlton
Lead Hydraulic Engineer:	Ed Zapel, P.E.
Lead Fish Passage Designer:	Michael Garello, P.E.
Constructability and Cost Estimating:	Jeffrey Allen
Project Support:	Gokhan Inci, Ph.D., Geotechnical Engineer Mathew Prociv, P.E., Fish Passage Design Shaun Bevan, P.E., Fish Passage Design John Ferguson, Ph.D., Fish Passage Biology (Anchor QEA) John Hess, Materials Engineering

Additional staff support provided for drawings, document production, and quality control.

1.4 Statement of Limitations

The CHTR fish passage facility layouts and designs for the FRO and FRO-X dam alternatives, operated as run-of-river impoundment structures, have been developed to a preliminary design level. Preliminary design development of this CHTR, summarized in this report, provides the general design concept, guidance for the development of the design in the form of design decisions made by the Fish Passage Technical Subcommittee, preliminary design criteria and assumptions, physical and temporal operating parameters, and main elements of the facility. Preliminary design drawings are provided in Appendix B. The preliminary design, including the design drawings and this report, is a work in progress and is not suitable for construction. Further design development of the preliminary design presented in this report is required to bring the design to a level adequate for construction. All elements of this study are prepared with the information available at the time of development and are subject to change in future phases of development as defining assumptions, directives, or available information are further refined.

2 DESIGN CRITERIA

2.1 Hydrology and Hydraulic Conditions

2.1.1 Facility Operation

2.1.1.1 Proposed Hydraulic Operation of FRO or FRO-X Dam

The FRO and FRO-X dams are operated as run-of-river facilities. During normal operation, the Chehalis River passes uncontrolled via conduits through the dams. Fish passage past the FRO and FRO-X dams takes place under normal operating conditions via the same conduits. The dams only impound water during high flow and anticipated floods. The FRO and FRO-X dams will temporarily store floodwater only when the river is forecasted to rise above 38,000 cfs within 48 hours at the downstream river monitoring gage at Grand Mound, Washington. The temporary storage events are estimated to have only a one in 7-year recurrence interval (HDR 2016). After flood regulation operations have been initiated and the conduit gates begin regulating outflows, fish passage through the conduits would no longer be available and operation of the CHTR facility for fish passage would commence.

2.1.1.2 Selection of Design River Flows

NOAA Fisheries and WDFW provide guidelines for when fish passage facilities must be operated throughout the full range of river flows. These are referred to as the fish passage design flows. Fish passage design flow criteria influences several factors associated with fish passage facility size and complexity. Guidelines presented by NOAA Fisheries and WDFW are based on exceedance calculations of mean daily flows, but can be modified to suit site-specific requirements. The exceedance flows statistically represent the flow equaled or exceeded during certain percentages of the time when migrating fish may be present or collected at a facility. The established guidelines are used to set instream flow depths, flow velocities, debris and bedload conditions, fish attraction requirements, tailwater fluctuations, and numerous other factors that a facility might experience while target fish species are migrating.

NOAA Fisheries (2011) requires the high fish passage design flow to be the mean daily streamflow that is exceeded 5% of the time during periods when target fish species are migrating. WDFW (2000a) suggests a 10% exceedance flow be used as a high design flow. NOAA Fisheries (2011) requires a low fish passage design flow equal to the mean daily streamflow that is exceeded 95% of the time during periods when migrating fish are typically present. WDFW recommends that a low flow be established based upon site-specific conditions.

Mean daily flows for water years 1940 through 2012 from U.S. Geological Survey gage 12020000 near Doty were reduced using Basin area and mean annual precipitation to estimate flows at the proposed dam site. An exceedance analysis was then performed on the estimated flows at the proposed dam site. The probability for exceedance of mean daily flows is summarized in Table 2-1.

At the dam site, 5% and 95% exceedance flows were also calculated for each adult species using their respective upstream migration timing. These results are provided in Table 2-2. The lowest 95% exceedance flow and the largest 5% exceedance determined the fish passage design flow for which both FRO and FRO-X upstream fish passage facilities will be designed. The lowest 95% exceedance flow is the 95% exceedance flow of 16 cfs, which occurs during the fall-run Chinook salmon migration period. The highest 5% exceedance flow is 2,197 cfs, which occurs during the coho salmon migration period. Therefore, fish passage facilities will be designed to operate from a low fish passage flow of 16 cfs to 2,200 cfs.

Table 2-1
Annual Flow Exceedance at the Proposed Dam Site

TIME EXCEEDED	FLOW (CFS)
99%	15
95%	19
90%	24
80%	37
75%	48
50%	171
25%	437
10%	960
5%	1,447
1%	2,957

Table 2-2
Flow Exceedance During Fish Migration Periods at the Proposed Dam Site

FISH SPECIES	95% EXCEEDANCE (CFS)	5% EXCEEDANCE (CFS)
Spring-run Chinook salmon	18	882
Fall-run Chinook salmon	**16**	1,592
Coho salmon	36	**2,197**
Winter-run steelhead	63	1,724
Coastal cutthroat trout	34	1,908
Pacific lamprey	17	737
Western brook lamprey	19	1,447

2.1.1.3 Selection of Tailwater and Reservoir Fluctuation Ranges

Anticipated tailwater fluctuations for the FRO and FRO-X dam configurations are significant factors in determining the type, size, and complexity of the CHTR fish passage facility. The fish ladder and fish ladder entrance of the CHTR facility must provide a continuous hydraulic connection throughout the anticipated range of tailwater elevations. In addition, any pump station supplying water for the CHTR facility 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. As tailwater fluctuations become larger, the facilities become larger and more complex. In some cases, certain fish passage and water supply technologies can be dismissed because they are unable to accommodate large tailwater fluctuations.

Historical river flows were used to calibrate the HEC-HMS simulation model to estimate the flood flows (WSE 2016). HDR performed additional hydraulic modeling of the stilling basin for the FRO dam to develop a tailwater rating curve that associates the tailwater elevations in the stilling basin with flows passing through the stilling basin. The design fish passage flows and select floods associated with their respective tailwater elevations in the stilling basins are provided in Table 2-3.

Table 2-3
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
Probable maximum flood	69,800	444.0

The FRO reservoir will only hold a pool during impoundment events. The water surface elevation in the reservoir will vary corresponding to the dam operations plan (Anchor QEA 2016). Operation of the reservoir during an impoundment event is presented in Section 2.4.1. Flow past the dam is controlled by the conduits and auxiliary water supply (AWS) system for the CHTR during impoundment events 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).

2.1.2 Water Supply

2.1.2.1 Summary of Flows

Multiple design elements of the CHTR fish passage facility require water to operate. The design flows for each element are provided in Table 2-4. The basis for these design flows are discussed in Sections 2.1.2.2 and 2.1.2.3.

Table 2-4
Water Supply Flows for Collect, Handle, Transfer, and Release Facility 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

2.1.2.2 Auxiliary Water Supply

NOAA Fisheries (2011) states that attraction flows from the entrance of the fish ladder should be greater than 10% of the high fish passage design flow. The minimum attraction flow for the CHTR facility should then be at least 220 cfs. However, the Fish Passage Technical Subcommittee decided in their March 22, 2017, meeting that, since 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 CHTR 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.

2.1.2.3 Gravity and Pumped Water Supply

Water is supplied to the CHTR facility via gravity throughout most of the CHTR operating period. When water levels in the reservoir are too low to supply water via gravity (see Section 2.4.2), 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. A pump station draws water from the tailwater pool. The adult fish ladder, juvenile fish ladder, and lamprey ramp are supplied by a single pump or a set of pumps, depending on the amount of pumped flow required. A single backup pump will remain available for use if needed. A single pump will be provided to supply water to the backwash screen cleaning system for the pump station intake screens.

2.2 Biological Design Criteria

As part of the Chehalis Basin Strategy, WDFW has led an extensive field sampling program to collect data and better understand the phenology, abundance, habitat requirements, distribution, and migration patterns of fish present within the Chehalis River, and more specifically, in the potentially impacted areas of the dam structure and inundation limits of the reservoir. Using new and historically available data, WDFW has assisted the Fish Passage Technical Subcommittee with biological criteria development in collaboration with other participating technical committee members. The three primary types of biological design criteria that have the most influence on facility type, size, and configuration relate to the following:

- **Selected species and migration timing:** Informs the selection of species and life stages targeted for fish passage design as well as their seasonality, anticipated hydrologic conditions, and duration of periods where these target fish species may be expected to migrate upstream and/or downstream of the dam location
- **Species abundance:** Informs the annual number of fish that require passage as well as the peak daily rate of migration that influences facility size and operation requirements
- **Trapping and holding criteria:** Informs the requirements for fish trapping and holding, including, but not limited to, holding volume, duration, temperature, and water supply

2.2.1 Selected Species and Migration Timing

The selection of fish species and life stages for fish passage design was derived from field-specific data obtained by WDFW in 2015 and 2016 in addition to readily available historical documentation developed for the Chehalis Basin. In general, Washington State interprets its regulatory authority (RCW 77.57.030, *Fishways required in dams, obstructions – penalties, remedies for failure*) to require provision for passage of all fish and fish life stages believed to be present in the system. For the purposes of the development of the CHTR fish passage facility, 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. These primary species and their known swimming and leaping abilities were used to influence development of specific technical design criteria. Species known to occur downstream of the dam site were selected for consideration, but did not directly influence the development of specific technical design criteria.

The life histories and specific life stages of each target species were also considered relative to their known occurrence, distribution, and movement through the dam site. Life stages of specific species were selected if they have been observed moving – or are believed to move – through the dam site (either upstream or downstream).

Target fish species and their respective life stages that were selected for the purposes of design development in this study are presented in Table 2-5.

Table 2-5
Target Fish Species and Life Stages Selected for Collect, Handle, Transfer, and Release Design

SPECIES	UPSTREAM	DOWNSTREAM
Spring-run Chinook salmon	Adult, juvenile	Not applicable
Fall-run Chinook salmon	Adult, juvenile	Not applicable
Coho salmon	Adult, juvenile	Not applicable
Winter-run steelhead	Adult, juvenile	Not applicable
Coastal cutthroat trout	Adult, juvenile	Not applicable
Pacific lamprey	Adult	Not applicable
Western brook lamprey	Adult	Not applicable
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	Not applicable

Bull trout are believed to occur only downstream of the proposed dam location so they were removed by the Fish Passage Technical Subcommittee as a target species but remained a species of consideration throughout alternative development and concept design. Of the species and life stages targeted for upstream passage, juvenile salmonids, resident fishes, and lamprey exhibit the most variable life history, are the weakest swimmers, and represent the most challenging species and life stages requiring passage. Therefore, technical design criteria used to target the passage requirements of these species and life stages were believed to also accommodate the requirements of bull trout.

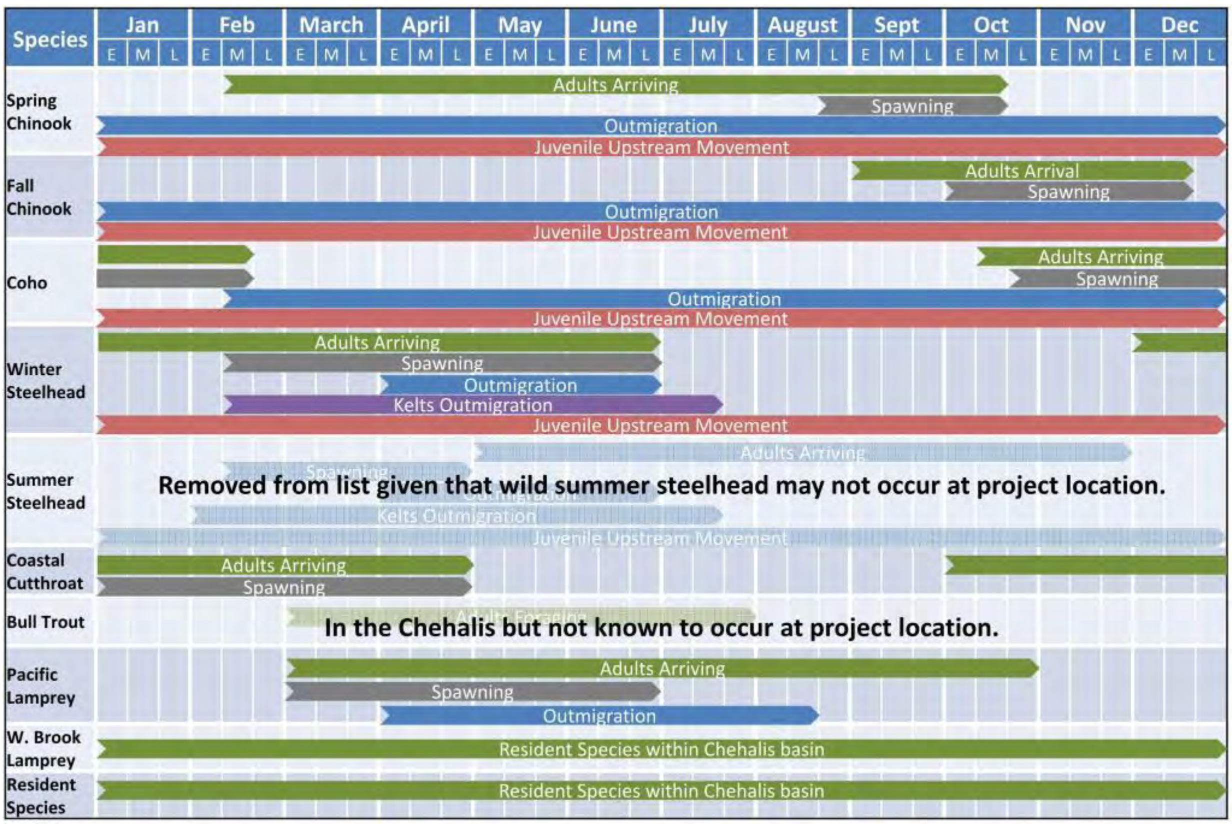
Passage technologies for lamprey are relatively new, and few facilities exist in the western United States that target lamprey for passage or collection and transport above dams. Where applicable, readily available best practices, lessons learned from experimental facilities on the Columbia River, and interviews with researchers who specialize in the understanding of lamprey behavior and navigational capabilities were used to inform lamprey passage facility requirements and anticipated performance. In addition to salmonids and the anadromous Pacific lamprey, multiple resident fish species and two species of resident lamprey (western brook and river) are believed to inhabit and transit the proposed dam area. A list of resident species believed to be in or that transit the dam site is provided in Table 2-5. Pursuant to RCW 77.57.030, passage facilities must also accommodate transit of resident species. As such, these resident species are also included as target species.

Many of the target species are known to have unique migration behaviors and are believed to pass upstream or downstream through the dam site at specific times of the year. Fish species migration timing and duration influence the design and operation of proposed fish passage facilities by defining the physical, operational, and environmental conditions expected to occur while passage is required. The migration timing and duration for each selected fish species and life stage was discussed at Fish Passage Technical Subcommittee meetings as new information was collected in the field and from literature sources. The resulting conclusions were used in fish passage alternative design development and are shown in Figure 2-1.

The selected values provide a summary of upstream migration, spawning, and outmigration periods suitable to inform robust fish passage designs. The periods shown in Figure 2-1 incorporate anecdotal data of species presence at the extreme ends of known movement periods and thereby are potentially broader than what may actually be found in the river. Aquatic target species' actual migration and spawning periods are far more complicated and nuanced. For the purposes of preliminary design, these nuances are not anticipated to be controlling factors in the design of the CHTR facility.

Figure 2-1

Anticipated Migration Periods of the Targeted Species and Life Stages (Periodicity)



2.2.2 Species Abundance

Fish abundance was evaluated by WDFW and discussed during Fish Passage Technical Subcommittee meetings held in 2016. Abundance was described in terms of peak annual and peak daily rates of migration. The peak daily rate of migration for upstream migrating fish influences the size of many components to fish passage alternatives. Documents and information provided by WDFW during Fish Passage Subcommittee Meetings (WDFW 2016a, 2016b) were used to assess CHTR facility sizes and capacities. The resulting assumptions used for facility sizing are summarized in the paragraphs below.

The peak rate of annual migration for adult salmonids moving upstream is provided in Table 2-6.

Table 2-6
Peak Number of Annual Upstream-Migrating Fish

SPECIES	PEAK ANNUAL MIGRATION
Spring-run Chinook salmon	1,350
Fall-run Chinook salmon	3,900
Coho salmon	12,900
Winter-run steelhead	5,630

Numbers for adult upstream migrating Pacific lamprey, cutthroat trout, resident fish, and juvenile salmonids were not available for the CHTR preliminary design. Because the design separates adult salmonid facilities from the facilities for these other species and life stages, the number of fish from these species passing the dam does not influence the size of the adult salmonid sorting and holding facilities, adult fish ladder, and adult hopper. The use of resident fish information in the sizing of the juvenile salmonid hopper is discussed in Section 2.2.3.

The peak daily counts of salmon and steelhead migrating upstream were estimated as 10% of the maximum annual run (WDFW 1992), and peak hourly counts were estimated as 20% of the peak daily count based on Bell (1991) and as cited in NOAA Fisheries (2011). Applying both criteria results in the peak hourly count being 2% of the annual run for each species. Using this methodology and based on the run timing information in the periodicity chart (Figure 2-1), a combined peak daily count of roughly 2,000 adult salmonids and a peak hourly count of 400 adult salmonids was used in the design development of the preliminary CHTR design. Calculations of CHTR holding and flow capacities needed to support species movement during the period of the migration based on the peak daily count and fish size, flow, and volume criteria (Table 2-9) are shown in Table 2-10.

2.2.3 Resident Fish

Guidelines have been established by NOAA Fisheries (2011) and WDFW (2000a, 2000b) for the design of passage facilities for salmonids, but little data exists regarding the passage of lamprey and resident fish species through fish passage facilities. The Fish Passage Technical Subcommittee, with support from the team's U.S. Fish and Wildlife Service representative, assembled relevant biological data for the target resident species, as well as for lamprey and salmonids. The subcommittee was not able to find data on all target resident species. A summary of what data was compiled for each species is provided in Table 2-7. Through continued collaboration with the Fish Passage Technical Subcommittee, the CHTR facility is being designed to accommodate trap and transport of these resident species listed in Table 2-7 to the extent possible, and without adversely affecting facility performance for listed priority species (salmonids, cutthroat trout, and lamprey).

Table 2-7
Locomotive and Biological Data Availability

SPECIES		DATA COLLECTED*	
LIFE STAGE	**COMMON NAME**	**SWIM SPEED**	**JUMP HEIGHT**
Adult	Spring-run Chinook salmon	●	●
Adult	Fall-run Chinook salmon	●	●
Adult	Coho salmon	●	●
Adult	Winter-run steelhead	●	●
Adult	Summer-run steelhead	●	●
Juvenile	Spring-run Chinook salmon	●	●
Juvenile	Fall-run Chinook salmon	●	●
Juvenile	Coho salmon	●	●
Juvenile	Winter-run steelhead	●	●
Juvenile	Summer-run steelhead	●	●
Adult	Coastal cutthroat trout	●	●
Adult	Bull trout	●	●
Adult	Pacific lamprey	●	Not applicable
Adult	Western brook lamprey	●	Not applicable
Adult	River lamprey	●	Not applicable
Adult	Largescale sucker	●	
Adult	Salish sucker	●	
Adult	Torrent sculpin	Not applicable	
Adult	Reticulate sculpin	Not applicable	
Adult	Riffle sculpin	Not applicable	
Adult	Prickly sculpin	Not applicable	
Adult	Speckled dace	●	
Adult	Longnose dace	●	
Adult	Peamouth	●	
Adult	Northern pikeminnow	●	
Adult	Redside shiner	●	
Adult	Rainbow trout	●	
Adult	Mountain whitefish	●	

Note:
● = Indicates that a data source has been identified

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. The design team was unable to locate data to inform how many resident or juvenile fish may enter the low volume, low velocity entrance and ascend the fish ladder. Therefore, it was decided that the hopper and transport tank for the juvenile/resident fish ladder will be sized to match the hopper for adult salmonids. Similarly, there is little data available regarding trap and holding requirements for the target resident fish species. Therefore, the juvenile and resident fish hopper and transport tank were sized using adult salmonid criteria, which are provided in Section 2.2.4.

2.2.4 Trapping and Holding Criteria

The criteria for fish trapping and holding are provided in Table 2-8 and Table 2-9.

Table 2-8
Trapping and Holding Criteria

CRITERIA	VALUE	REFERENCE
Holding duration – holding gallery	• 24 hours, maximum	NOAA Fisheries (2011)
Holding duration – hopper and transport tank	• 24 hours, maximum • 1/2 hour, maximum during peak run rates	NOAA Fisheries (2011)
Temperature	• 50°F	NOAA Fisheries (2011)
Dissolved oxygen	• 6 to 7 parts per million	NOAA Fisheries (2011)
Water supply, holding, fry	• 0.0075 gallons per minute (gpm) per fish	Piper et al. 1982
Water supply, holding, smolts	• 0.13 gpm per fish	Piper et al. 1982
Water supply, holding, adults	• 0.67 gpm per fish	NOAA Fisheries (2011)
Adult jump provisions	• Required	NOAA Fisheries (2011)
Segregation of fish	• Capability required	Not applicable
General	Decrease poundage of fish held by 5% for every degree over 50°F	

Table 2-9
Fish Size, Holding Volume, and Long-Term Holding Flow Criteria

SPECIES	AVERAGE ASSUMED WEIGHT/FISH (POUNDS)	LONG TERM HOLDING: FLOW/FISH (GPM)	HOLDING VOLUME (CF/POUNDS)
Spring-run Chinook salmon	23	1	0.25
Fall-run Chinook salmon	23	1	0.25
Coho salmon	9.5	0.5	0.25
Winter-run steelhead	9	2.0	0.25
Summer-run steelhead	8	2.0	0.25
Coastal cutthroat trout	1	Unknown	0.25
Lamprey	Unknown		
Resident species	Unknown		

Notes:
gpm = gallon per minute
cf = cubic foot
Holding volume and long-term holding flow requirements per NOAA Fisheries (2011)
Long-term flow requirements are for emergency situations where fish must be held for more than 72 hours
Adult fish sizes per Bell (1991).

Fish holding volume requirements do not change based on the amount of time held. However, flow requirements are contingent upon holding time, and fish held longer than 72 hours require more flow than fish held less than 72 hours. The Fish Passage Technical Subcommittee did not address fish holding periods during emergencies (e.g., a situation where washed out roads prevent fish transportation activities) Fish holding during emergency situations where holding may be required for more than 72 hours will be addressed in the next phase of design development. Flow requirements for long-term holding are provided in Table 2-9 for reference in future design development discussions.

Volume and flow needed for the holding gallery, fish hoppers, and transport tanks were determined using the trapping and holding criteria presented in Table 2-8 and the peak daily and hourly number of fish as determined in Section 2.2.2. The number of fish used to size these design elements is as follows:

- Holding gallery
  - Flow: Peak daily number of fish
  - Volume: Peak daily number of fish
- Hopper
  - Flow: Half the peak hourly number of fish
  - Volume: Half the peak hourly number of fish
- Transport tank
  - Flow: Not applicable
  - Volume: Half the peak hourly number of fish

The hoppers hold half the peak hourly count of fish to limit the size of the hoppers. Fish hoppers will be emptied frequently during peak short-term runs (e.g., every 20 minutes). However, during most of the trapping period, it is expected that low numbers of fish will enter the low volume, low velocity entrance each day, so the hopper will be emptied less frequently (e.g., every few hours). While the hopper may hold fish for up to 24 hours, the hopper will be operated such that no more than half the peak hourly count of fish is held at any time. Receptacles for life support systems will be provided on the outside wall of the hopper vessel (e.g., oxygen tanks). Use of such equipment will be evaluated based upon need during the commissioning and demonstration period.

Calculations determining the size of these elements are provided in Table 2-10 and Table 2-11.

Table 2-10
Adult Holding Gallery Sizing

CRITERIA	NO. OF FISH	POUNDS OF FISH	CF REQUIRED	FLOW (GPM)
Spring-run Chinook salmon	135	3,105	776.25	
Coho salmon	1,290	12,255	3,063.75	
Winter-run steelhead	563	5,067	1,266.75	
Subtotal			*5,107*	
Factor of Safety		*20%*	*1,022*	
Total	**1,988**	**20,427**	**6,130**	**1,332**

Notes:
cf = cubic foot
gpm = gallon per minute
Holding gallery sized for 1 day of peak-day run.

Table 2-11
Hopper and Transport Tank Sizing

CRITERIA	NO. OF FISH	POUNDS OF FISH	CF REQUIRED	FLOW (GPM)
Adult hopper and transport tank	200	2,043	511	134
Juvenile/resident hopper and transport tank	Same as adult hopper and transport tank			

Notes:
cf = cubic foot
gpm = gallon per minute
Juvenile/resident hopper and transport tank sized to match adult hopper and transport tank.

2.3 Technical Design Criteria

This section identifies technical design criteria, sources, and guidance relevant to the development of fish passage designs. Technical fish facility design criteria typically fall into two categories – criteria and guidelines. Criteria are specific standards for fish passage design that require an approved variance from the governing state or federal agency before a design can deviate from the established criteria. Deviating from an agency-established criterion requires establishing a site-specific, biological- or physical-based rationale for the deviation. In contrast, guidelines provide a range of values, or in some instances, specific values that the designer should seek to achieve, but that can be adjusted in light of project-specific conditions, if needed, to achieve the overall fish passage objectives by supporting better performance or solving site-specific issues. Adjustments to a design may be requested by the governing agencies during development of the design.

The list of criteria provided here is not intended to be an all-inclusive list of criteria used for the design of the CHTR facility, but this list did guide alternative formulation and concept development. The following documents provide the criteria and guidelines that were used during preliminary design of the CHTR. If two or more agencies provide differing guidance on a specific design criterion, the most conservative guidance from a fish passage and protection standpoint was followed.

- *Anadromous Salmonid Passage Facility Design* (NOAA Fisheries 2011)
- *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)

2.3.1 Fishway Criteria

Designs of upstream fish passage facilities at dams are developed based on criteria and guidelines developed to successfully pass adult salmonids. The fishway is comprised of two major components: the fishway entrance(s) and the fish ladder. Table 2-12 and Table 2-13 list the primary design criteria for the fishway entrance(s) and fish ladder, respectively.

Table 2-12
Fishway Entrance Criteria

CRITERIA	VALUE	REFERENCE
Location	Easily located by fish	NOAA Fisheries (2011), WDFW (2009)
Width	4 feet, minimum	NOAA Fisheries (2011)
Depth	6 feet, minimum	NOAA Fisheries (2011)
Head differential, adults	1 – 1.5 feet	NOAA Fisheries (2011), WDFW (2009)
Head differential, juveniles	0.13 inches	NOAA Fisheries (2011)
Attraction flow	5% – 10% of the maximum of the 5% exceedance flows for the migration period of each species	NOAA Fisheries (2011)
AWS energy dissipation factor	16 foot-pounds/second/cubic foot	NOAA Fisheries (2011)
AWS diffuser velocity, vertical	1 foot/second, maximum	NOAA Fisheries (2011)
AWS diffuser velocity, horizontal	0.5 foot/second, maximum	NOAA Fisheries (2011)
AWS diffuser bar spacing	1.75 millimeter, maximum (juvenile criteria)	NOAA Fisheries (2011)
Fish darting speed	27 feet per second, maximum	Bell (1991), pg. 6.3 (steelhead)
Fish darting duration	10 seconds, maximum	Bell (1991), pg. 6.2
Depth required for jumping	2 feet, minimum	USFS (2001), Adult Salmonid Migration Blockage Table (adapted)

Table 2-13
Fish Ladder Criteria

CRITERIA	VALUE	REFERENCE
Type	• Half-Ice Harbor	NOAA Fisheries (2011)
Head differential, juveniles	• 0.7 feet, maximum	NOAA Fisheries (2011)
Head differential, adults	• 1.0 feet	NOAA Fisheries (2011)
Energy dissipation factor	2 foot-pounds/second/cubic foot (juvenile criteria)	NOAA Fisheries (2011)
Turning pool	• Radius corners	NOAA Fisheries (2011), WDFW (2009)
Pool width	• 6 feet, minimum	NOAA Fisheries (2011)
Pool length	• 8 feet, minimum	NOAA Fisheries (2011)
Pool depth	• 5 feet, minimum	NOAA Fisheries (2011)
Baffle orifice dimensions	• 18 inches high x 15 inches wide	WDFW (2009)
Freeboard	• 3 feet, minimum	NOAA Fisheries (2011), WDFW (2009)

2.3.1.1 Fish Ladder Type Selection for Passage of Juvenile Salmonids and Resident Fish

The Fish Passage Technical Subcommittee identified two types of fish ladders that were expected to provide the best performance for target and resident species – a half-Ice Harbor fish ladder and a vertical slot fish ladder. Hydraulic analysis of half-ice harbor and vertical slot type fish ladders resulted in calculated orifice and slot velocities of 4.1 feet per second (fps) and 4.8 to 5.0 fps, respectively for passage of juvenile and resident fish. Data collected on the swimming speeds of target and resident fish indicates burst swimming speeds as low as 3.5 fps. Given that the half-Ice Harbor type ladder is believed to provide lower through-orifice velocities, it was selected as the preferred type of fish ladder and is believed to provide better passage performance than the vertical-slot.

2.3.2 Lamprey Passage Criteria

Throughout the preliminary design of the CHTR, the best available science relating to the lamprey passage at dams and in fishways was discussed, used to inform fish passage facility requirements, and incorporated into the design. This included information contained in the scientific literature, lessons learned from experimental facilities at U.S. Army Corps of Engineers dams on the Columbia River, and interviews with researchers who specialize in studying lamprey behavior and navigational capabilities. The following resources outline the experimental facilities and best practices used in the CHTR design for adult lamprey:

- *Best Management Practices to Minimize Adverse Effects to Pacific Lamprey* (USFWS 2010)
- *Adult Pacific Lamprey Passage: Data Synthesis and Fishway Improvement Prioritization Tools* (Keefer et al. 2012)

- *Pacific Lamprey and NRCS: Conservation, Management and Guidelines for Instream and Riparian Activities* (USDA 2011)
- *Pacific Lamprey Protection Guidelines for USDA Natural Resources Conservation Service Instream and Riparian Activities* (USDA 2010)
- *Lamprey Passage in the Willamette Basin: Considerations, Challenges, and Examples* (USFWS 2011)
- *Adult Pacific Lamprey: Known passage challenges and opportunities for improvement* (Keefer et al. 2014)
- *Evaluation of Adult Pacific Lamprey Fish Passage at Snake River Dams* (Stevens et al. 2015)

Based on information contained in these resources, the lamprey passage design criteria listed in Table 2-14 were used for the preliminary design of the lamprey passage components of the CHTR.

Table 2-14
Lamprey Passage Design Criteria

CRITERIA	VALUE	REFERENCE
Flow velocity	6 feet per second, maximum	USDA (2010)
Ramp width	1.0 feet minimum	USACE (2015)
Distance between resting pools	20 feet maximum	USACE (2015)
Water depth in ramp	3 inches, minimum	USACE (2015)
Wetted surface finish	Smooth	USACE (2015)

2.3.3 Pump Station Intake Criteria

The CHTR preliminary design includes the use of pumped flow from the dam stilling basin to supply flows to multiple CHTR components. The intake for the pump station (see Figure 1-4) is designed in accordance with the Hydraulic Institute's (2012) pump intake design guidelines and NOAA Fisheries (2011) salmonid passage facility design guidelines. The intake to the pump station will be screened according to NOAA Fisheries (2011) guidelines, which include the values shown in Table 2-15.

Table 2-15
Intake Screen Design Criteria

CRITERIA	VALUE
Screen bar spacing	1.75 millimeter
Approach velocity	0.40 fps, maximum
Screen cleaning	Active

2.3.4 Freeboard

The elevation of the finished ground at the sorting facility (see Figure 1-4) will be at least 440.0 feet mean sea level. The exterior walls of the fish ladder and the pump station will have a top elevation no less than 445.0 feet mean sea level. Designing these features to these elevations provides at least 6 feet of freeboard above the 100-year flood elevation.

2.3.5 Landslide Mitigation

The adult and juvenile fish ladders and fish ladder entrances are proposed to be located on the right bank of the Chehalis River adjacent to the stilling basin and at the toe of an identified landslide (Shannon & Wilson 2016). Shannon & Wilson (2016) note that more substantial retrogressive-type failures for this landslide are unlikely. Nonetheless, Shannon & Wilson (2016) recommend implementing mitigation measures such as monitoring the landslide for movement and installing deep drains, structural reinforcements, and stability berms. The recommendations provided by Shannon & Wilson (2016) are preliminary. In addition, the landslide report recommends that incorporating stability measures into the CHTR design should further evaluated in future design.

2.4 Operating Criteria

The CHTR fish passage facility 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 a FRO or FRO-X dam. While adult salmon and steelhead only pass upstream during certain periods of the year, the CHTR must be capable of operating at any time of year to accommodate resident fish, lamprey, and juvenile salmon and steelhead that currently traverse this reach of the Chehalis River and may wish to move upstream. Impoundment events are estimated to occur, on average, once every 7 years, but may occur more than once in any given year, and may occur with little forewarning.

Impoundment events occur when the flow rate in the Chehalis River at Grand Mound, Washington, is anticipated to be 38,000 cfs or more. Based on the hydrologic record from 1988 to 2016, the operational model indicates that these events are equivalent to about a 7-year recurrence interval (15% chance of occurrence in any year). Under future climate change conditions, it is estimated that these impoundment events would occur more frequently. Water will also be impounded in the reservoir when the natural flow of the river is greater than the capacity of the conduits, but not large enough to trigger an impoundment event at Grand Mound. Such situations are estimated to occur approximately once per year and last an average of 1 day. During these water retention events, the conduit gates would not be operated and remain fully open. Impoundment events in this report refer to flood operations triggered by high flows at Grand Mound and do not include events where some water is retained in the reservoir due to high flows at the dam but not at Grand Mound.

Downstream passage of outmigrating fish will be delayed during impoundment events coincident with flood retention activities. Since the primary flood control gates are almost closed and water is retained upstream of the dam, any outmigrating fish entering the impoundment at this time would also be temporarily retained. The passage of fish downstream would occur as the flood operations cease to occur and the reservoir is drained. Downstream passage would resume as normal operations of the dam structure resumed.

In addition, the CHTR fish passage facility will have to be maintained throughout its dormant periods to ensure that it is ready to operate with less than 48 hours of notice. The operating criteria herein define how design elements of the CHTR facility will operate, what components of the facility must be maintained, and how often maintenance will be required.

2.4.1 Operation Schedule

The FRO and FRO-X dams are operated as run-of-river facilities. Fish passage past the FRO and FRO-X dams takes place under normal operating conditions via conduits through the dam that are placed at river grade. The dams will impound water during anticipated floods at Grand Mound, Washington, (impoundment events) and during high flows at the dams. Fish passage is supplemented during impoundment events by opening and operating the CHTR facility. Supplemental fish passage via the CHTR will be designed to operate for 24 hours a day, 7 days a week for a maximum of 2 months.

How the CHTR would operate during a sample impoundment event (e.g., January 2009 flood) is shown in Figure 2-2 (Anchor QEA 2016). As indicated on the hydrograph, the CHTR facility would operate as soon as the gates began reducing flow through the conduits until the impoundment pool is emptied and run-of-river operations resume. There is a short period at the beginning of the CHTR operation where the river flow through the conduits is well above the high fish passage design flow (2,200 cfs; see Section 2.1.1.2). While NOAA Fisheries and WDFW guidelines do not require that fish passage be provided during these periods (above the high fish passage design flow), the CHTR will continue to operate during this period, as indicated in Figure 2-2.

2.4.2 Auxiliary Water Supply

Fish ladder flow is supplemented by an AWS to meet the fish ladder entrance attraction guidelines provided by NOAA Fisheries (2011). The Fish Passage Technical Subcommittee agreed that the auxiliary flow should be sufficient to meet the 300 cfs attraction flow requirement described in Section 2.1.2.2. The subcommittee further agreed auxiliary water could be provided solely via gravity from the impoundment pool when the impoundment pool depth exceeded 50 feet above the crown of the highest operating outlet. However, this would result in periods during CHTR operation when additional attraction water (i.e., AWS) would not be provided because the depth in the reservoir was too low (e.g., about half of 1/6/2009 and approximately 1/31/2009 through 2/5/2009 in Figure 2-3). Despite this, the attraction water requirements (10% of river flow) are still met during most of the time the AWS is not operating (e.g., approximately 2/1/2009 through 2/5/2009 in Figure 2-3) because the fish ladder flow alone provides attraction flow greater than 10% of the river flow. The periods where AWS flow is not provided were also accepted by the Fish Passage Technical Subcommittee (Appendix A). Figure 2-3 shows when attraction water guidelines are met and when auxiliary water may be supplied via gravity (without the use of pumped flow) for a sample impoundment period. During the evacuation period, the

CHTR is operating, but flow releases are anticipated to be high and fish passage effectiveness is expected to be low.

