Combined Dam and Fish Passage Alternatives

Technical Memorandum

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

LIST OF ACRONYMS AND ABBREVIATIONS

AACE	Association for the Advancement of Cost Engineering
ASR	alkali-silica reactivity
AWS	auxiliary water supply
cfs	cubic feet per second
CHTR	collect, handle, transfer, and release
FGS	Fish guidance system
FRO	flood retention only
FSC	Floating Surface Collector
FTE	full time equivalent
ft/sec	feet per second
g	gravity
I-5	Interstate 5
HEC-RAS	Hydrologic Engineering Center River Analysis System
LWM	large woody material
MW	megawatts (power)
MWh	megawatt-hours (energy)
mm	millimeter
O&M	operations and maintenance
OFM	Office of Financial Management
PFFC	portable floating fish collector
pga	peak ground acceleration
PMF	probable maximum flood
RCC	Roller compacted concrete
SR	State Route
TM	Technical Memorandum
USACE	United States Army Corps of Engineers
Work Group	Chehalis Basin Work Group
WSDOT	Washington State Department of Transportation

Executive Summary

1 Executive Summary

1.1 INTRODUCTION

This report represents the culmination of a Dam and Fish Passage Alternative study to evaluate alternative types of dams and fish passage systems that are technically feasible at the Chehalis River Retention Structure site near Pe Ell, Washington.

1.2 CHEHALIS DAM STRUCTURE OPTIONS

The Dam Design Study team identified no fatal flaws that would preclude construction of a flood retention only (FRO), or a multi-purpose dam at the proposed site. Based on the available information, the identified dam site is best suited for either a roller compacted concrete (RCC), or a central clay core rockfill dam. Based on the above conclusions, three of the following technically feasible dam options were selected for further configuration development conceptual design:

1. Flood retention only RCC dam
2. Multi-purpose RCC dam
3. Multi-purpose central clay core rockfill dam

1.3 FISH PASSAGE OPTIONS

Both upstream and downstream fish passage for the flood retention only RCC dam configuration would be accomplished by the construction and operation of up to nine, 9-foot-high-by-12-foot-wide run of the river conduits through the base of the dam. These conduits would also allow debris and sediment transport during non-flood operation conditions. These conduits would be designed to mimic stream flows in the natural channel and will be equipped with debris guards and racks on the upstream entrances to prevent large debris from entering/clogging the conduits. The fish passage system will have gated control designed to close under low head conditions when operation of the reservoir is changed to a flood retention objective. The upstream gates would be used for normal operations and could be either top sealing radial or vertical slide gates. The downstream gates would be closed to prevent water from the spillway stilling basin backing up into the conduits during high flows that would occur during emergency spillway flow conditions. Hydraulic evaluations indicate that flow conditions through conduits could meet a target 2 feet per second requirement for effective fish passage.

For the multi-purpose dam structure, the Fish Passage Design Team identified a variety of feasible upstream and downstream passage systems. These systems can be paired in a variety of ways to provide a complete fish passage system. The upstream passage systems include a collect, handle, transfer, and release (CHTR) facility, conventional fishway and an experimental fishway. Downstream passage systems include combinations of collection facilities near the outlet structures for the dam or in the forebay area.

1.4 COMBINED DAM AND FISH PASSAGE ALTERNATIVES

Table 1-1 lists the four primary alternatives (A through D) that were identified for the project involving a range of dam types and fish passage systems.

Table 1-1
Combined Dam and Fish Passage Alternatives

ALTERNATIVE	DAM OPTION	UPSTREAM FISH PASSAGE OPTION	DOWNSTREAM FISH PASSAGE OPTION
Alternative A	Flood Retention RCC Dam	Run of the river tunnels at the base of the dam with CHTR facility	Run of the river tunnels at the base of the dam
Alternative B	Multi-purpose RCC Dam	CHTR facility	Combination collector facilities
Alternative C	Multi-purpose RCC Dam	Conventional fishway with an experimental fishway exit structure	Forebay collector facilities
Alternative D	Multi-purpose Rockfill Dam	Conventional fishway and exit structure	Forebay collector facilities

Subsequent to the development and evaluation of these alternatives, potential climate change scenarios were identified and two sub-alternatives to Alternative A were developed. Scenario 1 predicted an 18 percent increase in the routed flood storage requirement for the project. This corresponds to a 75,000 acre-foot flood retention storage volume when sediment and debris are considered. Scenario 2 predicted a 90 percent increase in the required flood storage volume in the reservoir. This translates to approximately 130,000 acre-feet of total flood, debris and sediment retention storage volume.

1.5 HYDROPOWER

The Dam Design Study team completed a planning level evaluation of the hydropower potential of the Chehalis project dam site. Findings from that evaluation include the following:

- The estimated total construction cost for hydropower at the site would be $14 to $18 million. This cost does not include the intake structure and outlet penstock because they would be constructed with the dam. The cost of those structures is included as part of the probable cost estimates for the water quality outlet works with the multi-purpose dam configurations.
- The cost estimate includes a 30 to 35 percent contingency for allied costs for permitting and licensing, engineering, construction management, finance, legal/administration. This contingency total $5 million.
- Additional fish screening requirements could add $1 to $2 million to hydropower system costs.
- Total of all hydropower costs would range from $20 to $25 million.
- Estimated average annual energy production would be about 24,000 MWh (megawatt hours).
- The study results suggest a moderate to high likelihood of a positive cost benefit ratio for hydropower facilities at the site depending on power purchase agreement terms.

1.6 OPINIONS OF PROBABLE COST

An AACE (Association for the Advancement of Cost Engineering) Class 4 estimate of probable costs was completed with an overall expected accuracy of -30 percent to +50 percent. A summary of the component and total combined probable costs for Alternatives A through D is provided in Table 1-2. The primary numbers in this table are the average estimated combined costs for the Chehalis alternatives. The expected variance shown in this table, based on a limited number of uncertainties considered in the probable cost evaluation, is also indicated.

Table 1-2
Summary of Probable Project Costs

ALTERNATIVE	UPDATED PRELIMINARY CLASS 4 COST ESTIMATE 2014 $MILLION, AVERAGE ESTIMATED VALUE AND -/+ RANGE				
	DAM	FISH PASSAGE UPSTREAM	FISH PASSAGE DOWNSTREAM	HYDROPOWER	TOTAL
A (FRO/RCC)	266 217-314	14 (10-19)			280
B (Multi-purpose/RCC, CHTR, CC)	336 276-395	13 10-18	22 17-30	22 20-25	393
C (Multi-purpose/RCC, FW, FC)	336 276-395	37 30-52	33 27-47	22 20-25	428
D (Multi-purpose/RF, EFW, FC)	491 412-570	50 40-70	33 27-47	22 20-25	596

Notes: CC = climate change EFW = experimental fishway FC = floating collector
CHTR = collect, handle, transfer, release
FRO/RCC = Flood Retention Only roller compacted concrete
RF = rockfill

Estimates of probable project costs for climate change Scenario 1 (Alternative A1) and Scenario 2 (Alternative A2) as compared to the Base Case Alternative A are summarized in Table 1-3:

Table 1-3
Alternative A Climate Change Cost Impacts

PRELIMINARY CLASS 4 COST ESTIMATE 2014 $MILLION AVERAGE ESTIMATED VALUE (AND -/+ RANGE OF COSTS)		
BASE CASE A	A1 CLIMATE CHANGE SCENARIO 1	A2 CLIMATE CHANGE SCENARIO 2
280 227-333	303 246-361	400 322-478

1.7 CONSTRUCTION SCHEDULES

Based on experience with other similar projects involving large dam and fish passage facilities, the following overall project schedule should be achievable:

1. Completion of planning and permitting	up to 2 additional years
2. Final design including site characterization	up to 2 years
3. Bidding and award	4 to 6 months
4. Construction	2 to 3 years
	Total: 6.5 to 7.5 years

These findings are consistent with the anticipated project schedule outlined in the Final October 30, 2012, Chehalis River Basin Flood Retention Structure 8-Year Project Planning Document (EES Consulting et al. 2012).

1.8 SUMMARY COMPARISON OF ALTERNATIVES

Table 1-4 presents a comparative summary of the characteristics and costs for each of the alternatives described and evaluated in this TM.

Table 1-4
Summary Comparison of Combined Alternatives

COMBINED ALTERNATIVE	A			B	C	D
SUB-ALTERNATIVE	A BASE	A1	A2			
Purpose	Flood retention only			Multi-purpose	Multi-purpose	Multi-purpose
Dam Type	Roller compacted concrete			Roller compacted concrete	Roller compacted concrete	Central clay core rockfill
Dam Structural Height (feet)	254	263	313	313	313	316
Spillway Crest Elevation (feet)	628	637	687	687	687	687
Emergency Spillway Type	Dam Crest			Dam crest	Dam crest	Side channel
Reservoir Storage Volume (1,000 AF)	65	75	130	130	130	130
Upstream Fish Passage	Flow through channels and CHTR			CHTR	Conventional fishway	Experimental fishway
Downstream Fish Passage	Flow through channels			Head of reservoir collectors	Dam attached floating collector	Dam attached floating collector
Future Hydropower Potential (MWh/year)	None			24,000	24,000	24,000
Construction Period (months)	24	25	30	30	30	36
Estimated Dam and Fish Passage Project Costs (2014 $million)	280	303	400	393	428	596
Estimated Annual O&M Costs ($2014 $1,000)	793			1,539	1,391	1,624
Optional Hydro Total Capital Costs ($2014 $million)	NA			22	22	22
Optional Hydro Yearly O&M Costs ($2014 $1,000)	NA			485	485	485

Notes:

CTHR = collect, handle, transfer, and release MWh/year = megawatt hours per year NA = Not applicable

O&M = operations and maintenance

1.9 RECOMMENDATIONS

The configurations and corresponding estimates of probable construction costs presented in this report are considered reasonably conservative for their intended purposes. However, several of the following important considerations were identified that should be addressed as early as possible during subsequent planning and design phases:

1. Seismic hazards may include large magnitude earthquakes from up to three separate sources: large local crustal faults, interslab, and the Cascadia Subduction Zone. The expected frequency content, duration, and peak ground accelerations at the dam site will have a significant influence on the design of the various facilities and it is recommended that the next phase of work include a design level assessment of seismic hazards and appropriate structural and geotechnical analyses necessary to confirm the dam configuration requirements.
2. The configuration designs for the dams evaluated during this study are based on the dam study teams experience and general understanding of the geology of the site. A preliminary design level site characterization study involving supplemental geologic mapping, geophysical testing, borings, in situ testing, and lab testing of samples from the site should be completed to update the concepts and cost estimates.
3. The development of construction materials such as RCC aggregate, rockfill, and transition/filter/drain materials in close proximity to the site will significantly influence project costs and impacts associated with dam construction. A more detailed evaluation of construction materials along with a preliminary mix design for the RCC dam should be developed during the next planning and design phase.
4. Significant landslide, debris, and sediment hazards have been identified that will significantly influence design and operation of a dam at this site. Preliminary design work should include additional assessment of these hazards and concerns and incorporation of specific design elements into the planning documents and cost estimates to address these hazards.
5. A detailed hydrology study including reservoir routing should be performed for preliminary designs to update the configuration requirements for the emergency spillway and for construction diversion using the flood control outlet works. Construction flood routing requirements should be confirmed based on construction risk evaluation methods such as those used by the US Bureau of Reclamation. Subsequent risk evaluations may indicate that a separate construction diversion tunnel is required that would have to be plugged before operation of the project would begin. A separate construction diversion tunnel is not currently included in the conceptual designs for the dams.
6. There is a need to verify fish passage survival and collection efficiency rates for particular types of downstream passage technologies. The assumed rates have significant impacts on the results of the EDT modeling in the *Effects of Flood Reduction Alternatives and Climate Change on Aquatic Species* Report.
7. More study is needed to develop an estimate of the likelihood of more frequent triggering of water retention operations for a flood retention only dam under climate change scenarios when compared to baseline conditions.

Introduction

2 Introduction

This section provides a brief project background, describes the project purpose and objectives, and outlines the scope of project services described in the TM.

2.1 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. 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 in the basin. The Flood Authority has been studying water retention in the upper Chehalis River Basin along with smaller flood hazard mitigation projects lower in 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 a report in December 2012 titled Chehalis Basin Flood Hazard Mitigation Alternatives Report. That report compiled existing information on the potential flood hazard mitigation projects that seem of most interest to basin leaders and decision makers at this time. Potential flood hazard mitigation benefits, adverse impacts, costs, and implementation issues are summarized for each project to the degree such information was available. Along with that effort, the Work Group, comprising Chehalis Basin leaders, recommended to then Governor Gregoire a series of actions that, taken together, would represent a significant investment to reduce flood damages, 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 (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.

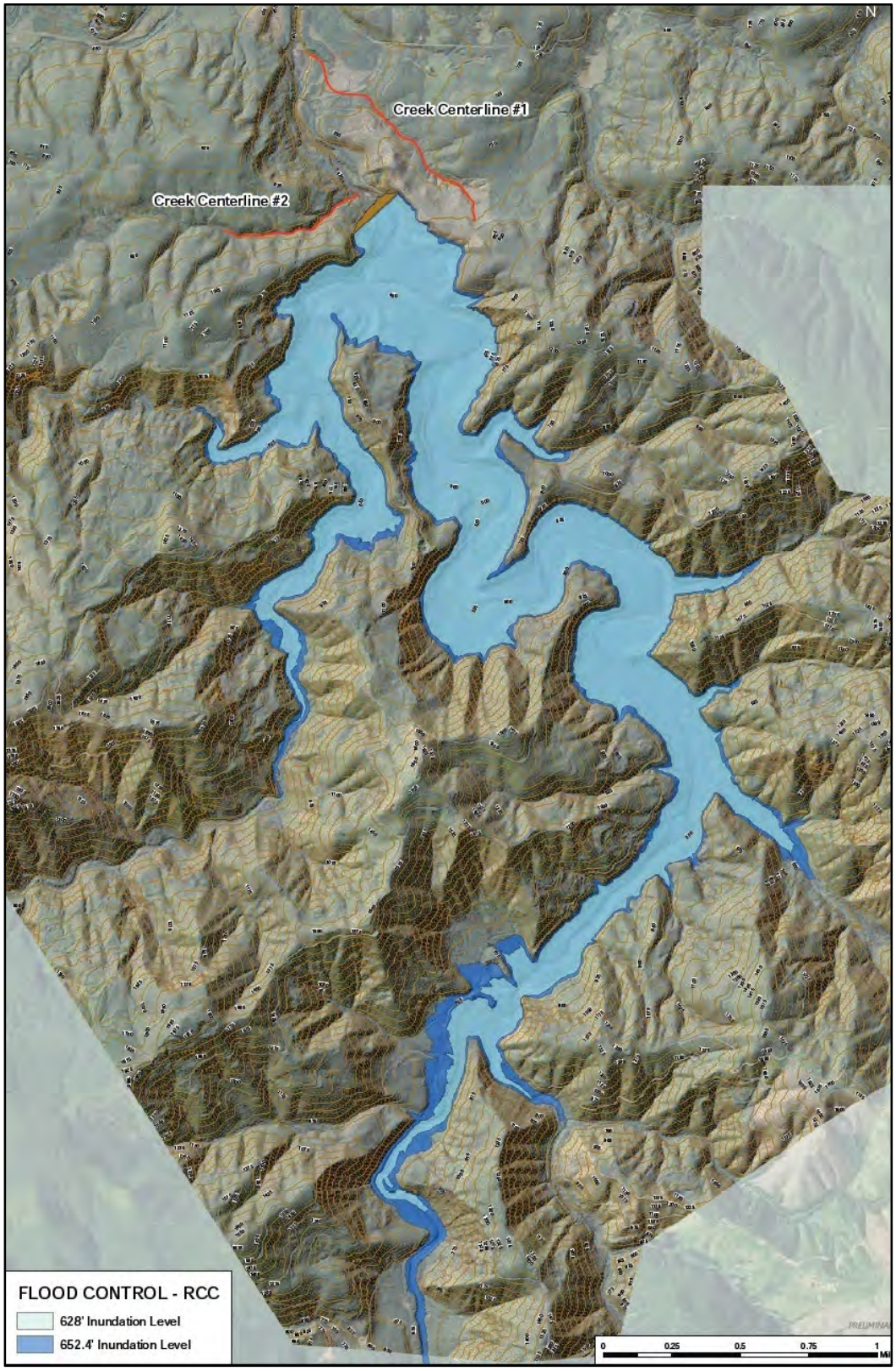
The Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project is evaluating the feasibility of mitigating flood hazards within the basin while exploring opportunities to enhance ecological conditions, aquatic habitat, and abundance of fish in the basin. The scope of the Project includes studying alternative water retention structures (dams), options for protecting Interstate 5 (I-5) with or without a dam, and other small flood reduction projects throughout the Chehalis Basin (basin) with or without a dam. The Project will provide information needed by the Work Group and stakeholders in the region in support of their decision on whether to advance the Project to the next phase of feasibility studies and project permitting.

2.2 PROPOSED PROJECT DAM LOCATION

The proposed dam site was selected in previous studies (S&W 2009a; 2009b) and this decision was not revisited in the current study. The dam would be located south of State Route (SR)-6 in Lewis County, Washington on the mainstem of the Chehalis River about 1 mile south of Pe Ell (the southwest corner of Section 3, Township 12N, Range 5W). Figure 2-1 shows the dam site location and Figure 2-2 shows an example of the approximated reservoir inundation area for a flood retention only dam.

Figure 2-1
Dam Site Location Map

Figure 2-2
Chehalis Flood Retention Dam Inundated Area



2.3 PURPOSE AND OBJECTIVES

The work performed for the Chehalis Basin Strategy includes this Dam and Fish Passage Alternatives Study. The objectives of the Dam and Fish Passage Study are to develop technically feasible alternatives for an integrated dam and fish passage structure at the proposed dam site. The work was completed under three primary work tasks: Dam Design Study, Fish Passage Design Study and the current, Combined Dam and Fish Passage Alternatives Study.

The objectives of the Dam Design Study were to identify any fatal flaws that would limit or preclude construction of a water retention structure on the mainstem of the Chehalis River, and to develop technically feasible options for a flood retention or a multi-purpose dam at the identified site. The Fish Passage Design Study team was tasked to evaluate potential fish passage technologies, establish design criteria and develop options for upstream and downstream passage of adult and juvenile fish that could be integrated with feasible water retention (dam) structures.

A multi-purpose dam will retain a permanent pool and be designed to provide flood retention, water storage, and hydropower, while enhancing certain aquatic species habitat. Reservoir releases will be regulated to attenuate floods and control water quality, transport sediment and small debris, and to control channel scour and erosion downstream of the dam. Reservoir releases for hydropower generation have not been explicitly considered in the outlet configurations presented in this report and will be incorporated as the project planning advances.

A flood retention only dam will only retain water temporarily and will be designed to provide flood retention and fish passage benefits. The intent of the flood retention only structure is to pass unimpeded Chehalis River flows up to a specified level of flooding, and then transition to flood retention operations without fish passage to attenuate larger flood events. Recently revised debris management operational plans and operational responses to climate change scenarios have necessitated consideration of a redundant fish passage facility during flood retention.

The Combined Dam and Fish Passage Alternatives Study builds upon the findings of the Dam Design Study and Fish Passage Design Study 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. Alternative approaches to combined engineered water retention and fish passage structures are evaluated and compared with respect to water retention and fish passage effectiveness, relative costs, and environmental sustainability—to determine the system that provides the desired level of flood protection, minimizes fisheries impacts, and may help to provide long-term downstream habitat improvements.

The results presented in this Combined Dam and Fish Passage Alternatives TM are interdependent with outcomes from the Hydrology and Hydraulic Study, Sediment Transport Study, Environmental Study, Reservoir Operations Study, and other work being performed for the Project. This TM also includes separate memoranda in appendices which discuss reservoir area vegetation and debris management, reservoir area slope stability, and dam construction materials availability. The purpose of this Combined Dam and Fish Passage TM is to present the combined dam and fish passage alternatives, summarize key combined design considerations and present the final concepts for the flood retention only, and multi-purpose dam and fish passage systems for consideration and review by the Water Retention Technical Committee and other technical committees working on the project.

This TM provides planning level information related to probable project costs and benefits for input to economics analysis as well as recommendations for next steps in the development of a combined dam and fish passage water retention facility. If economically feasible, it is expected that the selected combined dam and fish passage concepts will be further refined as information on conditions specific to the project are obtained from site characterization and geotechnical studies; geotechnical, hydrologic, hydraulic, and structural engineering evaluations, and information developed by other project technical committees. It is expected that conceptual layouts of the dam and fish passage alternatives contained within this TM will be modified and adjusted as additional information becomes available.

Other than a construction materials availability study, no additional site geotechnical investigations were performed as part of the scope of services for this TM. Several technical and cost feasibility issues were identified and a limited site geotechnical characterization program was recommended to provide additional site geotechnical information for this TM, but that study was not approved or funded within the time frame for completion of this study. As a result, only existing and available seismic hazard, geologic, geotechnical, and hydrologic and hydraulic information and engineering judgment and experience in the region were used in the development of the combined dam and fish passage alternatives. The concepts for various dam and appurtenant hydraulic structure and fish passage configurations presented in this report are believed to be conservative and decision making should be based, in part on the degree of conservatism that has been adopted due to the limited site characterization information available at the time the concepts were developed. One of the recommendations of this TM is that if a project involving a dam is deemed appropriate for advancement to the next phase of design, that a more extensive site geotechnical characterization study, along with appropriate geotechnical, hydraulic, and structural analyses be completed as soon as is practical.

2.4 PREVIOUS REPORTS

Previous reports and technical memoranda (TM) listed below form the basis for the Combined Dam and Fish Passage Alternatives presented in this TM:

- Interim Fish Passage Design Criteria TM, October 2013
- Draft Dam Design TM, March 2014
- Fish Passage Design TM, May 2014

2.5 SCOPE OF SERVICES

The work performed to develop this Combined Dam and Fish Passage Alternatives TM includes the following tasks:

- Refinement of selected flood retention and multi-purpose dam and fish passage options.
- Combined flood retention and multi-purpose dam and fish passage alternatives development including potential for hydropower development for the multi-purpose configurations.
- Workshop presentation to the Water Retention Committee and stakeholders regarding the proposed combined dam and fish passage alternatives.
- Evaluation and comparison of combined flood retention and multi-purpose dam and fish passage alternatives.
- Research and presentation of operations and maintenance issues and costs for flood retention and multi-purpose flood retention facilities.
- Development of planning level estimates of probable project costs for the selected alternatives.
- Recommendations for next steps in project development.
- Preparation of draft and final Combined Dam and Fish Passage Alternatives TMs summarizing the above information.
- Flood Retention Committee workshop presentation of the final of the findings and recommendations.

Chehalis Dam Structure Options

3 Chehalis Dam Structure Options

This section presents descriptions of the configurations of the selected dam options that will be combined with different fish passage options to form the combined dam and fish passage alternatives. As described in the Dam Design Study TM (HDR 2014a), the Dam Design Study team identified no fatal flaws that would preclude construction of a flood retention only, or a multi-purpose dam at the proposed site. Based on the available information, the identified dam site is best suited for either a roller compacted concrete (RCC), or a central clay core rockfill dam (CCCRF or also referred to as RF). Based on the above conclusions, as part of the dam design study work completed in late 2013 three technically feasible dam options were selected for further development and consideration:

- Flood retention only RCC dam
- Multi-purpose RCC dam
- Multi-purpose central clay core rockfill dam

The RCC dam options would have the smallest impact footprint during construction and have significant appurtenant structure and fish passage advantages. The rockfill dam alternative will have seismic response advantages. Traditional earth fill dam configurations were eliminated from the alternative development process due to the apparent scarcity of earthfill dam construction materials at the site, and the advantages that the other dam types offer to the site conditions and design/operation requirements of the project. This configuration could be reconsidered at a later date if sources of suitable rockfill materials for the rockfill dam configuration are not confirmed at the site.

As described further in Section 3.2, two additional sub-options were recently introduced to the study to account for a range of possible climate change conditions in the basin. Based on preliminary evaluation of the three dam types described above, the flood retention only RCC dam configuration was selected as the basis for the climate change scenario evaluation including cost estimates for the various project configurations. The three original configurations are henceforth referred to as the “baseline” configurations and the two additional scenarios referred to as the “climate change” configurations.

A summary of the dam configurations, reservoir storage volumes and corresponding dam heights included in this TM is provided in Table 3-1. Each configuration is described further in the subsections of this TM chapter. It should be noted that the required dam heights included in this table correspond to the elevation of the emergency spillway crest control structure. Additional flood routing freeboard is required based on the assumed emergency spillway configuration and dam safety freeboard requirements for routing a probable maximum flood through each dam type and configuration described in each corresponding sections of this chapter. The estimated maximum structural height shown in this table includes both design flood routing freeboard and foundation excavation anticipated to achieve a target objective rock quality required for these large dams. It represents the maximum total height of the dam.

Table 3-1
Summary of Reservoir Storage Volume and Dam Height Requirements

ALTERNATIVE	RESERVOIR STORAGE VOLUME (ACRE-FEET)			REQUIRED DAM HEIGHT (FEET)[1]	EST. MAX. STRUCTURAL HEIGHT (FEET)[2]
	FLOOD	OPERATIONS	TOTAL		
A - Baseline – Flood Retention RCC	65,000	0	65,000	228	254
A1 – Climate Change Scenario 1	75,000	0	75,000	237	263
A2 – Climate Change Scenario 2	130,000	0	130,000	287	313
B and C – Multi-purpose RCC	65,000	65,000	130,000	287	313
D – Multi-purpose Rockfill	65,000	65,000	130,000	287	316

Notes: 1. The indicated dam height is approximate. It is an estimate of the maximum hydraulic height and corresponds to the distance from the bottom of the stream channel (approximate elevation 400) at the maximum section to the crest of the emergency spillway. Additional height is required to provide probable maximum flood routing.
2. The maximum structural height includes the estimated flood routing freeboard and foundation excavation required for the dam. This represents the maximum total height of the dam structure from the base at the foundation excavation contact to the top of the parapet wall (RCC) or rockfill dam crest along the upstream axis (RCC), or edge of the dam that is the highest point on the dam.
RCC = roller compacted concrete

Prior to developing the planning level detail of each alternative dam type summarized in Table 3-1, the basic design criteria and requirements for the dam and appurtenant structure (excluding fish passage components described separately) were identified. These overall design criteria/requirements include the following:

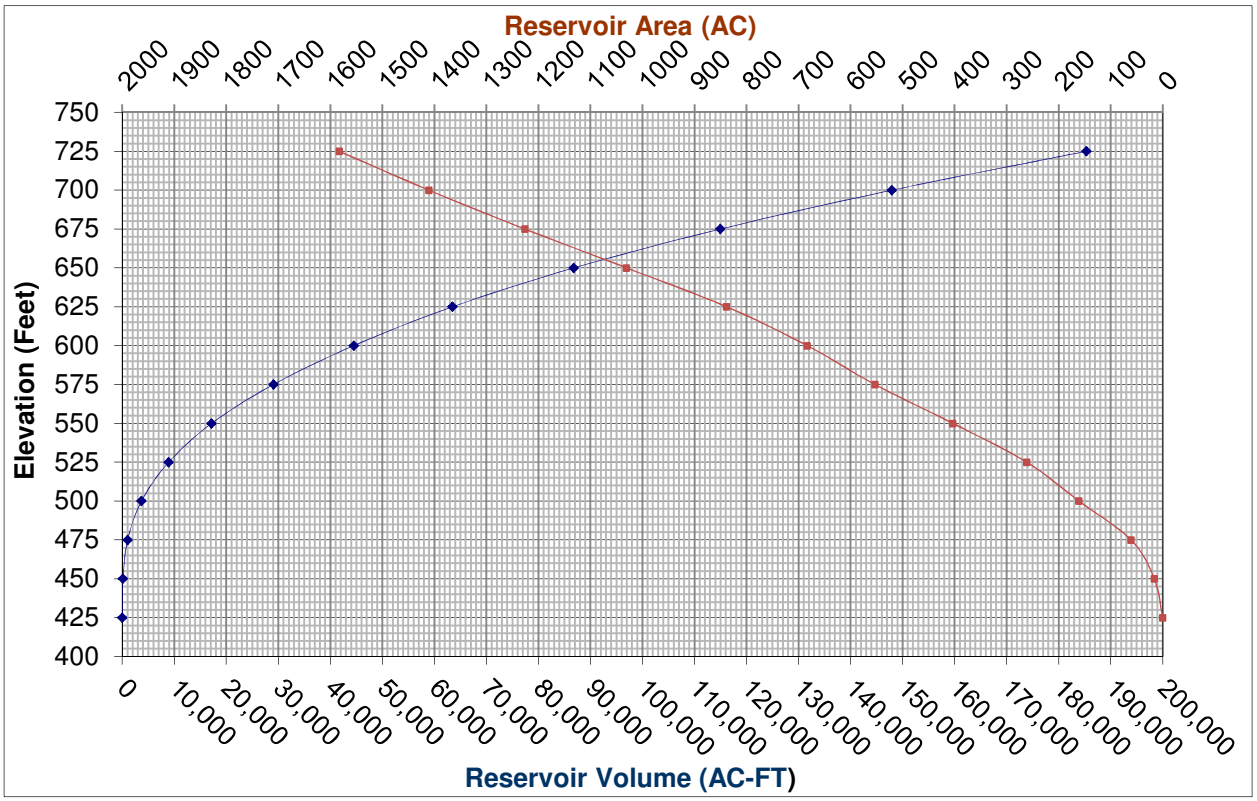
Reservoir Storage Capacity: See Figure 3-1 (reproduced from Figure 5 in the March, 2014 Dam Design TM)

Emergency Spillway: Safely pass a Probable Maximum Flood (PMF)

Flood Routing During Construction: 12,000 to 15,000 cubic feet per second (cfs)

Seismic Hazard: Operational Basis Earthquake, approximately 1 in 500-year event, Design Basis Earthquake, Maximum Credible Earthquake or 1 in 10,000-year event, whichever is greater.

Figure 3-1
Chehalis Main Stem Reservoir Volume and Area Curve



Note: ac = acre AC-FT = acre-feet

Detailed extreme flood hydrology, hydraulic, and seismic hazard analyses have not been performed and will be required during subsequent study phases. The design criteria have been approximated and evaluated at only a planning level for development of the concepts presented in this report. Some configuration modifications may be required depending on the outcome of more rigorous assessment of these design criteria. Additional information and discussion of these criteria are provided in the Dam Design TM.

3.1 BASELINE FLOOD RETENTION ONLY ROLLER COMPACTED CONCRETE DAM

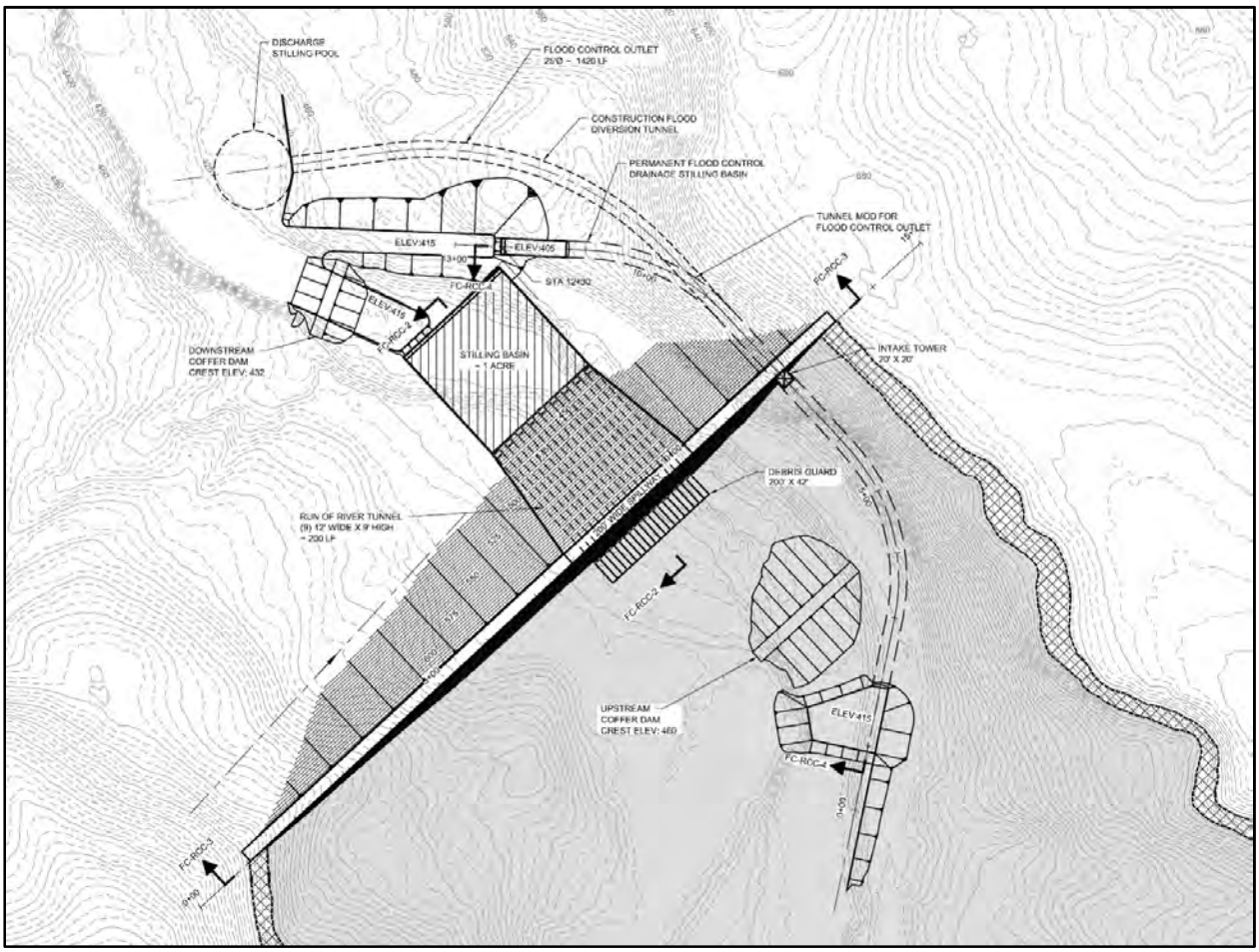
3.1.1 DAM AND APPURTENANT STRUCTURE LAYOUT

As summarized in Table 3-1, in order to provide the target flood retention storage, the flood retention only RCC dam (FRO-RCC Dam) would be approximately 228 feet tall and have a base width of about 220 feet at the maximum section of the dam. The dam would provide the target temporary flood storage capacity of approximately 65,000 acre-feet with an emergency spillway crest Elevation of 628 feet.

The structure would include up to nine run of the river fish passage conduits integral to the dam structure, a flood operations outlet works and an emergency spillway. The spillway and the fish passage conduits would be located near the maximum section of the dam at the current river channel location. The flood operation outlet works conduit would be located in the lower right abutment area. Figure 3-2 (extracted from drawing FRO-RCC-1 in Appendix A) shows the flood retention only RCC Dam site plan including locations of temporary construction cofferdams. Additional conceptual level drawings of this dam and appurtenant structures configuration are included in Appendix A of this TM.

Based on simplified probable maximum flood (PMF) routing assumptions and hydraulic analyses, the dam crest was set at elevation 651, one foot above the maximum estimated reservoir flood pool elevation. A 3-foot parapet wall at the upstream edge of the crest would provide flood routing freeboard. The crest width was set at 20 feet. The top of the dam includes a 20 foot high "chimney section". Below the chimney section the downstream slope of the dam would be inclined at 0.85H:1V (Horizontal to Vertical). The upstream face of the dam has a uniform slope of 0.1H:1V. The upstream and downstream faces of the dam would be finished with an integral conventional concrete. A typical cross-section of the dam is shown on drawing FRO-RCC-2, and a profile along the upstream axis of the dam is shown on drawing FRO-RCC-3 in Appendix A-1. A profile through the flood control outlet works tunnel is shown on drawing FRO-RCC-4. The locations of the cross-sections and profiles are illustrated on the plan view of the project illustrated on Figure 3-2.

Figure 3-2
Flood Retention RCC Dam Site Plan



A number of seepage control provisions are not shown in detail on the drawings, but will eventually be included in the design. These provisions would include treatment of the foundation excavation surface, installation of a foundation grout curtain, control joints through the dam, crack inducers (with water stops), a drainage gallery, foundation and dam drainage holes, and a suite of dam and foundation instrumentation to monitor the performance and safety of the dam. Adequate planning contingencies have been included in the cost estimate to cover these items until such time that they are explicitly in the design and cost estimate.

Typical dam operation would consist of passing the river through low level fish passages during normal river flows up to approximately 2,000 to 5,000 cfs. At some flow still to be determined, operations would begin to change from a fish passage criteria to a combined fish passage and flood operation and the flood control outlet would begin to operate. During this period, the flood control outlet works would be operated to augment flows through the fish passage conduits and control velocities for fish passages as long as possible. Once the flow reaches the point where fish passage flow conditions are no longer possible through the low level conduits, they would be closed and all of the flow would be directed through the flood control outlet. The flood control outlet would continue to operate throughout a flood event and until such time as fish passage operations can be restored. Once the capacity of the flood control outlet was reached, the reservoir would begin impounding water up to the point where it reaches the emergency (auxiliary) spillway crest. The discharge capacity of the spillway would meet dam safety criteria for a PMF event.

3.1.2 FISH PASSAGE CONDUITS

The flood retention RCC dam would include up to nine 12-foot-by-9-foot conduits at the base of the dam to facilitate upstream and downstream fish passage and allow debris and sediment transport during non-flood operation conditions. These run of the river conduits will be designed to mimic stream flows in the natural channel and will be equipped with debris guards and racks on the upstream entrances to prevent large debris from entering/clogging the conduits. The fish passage system will have gated control designed to close under low head conditions when operation of the reservoir is changed to a flood retention objective. The upstream gates would be used for normal operations and could be either top sealing radial or vertical slide gates. The downstream gates would be closed to prevent water from the spillway stilling basin backing up into the conduits during high flows that would occur during emergency spillway flow conditions.

A preliminary HEC-RAS (Hydrologic Engineering Center River Analysis System) model constructed by Watershed Science & Engineering was used as the basis for the conceptual level planning of the fish passage and flood control outlet works for the dam. The intent of the hydraulic modeling was to estimate the number of fish passage conduits required to meet a target 2 feet per second (ft/sec) requirement for fish. Multiple runs were completed using various fish passage and flood outlet sizes and intake invert elevations. With staggered fish passage conduit invert elevations and with roughened conduit surfaces the model indicated that flow velocities through nine conduits could be reduced to the target value of 2 ft/sec. Additional hydraulic analysis, numerical and physical modeling will need to be conducted during final design to refine the flow-through fish passage hydraulics.

All nine fish passage conduits have the same dimensions. One of the conduits will have smooth concrete surface and will be utilized for sediment discharge when needed. The other eight will be designed to simulate natural channel flow conditions. The five central fish passage conduits, including the smooth conduit, will have the same invert elevation. Two more will be set with an intake invert 2 feet higher and the final two conduits will have an intake invert 4 feet higher than the central conduits. The staggered configuration extends the window of operation for fish passage conduits during a flood event.

3.1.3 FLOOD CONTROL AND CONSTRUCTION DIVERSION OUTLET

The current configuration of the flood control outlet works includes a concrete lined, horseshoe shaped pressurized tunnel upstream of a gate chamber that includes two, 12-foot-by-25-foot slide gates, and a concrete lined, horseshoe-shaped discharge tunnel downstream of the control gates designed for open channel flow. The gates for control of flow through the outlet works would be located in an excavated chamber at the base of a vertical shaft attached to the upstream face of the dam. This gate system would be designed to operate under unbalanced head conditions. The gate chamber and operating system would be accessible during all flooding conditions through the vertical shaft from the crest of the dam. The intake for the outlet system would be designed to attract and discharge sediment through the outlet but to prevent clogging by large timber debris.

Smaller timber debris would be passed through the outlet and discharged to the downstream channel. A discharge structure would be constructed on the downstream end of the outlet conduit for energy dissipation and for directing flows back into the stream channel downstream of the spillway stilling basin described in the following paragraphs.