Figure 2-2

Collect, Handle, Transfer, and Release Operating Period During a Sample Impoundment Event

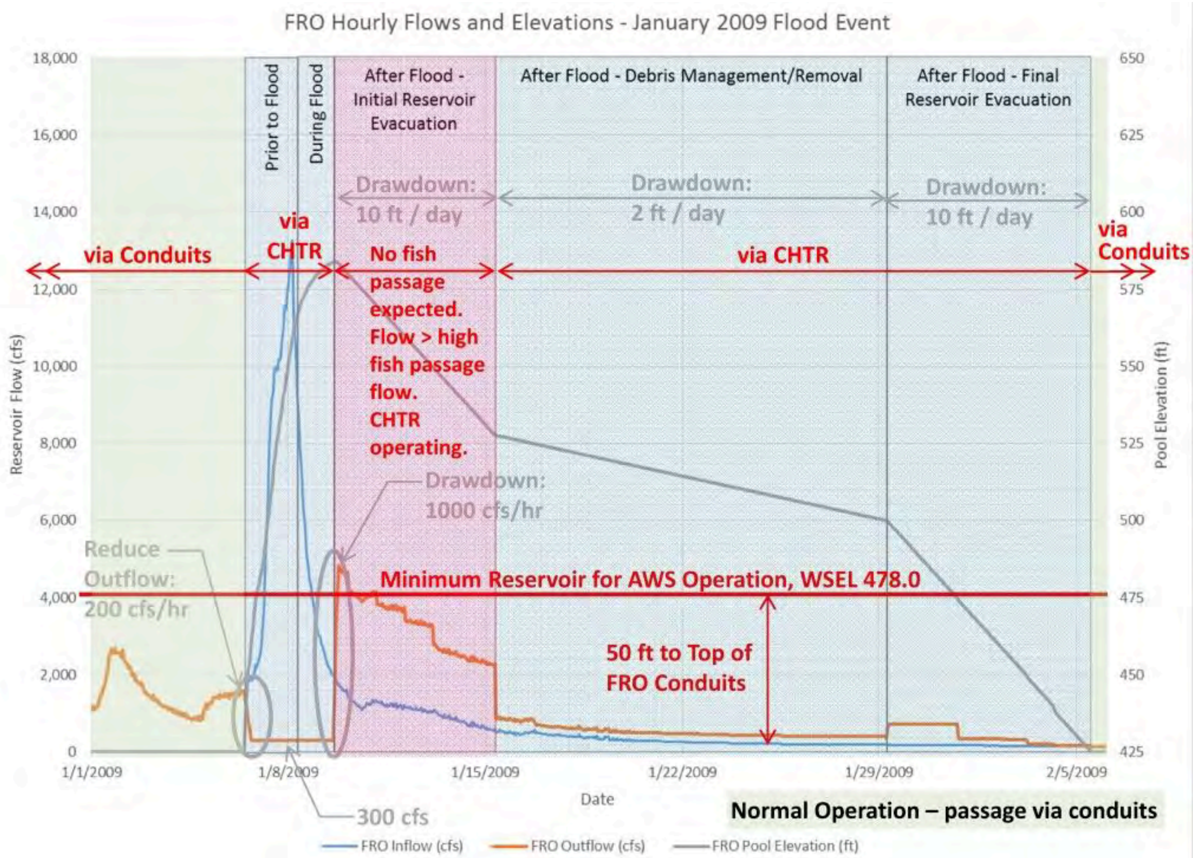
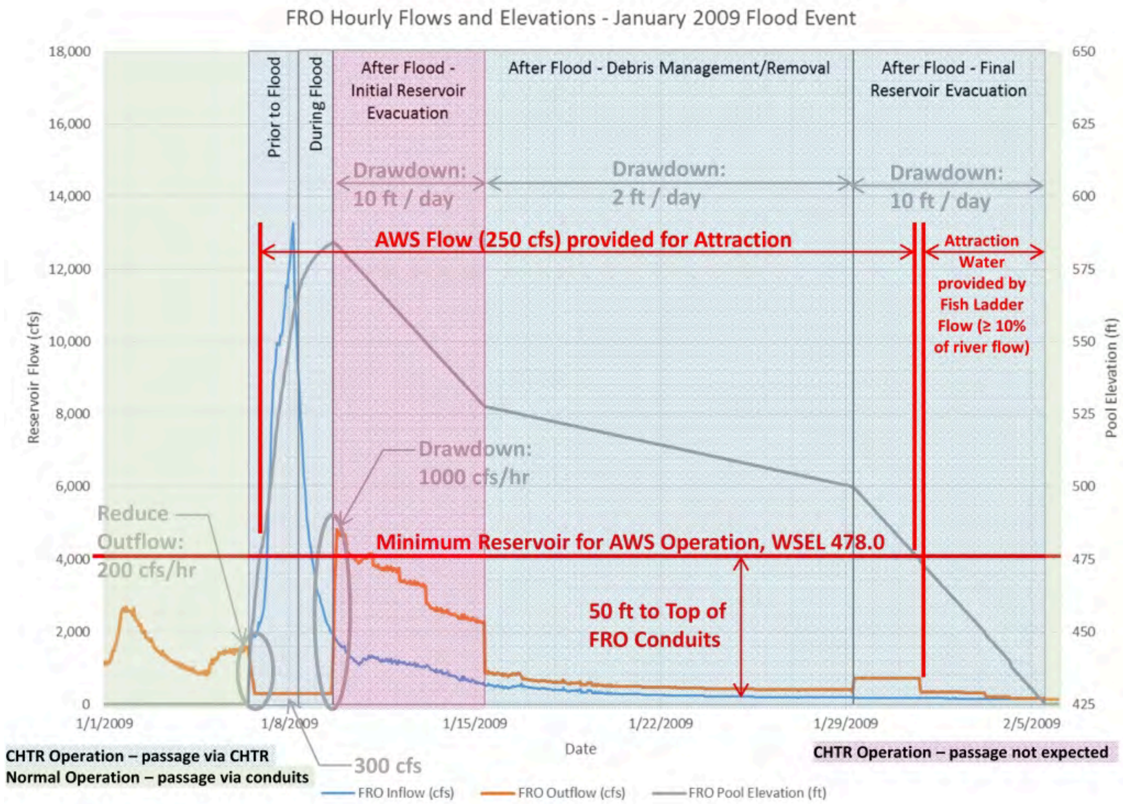


Figure 2-3
Attraction Water and Auxiliary Water Supply Durations during a Sample Impoundment Event

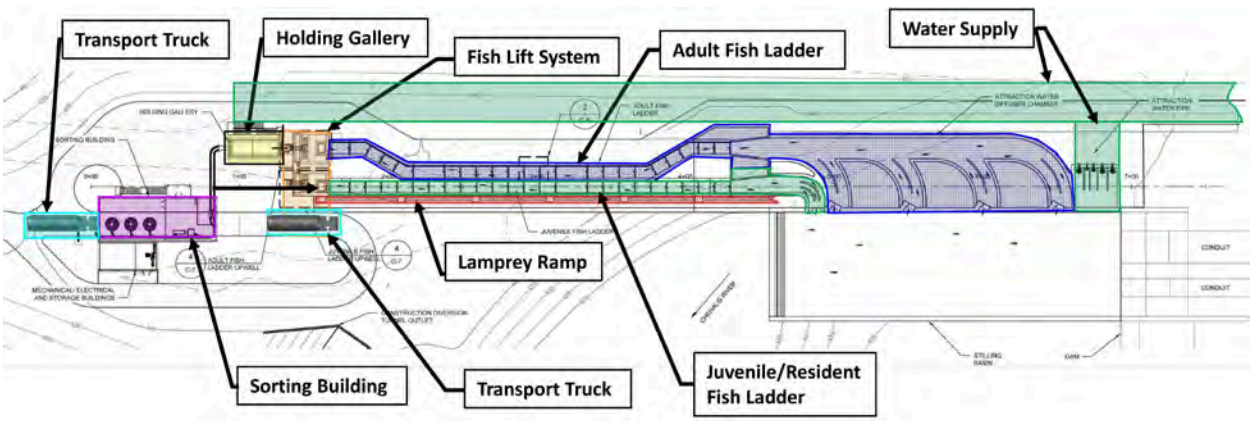


3 CHTR DESIGN ELEMENTS

3.1 Description of Design Elements

The CHTR facility consists of multiple design elements, which are described in this chapter. Figure 1-4 shows a layout of the design elements. A detailed layout of the CHTR features described in the subsections below is shown in Figure 3-1.

Figure 3-1
Collect, Handle, Transfer, and Release Features



3.1.1 Fish Ladders and Lamprey Ramp

3.1.1.1 Fish Ladder and Lamprey Ramp Entrances

The Adult Fish Ladder Entrance is a concrete structure on the right bank of the stilling basin that consists of four fish ladder entrance gates, an entrance pool, and an auxiliary water upwell and diffusion chamber. The entrance structure is about 230 feet long by 50 feet wide by 30 feet high. Four short channels branch off the entrance pool to the west where they tie into the east wall of the stilling basin. The stilling basin wall has an 8-foot-wide by 15-foot-tall rectangular opening at the end of each of the channels. Each opening contains a fish ladder entrance gate capable of covering the entire penetration or leaving it completely unobstructed. The juvenile fish, resident fish, and adult lamprey ladder entrance is located adjacent to the northernmost entrance gate, at the downstream end of the stilling basin. It consists of a 6-foot-wide by 15-foot-high penetration through the stilling basin wall with a gate that is also capable of blocking the full opening or leaving it fully open. A 33-foot-long by 21-foot-wide by 30-foot-high pool connects the juvenile fish ladder entrance gate to the juvenile fish ladder and lamprey entrances. A provision for allowing adult fish that enter the juvenile fish ladder entrance to access the adult fish ladder will be addressed in a future design phase.

Two lamprey entrances are located within the juvenile fish ladder entrance pool. Lamprey pass through the juvenile fish ladder entrance gate and entrance pool. Lamprey are attracted to the low velocity and smooth surfaces of the lamprey flume where it passes through the west wall of the juvenile fish ladder entrance pool. Both lamprey entrances are the same width and height as the lamprey ramp. One entrance penetrates the wall immediately downstream of the first juvenile fish ladder baffle. The invert of this entrance is located such that the water surface in the flume matches the water level in the juvenile entrance pool at the high fish passage design flow, water surface elevation 419.1. The second entrance is located downstream of the first lamprey ramp entrance and at a lower elevation corresponding to the low fish passage flow, water surface elevation 417.6.

Water is supplied to the adult and juvenile fish ladder entrance pools by their respective fish ladders and the auxiliary water upwell and diffuser chambers (see Section 3.1.5). The fish ladders connect to the north end of the entrance pools. The auxiliary water chambers are located adjacent to the adult fish ladder connection, on the east wall of the adult entrance pool, and between the adult and juvenile entrance pools. The floor of the chambers is several feet below the floor of the fish ladder entrance pools, allowing auxiliary water to pass from the upwell to wall screens in the diffuser chambers and to floor screens set in the floor of the juvenile and adult entrance pools.

The walls of the two diffuser chambers form a V-shaped channel with a flat floor in the adult entrance pool. Three 7-foot-long by 4-foot-high diffuser screens are set into the walls of the diffuser chambers on each side of the adult fish ladder entrance pool. A single 6-foot-wide by 21-foot-long floor screen is set in the floor between the diffuser chambers.

The west wall of the diffuser chamber, located between the adult and juvenile fish ladder entrance pools, and the straight west wall of the juvenile fish ladder entrance form a half-V-shaped channel with a flat floor in the juvenile fish ladder entrance pool. Three 7-foot-long by 4-foot-high diffuser screens are set into the wall of the diffuser chamber. Wall screens are only provided on this wall of the juvenile fish ladder entrance. Similar to the adult fish ladder, the juvenile fish ladder entrance has a single 6-foot-wide by 21-foot-long floor screen set in the floor west of the diffuser chamber.

While the tops of the walls remain at elevation 440 throughout entire fish ladder entrance, including the auxiliary water chambers, the floor in the upwell drops an additional 14 feet below the floor in the entrance pool and the auxiliary water diffuser area.

The fish ladder entrance is cut into hard rock on the right river bank. The adjacent stilling basin is cut deeper into the rock by over 30 feet; however, it is likely the deeper cut for the stilling basin walls will be nearly vertical. In the southwest corner, the fish ladder entrance will require about 15 feet of concrete fill as the rock bank drops at nearly a 1:1 slope in this area. The rock slope rises quickly to the northeast, leaving most of the fish ladder entrance to be founded on rock cut. Outside of the southwest corner, about half of fish ladder entrance is cut into the rock up to 50 feet deep. The remaining structure is cut 60 or more feet into rock.

In order to access the fish ladder entrance, an access road must also be cut into the hillside. The access road branches off the existing access road north of the sorting building, passes through the sorting facility area, and runs adjacent to the east wall of the fish ladder and fish ladder entrance. It is cut into the soil just above the rock line.

3.1.1.2 Adult Fish Ladder

A half-Ice Harbor-type fish ladder was selected for the CHTR, in part due to its ability to accommodate passage of aquatic species with a wide range of swimming and jumping capabilities. The fish ladder is about 270 feet long, extending from the fish ladder entrance to the fish lift. It consists primarily of 8-foot-wide by 10-foot-long pools. Baffles are located at the upstream and downstream end of each pool. The baffles are 4 feet wide by 5 feet 6 inches tall. The baffle rises to the 9 feet 6 inches tall for the remaining 4 feet of width. At the floor, in the center of short section of the baffle is an 18-inch square orifice. The floor slopes at a 1-foot vertical drop across each pool, which meets NOAA Fisheries' criteria for adult salmonids. The overall fish ladder has a slope of approximately 9.2%.

There are four turning pools and two resting pools in the fish ladder. The resting pools are the 10th and 18th pools upstream from the fish ladder entrance. The resting pools are twice as long as the standard pools. The turning pools are located at the upstream and downstream ends of the fish ladder to accommodate the water supply upwells and the diffuser chamber, respectively.

The fish ladder is a slab-on-grade structure cut into the hillside. In the lower portion of the fish ladder the structure is benched into rock. The fish ladder continues to be founded on rock for its full length but the depth of the rock cut lessens as the fish ladder ascends. The fish ladder pools are accessible via the grating over every pool.

3.1.1.3 Juvenile Fish Ladder

The juvenile fish ladder is nearly identical to the adult fish ladder except for a few minor differences:

- The floor slopes at a 0.7-foot vertical drop across each pool, which meets NOAA Fisheries' criteria for juvenile salmonids
- The overall fish ladder has a slope of approximately 6.5%
- There are no turning pools in the juvenile fish ladder
- The 10th pool upstream from the fish ladder entrance is the only resting pool in the fish ladder
- The juvenile fish ladder has one more pool than the adult fish ladder
- The juvenile fish ladder is located immediately adjacent to the adult fish ladder
    - The two fish ladders share a common wall between them for much of their lengths

A set of pools and a single baffle connect the adult fish ladder and the juvenile fish ladder just upstream of the first juvenile fish ladder pool. The water surface drops 1 foot across the baffle to accommodate adult fish but exclude juvenile fish. The cross connection allows adult fish that enter the juvenile fish ladder entrance a path to move upstream to the appropriate collection hopper. Future development of the CHTR will consider mechanisms to exclude adults from moving up the juvenile fish ladder past the cross connection, such as a bar rack.

3.1.1.4 Lamprey Ramp

A 1-foot-6-inch-wide, Bonneville-style steel flume is located adjacent to the west wall of the juvenile fish ladder. The flume is a free-standing structure mounted to a continuous concrete foundation. The flume is bolted together at legs located about every 5 feet. The concrete foundation buried to the crown in a gravel area between the juvenile fish ladder (east) and river protection retaining wall (west). The gravel area serves as an access path adjacent to the lamprey ramp for its full length. The lamprey ramp extends from the entrances in the west wall of the juvenile fish ladder entrance northeast to the fish lift. Resting boxes measuring 4 feet wide by 5 feet long by 3 feet deep are located every 50 feet along the full length of the ramp.

3.1.2 Fish Lift System

The fish lift system consists of a gantry crane, adult fish hopper and trapping mechanism, a juvenile/resident fish hopper and trapping mechanism, and a lamprey tank and trapping mechanism. The adult and juvenile/resident trapping mechanisms and hoppers sit inside concrete sumps, into which the hoppers are lowered during trapping. Side clearances between the hoppers and sump sidewalls shall not exceed 1 inch, thereby minimizing fish access below the hopper. Flexible side seals or brushes are used to ensure that fish do not pass below the hoppers. The sumps are 10 feet wide by 10 feet long. The concrete floors of the sumps are 4 feet below the fish ladder floors and roughly 10 to 18 feet below existing grade. Water is supplied to an upwell and diffuser system upstream of the hopper sumps. Water enters the upwell through a perforated pipe at the bottom of the upwell. Water passes over a weir and is diffused through 4-foot-wide by 3-foot-tall screens in the bottom walls of the sumps. One screen is located on each of the three upstream walls of adult sump while screens are only located on the north and east walls of the juvenile sump due to space constraints.

The trapping mechanisms for the adult and juvenile/resident hoppers are vee-traps built into the hoppers, which allows fish to volitionally enter, but not exit, the hoppers. All components exposed to fish have welds and sharp edges ground smooth to the touch, and other features as required to minimize injuries. The vee-trap allows for temporary closure to avoid conflict with hopper lifting and loading operations. The vee-traps do not allow fish entry into unsafe areas such as behind or under the hoppers.

A full-sized hopper is located inside the sump. Hopper freeboard, the distance from the water surface in the hopper to the top of the hopper bucket, is greater than the water depth within the hopper, to reduce risk of fish jumping out during lifting operations. Fail-safe measures are provided to prevent entry of fish into the holding pool area to be occupied by the hopper before the hopper is lowered into position. The hopper interior is smooth, and designed to safeguard fish.

The lamprey tank and trapping mechanism have not yet been developed. Generally, the lamprey tank is expected to be about the same size as a resting box – 4 feet wide by 5 feet long by 3 feet deep – and constructed out of smooth steel. The trapping mechanism is expected to simply be a 1- to 2-foot drop from the lamprey ramp into the tank. Water would be supplied at the upstream end of the lamprey ramp, allowing some water to fall into the tank and some to continue down the ramp, thus providing a continuous hydraulic path for lamprey into the tank. This type of trapping mechanism has proven to be successful for the U.S. Army Corps of Engineers in trapping lamprey (USACE 2015).

The motor lifts all the fish trapped in the hopper by the trapping mechanism over 80 feet vertically to a flume that leads to the handling and transport facility. The same water supply that feeds the fish lift and fish ladder also supplies the flume. The flume is sloped down toward the holding galleries.

The gantry crane extends over the adult and juvenile fish ladder upwells and the lamprey upwell. The hoppers and tank are aligned along their centerlines. The gantry crane is centered over the common centerline of tanks and hoppers. Cables from the gantry crane attach to the structural supports at the top of the hoppers and tank during the lift process. A motor lifts all the fish and lamprey trapped in the hoppers and tank by their respective trapping mechanisms over 30 feet vertically. The end trucks drive the bridge girder, trolley frame, hoist, and hopper or tank horizontally to its required destination.

The gantry crane extends across the access road to allow the hoppers and tanks to be directly mounted to or unloaded into the transport trucks. Fish from the juvenile/resident hopper will be moved to a truck-mounted transport tank via direct water-to-water transfer with the aid of the gantry crane. Lamprey tanks will be deposited directly onto transport trucks by the gantry crane. The adult fish hopper will normally be lifted in-place to facilitate transfer of the fish into the holding gallery. A motorized lift mechanism is triggered automatically according to a user set frequency programmed into the programmable logic controller. The lift mechanism raises the adult hopper about 30 feet to an elevated flume at the upstream end of the holding gallery. A mechanical trip automatically opens a door on the hopper and safely releases fish into the wetted flume. Inside the flume, fish travel by gravity to the holding gallery. The fish lift system will also have provisions allowing the adult hopper to be moved over the road for water-to-water transfer directly into a truck-mounted tank.

3.1.3 Sorting Facility

3.1.3.1 Holding Gallery

The holding gallery is provided for adult salmonids and steelhead. It is part of the sorting facility and is located north of the adult hopper sump. The holding gallery and adult hopper centerlines are coincident.

Aligning the holding gallery in this way allows for the hopper to be emptied into the holding gallery directly, via flume, without any need to move the hopper horizontally.

The holding gallery is an elevated concrete structure, sized to hold the estimated peak daily fish run for up to 24 hours. It has a separate water supply and drain system. The holding gallery includes provisions to minimize adult jumping that may result in injury or mortality. High freeboard on holding pool walls (5 feet or more) reduces the ability of fish to detect movement above the holding gallery. Sprinklers above the holding pool water surface will be used to minimize jumping.

The holding gallery has a motorized crowder to encourage fish to move towards the exit of the holding gallery, over the false weir, and into flumes that will carry them to the sorting building. The crowder has a maximum clear bar spacing of 7/8 inch. The side gap tolerances do not exceed 1 inch. The holding gallery also has side and bottom seals sufficient to allow crowder movement without binding, and to prevent fish movement behind the crowder panel.

A false weir is included at the upstream side of the holding gallery. The vertical flow provided through the false weir attracts fish to volitionally exit the holding gallery and supplies water to the distribution flumes. A bifurcation gate and distribution flumes route the fish to specific areas of the sorting building.

3.1.3.2 Sorting Building

The sorting building is located northwest and downstream of the holding gallery. It is an elevated building that contains flumes, tanks, tables, and other equipment necessary for the manual sorting and handling of adult salmonids and steelhead. The building is elevated to allow fish transport trucks to drive directly under the holding tanks for water-to-water transfer of fish. The bifurcation gate at the holding gallery directs fish to one of two visual inspection tanks via the distribution flumes. The operator determines where the fish should be held and directs the fish back into the adult primary transport flume. Fish are then sorted by actuated panel gates within the adult primary transport flume. Fish may be sent to an adult anesthesia tank and work-up station, any of three circular holding tanks, or bypassed directly to a transport tank located on a fish transport truck.

Anesthetized fish, after handling at the work-up station, are routed to any of the three circular holding tanks to allow monitoring of fish to ensure full recovery from the anesthetic effect prior to transport. Hydraulic conditions within the recovery tank ensure that partially or fully anesthetized fish are not impinged on an outflow grating or any other hazardous area.

The distribution flumes are used to route fish to anesthetic tanks, recovery tanks, pre-transport holding tanks, and directly into transport tanks located on a fish transport trucks. The flumes have smooth joints, sides, and bottom with no abrupt vertical or horizontal bends. Surfaces of the flumes are continuously wetted. Horizontal and vertical radius of curvature is at least 5 times the flume width, minimizing the risk of fish strike injuries. The minimum inside diameter of the distribution flume is 15 inches.

3.1.4 Transportation

All trapped target species are transported upstream of the dam, by truck, in transport tanks designed specifically for fish health and safety during transport. Transfer of trapped target species to the transport trucks takes place via the following three methods:

- Adult salmonids and steelhead are loaded into transfer tanks located on transport trucks via water-to-water transfer from overhead holding tanks located in the sorting building
- Juvenile salmonids and steelhead as well as resident fish are loaded into transfer tanks located on transport trucks stationed on the access road under the gantry crane; fish are moved from the hopper to the transport tank via water-to-water transfer
- Lamprey are trapped in their transport tank at the upstream end of the lamprey ramp; lamprey tanks are lifted out of their sump and placed directly on transport trucks waiting on the access road by the gantry crane

Water-to-water transfer is achieved through a vertical collar fixed to the bottom of the juvenile/resident hopper and each circular adult holding tank. The collar allows for some vertical and horizontal adjustment such that the collar can be locked to the receiving port on the top of the transport truck. The vertical collars and transport truck tanks are designed to minimize fish handling stress.

Fisheries personnel attach a vertical collar that extends from the bottom of the circular tank to the tank on the transport truck. The vertical collar has a minimum horizontal cross-sectional area of 3 square feet, and has a smooth transition that minimizes the potential for fish injury. Tanks on the transport trucks are completely filled with water prior to connection to the collar. After the connections are made, the collar is also filled with water to facilitate water-to-water transfer of the fish. A horizontal gate in the floor of the hopper or tank is opened, hydraulically connecting the hopper or circular holding tank to the tank on the transport truck. Water in the transport truck tank is slowly drained through juvenile criteria screens in the tank wall. The truck operator controls the rate of draining. This water is drained directly onto the pavement of the loading area or access road where it is carried via a floor drain or ditch, respectively, back to the river. As water is drained in the truck tank, the water elevation in the hopper or circular tank slowly recedes. The slopes in the floors of the hopper and circular tanks ensure that all the fish exit the hoppers and circular tanks through the floor gate and into the transport tank. After the water level is drawn down to the top of the transport tank, a gate in the floor of the circular tank is closed, the empty vertical collar is detached from the truck tank, and the door to the truck tank is sealed. An oxygen gas supply is provided to each transport tank to ensure proper dissolved oxygen levels are maintained in the tank during transport.

The transport trucks move the adult salmonids and steelhead, juvenile and resident fish, and lamprey upstream to predesignated points of release. Potential release points have not currently been sited and will be located in future phases of design development.

Figure 3-2
Water-to-Water Transfer from Holding Tank to Transport Truck, Courtesy of Puget Sound Energy

3.1.5 Water Supply

Multiple design elements of the CHTR fish passage facility require water to operate. Some design elements are supplied via gravity while others must be pumped. Table 3-1 identifies the water source and supply method for each design element that must be supplied with water.

Table 3-1
Water Supply Sources

DESIGN ELEMENT	WATER SOURCE	SUPPLY METHOD
Adult AWS	Impoundment pool	Gravity
Juvenile AWS	Impoundment pool	Gravity
Adult fish ladder	Stilling basin	Pumped

DESIGN ELEMENT	WATER SOURCE	SUPPLY METHOD
Juvenile fish ladder	Stilling basin	Pumped
Lamprey ramp	Stilling basin	Pumped
Sorting facility	Stilling basin	Pumped
Intake backwash system	Stilling basin	Pumped

Auxiliary water is provided to the adult and juvenile fish ladder entrances from the impoundment pool via a screened pipe penetration in the right bank wall of the right dam conduit, upstream of the conduit gate. The 6.5-foot-diameter, steel AWS pipe is encased in the concrete of the dam and buried adjacent to the fish ladder entrance. It carries the auxiliary water to an energy dissipation structure adjacent to the diffuser chambers at the fish ladder entrance. An energy dissipation valve located in a deep upwell within the energy dissipation structure reduces excess energy in the flow to bring the water to the elevation desired to supply attraction water to the fish ladder entrances. The energy dissipation valve is an automated valve that will adjust to meet the desired flow by burning more or less energy based on the reservoir elevation and the water level needed in the fish ladder entrance pools. Diffuser baffles for the AWS evenly distribute the flow to the fish ladder entrances. The baffles will be hydraulically tested at start-up and set once. No additional adjustment of the diffuser baffles should be necessary.

Water supply for the adult fish ladder, juvenile fish ladder, lamprey ramp, sorting facility, and intake backwash system is provided by a pump station and intake located upstream (south) of the adult fish ladder entrance. The intake draws water from the stilling basin through a set of fish screens meeting NOAA Fisheries' juvenile criteria. The screens total 26 feet wide by 8 feet tall. A water backwash system is located behind the screens to provide automated, active cleaning of the intake screens during pumping operation. The backwash system consists of a series of pipes carrying high pressure water from the backwash pump at the east end of the pump station to spray nozzles directed at the back side of the screens. High pressure water jets agitate debris collected on the screens push it back out into the stilling basin. Baffles are located behind the backwash system to control the flow through each screen section. The baffles will be balanced after installation to ensure a uniform approach velocity across the face.

The pump station is located at the east end of the intake. The pump station currently contains five vertical turbine pumps. The number of pumps used to supply water to each design element will be refined in future phases of design development. Each pump is located within its own pump bay. Pump bays are separated by internal walls that run from the east wall toward the intake screen, but only extend about half the length of the intake. The pump bay walls serve to provide straight, uniform flow to each pump. The pump station is designed to Hydraulic Institute (2012) standards.

The adult fish ladder, juvenile fish ladder, and lamprey ramp will be supplied by a single or a set of pumps with a single backup pump. The sorting facility will be supplied via a single or set of pumps. A backup pump will not be provided for the sorting facility water supply. A single pump will be provided to supply water to the backwash screen cleaning system for the pump station intake screens.

A common 36-inch-diameter pipe buried under the access road, parallel to the fish ladders, carries supply water to the adult and juvenile fish ladder upwells described in Section 3.1.2 and to the lamprey ramp. Pipes tee off the 36-inch fish ladder supply pipe to supply the upwells and lamprey ramp. The fish ladder supply pipe continues past these tees toward the river where a blow off is provided for safety and for drainage during maintenance activities.

An 18-inch-diameter pipe is also buried under the access road. It runs parallel to the 36-inch-diameter supply pipe to supply the sorting facility. At the holding gallery, a 12-inch-diameter pipe tees off of the sorting facility supply pipe to provide water to the Holding Gallery. Downstream of the Holding Gallery the 18-inch-diameter pipe reduces to a 12-inch-diameter pipe that runs toward the sorting building. At the sorting building the pipe branches to feed the multiple water needs of the sorting facility. A booster pump at the mechanical/electrical building provides higher pressure water to the sorting facility for maintenance and other needs. Similar to the fish ladder supply pipe, the sorting facility supply pipe continues, buried, past the supply branches to the sorting facility and toward the river. A blow off is provided for safety and for drainage during maintenance activities.

No potable water or sewer is provided at the CHTR facility.

3.1.6 Mechanical/Electrical and Storage Building

A prefabricated or concrete masonry unit building is located adjacent to the sorting building to house mechanical and electrical equipment and provide storage for equipment and materials associated with the CHTR. The buildings are secured facilities with outdoor lighting to reduce the risk of vandalism or theft. The building is separated into two sections. The mechanical/electrical section houses the transformer, distribution panel, circuit breakers, programmable logic controller, alarms, and other related items necessary for the performance of the CHTR. It will be accessible via a personnel door. The larger section provides storage for equipment and materials. Access to the storage building will be provided via a personnel door and a roll-up garage door. Interior access will not be provided between the two sections of the building in compliance with fire code. To access one section of the building from the other, personnel must exit the building then reenter.

3.2 Theory of Operation

3.2.1 Water Supply and Discharge

Water supplied to the adult fish ladder, juvenile fish ladder, lamprey ramp, and sorting facility is provided via gravity from the reservoir and via a pump station that draws water from the tailrace. Water will be supplied to these design elements continuously throughout the entire impoundment period from either water source. Pumping supply water from the tailrace ensures that these design elements may continue to operate regardless of how the impoundment pool is operated. However, when water is not provided by gravity, some water must be passed through the conduits to the stilling basin in order to supply the pump station. Additional information regarding operation of the gravity and pumped water supplies is provided in Section 2.1.2.3.

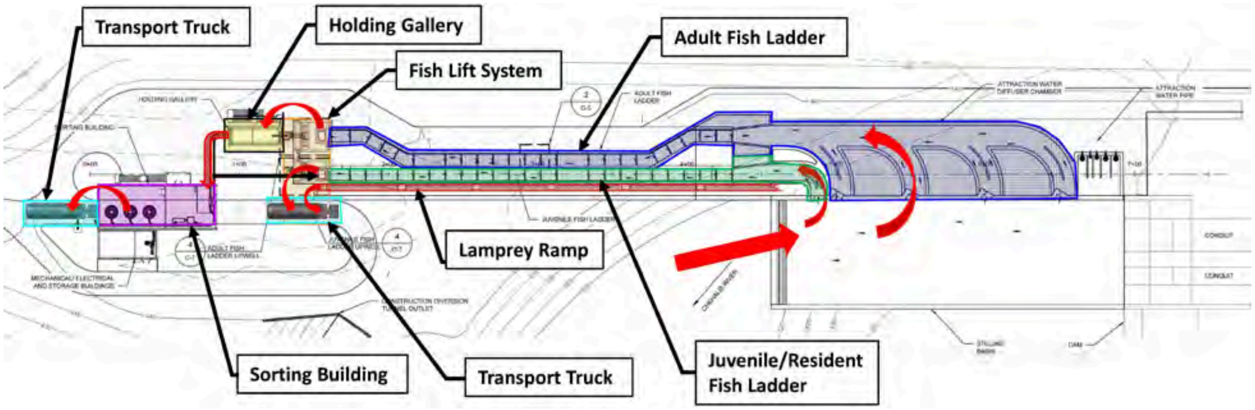
All the supply water for the CHTR elements (e.g., ladders, holding and sorting facilities, and ladder entrances) is discharged to the Chehalis River. Water supplied to the adult and juvenile fish ladders is combined with adult and juvenile AWS water at the adult and juvenile fish ladder entrances to provide adult attraction flows and conditions at the fish ladder entrances. Water from the fish ladder entrance gates discharges to the stilling basin. Supply water to the lamprey ramp discharges to the juvenile fish ladder entrance, where it joins the juvenile fish ladder and AWS water supplies. Water supplied to the sorting facility is not discharged to the stilling basin. Lamprey use a number of chemicals and pheromones to identify productive spawning habitat, coordinate spawning behaviors, and avoid risk (Buchinger et al. 2015). Also, anecdotal observations of lamprey behavior in fish ladders at Columbia River dams suggest that Pacific lamprey will not enter a confined channel that contains a concentrated scent of predator fish species (e.g., white sturgeon) or humans. Given this information, the Fish Passage Technical Subcommittee agreed that drainage water from the sorting facility should not be placed in the stilling basin near the ladder entrances and CHTR facility. The subcommittee was concerned that human and fish odors would be recirculated to the lamprey ramp water supply during the periods when the supply water was being pumped from the stilling basin. It was decided that water discharged from the sorting facility should be separated from other facility discharges and be returned to the Chehalis River downstream of the stilling basin.

3.2.2 Collect, Handle, Transfer, and Release

Target and resident species choosing to move upstream during impoundment events are collected, handled, and transported upstream for release via the CHTR. All species pass from the Chehalis River into the stilling basin. From the stilling basin, adult salmonids and cutthroat trout enter the adult fish ladder via the adult entrance gates while juvenile salmonids and cutthroat trout, resident fish, and lamprey enter the juvenile/resident fish ladder entrance via a single gate.

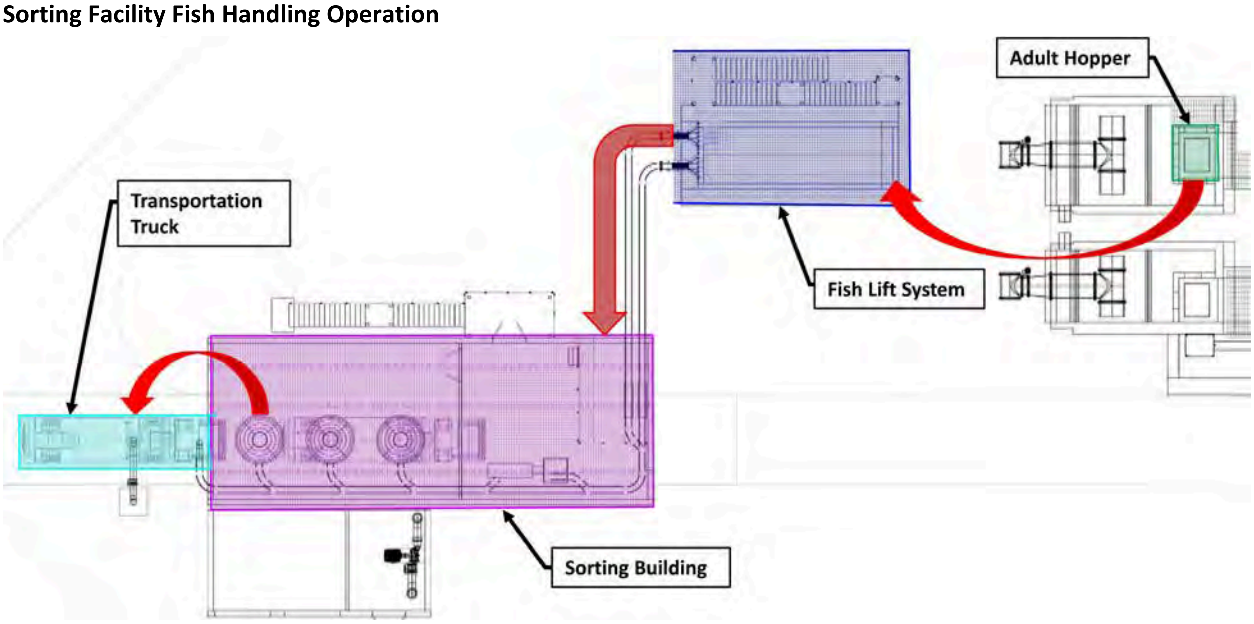
Lamprey exit the juvenile/resident fish ladder entrance prior to the first fish ladder weir and enter the lamprey ramp. Juvenile salmonids and cutthroat trout and resident fish ascend the juvenile fish ladder where they are trapped in the hopper at the top of the fish ladder. Similarly, lamprey ascend the lamprey ramp where they are collected in a holding tank. Upstream migrating juvenile salmonids, cutthroat trout, and resident fish as well as adult resident fish and lamprey will not be sorted. These species and life stages will be trapped in hoppers at the top of the juvenile/resident fish ladder and lamprey ramp, be transferred directly to transport tanks, and then transported upstream, as shown in Figure 3-3. Provisions will be provided for juvenile fish, adult resident fish, and lamprey that ascend the adult fish ladder and enter the sorting facility. They will be sorted and placed in their respective hoppers for regular transport.

Figure 3-3
Fish and Lamprey Collection and Handling



Upstream migrating adult salmonids and cutthroat trout will be collected in a hopper at the top of the fish ladder and moved through the sorting facility to transport trucks, as shown in Figure 3-4. The adults are moved from the hopper to the holding gallery using a fish lift system. These fish will stay in the holding gallery until personnel operating the CHTR crowd them to a false weir that guides the fish to enter a flume leading to the sorting building. In the sorting building, fish will be sorted and examined, if needed, then routed via flumes into holding tanks. Transport trucks will drive through the loading area directly beneath the holding tanks. Fish will be transferred from the holding tanks to a transportation truck via water-to-water transfer. The transport trucks will transport and release the fish upstream either on a routine schedule when fish numbers are low (e.g., daily) or when the holding tank on the transport truck reaches capacity during peak fish migration periods.

Figure 3-4
Sorting Facility Fish Handling Operation



3.2.3 Maintenance Schedule

Regular maintenance of the CHTR will be required. Maintenance activities can be broken into regular and infrequent periods.

Frequent maintenance is required throughout periods of operation. Frequent maintenance is anticipated to include the following:

- Frequent debris removal within the fish ladder and fish lift areas. Supply water is screened so debris is expected to come primarily from falling debris, such as leaves
- Frequent cleaning of tanks, holding galleries, work-up tables, flumes, and other holding and sorting facilities to ensure fish health

Many features of the CHTR require maintenance on a more periodic basis. Annual or semi-annual inspection and/or maintenance to exercise equipment and keep them in good working order will be necessary to prolong their functional life. Periodic maintenance tasks may include the following:

- Service manual and motorized gate operators
- Service trapping and lift systems, including repainting steel parts and service motors
- Inspect, service, and replace if necessary, all gates, crowders, and other fish handling and sorting equipment pumps
- Inspect fish screens and trashracks for damage

Infrequent maintenance primarily refers to maintenance that is conducted as needed. It may be several years between such maintenance activities. Infrequent maintenance activities may include the following:

- Repaint buildings and steel pipes and structures
- Replace trash rakes
- Replace gate operators, trapping system components, lift system components, crowder motors, and other fish handling and sorting equipment
- Replace water supply pipes outside of the dam penetration as necessary due to damage from exposure, debris, or floods
- Replace energy dissipation valves, baffles, screens, and other shutoff and control valves as necessary

4 COST DATA

4.1 Cost Summary

An opinion of probable construction and O&M costs were developed for the CHTR facility. Estimates of cost are largely based on unit price estimates developed from experience on similar projects. Cost is broken into two categories: construction and O&M.