The configuration of the outlet work is controlled to a significant degree by the need for stream and flood diversion during construction of the dam. A combination of the outlet works, and the upstream and downstream cofferdams would provide a target level of protection of the dam construction work. Simplified hydraulic evaluations suggest that the current configuration would provide for safe routing of river flows of up to 12,000 to 15,000 cfs without overtopping of the upstream or downstream cofferdams and inundation of the dam and foundation construction activities. Further hydraulic and risk evaluations should be performed during subsequent study phases to refine the concept and configuration for this outlet works.

3.1.4 AUXILIARY (EMERGENCY) SPILLWAY

The current configuration of the emergency spillway includes a 250-foot-wide ogee crest structure, a tapered and stepped discharge chute over the downstream face of the dam, discharging into a 200-foot wide by 200-foot long hydraulic jump stilling basin. For this configuration, the stilling basin would provide for containment and control of all flows over the emergency spillway up to, and including, the PMF. There are no operating gates, or operational requirements for the spillway. The configuration has been established to safely route up to 100,000 cfs over the dam during a PMF event without consideration of reservoir routing effects and is therefore conservative from the standpoint of the impact footprint of the structure and cost. Provided that there are no significant increases in the PMF inflow hydrograph for the site during subsequent hydrologic evaluations, future flood routing and hydraulic analyses for the spillway will likely result in reduced size and cost requirements. The stepped spillway chute and a stilling basin were configured based on preliminary calculations following the U.S. Army Corps of Engineers and U.S. Bureau of Reclamation standards and research papers for stepped spillways (Frizell [No Date], Kantoush et al. 2011, Gonzalez et al. 2005, and Sarfaraz et al. 2011)

3.1.5 CONSTRUCTION CONSIDERATIONS

Cofferdams will be required upstream and downstream of the dam footprint to facilitate excavation, foundation preparation, and construction of the dam and appurtenant structures for this configuration. The location and cross-section of the cofferdams included in the planning level configurations presented in this TM are based on limited and simplified hydraulic analyses, and HDR's experience with similar structures and stream diversion requirements. As noted above, the actual crest elevation and cross-section of the cofferdams will be established as part of future study phases based on hydraulic routing, geotechnical and construction risk evaluations. A temporary section of outlet tunnel, extending downstream of the lower cofferdam, will be blocked with a bulkhead after the permanent alignment, including the discharge stilling basin, are constructed and when river flows could be diverted through the fish passage conduits.

Construction materials and staging will be a significant aspect of the planning, design, and construction of the project. Significant access, power, and construction staging areas are required for the dam and appurtenant structures described in the preceding sections and significant additional work is required to incorporate these items into the configuration should this dam be selected for further evaluation. Additional discussion of construction materials is provided in a subsequent section of this report.

A number of landslide areas have been identified in the immediate vicinity of the dam site. The approximate locations of these features are illustrated on drawing FRO-RCC-1 in Appendix A-1. Complete or partial removal and remediation of these landslides will be required for the dam and appurtenant structures construction.

3.2 CLIMATE CHANGE FLOOD RETENTION ROLLER COMPACTED CONCRETE DAMS

3.2.1 CLIMATE CHANGE SCENARIOS

Two climate change scenarios were investigated and modeled by the Chehalis Team to identify the size of a flood retention only RCC dam that would be required to account for potential climate change impacts. Reservoir routing models were revised to estimate the amount of storage needed to retain a 100-year flood event, assuming higher flows in the basin and no changes to the existing operating rules. Climate change Scenario 1 predicted an 18 percent increase in the routed flood storage requirement for the project. This corresponds to a 75,000 acre-foot flood retention storage volume when sediment and debris are considered. This storage volume requirement is approximately 10,000 acre-feet more than the baseline study case estimate of 65,000 acre-feet for a flood-retention only dam described in Section 3.1. The second scenario (Scenario 2) predicted a 90 percent increase in the required flood storage volume in the reservoir. This translates to approximately 130,000 acre-feet of total flood, debris and sediment retention storage volume. The total reservoir storage volume required for a flood retention only dam for the second climate change scenario would be the same as the baseline study case for a multi-purpose dam, which has been sized to provide a total of 130,000 acre-feet of reservoir storage.

3.2.2 FLOOD RETENTION RCC DAM SIZE

The required storage volumes for the two climate change scenarios summarized in Section 3.2.1, were then used to identify the required dam height using the reservoir storage volume curve reproduced on Figure 3-1, developed by HDR from the most recent available topographic information for the reservoir basin area.

As seen in Table 3-1, depending on the assumed climate change scenario, the dam would need to be from 9 to nearly 60 feet higher than the baseline study case flood control only configuration structure.

3.3 MULTI-PURPOSE ROLLER COMPACTED CONCRETE DAM

3.3.1 DAM AND APPURTENANT STRUCTURE LAYOUT

As summarized in Table 3-1, in order to provide the target flood detention and operating pool storage capacity of the baseline 130,000 acre-feet, the Multi-purpose RCC Dam (MP-RCC Dam) would be approximately 287 feet tall and have a base width of about 270 feet at the maximum section. The dam would consist of an RCC dam section in the center of the valley transitioning to a wing embankment dam in the upper right abutment area. The RCC section would be approximately 1,900 feet long. The embankment section would be approximately 550 feet long. Appurtenant structures would include a flood control outlet works, a water quality/hydropower outlet works and an emergency spillway with its Ogee crest structure set at elevation 687. Figure 3-3 shows a site plan for the multi-purpose RCC dam extracted from drawing MP-RCC-1 in Appendix A-2. Additional conceptual level drawings of this alternative dam and appurtenant structures configuration are included in Appendix A-2 of this TM.

The auxiliary (emergency) spillway would be located near the maximum section of the dam at the current river channel location. The flood control outlet works (labeled as FR Outlet on Figure 3-3) would be located in the lower left abutment area along the west (left looking downstream) side of the auxiliary spillway. The water quality/hydropower outlet would be located in the lower right abutment area along the east (right) side of the emergency spillway and include provisions for discharging to the emergency spillway stilling basin. Adequate area has been incorporated into the layout of this dam for inclusion, or future construction of a hydropower plant at the downstream toe of the dam and immediately above the right retaining wall of the emergency spillway stilling basin. Additional elements of the project not included at this time in the conceptual design include a stream crossing bridge and roadway downstream of the dam for access to the flood control outlet works, additional road provisions in the upper right abutment of the dam for access to the dam crest and water

quality/hydropower outlet operating systems, stabilization of the landslides within the influence area of the dam facilities, power supply, communication, and an operation and maintenance facilities.

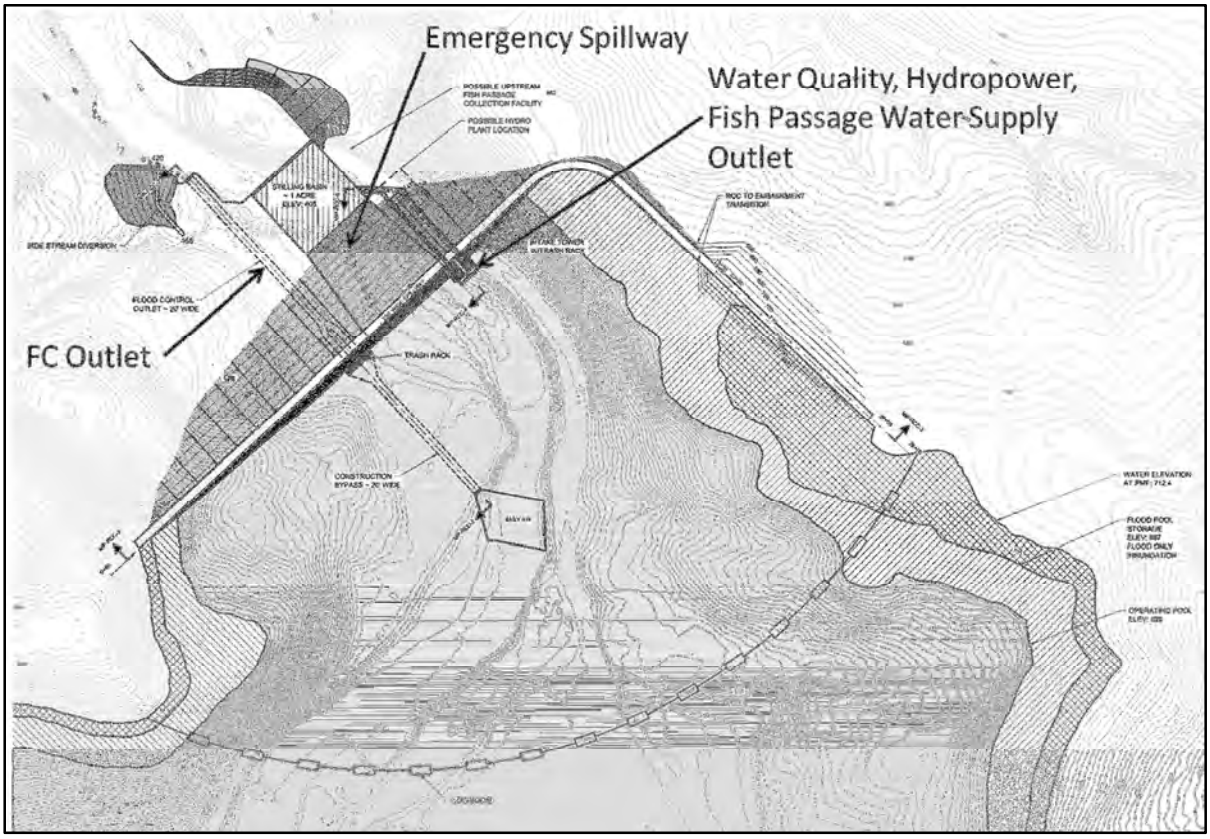
Based on simplified PMF routing assumptions and hydraulic analyses, the dam crest has been set at about elevation 710 feet and a 3-foot parapet wall is included along the upstream edge of the crest to provide flood routing freeboard. The crest width was set at 20 feet and the top of the dam includes a 20-foot-high "chimney section." The upstream slope of the dam would be inclined at 0.1(H):1(V). Below the chimney section, the downstream slope of the dam would be inclined at 0.85(H):1(V). Similar to the "baseline" FC only RCC dam, the upstream and downstream faces of the dam would be finished with integral conventional concrete. A typical cross section of the dam is shown on drawing MP-RCC-2, and a profile along the upstream axis of the dam is shown on drawing MP-RCC-3 in Appendix A-3.

A number of seepage control provisions are not shown in detail on the drawings, but will eventually be included in the design. These provisions would include treatment of the foundation excavation surface, installation of a foundation grout curtain, control joints through the dam, crack inducers (with water stops), a drainage gallery, foundation and dam drainage holes, and a suite of dam and foundation instrumentation to monitor the performance and safety of the dam. Adequate planning contingencies have been included in the cost estimate to cover these items until such time that they are explicitly in the design and cost estimate.

The maximum reservoir operating pool at elevation 628 feet would provide approximately 65,000 acre-feet of storage for water quality and other purposes including hydropower. The limits of this pool are indicated on Figure 3-3. The upper 65,000 acre-feet of storage is reserved for temporary flood retention. Operation of the permanent pool within the multi-purpose reservoir would be in accordance with operating rules designed to meet minimum stream flow, water quality, fish passage, and possibly power generation objectives. Typical dam operation under normal conditions would involve upstream and downstream fish passage and water quality and hydropower releases. Note that the fish passage systems are described in Chapter 4 of this TM. The right abutment outlet tower configuration includes provisions for selective withdrawal through slide gates installed at different elevations below elevation 628 feet. The concept design currently includes three gate locations as shown on drawing MP-RCC- 3.

The upper portion of the reservoir would be operated to meet flood control objectives. The operation of the upper portion of the reservoir is being evaluated separately from this TM. The operating storage permanent pool will be kept at or below elevation 628 feet in order to maintain enough flood retention storage capacity. As noted above, under normal conditions, reservoir releases will be through water quality/hydropower and fish passage outlet works. Flood control conduit will operate when additional reservoir drawdown capacity is required. Once the capacity of the flood control outlet is reached during a flood event the reservoir would fill up to the emergency spillway crest elevation. For floods exceeding an estimated 1 in 100 year recurrence interval, excess flows up to and including the PMF would discharge over the emergency spillway structure.

Figure 3-3
Multi-purpose RCC Dam Site Plan



3.3.2 FLOOD CONTROL AND CONSTRUCTION DIVERSION OUTLET

The current configuration of the flood control outlet works includes an open excavation approach channel from the upstream river channel to the intake structure on the upstream face of the dam, an emergency guard gate control chamber, a 20-foot diameter concrete encased steel penstock through the base of the RCC dam and downstream toe area, a transition structure consisting of a manifold of two or more transitions from the 20-foot diameter to 8-foot diameter steel discharge penstocks, and discharge through 8-foot diameter hollow jet or plunge valves to a plunge pool area in the stream channel downstream of the emergency spillway stilling basin area. Isolation valves are included in the configuration immediately upstream of the hollow jet or plunge valves so that each 8-foot diameter penstock can be isolated during construction for discharge valve installation, or for routine valve maintenance during the operating life of the dam. Guard gates in the upstream valve chamber are provided for emergency or construction closure of the outlet works.

The intake to the open excavation approach channel and the approach channel itself would be designed to facilitate flow of debris and sediment through the flood control outlet structure. It is likely that most sediment would be retained in the upstream reservoir areas and sediment flows through the outlet would be limited to fine-grained materials that find their way through the base of the reservoir during normal operations. It may be possible to operate the permanent pool so that some additional sediment movement through the system would be accomplished on a periodic basis. Such operating rules would be examined further during future planning phases if this configuration is advanced for further development.

The upstream portion of the outlet through the dam consists of two 15-foot by 15-foot conduits from the upstream face of the dam through the base of the guard gate chamber. Downstream of the guard gate chamber, these conduits combine through a transition zone into the 20-foot diameter concrete encased steel penstock. A combined steel and reinforced concrete trash rack structure protects the entrance and prevents the entry of large woody debris that would not pass through the downstream control valves. Occasional maintenance of the intake to remove accumulated debris will be required. With hydraulic controls located at the downstream end, the entire length of the conduit will remain pressurized during flood control operations. The guard gates will be designed for operation under unbalance head and accessible from a guard gate chamber within the base of the dam should emergency closure of the system be required.

The basic configuration of the outlet work described above is controlled to a significant degree by the need for stream and flood diversion during construction of the dam. A combination of the outlet works, and the upstream and downstream cofferdams would provide a target level of protection of the dam construction work. Simplified hydraulic evaluations suggest that the current configuration would provide for safe routing of river flows of up to 12,000 to 15,000 cfs without overtopping of the upstream or downstream cofferdams and inundation of the dam and foundation construction activities. Further hydraulic and risk evaluations should be performed during subsequent study phases to refine the concept and configuration for this outlet works.

3.3.3 AUXILIARY (EMERGENCY) SPILLWAY

The current configuration of the emergency spillway includes a 250-foot-wide ogee crest structure set at elevation 687, a tapered and stepped discharge chute over the downstream face of the dam, discharging into a 200-foot wide by 200-foot long hydraulic jump stilling basin. For this configuration, the stilling basin would provide for containment and control of all flows over the emergency spillway up to slightly less than the PMF. For very large flows over the emergency spillway, some overtopping of the downstream portion of the stilling basin retaining walls may occur. In addition, there may be times when the hydraulic jump and energy dissipation in the structure is not fully contained. This allowance was included in the configuration design due to significant site constraints in the downstream toe area of the dam. Further hydraulic evaluations and design refinement will be required during subsequent design phases to confirm the acceptability of this design assumption should this configuration be selected for further evaluation.

There are no operating gates, or operational requirements for the spillway. Similar to the FC only dam configuration, this configuration has been established to safely route up to 100,000 cfs over the dam during a PMF event without consideration of reservoir routing effects and is therefore conservative from the standpoint of the impact footprint of the structure and cost. Provided that there are no significant increases in the PMF inflow hydrograph for the site during subsequent hydrologic evaluations, future flood routing and hydraulic analyses for the spillway will likely result in reduced size and cost requirements.

The stepped spillway chute and a stilling basin were configured based on preliminary calculations following USACE and U.S. Bureau of Reclamation standards and research papers for stepped spillways (Frizell [No Date], Kantoush et al. 2011, Gonzalez et al. 2005, and Sarfaraz et al. 2011).

3.3.4 WATER QUALITY CONTROL AND HYDROPOWER OUTLET

Outlet works for water quality control and potential hydropower generation would be located at the base of the right abutment, to the right (looking downstream) of the emergency spillway stilling basin, and near the maximum section of the dam providing access to the entire reservoir pool during normal operation periods. The outlet works consists of an intake tower attached to the upstream face of the dam, a 6-foot-diameter hydropower penstock and a 3-foot-diameter water quality penstock for minimum stream flow and water quality regulation. As previously noted, ample room has been included in the configuration for installation of a hydropower plant at the toe of the dam and for discharge of both the water quality and hydropower penstocks

under/through the right emergency spillway training wall and discharge to the emergency spillway stilling basin at the toe of the dam. Butterfly values operated from a small gate chamber in the dam would provide for emergency shutoff of either or both penstocks during construction or normal operations.

The intake tower to the right abutment outlet works would be attached to the upstream face of the dam as shown on drawing MP-RCC-4. The tower would serve as a wet well allowing for releases through either, or a combination of the water quality release penstock, or the hydropower release penstock. Slide gates on the exterior of the intake tower would be opened based on reservoir temperature readings at multiple locations through the reservoir profile near the outlet intake tower. The system would be designed so that blending of flows from multiple partial gate openings could occur. Each intake to the tower would be equipped with trash racks and fish screens. Provisions for hoisting and cleaning the trash racks and fish screens will be included at the top of the tower with access from the dam crest.

3.3.5 CONSTRUCTION CONSIDERATIONS

The multi-purpose dam and appurtenant structure configuration described in the preceding sections would be constructed in stages. Drawings MP-RCC-5 through 8 provided in Appendix A-2 show a likely three phase construction sequence described as follows:

Phase I: Construction of the intake channel and construction of the flood control outlet works excluding installation of the downstream control valves. This work would be protected by local cofferdams and diversion channels as required for protection of the work. The flood control outlet alignment crosses three landslides mapped on the left abutment. Partial removal and stabilization of these and three other landslides located on the opposite side of the river would be included in Phase I construction.

Phase II: Phase I cofferdams and protection structures would be removed and the stream diverted through the flood control outlet works. The contractor would then install the primary cofferdams (Phase IIA). Once the cofferdams are in place, the contractor would construct the water quality/hydropower intake structure, penstocks and outlet release structures (Phase IIB) integral with construction of the dam, spillway stilling basin and associated downstream channel modifications (Phase IIC).

Phase III: The flood control outlet would be shut off and stream diversion would be through the water quality/hydropower outlet system. The Contractor would install any minor downstream coffer structures and then proceed to finalize the construction of the flood control outlet works through the installation of the downstream control values and operating systems. Once installed, all gate operating systems would be tested, final instrumentation systems installed and verified, and the reservoir would be taken through the initial filling sequence with final testing of all operating systems under normal head and operation protocols.

The location and cross-section of the cofferdams included in the planning level configurations presented in this TM are based on limited and simplified hydraulic analyses, and HDR's experience with similar structures and stream diversion requirements. As noted above, the actual crest elevation and cross-sections of the cofferdams and construction sequence will be established as part of future study phases based on hydraulic routing, geotechnical, and construction risk evaluations.

Landslide mitigation would be constructed during the appropriate phase of construction outlined above. Similarly, the construction phasing would include the required access road, power, communication, and operation, and maintenance facilities incorporated during subsequent project planning phases.

Construction materials and staging will be a significant aspect of the planning, design, and construction of the project. Significant access, power and construction staging areas are required for the dam and appurtenant structures described in the preceding sections and significant additional work is required to incorporate these items into the configuration should this dam be selected for further evaluation. Additional discussion of construction materials is provided in a subsequent section of this report.

3.4 MULTI-PURPOSE CENTRAL CLAY CORE ROCKFILL DAM

3.4.1 DAM AND APPURTENANT STRUCTURE LAYOUT

As summarized in Table 3-1, in order to provide the target flood retention and operating pool storage capacity of the baseline 130,000 acre-feet reservoir, the Multi-purpose central clay core rockfill dam (MP-RF Dam) would be approximately 287 feet high and have a base width of about 1,200 feet at the maximum section. The dam would include a central clay core for seepage control. The seepage and mass stability of the central clay core would be provided by upstream and downstream filter transition zones and rockfill shells. Appurtenant structures would include a flood control outlet and water quality/hydropower outlet through tunnels excavated through the lower left abutment and an uncontrolled RCC lined auxiliary (emergency) spillway with its Ogee crest structure set at elevation 687 located in the upper the right abutment. The emergency spillway would discharge into a tributary channel, which merges with the mainstem of the Chehalis River approximately 3,600 feet downstream of the dam. The flood control and water quality/hydropower outlet conduits would discharge into a plunge pool excavated into sound bedrock in the downstream river channel. Finally, some modifications to a tributary drainage channel that enters the lower left area of the proposed configuration (looking downstream) will be required to provide for routing of natural stream flows away from the discharge end of the flood control outlet works. Figure 3-4 shows a site plan of the multi-purpose rockfill dam. Additional conceptual level drawings of this dam and appurtenant configuration are included in Appendix A-3 of this TM.

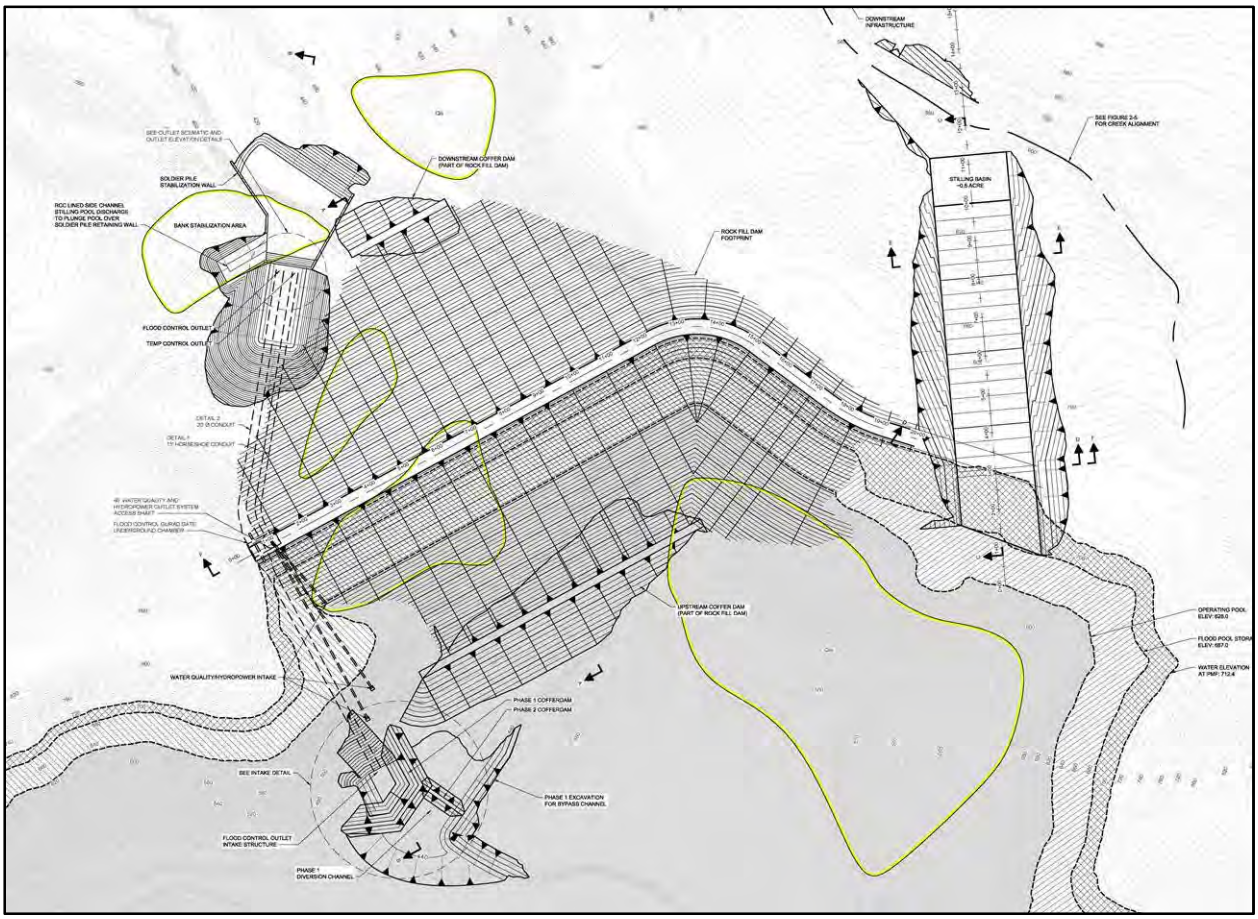
Based on simplified PMF routing assumptions and hydraulic analyses, the dam crest has been set at about elevation 717. The dam would have a 40-foot-wide crest and 2H:1V upstream and downstream slopes. The dam crest elevation provides for routing of the PMF and an additional 5 feet of freeboard for wave run-up. The upstream slope protection would consist of a 5-foot-thick layer of riprap underlain by a 5-foot-thick transition and bedding layer. The minimum width of the core at the top is 20 feet. The upstream and downstream side slopes of the core are inclined at 0.5H:1V. Zone 1 core material would be protected with a 10-foot wide filter/transition zone on the upstream side and a 10 foot-wide filter/chimney drain on the downstream side. A horizontal 10-foot thick blanket drain system at the base of the downstream shell would collect and safely discharge seepage through and under the dam. The upstream and downstream shells would be constructed with maximum 18-inch well graded high quality rockfill materials placed and compacted in lifts of 24 to 32-inch thickness. It is anticipated that rockfill will be produced from an on-site borrow area. Construction cofferdams would serve as starter dikes for the rockfill shells.

As noted during early phases of the dam study, the Chehalis site is likely to have a significant seismic hazard including ground motions from local crustal, interplate, and subduction zone sources. Our initial evaluation of the seismic hazard at the site using information available from the U.S. Geological Survey indicated estimated peak ground accelerations (pgas) of 0.56 g for a 1 in 2,500-year event, and 0.72 g for a 1 in 5,000-year event. Based on HDR's experience with design of rockfill dams subjected to these levels of ground motion hazard, we have assumed that all overburden soils at the site will need to be removed from the foundation of the dam including the area under both the upstream and foundation shells. These overburden soils would be removed

down to the top of weathered bedrock under the shells of the dam. Additional excavation to a suitable quality of rock would be completed under the central clay core. Foundation treatment at the base of the central clay core would include installation of a grout curtain, and placement of dental concrete in order to achieve a fully integrated objective for seepage safety and control of both embankment and foundation seepage during normal and flood operation of the structure. Adequate planning contingencies have been included in the cost estimate to cover these items until such time that they are explicitly in the design and cost estimate.

The maximum reservoir operating pool at elevation 628 feet would provide approximately 65,000 acre-feet of permanent storage for water quality and other purposes including hydropower. The limits of this operating pool are indicated on Figure 3-4. An additional 65,000 acre-feet of storage would be reserved for flood retention, for a total of 130,000 acre-feet of reservoir capacity. The reservoir would reach elevation 687 feet when auxiliary spillway begins to operate. Operation of the multi-purpose rockfill dam would be similar to the multi-purpose RCC dam discussed previously.

Figure 3-4
Multi-purpose Rockfill Dam Site Plan



The flood control outlet works along with the water quality/hydropower outlet works are separate systems located adjacent to one another in the lower left abutment area. Planning level designs are based on separate operating objectives and rules. Similar to the MP-RCC dam alternative, the flood control outlet works provides for routing of stream flows during construction and protection of construction activities, as well as control of the upper flood storage pool. The water quality and hydropower outlet works provide for control of the multi-

purpose pool to achieve a range of minimum stream flow, water quality, and possible hydropower releases during normal operation. As subsequently discussed, some flows for fish passage operations downstream of the dam would be also handled through this outlet works system. A separate technical committee is evaluating various reservoir operation scenarios. The operating storage within the permanent pool will be kept at or below Elevation 628 feet in order to maintain enough flood retention storage capacity. The water quality outlet system includes provisions for selective withdrawal through three separate intake tunnels at different elevations. These tunnels extend from the control shaft in the left abutment near the centerline axis of the dam to the reservoir. As discussed further below, these three tunnels are 48 inches in diameter and are controlled by separate slide gates to allow for selective withdrawal and blending of flows to meet water quality objectives.

The flood control outlet would begin to operate when additional reservoir control is required to meet downstream flood flow objectives. Once the maximum discharge capacity to meet these downstream flood control objectives are met, the flood control outlet would be operated to retain flood flows in the reservoir pool which would fill up to the emergency spillway crest elevation depending on the magnitude and duration of the flood event. For floods exceeding the estimated 1 in 100-year recurrence interval objective, excess flows up to and including the PMF would discharge over the emergency spillway structure after the reservoir pool has filled.

Additional elements of the project that are not included at this time in the conceptual design include a stream crossing bridge and roadway downstream of the dam for access to the flood control and water quality outlet works, additional road provisions in the upper right abutment of the dam for access to the dam crest, stabilization of the landslides within the influence area of the dam facilities, power supply, communication, and an operation and maintenance facilities.

A more complete drawing of the plan view of the dam and appurtenant structures is provided on MP-RF-1. A typical cross section through the maximum section of the dam is shown on MP-RF-2, and a profile along the axis of the dam illustrating the concepts for the dam and emergency spillway structures is shown on MP-RF-3. A profile and various sections through the emergency spillway are shown on drawing MP-RF-4. Profiles, sections, and details of the flood control and water quality outlet works are shown on drawings MP-RF-5 and MP-RF-6.

3.4.2 FLOOD CONTROL OUTLET WORKS

The current configuration of the flood control outlet works includes 1) an inlet area/structure for debris control, 2) a 20-foot diameter circular reinforced concrete lined intake tunnel, 3) a guard gate chamber with inlet and outlet transitions to/from two, 12-foot-by-16-foot rectangular conduits and slides gates for emergency shutoff, 4) a 20-foot diameter steel lined pressure discharge conduit from the guard gate chamber, under the dam and downstream toe area to, 5) a discharge manifold providing for distribution of outlet flows to two, 8 to 12-foot diameter steel penstocks equipped with, 6) two 8- to 12-foot-diameter hollow jet (or similar plunge) valves, discharging to 7) an excavated plunge pool area. Two 8- to 12-foot diameter butterfly isolation values would be installed in the discharge penstocks for isolation of flows needed for installation or maintenance of the hollow jet valves. Access to the guard gate chamber of the flood control outlet would be through the vertical access shaft or the downstream water quality discharge pipe tunnel described in a subsequent section. Although the guard gates are expected to operate only under emergency conditions, provisions to service these gates, should it be required, would be provided by installation of bulkheads at the inlet structure, and removal of the gates through the chamber connection to the base of the 40-foot diameter access shaft for the water quality outlet system. The 100-foot-wide-by-100-feet-long-by-80-feet-high control room chamber would be located more than 300 feet underground and would be excavated into sound bedrock.

A reinforced concrete trashrack intake structure with structural steel bars protects the entrance and would prevent the entry of large woody debris that would not pass through the downstream control valves.

Occasional maintenance of the intake structure to remove accumulated debris will be required. With hydraulic controls located at the downstream end, the entire length of the conduit will remain pressurized during flood control operations. The guard gates will be designed for operation under unbalance head conditions.

The basic configuration of the outlet work described above is controlled to a significant degree by the large footprint of the rockfill dam, and the need for stream and flood diversion during construction of the dam. A combination of the outlet works, and the upstream and downstream cofferdams would provide a target level of protection of the dam construction work. Simplified hydraulic evaluations suggest that the current configuration would provide for safe routing of river flows of up to 12,000 to 15,000 cfs without overtopping of the upstream or downstream cofferdams and inundation of the dam and foundation construction activities. Further hydraulic and risk evaluations should be performed during subsequent study phases to refine the concept and configuration for this outlet works.

3.4.3 AUXILIARY (EMERGENCY) SPILLWAY

The auxiliary (emergency) spillway would be located in the upper right abutment area. It would consist of 1) an excavated approach channel, 2) an RCC crest control structure with a crest at elevation 687, 3) an RCC lined discharge channel, and 4) a 200 foot wide by 100 foot long stilling basin also lined with RCC. The RCC lined channel would be approximately 900 feet long and 200 feet wide at the bottom with side slopes inclined at 1.5H:1V. The spillway would discharge to an existing tributary stream downstream of the right abutment of the dam. Some stream channel widening and stabilization below the stilling basin will likely be required to protect the channel against potential erosion and headcutting.

There are no operating gates or operational requirements for the spillway. Similar to the previously described flood control and multi-purpose configurations, this spillway has been designed to safely route up to 100,000 cfs during a PMF event without consideration of reservoir routing effects. The configuration is therefore conservative from the standpoint of the impact footprint of the structure and cost.

3.4.4 WATER QUALITY CONTROL AND HYDROPOWER OUTLET

The outlet works for water quality control and potential hydropower generation would be located adjacent to the flood control outlet works at the base of the left abutment. The water quality/hydropower outlet works consists of 1) three, 48-inch diameter intake tunnels with inlet structures at elevations ranging from 500 to 620 in the upstream left abutment area, 2) a 40-foot-diameter, 290-foot deep access shaft , 3) connection of the intake tunnels and corresponding tunnel lining pipelines to a 60-inch diameter steel pipe manifold system in the access shaft leading to discharge at the base of the shaft into 4) a 15-foot-diameter, concrete lined and horseshoe shaped access tunnel containing a 60-inch-diameter discharge penstock, 5) transition to a downstream manifold directing flows to a low flow hollow jet discharge valve, or 24-inch and 60-inch penstocks to two hydropower turbines. The outlet system would discharge to the same plunge pool excavated for the flood control outlet system. Ample room has been included in the configuration for installation of the control facilities for the flood control outlet works, a hydropower plant at the toe of the dam, and for discharge facilities needed for low flow or fish passage discharges. Butterfly isolation values could be installed and operated immediately upstream of the low flow discharge valve or the hydropower turbines for isolation of flows that would allow for simultaneous operation and maintenance activities.

Slide gates would be installed on each intake tunnel pipeline discharging to the access shaft manifold. These gates would be operated based on reservoir temperature readings at multiple locations through the reservoir profile near the outlet intake tunnel inlet structures. The system would be designed so that blending of flows from multiple partial gate openings could occur. Each intake to the inlet tunnels would be equipped with an intake structure consisting of trash racks and fish screens. There are currently no design provisions for hoisting

and cleaning these trash racks and fish screens. Such activities would have to be included as part of operation and maintenance activities when required.

3.4.5 CONSTRUCTION CONSIDERATIONS

Similar to the multi-purpose RCC dam described in the preceding section, the multi-purpose rockfill dam and appurtenant structure configuration described in the preceding sections would be constructed in phases. A likely three phase construction sequence would include the following:

Phase I: Rerouting of the stream channel near the flood control outlet works intake structure along with construction of the intake flood control outlet works excluding installation of the downstream control valves would commence during this phase. This work would be protected by local cofferdams as required for protection of the work. The flood control and water quality outlets exit in the vicinity of a landslide mapped in the lower left abutment area. Partial removal and stabilization of this landslide along with redirections of a tributary stream channel near the same location would be completed during this phase.

Phase II: Phase I cofferdams and protection structures would be removed and the stream diverted through the flood control outlet works. The contractor would then install the primary cofferdams (Phase IIA) and construct the water quality/hydropower intake tunnel, access shaft, discharge tunnel and penstock and outlet release structures (Phase IIB) integral with construction of the dam, spillway stilling basin and associated downstream spillway channel modifications (Phase IIC).

Phase III: Each discharge manifold of the flood control outlet would be shut off and the discharge valves installed. The Contractor would install any minor downstream coffer structures and then proceed to finalize the construction of the flood control outlet works through completion of the plunge pool, and outlet works operating systems. Once installed, all gate operating systems for both the flood control and water quality outlet systems would be tested, final instrumentation systems installed and verified, and the reservoir would be taken through the initial filling sequence with final testing of all operating systems under normal head and operation protocols.

The location and cross-section of the cofferdams included in the planning level configurations presented in this TM are based on limited and simplified hydraulic analyses, and HDR's experience with similar structures and stream diversion requirements. As noted above, the actual crest elevation and cross-sections of the cofferdams and construction sequence will be established as part of future study phases based on hydraulic routing, geotechnical and construction risk evaluations. As currently configured, the cofferdams would serve as starter berms for the dam itself and become an integral part of the dam once it is completed.

Other landslide excavation and mitigation would be constructed during the appropriate phase of construction outlined above. Similarly, the construction phasing would include the required access road, power, communication, and operation and maintenance facilities incorporated during subsequent project planning phases.

Construction materials and staging will be a significant aspect of the planning, design, and construction of the project. Significant access, power, and construction staging areas are required for the dam and appurtenant structures described in the preceding sections and significant additional work is required to incorporate these items into the configuration should this dam be selected for further evaluation. Additional discussion of construction materials is provided in a subsequent section of this report.

3.5 CONSTRUCTION MATERIAL QUANTITY ESTIMATES

HDR has estimated quantities of materials necessary for construction of the selected dam options summarized in Table 3-1. These quantities provide the basis for an AACE Class 4 (AACE 2011) construction cost estimates and were used to develop and compare various dam and fish passage combined structure alternatives. Please refer to Section 7 of this TM for additional cost estimate discussion.

AACE is an international non-profit professional educational association that provides guidance and standards related to cost estimating, cost/schedule control, and project management to a wide range of professions and industries. AACE defines five levels, or classes, of cost estimates for a project. A Class 4 estimate is typical of project with a degree of definition (expresses as a percentage of complete definition) in the range of 1 percent to 15 percent and is appropriate for concept or feasibility studies, such as this TM. On the other hand, a Class 1 estimate represents the most detailed cost estimate for a project at a 70 percent to 100 percent degree of definition and is appropriate for evaluation of a contractors bid or tender. The typical accuracy range for a Class 4 estimate has an expected variation on the low end, L, from -15 percent to -30 percent and variation on the high end, H, from +20 percent to +50 percent (AACE 2011).

Quantity takeoffs for each dam option were developed using the methods and the level of effort commensurate with the requirements of an AACE Class 4 cost estimate. In particular, quantities were estimated based on the conceptual layouts provided in Appendix A and described in the preceding section s of this chapter. Simplified hand calculations were completed for the major construction items, consistent with our past experience with similar projects.

It should be noted that there have been no site characterization studies completed at the dam site to support the development of the configuration designs presented in this report. Hence, there are significant uncertainties in the quantities and unit prices estimated for a number of the required construction components of work. The configuration design is considered at this stage to be conservative and the estimated quantities presented in this TM are judged to have a greater than 50 percent chance of being less than or equal to the numbers indicated in the cost estimating tables. Consistent with the guidelines for construction cost estimating for dams (USSD 2012), the listed quantities represent actual estimated rounded up values and do not include uncertainties or related quantity contingencies. Estimates of uncertainties in the cost estimate have been included based on potential variation of unit prices for significant elements of work and in the applied planning contingency factors. The assumptions related to these uncertainties are described further in Section 7.

Table 3-2 provides a summary of the estimated quantities required for construction of the major elements for the baseline and two climate change scenarios for the RCC flood retention only dam (FRO-RCC Dam). Quantities of the major items of work required for construction of a baseline multi-purpose RCC Dam are summarized in Table 3-3 (MP RCC Dam). Likewise, quantities for the major items of work required for a baseline multi-purpose central clay core rockfill dam are summarized in Table 3-4 (MP RF Dam).

Table 3-2
Estimated Construction Material Quantities for the Baseline Flood Retention Only-RCC Dam

ITEM DESCRIPTION	QUANTITIES (CY)		
	BASE CASE	CLIMATE CHANGE 1	CLIMATE CHANGE 2
General Excavation	570,500	620,800	761,900
Rock Excavation	111,500	123,700	202,600
Roller Compacted Concrete	750,000	830,000	1,320,000
Conventional Concrete, Reinforced	27,000	30,000	34,000
Conventional Concrete, Unreinforced	75,000	75,000	94,000
Fill for Cofferdams and Foundation Backfill	340,000	376,000	479,000

Note: CY = cubic yards

Table 3-3
Estimated Construction Material Quantities for the Multi-purpose-RCC Dam

ITEM DESCRIPTION	QUANTITY (CY)
General Excavation	905,000
Rock Excavation	258,000
Roller Compacted Concrete	1,319,700
Conventional Concrete, Reinforced	45,000
Conventional Concrete, Unreinforced	64,000
Fill for Coffer Dams and Foundation Backfill	480,000
Fill Wing Embankment	118,000

Note: CY = cubic yards

Table 3-4
Estimated Construction Material Quantities for the Multi-purpose-RF Dam

ITEM DESCRIPTION	QUANTITY (CY)
General Excavation	2,588,800
Dental Concrete Foundation Treatment	31,000
Central Clay Core (Zone 1)	1,982,556
Filter (Zone 2)	539,316
Rockfill (Zone 3)	4,534,812
Riprap (Zone 4)	227,880
Fill for Coffer Dams	254,000
Reinforced Concrete for Outlet Works Structures	34,000
Spillway RCC Lining	36,000

Note: CY = cubic yards

The largest volume of material required for construction of an RCC dam is the aggregate used for production of the roller compacted concrete. The flood retention only RCC dam contains approximately 750,000 cubic yards of roller compacted concrete. As will be subsequently presented, the costs for RCC represents approximately one-third of the FRO-RCC dam total estimated base construction cost. The multi-purpose RCC dam contains approximately 1.3 million cubic yards of roller compacted concrete representing approximately half of its total estimated base construction cost.

The largest volume of construction material required for the central clay core rockfill dam is Zone 3 rockfill for the outer shells. Approximately 4.5 million cubic yards rockfill will be required representing nearly half of the total base construction cost for the multi-purpose rockfill dam.

3.6 DAM CONSTRUCTION MATERIALS AVAILABILITY

Because of the large quantities involved, the availability and proximity of the primary dam construction materials indicated in Table 3-2 through 3-4 is one of the major factors influencing the estimated construction cost for the dam configurations under evaluation. This section summarizes the results of a Quarry Rock Study prepared by Shannon & Wilson as well as the conclusions and recommendations of HDR relative to the dam design work. The S&W report is provided in Appendix D of this TM.

The Quarry Rock Study was completed in response to recommendations from the dam design team to verify that suitable aggregate for the RCC dam types could be found in reasonable proximity to the site. Both local rock and sand and gravel sources were evaluated to determine if the available materials were of suitable quality and quantity for either 1) batching as aggregate for an RCC dam, or 2) for use as Zone 3 or filter/drain/transition material for a rockfill dam. As part of this study the Washington Departments of Natural Resources and Transportation (WDNR and WSDOT) and other private parties were contacted to identify existing or dormant quarry locations in the Pe Ell to Chehalis, Washington area and to obtain information on the quality and quantity of materials that may be available from those sources. Samples of rock from the dam site, along with information from 12 existing rock quarries, and seven existing sand and gravel quarries were evaluated. The haul distances from the existing or dormant rock quarry sources to the dam site varied from 6 to 24 miles. The distances from the existing sand and gravel quarries to the dam site varied from 32 to 37 miles.

For the rockfill dam material it is important that construction material sources meet minimum standards for strength and durability, as shown by test results for specific gravity, absorption, abrasion, and degradation. For use as aggregate in roller-compacted or other structural concrete, it is also important that the aggregate is not “reactive” as shown by low alkali-silica reactivity (ASR) test values. Available rock testing data from the identified quarries along with additional ASR testing on three quarry rock samples and on a sample of rock from the dam site are included in Appendix D.

3.6.1 QUARRY ROCK STUDY RESULTS

Conclusions presented in the Quarry Rock Study TM were as follows:

- The rock sampled and tested from the dam site indicates that these materials could, with appropriate mining, crushing and screening procedures, be a viable source of concrete aggregate for a RCC dam. They also suggest that suitable materials exist at the site for Zone 3 shell or filter/drain/transition material for a rockfill dam.
- In addition, there are materials within 10 miles of the dam site that also appear to be suitable for use as concrete aggregate, filter, drain, or transition zone materials.
- It is yet to be determined if there are adequate quantities of the suitable aggregate or rock fill material at or near the dam site to meet the needs of each of the dam alternatives.

- Most tested quarry samples met test requirements for absorption, specific gravity, and abrasion, but not all sources have been tested for ASR. ASR testing of these sources would be required to verify material suitability.
- One nearby (9 miles) rock quarry sample met aggregate quality requirements except degradation. This potential material source would likely have to be retested if considered for use during subsequent evaluations.
- All natural sand and gravel sources are at least 32 miles from the dam site and some would require additional long-term ASR testing to demonstrate suitability for use as aggregate in structural concrete. However, two quarry owners indicated that they were not aware of any reported ASR problems with local aggregates obtained from these sources. WSDOT currently uses some of these materials for structural concrete aggregate.

Based on the information presented in this TM and our experience with other RCC and rockfill dams in Oregon and the western United States, the design team believes that there is a reasonable likelihood that suitable concrete aggregate, filter, drain, transition zone, and rockfill materials can be obtained within reasonable proximity of the site. Our experience with mining, crushing, and screening operations for production of coarse and fine aggregate for concrete from hard rock quarries suggests that desired quantities of finer sand materials can be lacking and expensive to produce. It is not uncommon to obtain the substantial majority of concrete aggregates from processing of materials from local hard rock quarry's and to supplement those materials as appropriate (cost effective) with imported sand materials. Completion of material balance studies needed to identify the total quantity of construction materials from on-site verses off-site sources should be performed as part of future evaluations to support environmental permitting and refinement of cost estimates.

Fish Passage Options

4 Fish Passage Options

This section presents the selected fish passage options that will be added to the dam options to produce the combined dam and fish passage alternatives. Fish passage options are divided into two categories: upstream passage and downstream passage options. These options were selected and formulated through the fish passage design study (HDR 2014b). The design elements formulated for each fish passage option were selected based upon the needs of the project, overall fish passage objectives, and professional experience with similar high dam passage projects throughout the western United States. Subsequent subsections include narratives and illustrative concepts for each selected fish passage alternative. Also included is a discussion on dam operations as it relates to fish passage.

For the flood retention dam structure, the fish passage tunnels through the structure were intended to provide the singular mode of both upstream and downstream fish passage. However, the anticipated duration of water retention events combined with additional retention periods required for debris management and operational restrictions resulted in significant periods when the conduits could not be used for fish passage. Based upon flow modeling scenarios, some of these closed conduit periods would be coincident with upstream migration periods of target fish species. Pursuant to the state of Washington's fish passage laws and guidelines, it is therefore necessary to include a redundant form of fish passage to be operated during most anticipated retention events. To accomplish this objective, a CHTR facility has been added to the flood retention dam structure.

For the multi-purpose dam structure, any combination of upstream and downstream alternatives presented below can be paired to provide a complete fish passage system.

4.1 UPSTREAM FISH PASSAGE OPTIONS

The following subsections provide a more detailed narrative of each upstream, multi-purpose fish passage option, including descriptions of major design components, considerations, and concept sketches.

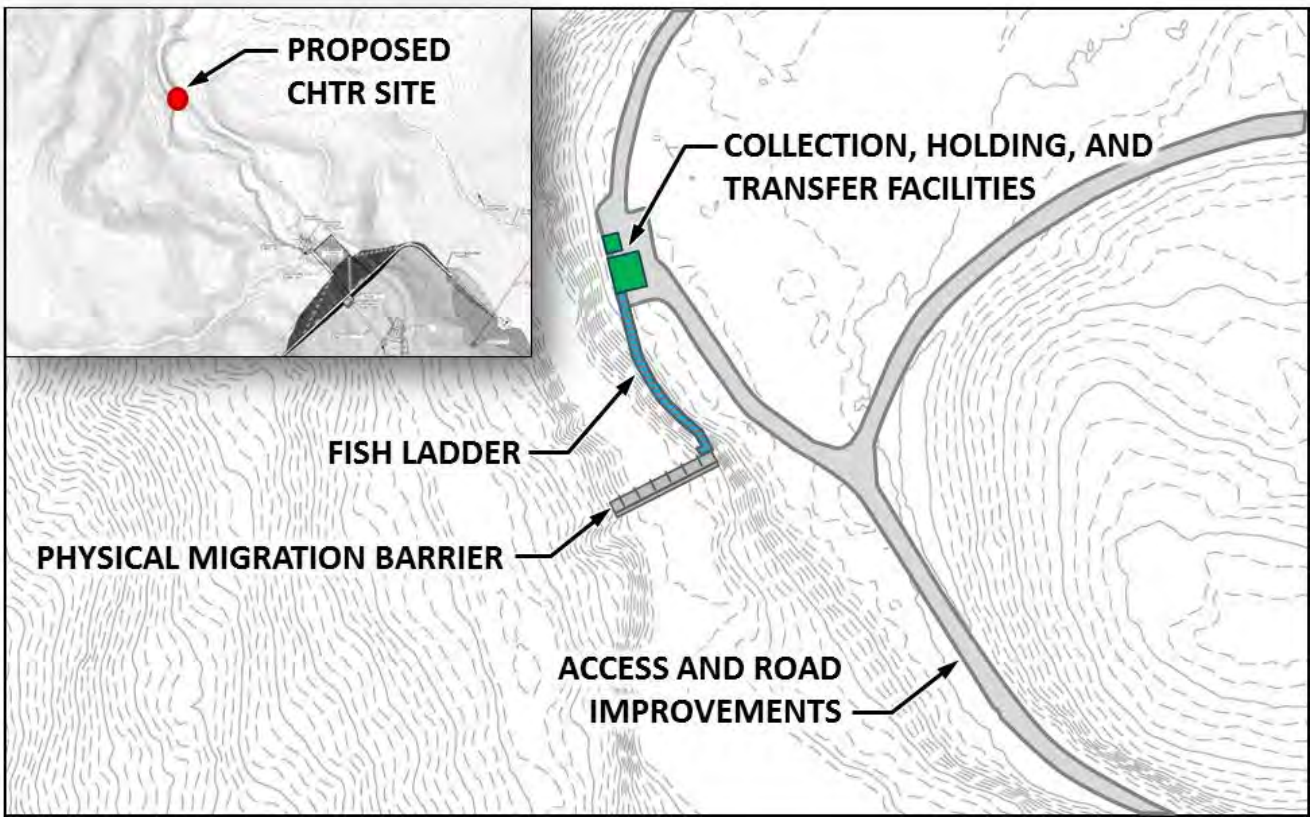
4.1.1 CHTR FACILITY AT BARRIER DAM

A CHTR facility is one of the potential options being considered for the multi-purpose dam project. This option includes the necessary system components for the safe and effective collection, handling, transfer, and release of fish to a suitable location upstream of the reservoir. Facility operations may be performed at a range of frequencies dependent upon the presence and migration tendencies of the target fish species. Facilities of this nature are not typically considered unless implementation of a conventional fishway is identified as an impractical or non-viable project option. In high dam applications such as this, there is not a precedent for providing volitional fish passage as defined. The primary challenge for providing volitional upstream fish passage at a high head dam such as this is the ability to accommodate fishway hydraulic connectivity over a wide range of reservoir water surface elevations that fluctuate over the course of fish migration seasons.

CHTR facilities typically include a channel-spanning passage barrier which guides fish towards a fishway entrance. Fish enter the fishway and ascend upstream into a collection facility. Here, fish remain in collection

and holding areas until they are transferred to a haul truck and/or handled for monitoring purposes. Fish could then be transported around the dam to release locations upstream of the reservoir. This operation takes place at least once a day and sometimes more often dependent upon the size of the facility and the number of fish ascending upstream during seasonal migration fluctuations. Figure 4-1 shows for a conceptual layout of the CHTR facility.

Figure 4-1
Conceptual Layout of CHTR Facility at Barrier Dam



The disadvantage of CHTR systems is that migration is not volitional; therefore, fish will experience stresses associated with increased holding, handling, and transportation. Additionally, CHTR facilities require a high level of attention and effort by trained individuals which requires higher levels of annual operation and maintenance funds in perpetuity compared to other types of fish passage facilities.

A primary advantage to the CHTR alternative is that the facility operates independent of forebay fluctuation, which commonly limits the applicability of conventional fishways. The CHTR facility also allows for captured fish to be transported above the reservoir, eliminating the need for fish to navigate the reservoir and potentially furthering migration delay. Further, more conventional passage technologies, such as fishways, fish lifts, and fish locks have limited applicability and reduced efficiency at high head dams like proposed multi-purpose dam.

Two locations were considered for siting a CHTR facility: the first near the tailrace or stilling basin of the dam and a second location further downstream of the dam. The current preference is to locate the CHTR facility downstream of the dam due to easier vehicle access and uncertainty of hydraulics within the dam stilling basin, possibly reducing capture efficiency. Because the CHTR facility would be sited downstream of the dam, a physical migration barrier would be required.

4.1.1.1 Channel Spanning Migration Barrier

In fish passage applications such as this where the base of a dam is difficult to access or construct fish passage facilities, fish migration barriers are used to limit upstream migration at a specified location and to guide fish to the entrance of a collection facility. Any fish that moved past the entrance would likely move to the base of the proposed dam where they would be subject to confusion, expenditure of additional energy, additional aging in fresh water prior to spawning, and subsequent migration delay. Although attraction water would be released at the collection facility, it is unknown whether fish would fall back and find the entrance of the collection facility efficiently. The addition of a migration barrier minimizes this potential for delay.

Several types of migration barriers could be considered for this application. Various types of physical, hydraulic, and behavioral systems are all applicable - each one exhibiting different tradeoffs that should be considered if future iterations of design development occur. For the purposes of this document, a fixed velocity weir was selected as the preferred approach for the purposes of estimating project costs.

A velocity weir could be composed of a new channel spanning cast-in-place reinforced concrete weir installed in the bed of the Chehalis River. The intent of the velocity weir is to increase hydraulic velocity and lower hydraulic depth over a sufficient distance that exceeds the swimming capability of target fish species and can also spread the flow out along the weir length to make it less attractive to the fish as compared to a fishway entrance. The weir is typically compact in size with the limiting design factors generally related to structural stability and durability rather than related to its hydraulic function. There are also many different configurations. Conceptually, the weir could span the Chehalis River at a slight angle to the flow. The weir span would be approximately 100 feet long with a width ranging from 30 to 40 feet and height ranging from 10 to 12 feet above the target thalweg elevation of the river. In profile, the weir crest could be sloped or stepped to help balance flow mass and facilitate low flow hydraulic preference towards the collection facility. Features downstream of the weir could be installed to dissipate energy, limit bed degradation, and return hydraulics to a more natural condition.

One example of a hydraulic migration barrier that functions similar to the concept described above is the adjustable velocity weir located on the Feather River in California. This weir incorporates an adjustable sill to meet seasonal project requirements such as flood control and recreation. An image of this structure is provided in Figure 4-2.

Figure 4-2
Example Hydraulic Migration Barrier on Feather River, California

Note: Photograph by HDR

4.1.1.2 Fishway

After encountering the velocity barrier described in the previous section, fish will be guided to river right toward the entrance to a fishway. The purpose of the fishway is to allow fish that are located at a lower elevation in the river to ascend volitionally upstream to a higher elevation where the collection and holding facility would be located. The collection and holding facility should be sited at an elevation that is above flood levels and at a suitable location that provides easy access and egress by personnel and vehicles. In this case, it is assumed that the maximum vertical distance between the river and a future collection facility would be on the order of 20 feet.

Conventional fish ladders include a range of potential types of fabricated structures used to facilitate passage of fish over or around an obstacle, dam, or other migration barrier. Although there are multiple variations, the three most common conventional fish ladders are pool and weir (including pool and chute and ice harbor), baffle (Denil, Alaskan steep pass, or other baffle configurations), and vertical-slot. One or more types of ladders can be used in combination to create a passage structure that meets site specific conditions. The overall slope of conventional fish ladders typically ranges from 8 percent up to 15 percent or slightly more depending on the type and configuration of the ladder or series of ladder segments. An example photograph of a vertical-slot controlled fishway is shown in Figure 4-3 and an example photograph of a pool and weir fishway is shown in Figure 4-4**Error! Reference source not found.**.

Figure 4-3
Example Vertical-slot Fishway at Coleman National Fish Hatchery on Battle Cr. near Red Bluff, California

Note: Photograph by HDR

Figure 4-4
Example Pool and Weir Fishway at a Diversion Dam on Corralitos Creek Near Santa Cruz, California

Note: Photograph by HDR

Another important design consideration is the development of sufficient attraction flow. The rule of thumb for providing attraction flow suggests that a fishway must exhibit 10 percent of the high fish passage design flow. Generation of attraction flow is more important as the width and hydraulic complexity of a river system increases. Fish passage design flows on the Chehalis River range from 14 to 2,000 cfs. This suggests that a fishway located at a proposed dam site may need to provide a 10 percent attraction flow of up to 200 cfs. It should be noted that the 10 percent attraction flow requirement may be reduced with the use of a guidance barrier but would still require a design variance approval from the agencies. Most fishways operate effectively within a range of 2 cfs (pool and weir) to 100 cfs (ice harbor). Vertical slot controlled fishways can be configured to operate within a range of 10 to 80 cfs effectively. Given that the total attraction flow which could be required for this project is greater than the typical capacity of conventional fishway systems, an auxiliary water supply system will likely be required. Discussion of a potential auxiliary water supply (AWS) system is discussed in the next section of this document.

Given the design considerations for this unique application, one potential concept could be to construct a vertical-slot or ice harbor type fishway which would allow fish to ascend from the river to the proposed collection and holding facility. For the purposes of this document, it is assumed that each baffle would be designed for a hydraulic differential of 0.9 feet. Each pool could have dimensions of 8 feet wide by 10 feet long. The height of the fishway would need to be at least 9 feet tall but would likely be designed to accommodate surrounding topographic features and protection from higher flow levels. Therefore, it is assumed that the fishway walls could be on the order of 12 feet tall. With a total target height differential of 20 feet and 0.9 foot of vertical drop per pool, it can be assumed that the fishway could have a minimum of 23 pools. By assuming that each pool is 11 feet long (1 foot of additional pool length to accommodate baffling) and that the entrance and exit pools are 30 feet long, the minimum fishway length could be on the order of 250 feet long. Additional length could be required for a single turning pool, resting pools, and for the auxiliary water supply. Therefore, in concept, the fishway could be approximately 350 to 375 feet long. This configuration and length could accommodate the assumptions provided above and would likely change if the location and site conditions were to be revised as part of further concept development activities. For example, if the total vertical height from the river to the collection and handling facility is less than 20 feet, the resulting fishway could likely be shorter.

Fishway construction is composed primarily of cast-in-place reinforced concrete foundations, slabs, and walls. There are a number of miscellaneous metal fabrications including embeds, debris racks, ledgers, guide slots, grating, and safety railing. In addition, there are typically hydraulic control, fish monitoring, and maintenance systems that are all a part of fishway design and construction. These items, although not explicitly discussed, are assumed to coincide with fishway design and construction.

4.1.1.3 Auxiliary Water Supply System

An AWS system is typically used to supplement attraction flow at the entrance to a fishway. Fishways are typically designed to convey a range of flows to accommodate the migration of fish that enter the fishway entrance. The design flow is based upon several factors including: the availability of water, hydraulic variation of entrance and exit conditions, and the type of fishway that is desired. Typically, a smaller fishway that accommodates the target fish species with a lower flow capacity is more cost effective than a fishway that is designed to accommodate the entire range of attraction flows. If a selected range of design flows remain lower than the total attraction flow required for the project, an AWS system is integrated which bypasses the fishway and adds supplemental flow to or near the fishway entrance through chambers and coarse screens to dissipate the energy and optimize the attraction flow field attributes of the AWS flow. The source of water for the AWS could be an intake in the reservoir, a penstock from a hydropower plant, or other options depending on the type of the dam and outlet works alternatives selected.

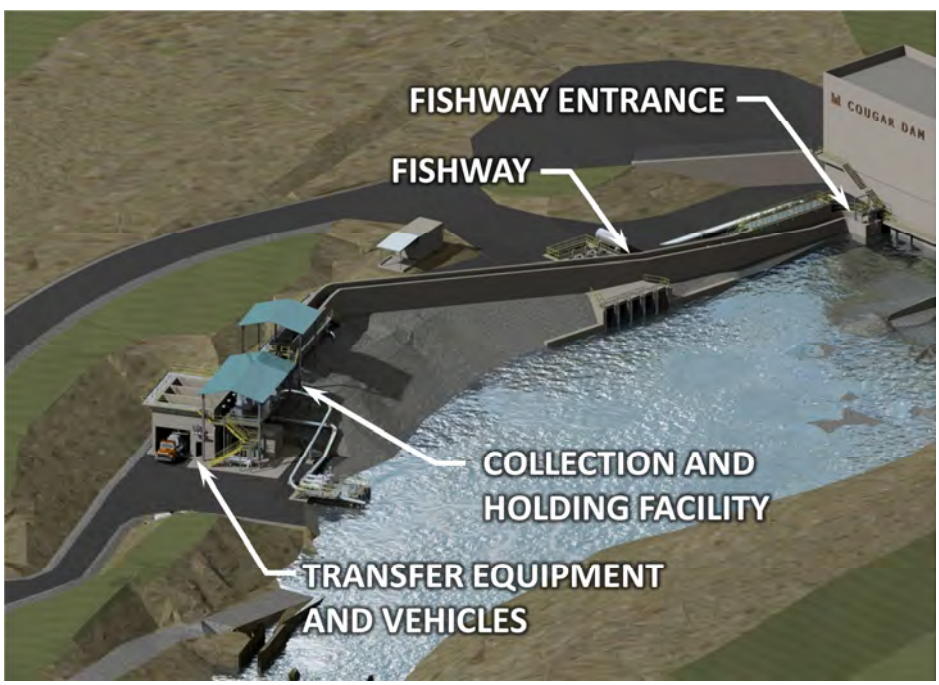
In concept, the ice harbor or vertical slot fishway concept described above could operate within a flow range of 10 to 50 cfs. Because the attraction flow requirement for this project maybe up to 200 cfs (lower if a variance is approved with the use of a guidance barrier) when river flows are up to 2,000 cfs during migration periods, a supplemental source of water could be used to convey the remaining 150 cfs. This keeps the overall cross-section of the fishway economical and provides flexibility during operational periods.

Construction of the AWS system could include: addition of diversion bifurcation structure at the migration barrier; addition of throttling gates/valves to control the primary source of water to the fishway, as well as the AWS; miscellaneous piping; fabricated metals; and cast-in-place structures.

4.1.1.4 Fish Collection and Holding Facility

After fish enter the fishway entrance and ascend to the top of the fishway, they enter the collection and holding facility. Facilities of this nature are composed of a wide range of structural, civil, hydraulic, aquaculture, and mechanical design systems. In general, there are typically one or more holding pools that are separated by fykes, picket panels, and segregation screens. Fish move into the first collection pool and move past a fyke or false weir which does not allow them to swim back down into the previous section of the fishway or collection pool. After they enter the initial collection pool, fish can be allowed to continue upstream into the next holding area, physically crowded to separate holding areas, and/or separated into species or size class. Here, fish are collected and handled for additional monitoring purposes or they are transferred via water to water transfer to a transport vehicle. An example rendering of the collection and holding facility developed for the CHTR facility at the base of Cougar Dam in Oregon is provided in Figure 4-5**Error! Reference source not found.**.

Figure 4-5
Rendering of Existing CHTR Facility at Cougar Dam for the U.S. Army Corps of Engineers



Source: USACE 2014

4.1.1.5 Fish Transport Vehicle

Fish that are collected in the CHTR facility will require transfer to the desired release location upstream of the reservoir. Fish could be moved from the holding area to a specially equipped vehicle via water-to-water transfer piping (Figure 4-6). Vehicles used for this purpose are sized for the number and size of fish that are anticipated to be collected on a daily basis in addition to the water that is required to safely accommodate them. Each vehicle is equipped with life sustaining and water conditioning equipment to maintain adequate temperature, dissolved oxygen, and carbon dioxide levels during transfer. At a minimum, fish are typically transferred upstream once every 24-hour period. During periods of peak migration, more trips to the desired release site or additional transport vehicles may be necessary.

Figure 4-6
Example Transport Vehicle Used to Move Fish from the CHTR Facility to Release Location

4.1.1.6 Fish Release

Fish that are collected in the CHTR facility and transported past the dam will require release to a selected location upstream. This could be at the head of the reservoir or directly to the stream channel at a suitable location. The release location may require facilities to acclimate and protect the fish being transferred. These facilities could be coincident and combined with potential downstream fish passage alternatives at the head of or above the reservoir (i.e., dual-use). Fish could be moved from the transport vehicle to a holding pool via water-to-water transfer piping and then allowed to move out of the holding pool to the reservoir or the stream via a channel or additional piping. Considerations for release facilities could include requirements for separate water supply intakes with the possibility of pumping, water conditioning equipment, and other mechanical equipment for safe handling and release of the fish.

For the purposes of cost estimating and as a standalone alternative, it is assumed that fish will be released directly to the stream at a suitable location upstream of the reservoir via a fixed pipe (or pipes) from the top of bank where the transport vehicle can be positioned for water-to-water transfer. A covered vehicle concrete pad at the pipe inlet is assumed to ensure efficient and consistent connection to the pipe from the transport vehicle

and to be able to safely operate in variable weather conditions. It is also assumed that the transport vehicle will have the necessary pumping equipment to provide make-up water or acclimation water in the tank(s) for water-to-water transfer and will obtain the water from the stream via a separate screened water supply pipe adjacent to the fish release pipe. Considerations for siting the fish release pipe include ensuring a large receiving pool in the stream channel is available for the pipe outlet over a range of flow conditions that could potentially occur during fish releases. A design consideration of the fish release pipe also includes protection of the pipe(s) from flood flows. Additional siting and design considerations may include needs for security and predation control.

4.1.2 CONVENTIONAL FISHWAY

Similar to the CHTR facility, two fishway entrance locations were considered: the first at the downstream face of the dam and the second located further downstream of the dam. Site constraints due to the steep hillside, as well as the uncertainty of hydraulics within the dam stilling basin led to the preference of locating the fishway entrance downstream of the dam. A fishway entrance in this location requires a physical migration barrier that incorporates velocities and jump heights unattainable by fish and provides flow conditions to guide and attract fish toward the fishway entrance.

A conventional fishway from below the dam to the forebay would incorporate the following components: channel spanning migration barrier, AWS system, fishway, and a fishway exit structure to maintain hydraulic connectivity to the fluctuating forebay. See Figure 4-7
Conceptual Layout of Conventional Fishway**Error! Reference source not found.**.

4.1.2.1 Channel Spanning Migration Barrier

A channel spanning migration barrier identical to the one previously described for the CHTR facility would be implemented for the conventional fishway alternative. See the CHTR facility description for barrier details.

4.1.2.2 Auxiliary Water Supply System

An AWS system identical to the one previously described for the CHTR facility would be implemented for the conventional fishway alternative. The AWS would supply additional water at or near the downstream end of the fishway to increase attraction flow. See the CHTR facility description for details of this component.

4.1.2.3 Fishway

After encountering the velocity barrier, fish will be guided to river right toward the entrance to a fishway. See the CHTR description of fishways for a more detailed description of different fishway types.

The primary difference between the CHTR fishway and the conventional fishway alternative is the conventional fishway continues to the dam, resulting in an approximate vertical distance between the migration barrier and reservoir forebay of 200 feet.

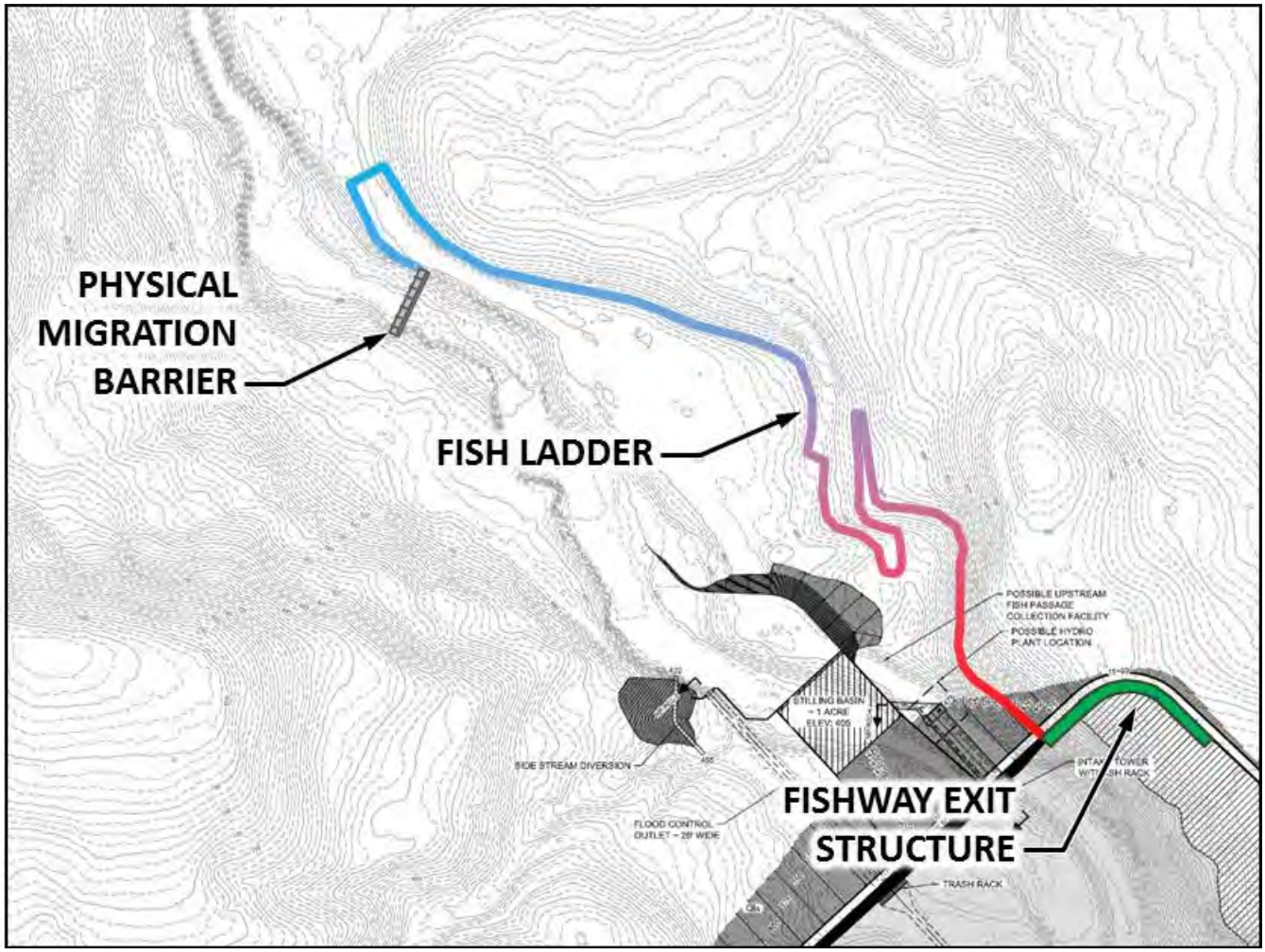
In order to develop a conceptual design, the following assumptions were made to estimate the fishway length:

- Head differential of 200 feet, estimated from dam operations and topography of the expected barrier weir location to the fishway exit structure outlet at the dam. Note that the total expected maximum operating head differential is 230 feet, which includes an additional 30 feet for the fishway exit structure to the normal operating reservoir elevation.
- Vertical slot pool length of 10 feet and 1 foot thick baffle, for a length of 11 feet per pool.
- Head drop of 0.9 foot per pool.
- Entrance pool length of 30 feet.
- Resting pool required every 100 feet.
- Resting pool length of 20 feet.

- Transition pool to reservoir fishway exit structure length of 30 feet.

Using the assumptions stated above yields 223 vertical slot pools in addition to 25 resting pools, resulting in an approximate fishway length of 3,000 feet downstream of the dam. The fishway pools for the exit structure are described separately below.

Figure 4-7
Conceptual Layout of Conventional Fishway



4.1.2.4 Fishway Exit Structure

In general, fish ladders can operate only over a narrow range of depth fluctuations in the fishway exit (upstream end) for maintaining hydraulic connectivity and be successful for fish passage out of the fishway. The variation in forebay and tailwater elevations is an important design consideration and can limit the efficacy of various types of fishways. The greater the fluctuation observed, the more difficult it is to provide upstream passage successfully over the range of anticipated migration flows without a series of added appurtenances such as complex hydraulic controls and multiple entrances and/or exits in order to maintain hydraulic connectivity.

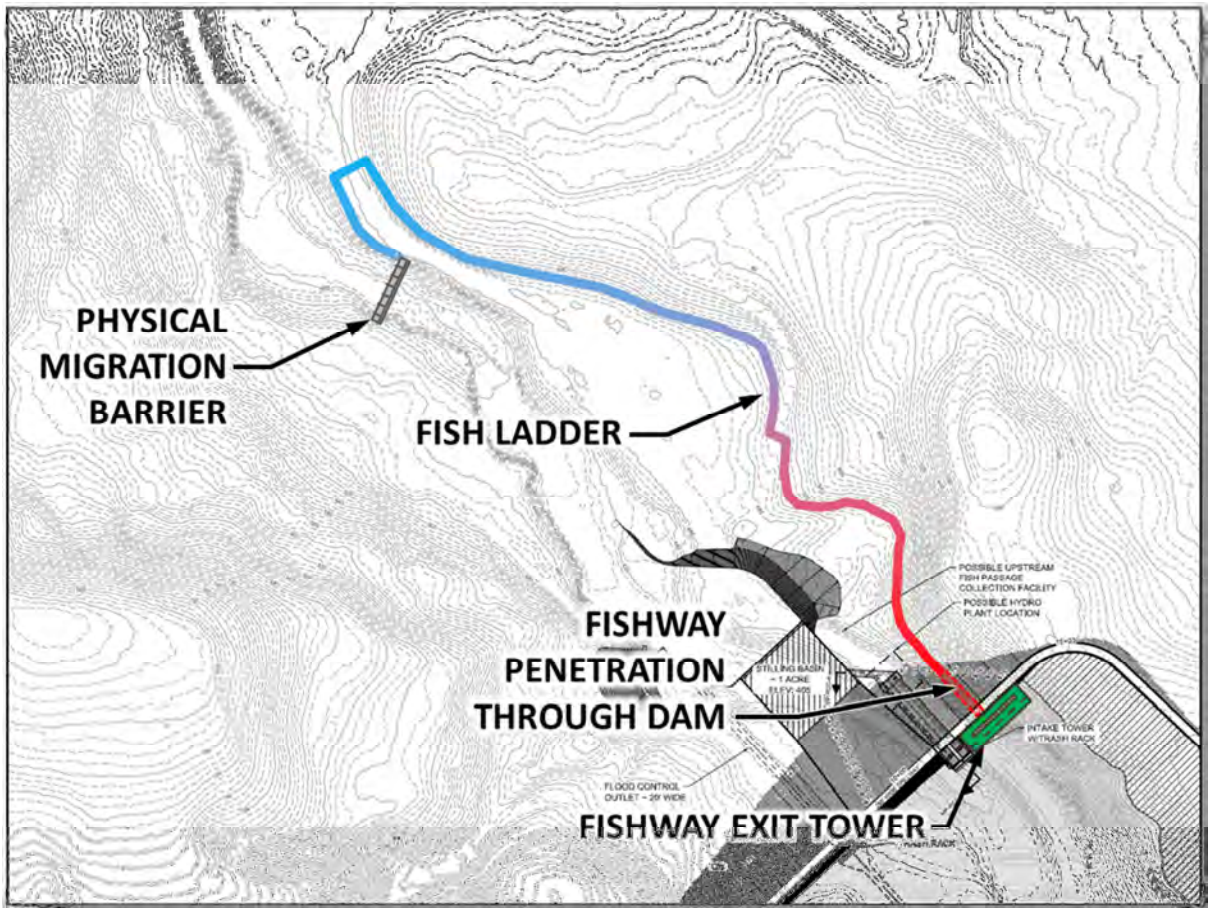
A fairly complex fishway exit structure would be required within the forebay in order to accommodate a range of reservoir elevations. Typically, fishways can be reasonably designed to accommodate up to 10 feet of forebay fluctuation with passive hydraulic controls like a vertical slot portion of fish ladder. However, for an effective design to accommodate larger ranges of forebay fluctuation, complex hydraulic controls of telescoping weirs in a flow control section or stair-cased sections of fish ladder with multi-level exit ports would be required. Currently the design team is estimating that a fishway exit structure could be developed to handle up to 30 feet of forebay fluctuation that includes a flow control section of at least 34 telescoping weirs. This flow control section would be housed inside a concrete channel or chamber running along the reservoir side for a length of the dam (Figure 4-7**Error! Reference source not found.**). Using the fishway geometry assumptions to estimate the length of the fishway exit structure in the reservoir yields an approximation of 450 feet for the fishway exit structure. The exact type and size of exit gates would be determined during design development along with operational and other design considerations.

4.1.3 CONVENTIONAL FISHWAY WITH AN EXPERIMENTAL FISHWAY EXIT TOWER

In addition to a conventional fishway with the fishway exit structure just described, a conventional fishway is also being paired with an experimental fishway exit tower. The multi-purpose dam operations indicate that forebay elevations will vary in excess of 125 feet. This large variation would mean a fish ladder would only be able to operate over a very limited range of possible forebay elevations and still provide the hydraulic connectivity for volitional fish passage without a more complex system. Therefore, a conventional fishway with an experimental fishway exit tower is included as an upstream fish passage alternative for the multi-purpose dam with an assumed ability to accommodate up to 80 feet of reservoir fluctuation which covers a substantially larger portion of reservoir water surface elevations during fish migration periods of targeted species.

The conventional fishway with an experimental fishway exit tower would have very similar components to the conventional fishway alternative described in the previous section. Fish passage components would include: a channel spanning migration barrier, AWS system, fishway, and an experimental fishway exit structure to accommodate approximately 80 feet of forebay fluctuation. For a conceptual layout of this alternative see Figure 4-8.

Figure 4-8
Conceptual Layout of Conventional Fishway with an Experimental Fishway Exit Structure



4.1.3.1 Channel Spanning Migration Barrier

A channel spanning migration barrier identical to the one previously described for the CHTR facility would be implemented for the experimental fishway alternative. See the CHTR facility description for details of this component.

4.1.3.2 Auxiliary Water Supply System

An AWS system identical to the one previously described for the CHTR facility would be implemented for the experimental fishway alternative. The AWS would supply additional water at near the downstream end of the fishway to increase attraction flow. See the CHTR facility description for details of this component.

4.1.3.3 Fishway

The fishway portion for this fish passage alternative would be the same conceptually as that described for the conventional fishway alternative and would continue from the barrier weir to the dam. The length of the fishway would be slightly shorter however, approximately 2,300 feet in length using the same ladder design assumptions as the conventional fishway. The only difference being that this alternative's conventional fishway portion would have an approximate head differential of 150 feet, 50 feet less than the conventional fishway alternative. The fish ladder portion of this alternative has a slightly lower head differential downstream of the dam because a substantial portion of the elevation gain is accommodated within the experimental fishway exit tower located within the reservoir.

4.1.3.4 Experimental Fishway Exit Tower

As previously described, conventionally designed fish ladders can operate only over a narrow range of depth fluctuations at the fishway exit (upstream end) and still maintain hydraulic connectivity and be successful for fish passage out of the fishway. It is unprecedented to accommodate more than 10 feet of forebay fluctuation with passive hydraulic controls. A more complex fishway exit structure with multiple levels of hydraulic control features such as gates would be required within the forebay in order to accommodate a larger range of reservoir elevations. The design team is including an option in this case that is considered to be an experimental fishway exit tower that will accommodate up to 80 feet of reservoir water surface fluctuation. This will not accommodate the total range of forebay fluctuation for the multi-purpose reservoir, which is over 150 feet from normal reservoir operating surface to the low pool elevation. However, 80 feet covers a substantial portion of reservoir water surface elevations during migration periods of the targeted fish species.