Construction cost is greater for the refined CHTR facility presented in this document compared to the FRO CHTR facility in the previous fish passage evaluation of fish passage alternatives (HDR 2017). The change in cost is primarily due to the addition of specific functions and design features to address project goals brought forward by participants of the Flood Damage Reduction Committee during the 2017 CHTR facility design activities. As described in more detail earlier in this document, design features added to the project which impact both O&M and construction costs include facilities to trap and transport juvenile fish and lamprey, a dedicated fish ladder for juvenile and resident fish, and a dedicated ramp and collection facility for adult lamprey. Refined construction costs also include updated landslide mitigation measures on the right bank above the CHTR facilities, which are anticipated to represent a substantial cost item. The updated middle range estimated construction cost for the CHTR facility is estimated to be $43.0 million, with a lower bound of $32.3 million and an upper bound of $64.5 million. A detailed summary of the estimated opinion of probable construction costs is provided in Appendix C.

The CHTR facility is a fully manned facility that requires several personnel to operate. When in operation, two technicians and one truck driver would be required, 7 days a week. Additional skilled staff would be needed during periodic maintenance activities. However, operation of the CHTR is of short duration and is episodic, only requiring operation during infrequent impoundment events on average, once in every 7 years. While the infrastructure for the CHTR increased substantively from the CHTR presented in the previous fish passage evaluation (HDR 2017), the increase in O&M effort and associated full time equivalent personnel is anticipated to be minimal due to infrequent use and a shorter, 4-week operational period. As such, the estimated O&M cost for the CHTR facility for the FRO and FRO-X dam (operated as run-of-river) is approximately $20,000 per year, similar to the last concept design iteration. A detailed summary of the estimated O&M costs is provided in Appendix C.

5 REFERENCES

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Appendix A
Fish Passage Technical Committee Meeting Notes

FISH PASSAGE TECHNICAL COMMITTEE MEETING NOTES

Date: February 15, 2017
Time: 9:00 am to 12:00 pm
Location: HDR Olympia Office
Participants: Mike Garello (HDR), Matt Prociv (HDR), Anna Mallonee (HDR), Eric Orton (HDR), John Ferguson (Anchor QEA), Justin Allegro (WDFW), Don Ponder (WDFW), Jeff Brown (NMFS), Mark Mobbs (Quinault Indian Nation), Miranda Plumb (USFWS)

Meeting Notes

1. Updates on action items already in progress
   A. The group discussed progress on comments regarding the Fish Passage section of the Draft Combined Design Report. A few subcommittee members mentioned that they had not received the draft report or that the SharePoint link for the report had expired.

      ACTION ITEM: John to email download instructions to Fish Passage Technical Subcommittee members so they can download the Draft Combined Dam and Fish Passage Concept Report and provide comments.

      1) Justin mentioned that comments have been provided on the PEIS only. They are being responded to now. Some comment responses have already been provided.
      2) Mark explained that the Quinault Indian Nation did not have people who have worked on specific areas comment on those sections; they provided independent reviewers. Members participating in subcommittees did not comment on documents produced within the discipline they were participating.
      3) While discussing the overall project schedule, Miranda questioned how the current work would feed into the PEIS process. Justin responded that the report is being completed now and will not be included in the PEIS but reflects work we've done and will be useful for moving forward during the next biennium.

   B. Final estimates for population abundance were discussed to confirm numbers to be used by the design team. Estimates have not been made for cutthroat, resident fish, and lamprey. Abundance values for adult for Coho, fall and spring chinook, and steelhead need to be finalized.

      ACTION ITEM: Justin to finalize adult population abundance estimates and provide on WDFW letterhead similar to Mara's memorandum provided for juveniles salmonids.

2. Introduction to Miranda Plumb, a biologist with USFWS. She is part of the Chehalis Habitat work group, the ASRP, and other Chehalis Basin Strategy work groups.
   A. Miranda provided comments on the PEIS but she has not been part of the fish passage technical committee or read the combined Dam/Fish Passage report. John will provide

SharePoint access to the documents. Mike welcomed Miranda to the design team and stressed the importance of having her agency's perspectives on the team going forward.

B. Mike pointed out that Jeff Fisher and John Ferguson reached out to USFWS multiple times over the course of Fish Passage Technical Subcommittee meetings. It is possible that they were not the contacting the right person in USFWS. Despite our efforts there has not been any USFSW participation in the fish passage subcommittee meetings until today.

3. Review of the overall approach and objectives for CHTR Facility (FRO vs. FRFA)
   A. John pointed out that a possible hybrid FRO/FRFA Dam is being considered. It would operate as run-of-river but could operate as FRFA later if necessary. Because of the many similarities between the CHTR designs for the FRO and FRFA, and given the potential for the hybrid alternative, the 30% preliminary design of the CHTR will focus on one design concept reflecting the fish passage needs of the FRO dam design alternative.
   B. Mike informed the subcommittee of the June 30 due date for the 30% design level drawings of the CHTR facility.
      1) John will discuss today's schedule of required agency input.
      2) For the design of the CHTR for the FRO dam to a 30% level, Jeff does not advocate for incorporating juvenile salmonid and non-salmonid species passage facilities into one combined CHTR design at this time. One possible solution would be to incorporate them as separate stand-alone facilities to ensure that the passage performance for adult salmonids is not affected by the inclusion of an entrance and facilities for juvenile salmonids and non-salmonids fish species.
      3) John explained how the current effort of the fish passage design team - the 30% preliminary design of the CHTR - will be incorporated into the project-level EIS, not the current PEIS.
      4) Mike informed subcommittee members of CHTR facility designs, mentioning that we will discuss FRO/FRFA CHTR facility similarities today and in future meetings to reduce duplication of work if the dam types are changed in the future.
   C. The passage issues associated with FRO that still need further discussion and assessment in this biennium and the next include:
      1) How to design the CHTR (ongoing; 2015-2017 biennium)
      2) How to design the downstream weir that backwaters the outlet tunnels to accommodate fish passage; installing a notch was discussed as the likely solution in the future
      3) Verify passage conditions through the FRO outlet tunnels when flow is <2,000 cfs in the future
      4) Evaluate passage conditions through the FRO outlet tunnels when flow is >2,000 cfs in the future
      5) Evaluate passage conditions through the outlet tunnels during flood storage and when 300 cfs is being released through the tunnels in the future
   D. Attendees discussed FRO operations. The team projected and reviewed the graph of a storm hydrograph with the proposed flow, reservoir elevation, and time overlaid on the same graph developed as part of the FRO operations plan.
      1) While discussing the FRO drawdown strategy and reviewing the hydrograph slide, Jeff suggested using Fall Creek as a potential data source for identifying how drawdown influences juvenile and lamprey passage. Fall Creek in the

Willamette basin performs seasonal drawdown at the end of summer to return the river to a run-of river condition. There may be data on how juvenile salmonids in the reservoir survive when passing through the dam structure at drawdown.

2) Miranda and Don posed questions about the pressure differential for fish that go through throttled (FRO) gates.
    a. Mark continued the conversation by asking: Do residents move into tributaries or higher in reservoirs because of pressure due to 300' of water when they are used to under 15''?
    b. Mike mentioned that literature has shown salmonids generally do not sound more than 40-50 feet. This is what we have assumed in development of these passage alternatives.
3) Miranda brought the following questions to the discussion: What will stop fish from going through FRO gates at high flows during drawdown? Especially other species like lamprey, sculpin, sucker feeders, etc.?
    a. Miranda asked the group if there are there studies from other reservoirs.
    b. John reviewed available information on juvenile lamprey passage through turbines at Columbia River dams and pointed out that in our design review efforts to date we have focused on passage of juvenile salmonids through the FRO outlet tunnels.
4) Miranda asked, will bottom feeders sit at the bottom, move higher, or move to tributaries?
    a. Mike responded that we are uncertain whether they will migrate away from the deeper portions of the reservoir to maintain exposure to acceptable pressures, or if they will expose themselves to greater depths and be close to the outlets. This topic would still need to be investigated.

E. While discussing a few of the outlined pieces of the hydrograph, Jeff asked the design team: How is the 300 cfs getting through when the FRO conduit gates are closed?
    1) Mike responded, saying that the gates are not fully closed but throttled down to only allow 300 cfs to be discharged.

F. Next, the group reviewed a chart showing the frequency of flood closures that also represent the durations of CHTR facility operation.
    1) Mark asked if the operational curves use data over the last 20 years. Yes.
    2) Justin asked if the predicted forecast at Grand Mound gage also take into account where the precipitation is going to fall within the watershed. Yes.
    3) Mike explained that the model already accounts for storm center location. Larry Karpack, WSE have prepared technical documents that address the basin modeling methods.
    4) Don asked, what is the 32 days of the FRO conduits being impassable that keeps being brought up? John referred participants to the PEIS Appendix that contained the predicted flood events which triggered gate closure.
        a. In response, Mike explained how the days listed for each month aren't necessarily one single, long event for that month (of the table shown on the PowerPoint (slide 8) being viewed).

b. Don suggests adding a column to show events and/or dates of events.
c. Miranda recommends the design team considers adding information on where the storm centers occur (i.e. – headwaters, Newaukum, etc.).

G. As the group went over a brief explanation of dam operations for both FRO and FRFA Dam options, Miranda asked if there is there research or information about how species fare in long tunnels with similar substrates.
   1) John pointed out that the tunnels are set to grade, are backwatered, and are normally full of sediment. The tunnels are very similar to naturally bedded culverts.
   2) Mike explained how there is limited data for non-salmonids. The best analogy for the tunnels is large culverts under roads, and there is quite a lot of information on fish passage through these types of conduits (e.g., bottomless arched culverts).

H. While discussing details of the fish ladder entrance, Miranda posed a question regarding the FRO conduits, asking: what has been modeled to figure out how fast sediment reappears after sluicing wipes it away?
   1) Miranda also asked how fast sediment will accumulate after being evacuated.
      a. Mike explained that sediment begins to settle in immediately after conduit operations return to normal. Modeling suggests it could take months for the sediment in the conduits to return to equilibrium. The process is highly variable and depends on the types and durations of storm events that occur in the basin. Bedload begins to mobilize at flows of 4,000 to 6,000 cfs. Flows of that magnitude will need to occur upstream before natural bed substrate moves back into the conduits after a flood event.
   2) Miranda asked the group: how will the sediment accumulation process influence fish populations?
      a. Miranda expressed concern that resident fish may not navigate through the outlet tunnels without resting. A natural riverbed provides velocity refugia where resident fish can rest. What is the range of depths over the end sill? At station 108.216 for flows 250-2000 cfs, the depths over the end sill range from 2.99’ to 5.06’ with an average of 4.1’ over the range of flows.
      b. Justin reiterated that the fish ladder is in use while gates are closed, and sediment is cleared out from conduits and stilling basin which gives static condition at stilling basin.

I. Mark asks that the team review operations modeling data to see what years in the past FRO would have closed and for how long.

J. During a discussion on the fish ladder entrance and attraction flows, John asked the design team, does the outflow for the FRO all need to go through the conduits and stilling basin or could some go out through a low velocity fish ladder entrance?
   1) Mike explained that this has not been decided yet and further coordination with the dam design team is needed to address that question. More information relative to conduit and stilling basin hydraulics will become available as additional modeling is performed throughout future design efforts.

K. Jeff asked if flood events are rain only or rain on snow.

1) Matt stated that flood flows are typically generated by rain only events, and that the Chehalis basin typically occurs at lower elevations.

L. Mike informed the subcommittee that HDR will try to reduce the height of the fish lift as the concept moves forward. This can be done by increasing the amount of cut that occurs on the right bank. Likely, there will be changes to the facility orientation that will help resolve elevation difference as well.

M. Jeff recommends that the team consider having a shorter ladder for juvenile salmonids and resident species so they do not have to ascend the full length of the adult salmonid fish ladder.

1) Jeff offered that the team should consider a vertical slot fish ladder. It provides a swim-through condition and would accommodate the non-salmonid species.
   a. Mike told the committee that the design team did take a look at a vertical slot ladder. One tradeoff is that use of a 12- to 15- inch slot requires more water to operate. HDR will look at the feasibility of gravity water supply and use of vertical slot.
   b. DECISION: Participants generally favored Jeff's proposal to separate the low velocity entrance and collection system from the adult salmonid entrance, ladder and lift systems. The concept would include an intertie with the adult salmonid ladder and CHTR facility to allow fish to enter the low velocity entrance to access the adult fish ladder, using adult salmonid criteria for the intertie (i.e., 1 foot head differential over weirs).

ACTION ITEM: HDR will examine the feasibility of a juvenile entrance/ladder separate for FRO and FRO/FRFA hybrid dams.

2) Consider separate exit from fish ladder for fish that will have trouble ascending the full ladder at 0.7 vertical jump. This would require two traps and lifts – the current trap for adults and a trap for "weaker" fish after only a few pools. Need to allow a way for adult salmon and steelhead that enter the low velocity entrance to exit from the juvenile system and back into the adult ladder. The adult fish ladder can be set up for head differentials across the weirs that meet adult salmonid criteria.
3) A recommendation was made to design the facility as discussed, but componentize the elements so that the lamprey ramp or low-head entrance and juvenile/resident fish ladder can be dropped from the design at a later date if necessary.
4) John recommends the team gather more details about "possible adverse entrance conditions at low flow"?
   a. Consider fish energetic requirements.
   b. Consider predation.

N. While examining details of the fish ladder entrance, Miranda asked what the difference in required cfs is for the low velocity (juvenile salmonid and non-salmonid species) entrance and collection system and the adult salmonid CHTR.

1) Mike responded that the current fish ladder concept requires approximately 20-25 cfs and accommodates both juvenile and adults. Flow is based primarily on

leap height, velocity, and type of ladder. This concept has 0.7' of drop from pool to pool. If two ladders were required, flow requirements would increase. The adult ladder would be designed with 1.0' drops while the juvenile ladder would be designed with 0.7' drops. Attraction flow estimates are on the order of 200 cfs which will be dispersed in the adult ladder entrance through the use of diffuser chambers.

ACTION ITEM: Miranda to provide the design team with information and results of recent work by USFWS on sculpin passage.

O. While discussing the sorting station design for the potential CHTR facility, the following topics were brought up:
   1) Justin commented that WDFW will probably want to bring in their biologists for discussion on this portion of the design as they may be running the CHTR in the future.
   2) Jeff mentioned that the holding criteria from NMFS is soon to change.
   3) Mike commented that we do not need an automated sorting facility. The facility could be based on hand sorting to reduce cost and complexity.
   4) Don asked if all the fish go to the same release point.
      a. Mike explained that for the current design, all fish are released from one of four tanks into trucks via water-to-water transfer. At this point, the release sites have not been identified and is up for further discussion. Fish could be released above the dam in the mainstem river or transported to and released in tributaries above the FRO facility

4. Verification of target species and life stages (FRO/FRFA)
   A. Justin pointed out that at this point the 2017-2019 biennium budget request includes the following:
      1) Updating a salmon and steelhead population model to evaluate the effects of FRO impoundments at the population level (e.g., the effects of 3 FRO impoundment events/year versus the current estimate of 1 impoundment event/7 years).
      2) Funding for WDFW whitefish and lamprey density and distribution studies as well as snorkel surveys for lamprey distribution and abundance. The studies might contribute numbers of lamprey that would be useful to this design effort.
   B. USFWS pointed out that Miranda will focus on passage requirements of lamprey, sculpin, and other resident fish species.
   C. Miranda pointed out that resident fish present in the river include whitefish, 5 sculpin sp., and sucker sp.
   D. Justin asked if there is an assumption that resident species would not use the adult salmonid ladder. Yes, the leap heights and velocities present in the adult ladder would not be favored by juvenile salmonids and resident fish.
   E. While reviewing previous action items, specifically adult annual abundance, Jeff asked: Do we expect resident fish to be moving at flood flows? What do they do behaviorally?
      1) Mark responded that there might be some movement at the start of the hydrograph. While he is uncertain if that will be the case, resident fish will likely

be hunkering down but adult salmonids will be triggered to move with the initial increase of the hydrograph.

F. While discussing which species should be included in the design process for fish passage, Miranda informed the subcommittee that the Chehalis tribe is doing an eDNA study in the tributaries and dam area next year based on funding from USFWS. The study will include looking for bull trout.

G. Miranda and USFWS do not believe there are bull trout at or above the proposed dam location.

5. Receive initial comments and direction from the Fish Passage Subcommittee
   A. Mike informed the subcommittee that we have not yet received comments on the fish passage section of the combined design report. Comments received will be incorporated into the detailed fish passage appendix that is part of the final Combined Fish Passage and Dam Design Report. The consultant team is trying to distribute the draft fish passage appendix soon.

6. Schedule next meeting (s)
   A. Next meetings (2) will be at HDR Olympia office on:
      1) March 22nd at 11:00 AM to 2:00 PM
      2) April 19th 10:00 AM to 1:00 PM.

ACTION ITEM: Mike Garello to send out PowerPoint, meeting notes, and next meeting invite.

FISH PASSAGE TECHNICAL COMMITTEE MEETING NOTES

Date: March 22, 2017
Time: 11:00 am to 2:00 pm
Location: HDR Olympia Office
Participants: Mike Garello (HDR), Matt Prociv (HDR), Anna Mallonee (HDR), Eric Orton (HDR), John Ferguson (Anchor QEA), Justin Allegro (WDFW), Don Ponder (WDFW), Jeff Brown (NMFS), Mark Mobbs (Quinault Indian Nation), Miranda Plumb (USFWS)

Meeting Notes

1. Updates on Action Items already in progress
   A. The group discussed the distribution of the Draft Combined Dam and Fish Passage Concept Report. Every member of the subcommittee now has access to the report, and comments need to be received. John made a point that the group needs to create a concrete schedule for comments on the combined report.
      1) Per Mike's suggestion, the group agreed on a comment due date of April 7th, which will give the design team time to circle back on these comments before the next meeting (April 19).

   B. John and Justin are working together, separate from the subcommittee, to finish the adult population abundance estimates. This item is still in progress.

      <u>ACTION ITEM</u>: John and Justin to continue working on adult population abundance estimates together.

   C. HDR has looked at the feasibility of a separate juvenile entrance and ladder – the following presentation on the refined CHTR concept includes this action item.
   D. As an action item from the previous meeting, Miranda was to provide the team with USFW data on results of their passage work.
      1) Miranda has received information on sculpin (torrent) passage from a colleague at USFW that she thinks may be useful, although it is not entirely applicable. This study has information regarding the following: short jump height (6"), movement of migratory sculpin in the lowlands, and torrent sculpin movement.
      2) Miranda explained that her colleague mentioned the fact that fluvial species tend to be weaker than migratory, and that this should be taken into consideration.
      3) Miranda also did research on other studies regarding sculpin passage, and referenced a study done by the University of Washington on a specific species of sculpin. She pointed out that the study recognized water velocities greater than 4 m/s as a barrier for specific species of sculpin. Another study she found from 1969 indicates a 1 m/s barrier for other species of sculpin.
      4) John suggested matching current passage condition for this reach at a natural condition, as there is not much data on sculpin passage thus far.

E. Once discussion on sculpin data and research began, a more detailed discussion on sculpin continued.
    1) Miranda brought attention to the fact that we cannot limit sculpin to one area of the stream, as they are a benthic fish that feed on benthic invertebrates. This relationship greatly influences the food web in the area.
    2) Miranda also mentioned the fact that backwatering of the culverts can cause sediment buildup, causing higher velocities which create localized barriers for sculpin passage.
    3) Mark explained to the group that the important piece in designing the system for passage is to create one that is as close to a natural system as possible.
    4) John asked the group if we have design criteria on nature-like fishways.
        a. Mike responded, explaining that there is currently no criteria, only helpful documents and research from WDFW.
        b. Jeff brought up the fact that there might already be localized and natural "barriers" that fish are currently and effectively able to pass upstream. The most important criteria is then to design something that is also passable based upon the existing "barrier."
        c. John asked why the group would not compare tunnels and nature-like bypass systems.
            i. Mike responded that the design team has already adopted that approach when comparing the velocities in the proposed conduits and the existing river channel.
    5) Mark asked the group if we know of the seasonal or lifetime migration of sculpin, and what the movement needs of sculpin are. He also asked if, like some salmon, they move around just because they "want to"?
        a. Miranda commented that the torrent located in the Chehalis are resident fish, and that they do not migrate out. She also explained that they are migratory in the lowlands but she does not know for certain about the riverine species.

2. Review new information and refinements to design criteria or objectives
    A. The group moved on from previous action items and sculpin discussion to talk about new criteria to accommodate those additional resident species.
    B. Mike reiterated that there is a general interest to provide passage of these other resident species, and asked the group if the design team should include these other species as target species. He also asked the group to keep in mind that we are designing two things for this concept: the FRO conduits and the CHTR facility.
    C. John asked that the subcommittee recognize the fish passage scope for the remainder of this meeting (and the next) does not include the FRO conduits, and should focus only on the CHTR. He says that comments pertaining to the FRO conduits should be noted, but not explored at this point in time.
        1) Justin explained to the subcommittee that there is budget allocated in the 2017-2019 biennium to research lamprey and whitefish, and that some of that work in the next biennium would benefit the scope for this biennium.
    D. Mark recommends the group look at how well criteria for salmonids encompasses criteria for other fish that may be included in the target species.

E. John wondered what the group is thinking in regards to passage through the conduits, and how (and if) that thinking influences our work on the low velocity CHTR entrance.
    1) Jeff disagreed with John's original statement, explaining how the CHTR and conduits will not be run simultaneously, so it should not matter.
    2) John argued that they will be run simultaneously at 300 cfs, and that his main point comes back to the idea of keeping the CHTR facility in the front of our minds. We should not get lost in the conduits, as our focus needs to remain on the CHTR facility.
F. Mike explained to the group that what is really needed is a list of all species we must accommodate in our design. HDR can work on the 30% design and then the group can reassess what else must be included to do so.
    1) Justin added that while it is not in the group's scope, the project specific EIS might need information from us on what the impacts might be.
G. In regards to the list of species, Matt mentioned that he had gone through the 2014 Aquatic Enhancement Plan and created a list of potential species within the Chehalis River Basin.
    1) Miranda informed the group of the 2015 Fish Study, located on the strategy's website. She explained how this study should have a list of species, and that all of them should be considered in the design.
    2) Mike pointed out that the team should make sure the species we accommodate for are at the dam site, not just in the Chehalis.
        a. Miranda agreed, and directed the team to a graphic in the same document showing on a map the location of the species present.
    3) Justin suggested that once a list is finalized, WDFW should be contacted to confirm the list's accuracy.

<u>ACTION ITEM</u>: HDR to create a list of species and circulate to subcommittee for comments and confirmation.

    4) Matt says that he thinks the group needs to go over whether or not some species need to be accommodated based on their life history.
        a. Jeff asked, what variables can we actually change that will affect passage for these other species? He feels there is a limited extent to what we can do for these species.
        b. Mike said that research must be completed to possibly improve the passage for these species but we first need to figure out what these species are.
            i. Justin agreed, and added that the final step would be to circle back and see if the species is actually worth adding based on the life history.
    5) John suggested the team look at lumping of design criteria based on body types, sizes, swimming capabilities, life stages, jump height, etc.
H. Mark suggested the group look into the way fish interact with each other, especially at the holding facilities.
    1) John explained that at the moment, the group is looking at separate facilities with a connection. He also brought up that we are not yet making a

recommendation to put in a low entrance but a decision on this needs to be made. John also makes it clear that our notes must reflect what we are discussing in these meetings, and that they will influence the conduit design.
2) Jeff described the assumption that if fish are able to get through the higher velocities, they will be able to make it all the way up the ladder. This begs the question: what happens when these fish make it to the top?

I. The team reviewed the last technical committee's meeting notes. John cleared up a few items in those notes:
    1) Items discussed in the section pertaining to continued discussion and assessment of passage issues associated with the FRO were claimed to be done in the next biennium. John pointed out that while those tasks might be done in the next biennium, they are not technically within the budget and scope.
    2) Unless these items come up as needed for the project level EIS, fish passage design is not in the next biennium.
    3) Justin confirmed the lack of funding for fish passage in the next biennium, explaining that the budget for fish passage design work has been cut out.

J. While discussing the basic, simplified CHTR processes for adult and juvenile salmonids and lamprey, Jeff recommended adding release sites, as there is sometimes construction that must go along with these sites.
    1) Mike agreed, but touched on the fact that we must address the where before the how.

K. HDR presented the updated site plan for the CHTR facility, and Don asked the group if this configuration required less grading of the site.
    1) Mike explained that this new configuration would actually require more grading, but is a cleaner design that would require less slope stabilization. This configuration is closer to water level and is more efficient from an operational standpoint.
    2) The subcommittee appreciated this new site plan and preferred it over the previous iteration.

L. The thorough and more detailed process diagram was presented to the subcommittee, along with isometric views of the proposed system.
    1) Jeff asked if the holding tanks are in parallel or in series, or in some other configuration.
        a. Eric explained that the current system is a dual system, where fish are put in one of two visual inspection tanks, then they go into tanks (in series), and trucks are lined up under the tanks for water to water transport.
    2) John asked if the hopper is operated with a gantry crane.
        a. Eric explained that gantry crane operation is the current setup in the design.
    3) Jeff asked the design team if fish are metered out to the tanks or simply dumped into them.
        a. HDR responded that fish are dumped into appropriately sized tanks.
    4) The vertical hopper lift was questioned by multiple members of the subcommittee and was found to be 25'.

a. Jeff asked the team how water would be fed into the hopper. He also asked if there is an overflow weir, and how fish will get into the hopper. He explained that it must be sized large enough to get water to the bottom of the ladder.
   i. Eric responded that water would be provided to the ladder via diffuser screens at the bottom of the hopper well.
   ii. Mike explained that the fish are to swim nose up from the last baffle straight into the hopper.
b. Jeff suggested keeping water separate for juvenile and adult ladders,
   i. Eric explained that the drains from the adult ladders do not go into the juvenile ladders, and that the drain water goes back into the same system.
   ii. Mike added that the water does, however, combine at the entrances. The water remains separate throughout the ladder but not at the ladder entrances.
c. John informed the subcommittee that lamprey are particularly sensitive to human smell, and will react to it if they are exposed to waters touched by humans. He wanted to highlight this fact and explained that discharging the handled water to the entrances will create an issue. While he is not sure what to do about it, the concept must be discussed, as handling drain water into any ladder could prevent lamprey from entering the facility.
   i. Eric suggested dumping water further downstream.
   ii. Jeff commented that water supply is not a large issue in this design, and not something we will have to worry about when accommodating for the lamprey and handled water.

5) Miranda asked the group what is known about survival in juvenile ladders. She asked, is there something out there that can give us insight?
   a. Mike said that as far as we know, survival is incidental. Most studies or existing ladders do not focus specifically on juvenile ladders.
   b. Jeff explained how in Oregon, they have worked on juvenile passage and have been successful on a much smaller scale, although these juveniles do not use a separate entrance.
6) Eric asked about the separation of native fish, specifically if they all need to be separated, since all adults need to be handled.
   a. Mark made a point that the design team must be aware of the potential for listing chinook and cutthroat as target species.
      i. Jeff believes this possibility should not influence design much, and that the cross connections will take more work.
7) Mike informed the group of thoughts on putting in a bar rack for adults to exit the juvenile ladder.

M. While discussing the current lamprey ramp design, Jeff asked the team how big the opening is for 75 cfs.
   1) Mike explained that the opening would have to be rather large. Matt clarified by adding that it would be a 6x10 opening, or something on that order.

N. The design team then presented the auxiliary water supply site plan to the subcommittee.
    1) Justin asked if HDR had discussed the slope and grading with geotechnical engineers.
    2) Matt explained that for the AWS, the artificial assumption is that 50' of impoundment is required.
    3) The discussion continued into talk of water required for running a vertical slot. Jeff asked the team for clarification on whether or not 25 cfs is enough to run a vertical slot.
        a. Mike responded that 25 cfs is not enough for the 12" slot, and the 9" slot would be about 7-10 cfs.

O. In continuing the AWS conversation, the hydrograph for the 2009 storm was presented with detail pertaining to AWS flow, fish ladder flows, and minimum reservoir required for AWS flow.
    1) Justin asked what the delay in the front and back end of the AWS was pertaining to.
        a. Mike explained that the facility will not all of a sudden have enough water to operate the supply. HDR has not yet discussed with the dam group whether the water will be running through the FRO also or just the CHTR.
        b. Jeff commented that flow only going through the CHTR is best for attraction flow, and that a day or two of delay is not a "deal killer."
        c. Mike responded, saying that there will not really be a delay; everything will still be operating but the additional attraction flow will not be there.
    2) John asked what the reason for pumping was.
        a. Matt commented that HDR has looked at pumping 300 cfs a month.
        b. Mike explained that with just gravity, it is not possible to operate the facility ahead of a storm, and nothing can be double checked at any point unless you close the gates and create an impoundment.
        c. Don believes that gravity is still the best solution, and the facility should only use pumps when required, and only for as much as required.
        d. Jeff believes that gravity is the best solution. Suggests provide fish ladder flow by gravity too when possible.
        e. General team agreement for gravity water supply to the AWS as described.
    3) Mark asked the group if anyone had looked into the how often power may be interrupted in this area.
    4) Justin asked why the 50' measurement to the top of the FRO conduits was important.
        a. Matt explained that fish will probably not sound more than 50'.
        b. Mike added that there is less of a possibility of entrainment in the pipe and going into diffuser screens.

P. Following discussion of the AWS, 2D model results for velocity were shown for the FRO conduits and stilling basin, for flows 250 – 2000 cfs, with models for flows of 250 – 1000 cfs containing sediment and models for flows of 1250 – 2000 cfs containing no sediment.

1) Jeff commented that attraction flows appear to be quite good for the model showing velocities as 250 cfs. At 750 cfs, he commented that the high flows going into the stilling basin on river left are concerning. While looking at the 1000 cfs graphic, Jeff asked what the cross sectional area of the conduits was.
    a. There are two 10x16 conduits and a single 12x20 conduit (560 sq ft).
2) While reviewing the final graphic at 2000 cfs there was some discussion of changing the geometry of the conduits.
    a. John asked, what's the driving factor for the size of wall between the conduits?
    b. Mike explained that the larger conduit acts as a "workhorse" sluice, while the smaller conduits are mainly for flow throttling.
3) John asked the design team if attraction flow will come into play for the FRO conduits.
4) Mike asked the subcommittee if HDR had effectively captured all thoughts and main ideas that had been discussed previously.
    a. Jeff commented that the new facility appears to be very appealing.
    b. Mark said that the only issue he sees is the modeling and the increase of velocities at the downstream end of the conduits.
        i. Mike offered vertical differential as a potential factor for this velocity increase.
        ii. Matt explained that another possible factor could be that the sediment in the stilling basin is modeled as higher than the sediment in the conduits.
        iii. HDR will continue working to improve the models.
    c. Jeff commended the improvements to the site plan and the flexibility of the pump station. He asked the design team if it is feasible to operate the ladder and the entrances at any time.
        i. Mike explained that the pump station is located there for that exact reason.
    d. Jeff also asked if the pump station could be common to the AWS, so when there's 50' of head the pumps can be turned off.
        i. Mike explained that the idea was to connect the two for just that.

3. Discuss design tradeoffs based on refined concepts
    A. Tradeoffs for the refined concepts as well as any additional comments for the refined design elements were discussed after reviewing these refinements.
    B. Jeff believes that the adult standalone ladder is great for salmonids. He suggests considering a juvenile ladder with a vertical slot with less flow that would accommodate juveniles better than a half ice harbor.
        1) Mike commented that a 12" adult slot would be adequate, with a 9" slot for juveniles.
    C. John believes there are no tradeoffs, only benefits. He mentions the benefits as the following:
        1) Adding flexibility on how we operate (we are accommodating a lot of uncertainty for the design)

2) The separation of two systems (separation of water supplies for juvenile, adult, and lamprey; separation of pumped and gravity systems as necessary)
3) Including pump capacity at the tail end of flood storage
   a. Jeff asked the group if thought had been put into where the pump intake would be.
   b. Mike explained that HDR had discussed the circulation patterns in the stilling basin.

D. John explained that the low volume and the normal adult salmonid CHTR is feasible, and the feasibility of the structure is no longer in question.
   1) Mike mentioned that while technically feasible, there is uncertainty with the biomechanics involved, as a system like this has not been created before.
   2) John recommends the CHTR for inclusion (it's feasible), and comments that the team has been able to incorporate other species for as much information as we have to do so, and have built in flexibility into the system.
      a. Jeff said he would add that the current concept includes biomechanical certainty and follows convention for adults.
   3) John reiterates a main takeaway: the stilling basin hydraulics look favorable at first glance, and there appear to be generally good conditions up to 750/1000 cfs.

E. Jeff explained that he owed the group some probable holding criteria, and provided the following information:
   1) Short term criteria does not change.
   2) Long term criteria will be twice the current values.
   3) Temperature modifications are not concrete, one interpretation is that the system will break at 70 degrees, but there is activity pending on this issue.

F. Eric raised the issue of creating a more complex handling facility.
   1) Mike suggested adding a flume to separate the fish.
   2) Jeff suggested adding a return to river pipe in the sorting facility that would lead to transport holding tanks.

G. Jeff believes that the idea of a bar rack needs to be fleshed out for feasibility.

H. John mentioned that there is literature on resident fish passage.
   1) Miranda explained that the 2015 Study has a fairly short list.

ACTION ITEM: Miranda to pass on any sculpin references she comes across.

4. Next meeting and summary of action items

A. Next meeting originally scheduled for April 19 (10:00 AM to 1:00 PM), group discussed possibility of pushing meeting back.

ACTION ITEM: Mike Garello to send out PowerPoint, meeting notes, and next meeting invite.

B. The following action items were identified during the March 22 fish passage subcommittee meeting:
   1) John and Justin to continue working on adult population abundance estimates together.

2) HDR to create a list of species and circulate to subcommittee for comments and confirmation.
3) Miranda to pass on any sculpin references she comes across.
4) Mike Garello to send out PowerPoint, meeting notes, and next meeting invite.

FISH PASSAGE TECHNICAL COMMITTEE MEETING NOTES

Date: June 1, 2017
Time: 10:00 AM to 12:00 PM
Location: HDR Olympia Office
Participants: Mike Garello (HDR), Matt Prociv (HDR), Anna Mallonee (HDR), John Ferguson (Anchor QEA), Jeff Brown (NMFS), Mark Mobbs (Quinault Indian Nation), Miranda Plumb (USFWS), Jessica Hausman (WA Dept of Ecology)

Conference Call Information: 866-583-7984, 74660139#; Leader-Pin: 532659#

Meeting Notes

1. Updates on action items already in progress
   A. John and Justin to continue working on adult population abundance estimates together.
      i. If population abundance estimates are unable to be refined (and put on WDFW letterhead) by the end of this biennium, they should be finished in the next. Values for population abundance are not anticipated to change between the next biennium.
   B. HDR and Anchor to refine list of target resident species.
      i. This list is now located in the basis of design document (page 19 of the pdf).
      ii. Miranda acknowledged that a lot of the information for this list might not be included, or is unavailable.
      iii. John explained that in addition to the list located in the basis of design document, there is a spreadsheet that contains performance and swim speed data, jump heights for juvenile and adult species, and references where the data came from.

      ACTION ITEM: Matt to send spreadsheet containing performance and swim speed data out for review.

      ACTION ITEM: Miranda (and others) to give any extra information and feedback that could be added to the spreadsheet by June 9, 2017.

   C. Miranda to pass on any sculpin references she comes across.
      i. In progress
2. Review CHTR Facility Drawings and Design Details
   A. Identify and discuss any comments provided by subcommittee participants.
      i. Jeff suggests considering cross over between adult and juvenile ladders. They are currently “black boxed.” We may not have time to refine this design before the next biennium but it should remain a consideration in future design efforts.

a. Mike explained one idea is to have a bar rack allowing passage of smaller fish. This bar rack would have spacing that diverts larger fish to the adult ladder and allows smaller fish to pass through.
b. Group agrees that 1.25’ drop is a good target to shoot for that allows adults to exit juvenile ladder, and that 1” clear spacing should prevent gilling of adult salmonids.
c. John suggests making the spacing of the racks adjustable, allowing for flexibility in operations. If the bar rack did not work for any reason, it could be changed out to take advantage of behavior. It is difficult to judge conditions prior to implementation, but they can be addressed in the field.
    1. In Mike’s experience, having multiple sizes of racks is usually more cost effective. This cross over will be addressed very briefly, but cannot be vetted out fully at this point.

ii. Miranda wants to make sure the group is accommodating passage for as many species as possible. She understands that there is a lack of information and data on some species, but urges that accommodations are made to the best of our abilities.
a. Is downstream passage being considered? Yes – conduits provide downstream passage except during flood events.
b. In general, the group lacks data. Matt suggests documenting the things we do not know.
c. Mike explained that there are a variety of design elements that have been considered in an effort to accommodate a wider range of species, but that there is still a lot of uncertainty of motivation and behavior of fish.
d. Miranda agrees, believes the approach is sound given the information the group has at the moment.

iii. Mark questions if Olympic mud minnow need to be included in list of species.
a. Most have dismissed mud minnow as a species of concern.

<u>ACTION ITEM:</u> Matt and John to provide statement in document similar to: “based on information we are aware of, the assumption has been made that mud minnows are distributed further downstream and do not need to be included in this fish passage design.”

B. Discuss schedule for completing the final draft.

<u>ACTION ITEM:</u> All comments should be sent to John *and* Mike by June 9, 2017.

3. Discuss Fish Passage Design Tasks required in the next biennium to support ongoing fish passage project needs

A. John described that there is no budget for engineering design work in the next biennium, but the preliminary (internal) work plan items will be covered in the EIS component/budget. Important to keep line of communication open even without engineering design work.