Using the statistics estimated for reservoir operation of multi-purpose dam alternatives, the rationale and assumptions for choosing the experimental fishway exit tower operating range can be provided. See Appendix A for the fish passage assessment relative to proposed operations of the multi-purpose dam alternative. The annual reservoir water surface elevation ranges from elevation 628 feet for a 1 percent duration exceedance (which represents the normal operating pool elevation) down to elevation 563 feet for a 90 percent duration exceedance. In other words, less than 1 percent of the time the reservoir is above an elevation of 628 feet and 90 percent of the time the elevation is expected to be above 563 feet on an annual average basis. This is a total range of 65 feet. However, examining select periods of the year for anticipated migration seasons shows this does not accommodate coho salmon, Chinook salmon, and pacific lamprey fall migration seasons very well as the expected reservoir elevations during the fall are only expected to be above approximately 535 feet 90 percent of the time on average (see Figure 4 in Appendix A for stage duration curves for each species' migration period). To accommodate these species for 90 percent of the time during their migration period, on average of the expected reservoir elevations a forebay fluctuation of up to 93 feet would be needed. By accommodating up to 80 feet in reservoir range the experimental fishway exit tower provides up to 80 percent of the time on average for the coho salmon (*Oncorhynchus kisutch*), Chinook salmon (*Oncorhynchus tshawytscha*), and Pacific lamprey (*Entosphenus tridentatus*) fall migration seasons and at least 90 percent of the time on average for the other species' migration seasons. The design range of this alternative would be further refined as the project progresses considering potential operational changes and refinement to analyses. For concept-level cost estimating purposes, 80 feet is chosen to represent a reasonable range of reservoir elevation fluctuation.

As just described, to accommodate the designated range of reservoir elevations for the multi-purpose dam alternative the experimental fishway exit tower is proposed. It is considered experimental because it is unprecedented at this scale. This option would consist of a tunnel at approximate elevation 548 feet connecting the fishway portion that is downstream of the dam, either through or around the dam, to the base of a concrete tower in the reservoir. The tower contains flights (stair-cased sections) of a fish ladder channel and exit channel with gated multi-level exit ports that serve as the hydraulic inlet and connectivity from the forebay water surface elevation to the corresponding pools in the fish ladder. Location and other design aspects of the tower will be dependent on the type of multi-purpose dam being considered (RCC or rock fill) and structural and foundation considerations for this type of structure.

This tower concept is most analogous to a multi-level gated intake tower that may be used for depth-selective water withdrawal for water quality control in a deep reservoir, except that to accommodate flights of a step-wise fish ladder channel and gallery of multi-level gates inside, it will need to be larger. Using the fishway geometry assumptions to estimate the length of a flight of fish ladder section in the tower yields an approximation of 150 feet for the tower structure length. This would provide enough length for 12 pools per flight of fish ladder channel. The width of the tower would likely need to be at least 25 feet to provide interior room for the fish ladder channel flights, the exit channels with the gate gallery, the transition/turning pools, and

shafts for light and access. For the 80 feet of range from the normal operating pool elevation of 628 feet to the low operating forebay elevation of 548 feet there would be a minimum of 89 fish ladder pools and exit gates in the tower spread out over eight flights (12 pools per flight plus extra room for transition/turning pools). It is anticipated that the gates could be automated to open and close for fluctuating forebay elevations and connect the corresponding fish ladder pool. Operational and other design considerations would be developed as the project progresses and alternatives are selected. The type and size of exit gates would also be determined during design development. For preliminary cost estimating purposes, it is assumed that 4 feet wide by 5 feet tall slide gates with motor operated actuators to be remotely controlled would be used. A concept-level schematic of a plan view of one of the flights and a partial elevation view of a series of gates of the fishway exit tower is shown in Figures 4-9**Error! Reference source not found.** and 4-10 respectively.

Figure 4-9
Concept-level Plan View of One Flight of the Fishway Exit Tower

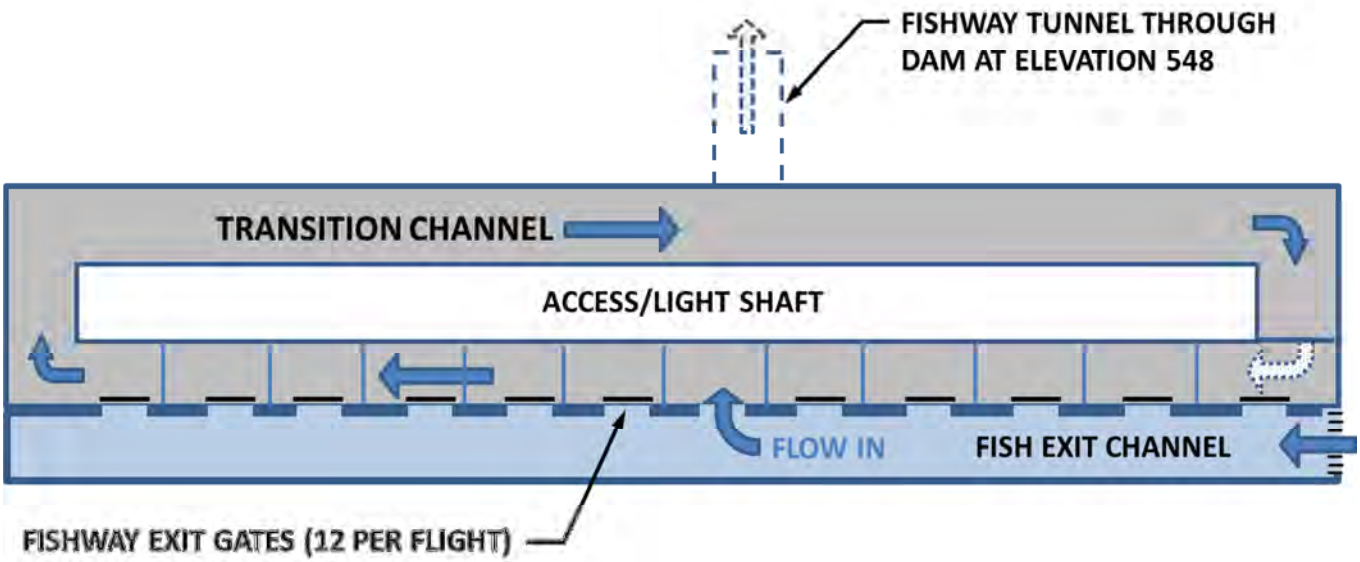
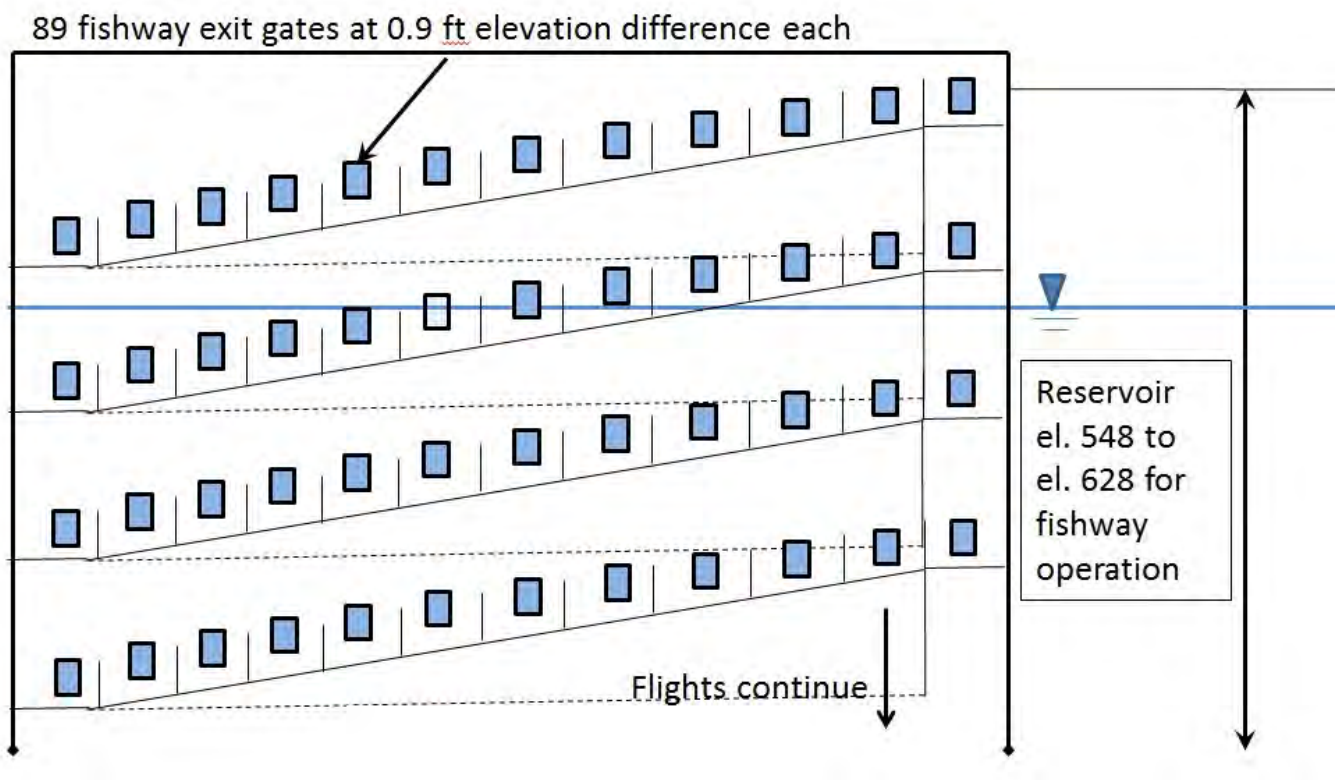


Figure 4-10
Elevation View Through Exit Channel of Experimental Fishway Tower



4.2 DOWNSTREAM MULTI-PURPOSE FISH PASSAGE ALTERNATIVES

The following subsections provide a more detailed narrative of each downstream multi-purpose fish passage alternatives, including descriptions of major design components, considerations, and concept sketches.

4.2.1 FLOATING FOREBAY COLLECTOR

The fish passage system selected to represent a facility that can provide downstream passage of juvenile fish is a floating surface collector (FSC). FSCs include a class of similar type structures that are used to attract, collect, and hold out-migrating fish that are routed through a reservoir. Structures of this nature float on the reservoir surface and can typically be designed to accommodate up to 100 feet of water surface fluctuation. At the time this document was prepared, there are approximately four to five (total) modern fixed and FSCs under full scale operation, all of which are located in the Pacific Northwest. This does not include the number of fixed surface-spill type corner collectors located on Columbia River dams and east coast river systems. Although initial data appears to show that collection efficiencies of surface collectors are high, they are still classified as a relatively new and experimental technology with only a few years of actual full scale operation and fish monitoring data. The most prominent of these facilities is the Upper Baker Dam FSC which began operation in 2008. The other currently operating facilities include the following:

- Swift FSC on the North Fork Lewis River, Washington (2012)
- River Mill FSC on the Clackamas River, Oregon (2012)
- Round Butte Fish Collection System on the Deschutes River, Washington

Facilities currently in various stages of design and construction include the following:

- Cushman FSC on North Skokomish River, Washington
- Upper Cowlitz Fixed Surface Collector on the Cowlitz River, Washington
- Cougar Dam Portable Floating Fish Collector on the McKenzie River, Oregon

In general, FSCs are composed of a number of design elements ranging from very basic in function to highly complex automated and mechanical systems. The floating platform uses an array of low-head, high-volume pumps to generate attraction flow towards the fish collection module inlet. Fish sense the outgoing velocity and tend to swim towards the outgoing flow. Their path may be intercepted by a fish guidance system (FGS) comprises panels or nets suspended in the top third of the water column. The FGS guides fish towards the collector inlet. Both fish and water enter the collector inlet. The water and fish are then gradually accelerated until a capture velocity is reached. A high proportion of total collector flow is gradually removed through screens from the collector module and bypasses the collection and holding system during flow acceleration and/or after fish capture and flow deceleration. Collected fish and low volumes of water are then conveyed to holding areas. Fish can then be crowded to separate holding areas, and/or separated into species or size class where they can be sampled for monitoring purposes. Fish are then transferred from the holding areas to a transport vehicle via a bypass conduit or hopper system. A schematic showing one example of a collection and transfer process is illustrated in Figure 4-11. Figure 4-12**Error! Reference source not found.** provides a concept sketch of how a FSC facility could be incorporated in the project just upstream of the proposed dam.

Figure 4-11
Schematic Illustration Showing the Fish Collection and Transfer Process Which Occurs in an FSC

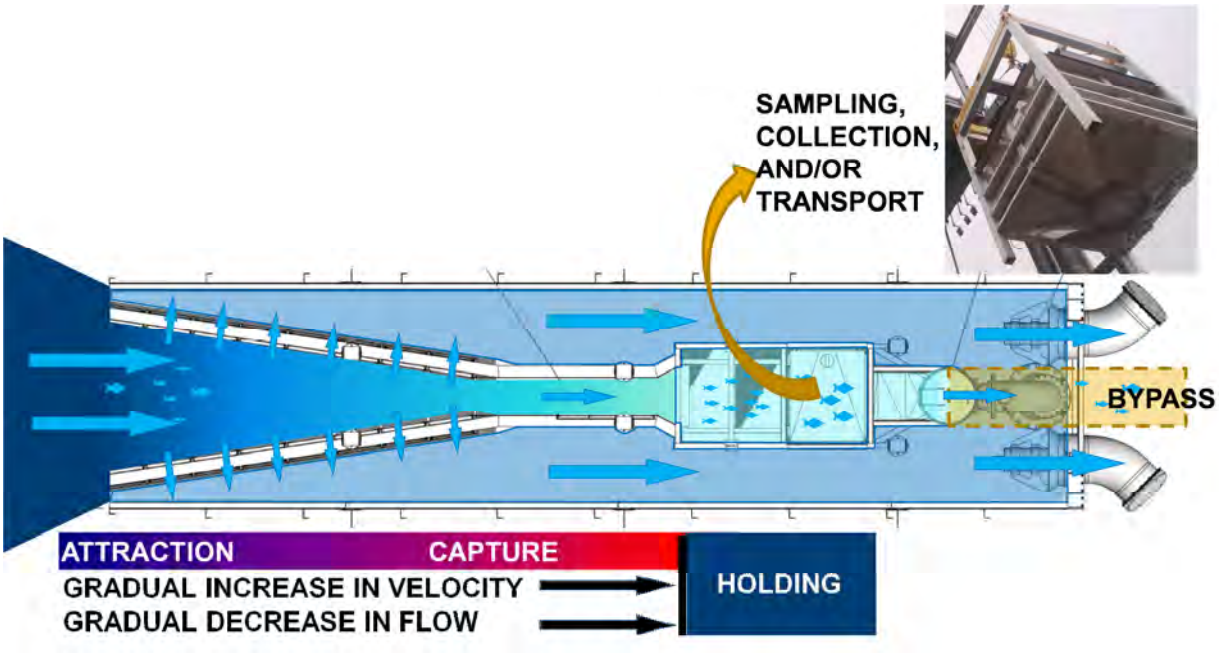
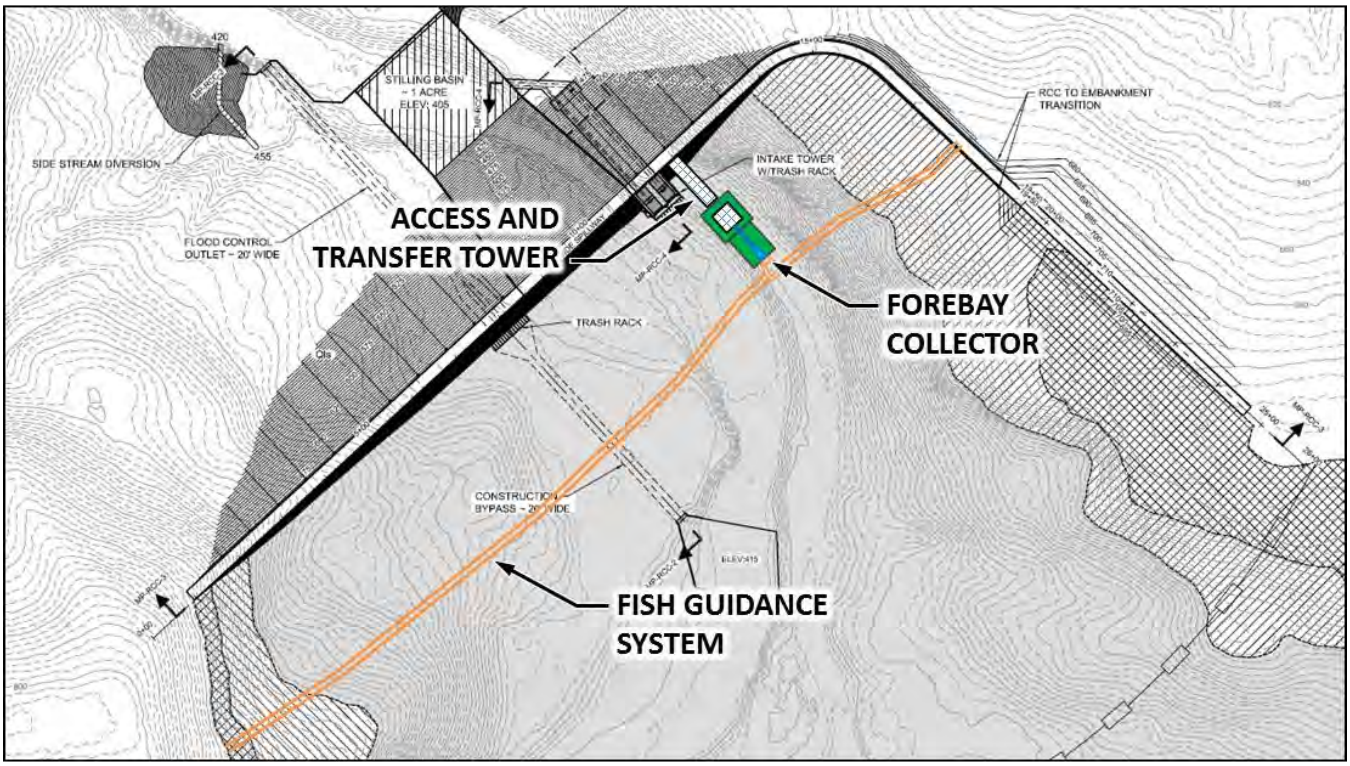


Figure 4-12
Conceptual Layout of Floating Forebay Collector



The primary advantage to FSCs is that they are designed to operate over a large range of reservoir pool elevations and conditions. This large operation range is possible because the collector rises and falls with the fluctuation of the forebay. In addition, an FSC provides placement flexibility which could maximize capture efficiency based on flow and fish migration patterns within the reservoir. Another benefit is if this alternative is paired with a CHTR facility for upstream passage, then fish that are already being transported upstream via trucks; the same trucks could also be used to transport the collected out-migrating juveniles to a release point downstream of the dam or at the upstream passage facilities of the CHTR.

Disadvantages to FSCs include their high capital and operation cost, complex system design, and power requirements. They work well in relatively still pool conditions where debris can be redirected. Systems such as this are not designed for and do not perform well in natural river-like conditions.

4.2.1.1 Debris Management System

One of the most important operation and maintenance activities that affect the performance of all FSCs is debris management. Debris management will need to occur at several positions ahead of the FSC to reduce the amount of large and small debris that interferes with water flowing into the collector inlet. The most important feature in redirecting large debris out from in front of the collector is the floating debris boom. Floating debris booms are composed of a series of plastic, steel, or wood constructed boom segments connected via lengths of chain or wire rope. Each boom system can possess two or more points of anchorage to maintain a specified angle and shape under loaded conditions. When configured appropriately, floating debris can be captured or redirected in a desired direction. An example floating debris boom is shown in Figure 4-13**Error! Reference source not found.**.

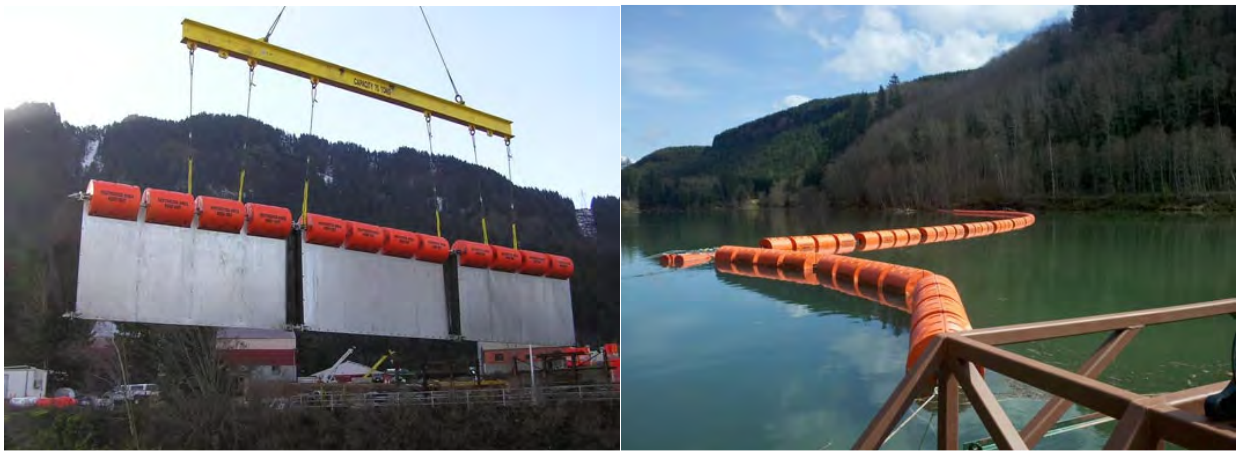
Figure 4-13
Example Debris Boom Used to Redirect Debris Away from a Surface Collector Entrance

Note: Courtesy of Worthington Products.

4.2.1.2 Physical Fish Guidance System

Physical FGSs have been proven to improve fish collection efficiencies significantly and are typically a component of all modern fixed and floating fish collection systems. Physical FGSs can be composed of solid panels and/or netting suspended from a series of floating booms. The boom systems are usually configured in a large "V" type configuration to funnel fish towards the collector inlet. In dam forebay applications, FGSs can simply be placed parallel to the dam to minimize fish being bypassed into hydropower inlets. An example FGS installation is provided in Figure 4-14.

Figure 4-14
Photographs Showing FGS Panels (Left) and Floating Boom Installation (Right)



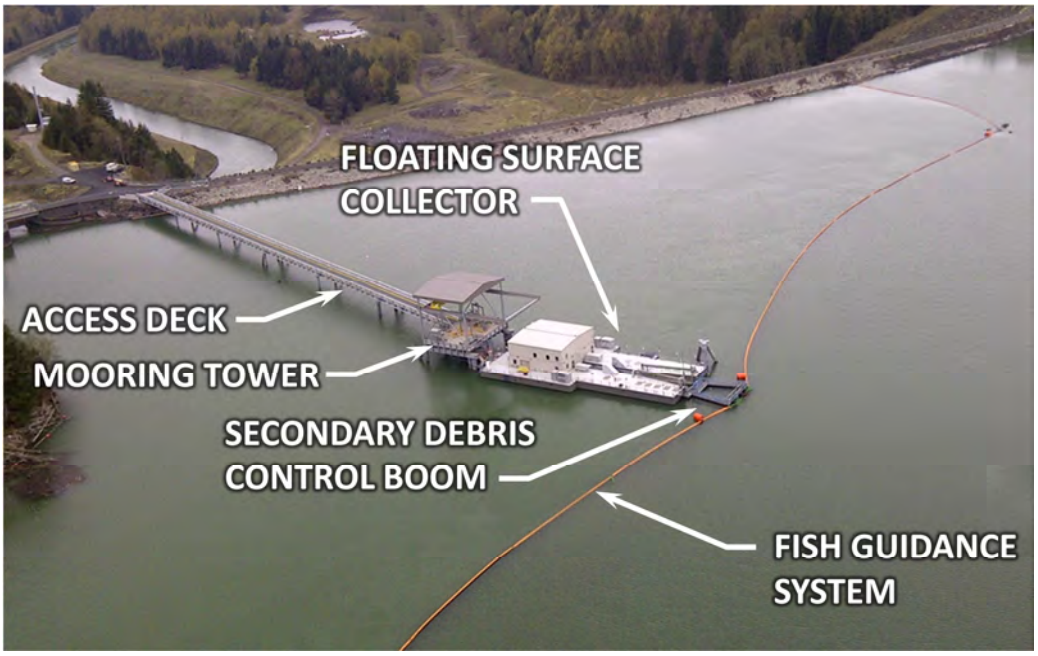
Note: Courtesy of Worthington Products.

4.2.1.3 FSC and Fish Collection Module

The most recent example of a modern and currently operating FSC at the time this document was prepared is the Swift Dam FSC on the North Fork Lewis River, Washington (see Figure 4-15**Error! Reference source not found.**). The application of this facility is similar to proposed multi-purpose dam on the Chehalis River in the following ways:

- Multiple target fish species
- Potential for large changes in pool elevations
- Requirement for high level of debris management
- Requirement for FGS to prevent fish from bypassing the collector

Figure 4-15
Swift Dam Floating Surface Collector, North Fork Lewis River, Oregon



Source: PacifiCorp 2012

Another very similar application is the Portable Floating Fish Collector (PFFC) which is currently being commissioned by USACE at Cougar Reservoir on the McKenzie River, Oregon. This is a unique facility designed to be self-mooring so that its position can be changed within the reservoir as required to effectively collect fish. The PFFC also uses an array of pumps to generate attraction flow up to a flow rate of 100 cfs. Power is provided from on-shore service and is provided to the floating platform via a shielded sub-marine cable and an automatic cable spooler which is located on the deck of the PFFC. For the purposes of this document, it is assumed that the application at the proposed project would incorporate many aspects of both of these facilities.

As alluded to earlier, FSCs are composed of a number of systems that will not be explained in high detail as part of this document. In general, the concept of a FSC for the multi-purpose dam and reservoir alternative could include the following primary components:

- **Access Facilities** – Facilities would need to be improved to facilitate daily access by operations personnel as well as fish transfer vehicles. Daily access may not necessarily be the same access point that would

be desired for construction but would consist of an easily accessible route with shortest travel time from the proposed dam. Floating walkway and dock facilities may be required for service boat moorage.

- **Anchorage** – Similar to the Cougar PFFC, it may be desired to incorporate a self-mooring anchoring system. The anchoring system would incorporate three to five anchors, automatic pay-out, and self-tensioning winches, and supporting equipment to accommodate wire rope layout and manage directional changes on-board the floating barge.
- **Floatation** – Both custom and/or "off the shelf" floatation systems could be considered to create the floating platform. The floatation system would require ballast tanks and water pumping systems so that the notional water line can be adjusted up or down to accommodate different depths or hydraulic conditions within the collector inlet. The final configuration would likely be developed in further design tasks which are beyond the scope of this preliminary assessment.
- **Collector Inlet** – A large funnel is typically formed at the bow of the collector which guides fish downstream while gradually increasing flow velocities. The size of the collector inlet is often dictated by the total collector maximum design flow and swimming behavior of the fish species being targeted as well as target capture velocities required just upstream of the next transport channel.
- **Water Management and Removal Systems** – As water and fish are drawn into the collector, dewatering screens, porosity control panels, and systems of weir gates are used to incrementally decrease the amount of water handled in the collection module while safely guiding fish further into the collector towards the holding facility. Water that is drawn out of the collector inlet through dewatering screens bypasses the collection module and is conveyed out through the attraction pumps. Dewatering can occur in the collector inlet while velocities are accelerating and/or in a transport channel after velocity deceleration occurs. Water that remains in the transport channel is conveyed downstream to a holding area. Eventually, the water is removed from the system and discharged to the reservoir using one or more screened leveling pumps.
- **Fish Holding and Sampling Facilities** – Upon collection of fish, there are a number of potential processes that could take place dependent upon the various objectives of the facility. Two basic objectives could be: 1) hold fish in a live well until they are transferred to a transport vehicle, and 2) facilitate on-board sampling activities to monitor biometric factors such as species, fork length, weight, length, genetics, etc. To accomplish these objectives, channels incorporating systems of graders, separation panels, crowder panels, braille equipment, and hoppers could be incorporated to move, sort, and handle fish as required. On the deck of the FSC platform, tables, tanks, monitoring equipment such as pit tag readers, and small auxiliary water supply lines could also be incorporated to facilitate monitoring and data collection objectives.
- **Fish Attraction Pumps –** Fish attraction is generated through the use of an array of low-head, high-volume, and low-noise fish attraction pumps. Fish attraction requirements vary widely but can be based upon two primary factors: 1) knowledge of fish behavior at a specific reservoir location, and 2) minimum flow requirements required to achieve the design capture velocity. This range of flows can be as low as 72 cfs and as high as 1,000 cfs based on available case studies. For the purposes of this report, a lower flow requirement of 100 to 200 cfs was selected which is similar to that of the Cougar PFFC.
- **Electrical Instrumentation and Control Equipment –** A variety of electrical supply, instrumentation, and control equipment would be required to operate all on-board systems. Although not specific in nature at the conceptual level, costs for such equipment should be accounted for in construction cost development.
- **Shore-Based Electrical Supply Systems –** For this concept, it is assumed that primary electrical power would be provided through a shore-based supply system. Service power will originate from a source to be identified in the future and would be brought to a local electrical panel. From the local electrical panel, power to the FSC would be provided via a shielded sub-marine cable to a spooler located on the FSC deck.

4.2.1.4 Fish Transfer System

Upon collection, fish will need to be transferred from the FSC to a transport vehicle so that they can be moved downstream and released below the proposed dam. The preferred method for transfer is one that does not require direct handling of fish but rather a method that includes a water-to-water type transfer. In water-to-water transfers, fish are never taken out of the water with a net or braille. One option would be to use specially equipped fish pumps to transfer fish from one vessel to another. Fish pumps are used regularly at various types of fish facilities including other fish collection sites, hatcheries, and commercial aquaculture facilities.

For this concept, fish could be crowded to a transfer tank where they could be pumped from the FSC to a live well, located on a service barge or directly onto an equipped service boat. The service vessel could then be navigated to the shore of the reservoir. Here, fish would be pumped directly into a transfer vehicle for transport downstream.

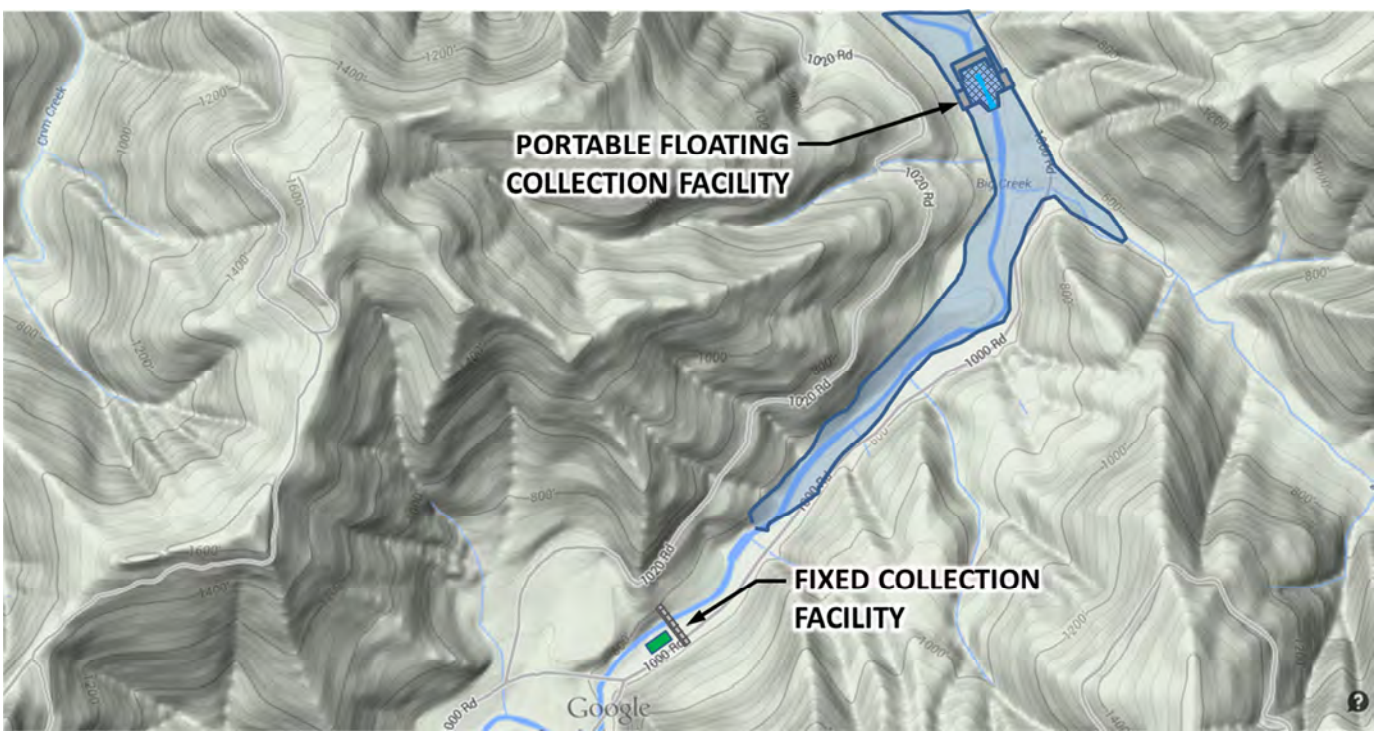
4.2.1.5 Fish Transport Vehicle

If an upstream fish passage facility also utilizes fish transport vehicles, the same vehicle could also be used to transport the collected downstream migrating juveniles to a release point downstream of the proposed dam. The type of vessel and considerations associated with fish transport are discussed in earlier sections of this document.

4.2.2 FLOATING COLLECTOR NEAR HEAD OF RESERVOIR COMBINED WITH AN ABOVE RESERVOIR COLLECTION FACILITY

The fish passage system selected to represent a combination of facilities that can provide collection and downstream passage of juvenile fish at and near the head of the reservoir includes a combination of a portable floating collection facility in the head of the reservoir and a fixed collection facility operating in the river channel just upstream of the normal operating reservoir elevation. These different components are described below and a conceptual-level schematic of the facilities and their respective locations are shown in Figure 4-16.

Figure 4-146
Head of Reservoir Combination Collection Facilities



4.2.2.1 Portable Floating Collection Facility

The portable floating collection facility is very similar to the floating forebay collector previously described except that the location is at the head of reservoir and different access considerations are needed. This concept includes an FSC that would be the most similar to the Cougar PFFC in that it could be more easily repositioned using the self-contained mooring system as the reservoir water surface fluctuates and moves upstream and downstream. This may also provide the advantage of moving the facility to a harbor area during floods that may bring large amounts of debris into the reservoir. However, flexibility of debris management and fish guidance devices will be an important consideration when operating this facility in response to changing flow and reservoir conditions. The other components of this facility include a fish transfer system and transport vehicle. Refer to the descriptions provided in the previous section for these components. Operational and other design considerations would be developed as the project progresses and alternatives are selected.

4.2.2.2 Fixed Collection Facility

The fixed collection facility located in the river above the head of the reservoir could consist of a small diversion dam to divert a portion of the flow with most of the downstream migrating fish to an off-channel screening and collection facility. The small, low-head diversion dam provides gravity-driven flow and some guidance of fish toward an intake positioned on the side of the river channel that is then screened down to better collect and handle fish for transporting. A typical irrigation diversion with fish screens and bypasses found on many rivers is the most analogous to this concept, except that all diverted flow would return to the river channel downstream of the diversion dam. Also, instead of bypassing fish at the downstream end of the screen back to the river channel, they would be collected into a holding facility that is very similar to the one described for an FSC. The other components of this facility include a fish transfer system and transport vehicle. Refer to the descriptions provided in the previous section for these components. Collected downstream migrants would be transported downstream via trucks and released at the tailwater of the dam or another suitable location.

A primary benefit to this option is the exclusion of downstream migrants from the reservoir and those fish needing to transit the reservoir to reach other facilities. However, fixed downstream collection facilities often lose collection efficiency at higher flows when the diversion dam is overtopped, which would result in fish entering the reservoir. Another disadvantage of this concept is its susceptibility to flood flows and damage depending on local siting conditions and trade-offs associated with obtaining high collection efficiencies. Additionally, upstream passage of fish at the diversion dam will be an important requirement and design consideration. Operational and other design considerations would be developed as the project progresses, and alternatives are selected.

4.3 FLOOD RETENTION ONLY FISH PASSAGE ALTERNATIVE

The following subsections provide a more detailed narrative of the flood retention only fish passage alternatives, including descriptions of major design components and considerations.

4.3.1 TUNNEL THROUGH DAM FOR BOTH UPSTREAM AND DOWNSTREAM PASSAGE

Fish passage conduits through the base of the flood retention only structure was rated far above all other flood retention fish passage options. Because the dam design team believes it is also technically feasible, it is the only alternative currently being considered as the primary mode of upstream fish passage for further evaluation.

The fish passage tunnels would allow the river to freely pass under the dam and have hydraulics conducive to fish passage. This constrains this option to only being applicable to a flood retention structure that only impounds water during higher (flood) flows. No operating pool would exist, as a reservoir would only form when the tunnels are shut off at some agreed upon flood flow trigger. During the majority of river flows, the

tunnel would operate in an open channel condition up to the high fish passage design flow, approximately 2,000 cfs.

The concept design currently consists of nine fish passage conduits, each 9 feet tall by 12 feet wide. Baffles or other roughness features will likely be implemented to slow velocities and provide fish resting locations within the conduits. In addition, the tailwater control below the dam will be set to backwater the tunnels to provide adequate depths and velocities during low flows. Further details of the fish passage conduits can be found in Appendix A-1.

4.3.2 CHTR FACILITY AT TEMPORARY BARRIER DAM

A CHTR facility is one of the potential options being considered to supplement fish passage during prolonged flood retention periods. This option includes the necessary system components described previously in Section 4.1.1. Facility operations would be performed on an as-needed basis when prolonged tunnel closures and debris management operations are anticipated to impact the upstream migration of target fish species. Essentially, this facility would remain idle unless flood retention operations are anticipated, at which time, the facility would be brought into an operational status.

CHTR facilities typically include a channel-spanning passage barrier which guides fish towards a fishway entrance. Implementation of a permanent barrier would likely cause long term passage issues when not in use. When coupled with the Flood Retention Only Dam Structure, this CHTR Facility option would include a series of temporary picket panels mounted on a permanent frame which could be mechanically adjusted to the desired height on an as-needed basis. During periods of non-migration or non-flood prone periods, the panels and frame can be removed from the river and placed in a storage area to preserve material and structural longevity of the picket weir components.

Combined Dam and Fish Passage Alternatives

5 Combined Dam and Fish Passage Alternatives

Four combined dam and fish passage alternatives were formulated from the technically feasible "baseline" dam configurations developed as part of the dam design study summarized in Chapter 3, and the practical fish passage options recommended from the Fish Passage Study and summarized in Chapter 4. The four selected combined structure alternatives are summarized in Table 5-1. Details related to each of these combined alternatives are presented in the subsections of this chapter. Conceptual level design drawings for each alternative are presented in Appendix B.

Table 5-1
Combined "Baseline" Dam and Fish Passage Alternatives Summary

ALTERNATIVE	DAM OPTION	UPSTREAM FISH PASSAGE OPTION	DOWNSTREAM FISH PASSAGE OPTION
Alternative A	Flood Retention RCC Dam	Run of the river tunnels at the base of the dam with CHTR facility	Run of the river tunnels at the base of the dam
Alternative B	Multi-purpose RCC Dam	CHTR facility	Combination collector facility
Alternative C	Multi-purpose RCC Dam	Conventional fishway with an experimental fishway exit structure	Forebay collector facilities
Alternative D	Multi-purpose Rockfill Dam	Conventional fishway and exit structure	Forebay collector facilities

Notes: CHTR = collect, handle, transfer, and release RCC = roller compacted concrete

As noted in the introduction of Chapter 3, following the development of the Alternatives in Table 5-1, two additional scenarios were recently introduced to the study to account for a range of possible climate change conditions in the basin. Based on the dam study team's preliminary evaluation of the three dam types described in Chapter 3, the flood retention only RCC dam configuration was selected as the basis for the climate change scenario evaluation including the development of cost estimates. Hence, the three original dam configurations used for development of the combined alternatives in Table 5-1 are relative to the "baseline" configuration requirements of the project. The two additional climate change scenarios are referred to as the "climate change" configuration requirements and are sub-alternatives of Alternative A.

5.1 ALTERNATIVE A – FLOOD RETENTION ROLLER COMPACTED CONCRETE DAM

Alternative A consists of the following dam and fish passage components:

- **Dam:** Roller compacted concrete flood retention only structure (Section 3.1 and Appendix A-1)
- **Upstream fish passage:** Nine fish passage conduits, each measuring 9 feet tall by 12 feet wide. One CHTR facility located downstream of the dam structure with temporary picket weir.
- **Downstream fish passage:** Utilizes the same nine fish passage conduits.

5.1.1 FLOOD RETENTION ONLY STRUCTURE - CLIMATE CHANGE IMPACTS

The purpose of these sub-alternatives to Alternative A is to describe the potential cost impacts related to the construction of higher dams to account for potential climate change impacts. Although the dam would be higher as noted in the introduction above, the configuration of the fish passage conduits through the base of the dam along with the configuration of the flood control outlet and emergency spillway remained unchanged.

Hydraulic analyses for the appurtenant flood control outlet works, emergency spillway, and fish passage structures have not been completed for these two climate change sub-alternatives. Hence the configuration of the hydraulic structures for each climate change scenario is the same as developed for the "baseline" configuration. Likewise, conceptual level drawings have not been prepared for these two sub-alternatives. Quantity estimates were extrapolated from the quantity estimates prepared for Alternatives A and B for the purpose of providing a total cost estimate for these two sub-alternatives. The cost estimates for these two sub-alternatives are presented in Chapter 7.

5.1.2 FLOOD RETENTION ONLY STRUCTURE – FISH PASSAGE CONSIDERATIONS

As the dam operations and debris management strategies were further formulated, it became apparent that longer flood retention durations are required to accommodate debris management activities. A detailed description of the anticipated debris management activities is presented in Appendix E titled Reservoir Vegetation and Debris Management, and Related Operational Considerations. An additional summary of the modeling and fish passage delay times anticipated for the flood retention only dam structure are provided in a Memorandum titled Revised Operations for Flood Retention Only Dam (Anchor QEA 2014) as well as in Appendix F – Revised Assessment of Fish Passage and Anticipated Operations of Proposed Dam Structure Alternatives (HDR 2014c). Conclusions from these supplemental assessments indicate that the fish passage tunnels would be closed to provide flood retention nine times during the 24 years of modeling data evaluated. Of the nine closures, seven would have triggered a prolonged retention period greater than 10 days to accommodate debris management activities. Of the seven prolonged retention events identified, durations ranged from 10 to 26 days. Pursuant to the State of Washington's fish passage laws and guidelines, it was determined by WDFW that it would therefore be necessary to include a redundant form of fish passage to be operated during most of the anticipated retention events. To accomplish this objective, a CHTR facility is combined with the flood retention only dam structure to achieve the required fish passage objectives as part of this alternative.

A CHTR facility would be included downstream to accommodate upstream fish passage of target species during prolonged periods of tunnel closure. The CHTR facility would be identical to the facility described in Section 4.1.1 of this report with the exception that the permanent barrier dam would be replaced with a temporary picket weir to be installed and used on an as-needed basis.

5.2 ALTERNATIVE B – MULTI-PURPOSE RCC DAM WITH CHTR AND COMBINATION COLLECTOR FACILITIES

Alternative B consists of the following dam and fish passage components. Conceptual drawings and details are provided in Appendix B-2.

- **Dam:** Roller compacted concrete multi-purpose structure (Section 3.2 and Appendix A-2)
- **Upstream fish passage:** Migration barrier and CHTR (collect, hold, transport, and release) facility
- **Downstream fish passage:** Includes a combination of facilities at or near the head of the reservoir—a portable floating collection facility in the head of the reservoir and a fixed collection facility operating in the river channel just upstream of the normal operating reservoir elevation

A CHTR facility in combination with a physical migration barrier will provide upstream fish passage. Conceptual design and operation of the CHTR facility are described in Section 4.1.1 of this TM. The CHTR facility would be located on the right side of river. A small fish ladder was added between the downstream barrier and CHTR facility to keep the structure out of the floodplain. A portable floating collection facility located near the head of the reservoir will be used in combination with a fix collection facility operating upstream of the reservoir to provide downstream fish passage.

5.3 ALTERNATIVE C – MULTI-PURPOSE RCC DAM WITH EXPERIMENTAL FISHWAY AND FOREBAY COLLECTOR

Alternative C consists of the following dam and fish passage components. Conceptual drawings and details are provided in Appendix B-3.

- **Dam:** Roller compacted concrete multi-purpose structure (Section 3.2 and Appendix A-2)
- **Upstream fish passage:** Below dam barrier and conventional fish ladder with experimental fishway exit tower
- **Downstream fish passage:** Floating forebay collector at the dam

The 2,225-foot-long fish ladder is located on the right side of the river (looking downstream). The fishway generally follows the existing topography and extends from elevation 398 feet at the river intake to elevation 548 feet at the toe of the dam. The best location for the fishway exit structure was found to be at the upstream face of the dam, immediately to the right of the water quality outlet works intake tower. However, there are some challenges with the selected location, because the fish exiting the structure could potentially be pulled back into the water quality outlet works intake tower. This potential conflict would need to be investigated further and mitigated if the two structures are to be near each other. A floating forebay collector will be installed in front of the water quality outlet works intake tower.

5.4 ALTERNATIVE D – MULTI-PURPOSE ROCKFILL DAM WITH CONVENTIONAL FISHWAY AND FOREBAY COLLECTOR

Alternative D consists of the following dam and fish passage components. Conceptual drawings and details are provided in Appendix B-4.

- **Dam:** Central clay core rockfill dam (Section 3.3 and Appendix A-3)
- **Upstream fish passage:** Below dam barrier and conventional fish ladder and fishway exit structure
- **Downstream fish passage:** Floating forebay collector at the dam

The fish ladder switchback is located on the left side of the river (looking downstream). The 3,730-foot fishway extends from elevation 415 feet at the inlet pool to elevation 548 at the downstream toe of the dam, where it transitions to 526-foot conduit through embankment foundation. The advantages of placing the fishway on the left side include a shorter tunnel under the dam, proximity of the ladder inlet pool to the attractant flow, and an advantageous location for the fish way exit structure at a natural knoll on the left abutment. There are potential conflicts with the water quality outlet works intake structures and tunnels that will need further investigation to minimize the potential for pulling fish back downstream through the water quality outlet works.

The floating forebay collector would likely be attached to the fishway exit structure. This would allow both facilities to share an access road and potentially allow fish migrating downstream to be directed into the fishway after being collected in the floating forebay collector.

5.5 MULTI-PURPOSE DAM OPTIONAL FUTURE HYDROPOWER

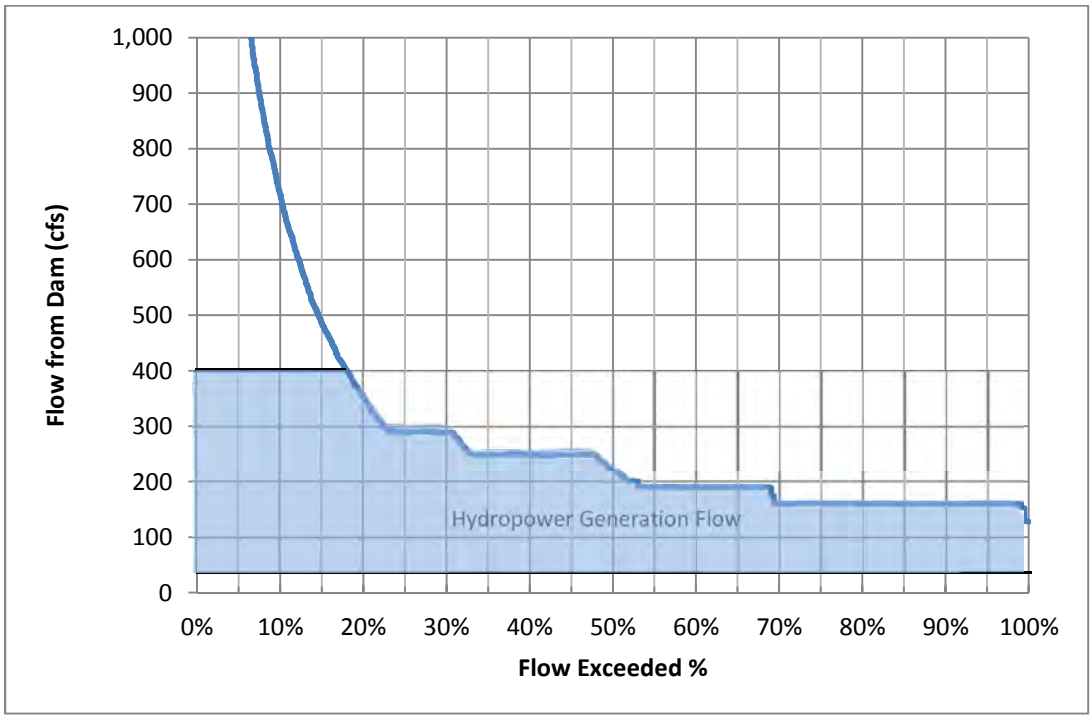
This section summarizes a planning level investigation of the potential for adding hydropower to a multi-purpose project alternative in the future.

5.5.1 HYDROPOWER FLOW CRITERIA

Twenty-four years of recorded daily flows at the Doty stream gaging station downstream of the dam site were proportioned to the basin size above the dam and then used to model multi-purpose reservoir storage elevations and flow releases for hydropower generation. These daily reservoir elevations and flow releases were used to develop a flow duration curve and spreadsheet model of simulated potential energy generation. The simulated multi-purpose dam flow duration curve is shown on Figure 5-1.

Based on the results summarized on Figure 5-1, a 400 cfs flow exceedance point on the flow duration curve was selected as the design flow for identifying the size and number of turbines at the site. It was also assumed that a base fisheries bypass flow of 25 cfs would not be available for hydropower generation.

Figure 5-1
Hydropower Flow Duration Curve



Note: cfs = cubic feet per second

5.5.2 HYDROPOWER GENERATION CONFIGURATION

The hydropower facility would be located below the dam and connected to a penstock from the dam water quality outlet works system. Although hydropower would not be installed with the initial dam project development, the configuration of the water quality outlet works and emergency spillway stilling basin was established to allow for addition of a future hydropower facility. The outlet penstock would be designed with a blind-flanged bifurcation to facilitate the future installation of hydropower facilities.

Based upon the simulated daily reservoir elevations and flows, Table 5-2 lists the criteria that were used for determining the energy benefits of an optional future hydropower turbine and generator installation.

Table 5-2
Hydropower Requirements

TURBINE/GENERATOR:	Single 4.6 MW Francis turbine
HYDRAULIC CAPACITY:	375 cfs
DESIGN HEAD:	160 feet
GENERATION HEAD RANGE:	110 feet to 190 feet
POWERHOUSE SIZE:	Approximately 30 feet by 50 feet

Note: cfs = cubic feet per second

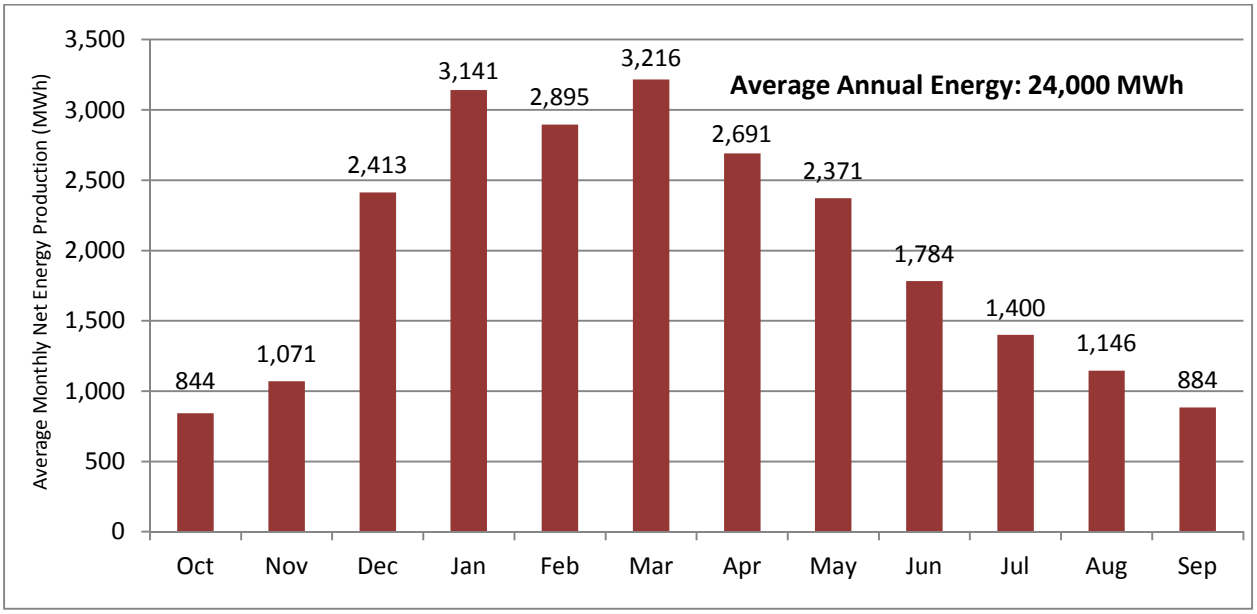
The turbine generator could either be in a vertical or horizontal shaft configuration. Additional unit optimization and sizing would be required during a future hydropower system design. For example, it could be determined

that it would be cost effective to install two turbine/generator units to maintain higher efficiencies over a wider range of flows and varying reservoir head conditions. Approximately 7,800 feet of new power line along the dam access road could connect the powerhouse to an existing substation in south Pe Ell.

5.5.3 ESTIMATED HYDROPOWER GENERATION

Using the 24 years of simulated flows and reservoir elevations along with the above hydropower design criteria, the simulated average monthly energy generation is shown on Figure 5-2. The total average annual energy generation would be approximately 24,000 MWh.

Figure 5-2
Average Monthly Hydropower Energy Generation



Operations and Maintenance

6 Operations and Maintenance

In 2014, Shannon & Wilson collected and evaluated information on the operations and maintenance requirements at existing dams owned and operated by USACE Seattle and Portland Districts. The purpose of this study was to develop an understanding of vegetation, debris, and sediment management best practices as well as general operational issues at similar dams in the region. In addition to data collection the Howard A Hanson and Mud Mountain dam site were visited and USACE personnel were interviewed to document the debris and sediment management facilities at each project and the procedures followed by operations staff during and following major events and generate significant debris and sediment flow into the reservoirs. A Technical Memorandum summarizing this evaluation is presented in Appendix E.

An additional study was also completed by Shannon & Wilson to evaluate the location(s) and geometry of landslides within the proposed Chehalis Dam and Reservoir footprint area. This study involved evaluation of aerial photographs, GIS data, and a site reconnaissance visit. This TM is presented in Appendix C.

6.1 DAM OPERATIONS AND MAINTENANCE

The probable operation and maintenance requirements of the proposed Chehalis Dam and Reservoir can only be approximated based on comparable dam facilities in the region. At the Howard A. Hanson and Mud Mountain Dams operations and maintenance activities include removal of debris and sediment carried into the reservoir during flood events, control of discharge of both water and sediment to downstream river reaches during and after flood events, cleaning and maintenance of outlet and gate structures, monitoring of reservoir and dam abutment slope stability, and control of vegetation growing in and near the reservoir inundation area. Based on the requirements at the Howard A. Hanson and Mud Mountain Dams up to eight full-time operations and maintenance personnel and an annual budget ranging from $3.5 to 3.8 million may be required for the flood control and multi-purpose dam configurations developed by the dam study team.

6.2 VEGETATION MANAGEMENT

Management of vegetation for the proposed Chehalis Dam site will vary significantly based on whether the flood control or multi-use option is selected. Vegetation removal including both clearing and grubbing for the flood control only RCC option will likely be limited to the construction area including the dam footprint, quarry areas, staging area, and within the right-of-ways required for temporary and permanent access roads. Some vegetation removal in the lower portions of the reservoir basin may also be prudent depending on future agreements with the landowner. Vegetation removal for the multi-purpose dam alternative would include vegetation removal for the much larger construction footprint including borrow areas in the reservoir basin as well as the area below the normal operating pool level. Removal of timber outside of construction areas would likely be limited to marketable timber within the inundation zone(s) for the two options. Vegetation not related to the construction and borrow area clearing and grubbing or harvesting of marketable timber will be left in place to reduce erosion in the reservoir and sedimentation near the dam. Long-term management of vegetation will involve allowing vegetation growth to self-regulate based on inundation frequency. As vegetation decomposes and is eroded by flood waters, it will be removed periodically in order to prevent accumulation of vegetation debris that could damage or affect the operation of fish passage and outlet structures.

6.3 SEDIMENT MANAGEMENT

Sediment accumulation and migration within the reservoir footprint will vary significantly between the flood control only and multi-purpose configurations. For the flood control only options, it is expected that coarser sediment will initially pass through the fish passage and flood control outlet works. However, as a flood management pool begins to develop, some coarse sediment will begin to accumulate in the reservoir poll area near the dam. As pool levels rise, sediment will drop out of suspension near the edge of the pool. Management of sediment accumulated in the reservoir basin including deposits near the dam would require either flushing of sediment through the low level openings of the fish passage or flood control outlet works, by sediment excavation and removal, or both. Allowing passage of sediment and small woody debris through the low level openings has the benefit of replenishment of sediment and debris in the river channel downstream of the dam but carries the risk of damage to the outlets structures, especially in the case of larger sized materials including small boulders, cobbles, and coarse gravels moving through the system. If larger cobble and boulder sized material is allowed to pass through the low level openings, partially open gates, or the tunnel/penstocks, then operation and maintenance budgets and schedules should be allocated for more frequent maintenance, and periodic repair of these structures. A combination of periodic removal of boulder and larger cobble sized material from the reservoir pool area upstream of the dam and allowing passage of gravel and smaller sediment through the low level outlet works and fish passage openings should be considered and the preferred method of sediment management.

For the Chehalis dam multi-purpose configurations, sediment accumulation immediately upstream of the dam would consist primarily of relatively small amounts of fine grained sand to clay sized particles. Coarser material would drop out of suspension when flow enters the upstream portion of the permanent reservoir pool. Deltas and alluvial fans will develop in these areas. During lower reservoir pool conditions these deltas and alluvial fans would be dissected by surface water flow that would transport some of the coarser material into lower reservoir pool areas. Excavation/construction of sediment traps in the upstream reservoir area to allow storage of sediment equal to a given number of years of expected sediment load would mitigate sediment management requirement. Recent experience with sediment management in large fire scar areas above major reservoirs and streams in Colorado and California indicate that such sediment management traps can be very effective and relatively low cost to construct. Relatively infrequent excavation and transport and/or disposal of sediment deposited in the aforementioned deltas, alluvial fans, or sediment trap locations for the purpose of stream bed sediment nourishment should be considered as the preferred method of sediment management for the multi-purpose project configurations.

6.4 DEBRIS MANAGEMENT AND REMOVAL

Debris management and removal is a critical aspect of operations and maintenance for either the flood control option or the multi-purpose configurations. Failure to control debris, especially large woody material (LWM) has the potential to reduce the effectiveness of both fish passage and outlet works structures. LWM is successfully managed at other USACE sites including Mud Mountain through the use of log booms that hold the LWM in the reservoir until it can be removed from the reservoir. Experience at Howard A. Hanson and Mud Mountain dams, indicates that local stakeholders should be allowed to make use of the LWM as desired.

Allowing some of the smaller portions of woody debris to pass through the low level outlets should be considered in order to enhance downstream habitat. LWM, if desired for downstream habitat enhancement, should be transported around the dam and not allowed to pass through the outlet works or spillway systems.

Operations and maintenance budgets should be allocated for management of general and LWM for each flood event. This includes allowing some general debris to pass downstream or hauling and transporting LWM out of reservoir. Budgets should include labor and equipment to perform LWM removal operations including multiple log booms at varying reservoir locations, tug boats to move and pull LWM to designated debris removal locations along the perimeter of the reservoir, equipment to load and haul the LWM out of the reservoir basin, and annual evaluation of LWM management plans.

Infrequent extended duration flood events are likely to deliver a higher volume of LWM to the reservoir than frequent, lower volume, shorter duration flood events. Therefore, following large flood events, the reservoir will likely need to be lowered more slowly or delayed by days to weeks to allow for LWM to be collected and removed. This slowed or delayed reservoir draw down increases fish passage delays. The analysis presented in Appendix G provides anticipated fish passage delays due to flood and debris management operations. It is estimated that fish passage would be blocked 1.5 percent of the time, on an annual basis, due to flood and debris management operations. Over a 24 year period of record flood and debris operations occurred nine times, with fish passage delays ranging from 0.5 to 26 consecutive days for each delay event.

Due to the potential of fish passage delays of up to 26 consecutive days for the flood retention only dam alternative, redundant passage is appropriate. A CHTR facility is the most applicable. The CHTR facility would be operated during period of flood and debris management operations.

6.5 RESERVOIR SLOPE STABILITY

The construction and operation of both the flood control only and multi-purpose projects may result in increased slope instability within the dam construction area, and within the reservoir pool. Potential construction-induced instability, particularly in the vicinity of the dam would be effectively treated through proper slope stabilization and excavation work activities. The conceptual level dam designs described in Chapter 3 have acknowledged this issue and the need to further evaluated slope stabilization requirements during subsequent design and cost estimating activities.

Reservoir rim slope stability will be a significant concern when slopes become saturated and the levels of storage in the reservoir change significantly over relatively short periods of time. This concern will be heightened at locations of existing landslides that are partially or fully within the limits of reservoir pool inundation. Stable slopes that show no evidence of previous movement or deformation as well as slopes within identified landslide areas may experience increased slope instability and small to very large deformations due to saturation and rapid unloading as reservoir levels drop during flood management activities.

Slopes in areas of existing landslides, that are steep, or that have had vegetation removed will be the most susceptible to slope failures. Shallow seated slope failures (3 to 6 feet deep) will be the most frequent and should be anticipated. In addition to shallow seated slope failures, Appendix D has identified a number of more deeply seated landslide masses that will have a portion of the toe of the landslide mass saturated as a result of flood or permanent pool reservoir elevations. Figure 2 in Appendix D shows the location and extent of these landslides and the portion of the estimated landslide mass that will be inundated by reservoir levels ranging from El. 660 and El. 740. The estimated quantity of each landslide is also presented in Appendix D. The estimated volumes of these landslides range from 160,000 cubic yards to 2.2 million cubic yards. As the project progresses to further design phases these landslides should be studied in greater detail in order to better analyze the effect(s) of saturation due to reservoir elevations. Potential slope stabilization remedial measures should be evaluated.

Slope stability on the upstream and downstream portions of the dam abutments could also present long term dam safety and operation issues. Abutment slopes including the five known landslides shown on the drawings in Appendix A should be thoroughly investigated during future design phases in order to accurately assess slope stability and design remedial measures to reach acceptable stability conditions for all reservoir operational conditions.

Slope failures of any size will provide sediment to the reservoir that can reduce reservoir capacity and potentially be transported to and through the outlet structures. Budget for annual inspections of reservoir slopes during operation should be allocated in order to assess slope stability issues. A contingency budget for removal or stabilization of material resulting from slope failures should be considered in the annual operation and maintenance budget estimate.

6.6 FISH PASSAGE

Each of the fish passage alternatives possesses a number of complex, specialized operations and maintenance activities that require frequent attention. General operations will require a number of daily activities to operate fish passage systems within the intended range of performance criteria and to meet mandated requirements set forth by State and Federal resource agencies. These mandated requirements are typically monitored and reported as specified in the various environmental, operational, and "take" permits. As complexity of each alternative increases the level of maintenance anticipated for each facility is expected to increase as well. For each proposed alternative there is a number of monitoring and control systems that are composed of specialized equipment which are operated both automatically and manually as needed. For alternatives that require fish handling and transport, there is a need for daily manual labor performed by multiple trained, specialized personnel. A general summary of potential activities associated with O&M of the proposed fish passage alternatives is provided in Table 6-1.

Of the fish passage alternatives proposed, Alternative B is anticipated to require the highest level of effort associated with O&M activities. This alternative combines fish collection and transport facilities to facilitate both upstream and downstream migration of a variety of fish species and age classes. These types of operations require a high level of specialized equipment, continuous oversight and operation, and daily handling of fish by trained personnel. Each day will require a number of activities to ensure that each of the three facilities is operating in a manner consistent with the performance criteria and safe for fish passing through the system.

Alternatives C and D require operation of a fishway coupled with the operation of a forebay collector. In either case, operation and maintenance of the fishway will require relatively low effort when compared to other options. The exception is that Alternative C incorporates a more complex fishway exit tower which increases the overall level of effort. However, the level of effort required for each fishway is still expected to be comparatively less than that of the forebay collectors. General operations of fishways require daily inspection, small-scale removal of debris from debris racks, and control of hydraulic conditions via adjustment of inlet and outlet gates. The majority of the annual maintenance may include inspections and general maintenance of control equipment such as gates and mechanical systems. Similar to the facilities proposed in Alternative B, forebay collectors require a high level of effort to operate and maintain. One of the major operational components of the forebay collector is the attraction flow and collection systems. The attraction flow systems require operation of an array of low head pumps which utilize significant amounts of electrical resources. The collection systems can be complex and can incorporate a number of automated and manual mechanical equipment to safely collect, segregate, and transfer fish to a fish bypass or transport vehicle.

Table 6-1
Summary of Potential Fish Passage Operations and Maintenance Activities

OPERATIONS AND MAINTENANCE ACTIVITY	FISH PASSAGE ALTERNATIVE			
	A	B	C	D
Daily Activities				
General inspection	X	X	X	X
Biological monitoring	X	X	X	X
Small scale debris removal	-	X	X	X
Flow control and adjustment	X	X	X	X
Verification of automated systems	-	X	X	X
Fish handling and transfer operations	-	X	X	X
Weekly and Monthly Activities				
Access road inspection	-	X	-	-
Transport vehicle maintenance	-	X	-	-
General maintenance of equipment	-	X	X	X
Documentation and reporting	X	X	X	X
Weekly and Monthly Activities				
Access road inspection	-	X	-	-
Transport vehicle maintenance	-	X	-	-
General maintenance of equipment	-	X	X	X
Documentation and reporting	X	X	X	X
Annual or Bi-Annual Activities				
General inspections and verifications	X	X	X	X
Cleaning, repairing, and replacing equipment and expendables	-	X	X	X
Access road maintenance	-	X	-	-
Lubrication, exercising, equipment tuning	-	X	X	X
Calibration and verification of gates, motors, transmitters	-	X	X	X
Verification and modification to SCADA and monitoring systems	-	X	X	X
Emergency Activities				
Major replacement of equipment	X	X	X	X
Flood proofing	-	X	-	-
Fish transfer and/or salvage	X	X	X	X
Debris and sediment removal	X	X	X	X
Shoring, erosion protection, and riverbed stabilization	X	X	X	X
Access repair	-	X	-	-

Note: SCADA = Supervisory Control and Data Acquisition

Alternative A is anticipated to require the least amount of O&M effort with respect to fish passage. Automated flow controls integrated into the dam and stilling basin itself are used to provide adequate hydraulic conditions through the fish passage tunnels to facilitate both upstream and downstream volitional passage of fish. The CHTR facility is anticipated to be operated on an infrequent as-needed basis when debris management operations prolong flood retention durations and impact fish passage.

6.7 OPTIONAL FUTURE HYDROPOWER

The conceptual designs of appurtenant structures associated with the multi-purpose dams (both RCC and rockfill options) include provisions for construction of a hydropower generation facility in the future. In order to construct this facility, a Federal Energy Regulatory Commission (FERC) hydropower generation license application and permit would be required. These permits require completion of appropriate environmental studies, as well as for the dam facility to meet FERC dam safety requirements. The FERC license would also require the submission of annual reports regarding the operation of the facility and other compliance requirements, including Part 12D dam safety inspections.

A future hydropower installation would add to the complexity of dam operation and maintenance including the need for additional operations and maintenance staff and budget. Although the facility would be largely automated, trained staff would be required to operate and maintain the hydropower facility on a year-round basis. In addition to personnel requirements, the hydropower facility would need an annual investment in maintenance, repairs, and replacement of equipment, and may require payment of additional taxes, insurance and licensing fees.

The facility could be remotely monitored and controlled via an on-line data acquisition and control system. However, typical operational requirements would also include daily visits to the facility to inspect the facility and verify that remote systems are correctly reflecting the actual operation of the systems. A trained hydropower operator could also serve as one of the required staff for operation and maintenance of the dam and other outlet works systems.

Opinions of Probable Costs

7 Opinions of Probable Costs

Estimates of probable costs (2014 basis) have been prepared for each of the dam and fish passage components used as part of the combined alternatives outline in Table 5-1. These component cost estimates have been combined into total planning level estimates for Alternatives A through D developed for the "baseline" project conditions, and for sub-alternatives A1 and A2 developed for two alternative "climate change" scenarios. This chapter presents a summary of the cost estimates for input to the overall project economic evaluations and selection of alternatives for further development during subsequent phases of the project.

Cost estimates for the dam alternatives were initially presented as part of a briefing to the Water Retention Committee in April 2014 following completion of the Dam Design TM. Subsequently, updates of some quantities, completion of a planning level aggregate and construction material availability evaluation, introduction of new climate change scenarios for the flood control only RCC dam, and further refinement of unit prices were completed and the overall estimates for probable costs updated and finalized. As noted in the sections below, cost estimates for the configurations and alternatives involving the RCC dams have been reduced to reflect primarily an adjustment in the estimated unit cost range. This adjustment reflects two of the following important considerations:

1. A reasonable likelihood that suitable RCC aggregate materials can be generated from borrow sources at the site or within a distance of less than 10 miles.
2. Further evaluation of a data base of unit costs for RCC dam projects completed throughout the United States. This evaluation has confirmed that projects involving large quantities of RCC similar to the quantities that would be required for the Chehalis project dam configurations have previously resulted in significant reductions in unit prices bid by qualified contractors.

As previously noted in Section 3.2.1, the climate change scenarios and corresponding flood control only sub-alternatives A1 and A2 presented in the sections below include consideration of the following:

- Scenario 1 – 18 percent increase in Chehalis River flow above the dam site
- Scenario 2 – 90 percent increase in Chehalis River flow above the dam site

7.2 METHODOLOGY, CONTINGENCIES, AND ASSUMPTIONS

The estimated of probable costs were develop to meet the guidelines for an AACE Class 4 estimate with an expected accuracy of -30 percent to +50 percent. As is common for this level of accuracy and at this stage of the planning process, a number of elements of the design are addressed through the application of planning stage "contingency factors." Those factors included in the estimates include the following:

- Items not developed in concept level design but that are known to be required. These items for the Chehalis Project dam configurations include temporary and/or permanent stream crossing bridges,

temporary and/or permanent access roads, landslide stabilization, and debris and sediment management provisions

- Site characterization including geologic studies, geophysical investigations, drilling, and laboratory testing programs; delineation of design criteria including seismic hazards, flood hydrology, topography and site survey controls
- Permitting
- Design
- Construction management and engineering support
- Construction change orders/claims

Uncertainty in the estimates of probable construction costs were considered by including potential ranges of 1) unit prices for key items of work and 2) for the contingency factors noted above. The lower and upper bounds for the unit prices and contingency factors used in the cost estimates are summarized in Table 7-1.

Table 7-1
Summary of Key Unit Price and Contingency Factor Ranges Included in Cost Estimates

Bid Item	Description	Unit	Lower Bound Unit Price	Upper Bound Unit Price
1	Clearing and grubbing, reservoir clearing, stripping topsoil, reclamation of disturbed areas	Acre	$29,560	$29,560
2	Fill - Coffer Dams	CY	$6.50	$6.50
3	Control Structures - Gates/Bulkheads Fab	LB	$12.00	$20.00
4	Regulating Intake Tower - Gates Fab	LB	$12.00	$20.00
5	Excavation - Foundation General	CY	$6.50	$6.50
6	Excavation - Foundation Rock	CY	$34.50	$34.50
7	Foundation Treatment - Grout Curtain	LF	$16.50	$16.50
8	Conventional Concrete Reinforced	CY	$700	$740
9	Conventional Concrete Non-Reinforced	CY	$400	$476
10	Dental Concrete	CY	$250	$325
11	Fill -Roller Compacted Concrete	CY	$65	$95
12	Embankment Fill - Foundation Backfill/Wingdam	CY	$4.50	$4.50
13	Fill - Central Clay Core (Zone 1)	CY	$6.50	$8.10
14	Fill - Filter Transition (Zone 2)	CY	$45.00	$53.80
15	Fill - Rockfill (Zone 3)	CY	$25.00	$34.50
16	Fill - Riprap (Zone 4)	CY	$28.00	$34.50
	Design Contingency		25%	30%
	Construction. CO/C Contingency		8%	8%
	Permitting		3%	6%
	Design and site Characterization		7%	9%
	Engineering Support During Construciton		9%	12%

Note: 1. The range of contingency costs included for each planning item in Table 7-1 was estimated by multiplying the total base construction cost (BCC) by the low and high percentages shown.

CY = cubic yard LB = pound LF = lineal foot

Items that are not included in the estimates of probable costs at this time include operation and maintenance, and state administration and taxes.

7.3 DAM STRUCTURE OPTIONS

7.3.1 FLOOD RETENTION ONLY RCC DAM

The estimated total cost for this dam and appurtenant structures configuration range from $217 million to $314 million with average of $266 million. A summary of these estimated costs is provided in Table 7-2.

Table 7-2
Summary Cost Estimate for FR-RCC Dam, "Baseline" Study Conditions

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
	Phase 1 - Prep Work						
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	25	Acre	$ 29,560.00	$ 29,560.00	$ 739,000	$ 739,000
2	Flood Control Outlet Tunnel 20 ft wide	1,420	LF	$ 7,670.00	$ 7,670.00	$ 10,891,400	$ 10,891,400
3	Excavation - General	112,000	CY	$ 6.50	$ 6.50	$ 728,000	$ 728,000
4	Control Structures - Reinforced Concrete	2,000	CY	$ 700.00	$ 740.00	$ 1,400,000	$ 1,480,000
5	Control Structures - Gates (Fab and Construct)	544,000	LB	$ 12.00	$ 20.00	$ 6,528,000	$ 10,880,000
6	Fill - Coffer Dams	26,000	CY	$ 6.50	$ 6.50	$ 169,000	$ 169,000
	Subtotal					$ 20,455,400	$ 24,887,400
	Phase 2 - Main Dam						
7	Excavation - Foundation General	458,519	CY	$ 6.50	$ 6.50	$ 2,980,374	$ 2,980,374
8	Excavation - Foundation Rock	111,488	CY	$ 34.50	$ 34.50	$ 3,846,336	$ 3,846,336
9	Fill - Roller Compacted Concrete	746,641	CY	$65.00	$95.00	$ 48,531,665	$ 70,930,895
10	Fill - Foundation Backfill	312,720	CY	$ 4.50	$ 4.50	$ 1,407,240	$ 1,407,240
11	Conventional Concrete Reinforced	25,000	CY	$ 700.00	$ 740.00	$ 17,500,000	$ 18,500,000
12	Conventional Concrete Non-Reinforced	75,000	CY	$ 400.00	$ 476.00	$ 30,000,000	$ 35,700,000
13	Foundation Treatment - Grout Curtain	15,000	LF	$ 16.50	$ 16.50	$ 247,500	$ 247,500
14	Fish Passage Gates - Radial (Fab and Construct)	450,000	LB	$ 12.00	$ 20.00	$ 5,400,000	$ 9,000,000
15	Fish Passage Gates - DS Vertical (Fab and Construct)	378,000	LB	$ 12.00	$ 20.00	$ 4,536,000	$ 7,560,000
	Subtotal					$ 114,449,115	$ 150,172,345
	Other						
16	Bulkheads	206,000	LB	$ 12.00	$ 20.00	$ 2,472,000	$ 4,120,000
	Subtotal					$ 2,472,000	$ 4,120,000
	Total Base Construction Cost (BCC)					$ 137,376,515	$ 179,179,745
	Design Contingency					$ 34,344,129	$ 53,753,923
	Construction. CO/C Contingency					$ 10,990,121	$ 14,334,380
	Subtotal					$ 182,710,764	$ 247,268,047
	Permitting					$ 5,481,323	$ 14,836,083
	Design and Site Characterization					$ 11,876,200	$ 22,254,124
	Engineering Support During Construction					$ 16,443,969	$ 29,672,166
	Total Cost (Rounded)					**$ 216,500,000**	**$ 314,000,000**

Notes: CY = cubic yard LB = pound LF = lineal foot

7.3.2 MULTI-PURPOSE ROLLER COMPACTED CONCRETE DAM

The estimated total cost for this dam and appurtenant structures configuration range from $276 million to $395 million with average of $336 million. A summary of these estimated costs is provided in Table 7-3.

Table 7-3
Summary Cost Estimate for MP-RCC Dam, "Baseline" Study Conditions

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
	Phase 1 - Prep Work						
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	28	Acre	$ 29,560.00	$ 29,560.00	$ 827,680	$ 827,680
2	Flood Control Outlet Cut/Cover	1,200	LF	$ 2,000.00	$ 2,400.00	$ 2,400,000	$ 2,880,000
3	Excavation - Outlet General	255,000	CY	$ 6.50	$ 6.50	$ 1,657,500	$ 1,657,500
4	Excavation - Outlet Rock	55,000	CY	$ 34.50	$ 34.50	$ 1,897,500	$ 1,897,500
5	Fill - Coffer Dams	26,000	CY	$ 6.50	$ 6.50	$ 169,000	$ 169,000
6	Control Structures - ReinforcedConcrete	5,000	CY	$ 700.00	$ 740.00	$ 3,500,000	$ 3,700,000
7	Control Structures - Gates	170,900	LB	$ 12.00	$ 20.00	$ 2,050,800	$ 3,418,000
8	Regulating Intake Tower - Reinforced Concrete	20,000	CY	$ 700.00	$ 740.00	$ 14,000,000	$ 14,800,000
9	Regulating Intake Tower - Gates	22,000	LB	$ 12.00	$ 20.00	$ 264,000	$ 440,000
	Subtotal					$ 26,766,480	$ 29,789,680
	Phase 2 - Main Dam						
10	Excavation - Foundation General	649,860	CY	$ 6.50	$ 6.50	$ 4,224,090	$ 4,224,090
11	Excavation - Foundation Rock	202,632	CY	$ 34.50	$ 34.50	$ 6,990,804	$ 6,990,804
12	Foundation Treatment - Grout Curtain	25,000	LF	$ 16.50	$ 16.50	$ 412,500	$ 412,500
13	Conventional Concrete Reinforced	20,000	CY	$ 700.00	$ 740.00	$ 14,000,000	$ 14,800,000
14	Conventional Concrete Non-Reinforced	64,000	CY	$ 400.00	$ 476.00	$ 25,600,000	$ 30,464,000
15	Fill -Roller Compacted Concrete	1,319,700	CY	$ 65.00	$ 95.00	$ 85,780,500	$ 125,371,500
16	Fill - Foundation Backfill	452,828	CY	$ 4.50	$ 4.50	$ 2,037,726	$ 2,037,726
	Subtotal					$ 139,045,620	$ 184,300,620
	Other						
17	Fill - Wingdam Embankment	117,768	CY	$ 4.50	$ 4.50	$ 529,956	$ 529,956
18	Large Diameter Valves	1	LS	$ 8,605,500	$ 10,756,875	$ 8,605,500	$ 10,756,875
	Subtotal					$ 9,135,456	$ 11,286,831
	Total Base Construction Cost (BCC)					$ 174,947,556	$ 225,377,131
	Design Contingency					$ 43,736,889	$ 67,613,139.30
	Construction. CO/C Contingency					$ 13,995,804	$ 18,030,170.48
	Subtotal					$ 232,680,249	$ 311,020,441
	Permitting					$ 6,980,407	$ 18,661,226.45
	Design and site Characterization					$ 15,124,216	$ 27,991,839.67
	Engineering Support During Construction					$ 20,941,222.45	$ 37,322,452.89
	Total Cost (Rounded)					**$ 275,700,000**	**$ 395,000,000**

Notes: CY = cubic yard LB = pound LF = lineal foot

7.3.3 MULTI-PURPOSE ROCKFILL DAM

The estimated total cost for this dam and appurtenant structures configuration range from $412 million to $570 million with average of $491 million. A summary of these estimated costs is provided in Table 7-4.