B. Meetings of this subcommittee would be useful to:

   i. Bring back latest performance data (e.g. surface collector performance)

   ii. Cover any new literature on passage, even if it's simply sent out to the group

   iii. Have conversations and review numbers based on discussions and most up to date information

   iv. Look more closely at ascending and descending limbs of the FRO tunnels and develop a design for the FRFA CHTR facility

C. The group reviewed the preliminary (internal) work plan for the next biennium. The objective of this work plan is to get tasks into the EIS budget.

   i. Information for draft EIS due December 2017

4. Next meeting and summary of action items

   A. The following action items were identified during the June 1 fish passage subcommittee meeting:

      1. Matt to send spreadsheet containing performance and swim speed data out for review.
      2. Miranda (and others) to give any extra information and feedback that could be added to the spreadsheet by June 9, 2017.
      3. Matt and John to provide statement in document similar to: "based on information we are aware of, the assumption has been made that mud minnows are distributed further downstream and do not need to be included in this fish passage design."
      4. All comments should be sent to John *and* Mike by June 9, 2017.

Appendix B
Preliminary Design Drawings

1 2 3 4 5 6 7 8

CHEHALIS BASIN STRATEGY
FRO AND HYBRID DAMS,
CHTR FISH PASSAGE FACILITY
PRELIMINARY DESIGN
- DRAFT -

WASHINGTON STATE

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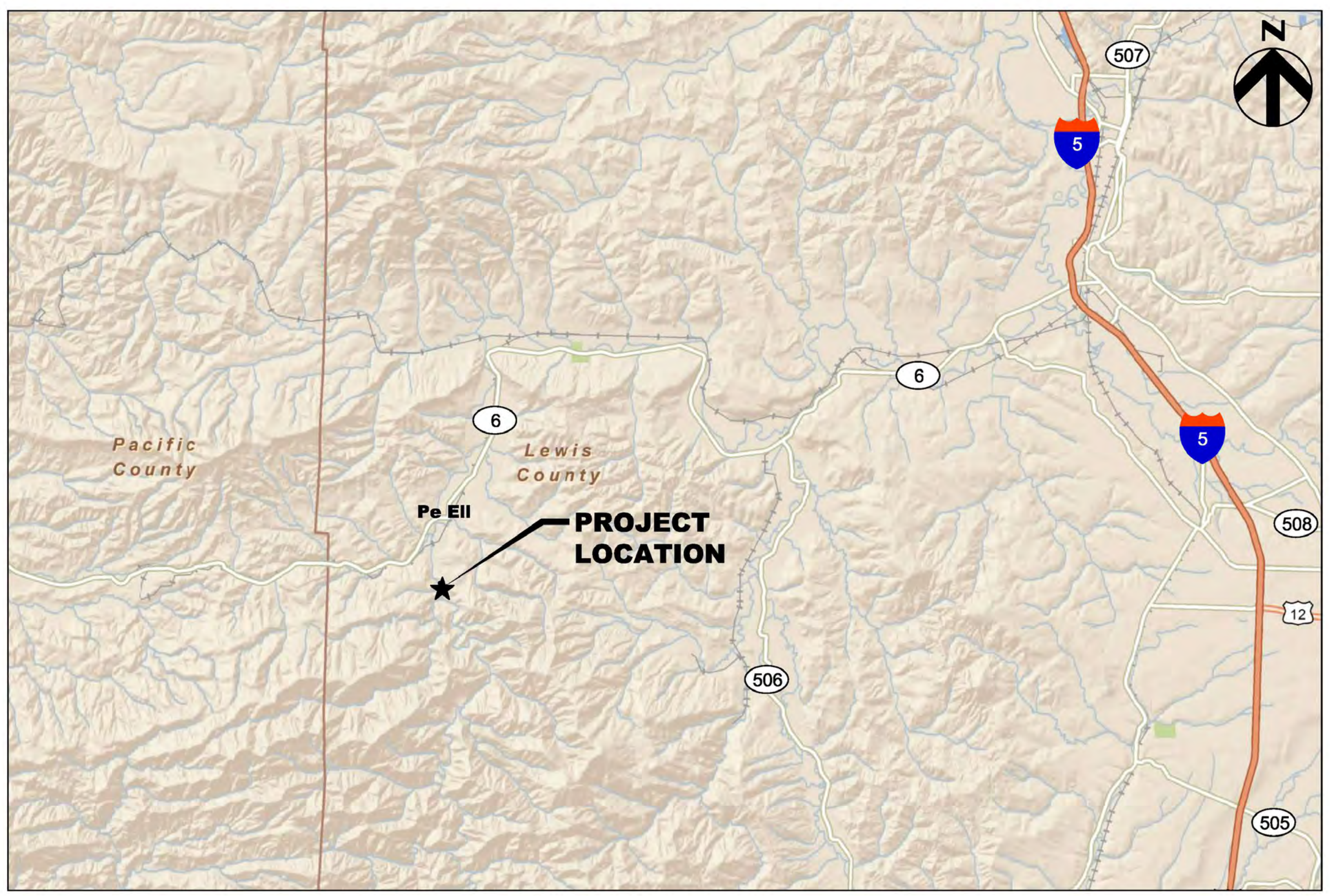
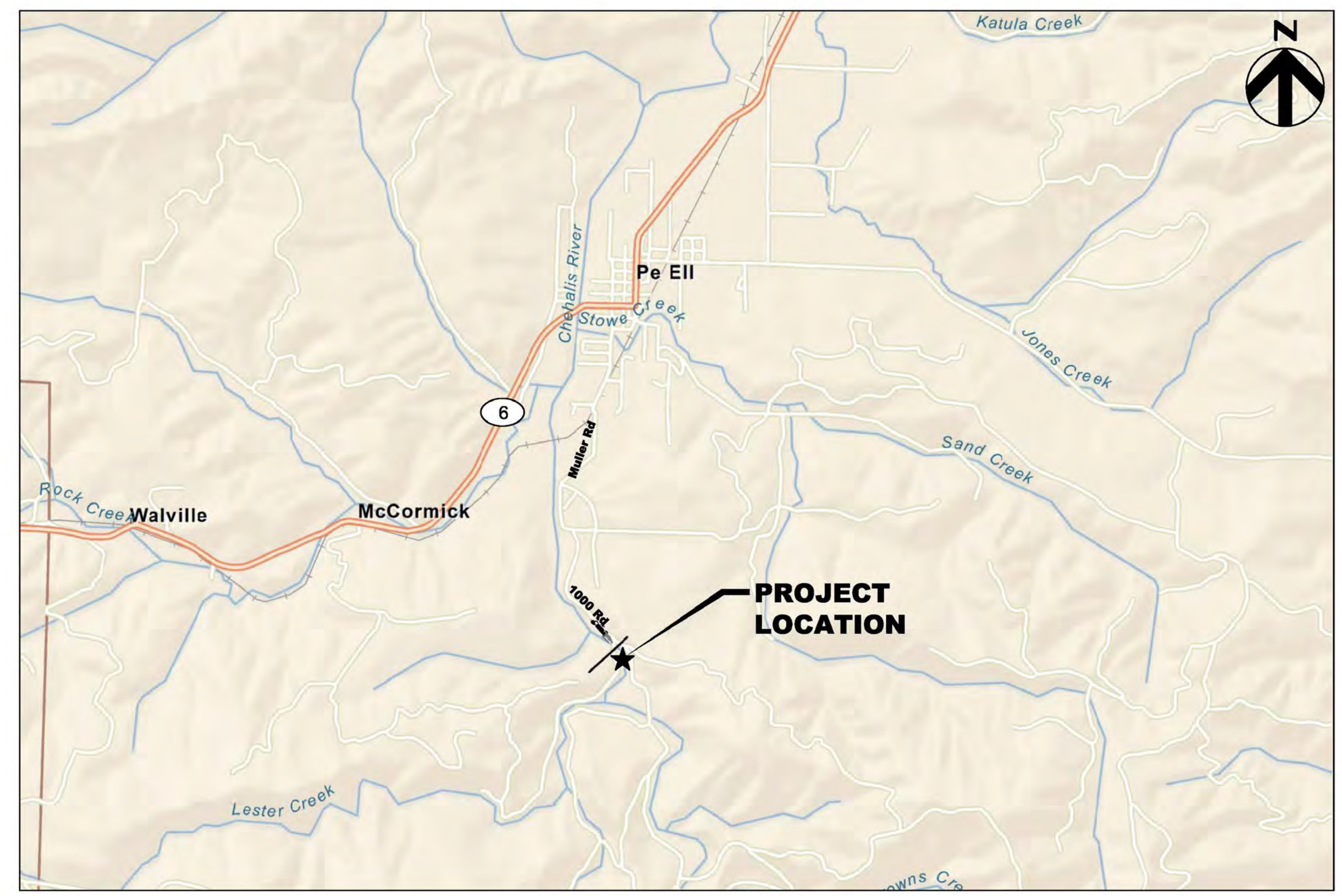


PROJECT VICINITY



PROJECT LOCATION

PRELIMINARY
NOT FOR CONSTRUCTION

HDR

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER	
CHECKED BY	MATT PROCIV
CHECKED BY	
DRAWN BY	DON DADE
DESIGNED BY	DON DADE
DESIGNED BY	
PROJECT NUMBER	268421

THE SEAL OF THE STATE OF WASHINGTON 1889

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

GENERAL

COVER SHEET, VICINITY AND LOCATION MAPS, AND DRAWING INDEX

0 1" 2"

DATE	JUNE, 2017
SCALE	NTS

SHEET G-1

A B C D

PROPOSED SITE PLAN SYMBOLOGY

- PROPOSED TEMPORARY FENCE
- BARBED WIRE OR CHAINLINK FENCE
- 100 YEAR FLOOD BOUNDARY
- PERMANENT EASEMENT
- TEMPORARY OR CONSTRUCTION EASEMENT
- PROPERTY LINES
- RIGHT OF WAY LINES
- RIGHT OF WAY CENTERLINE
- SEDIMENT FENCE
- CONSTRUCTION HIGH VISIBILITY FENCE
- B-4 SOIL BORING
- TP-2 TEST PIT
- CUT SLOPE
- FILL SLOPE
- CONTRUCTION STAGING AREA
- DIRECTION OF SURFACE RUNOFF

EXISTING SITE PLAN SYMBOLOGY

- MONUMENT
- MONUMENT IN CASE
- REBAR & CAP
- SCRIBED "X"
- PK NAIL
- METAL SIGN POST
- WOOD SIGN POST
- SANITARY SEWER MANHOLE
- STORM DRAIN MANHOLE
- CATCH BASIN
- WM WATER METER
- W WATER VAULT
- E ELECTRICAL VAULT
- PUMP
- EL ELECTRICAL JUNCTION BOX
- PM POWER METER
- TC TELECOMMUNICATION JUNCTION BOX
- JUNCTION BOX
- P POWER VAULT
- TEL TELECOMMUNICATION RISER
- UTILITY POLE
- UG UTILITY POLE W/ CONDUITS TO UNDERGROUND
- GUY ANCHOR
- GATE POST
- WATER LEVEL
- TEST HOLE (WITH FIELD IDENTIFIER)
- YARD LIGHT OR LUMINAIRE W/NO MAST
- GRATE INLET
- CPEP CORRUGATED POLYETHYLENE PIPE
- RCP REINFORCED CONCRETE PIPE
- CIP CAST IRON PIPE
- CMP CORRUGATED METAL PIPE
- DIP DUCTILE IRON PIPE
- WATER MANHOLE
- WATER VALVE
- OR FIRE HYDRANT
- TREE CANOPY OR SHRUB LINE
- PAVEMENT
- EXISTING GRAVEL ROAD EXPECTED TO BE PAVED FOR DAM OPERATION AND MAINTENANCE
- SOIL
- ROCK
- SD STORM DRAIN
- SS SANITARY SEWER
- FM FORCEMAIN
- UGE UNDERGROUND POWER
- UGF UNDERGROUND FIBER OPTIC
- UGT UNDERGROUND TELEPHONE
- OHE OVERHEAD POWER
- OHF OVERHEAD FIBER OPTIC
- OHT OVERHEAD TELEPHONE
- 20 MAJOR CONTOUR
- 21 MINOR CONTOUR
- DITCH OR STREAM CENTERLINE
- OHW ORDINARY HIGH WATER

GENERAL SYMBOLOGY

ARROW INDICATES DIRECTION OF PLAN NORTH

N

NORTH ARROW

SECTION LETTER (DO NOT USE I OR O)

FLAG INDICATES DIRECTION OF SECTION CUT

XX / X0.00

DRAWING WHERE SECTION IS SHOWN OR TAKEN

SECTION CUT MARKER

DETAIL NUMBER

XX / X0.00

DRAWING WHERE DETAIL IS SHOWN OR TAKEN

DETAIL MARKER

DETAIL NUMBER OR SECTION LETTER

DETAIL TITLE
DETAIL TITLE 2
SCALE: 1/4" = 1'-0"

XX / X0.00

DRAWING WHERE DETAIL OR SECTION IS SHOWN OR TAKEN

DETAIL OR SECTION TITLE

PHOTO NUMBER OR ELEVATION LETTER

XX / X0.00

DRAWING WHERE ELEVATION OR PHOTO IS SHOWN OR TAKEN

ELEVATION OR PHOTO MARKER

XX — KEYNOTE OR SPECIFIC NOTE DESIGNATION

WPXX — WORK POINT

XXXXX — EQUIPMENT DESIGNATION

GPXX — GRADE POINT

NOTES:

1. THIS IS A STANDARD DRAWING SHOWING COMMON SYMBOLOGY.
2. ALL SYMBOLS ARE NOT NECESSARILY USED ON THIS PROJECT.

PRELIMINARY
NOT FOR CONSTRUCTION

GENERAL NOTES

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2. SCREENING OR SHADING OF WORK IS USED TO INDICATE EXISTING COMPONENTS OR TO DEEMPHASIZE PROPOSED IMPROVEMENTS TO HIGHLIGHT SELECTED TRADE WORK.

ABBREVIATIONS

ABC	AGGREGATE BASE COURSE
ABN	TO BE ABANDONED
ABND	ABANDONED
AC	ASBESTOS CEMENT
ACP	ASPHALT CONCRETE PAVEMENT
AFF	ABOVE FINISH FLOOR
ALUM	ALUMINUM
ANSI	AMERICAN NATIONAL STANDARD INSTITUTE
AWS	AUXILIARY WATER SUPPLY
BF	BLIND FLANGE
BFV	BUTTERFLY VALVE
BOC	BEGINNING OF CURVE
BV	BALL VALVE
CL	CENTER LINE
CB	CATCH BASIN
CAV	COMBINATION AIR RELEASE VALVE
CCP	CONCRETE CYLINDER PIPE
CDF	COMPACTED DENSITY FILL
CE	CONCRETE ENCASEMENT
CI	CAST IRON
CO	CLEAN OUT
CONC	CONCRETE
CONSTR	CONSTRUCTION
CHTR	CAPTURE, HANDLE, RELEASE, TRANSPORT
CLR	CLEAR
CMP	CORRUGATED METAL PIPE
CMU	CONCRETE MASONRY UNIT
CTB	CONCRETE TREATED BASE
CV	CHECK VALVE
CU	COPPER
DEQ	DEPARTMENT OF ENVIRONMENTAL QUALITY
DF	DOUGLAS FIR
DIA	DIAMETER
DIM	DIMENSION
DIP	DUCTILE IRON PIPE
DR	DRAIN
DTL	DETAIL
D/W	DRIVEWAY
ED	EQUIPMENT DRAIN

EG	EXISTIING GRADE
EL, ELEV	ELEVATION
ELB	ELBOW
EOC	END OF CURVE
ETC	ET CETERA
EW	EACH WAY
EXIST	EXISTING
FCA	FLANGED COUPLING ADAPTER
FD	FLOOR DRAIN
FDN	FOUNDATION
FE	FLANGED END
FF	FINISH FLOOR
FG	FINISHED GRADE
FH	FIRE HYDRANT
FM	FLOW METER
GALV	GALVANIZED
GP	GRADE POINT
HPG	HIGH PRESSURE GAS
IE; INV	INVERT ELEVATION
LF	LINEAL FEET
MH	MANHOLE
MAX	MAXIMUM
MIN	MINIMUM
MIPT	MALE IRON PIPE THREAD
MJ	MECHANICAL JOINT
MSL	MEAN SEA LEVEL
MON	MONITORING
NGVD	NATIONAL GEODETIC VERTICAL DATUM
NIC	NOT IN CONTRACT
NTS	NOT TO SCALE
OC	ON CENTER
OF	OVERFLOW PIPE
OHE	OVERHEAD ELECTRIC
OHW	ORDINARY HIGH WATER
PE	POLYETHYLENE/ PLAIN END
PNL	PANEL
POC	POINT OF CONNECTION
PVC	POLYVINYL CHLORIDE
PG	PRESSURE GAGE

PL	PLATE
PRV	PRESSURE REDUCING VALVE
PS	PIPE SUPPORT
PT	POINT OF TANGENCY
(R)	HORIZONTAL STATION REFERENCE
R&R	REMOVE AND REPLACE PER EXISTING
RED	REDUCING, REDUCER
RR	RAILROAD
SAN	SANITARY
SD	STORM DRAIN
SH	SCHEDULE
SIM	SIMILAR
SPEC	SPECIFICATION
SQ	SQUARE
SSMH	SANITARY SEWER MANHOLE
SS	STAINLESS STEEL
STA	STATION
STD	STANDARD
STL	STEEL
STM	STORM
STN	STAINLESS
TESC	TEMPORARY EROSION & CONTROL
THK	THICK/THICKNESS
TYP	TYPICAL
UGE	UNDERGROUND ELECTRIC
UGT	UNDERGROUND TELEPHONE
UGTV	UNDERGROUND TV CABLE
UNO	UNLESS NOTED OTHERWISE
VIF	VERIFY LOCATION AND/OR FEATURES IN FIELD PRIOR TO CONDUCTING POTENTIALLY IMPACTED CONSTRUCTION ACTIVITY
WL	WETLAND
WP	WORK POINT
WSEL	WATER SURFACE ELEVATION
N	NORTH
E	EAST
S	SOUTH
W	WATER, WEST
℄	CENTERLINE
Ø	DIAMETER

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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

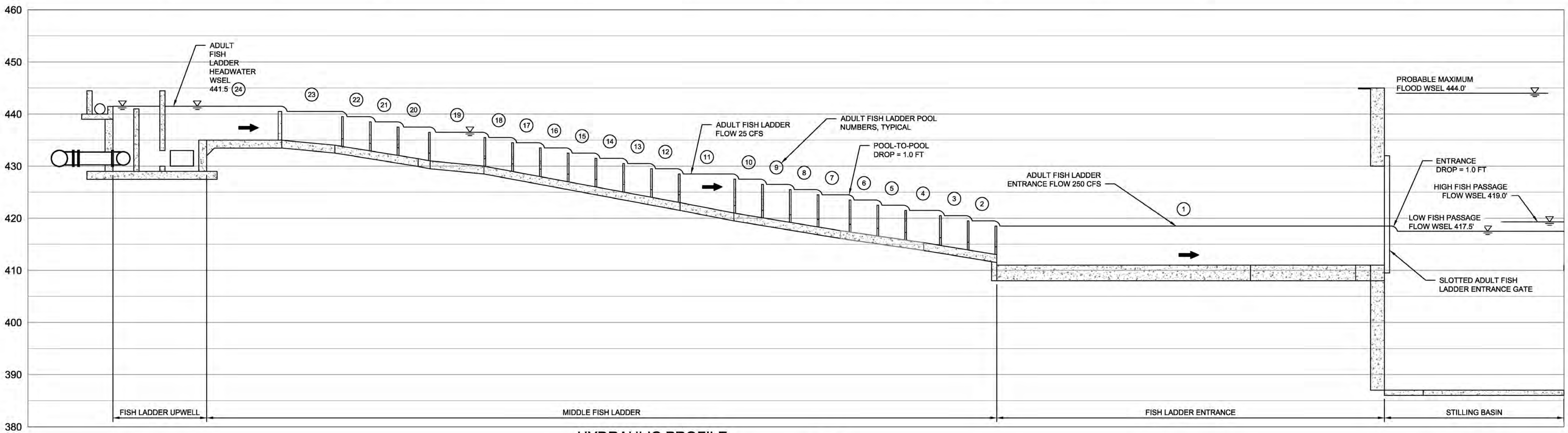
GENERAL

ABREVIATIONS, SYMBOLS AND GENERAL NOTES

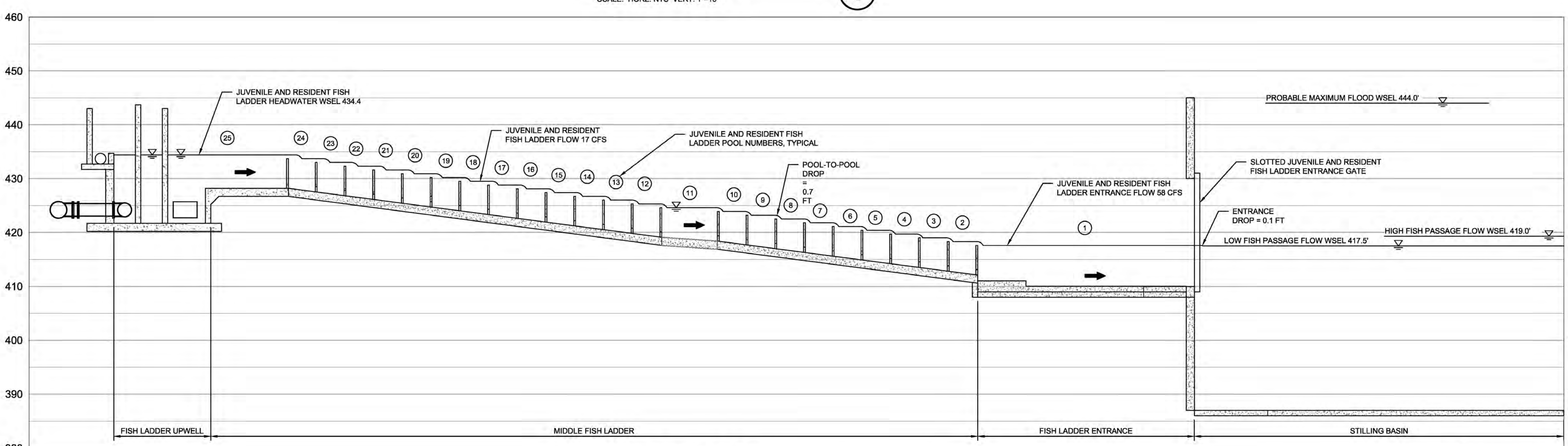
0 1" 2"

DATE: JUNE, 2017
SCALE: NTS

SHEET G-2



HYDRAULIC PROFILE
ADULT FISH LADDER (1)
SCALE: HORZ. NTS VERT. 1"=10'



HYDRAULIC PROFILE
JUVENILE AND RESIDENT FISH LADDER (2)
SCALE: HORZ. NTS VERT. 1"=10'

SCALE IN FEET

PRELIMINARY
NOT FOR CONSTRUCTION

HDR

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PROJECT MANAGER	
CHECKED BY	MATT PROCIV
CHECKED BY	
DRAWN BY	DON DADE
DESIGNED BY	DON DADE
DESIGNED BY	
PROJECT NUMBER	268421

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

GENERAL

HYDRAULIC PROFILES
ADULT AND JUVENILE FISH LADDERS

DATE	JUNE, 2017
SCALE	AS SHOWN
SHEET	G-3

HYDRAULIC PROFILE
FISH LADDER WATER SUPPLY

SCALE: HORZ. NTS VERT. 1"=10'

1 / -

HYDRAULIC PROFILE
SORTING FACILITY WATER SUPPLY

SCALE: HORZ. NTS VERT. 1"=10'

1 / -

10 0 10 20
SCALE IN FEET

PRELIMINARY
NOT FOR CONSTRUCTION

HDR

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THE SEAL OF THE STATE OF WASHINGTON 1889

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

GENERAL
HYDRAULIC PROFILES
FISH LADDER AND
SORTING FACILITY WATER SUPPLY

0 1" 2"

DATE	JUNE, 2017
SCALE	AS SHOWN

SHEET
G-4

PRELIMINARY
NOT FOR CONSTRUCTION

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER	
CHECKED BY	MATT PROCIV
CHECKED BY	
DRAWN BY	DON DADE
DESIGNED BY	DON DADE
DESIGNED BY	
PROJECT NUMBER	268421

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

GENERAL

SITE ISOMETRIC - AERIAL

DATE	JUNE, 2017
SCALE	NTS

SHEET
G-5

PRELIMINARY
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PROJECT NUMBER	268421

THE SEAL OF THE STATE OF WASHINGTON 1889

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

GENERAL

FACILITY ISOMETRIC - AERIAL

0 1" 2"

DATE	JUNE, 2017
SCALE	NTS

SHEET
G-6

PRELIMINARY
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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

GENERAL

FACILITY ISOMETRIC - OVERHEAD

0 1" 2"

DATE	JUNE, 2017
SCALE	NTS

SHEET
G-7

PROJECT SITE PLAN

PRELIMINARY
NOT FOR CONSTRUCTION

HDR

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER	
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STATE OF WASHINGTON
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CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

CIVIL
PROJECT SITE PLAN

0 1" 2"

DATE	JUNE, 2017
SCALE	1" = 50'

SHEET
C-1

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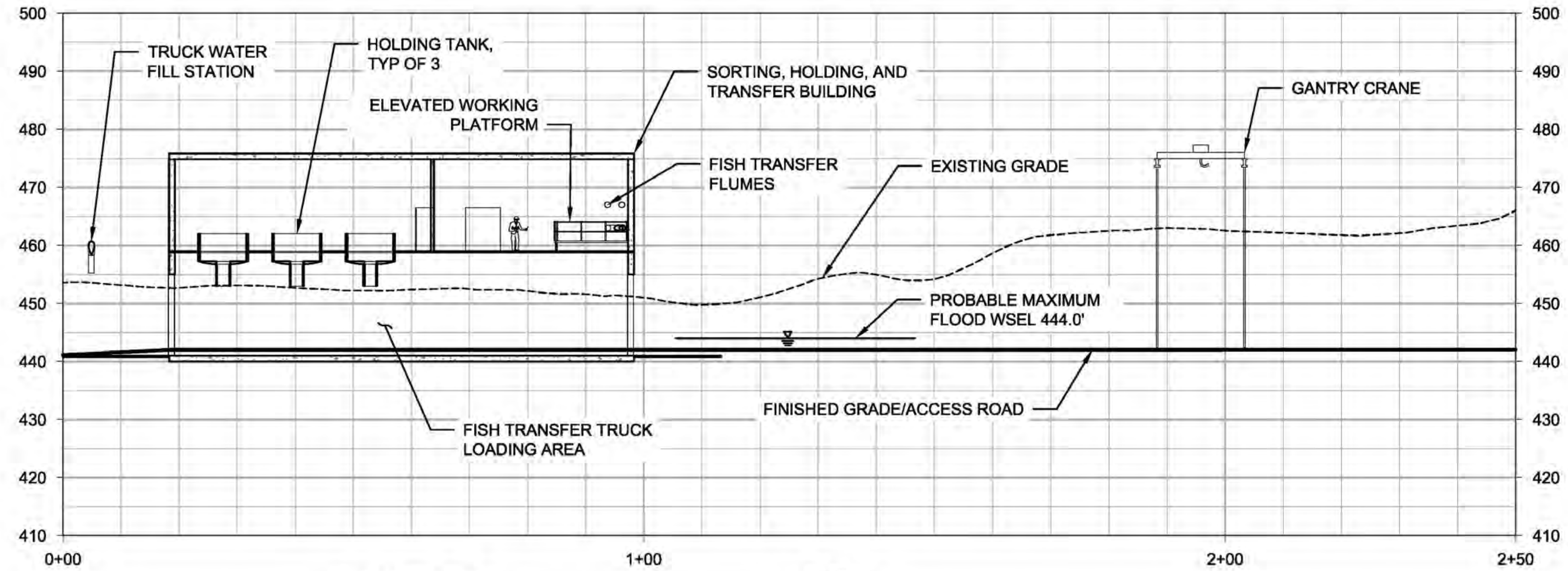
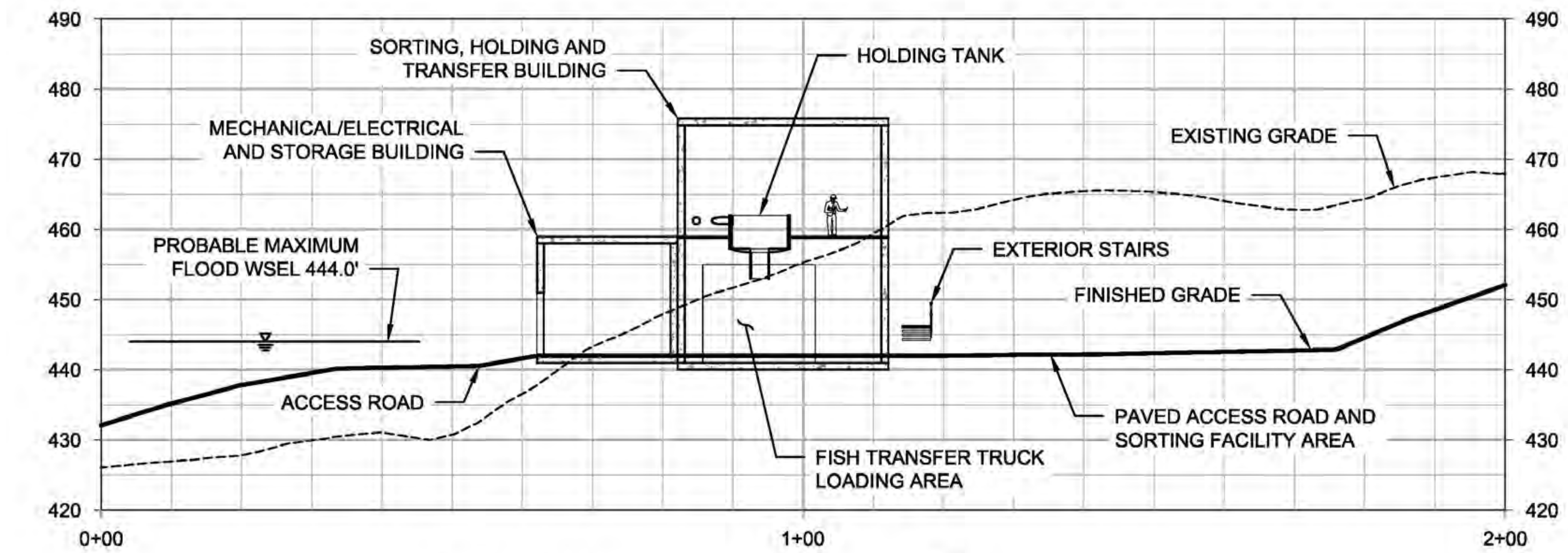
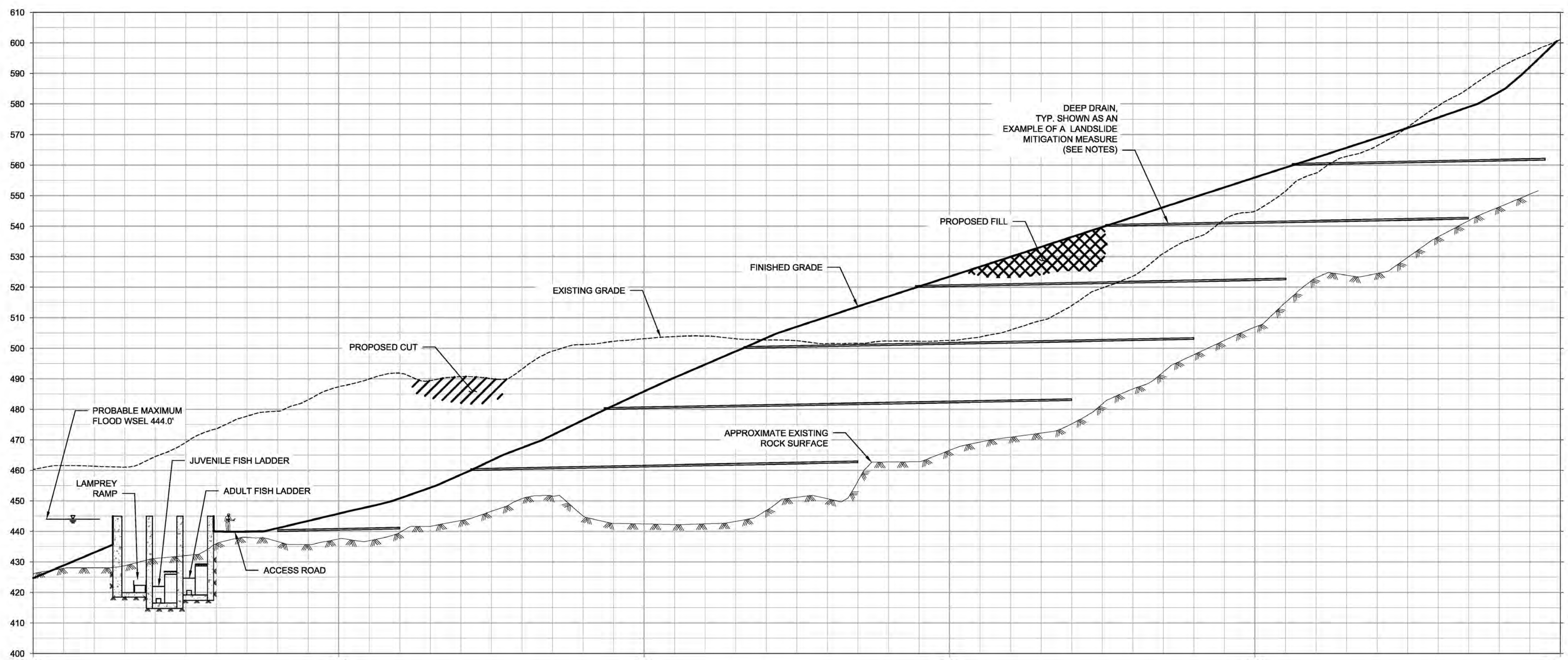
NOTES:
1. SEE SHEET C-1 FOR LANDSCAPE MITIGATION NOTES.