Table 7-4
Summary Cost Estimate for MP-Rockfill Dam, "Baseline" Study Conditions

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
	Phase 1 - Prep Work						
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	82	Acre	$ 29,560.00	$ 29,560.00	$ 2,423,920	$ 2,423,920
2	Flood Control Outlet Tunnel 20 ft dia	1,500	LF	$ 7,670.00	$ 7,670.00	$ 11,505,000	$ 11,505,000
3	Excavation - General (Intake and Outlet)	96,800	CY	$ 6.50	$ 6.50	$ 629,200	$ 629,200
4	Excavation - Rock (Intake and Outlet)	49,700	CY	$ 34.50	$ 34.50	$ 1,714,650	$ 1,714,650
5	Fill - Coffer Dams	254,000	CY	$ 6.50	$ 6.50	$ 1,651,000	$ 1,651,000
6	Control Structures - Reinforced Concrete	34,000	CY	$ 700.00	$ 740.00	$ 23,800,000	$ 25,160,000
7	Control Structures - Gates	140,000	LB	$ 12.00	$ 20.00	$ 1,680,000	$ 2,800,000
8	Regulating Intake Access Shaft	290	LF	$ 25,000.00	$ 32,000.00	$ 7,250,000	$ 9,280,000
	Subtotal					$ 50,653,770	$ 55,163,770
	Phase 2 - Main Dam						
9	Excavation - Foundation General	2,145,000	CY	$ 6.50	$ 6.50	$ 13,942,500	$ 13,942,500
10	Foundation Treatment - Grout Curtain	18,000	LF	$ 16.50	$ 16.50	$ 297,000	$ 297,000
11	Foundation Treatment - Dental Concrete	31,000	CY	$ 279.00	$ 300.00	$ 8,649,000	$ 9,300,000
12	Fill - Central Clay Core (Zone 1)	1,982,556	CY	$ 6.50	$ 8.10	$ 12,886,614	$ 16,058,704
13	Fill - Filter Transition (Zone 2)	539,316	CY	$ 45.00	$ 53.80	$ 24,269,220	$ 29,015,201
14	Fill - Rockfill (Zone 3)	4,534,812	CY	$ 25.00	$ 34.50	$ 113,370,300	$ 156,451,014
15	Fill - Riprap (Zone 4)	227,880	CY	$ 28.00	$ 34.50	$ 6,380,640	$ 7,861,860
	Subtotal					$ 179,795,274	$ 232,926,278
	Other						
16	Steel Pipe - Temp. Intake Tunnels (48") Drill	1,500	LF	$ 2,300.00	$ 2,800.00	$ 3,450,000	$ 4,200,000
17	Temperature Intake Tunnel (15' Horseshoe)	700	LF	$ 19,000.00	$ 22,000.00	$ 13,300,000	$ 15,400,000
18	Steel Pipe - Temperature Intake Tunnel (60")	700	LF	$ 1,350.00	$ 1,500.00	$ 945,000	$ 1,050,000
19	Spillway Excavation	347,000	CY	$ 6.50	$ 6.50	$ 2,255,500	$ 2,255,500
20	Spillway RCC	36,000	CY	$ 65.00	$ 95.00	$ 2,340,000	$ 3,420,000
21	Large Diameter Valves	1	LS	$ 8,718,000	$ 10,897,500	$ 8,718,000	$ 10,897,500
	Subtotal					$ 31,008,500	$ 37,223,000
	Total BCC					$ 261,457,544	$ 325,313,048
	Design Contingency					$ 65,364,386	$ 97,593,915
	Construction. CO/C Contingency					$ 20,916,604	$ 26,025,044
	Subtotal					$ 347,738,534	$ 448,932,007
	Permitting					$ 10,432,156.01	$ 26,935,920.41
	Design and site Charactgerization					$ 22,603,004.68	$ 40,403,880.61
	Engineering Support During Construction					$ 31,296,468.02	$ 53,871,840.82
	Total Cost (Rounded)					**$ 412,100,000**	**$ 570,100,000**

Notes: CY = cubic yard LB = pound LF = lineal foot

7.4 FISH PASSAGE OPTIONS

A summary of the estimated probable costs for the fish passage concepts summarized in Chapter 4 is provided in Table 7-5.

Table 7-5
Summary of Estimated Fish Passage Costs

FISH PASSAGE OPTION	LOWER BOUND COST ($MILLION)	MIDDLE COST ($MILLION)	UPPER BOUND COST ($MILLION)
CHTR	9.9	12.3	17.3
Conventional Fishway	29.3	36.6	51.3
Experimental Fishway	39.8	49.7	69.6
Combo Collection Facilities	17.0	21.2	29.7
Forebay Collector	26.4	33.0	46.2
Fish Passage Tunnels	Included with the cost of the FRO-RCC dam alternative		

Notes: CHTR = collect, handle, transfer, and release
FRO-RCC = Flood Retention Only roller compacted concrete

7.5 HYDROPOWER

A planning level evaluation of the hydropower potential of the Chehalis project dam site was completed and summarized in Section 5.5 of this report. Based on the results of this investigation, a concept level configuration of a powerhouse was formulated based on experience with similar size and flow/power generation potential sites and probable costs were estimated. A summary of this evaluation is as follows:

- The estimated total construction cost for hydropower at the site would be $14 to $18 million. This cost does not include the intake structure and outlet penstock developed and included as part of the probable cost estimates for the water quality outlet works included with the multi-purpose dam configurations.
- The cost estimate includes a 30 to 35 percent contingency for allied costs for permitting and licensing, engineering, construction management, finance, legal/administration. This contingency total $5 million.
- Additional fish screening requirements could add $1 to $2 million to hydropower system costs.
- Total of all hydropower capital costs would range from $20 to $25 million.
- Estimated average annual energy production would be about 24,000 MWH.
- The study results suggest a moderate to high likelihood of a positive cost benefit ratio depending on power purchase agreement terms.

7.6 COMBINED DAM AND FISH PASSAGE ALTERNATIVES COST COMPARISON

Using the various component estimates of probable costs summarized in Section 7.2 through 7.4 total project costs for the Baseline Alternatives A through D, as well as the Sub-Alternatives A1 and A2 were developed. These costs do not include hydropower development costs summarized in Section 7.5. They are presented in the follow sections.

7.6.1 ALTERNATIVES A THROUGH D

A summary of the component and total combined costs for Alternatives A through D is provided in Table 7-6. The primary numbers in this table are the average estimated combined costs. The expected variance, based on the uncertainties described in previous sections of the report, are also indicated.

Table 7-6
Summary of Combined Costs for Dam and Fish Passage Alternatives

ALTERNATIVE	PRELIMINARY CLASS 4 COST ESTIMATE 2014 $MILLION, AVERAGE ESTIMATED VALUE AND (-/+ RANGE OF COSTS)				
	DAM	FISH PASSAGE UPSTREAM	FISH PASSAGE DOWNSTREAM	HYDROPOWER	TOTAL
A (FRO/RCC)	266 (217-314)	14 (10-19)			280
B (Multi-purpose/RCC, CHTR, CC)	336 (276-395)	13 (10-18)	22 (17-30)	22 (20-25)	393
C (Multi-purpose/RCC, FW, FC)	336 (276-395)	37 (30-52)	33 (27-47)	22 (20-25)	428
D (Multi-purpose/RF, EFW, FC)	491 (412-570)	50 (40-70)	33 (27-47)	22 (20-25)	596

Notes: CC = climate change EFW = experimental fishway FC = floating collector FW = fishway
CHTR = collect, handle, transfer, release FRO/RCC = Flood Retention Only roller compacted concrete

7.6.2 CLIMATE CHANGE ALTERNATIVES A1 AND A2

A summary of the estimated costs for these climate change sub-alternatives are as follows:

Scenario 1 – 18 percent increase in Chehalis River flows above dam site. As previously noted, this scenario results in 10,000 acre-foot increase in required flood retention storage and corresponds to an increase in the dam height of 9 feet to 239 feet over the baseline flood control only RCC dam included as part of Alternative A. The corresponding combined alternative is referred to as Alternative A1. The total estimated costs range from $246 million to $361 million with an average of $303 million. A summary of these estimated costs are provided in Table 7-7. This corresponds to a $23 million increase over the "baseline" configuration FR-RCC dam estimate of probable costs (Table 7-2) for a corresponding 15 percent increase in the flood retention volume.

Table 7-7
Summary of Estimated Dam Costs for Climate Change Scenario 1 Used as part of Sub-alternative A1

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
	Phase 1 - Prep Work						
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	27	Acre	$ 29,560.00	$ 29,560.00	$ 798,120	$ 798,120
2	Flood Control Outlet Tunnel 20 ft wide	1,420	LF	$ 7,670.00	$ 7,670.00	$ 10,891,400	$ 10,891,400
3	Excavation - General	112,000	CY	$6.50	$ 6.50	$ 728,000	$ 728,000
4	Control Structures - Reinforced Concrete	2,089	CY	$ 700.00	$ 740.00	$ 1,462,300	$ 1,545,860
5	Control Structures - Gates (Fab and Construct)	568,091	LB	$ 12.00	$ 20.00	$ 6,817,092	$ 11,361,820
6	Fill - Coffer Dams	28,851	CY	$ 6.50	$ 6.50	$ 187,532	$ 187,532
	Subtotal					$ 20,884,444	$ 25,512,732
	Phase 2 - Main Dam						
7	Excavation - Foundation General	508,798	CY	$ 6.50	$ 6.50	$ 3,307,187	$ 3,307,187
8	Excavation - Foundation Rock	123,713	CY	$ 34.50	$ 34.50	$ 4,268,099	$ 4,268,099
9	Fill - Roller Compacted Concrete	828,515	CY	$65.00	$95.00	$ 53,853,475	$ 78,708,925
10	Fill - Foundation Backfill	347,012	CY	$ 4.50	$ 4.50	$ 1,561,554	$ 1,561,554
11	Conventional Concrete Reinforced	27,741	CY	$ 700.00	$ 740.00	$ 19,418,700	$ 20,528,340
12	Conventional Concrete Non-Reinforced	83,224	CY	$ 400.00	$ 476.00	$ 33,289,600	$ 39,614,624
13	Foundation Treatment - Grout Curtain	15,506	LF	$ 16.50	$ 16.50	$ 255,849	$ 255,849
14	Fish Passage Gates - Radial (Fab and Construct)	469,929	LB	$ 12.00	$ 20.00	$ 5,639,148	$ 9,398,580
15	Fish Passage Gates - DS Vertical (Fab and Construct)	394,740	LB	$ 12.00	$ 20.00	$ 4,736,880	$ 7,894,800
	Subtotal					$ 126,330,492	$ 165,537,958
	Other						
16	Bulkheads	215,123	LB	$ 12.00	$ 20.00	$ 2,581,476	$ 4,302,460
	Subtotal					$ 2,581,476	$ 4,302,460
	Total Base Construction Cost (BCC)					$ 149,796,411	$ 195,353,149
	Design Contingency					$ 37,449,103	$ 58,605,945
	Construction. CO/C Contingency					$ 11,983,713	$ 15,628,252
	Subtotal					$ 199,229,227	$ 269,587,346
	Permitting					$ 5,976,877	$ 16,175,241
	Design and Site Characterization					$ 12,949,900	$ 24,262,861
	Engineering Support During Construction					$ 17,930,630	$ 32,350,481
	Total Cost (Rounded)					**$ 236,100,000**	**$ 342,400,000**

Notes: CY = cubic yards LB = pound LF = lineal foot

Scenario 2 – 90 percent increase in Chehalis River flows above the dam site. As previously noted, this scenario results in a 65,000 acre-foot increase in the required flood retention storage and corresponds to an increase in the dam height of 59 feet to 287 feet. This is the same total storage and dam height as required for the MP-RCC dam configuration. The corresponding combined alternative is referred to as Alternative A2. The total estimated costs range from $322 million to $478 million with an average of $400 million. A summary of these estimated costs are provided in Table 7-8. This corresponds to a $120 million increase over the "baseline" configuration FR-RCC dam estimate of probable costs (Table 7-2) for a corresponding 100 percent increase in the flood retention volume.

Table 7-8
Summary of Estimated Dam Costs for Climate Change Scenario 2 Used as part of Sub-alternative A2

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
	Phase 1 - Prep Work						
1	Clearing and grubbing, reservoir clearing, stripping topsoil, reclamation of disturbed areas	28	Acre	$ 29,560.00	$ 29,560.00	$ 827,680	$ 827,680
2	Flood Control Outlet Tunnel 20 ft wide	1,420	LF	$ 7,670.00	$ 7,670.00	$ 10,891,400	$ 10,891,400
3	Excavation - General	112,000	CY	$ 6.50	$ 6.50	$ 728,000	$ 728,000
4	Control Structures - Reinforced Concrete	2,555	CY	$ 700.00	$ 740.00	$ 1,788,500	$ 1,890,700
5	Control Structures - Gates (Fab and Construct)	694,949	LB	$ 12.00	$ 20.00	$ 8,339,388	$ 13,898,980
6	Fill - Coffer Dams	26,000	CY	$ 6.50	$ 6.50	$ 169,000	$ 169,000
	Subtotal					$ 22,743,968	$ 28,405,760
	Phase 2 - Main Dam						
7	Excavation - Foundation General	649,860	CY	$ 6.50	$ 6.50	$ 4,224,090	$ 4,224,090
8	Excavation - Foundation Rock	202,632	CY	$ 34.50	$ 34.50	$ 6,990,804	$ 6,990,804
9	Fill - Roller Compacted Concrete	1,319,700	CY	$65.00	$95.00	$ 85,780,500	$ 125,371,500
10	Fill - Foundation Backfill	452,828	CY	$ 4.50	$ 4.50	$ 2,037,726	$ 2,037,726
11	Conventional Concrete Reinforced	31,250	CY	$ 700.00	$ 740.00	$ 21,875,000	$ 23,125,000
12	Conventional Concrete Non-Reinforced	93,750	CY	$ 400.00	$ 476.00	$ 37,500,000	$ 44,625,000
13	Foundation Treatment - Grout Curtain	25,000	LF	$ 16.50	$ 16.50	$ 412,500	$ 412,500
14	Fish Passage Gates - Radial (Fab and Construct)	574,866	LB	$ 12.00	$ 20.00	$ 6,898,392	$ 11,497,320
15	Fish Passage Gates - DS Vertical (Fab and Construct)	482,887	LB	$ 12.00	$ 20.00	$ 5,794,644	$ 9,657,740
	Subtotal					$ 171,513,656	$ 227,941,680
	Other						
16	Bulkheads	263,161	LB	$ 12.00	$ 20.00	$ 3,157,932	$ 5,263,220
17	Fill - Wingdam Embankment	117,768	CY	$ 4.50	$ 4.50	$ 529,956	$ 529,956
	Subtotal					$ 3,687,888	$ 5,793,176
	Total Base Construction Cost (BCC)					$ 197,945,512	$ 262,140,616
	Design Contingency					$ 49,486,378	$ 78,642,185
	Construction. CO/C Contingency					$ 15,835,641	$ 20,971,249
	Subtotal					$ 263,267,531	$ 361,754,050
	Permitting					$ 7,898,026	$ 21,705,243
	Design and Site Characterization					$ 17,112,390	$ 32,557,865
	Engineering Support During Construction					$ 23,694,078	$ 43,410,486
	Total Cost (Rounded)					**$ 312,000,000**	**$ 459,400,000**

Notes: CY = cubic yards LB = pound LF = lineal foot

Estimates of probable project costs for the dam and fish passage costs for climate change Scenario 1 (Alternative A1) and Scenario 2 (Alternative A2) as compared to the Base Case Alternative A are summarized in Table 7-9.

Table 7-9
Alternative A Dam and Fish Passage Climate Change Cost Impacts

PRELIMINARY CLASS 4 COST ESTIMATE 2014 $MILLION AVERAGE ESTIMATED VALUE (AND -/+ RANGE OF COSTS)		
BASE CASE A	A1 CLIMATE CHANGE SCENARIO 1	A2 CLIMATE CHANGE SCENARIO 2
280 (227-333)	303 (246-361)	400 (322-478)

7.7 OPERATIONS AND MAINTENANCE ESTIMATED ANNUAL COSTS

Dam, fish passage, and optional future hydropower O&M costs were estimated based upon experience with similar projects, estimated labor full time equivalent (FTE) costs, typical expenses, administrative reporting costs, and operational costs.

Dam and reservoir cost categories included vegetation management, debris handling, and disposal, environmental monitoring and reporting, dam operations, administrative management and reporting, legal and insurance, dam maintenance and repairs, safety inspections, and mechanical and structural repair/replacement funding.

Each of the fish passage alternatives requires a number of different specialized O&M activities ranging on the basis of frequency from daily to annually occurrence. Costs for these activities were generated from estimated labor, expendables, and replacement categories as well as by scaling values obtained from other example facilities. Labor was estimated based upon FTEs for various seasonal or full-time personnel. Expendables and replacement costs were estimated based upon escalated actual cost and/or by percentage of total O&M labor cost.

For the only periodically used flood retention RCC dam Alternative A, the annual dam repair and replacement funds were set at 0.4 percent (mechanical) and 0.1 percent (structural) of the estimated capital construction costs for each of those categories. For the multi-purpose dam alternatives, the repair and replacement funds were doubled to 0.8 percent and 0.2 percent respectively. These higher repair and replacement funds were also used for the multi-purpose dam fish passage facilities.

Referenced costs for vegetation management and debris handling varied significantly and the costs presented in the estimate are estimated long-term averages in annual 2014 dollars. Vegetation management and debris handling would be expected to be significantly higher than these long-term average costs during the first several years of reservoir operation. Table 7-10 summarizes preliminary estimated annual O&M Costs for the combined dam and fish passage alternatives.

Table 7-10
Estimated Annual Dam and Fish Passage Annual Operations and Maintenance Costs

COMBINED ALTERNATIVES	PRELIMINARY ESTIMATED ANNUAL O&M COSTS (2014 $1,000) AVERAGE ESTIMATED VALUE AND (-/+ RANGE OF COSTS)		
	DAM AND RESERVOIR ($1,000)	FISH PASSAGE ($1,000)	TOTAL ANNUAL ($1,000)
A (FRO/RCC)	603 (543-664)	190 (182-198)	793
B (Multi-purpose/RCC, CHTR, CC)	958 (862-1054)	581 (523-639)	1,539
C (Multi-purpose /RCC, FW, FC)	958 (862-1054)	433 (390-476)	1,391
D (Multi-purpose /RF, EFW, FC)	1,104 (994-1,214)	520 (468-572)	1,624

Notes: EFW = experimental fishway FC = floating collector FW = fishway
CHTR = collect, handle, transfer, release FRO/RCC = Flood Retention Only roller compacted concrete

Annual operations and maintenance costs for the optional future hydropower facility were estimated to be approximately $485,000 per year in 2014 dollars. Because hydropower is only a future option, this cost was not included in the above total annual alternative O&M costs.

More detailed annual O&M cost tables for each alternative are included in Appendix F.

Construction Schedules

Based on experience with other similar projects involving large dam and fish passage facilities, the following overall project schedule should be considered:

1.	Completion of planning and permitting	up to 2 additional years
2.	Final design including site characterization	up to 2 years
3.	Bidding and award	4 to 6 months
4.	Construction Schedules (details below)	2 to 3 years
	Approximate Total:	**7 to 8 years**

These finding are consistent with the anticipated project schedule outlined in the Final October 30, 2012, Chehalis River Basin Flood Retention Structure 8-Year Project Planning Document.

8 Construction Schedules

The following figures present the approximate construction schedules for each of the four combined alternatives. The lines within each schedule list the order of major summary construction activities that would be occurring for each alternative. The schedules are shown by construction year subdivided into quarters for each year. The darkened squares indicate which of the construction activities would likely be occurring in any given quarter.

As a way of predicting approximate cash flow, the numbers within each darkened square indicate the approximate expected percentages of the total construction cost for those major work items and that time period that would be occurring. Because the costs for the dam, upstream fish passage, and downstream fish passage were itemized separately for each alternative, the schedules are subdivided to separately show the level of percentages (totaling to 100 percent) for each of those project elements.

8.1 ALTERNATIVE A – FLOOD RETENTION RCC DAM SCHEDULE

8.1.1 FLOOD RETENTION RCC DAM WITHOUT CLIMATE CHANGE

Figure 8-1
Alternative A Construction Schedule

CONSTRUCTION YEAR	YEAR 1				YEAR 2				YEAR 3			
CONSTRUCTION ITEM	Q1	Q2	Q3	Q4	Q5	Q6	Q7	Q8	Q9	Q10	Q11	Q12
Mobilization/Site Prep	5											
Tunnel Diversion, Entry/Discharge Pools	5	12	4									
Cofferdams			4									
Concrete and RCC Dam Construction			8	16	16	14						
Spillway and Stilling Basin						4	4	4				

CONSTRUCTION YEAR	YEAR 1				YEAR 2				YEAR 3			
CONSTRUCTION ITEM	Q1	Q2	Q3	Q4	Q5	Q6	Q7	Q8	Q9	Q10	Q11	Q12
Final Cleanup, Remove Cofferdams								4				
In Operation									X			
FISH PASSAGE CONSTRUCTION												
Upstream Fish Passage CHTR		25	50	25								

Notes: CHTR = collect, handle, transfer, release

RCC = roller compacted concrete

8.1.2 FLOOD RETENTION DAM WITH CLIMATE CHANGE

The larger flood retention dams required for climate change would take longer to build. The Scenario 1 climate change would extend the RCC dam construction approximately one month. The much larger Scenario 2 climate change dam would be similar in size to the Alternative B multi-purpose dam and would have approximately the same construction period and cash flow as shown for Alternative B.

8.2 ALTERNATIVE B – MULTI-PURPOSE RCC DAM WITH CHTR AND HEAD OF RESERVOIR FISH COLLECTORS

Figure 8-2
Alternative B Construction Schedule

CONSTRUCTION YEAR	YEAR 1				YEAR 2				YEAR 3			
CONSTRUCTION ITEM	Q1	Q2	Q3	Q4	Q5	Q6	Q7	Q8	Q9	Q10	Q11	Q12
DAM CONSTRUCTION												
Mobilization/Site Prep	4											
Flood Contr. Outlet, Entry/Discharge Pools	4	6										
Cofferdams, Begin Outlet Diversion		3	4									
Water Quality Outlet Structure			6	4								
Concrete and RCC Dam				8	14	12	8	6				
Spillway and Stilling Basin							3	4	8			
Remove Cofferdams, Channel Modifications									2	4		
FISH PASSAGE CONSTRUCTION												
Upstream Fish Passage CHTR		25	50	25								
Downstream Fish Passage Collectors							50	50				
In Operation											X	

Notes: CHTR = collect, handle, transfer, release

RCC = roller compacted concrete

8.3 ALTERNATIVE C – MULTI-PURPOSE RCC DAM WITH EXPERIMENTAL FISHWAY AND DAM MOUNTED FLOATING COLLECTOR

Figure 8-3
Alternative C Construction Schedule

CONSTRUCTION YEAR	YEAR 1				YEAR 2				YEAR 3			
CONSTRUCTION ITEM	Q1	Q2	Q3	Q4	Q5	Q6	Q7	Q8	Q9	Q10	Q11	Q12
DAM CONSTRUCTION												
Mobilization/Site Prep	4											
Flood Contr. Outlet, Entry/Discharge Pools	4	6										
Cofferdams, Begin Outlet Diversion		3	4									
Water Quality Outlet Structure			6	4								
Concrete and RCC Dam				8	14	12	8	6				
Spillway and Stilling Basin							3	4	8			
Remove Cofferdams, Channel Modifications									2	4		
FISH PASSAGE CONSTRUCTION												
Upstream Fish Passage Fishway				20	25	30	25					
Downstream Fish Passage Floating Collector						30	40	30				
In Operation											X	

Notes: RCC = roller compacted concrete

8.4 ALTERNATIVE D – MULTI-PURPOSE ROCKFILL DAM WITH CONVENTIONAL FISHWAY AND FOREBAY COLLECTOR

Figure 8-4
Alternative D Construction Schedule

CONSTRUCTION YEAR	YEAR 1				YEAR 2				YEAR 3			
CONSTRUCTION ITEM	Q1	Q2	Q3	Q4	Q5	Q6	Q7	Q8	Q9	Q10	Q11	Q12
DAM CONSTRUCTION												
Mobilization/Site Prep	4											
Flood Contr. Outlet, Entry/Discharge Pools	4	4										
Cofferdams, Begin Outlet Diversion		4	3									
Water Quality Outlet Structure			6	4								
Rockfill Dam				6	9	9	9	8	8			
Spillway and Stilling Basin								2	2	8	4	
Remove Cofferdams, Channel Modifications											2	4
FISH PASSAGE CONSTRUCTION												
Upstream Fish Passage Fishway					20	25	30	25				
Downstream Fish Passage Floating Collector							30	40	30			
In Operation									**in 1st Quarter Year 4 ->**			

Alternatives Comparison and Recommendations

9 Alternative Comparison and Recommendations

9.1 SUMMARY COMPARISON OF ALTERNATIVES

The following Table 9-1 presents a comparative summary of the characteristics and costs for each of the alternatives described and evaluated in this technical memorandum.

Table 9-1
Summary Comparison of Alternatives

COMBINED ALTERNATIVE	A			B	C	D
SUB-ALTERNATIVE	A BASE	A1	A2			
Purpose	Flood retention only			Multi-purpose	Multi-purpose	Multi-purpose
Dam Type	Roller compacted concrete			Roller compacted concrete	Roller compacted concrete	Central clay core rockfill
Dam Structural Height (feet)	254	263	313	313	313	316
Spillway Crest Elevation (feet)	628	637	687	687	687	687
Emergency Spillway Type	Dam Crest			Dam crest	Dam crest	Side channel
Reservoir Storage Volume (1,000 AF)	65	75	130	130	130	130
Upstream Fish Passage	Flow through channels and CHTR facility			CHTR	Conventional fishway	Experimental fishway
Downstream Fish Passage	Flow through channels			Head of reservoir collectors	Dam attached floating collector	Dam attached floating collector
Future Hydropower Potential (MWh/year)	None			24,000	24,000	24,000
Construction Period (months)	24	25	30	30	30	36
Estimated Dam and Fish Passage Project Costs (2014 $Million)	280	303	400	393	428	596
Estimated Annual O&M Costs ($2014 $1,000)	793			1,539	1,391	1,624
Optional Hydro Total Capital Costs ($2014 $Million)	NA			22	22	22
Optional Hydro Yearly O&M Costs ($2014 $1,000)	NA			485	485	485

Notes: AF = acre-foot CHTR = collect, handle, transfer, release MWh/year = Megawatt-hours per year
NA = Not applicable O&M = operations and maintenance

9.2 RECOMMENDATIONS

The configurations and corresponding estimates of probable construction costs presented in this report are considered reasonably conservative for their intended purposes. However, several of the following important considerations were identified that should be addressed as early as possible during subsequent planning and design phases:

1. Seismic hazards may include large magnitude earthquakes from up to three separate sources: large local crustal faults, interslab, and the Cascadia Subduction Zone. The expected frequency content, duration, and peak ground accelerations at the dam site will have a significant influence on the design of the various facilities and it is recommended that the next phase of work include a design level assessment of seismic hazards and appropriate structural and geotechnical analyses necessary to confirm the dam configuration requirements.
2. The configuration designs for the dams evaluated during this study are based on the dam study teams experience and general understanding of the geology of the site. A preliminary design level site characterization study involving supplement geologic mapping, geophysical testing, borings, in situ testing, and lab testing of samples from the site should be completed to update the concepts and cost estimates.
3. The development of construction materials such as RCC aggregate, rockfill, and transition/filter/drain materials in close proximity to the site will significantly influence project costs and impacts associated with dam construction. A more detailed evaluation of construction materials along with a preliminary mix design for the RCC dam should be developed during the next planning and design phase.
4. Significant landslide, debris, and sediment hazards have been identified that will significantly influence design and operation of a dam at this site. Preliminary design work should include additional assessment of these hazards and concerns and incorporation of specific design elements into the planning documents and cost estimates to address these hazards.
5. A detailed hydrology study including reservoir routing should be performed for preliminary designs to update the configuration requirements for the emergency spillway and for construction diversion using the flood control outlet works. Construction flood routing requirements should be confirmed based on construction risk evaluation methods such as those used by the US Bureau of Reclamation. Subsequent risk evaluations may indicate that a separate construction diversion tunnel is required that would have to be plugged before operation of the project would begin. A separate construction diversion tunnel is not currently included in the conceptual designs for the dams.
6. There is a need to verify fish passage survival and collection efficiency rates for particular types of downstream passage technologies. The assumed rates have significant impacts on the results of the EDT modeling in the *Effects of Flood Reduction Alternatives and Climate Change on Aquatic Species* Report.
7. More study is needed to develop an estimate of the likelihood of more frequent triggering of water retention operations for a flood retention only dam under climate change scenarios when compared to base conditions.

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Appendix A – Chehalis Dam Concepts

Appendix A.1 – FR-RCC

DISCHARGE STILLING POOL

LANDSLIDE REMEDIATION REQUIRED

CONSTRUCTION FLOOD DIVERSION TUNNEL

PERMANENT FLOOD CONTROL DRAINAGE STILLING BASIN

TUNNEL MOD FOR PERMANENT FLOOD CONTROL OUTLET

FLOOD CONTROL OUTLET DISCHARGE CHANNEL (NOTE 2)

ELEV:415

13+00

FR-RCC-4

ELEV: 405

STA 12+30

10+00

15+00

FR-RCC-3

FR-RCC-2

ELEV:415

LANDSLIDE REMEDIATION REQUIRED

Qls

200' WIDE STILLING BASIN

200'

OUTLET CONTROL TOWER 20' X 20'

FLOOD CONTROL OUTLET 25'Ø ~ 1420 LF

PHASE 2 DOWNSTREAM COFFER DAM CREST ELEV: 432 (NOTE 1)

TRASH RACK 200' X 42'

SPILLWAY CHUTE

FISH PASSAGE TUNNELS (9) 12' WIDE X 9' HIGH ~ 225 LF

FLOOD CONTROL OUTLET INTAKE AND DEBRIS RETENTION RACK (NOTE 2)

LANDSLIDE REMOVAL AND REMEDIATION REQUIRED

RCC DAM

PHASE 2 UPSTREAM COFFER DAM CREST ELEV: 460 (NOTE 1)

ELEV:415

FR-RCC-4

0+00

LANDSLIDE REMEDIATION REQUIRED

LANDSLIDE REMEDIATION REQUIRED

FR-RCC-3

0+00

DAM AXIS

WATER ELEVATION AT PMF: 650

FLOOD POOL STORAGE ELEV: 628 FLOOD ONLY INNUNDATION

FLOW

LEGEND:

Qls — APPROXIMATE LIMITS OF LANDSLIDE

A A — CROSS-SECTION OR PROFILE LOCATION

NOTE:

1. CREST OF COFFER DAMS TO BE DETERMINED DURING FUTURE STUDY PHASE BASED ON RISK EVALUATIONS.
2. PROVISIONS FOR FLOOD PROTECTION (PHASE 1) WILL BE REQUIRED FOR CONSTRUCTION OF FLOOD CONTROL OUTLET INCLUDING INTAKE AND DISCHARGE STRUCTURES/CHANNELS AS WELL AS THE OUTLET CONTROL TOWER.

150 0 150 300

SCALE IN FEET

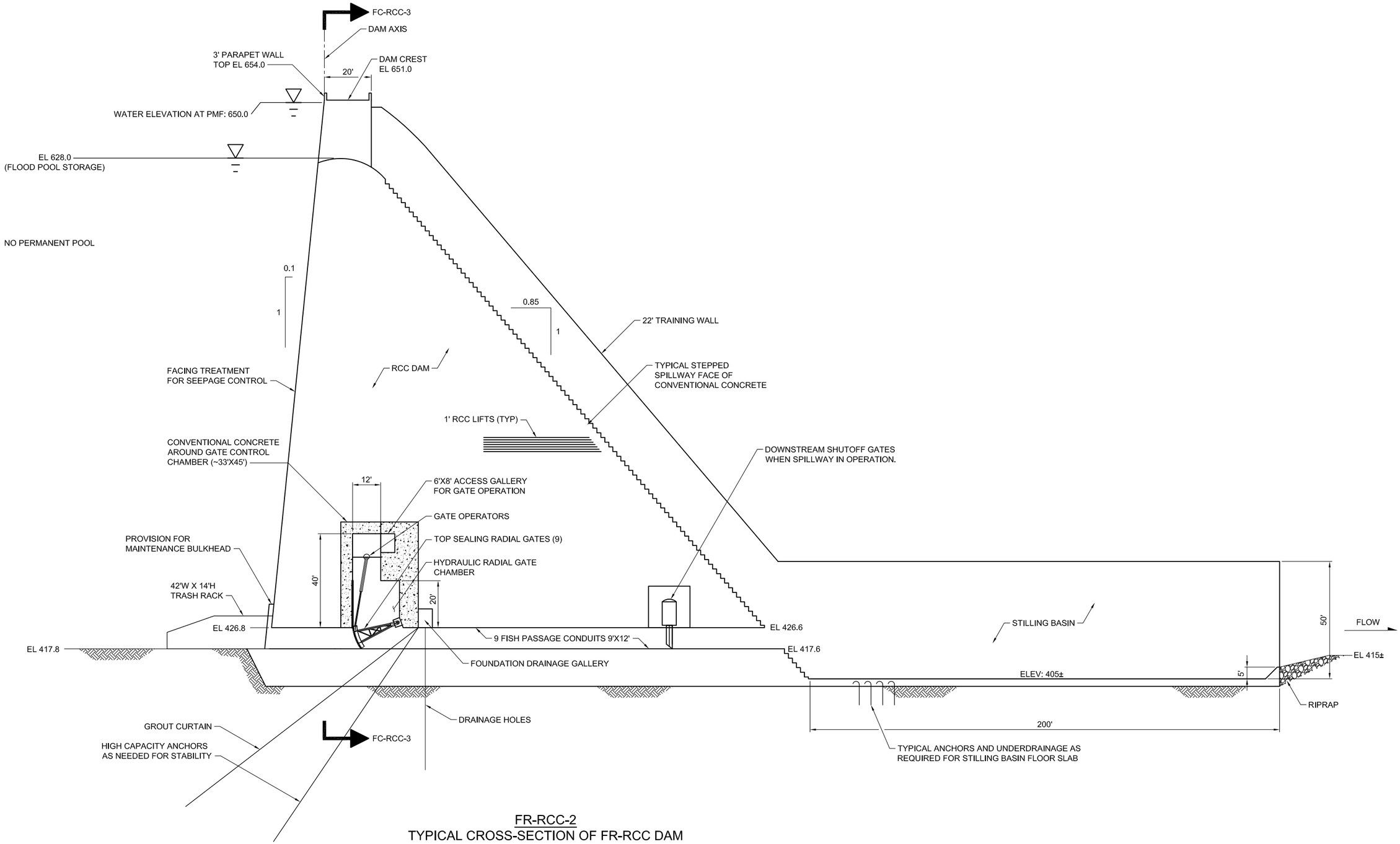
N

HDR

FLOOD RETENTION ONLY RCC DAM PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
FR-RCC-1



FR-RCC-2
TYPICAL CROSS-SECTION OF FR-RCC DAM

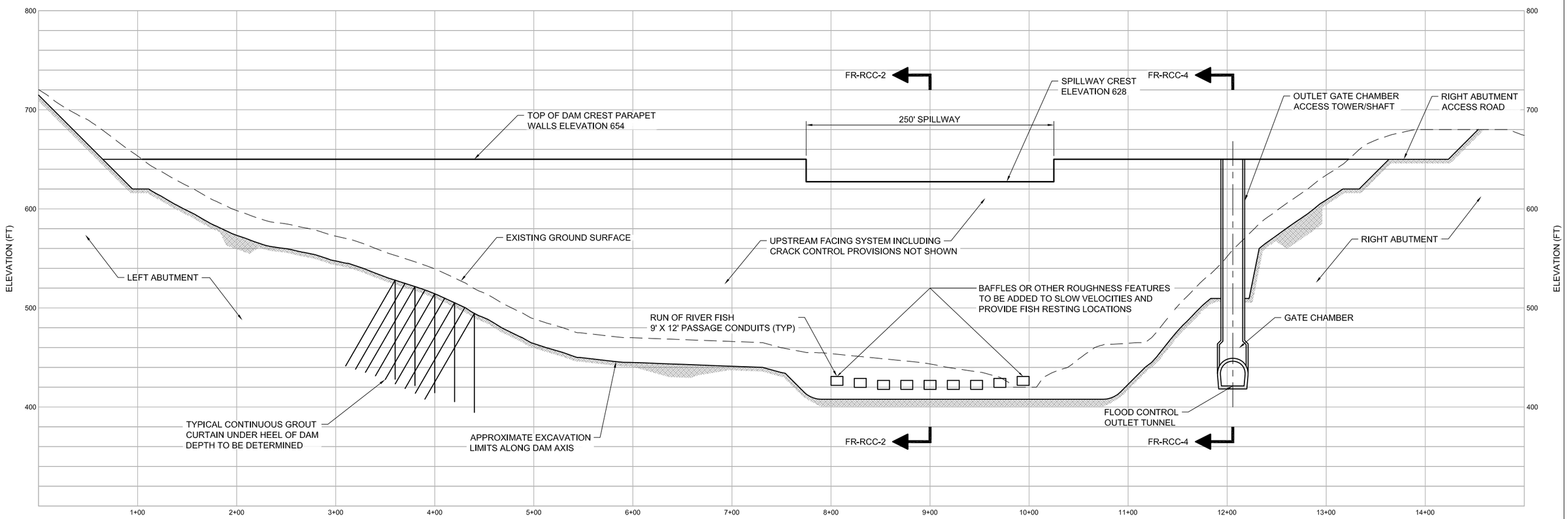
20 0 20 40
SCALE IN FEET

HDR

FLOOD RETENTION ONLY
RCC DAM TYPICAL CROSS-SECTION

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
	JULY 2014
FIGURE	
	FR-RCC-2



FR-RCC-3

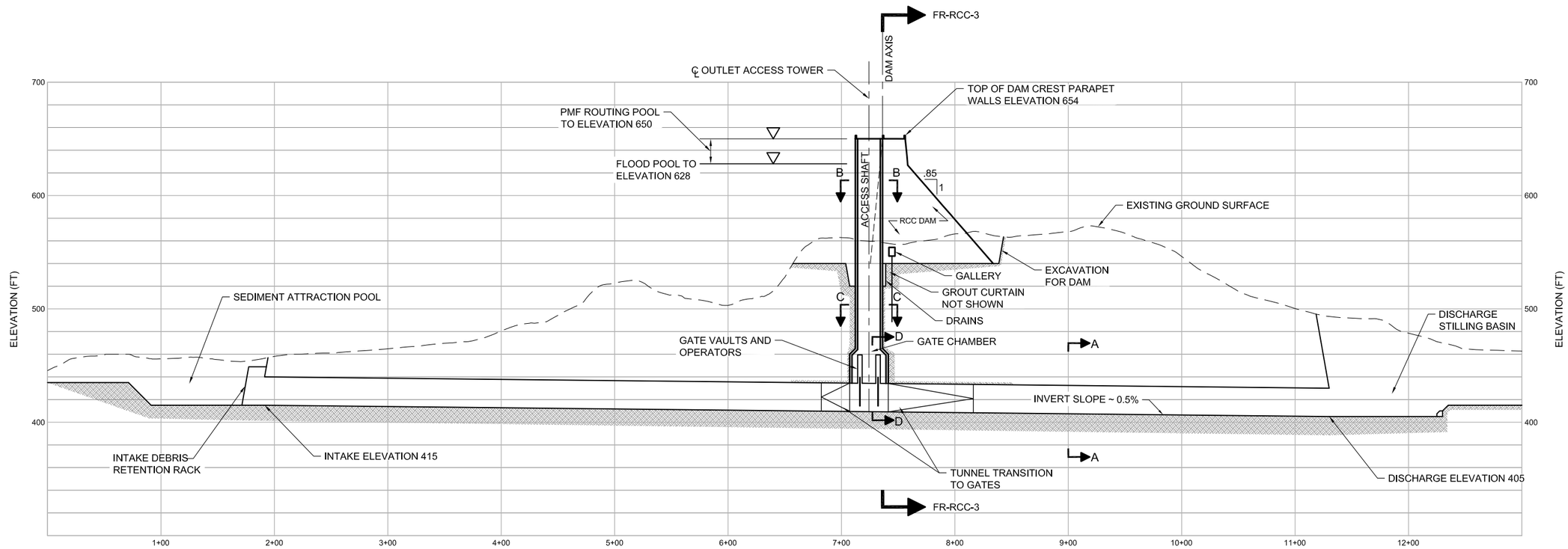
STATION ALONG AXIS (UPSTREAM FACE)
OF DAM (LOOKING DOWNSTREAM) (FEET)