SECTION A / C-1



SECTION B / C-1



SECTION C / C-1

20 0 20 40
SCALE IN FEET

PRELIMINARY
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THE SEAL OF THE STATE OF WASHINGTON 1889

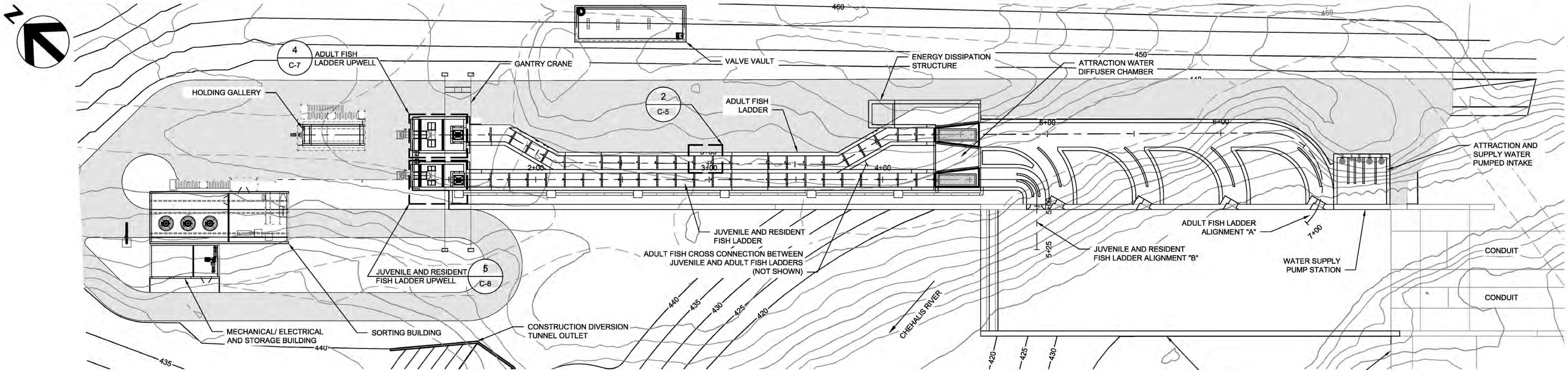
STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

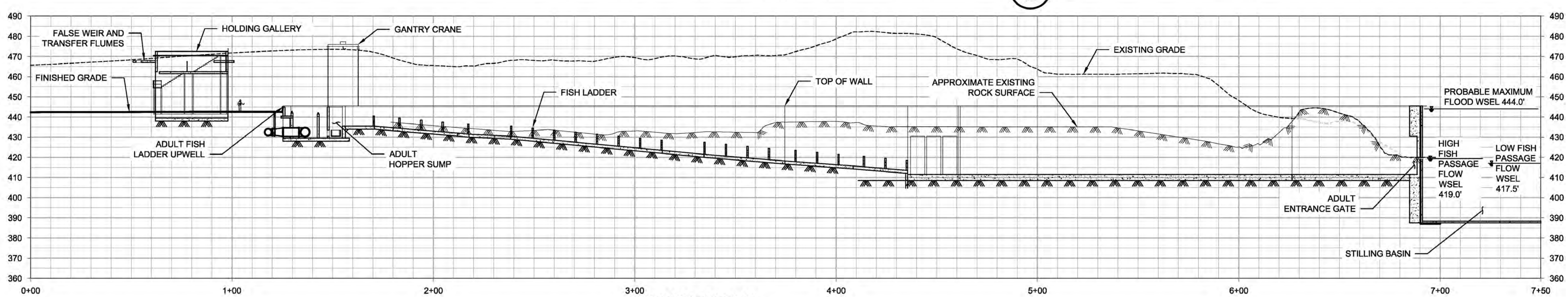
CIVIL
SITE SECTIONS AND ELEVATIONS

0 1" 2"

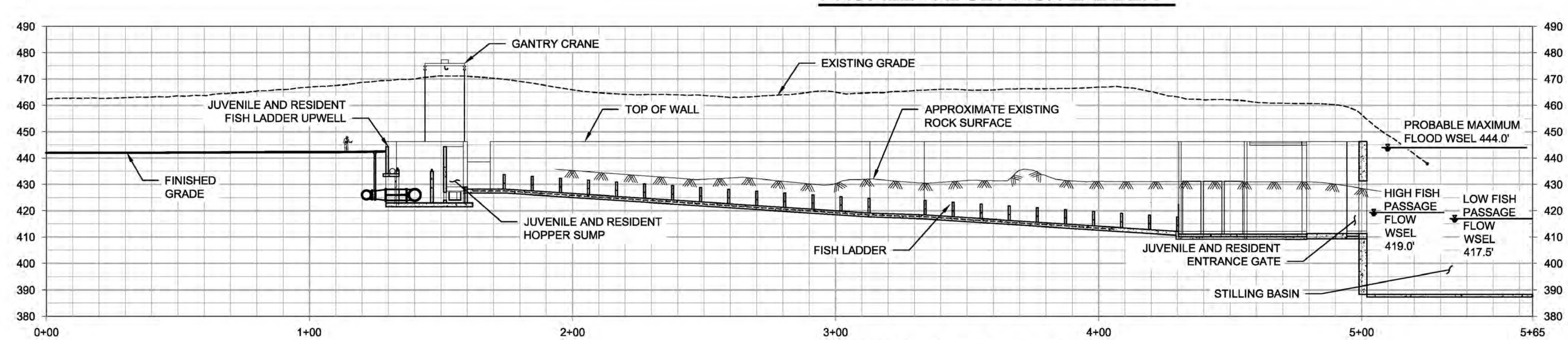
DATE JUNE, 2017
SCALE 1" = 20'
SHEET C-2



PLAN - ADULT AND JUVENILE FISH LADDERS 1 C-1



ALIGNMENT "A"
PROFILE - ADULT FISH LADDER



ALIGNMENT "B" PROFILE -
JUVENILE AND RESIDENT FISH LADDER

30 0 30 60
SCALE IN FEET

PRELIMINARY
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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

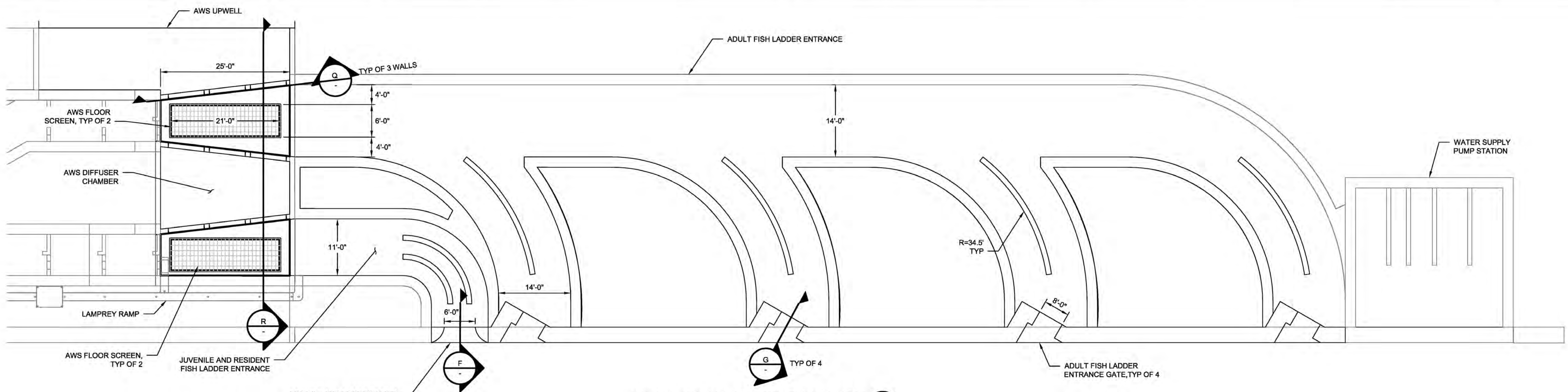
FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

CIVIL

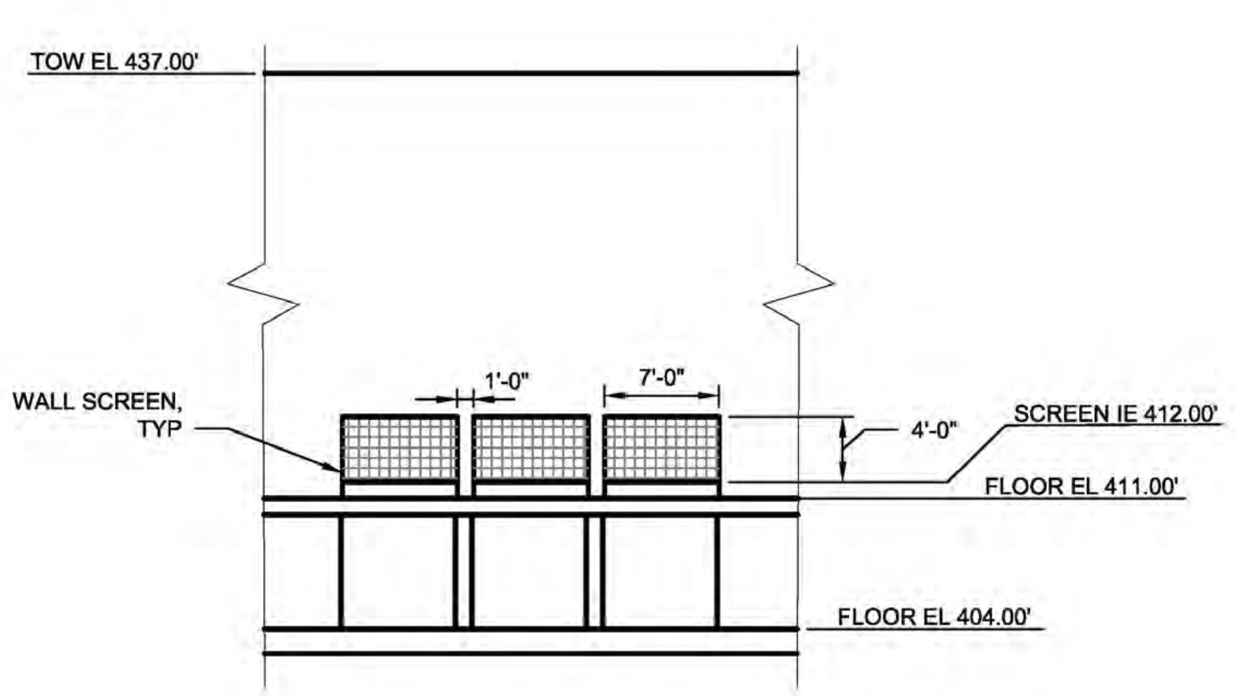
FISH LADDER
PLAN AND PROFILES

DATE	JUNE, 2017
SCALE	1" = 30'

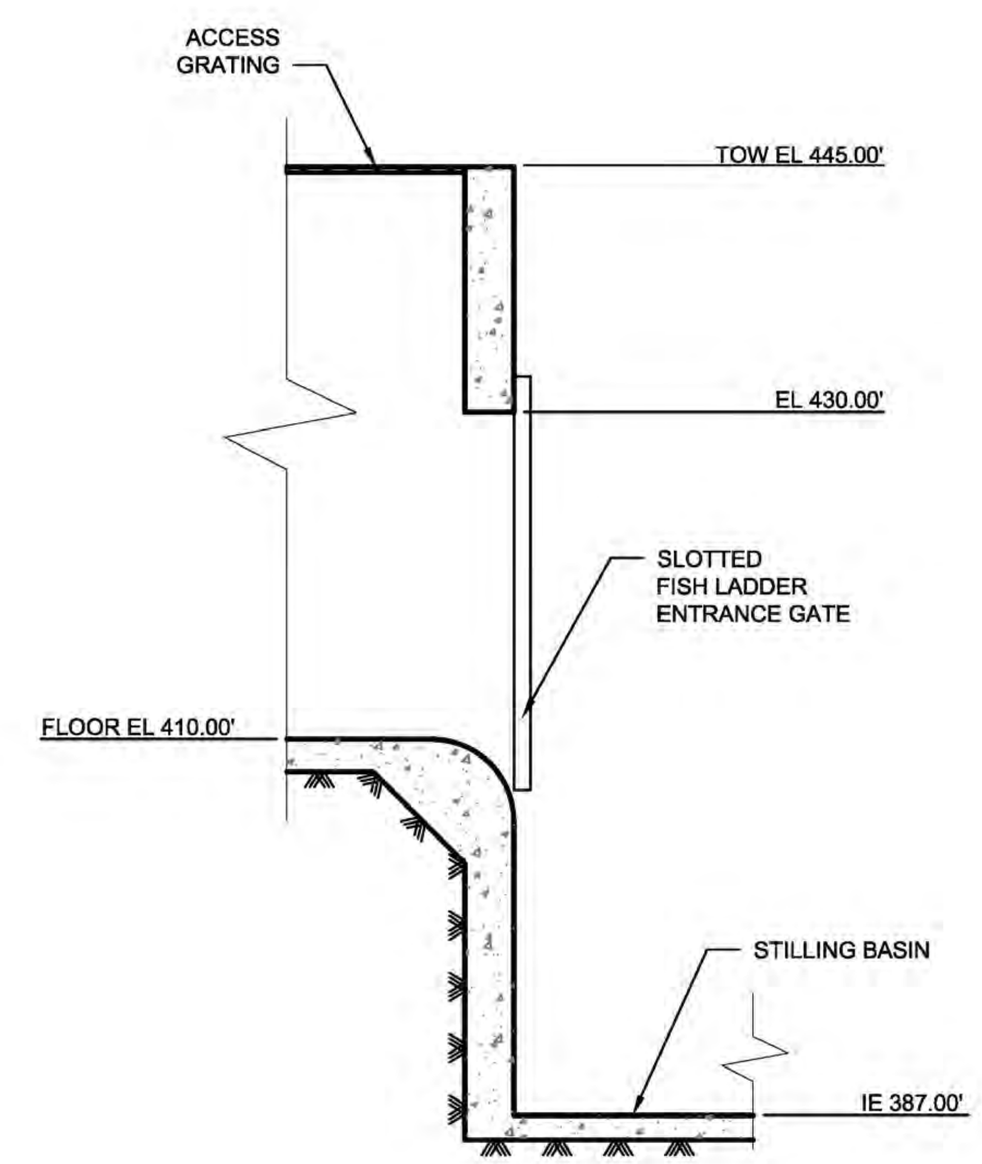
SHEET C-3



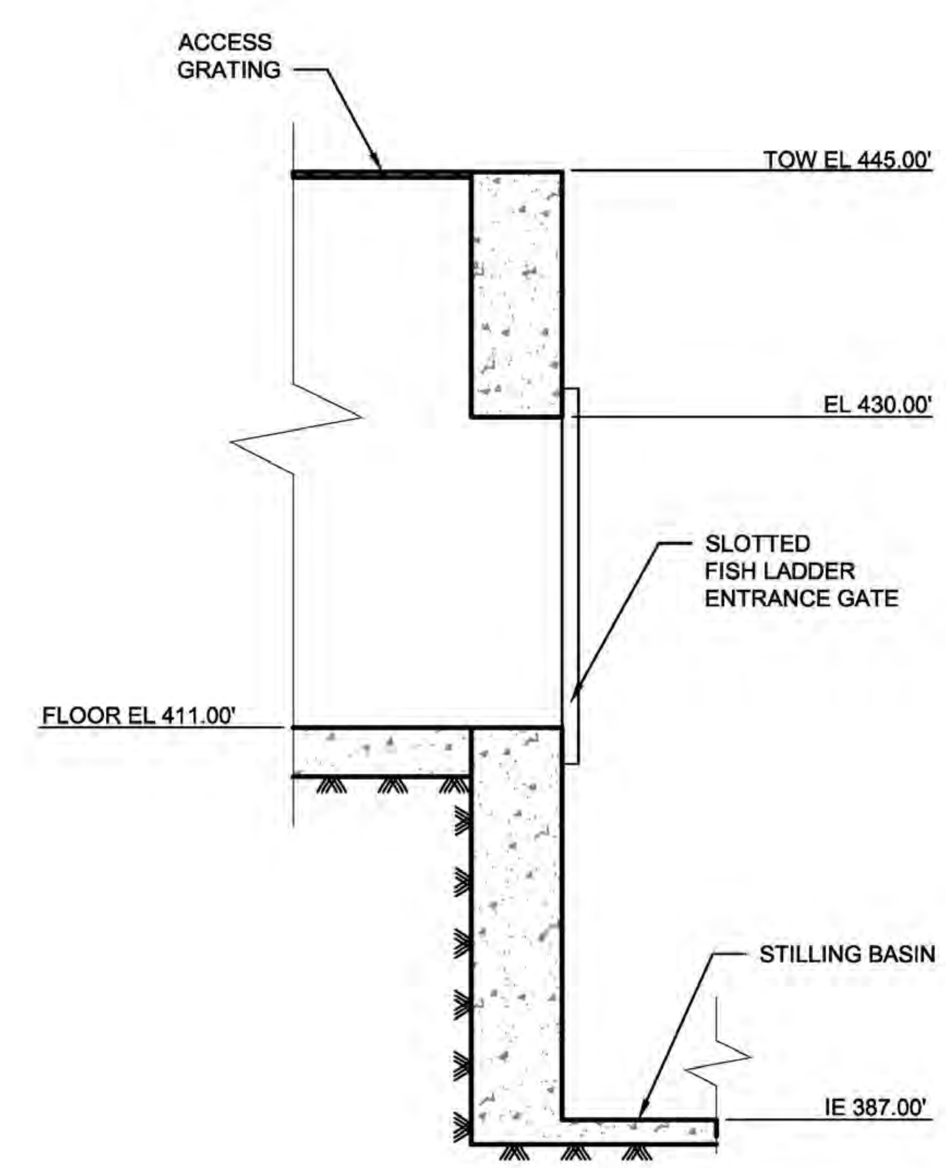
PLAN - FISH LADDER ENTRANCE (2 / C-1)



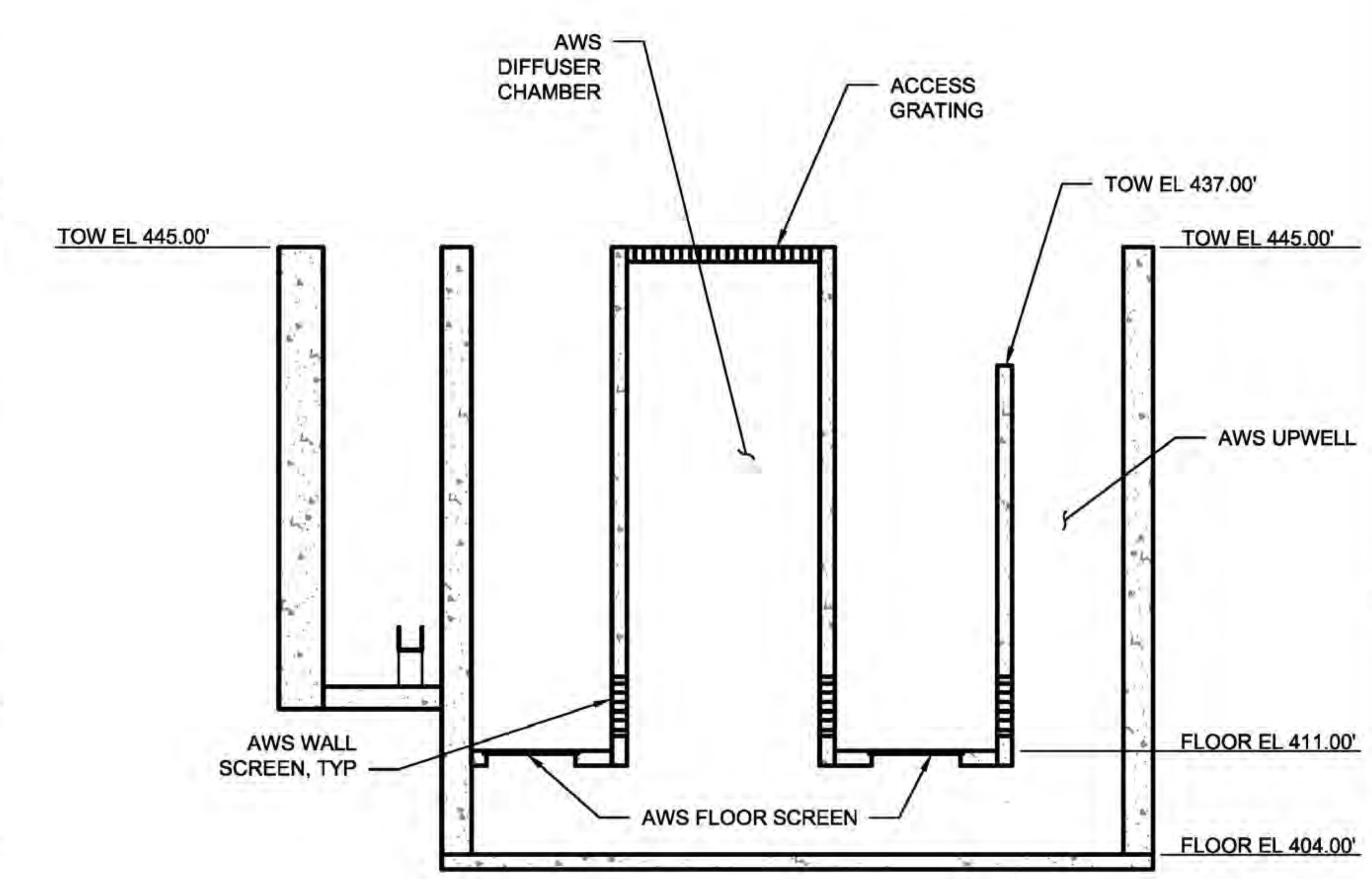
TYPICAL ELEVATION - AWS WALL SCREENS (Q / -)



SECTION - JUVENILE AND RESIDENT FISH LADDER ENTRANCE (F / -)



TYPICAL SECTION - ADULT FISH LADDER ENTRANCE (G / -)



SECTION (R / -)

10 0 10 20
SCALE IN FEET

PRELIMINARY
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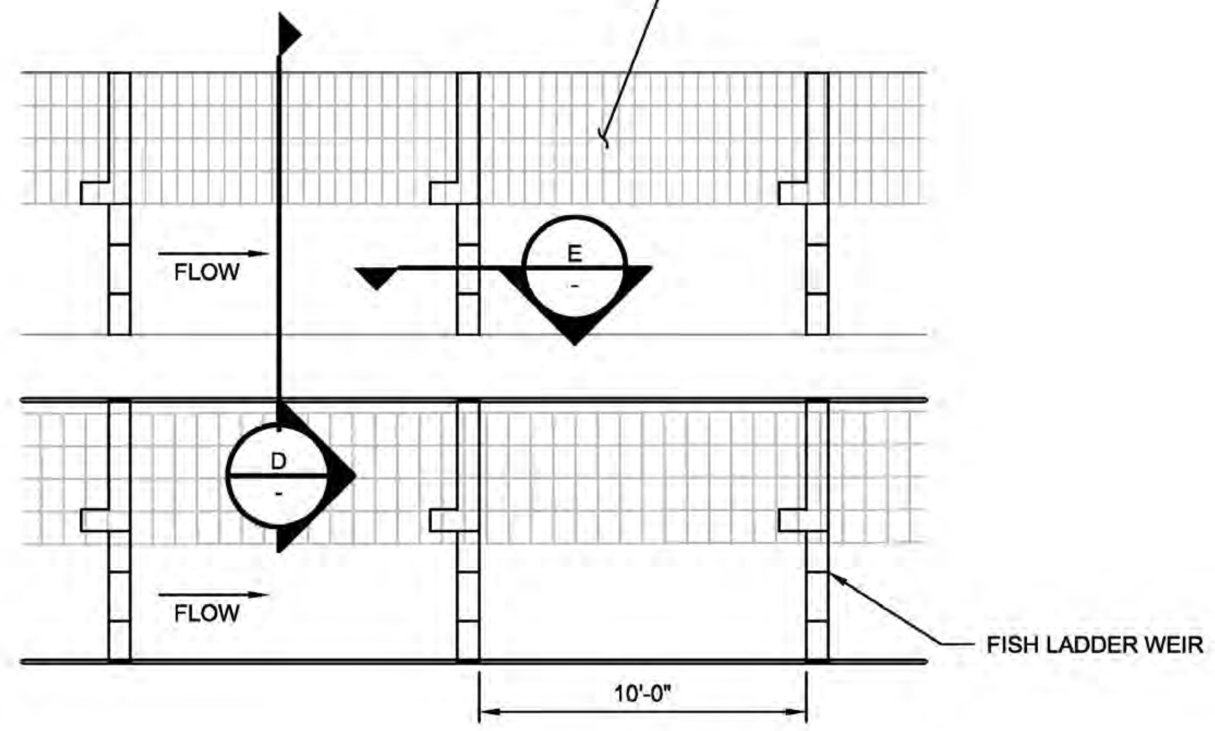
STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

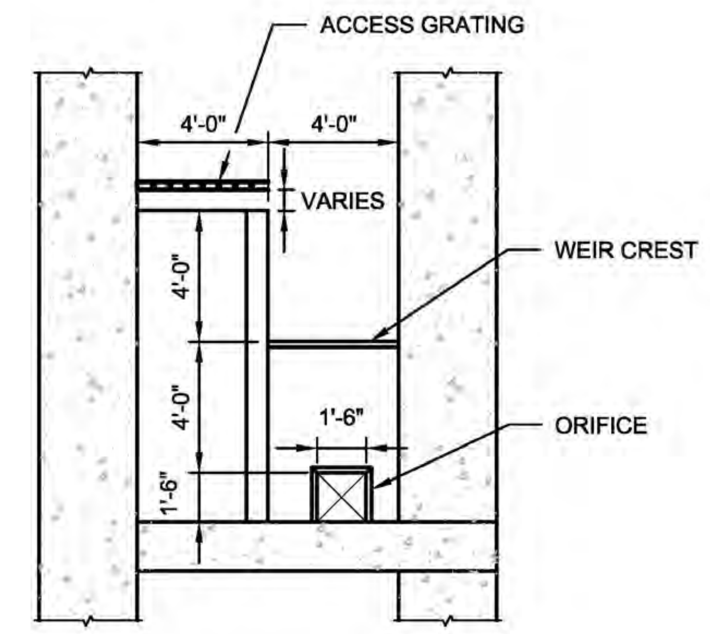
CIVIL

FISH LADDER ENTRANCE DETAILS

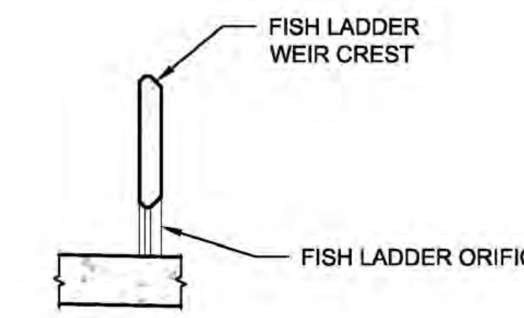
DATE	JUNE, 2017
SCALE	1" = 10'
SHEET	C-4



DETAIL - TYPICAL FISH LADDER POOLS (2 / C-3)



TYPICAL FISH LADDER WEIR ELEVATION (D / -)



TYPICAL FISH LADDER WEIR SECTION (E / -)

SCALE IN FEET

PRELIMINARY
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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

CIVIL

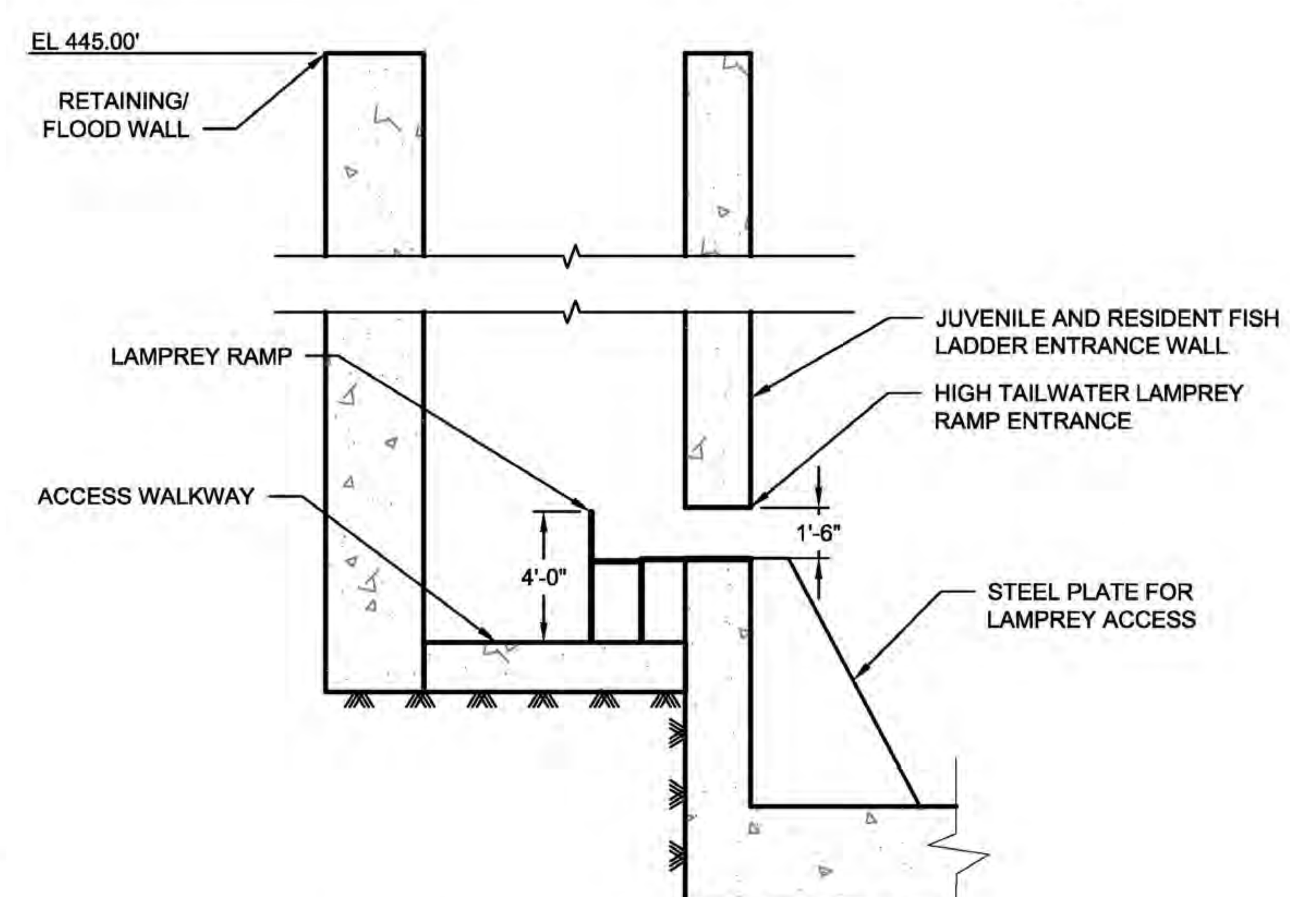
FISH LADDER
TYPICAL SECTIONS AND DETAILS

DATE	JUNE, 2017
SCALE	1" = 5'

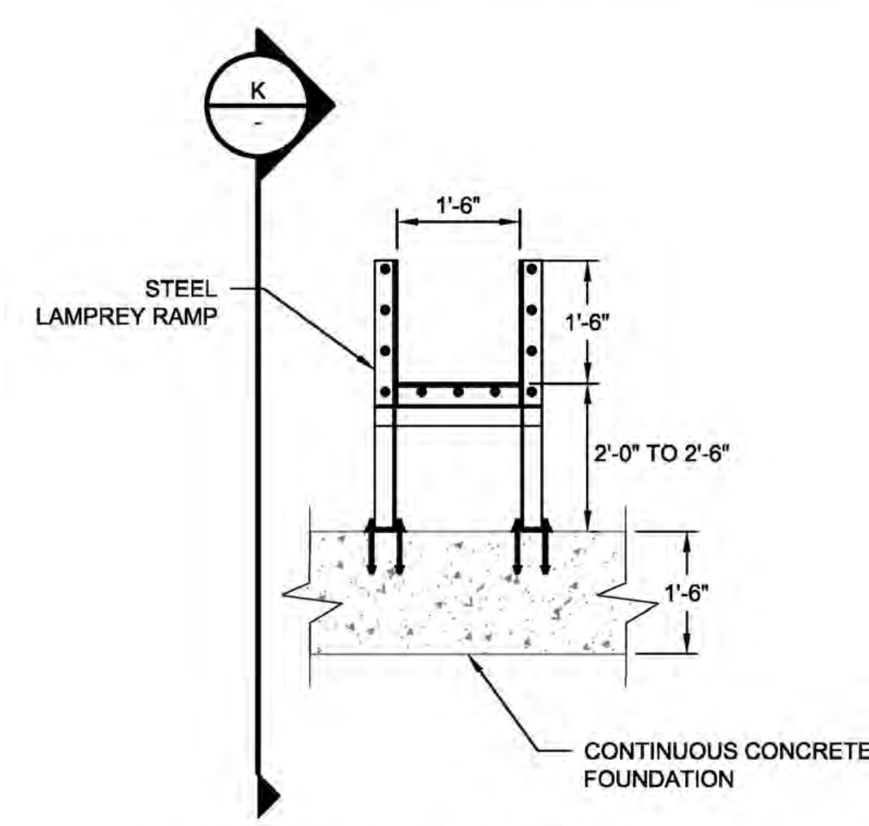
SHEET C-5

PLAN - LAMPREY RAMP
SCALE: 1" = 10'
3 C-1

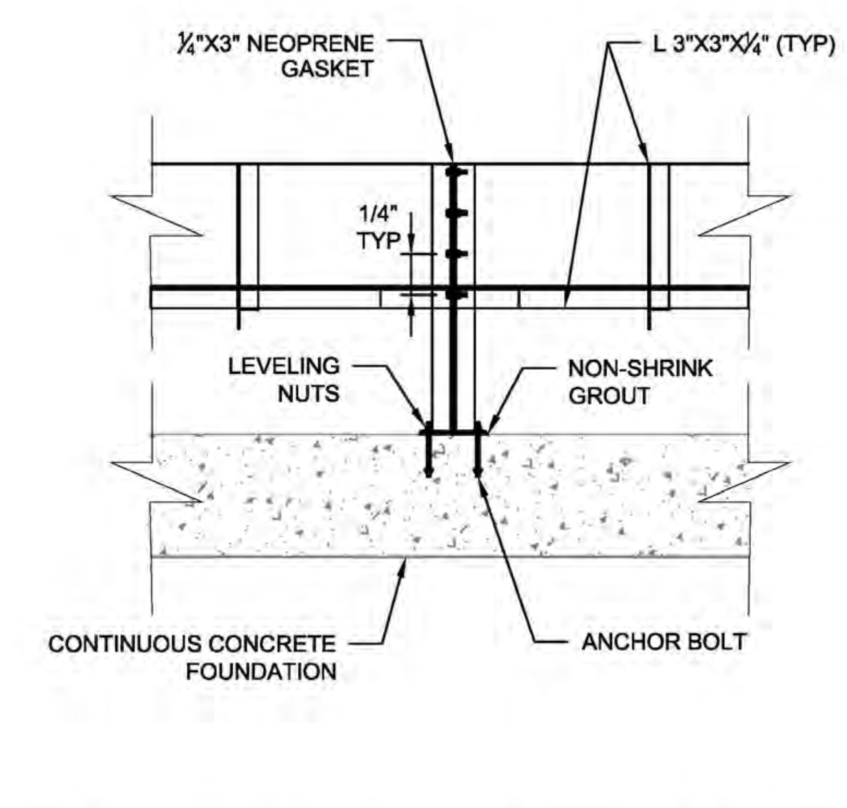
PROFILE - LAMPREY RAMP
SCALE: 1" = 10'



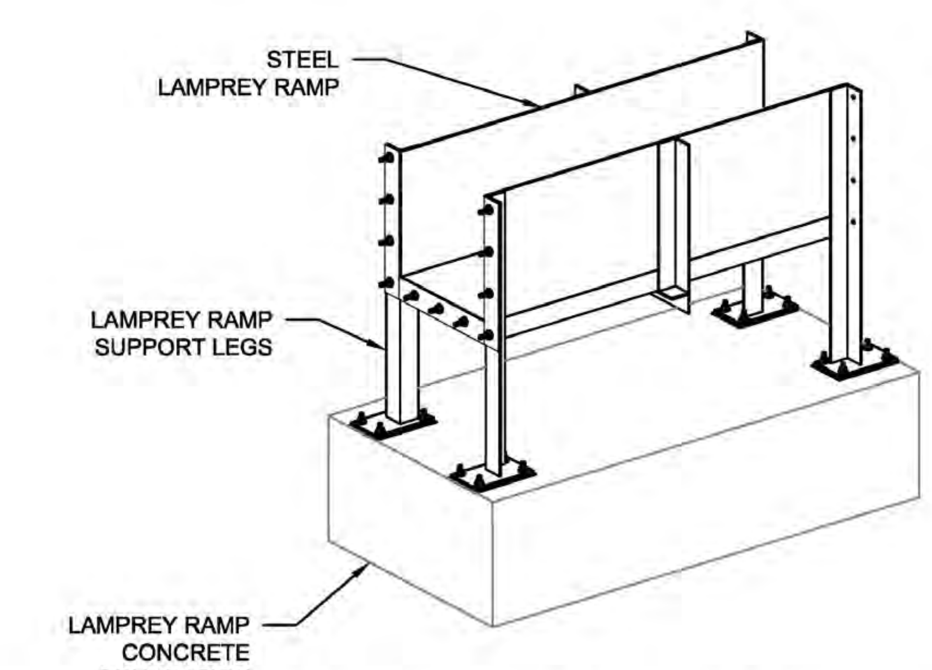
SECTION
SCALE: 1" = 5'
J



SECTION - TYPICAL LAMPREY RAMP SECTION
SCALE: 1" = 2'
H



TYPICAL ELEVATION - LAMPREY RAMP
SCALE: 1" = 2'
K



ISOMETRIC - LAMPREY RAMP
SCALE: NTS
6

SCALE IN FEET (2 0 2 4)
SCALE IN FEET (5 0 5 10)
SCALE IN FEET (10 0 10 20)

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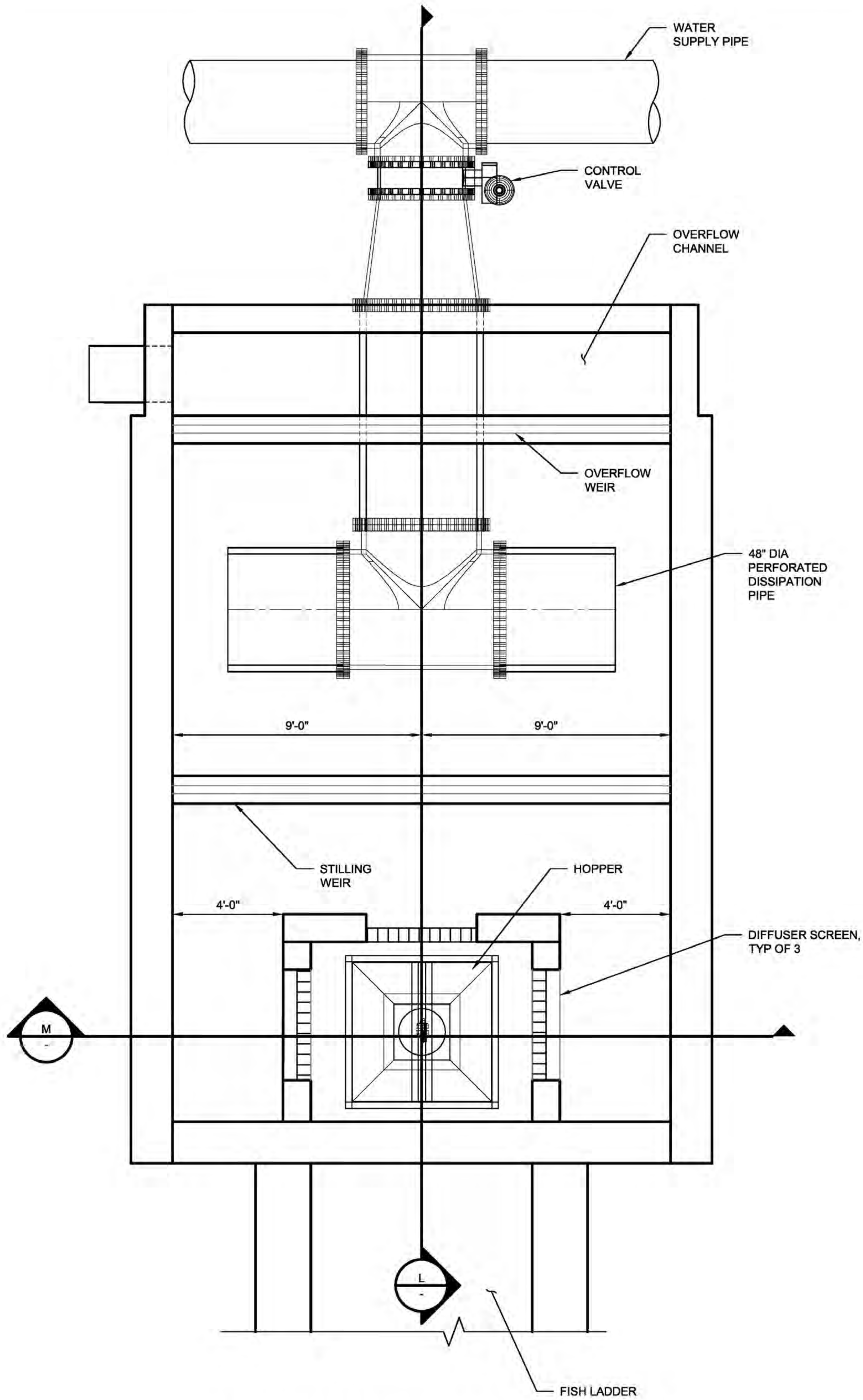
STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