HDR

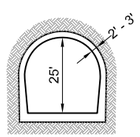
FLOOD RETENTION ONLY
RCC DAM CREST PROFILE

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
FR-RCC-3

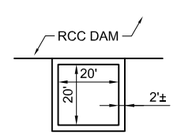


FR-RCC-4
STATION ALONG ℄ OF TUNNEL (FEET)



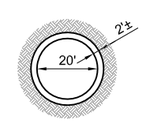
TYPICAL SECTION OUTLET TUNNEL

SECTION A-A



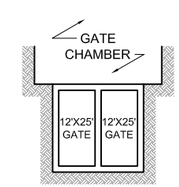
GATE CHAMBER ABOVE ACCESS TOWER FOUNDATION CONTACT

SECTION B-B



GATE CHAMBER ACCESS SHAFT BELOW FOUNDATION CONTACT

SECTION C-C



GATED SECTION OF OUTLET TUNNEL

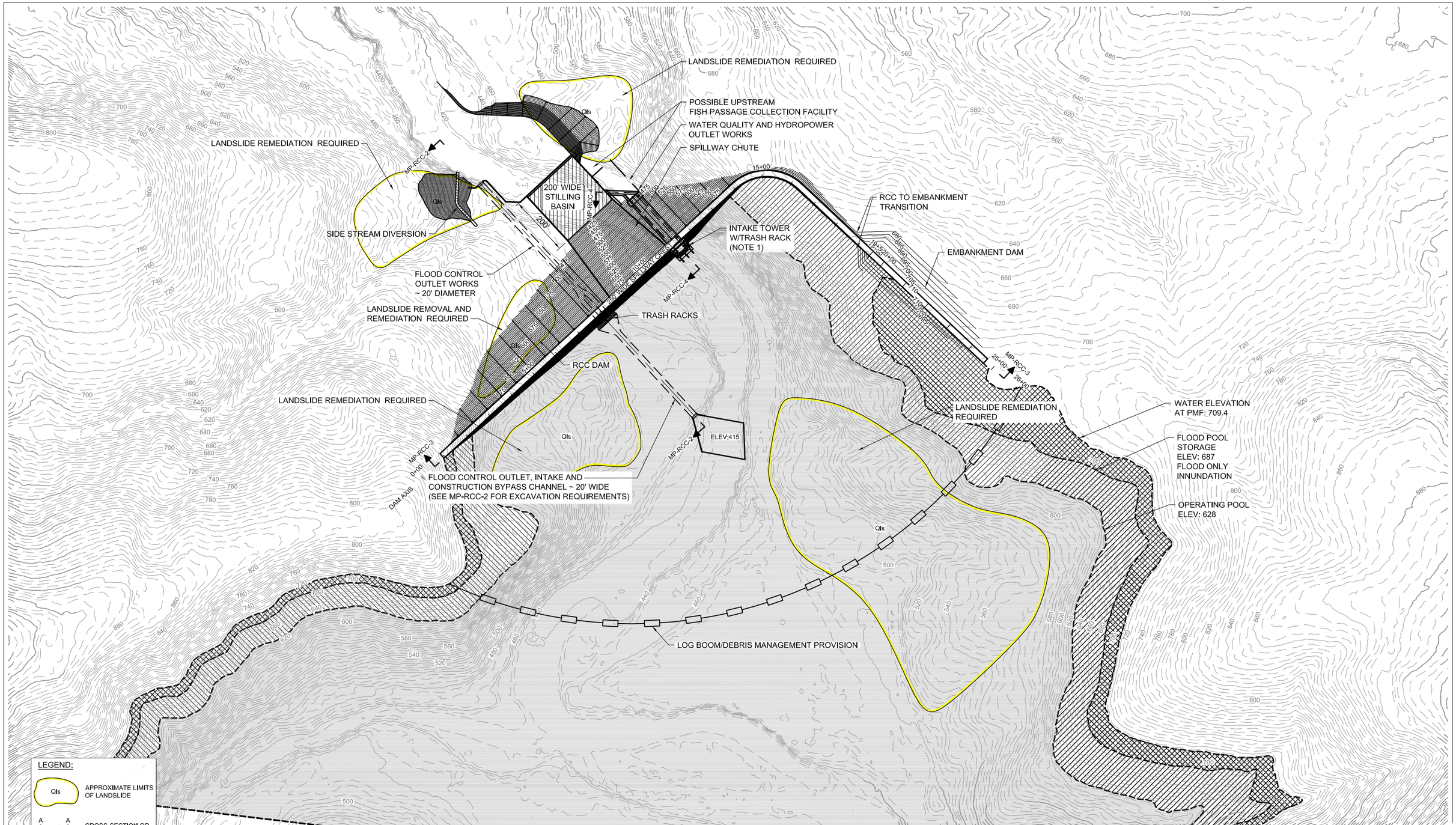
SECTION D-D

FLOOD RETENTION ONLY
RCC DAM FLOOD CONTROL OUTLET TUNNEL PROFILE

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
FR-RCC-4

Appendix A.2 – MP-RCC



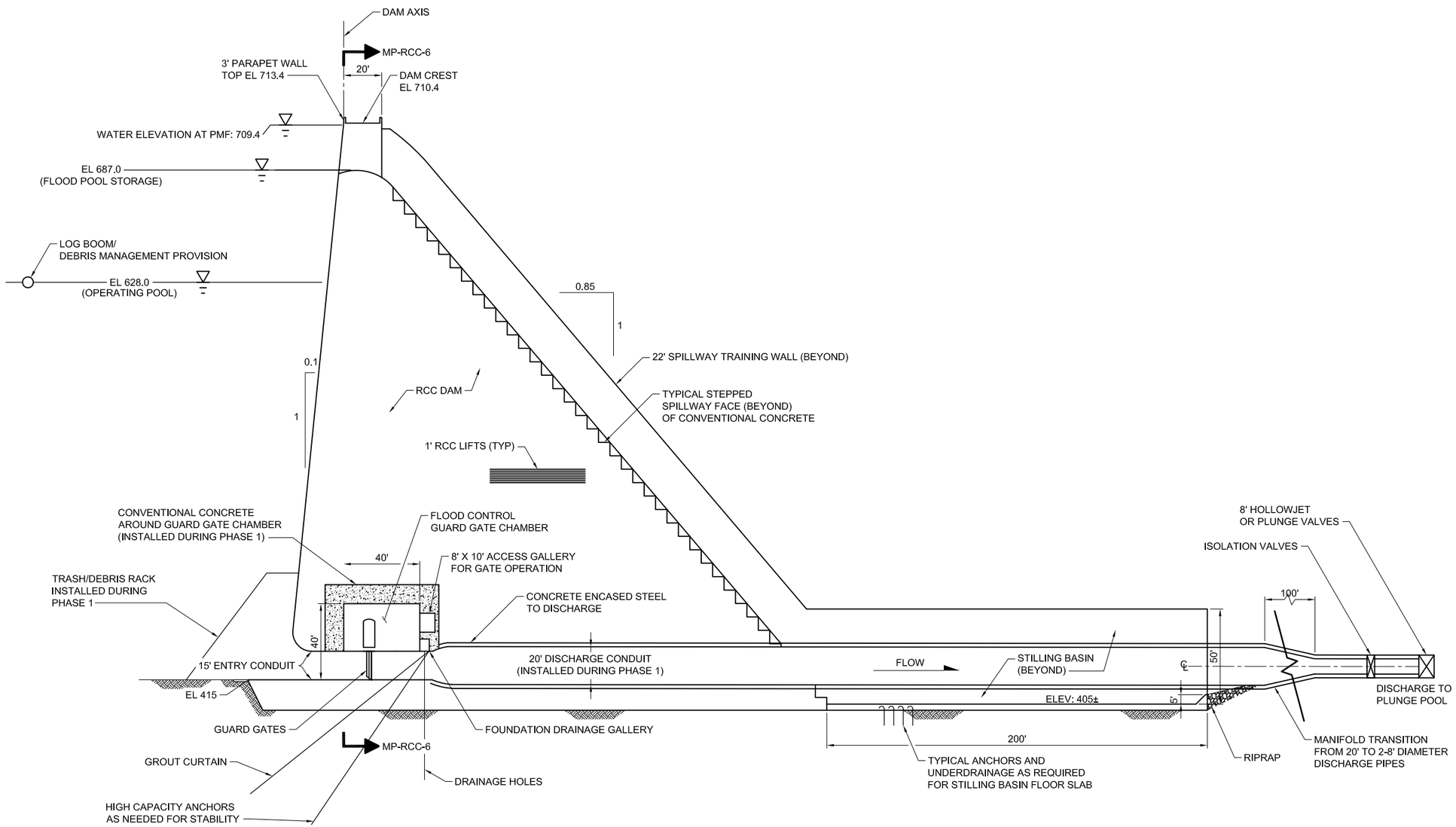
NOTE:
1. PROVISION FOR FLOOD PROTECTION (PHASE 1) WILL BE REQUIRED FOR FLOOD CONTROL OUTLET INCLUDING INTAKE AND DISCHARGE STRUCTURES/CHANNELS AS WELL AS THE OUTLET CONTROL SYSTEM TO BE INCORPORATED INTO RCC DAM.

HDR

MULTIPURPOSE - RCC DAM PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RCC-1



MP-RCC-2
TYPICAL CROSS-SECTION OF MP-RCC DAM AT
FLOOD CONTROL OUTLET ALIGNMENT

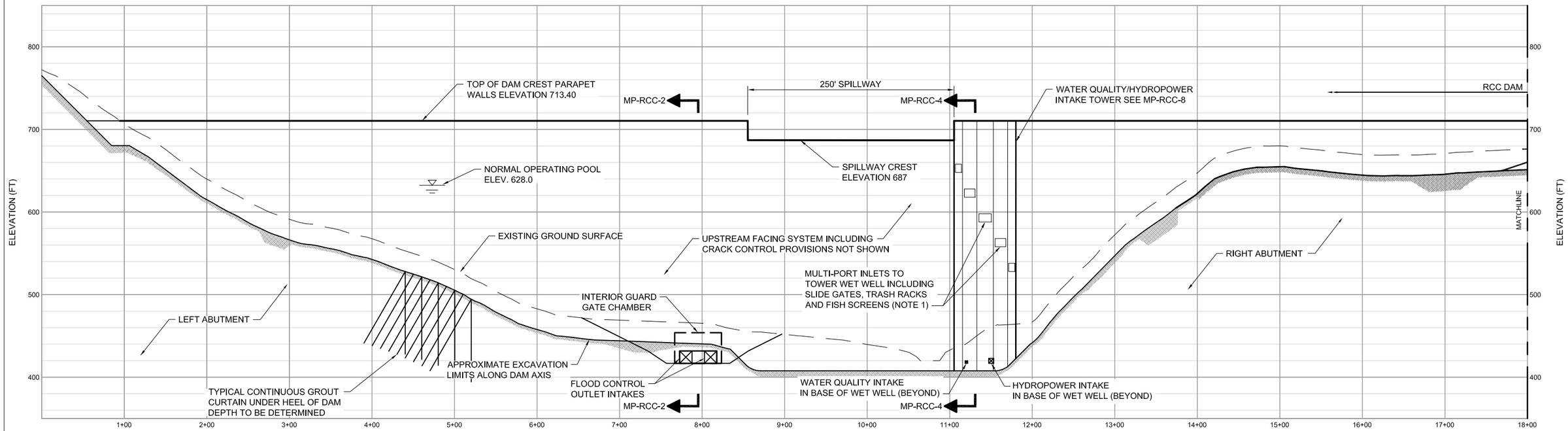
30 0 30 60
SCALE IN FEET

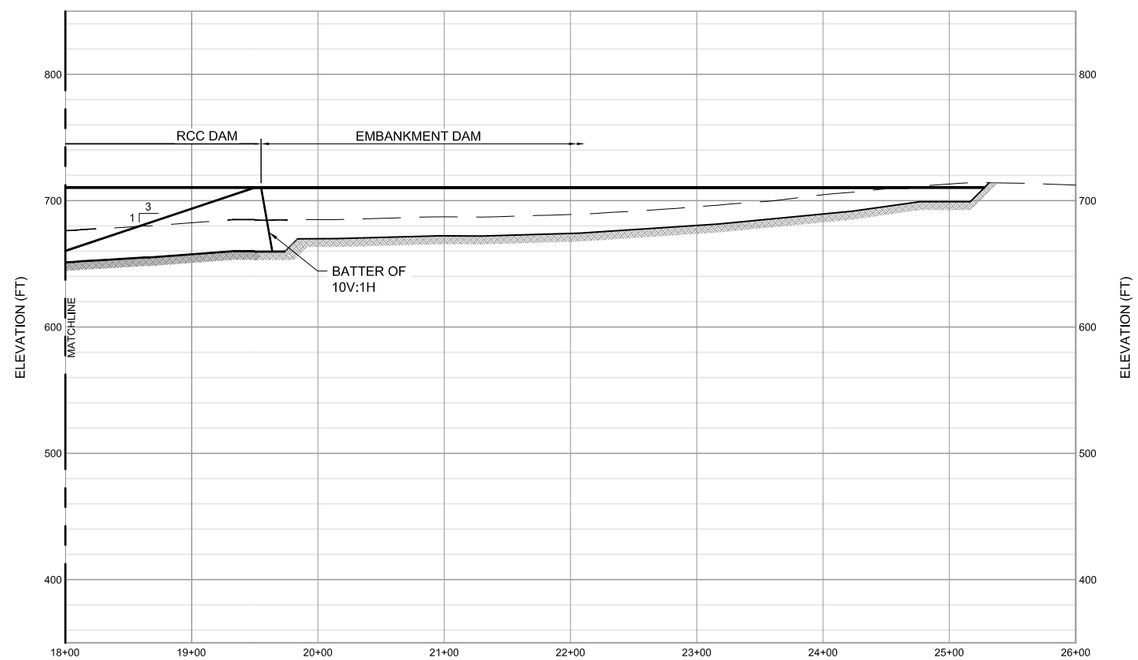
HDR

MULTIPURPOSE - RCC DAM
SPILLWAY TYPICAL CROSS-SECTION

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RCC-2





MP-RCC-3

STATION ALONG AXIS (UPSTREAM FACE) OF DAM (LOOKING DOWNSTREAM) (FEET)

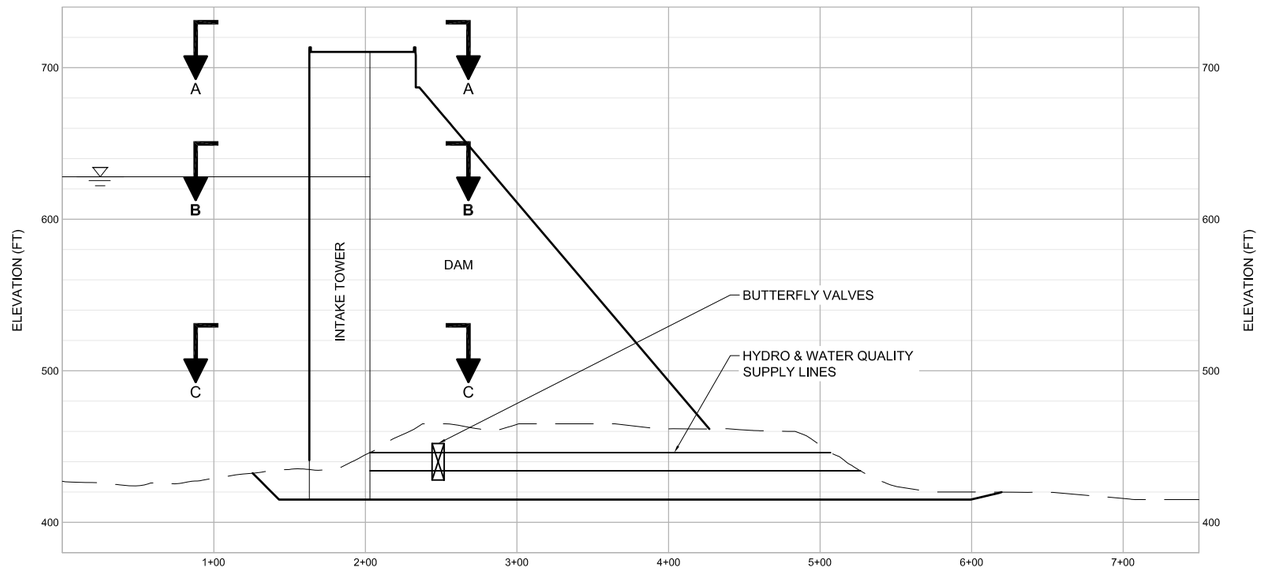
60 0 60 120
SCALE IN FEET

NOTE:
1. SIZE, LOCATION AND DETAILS OF INLET PORTS TO BE DETERMINED DURING FUTURE STUDIES.

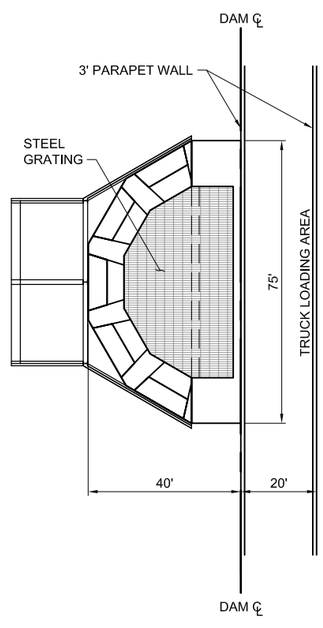
MULTIPURPOSE - RCC DAM CREST PROFILE

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

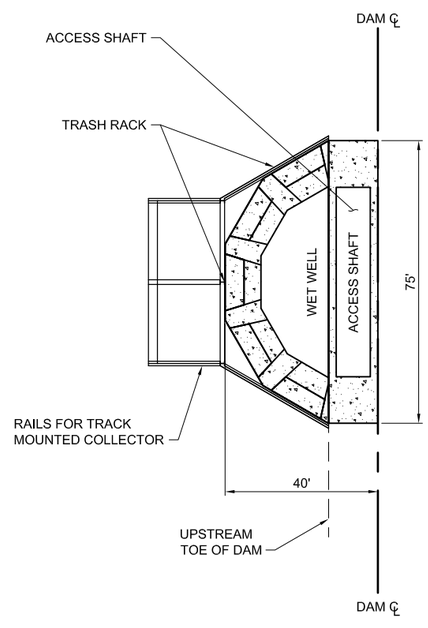
DATE	
JULY 2014	
FIGURE	
MP-RCC-3	



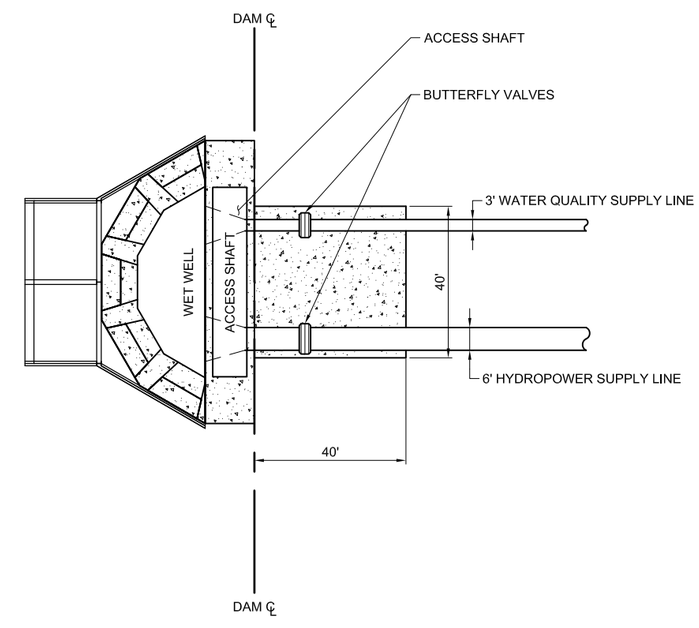
MP-RCC-4



SECTION A-A



SECTION B-B

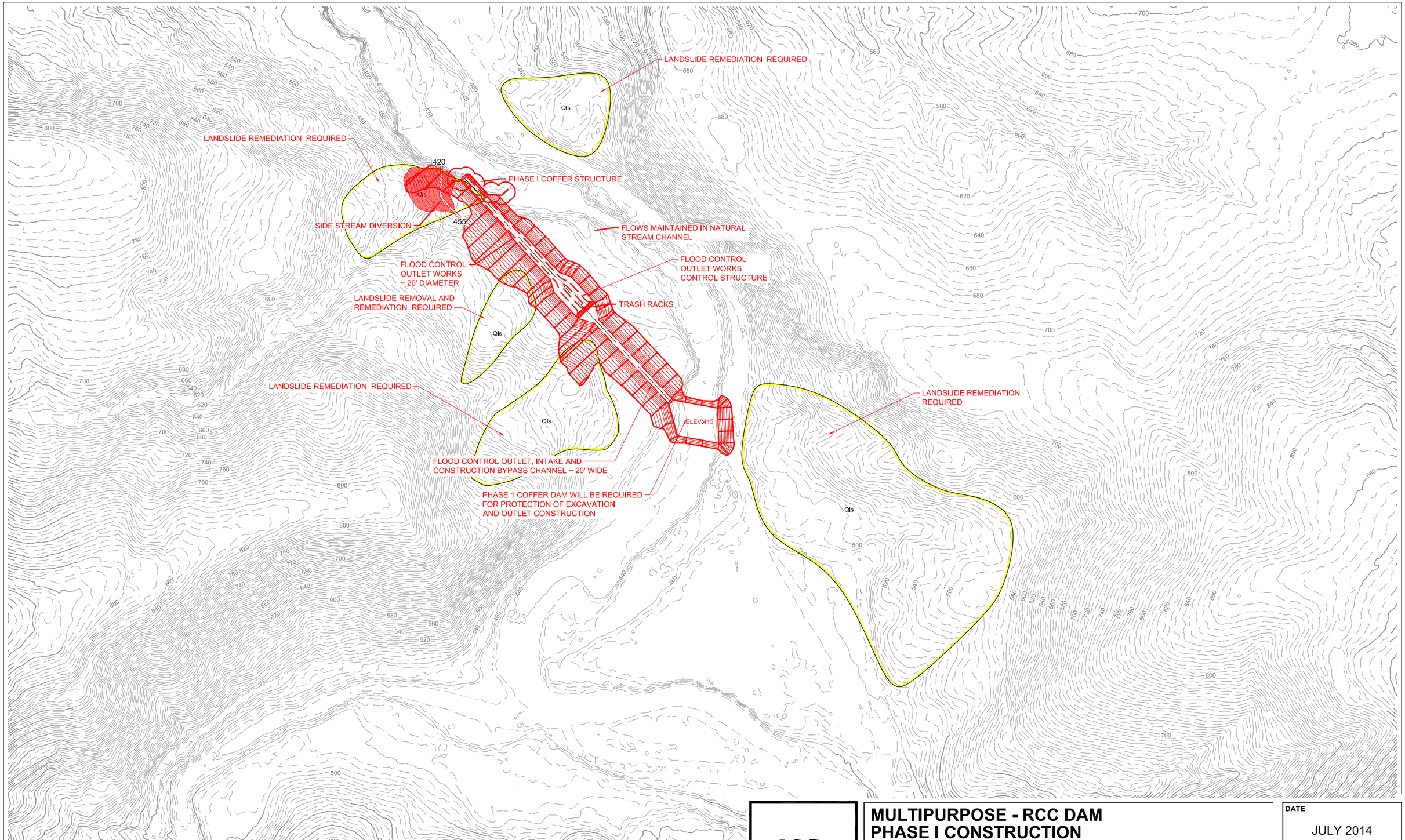


SECTION C-C

MULTIPURPOSE - RCC DAM
WATER QUALITY INTAKE TOWER
CROSS-SECTION AND DETAILS

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RCC-4

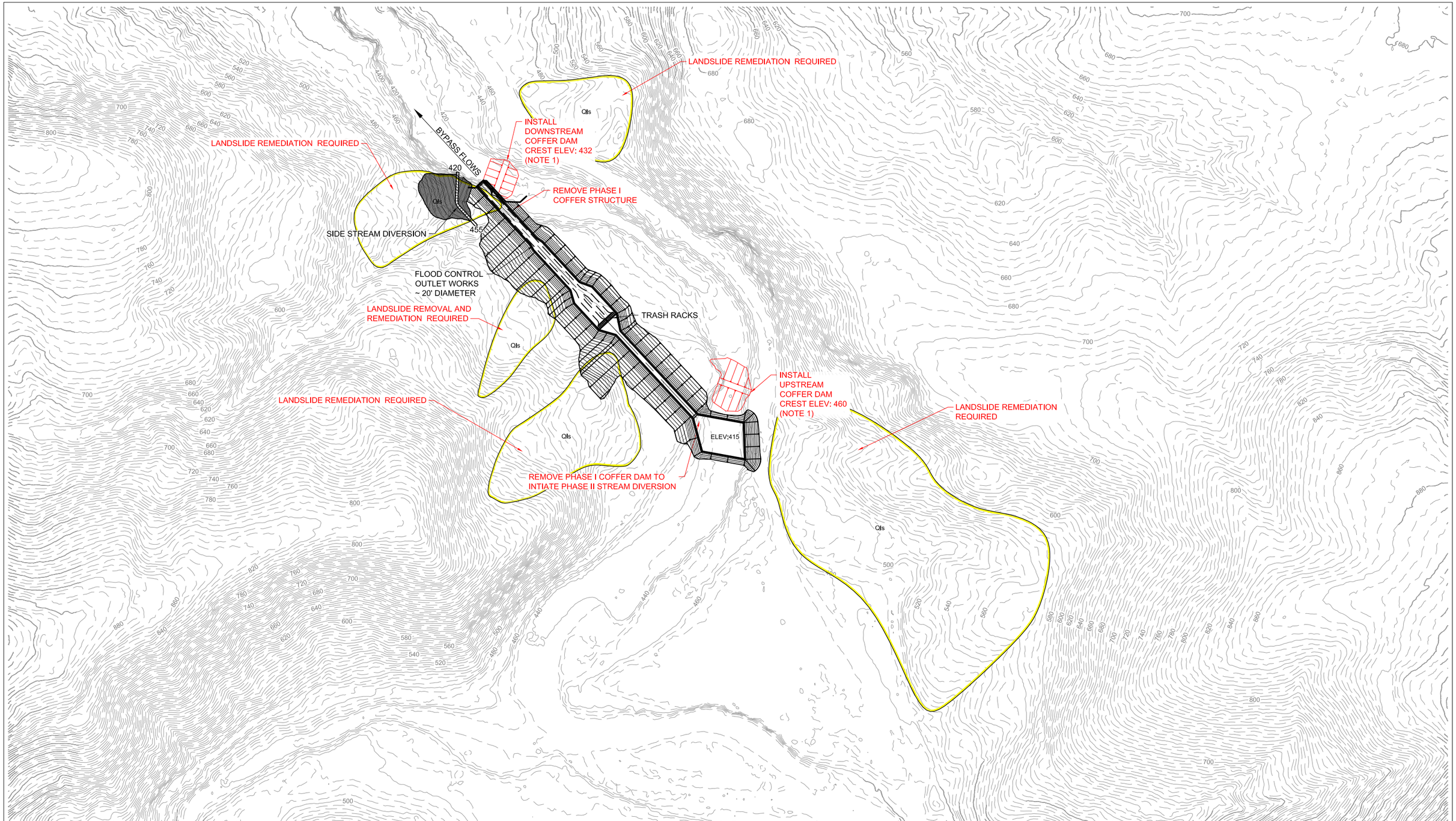


HDR

MULTIPURPOSE - RCC DAM
PHASE I CONSTRUCTION
FLOOD CONTROL OUTLET

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RCC-5



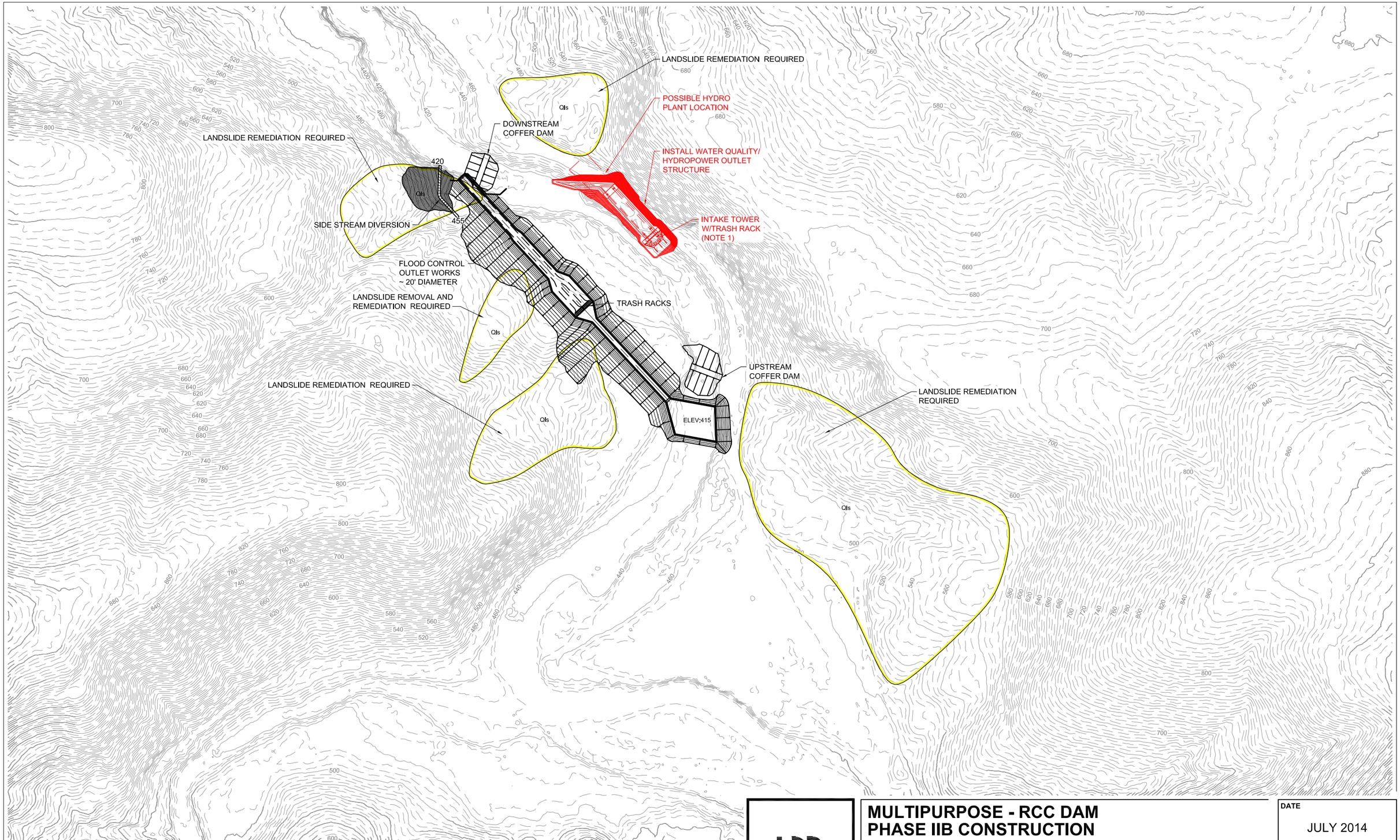
NOTE:
1. CREST OF COFFER DAM TO BE DETERMINED DURING FUTURE STUDY PHASES BASED ON RISK EVALUATIONS.

HDR

MULTIPURPOSE - RCC DAM
PHASE IIA CONSTRUCTION
STREAM DIVERSION

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RCC-6

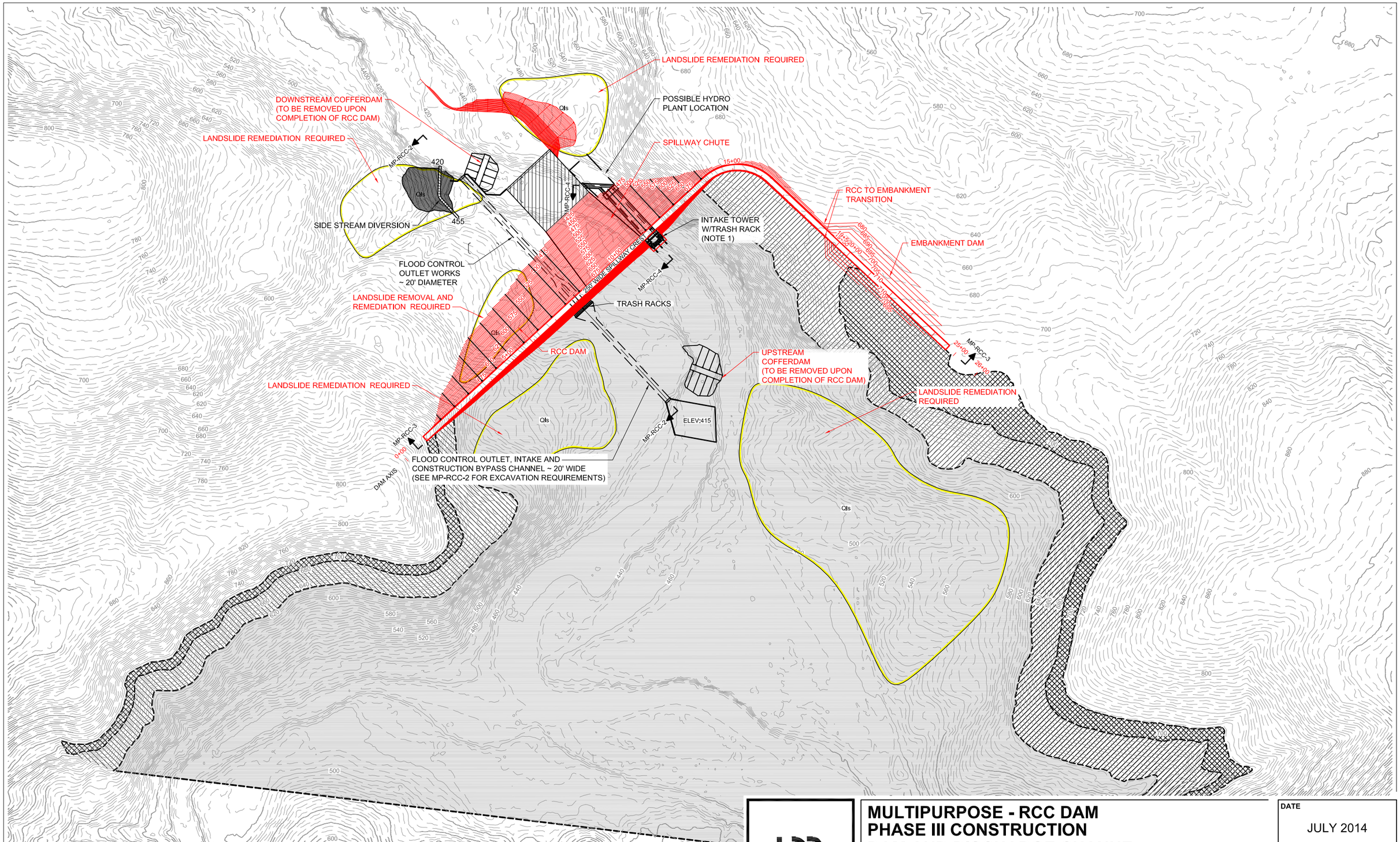


HDR

MULTIPURPOSE - RCC DAM
PHASE IIB CONSTRUCTION
WATER QUALITY/HYDROPOWER
OUTLET WORKS

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
JULY 2014	
FIGURE	
MP-RCC-7	



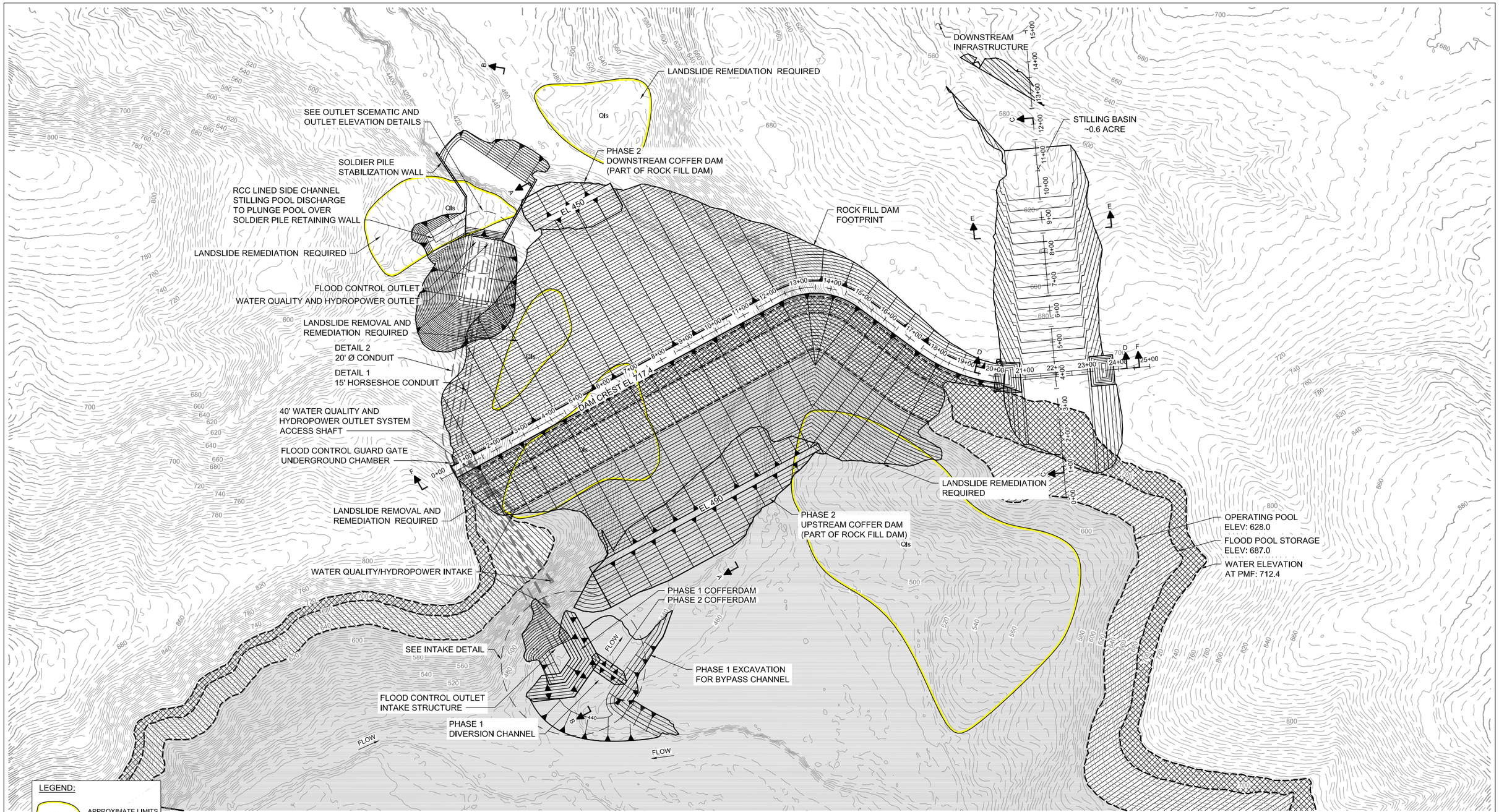
HDR

MULTIPURPOSE - RCC DAM
PHASE III CONSTRUCTION
DAM AND DISCHARGE CHANNE

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RCC-8

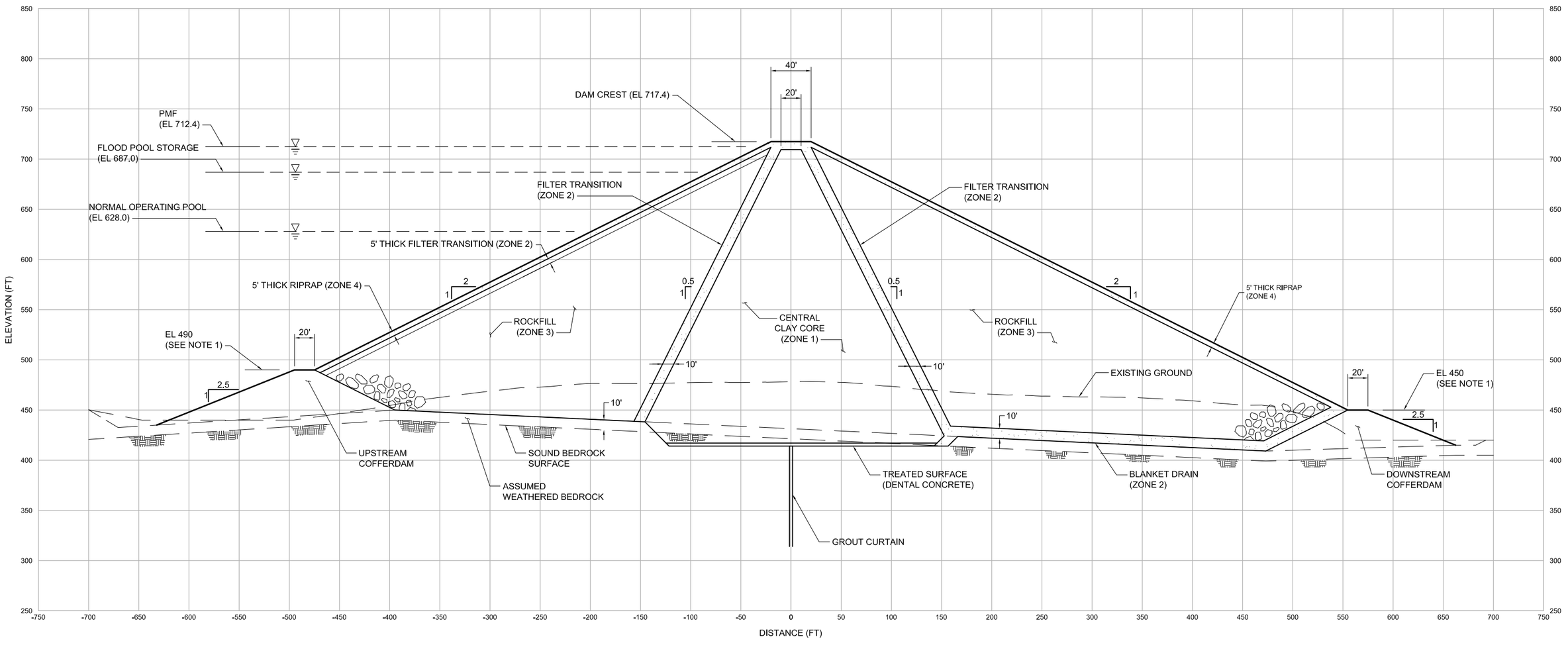
Appendix A.3 – MP-Rockfill



MULTI-PURPOSE - ROCKFILL DAM PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RF-1



SECTION A-A

50 0 50 100
SCALE IN FEET

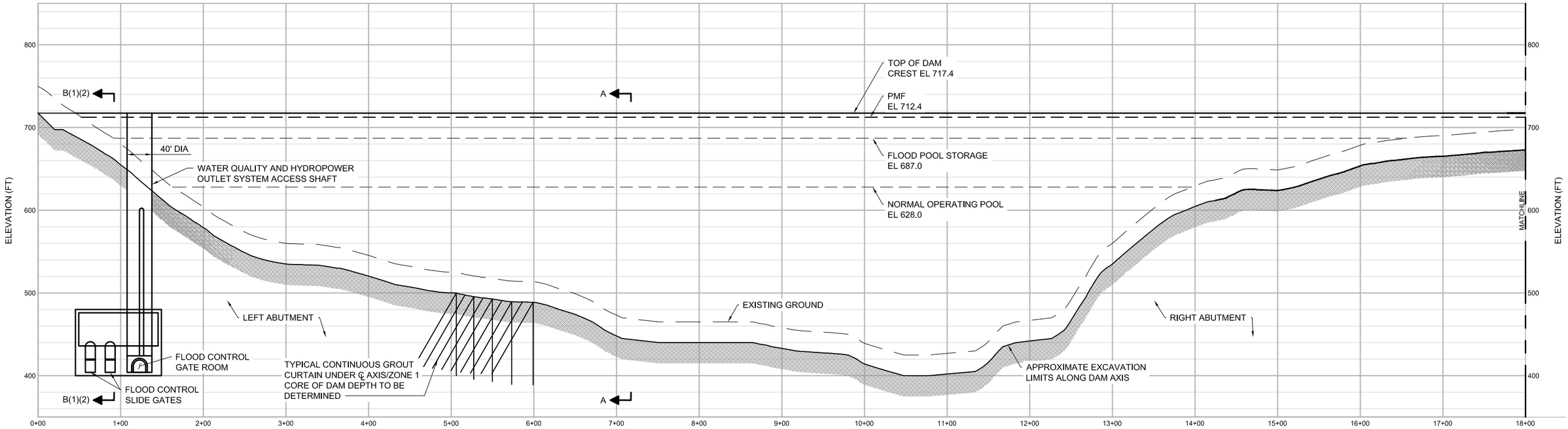
NOTES:
1. CREST OF COFFERDAM'S TO BE DETERMINED DURING FUTURE STUDY PHASE BASED ON RISK EVALUATIONS.