CIVIL

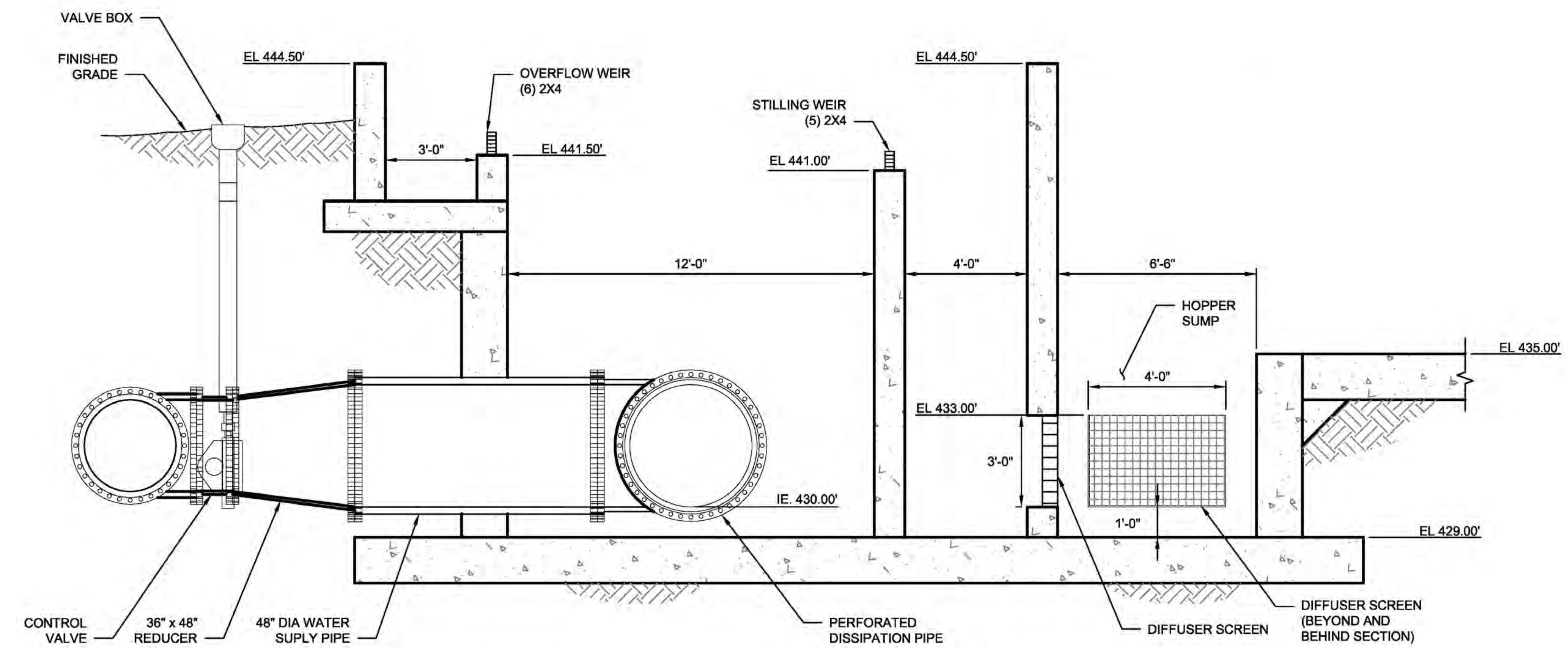
LAMPREY RAMP
PLAN, SECTIONS, AND DETAILS

DATE	JUNE, 2017
SCALE	AS SHOWN
SHEET	C-6

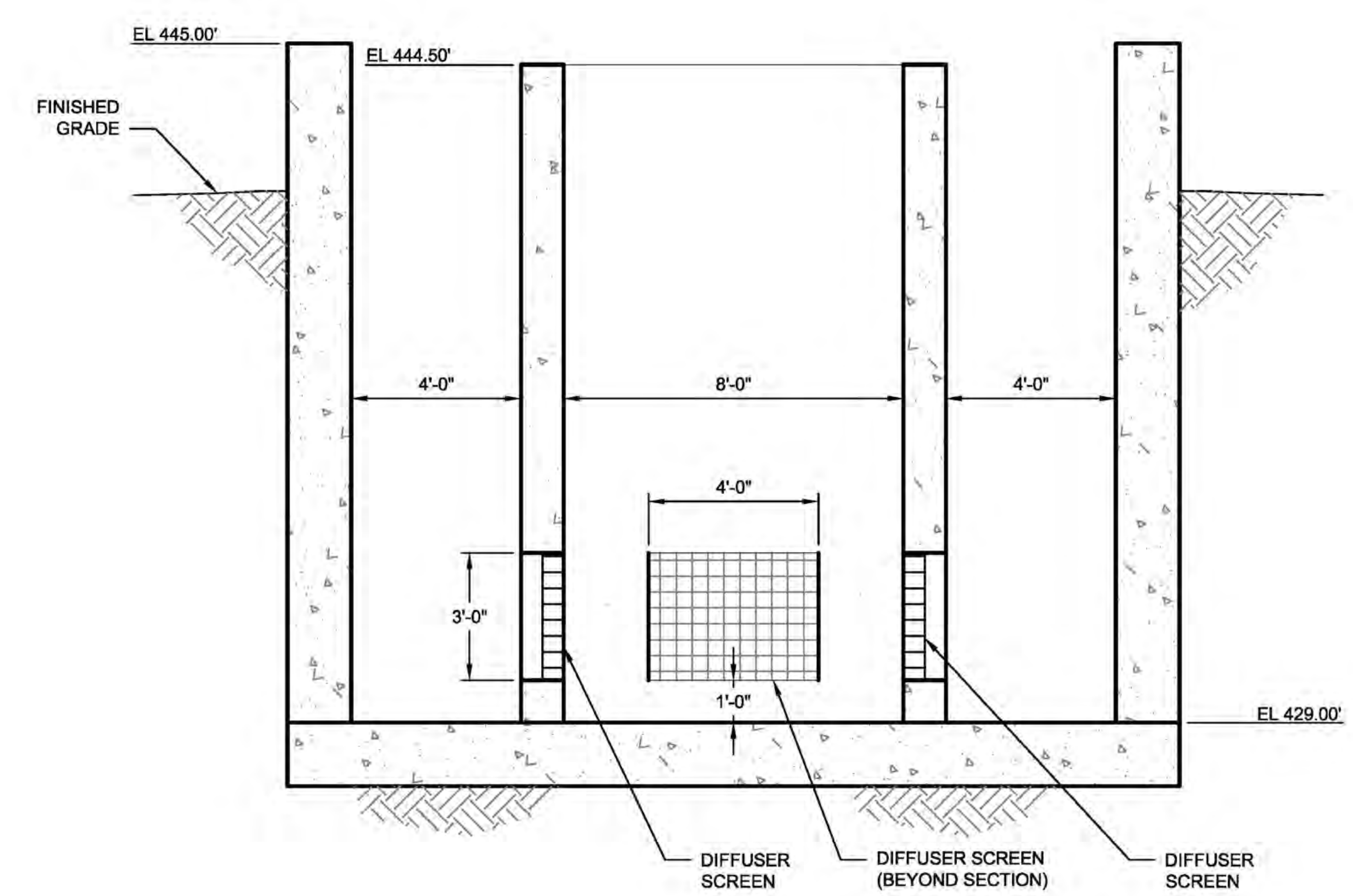


PLAN - ADULT FISH LADDER UPWELL (4 / C-3)

3 0 3 6
SCALE IN FEET



SECTION (L)



SECTION (M)

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THE SEAL OF THE STATE OF WASHINGTON 1889

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

CIVIL

ADULT FISH LADDER SUPPLY WATER
UPWELL SECTIONS AND DETAILS

DATE	JUNE, 2017
SCALE	1" = 3'

SHEET C-7

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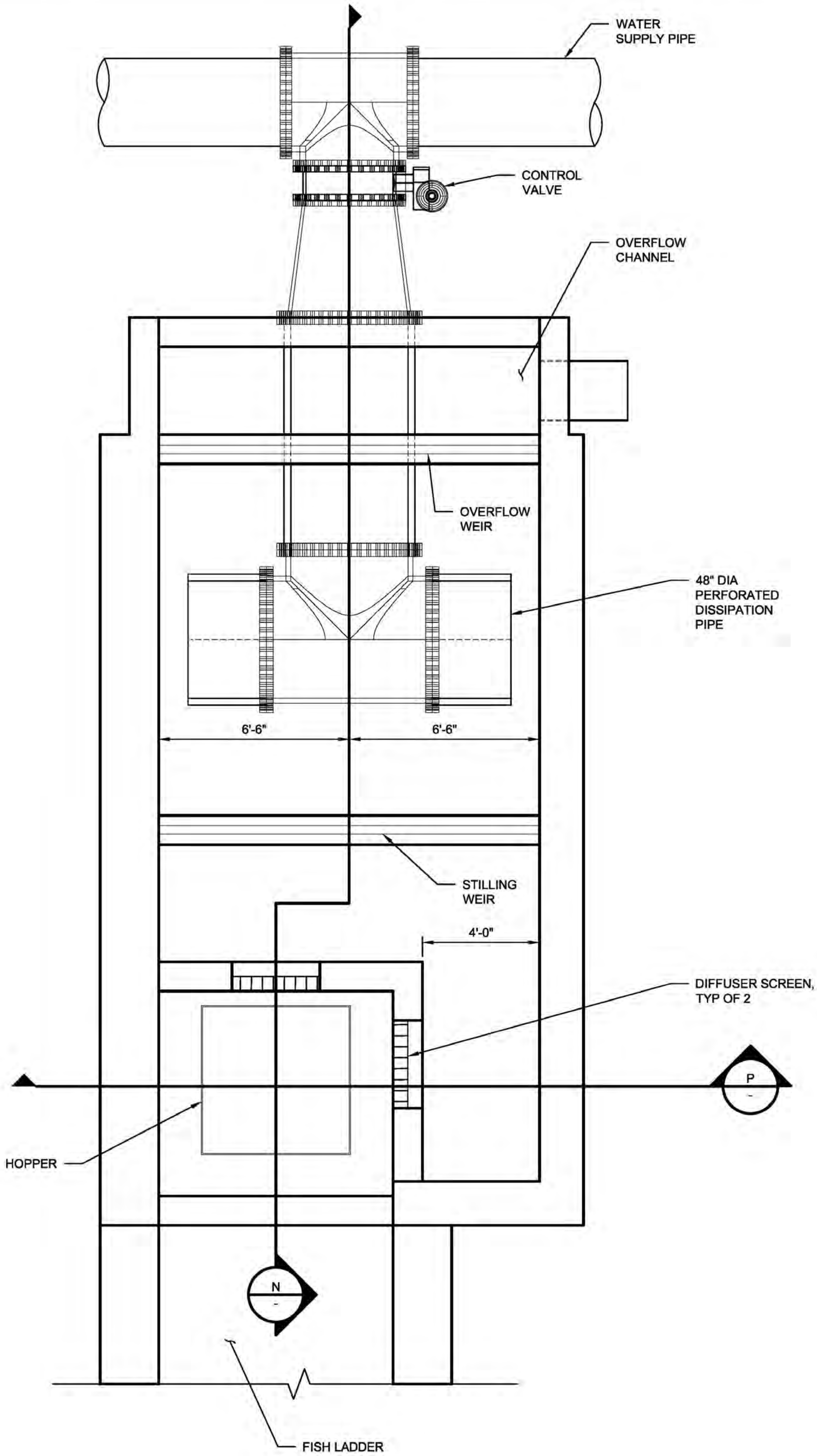
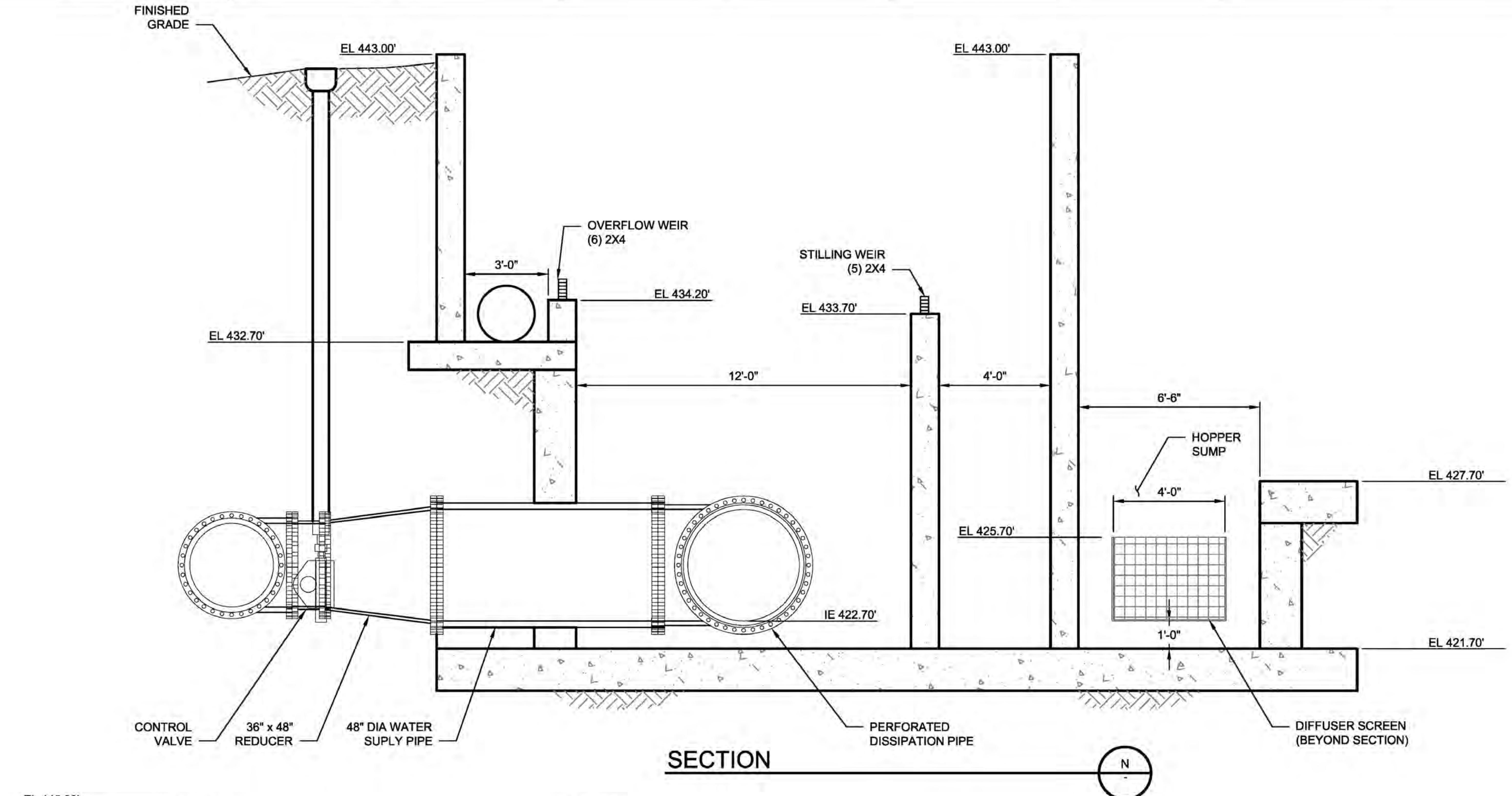
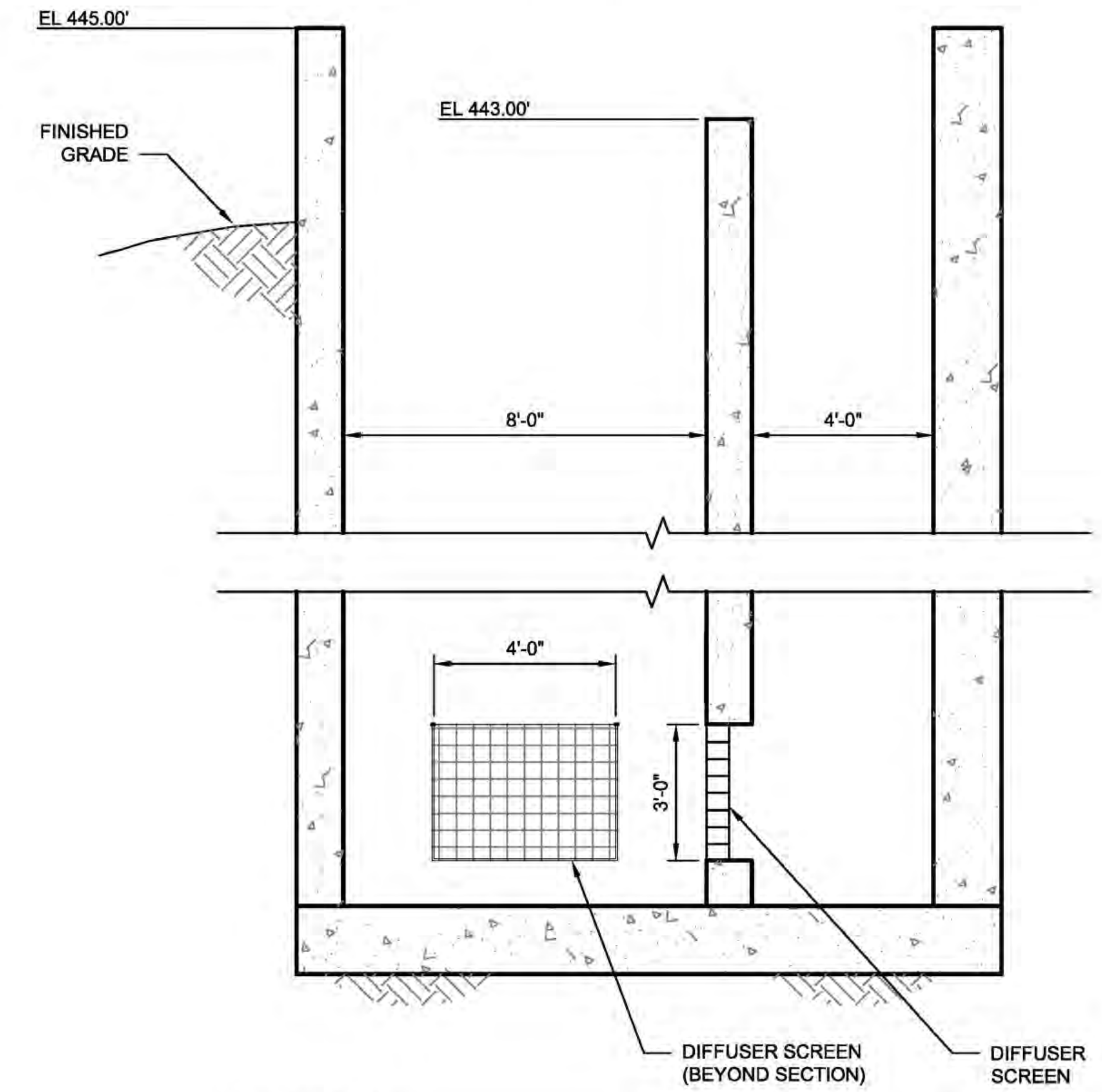


PLAN - JUVENILE FISH LADDER UPWELL 5 C-3

3 0 3 6
SCALE IN FEET



SECTION N



SECTION P

PRELIMINARY
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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

CIVIL

JUVENILE FISH LADDER SUPPLY WATER UPWELL - SECTIONS AND DETAILS

0 1" 2"

DATE	JUNE, 2017
SCALE	1' = 3'

SHEET C-8

PRELIMINARY
NOT FOR CONSTRUCTION

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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

CIVIL
WATER SUPPLY PLAN

0 1" 2"

DATE	JUNE, 2017
SCALE	1" = 20'

SHEET
C-9

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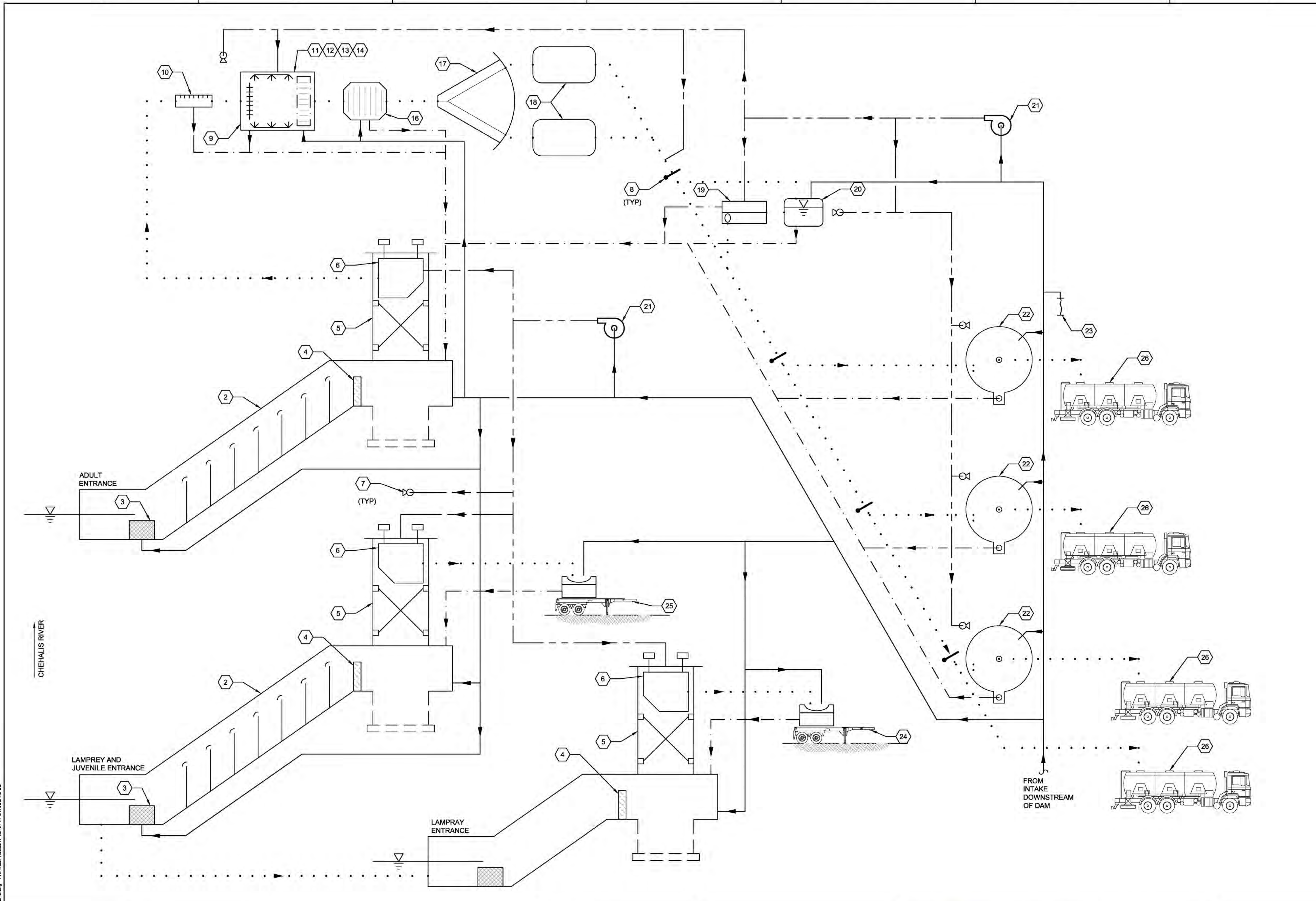

KEYNOTES:

1. ATTRACTION WATER SUPPLY
2. FISHWAY
3. AUXILIARY WATER SUPPLY
4. TRAP GATE
5. HOPPER LIFT TOWER
6. HOPPER BUCKET
7. HOSE BIB
8. DIVERTER GATE
9. ADULT HOLDING GALLERY
10. DEWATERING SCREEN
11. SPRINKLER SYSTEM
12. CROWDER
13. HOLDING GALLERY UPWELL
14. HOLDING GALLERY BRAIL
15. JUVENILE HOLDING GALLERY
16. FALSE WEIR
17. AUTOMATIC DIVERTER GATE
18. VISUAL INSPECTION TANKS
19. ADULT WORKUP TABLE
20. ADULT HOLDING TANK
21. JOB WATER BOOSTER PUMP
22. CIRCULAR HOLDING TANK
23. TRUCK FILL
24. LAMPREY TRANSPORT TANK
25. JUVENILE TRANSPORT TANK
26. ADULT TRANSPORT TRUCK

LEGEND:

- FISH FLUME (FF)
- DRAIN (DR)
- WATER SUPPLY (WS)
- JOB WATER (JW)

PRELIMINARY
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THE SEAL OF THE STATE OF WASHINGTON 1889

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

MECHANICAL
PROCESS DIAGRAM

0 1" 2"

DATE	JUNE, 2017
SCALE	NTS

SHEET
M-1

KEYNOTES:

- (A) BYPASS PIPE
- (B) 8' DIA x 5' DEEP CIRCULAR HOLDING TANK
- (C) ADULT ANESTHESIA TANK WITH BRAIL
- (D) ADULT WORK-UP TABLE
- (E) VISUAL INSPECTION TANKS
- (F) DIVERTER GATE
- (G) FALSE WEIR
- (H) BRAIL
- (I) CROWDER
- (J) JW BOOSTER PUMP
- (K) DISTRIBUTION FLUME
- (L) ELEVATED PLATFORM
- (M) TRUCK FILL
- (N) INSPECTION FLUME

PRELIMINARY
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THE SEAL OF THE STATE OF WASHINGTON 1889

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

MECHANICAL

SORTING FACILITY PLAN

0 1" 2"

DATE	JUNE, 2017
SCALE	NTS

SHEET
M-2

PRELIMINARY
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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

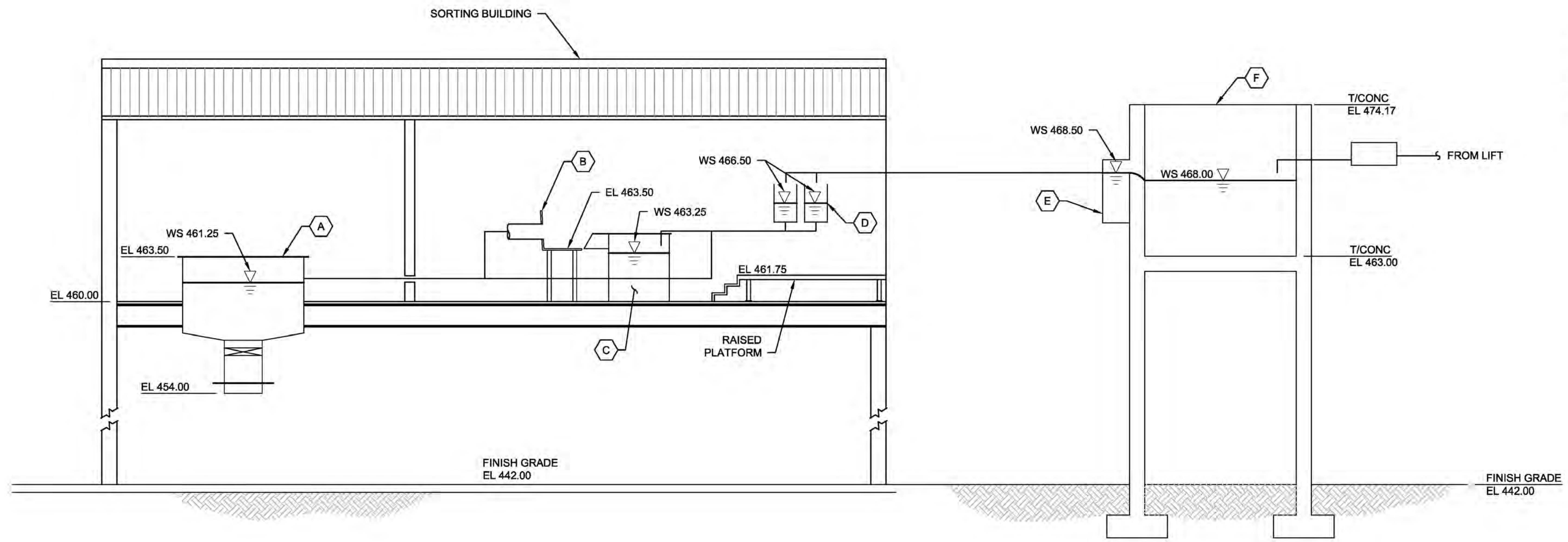
FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

MECHANICAL

SORTING FACILITY PIPING PLAN

DATE	JUNE, 2017
SCALE	1" = 10'

SHEET M-3



HYDRAULIC PROFILE

KEYNOTES:
- (A) CIRCULAR HOLDING TANK
- (B) ADULT WORK-UP TABLE
- (C) ADULT ANESTHESIA TANK WITH BRAIL
- (D) VISUAL INSPECTION TANKS
- (E) FALSE WEIR
- (F) ADULT HOLDING GALLERY

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STATE OF WASHINGTON
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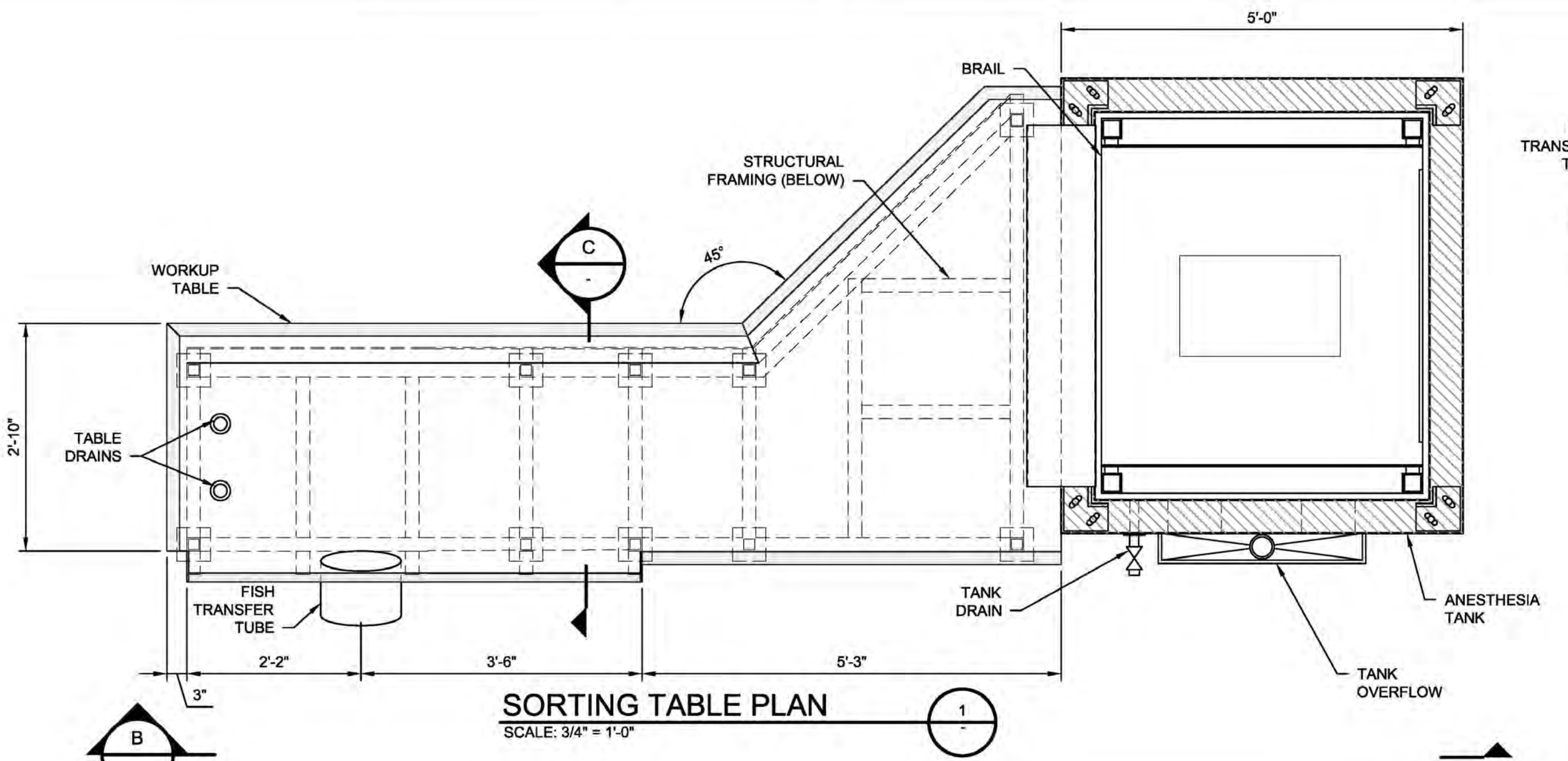
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

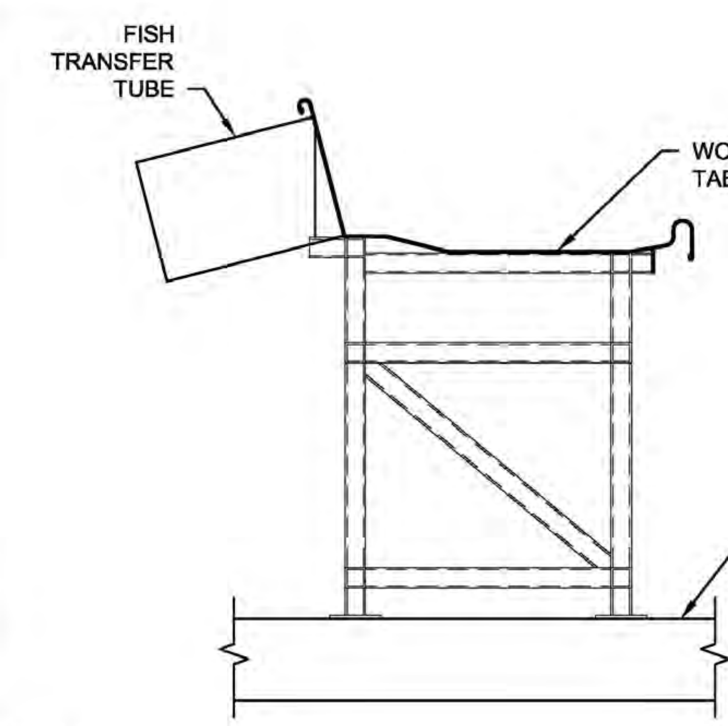
MECHANICAL
SORTING FACILITY SECTIONS

DATE	JUNE, 2017
SCALE	NTS

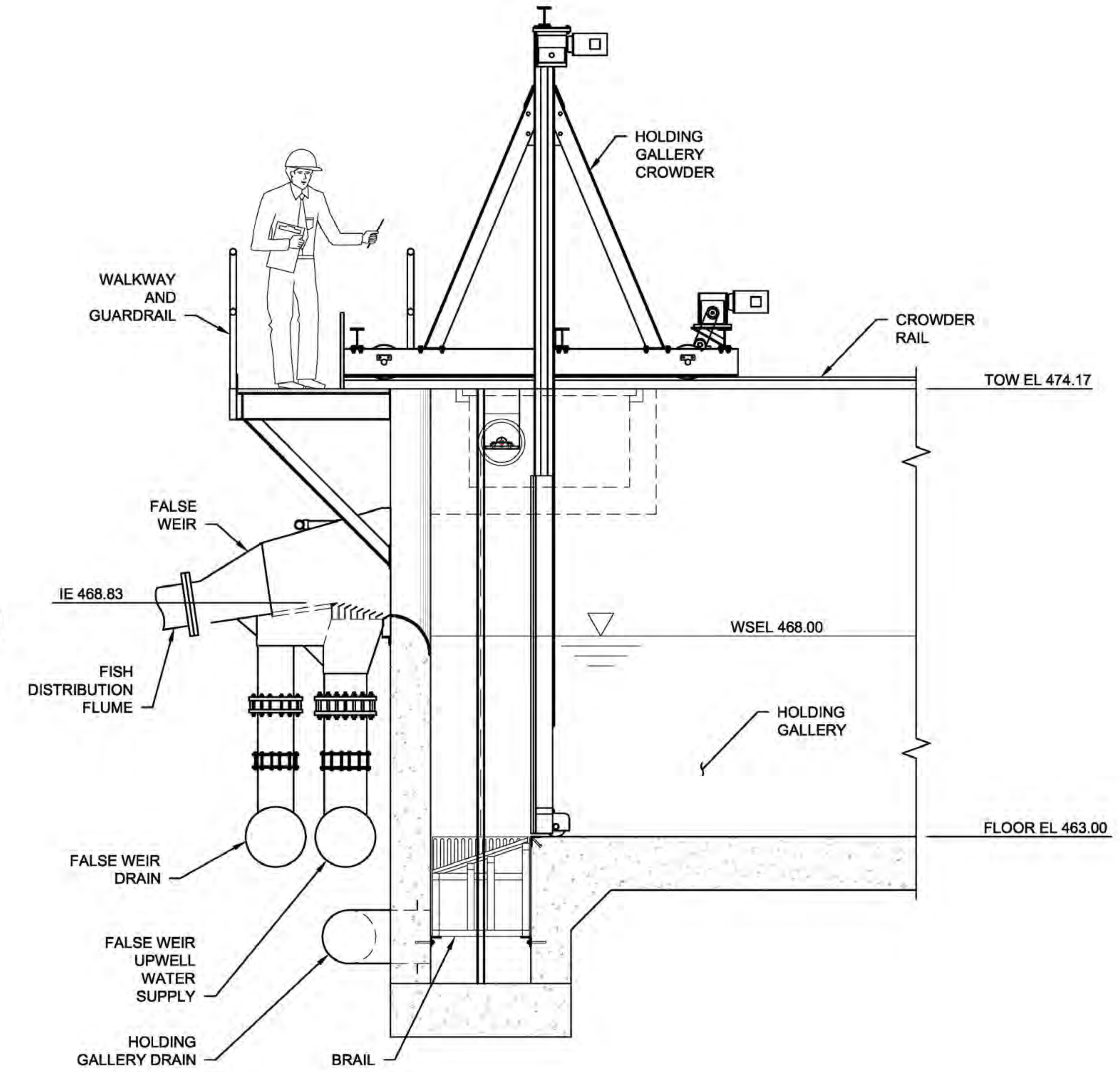
SHEET M-4



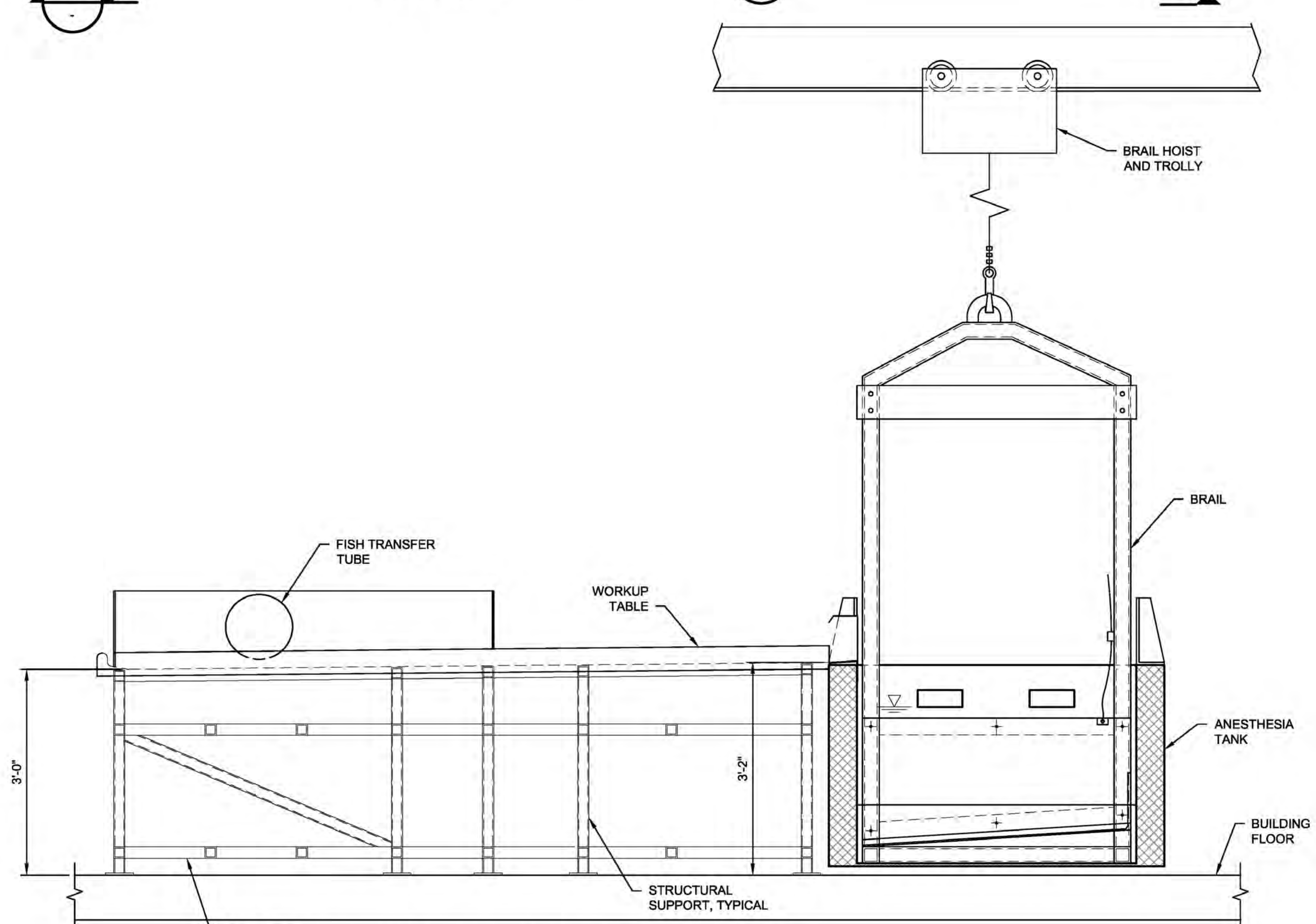
SORTING TABLE PLAN
SCALE: 3/4" = 1'-0"
1



SORTING TABLE SECTION
SCALE: 3/4" = 1'-0"
C



HOLDING GALLERY SECTION
SCALE: 3/8" = 1'-0"
A
M-2



SORTING TABLE ELEVATION
SCALE: 3/4" = 1'-0"
B

2 0 2 4
SCALE IN FEET

1 0 1 2
SCALE IN FEET

PRELIMINARY
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CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

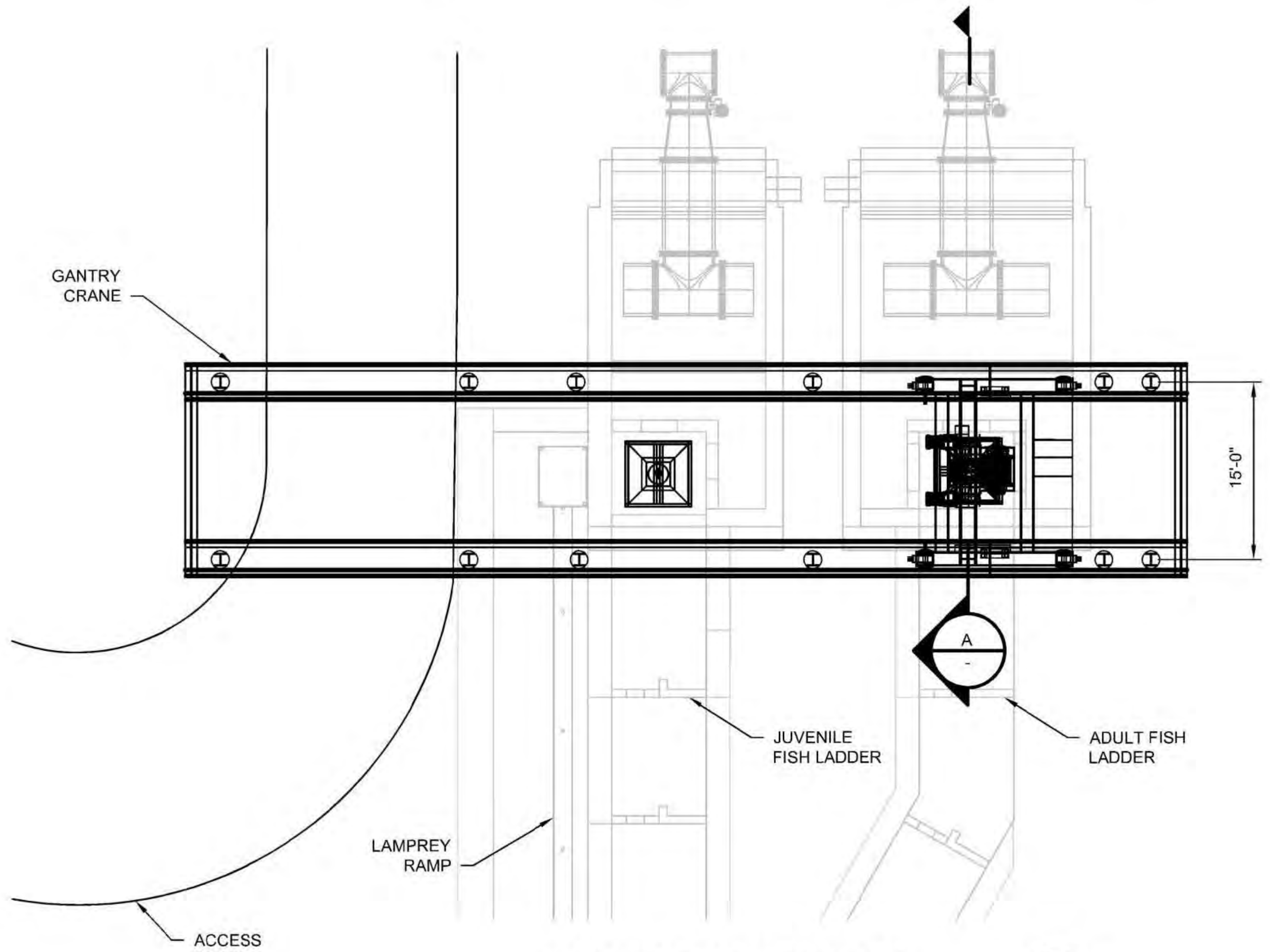
MECHANICAL

SORTING TABLE PLAN AND SECTIONS
AND TRAP BRAIL HOIST

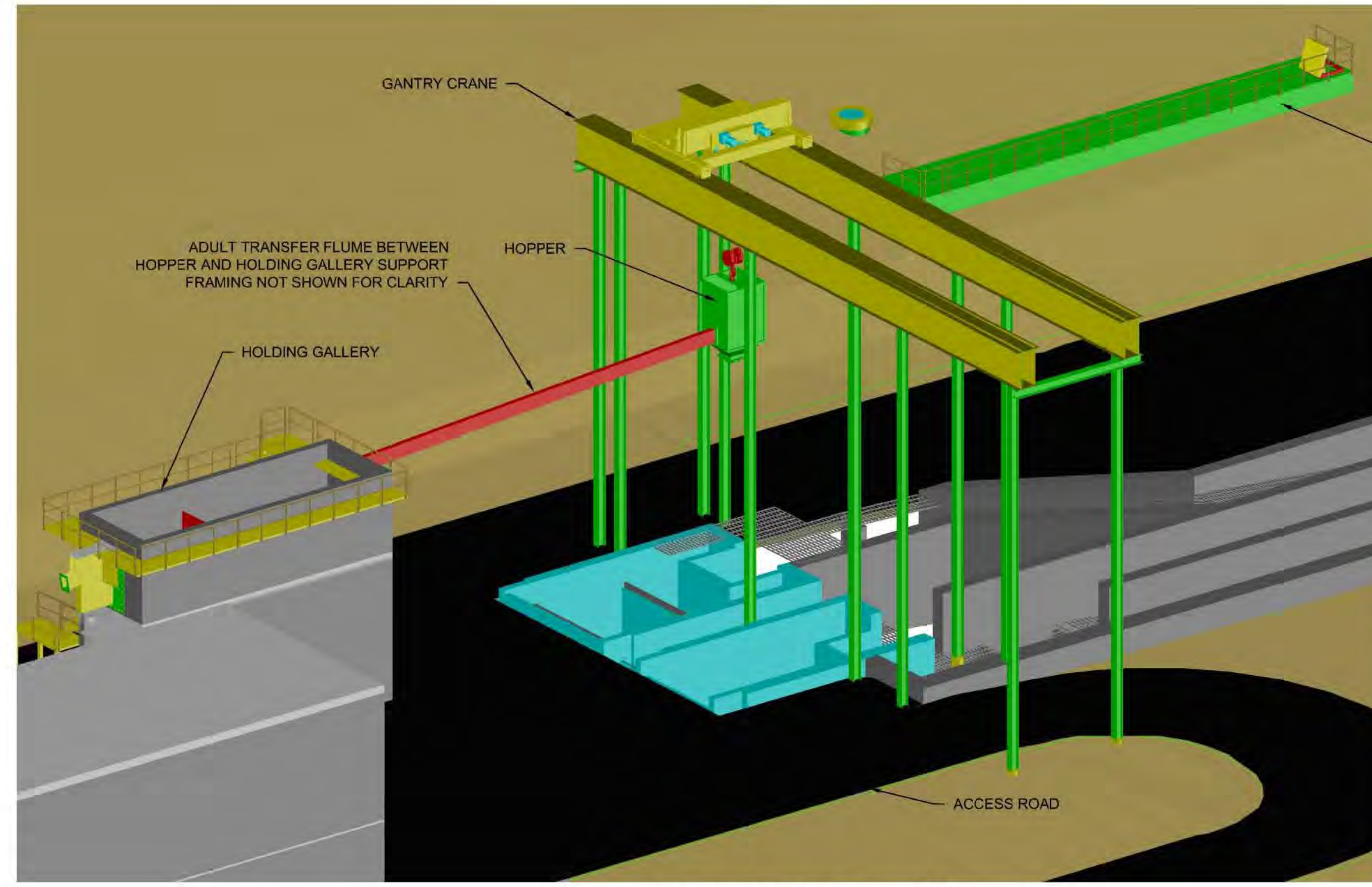
0 1" 2"

DATE	JUNE, 2017
SCALE	AS SHOWN

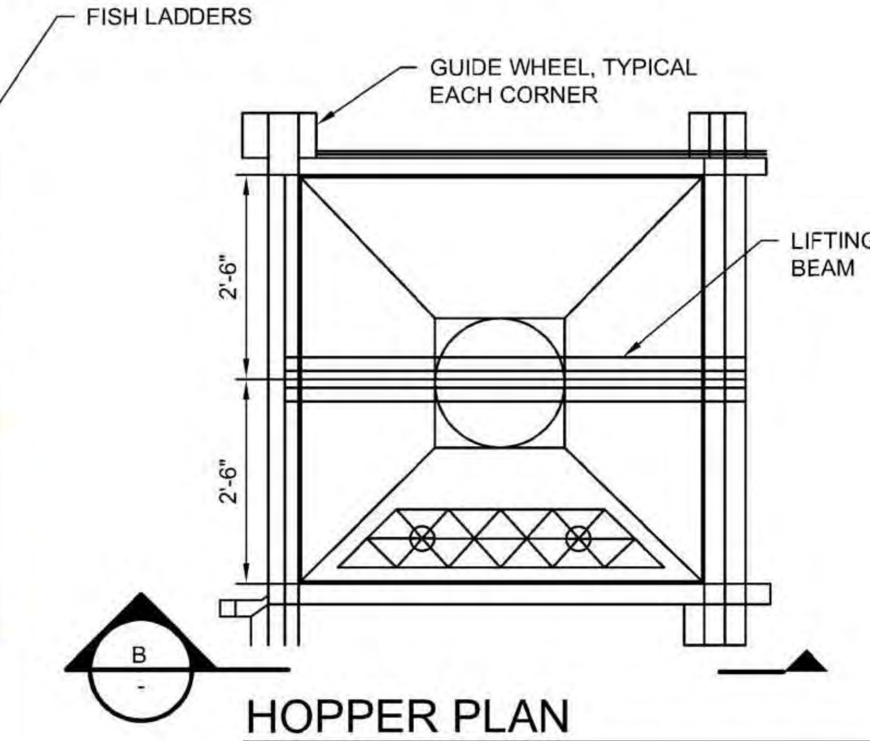
SHEET
M-5



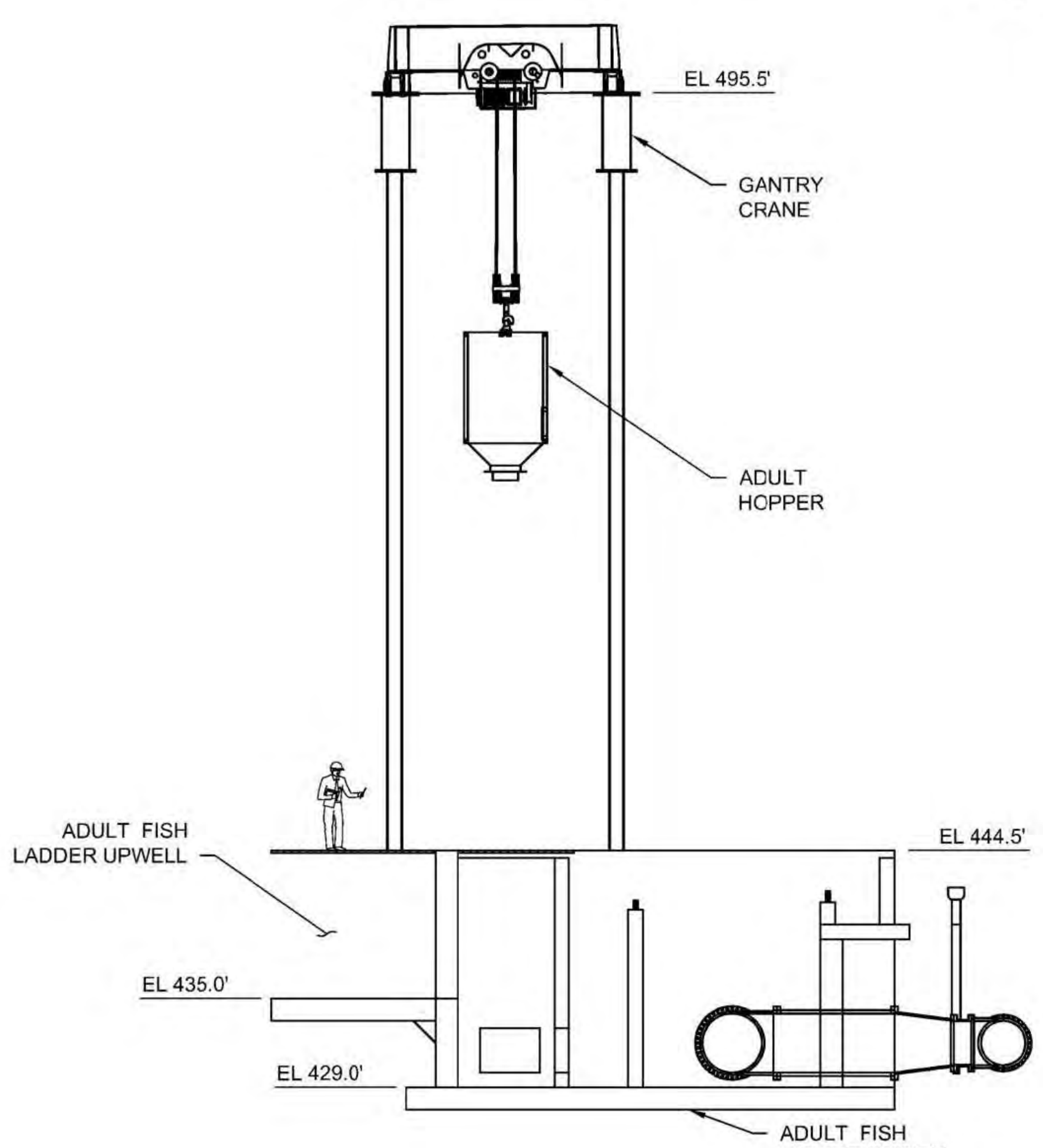
GANTRY CRANE PLAN 1
SCALE: 1" = 10'



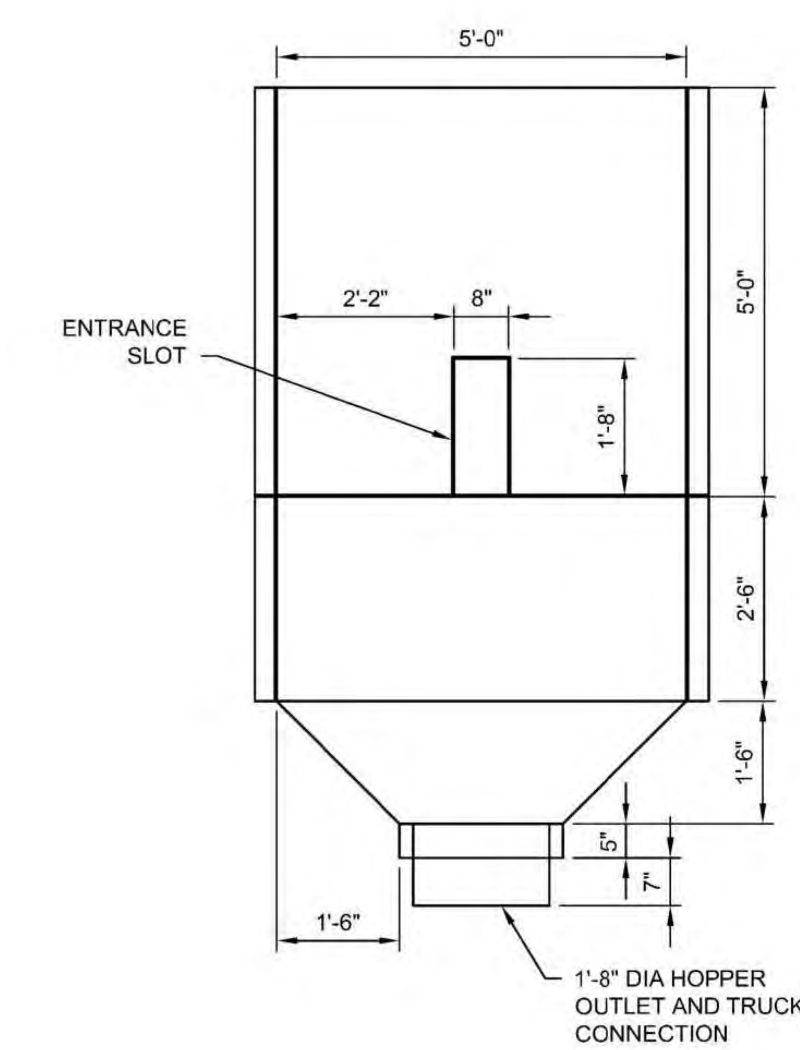
GANTRY CRANE ISOMETRIC
SCALE: NTS



HOPPER PLAN
SCALE: 1/2" = 1'-0"



GANTRY CRANE SECTION A
SCALE: 1" = 10'



HOPPER SECTION B
SCALE: 1/2" = 1'-0"

10 0 10 20 SCALE IN FEET

2 0 2 4 SCALE IN FEET

PRELIMINARY
NOT FOR CONSTRUCTION

HDR

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER	
CHECKED BY	MATT PROCIV
CHECKED BY	
DRAWN BY	DON DADE
DESIGNED BY	DON DADE
DESIGNED BY	
PROJECT NUMBER	268421

THE SEAL OF THE STATE OF WASHINGTON 1889

STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

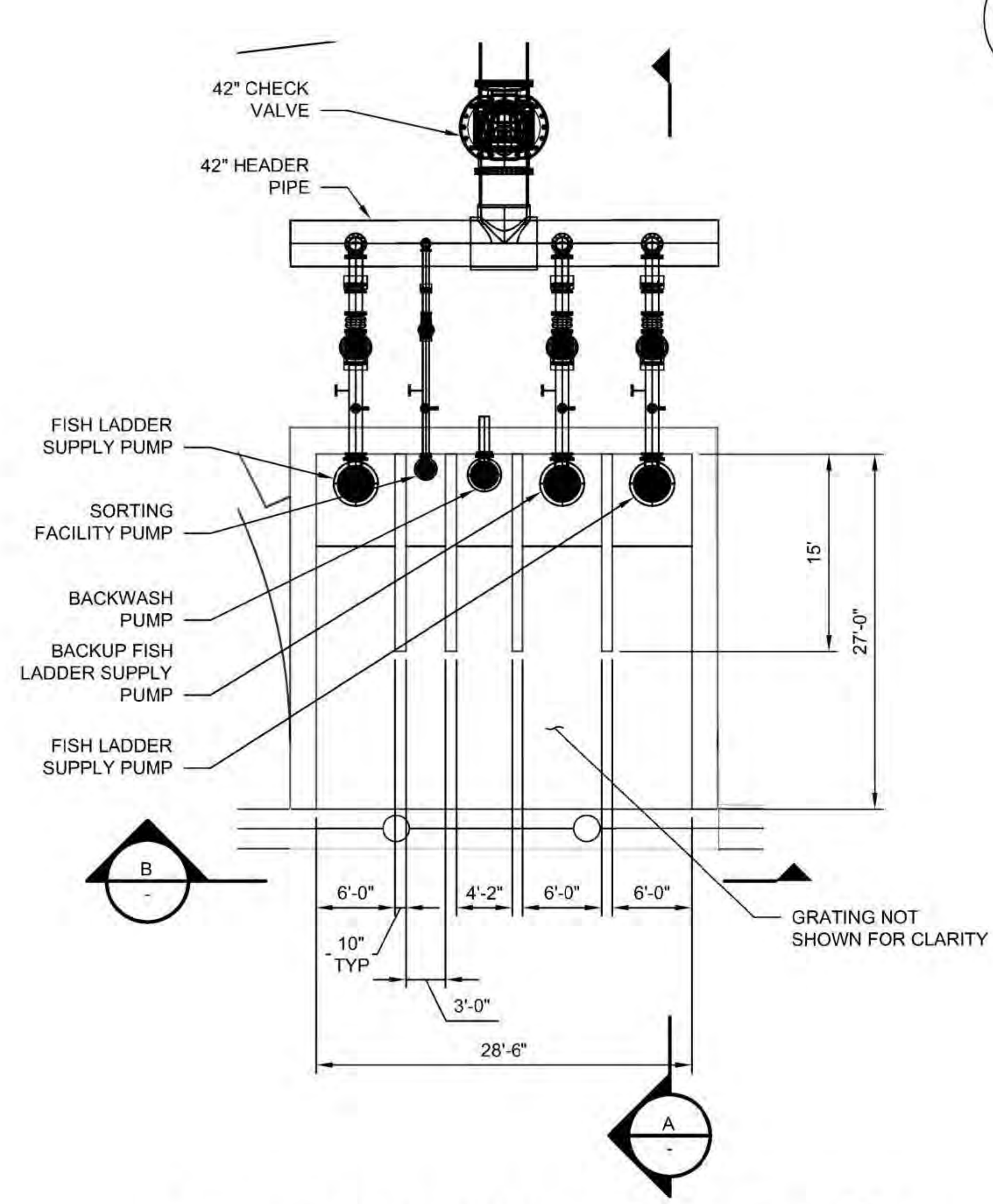
MECHANICAL

GANTRY CRANE AND HOPPER PLANS, SECTIONS AND DETAILS

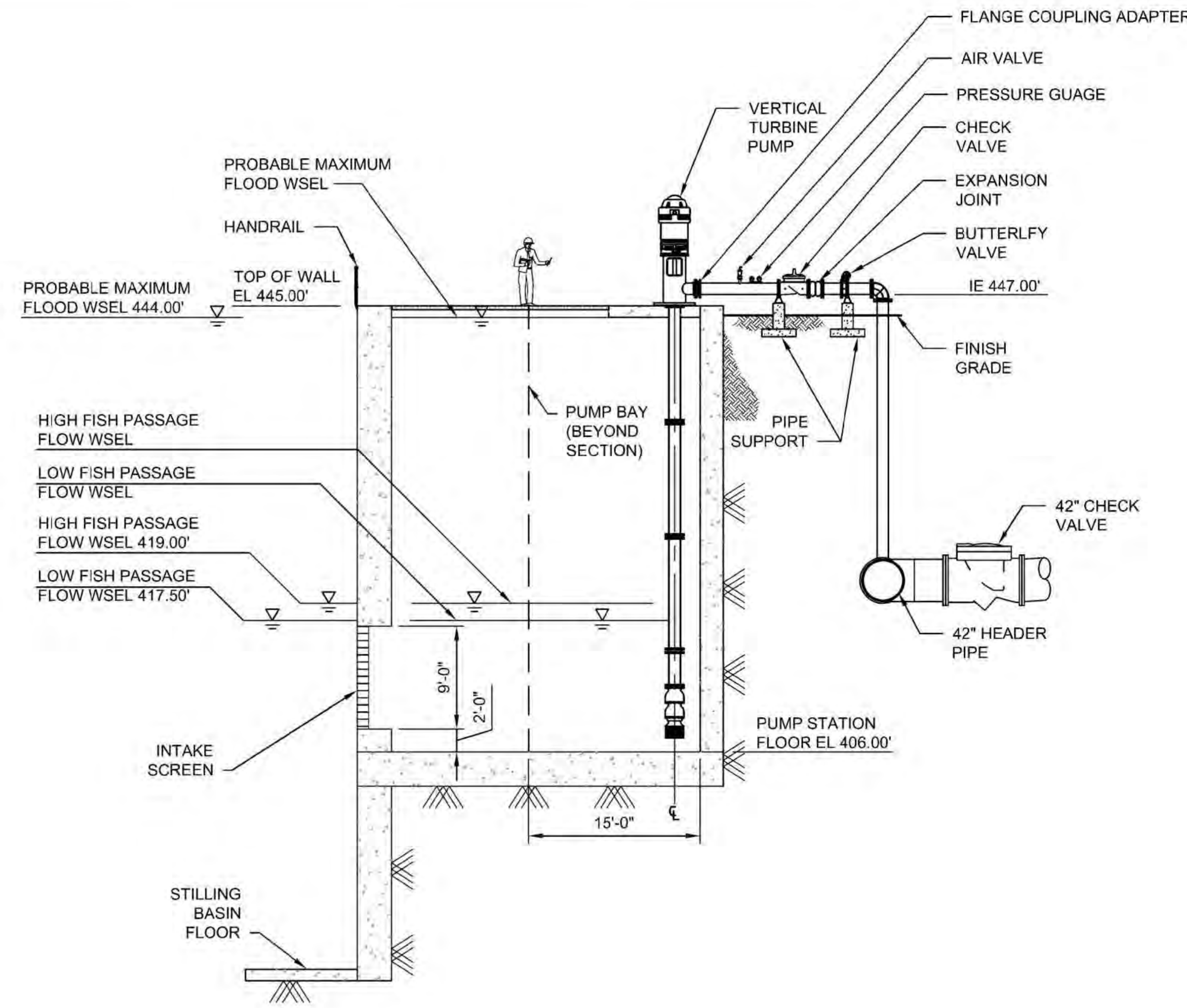
0 1" 2"

DATE	JUNE, 2017
SCALE	AS SHOWN

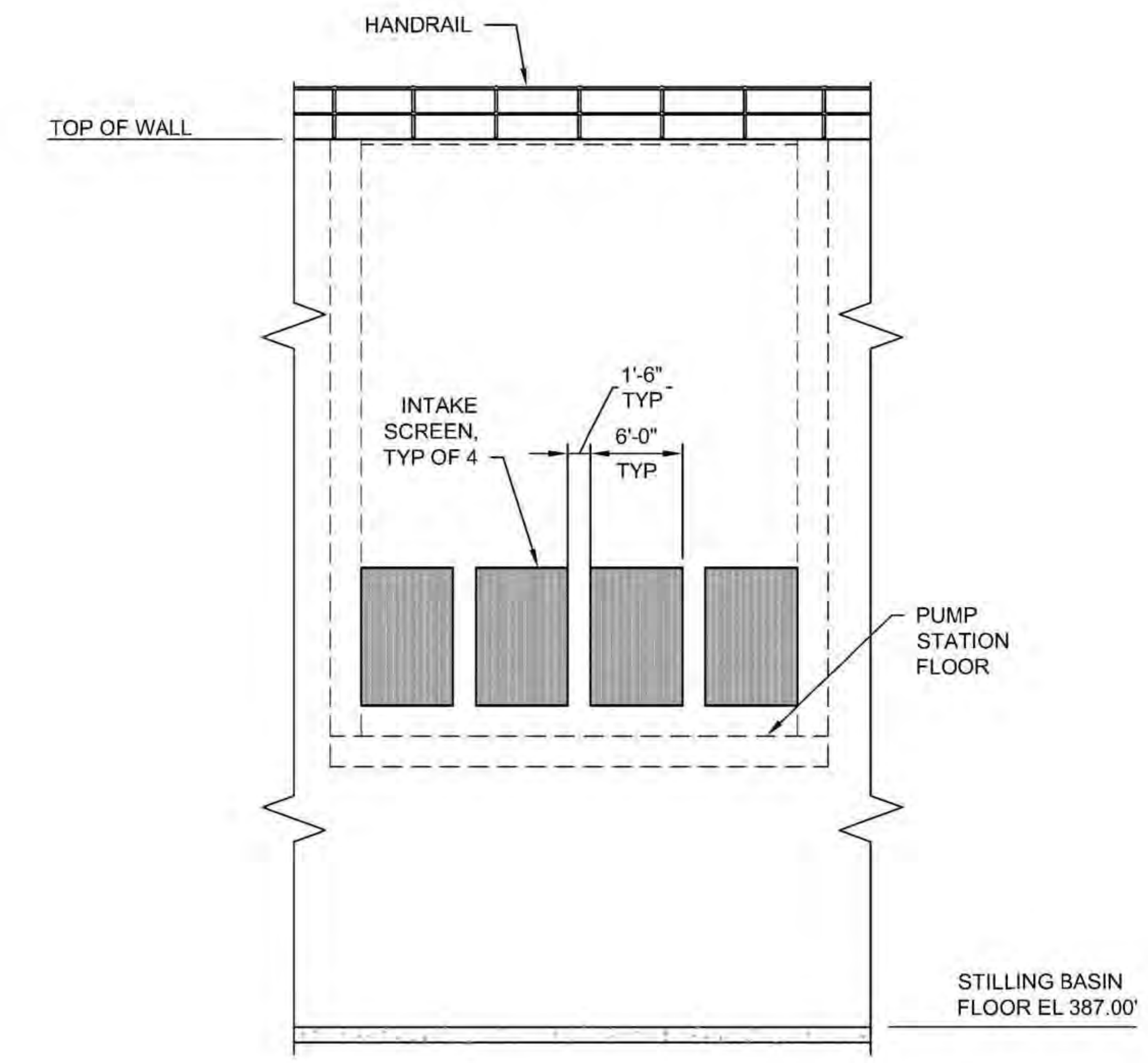
SHEET
M-6



AUXILLARY PUMP STATION - PLAN 1



SECTION A



SECTION B

10 0 10 20
SCALE IN FEET

PRELIMINARY
NOT FOR CONSTRUCTION

ISSUE	DATE	DESCRIPTION

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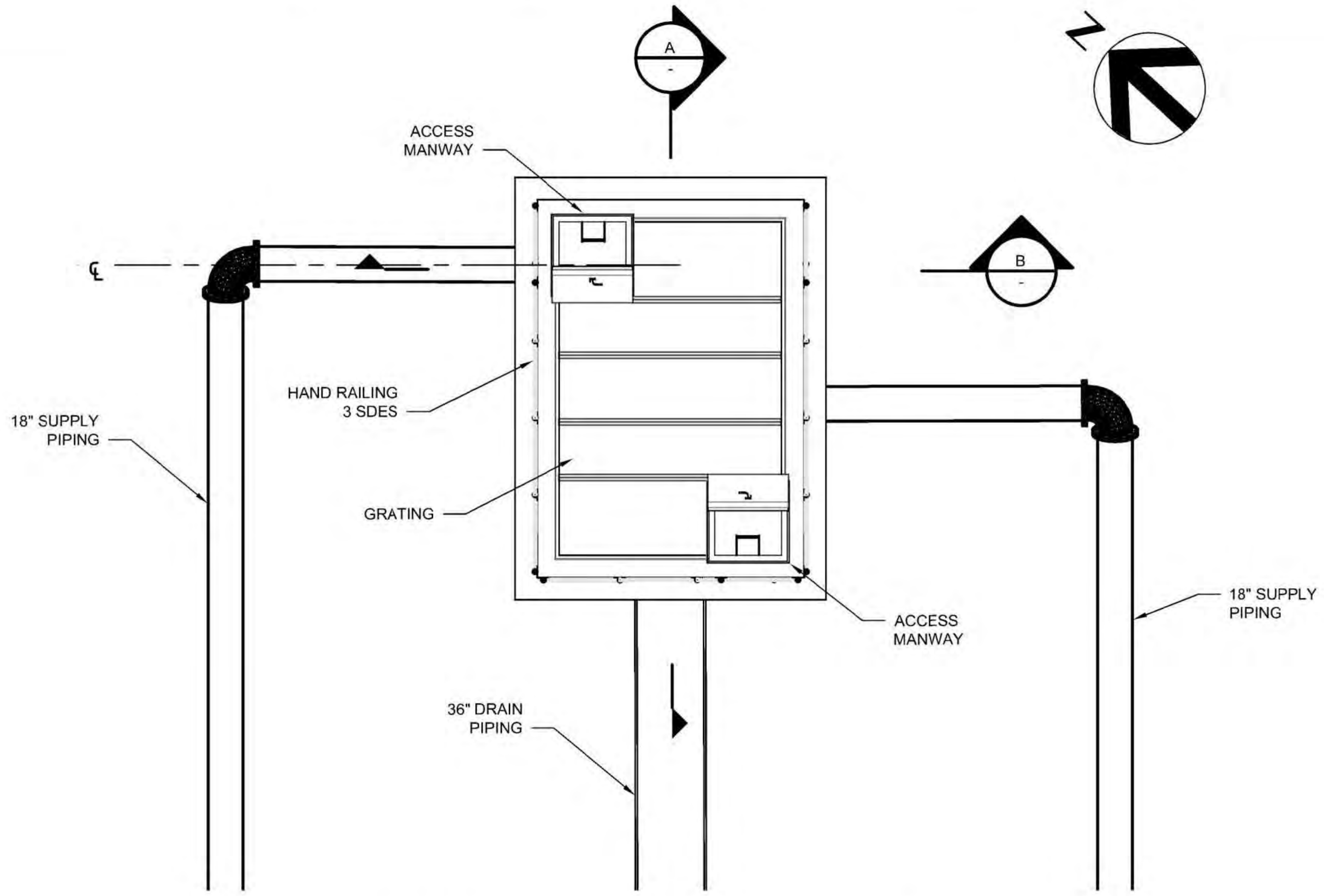
STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

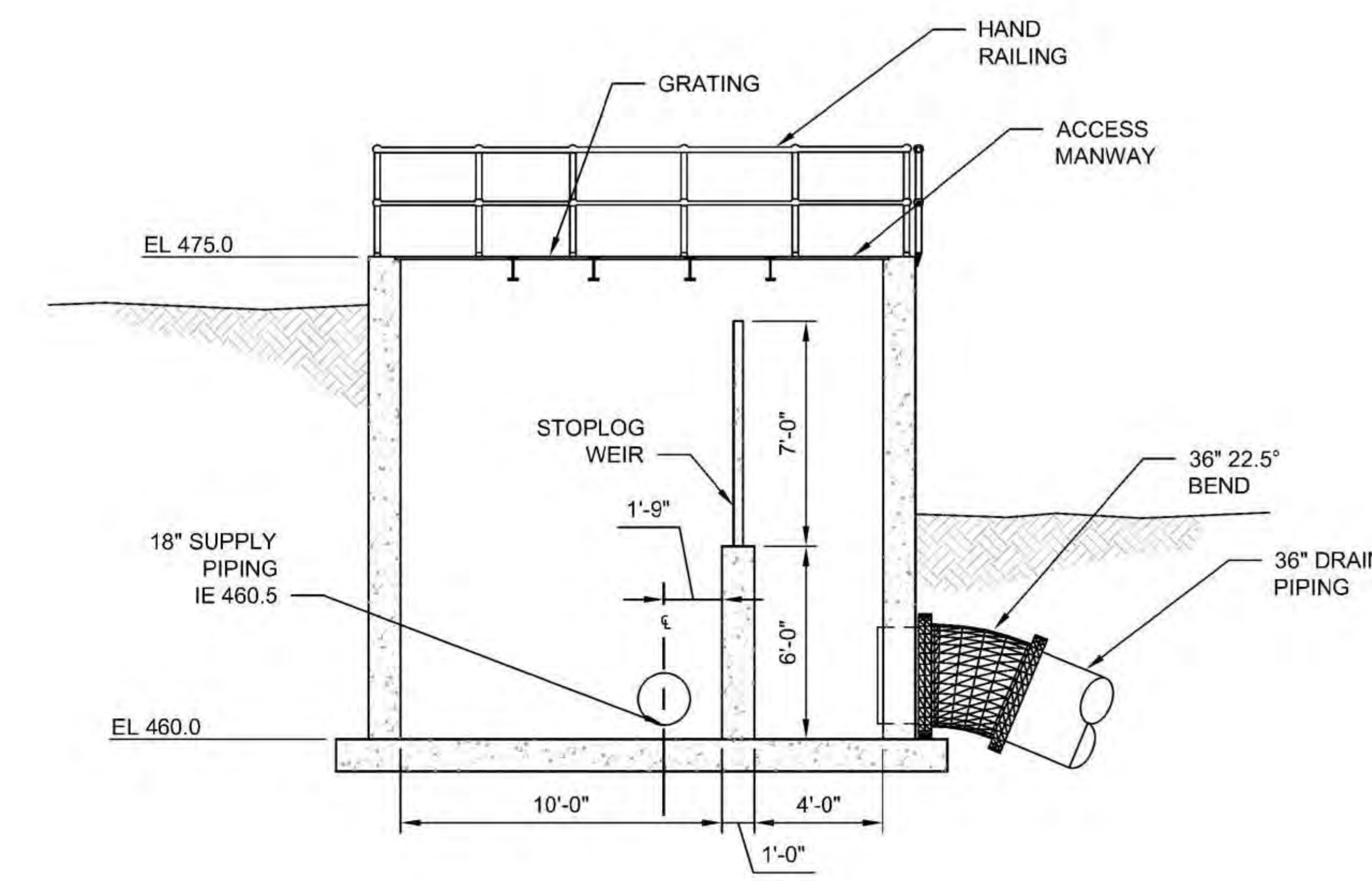
MECHANICAL

**AUXILIARY PUMP STATION
PLAN, SECTIONS, AND ISOMETRIC**

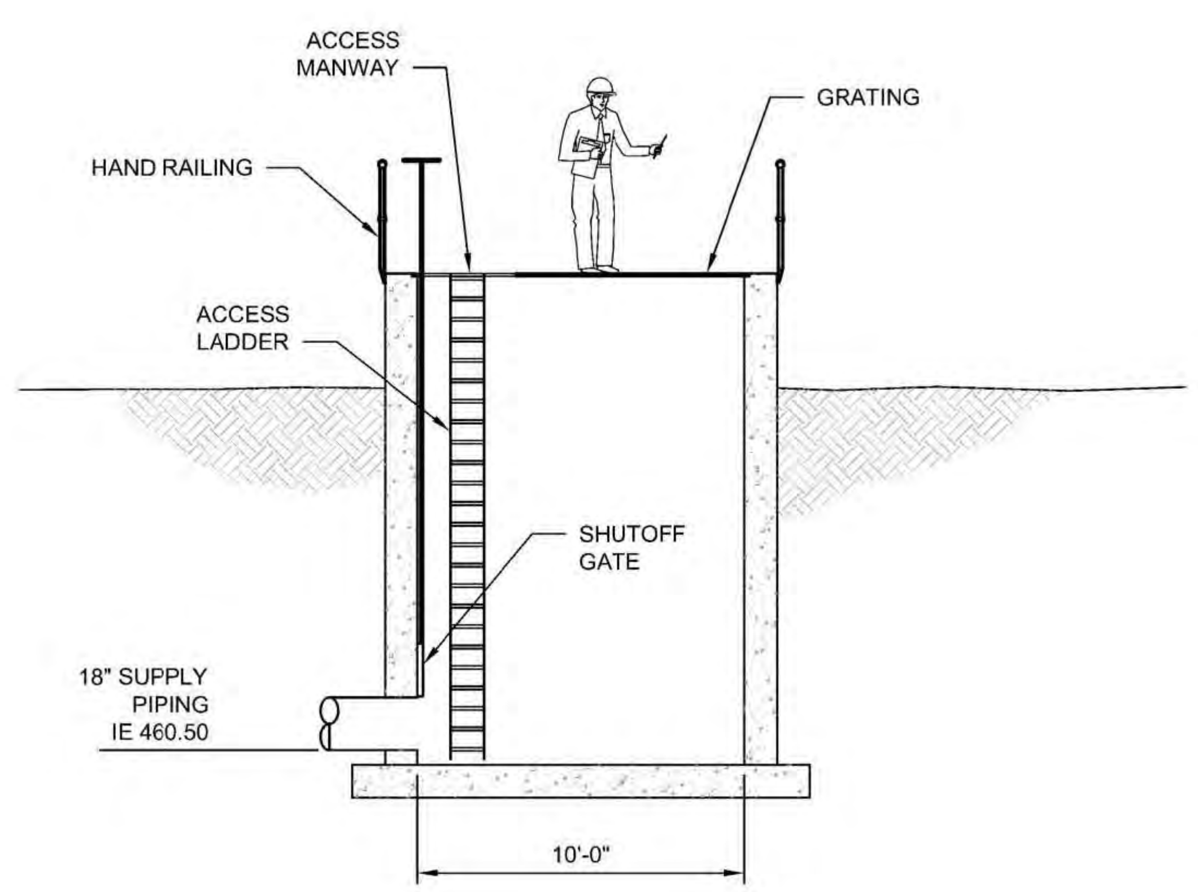
DATE	JUNE, 2017	SHEET
SCALE	1" = 10'	M-7



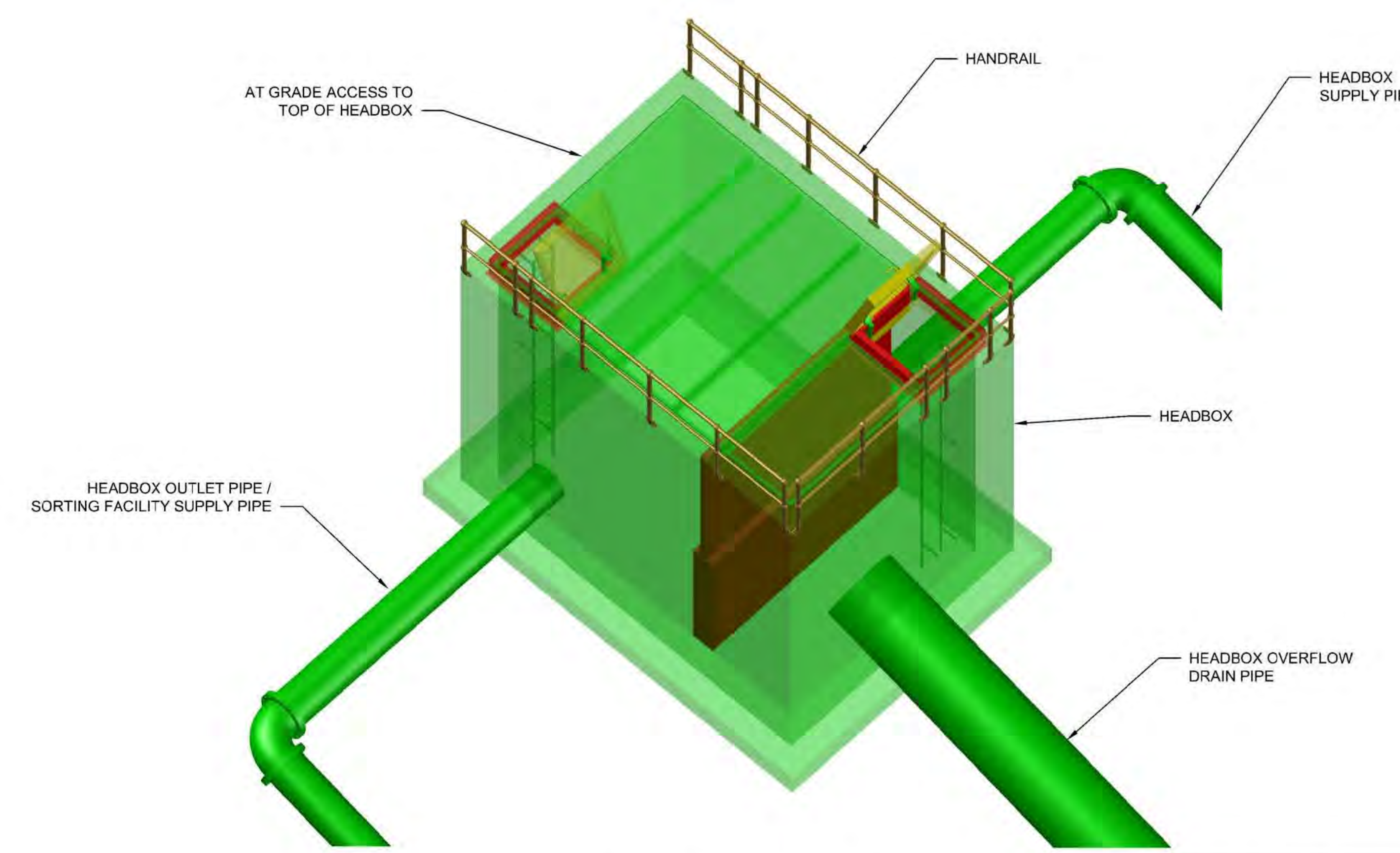
HEADBOX - PLAN
1" = 5'
1



HEADBOX - SECTION
1" = 5'
A



HEADBOX - SECTION
1" = 5'
B



HEADBOX - ISOMETRIC
NTS

5 0 5 10
SCALE IN FEET

PRELIMINARY
NOT FOR CONSTRUCTION

HDR

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CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
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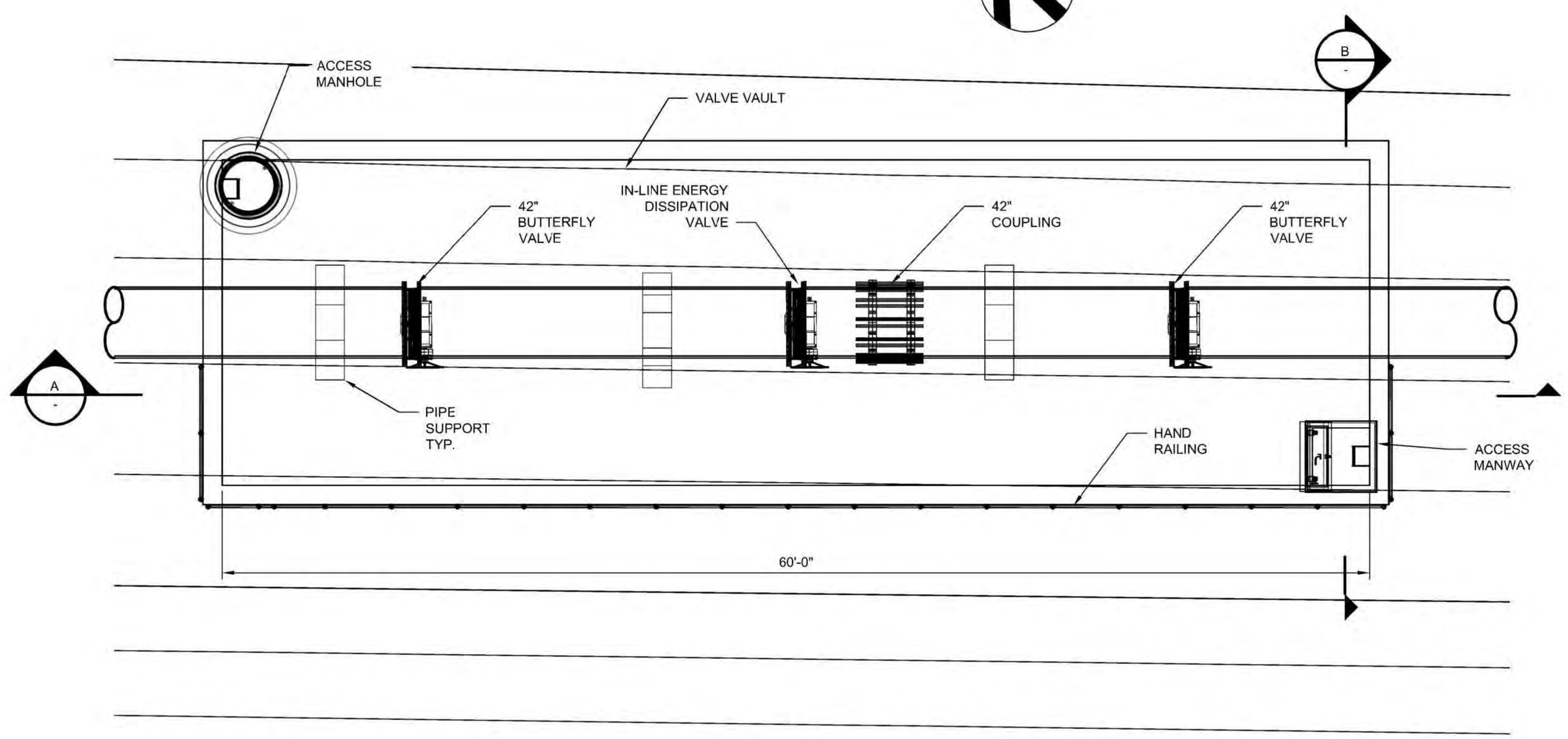
MECHANICAL

HEADBOX
PLAN, SECTIONS, AND ISOMETRIC

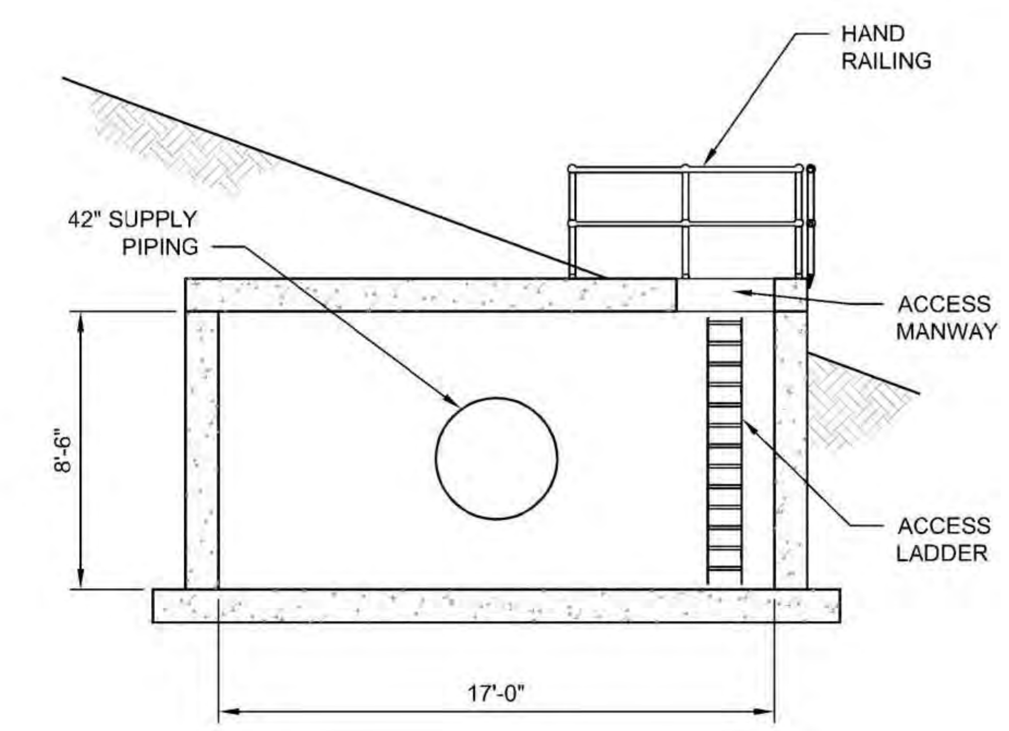
0 1" 2"

DATE	JUNE, 2017
SCALE	AS SHOWN

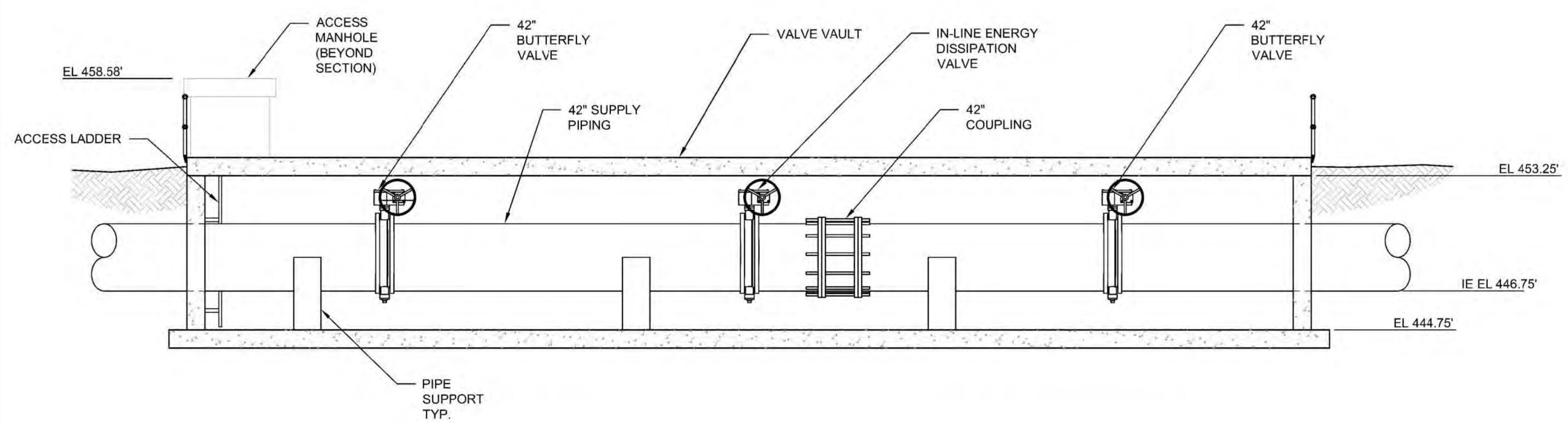
SHEET
M-8



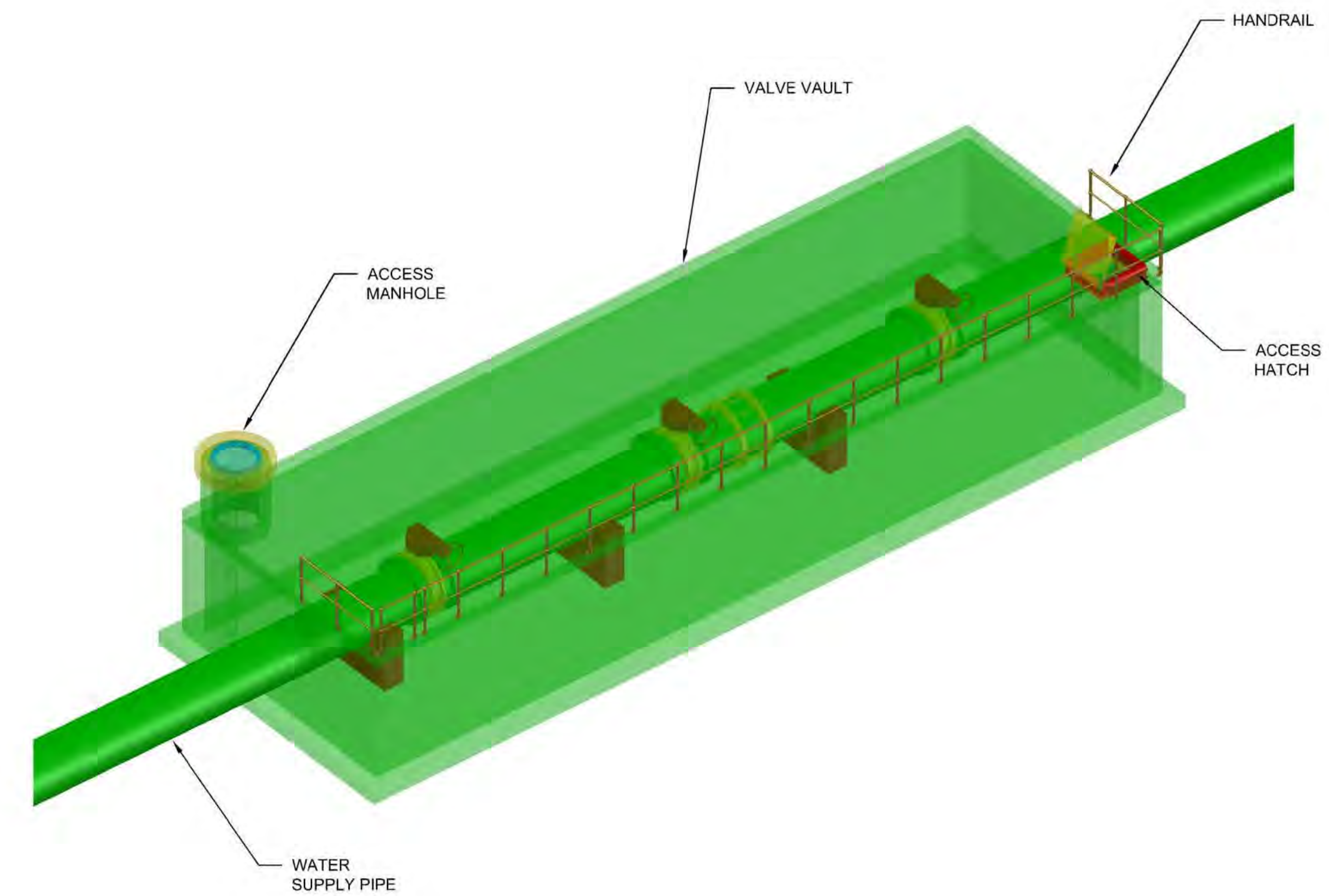
VALVE VAULT - PLAN
1" = 5'
1



VALVE VAULT - SECTION
1" = 5'
A



VALVE VAULT - SECTION
1" = 5'
B



VALVE VAULT - ISOMETRIC
N/A

5 0 5 10
SCALE IN FEET

PRELIMINARY
NOT FOR CONSTRUCTION

HDR

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STATE OF WASHINGTON
OLYMPIA
CHEHALIS BASIN STRATEGY

FRO AND HYBRID DAMS
CHTR FISH PASAGE FACILITY

MECHANICAL

VALVE VAULT
PLAN, SECTIONS, AND ISOMETRIC

0 1" 2"

DATE	JUNE, 2017
SCALE	AS SHOWN

SHEET
M-9

Appendix C
Cost Data

CHEHALIS BASIN STRATEGY
FISH PASSAGE DESIGN STUDY TEAM
OPINION OF IMPLEMENTATION AND OPERATION AND MAINTENANCE COSTS
SUMMARY OF CHTR FISH PASSAGE FACILITY COST BOUNDS AND O&M COSTS

Estimated Construction Cost

Fish Passage Alternative	Lower Bound Cost ($ Million)	Middle Cost ($ Million)	Upper Bound Cost ($ Million)
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES	$32.3	$43.0	$64.5
Lower Bound and Upper Bound Cost Margins	-25%	-	50%

Estimated Operation and Maintenance Cost

Fish Passage Alternative	Annual Cost ($)
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES	$20,000

*Note - O&M costs include an assumed Monitoring and Evaluation operation. M&E costs are a placeholder. .
LOE for M&E has not been discussed with the Fish Passage Subcommittee

CHEHALIS BASIN STRATEGY
FISH PASSAGE DESIGN STUDY TEAM
OPINION OF IMPLEMENTATION AND OPERATION AND MAINTENANCE COSTS
SUMMARY OF CHTR FISH PASSAGE FACILITY COSTS

FYI - NEGLIGIBLE ESCALATION FROM 2016 TO 2018 PER WSDOT DATA ON PRICE PER CY STRUCTURAL CONCRETE AND REBAR

Table 1 - Project implementation costs for all alternatives shown as a percentage of the OPCC.

PROJECT IMPLEMENTATION COSTS	PERCENTAGE OF OPCC
CONSTRUCTION MANAGEMENT	8.00%
APS PROCUREMENT	4.00%
ENGINEERING AND DESIGN	10.00%
BOND AND INSURANCE	2.50%
STATE TAXES	7.90%
BUSINESS AND OCCUPATION TAX	0.48%
PROJECT ADMINISTRATIVE (NOT INCLUDED)	0.00%
TOTAL PERCENTAGE OF OPCC	**32.88%**

Table 2 - Summary of concept OPCC (rounded to $100,000).

ALTERNATIVE	BASE OPCC W/ CONT
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES	$32,300,000

Table 3 - Summary of OPCC, implementation cost, and total project costs for each concept (rounded to $100,000).

ALTERNATIVE	BASE OPCC	IMPLEMENTATION COST	TOTAL PROJECT COST
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES	$32,300,000	$10,700,000	$43,000,000

Table 4 - Summary of anticipated Operations and Maintenance Costs (rounded to $1,000).

ALTERNATIVE	BASE O&M COST
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES	$20,000

CHEHALIS BASIN STRATEGY
FISH PASSAGE DESIGN STUDY TEAM
OPINION OF IMPLEMENTATION AND OPERATION AND MAINTENANCE COSTS
SUMMARY OF CHTR FISH PASSAGE FACILITY OPERATION AND MAINTENANCE LEVEL OF EFFORT

Table 1 - Project implementation costs for all alternatives shown as a percentage of the OPCC.

	STAFF FTEs			
ALTERNATIVE	OPERATIONS	BIOLOGICAL	MAINTENANCE	TOTAL
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES	0.03	0.02	0.04	0.09

Computed by: R. Sheean
Created: 06/07/2017
Printed: 1/23/2018
Checked by: M. Prociv
Checked Date: 06/28/2017

CHEHALIS BASIN STRATEGY
FISH PASSAGE DESIGN STUDY TEAM
OPINION OF IMPLEMENTATION COSTS
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES

ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTAL
MOBILIZATION AND DEMOBILIZATION (10%)	1	LS	$1,909,527	$1,909,527	**$1,909,527**
GENERAL CONDITIONS (20%)	1	LS	$3,819,055	$3,819,055	**$3,819,055**
LANDSLIDE MITIGATION					**$1,230,424**
LANDSLIDE MITIGATION	1	LS	$500,000	$500,000	
GENERAL EXCAVATION	51,856	CY	$8	$414,848	
FILL	35,064	CY	$9	$315,576	
SITEWORK AND ACCESS IMPROVEMENTS					**$4,135,393**
EROSION CONTROL	1	LS	$40,000	$40,000	
CLEAR AND GRUB	13	ACRE	$7,500	$97,500	
ROAD AND ACCESS IMPROVEMENTS	2,171	TON	$159	$345,189	
GENERAL EXCAVATION	155,371	CY	$8	$1,242,968	
ROCK EXCAVATION	41,637	CY	$30	$1,249,110	
FILL	109,514	CY	$9	$985,626	
SITE CLEAN-UP	1	LS	$15,000	$15,000	
PROJECT ISOLATION DURING CONSTRUCTION	1	LS	$60,000	$60,000	
LOCAL DEWATERING	1	LS	$100,000	$100,000	
RIVER DEWATERING (COVERED BY DAM EST.)	0	LS	$0	$0	
INTAKE AND PUMP STATION					**$1,076,156**
CONCRETE SLABS	107	CY	$800	$85,333	
CONCRETE WALLS	166	CY	$950	$157,911	
CONCRETE BAFFLES	63	CY	$1,100	$69,259	
SCREENS & BAFFLES	216	SF	$1,000	$216,000	
GRATING	770	SF	$35	$26,933	
HANDRAIL	120	LF	$56	$6,720	
BACKWASH SYSTEM	1	LS	$100,000	$100,000	
FISH LADDER SUPPLY PUMPS	1	EA	$138,000	$138,000	
SORTING FACILITY PUMP	1	EA	$138,000	$138,000	
BACKWASH PUMP	1	EA	$138,000	$138,000	
FISH LADDER ENTRANCE					**$3,340,727**
CONCRETE SLABS	1,392	CY	$800	$1,113,481	
CONCRETE WALLS	1,891	CY	$950	$1,796,731	
CONCRETE BAFFLES	134	CY	$1,100	$146,993	
FILL	3,824	CY	$9	$34,417	
ENTRANCE GATES	5	EA	$40,000	$200,000	
GRATING	1,243	SF	$35	$43,505	
HANDRAIL	100	LF	$56	$5,600	

Computed by: R. Sheean
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CHEHALIS BASIN STRATEGY
FISH PASSAGE DESIGN STUDY TEAM
OPINION OF IMPLEMENTATION COSTS
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES

ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTAL
AUXILIARY WATER SUPPLY					**$1,160,755**
CONCRETE ELEVATED SLABS	28	CY	$1,100	$30,433	
CONCRETE WALLS	336	CY	$950	$319,552	
GRATING	1,462	SF	$35	$51,170	
HANDRAIL	100	LF	$56	$5,600	
42" SLEEVE VALVE	1	LS	$250,000	$250,000	
SCREENS & BAFFLES	504	SF	$1,000	$504,000	
ADULT FISH LADDER					**$765,549**
CONCRETE SLABS	124	CY	$800	$99,200	
CONCRETE WALLS	578	CY	$950	$548,994	
CONCRETE BAFFLES	37	CY	$1,100	$41,230	
GRATING	1,335	SF	$35	$46,725	
HANDRAIL	300	LF	$56	$16,800	
SCREENS	36	SF	$350	$12,600	
JUVENILE FISH LADDER					**$1,187,282**
CONCRETE SLABS	118	CY	$800	$94,667	
CONCRETE WALLS	1,032	CY	$950	$980,083	
CONCRETE BAFFLES	39	CY	$1,100	$43,022	
GRATING	1,314	SF	$35	$45,990	
HANDRAIL	270	LF	$56	$15,120	
SCREENS	24	SF	$350	$8,400	
LAMPREY RAMP					**$961,146**
CONCRETE SLABS	146	CY	$800	$116,800	
CONCRETE WALLS	809	CY	$950	$768,796	
CONCRETE BAFFLES	45	CY	$1,100	$49,296	
STEEL RAMP ASSEMBLY - PLATE	1,416	SF	$12	$16,992	
STEEL RAMP ASSEMBLY - STRUCTURAL STEEL	2,646	LB	$3.5	$9,261	
HOLDING GALLERY					**$1,145,149**
CONCRETE ELEVATED SLABS	15	CY	$1,100	$16,337	
CONCRETE WALLS	95	CY	$950	$90,637	
BRAIL AND HOPPER	1	LS	$496,759	$496,759	
CROWDER	1	LS	$75,000	$75,000	
FALSE WEIR	1	LS	$50,000	$50,000	
STAIRS	48	RISER	$667	$32,016	
SUSPENDED GRATING	384	SF	$350	$134,400	
GANTRY CRANE	1	LS	$250,000	$250,000	

Computed by: R. Sheean
Created: 06/07/2017
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CHEHALIS BASIN STRATEGY

FISH PASSAGE DESIGN STUDY TEAM

OPINION OF IMPLEMENTATION COSTS

CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES

ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTAL
SORTING BUILDING					**$528,525**
CONCRETE SLABS	89	CY	$800	$71,022	
CONCRETE ELEVATED SLAB	21	CY	$1,100	$22,611	
CONCRETE WALLS	254	CY	$950	$241,687	
JOB WATER BOOSTER PUMP	2	EA	$50,000	$100,000	
CIRCULAR HOLDING TANKS	3	EA	$4,530	$13,590	
INSPECTION TABLE	1	EA	$2,000	$2,000	
DIVERTER FLUME	66	SF	$12	$792	
DIVERTER GATE	4	EA	$2,000	$8,000	
ANAESTHESIA TANK	1	LS	$10,000	$10,000	
WORKUP TABLE	1	EA	$2,000	$2,000	
STAIRS	29	RISER	$667	$19,343	
HANDRAIL	580	LF	$56	$32,480	
TRUCK FILL	1	LS	$5,000	$5,000	
MAINTENANCE/STORAGE BUILDING					**$248,767**
PRE-ENGINEERED BUILDING	800	SF	$250	$200,000	
CONCRETE FOUNDATION	51	CY	$950	$48,767	
HEADBOX					**$46,126**
CONCRETE SLAB	10	CY	$800	$7,881	
CONCRETE WALLS	30	CY	$950	$28,465	
CONCRETE WEIRS	3	CY	$1,100	$3,259	
ACCESS LADDER	2	EA	$2,000	$4,000	
HANDRAIL	45	LF	$56	$2,520	
VALVE VAULT					**$158,417**
CONCRETE SLAB	50	CY	$800	$39,822	
CONCRETE ELEVATED SLAB	43	CY	$1,100	$47,341	
CONCRETE WALLS	50	CY	$950	$47,254	
HATCHES	2	EA	$12,000	$24,000	
PIPING					**$1,681,858**
2" - 8" Steel Pipe	881	LF	$64	$56,384	
10" Steel Pipe	14	LF	$74	$1,030	
12" Steel Pipe	307	LF	$85	$26,095	
18" Steel Pipe	301	LF	$133	$39,928	
36" Steel Pipe	101	LF	$403	$40,703	
42" Steel Pipe	274	LF	$714	$195,636	
90" Concrete Pipe	647	LF	$756	$489,132	
2" - 8" Ductile Iron Bends, Wyes, and Tees	46	EA	$713	$32,798	
10" Ductile Iron Bends, Wyes, and Tees	4	EA	$985	$3,940	
12" Ductile Iron Bends, Wyes, and Tees	5	EA	$1,301	$6,505	
18" Ductile Iron Bends, Wyes, and Tees	18	EA	$3,104	$55,872	
36" Ductile Iron Bends, Wyes, and Tees	3	EA	$0	$0	
42" Ductile Iron Bends, Wyes, and Tees	3	EA	$0	$0	
42" In-Line Energy Dissipation Valve	1	EA	$100,000	$100,000	
3" - 4" Ductile Iron Backflow Valve	8	EA	$3,125	$25,000	
18" Cast Iron Gate Valve	1	EA	$22,596	$22,596	
36" Cast Iron Gate Valve	1	EA	$68,996	$68,996	
42" Cast Iron Gate Valve/ Backflow Valve/ Sleeve Valve	4	EA	$79,311	$317,244	
90" Cast Iron Backflow Valve	1	EA	$200,000	$200,000	

CHEHALIS BASIN STRATEGY
FISH PASSAGE DESIGN STUDY TEAM
OPINION OF IMPLEMENTATION COSTS
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES

ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTAL
FISH TRANSPORT VEHICLE					**$110,000**
FISH TRANSFER TRUCK	1	LS	$110,000	$110,000	
BASE ELECTRICAL EQUIPMENT & SUPPLY					**$1,319,000**
ELECTRIC SUPPLY	1	LS	$150,000	$150,000	
ELECTRICAL SERVICE SITE IMPROVEMENTS	1	LS	$425,000	$425,000	
BACKUP GENERATOR	1	LS	$275,000	$275,000	
SERVICE ENTRANCE	1	LS	$125,000	$125,000	
ELECTRICAL DISTRIBUTION EQUIPMENT	1	LS	$50,000	$50,000	
GROUNDING	1	LS	$30,000	$30,000	
POWER AND CONTROLS TO EQUIPMENT	1	LS	$35,000	$35,000	
LIGHTING	1	LS	$17,500	$17,500	
EQUIPMENT CONNECTIONS	1	LS	$5,000	$5,000	
INSTRUMENTATION AND CONTROL EQUIPMENT	1	LS	$25,000	$25,000	
TRANSMITTERS AND PRIMARY ELEMENTS	1	LS	$25,000	$25,000	
PIT TAG EQUIPMENT	1	LS	$85,000	$85,000	
PIT ANTENNAS	1	LS	$15,000	$15,000	
EQUIPMENT CONNECTIONS	1	LS	$6,500	$6,500	
SCADA	1	LS	$50,000	$50,000	
SUBTOTAL CONSTRUCTION COSTS					**$24,823,856**
UNDEFINED DESIGN AND CONSTRUCTION ITEMS (30%)					**$7,447,157**
SUBTOTAL W/ CONTINGENCY					**$32,271,013**
Construction Management	8%	LS	$32,271,013	$2,581,681	
APS Procurement	4%	LS	$32,271,013	$1,290,841	
Design Contingency	10%	LS	$32,271,013	$3,227,101	
Bond and Insurance	3%	LS	$32,271,013	$806,775	
TOTAL ROUNDED OPCC					**$40,177,412**
Sales Tax	7.90%	LS	$18,079,835	$1,428,307	
B&O Tax	0.48%	LS	$41,605,719	$201,372	
TOTAL OPCC					**$41,807,090**

CHEHALIS BASIN STRATEGY
FISH PASSAGE DESIGN STUDY TEAM
OPINION OF OPERATION AND MAINTENANCE COSTS
CHTR FISH PASSAGE FACILITY FOR FRO AND HYBRID (RUN-OF-RIVER) DAM ALTERNATIVES

Item	Quantity	Unit	Unit Cost	Amount	Total
LABOR	**1**	**LS**		**$7,576**	**$7,576**
Maintenance direct labor cost	0.008	FTE	$57,000	$470	
(average 30 hr/week for 1-month operating period every 7 years)					
Maintenance benefits @1.15 labor cost	0.008	FTE	$65,550	$540	
1- Fisheries technician direct labor cost	0.011	FTE	$60,000	$659	
(full time for 1-month operating period every 7 years)					
Fisheries technician benefits @ 1.15 labor cost	0.011	FTE	$69,000	$758	
2 - Seasonal technicians direct labor cost	0.015	FTE	$24,000	$369	
(average 28 hrs/week for 1-month intensive operating period every 7 years)					
Seasonal technician benefits @ 0.85 labor cost	0.015	FTE	$20,400	$314	
Annual inspections and Maintenance	0.035	FTE	$60,000	$2,077	
(assume 3 people for annual (1) 3-day period)					
Annual inspections and Maintenance	0.035	FTE	$69,000	$2,388	
FTE = Full time equivalent					
TRANSPORT (1 Diesel Vehicle at 10 MPG and $4/gallon)	**1**	**LS**		**$3,038**	**$3,038**
Assume 20 Mile Round Trip to a Release Site Above Reserovoir (Fuel)	86	MILES	$0	$38	
Assume 1 Trip per day for each of the year, all year					
Annual Maintenance	1	LS	$3,000	$3,000	
MONITORING & EVALUATION - PLACEHOLDER	**1**	**LS**		**$2,138**	**$2,138**
1- Fisheries technician direct labor cost	0.008	FTE	$60,000	$495	
(average 30 hr/week for 1-month operating period every 7 years)					
Fisheries technician benefits @ 1.15 labor cost	0.008	FTE	$69,000	$569	
2 - Seasonal technicians direct labor cost	0.011	FTE	$24,000	$264	
(average 28 hrs/week for 1-month intensive operating period every 7 years)					
Seasonal technician benefits @ 0.85 labor cost	0.011	FTE	$20,400	$224	
Associated science costs (e.g. - lab tests, etc.)	0.016	LS	$37,000	$587	
(for 1-month operating period every 7 years)					
EXPENDABLES AND REPLACEMENT COSTS	**1**	**LS**	**$3,000**	**$3,000**	**$3,000**
ELECTRICAL	**1**	**LS**		**$3,472**	**$3,472**
Assume average of 30 kWh/day	1,211	kWh	$0	$109	
Pumping Costs (assume 1 month operation every 7 years)	37,367	kWh	$0	$3,363	
TOTAL ANTICIPATED OPERATIONS AND MAINTENANCE COSTS					**$19,224**