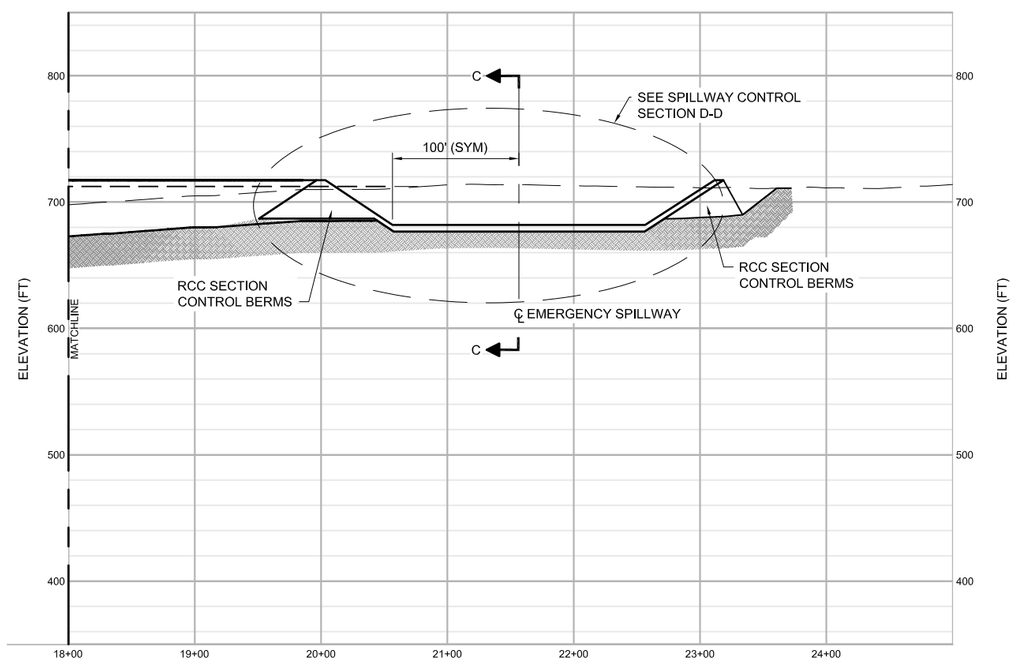
HDR

MULTI-PURPOSE ROCKFILL DAM
CREST PROFILE

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RF-2





SECTION F-F

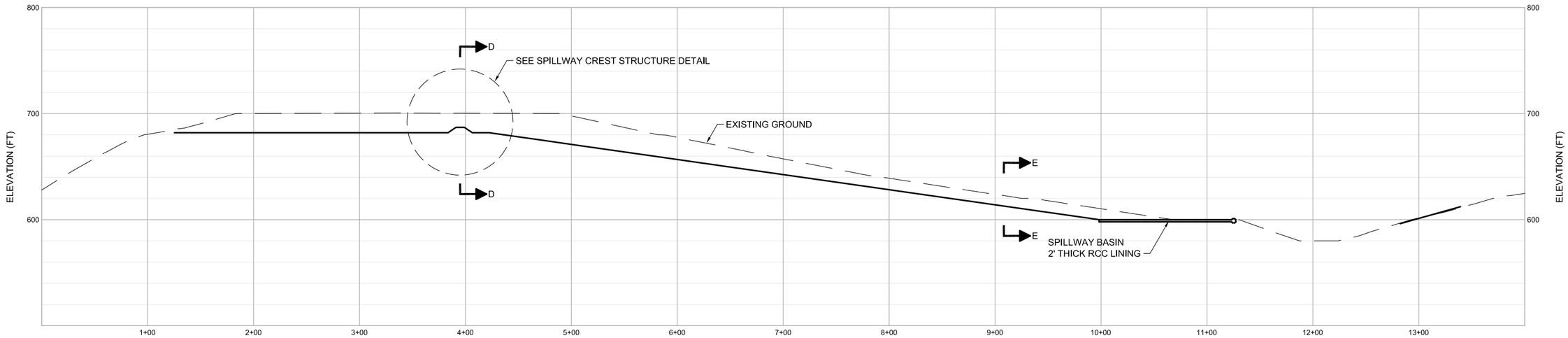
NOTE:
STATION ALONG ℄ AXIS LOOKING DOWNSTREAM

60 0 60 120
SCALE IN FEET

MULTI-PURPOSE ROCKFILL DAM
TYPICAL CROSS-SECTION

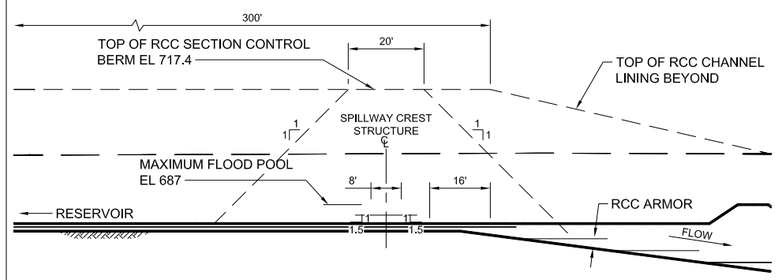
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RF-3



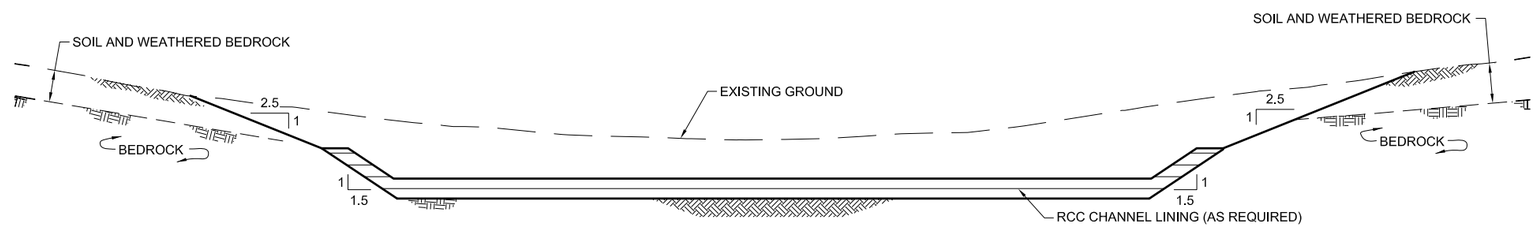
SECTION C-C

50 0 50 100
SCALE IN FEET



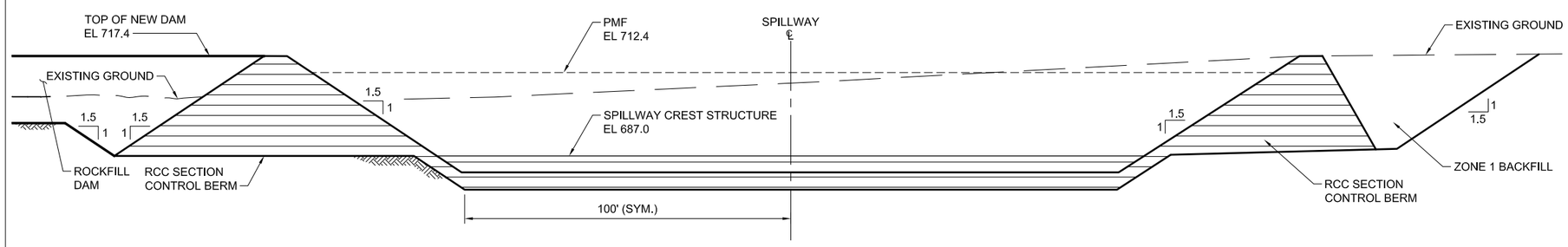
SPILLWAY CREST STRUCTURE DETAIL

20 0 20 40
SCALE IN FEET



SECTION E-E

20 0 20 40
SCALE IN FEET



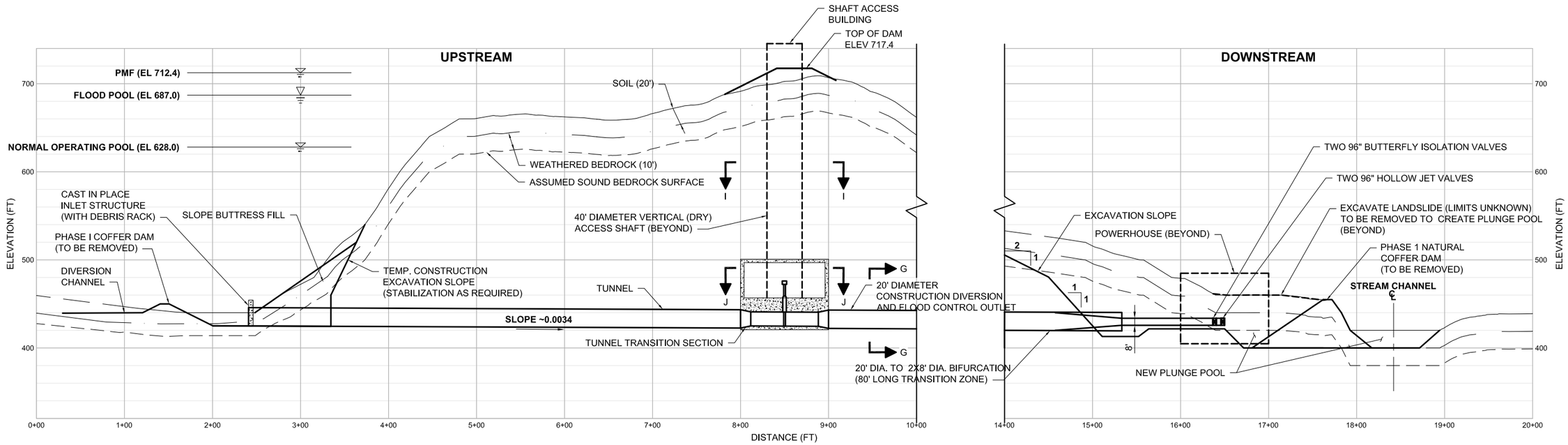
SECTION D-D
SPILLWAY CONTROL SECTION

20 0 20 40
SCALE IN FEET

MULTI-PURPOSE ROCKFILL DAM
AUXILIARY SPILLWAY PROFILE,
SECTION AND DETAILS

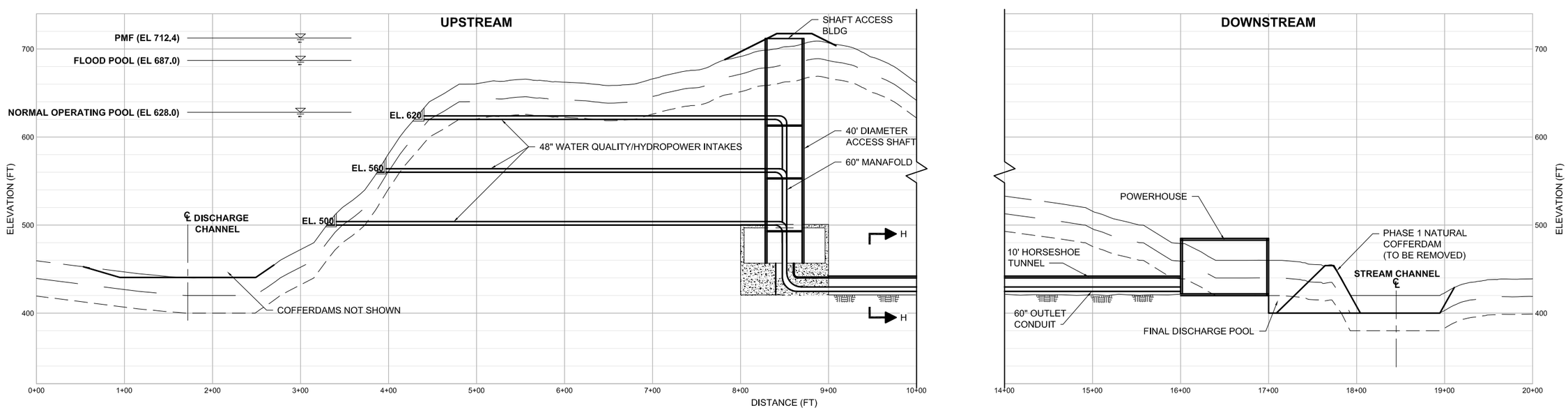
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
MP-RF-4



SECTION B-B (1)
FLOOD CONTROL OUTLET

SCALE: 1" = 60'



SECTION B-B (2)
WATER QUALITY AND HYDROPOWER OUTLET

SCALE: 1" = 60'

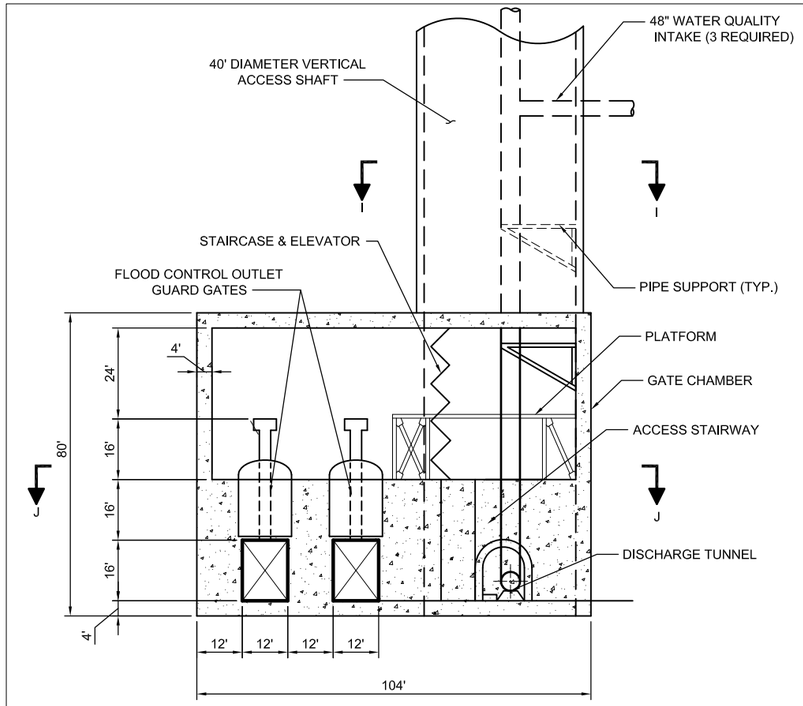
60 0 60 120
SCALE IN FEET

HDR

MULTI-PURPOSE ROCKFILL DAM
FLOOD CONTROL, WATER QUALITY AND HYDROPOWER OUTLET WORKS CROSS-SECTIONS

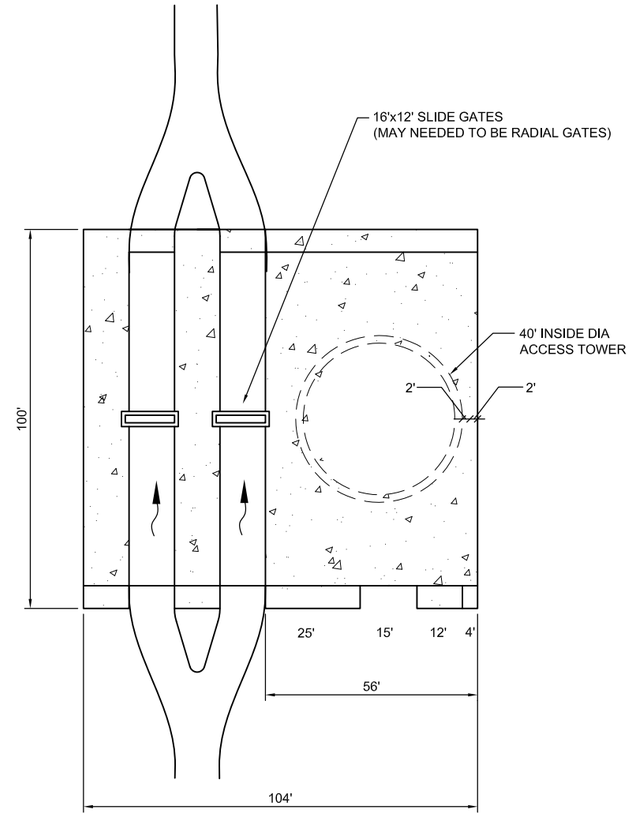
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
	JULY 2014
FIGURE	
	MP-RF-5



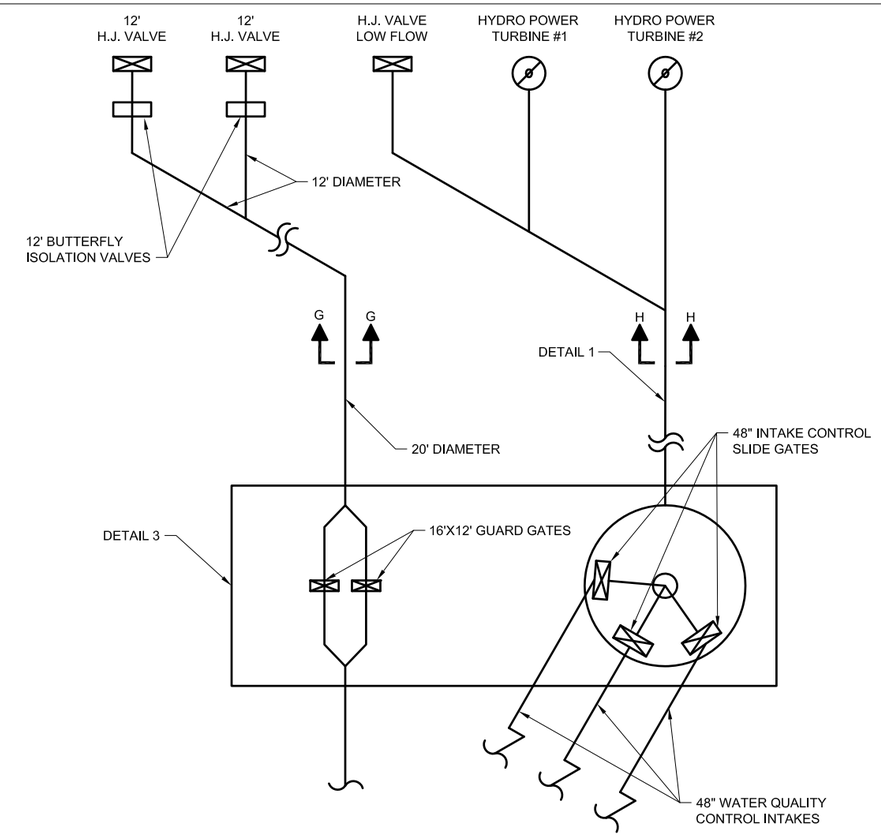
DETAIL 3
FLOOD CONTROL GATE ROOM AND WATER QUALITY CONTROL OUTLET TOWER

20 0 20 40
SCALE IN FEET

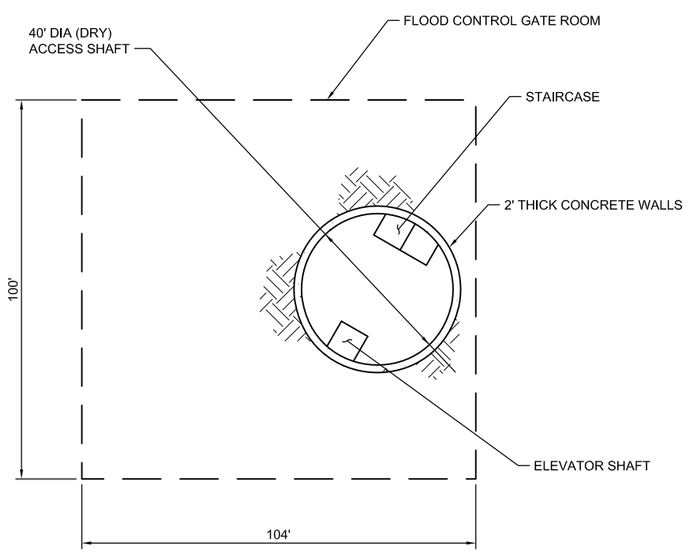


SECTION H-H

20 0 20 40
SCALE IN FEET



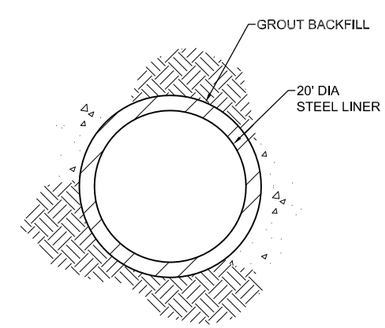
OUTLET SCHEMATIC
NTS



SECTION I-I

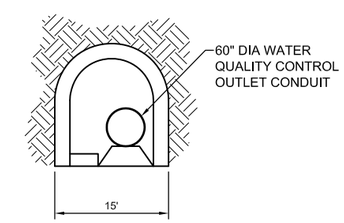
20 0 20 40
SCALE IN FEET

NOTE:
H.J. = HOLLOW JET VALVE



SECTION G-G
TYPICAL FLOOD CONTROL OUTLET TUNNEL

10 0 10 20
SCALE IN FEET



SECTION H-H
TYPICAL WATER QUALITY CONTROL CONDUIT TUNNEL

10 0 10 20
SCALE IN FEET

HDR

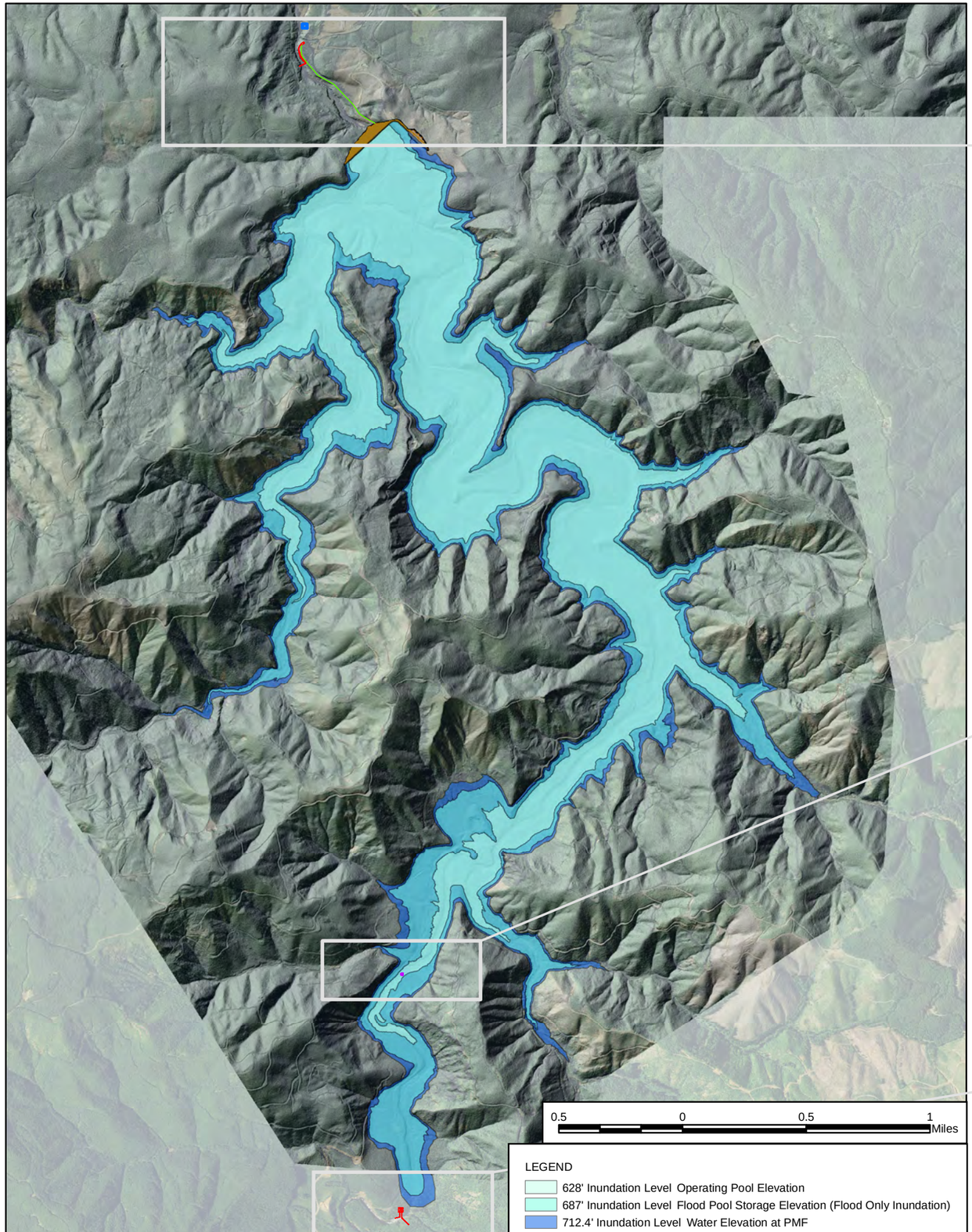
MULTI-PURPOSE ROCKFILL DAM
FLOOD CONTROL, WATER QUALITY AND HYDROPOWER OUTLET WORKS DETAILS

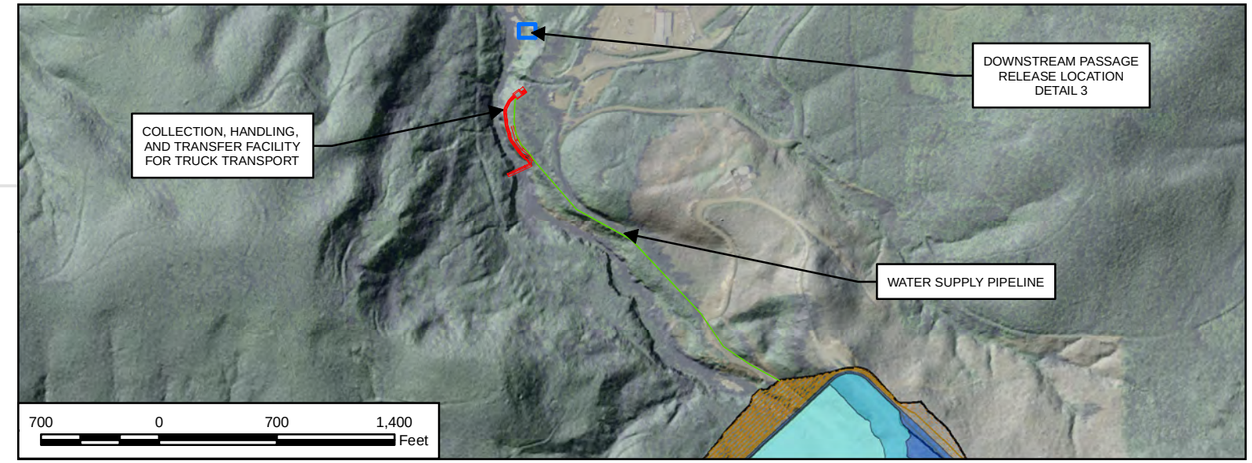
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

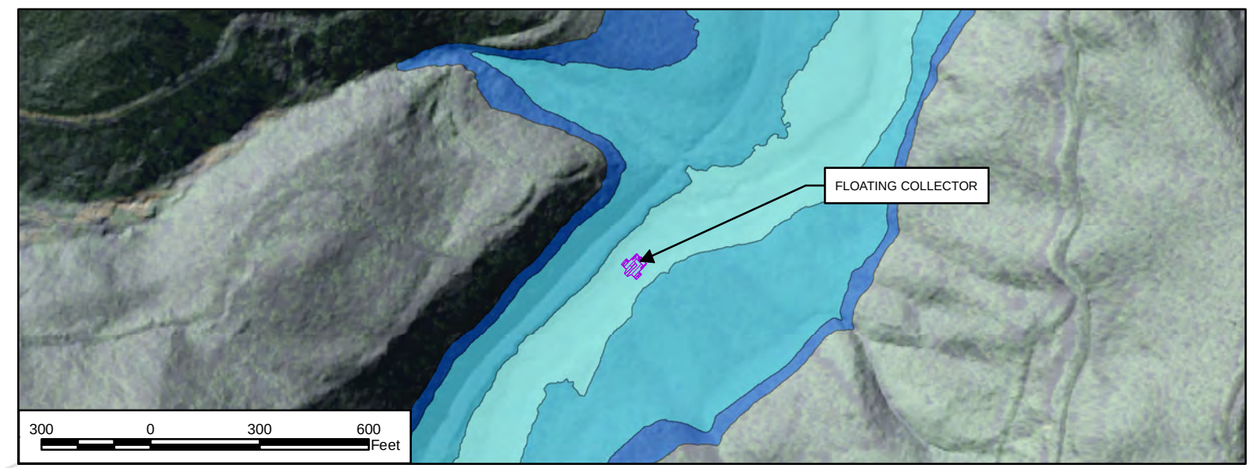
DATE	
	JULY 2014
FIGURE	
	MP-RF-6

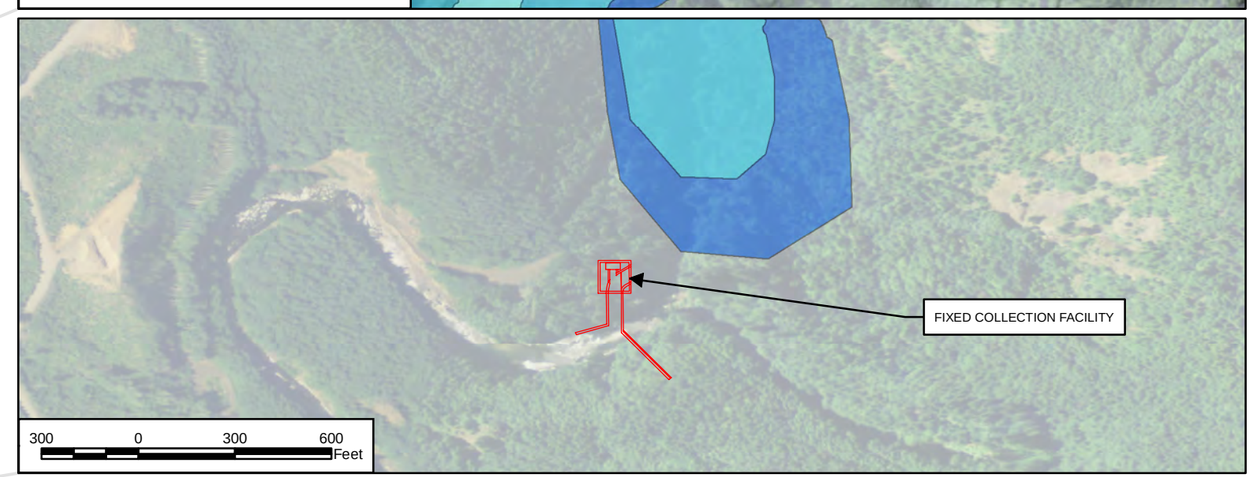
Appendix B – Combined Dam and Fish Passage Drawings

Appendix B.2 – Alternative B









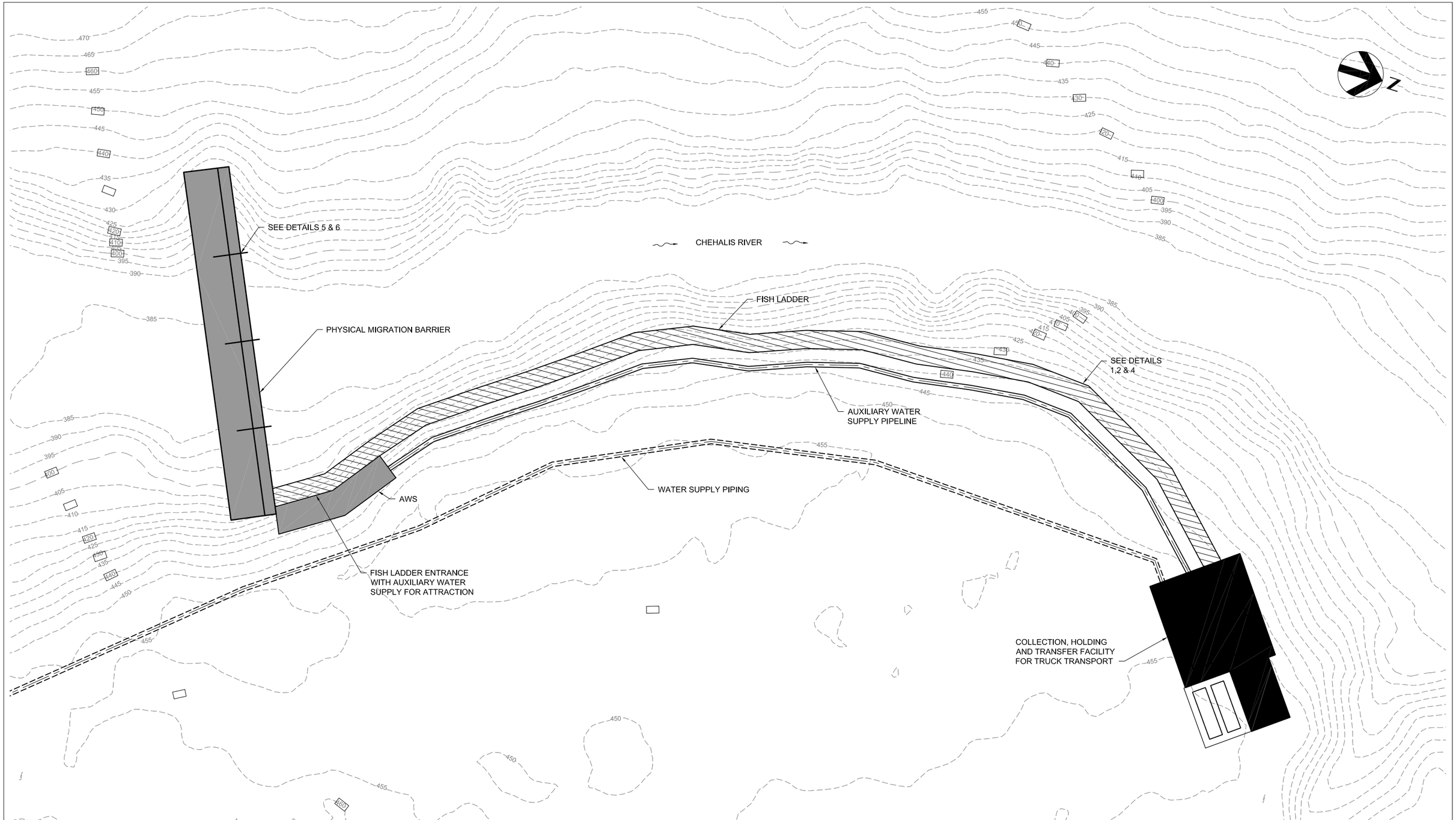
N

HDR

MULTI-PURPOSE - RCC DAM
INTEGRATED STRUCTURE PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	FIGURE
JULY 2014	ALT-B-1

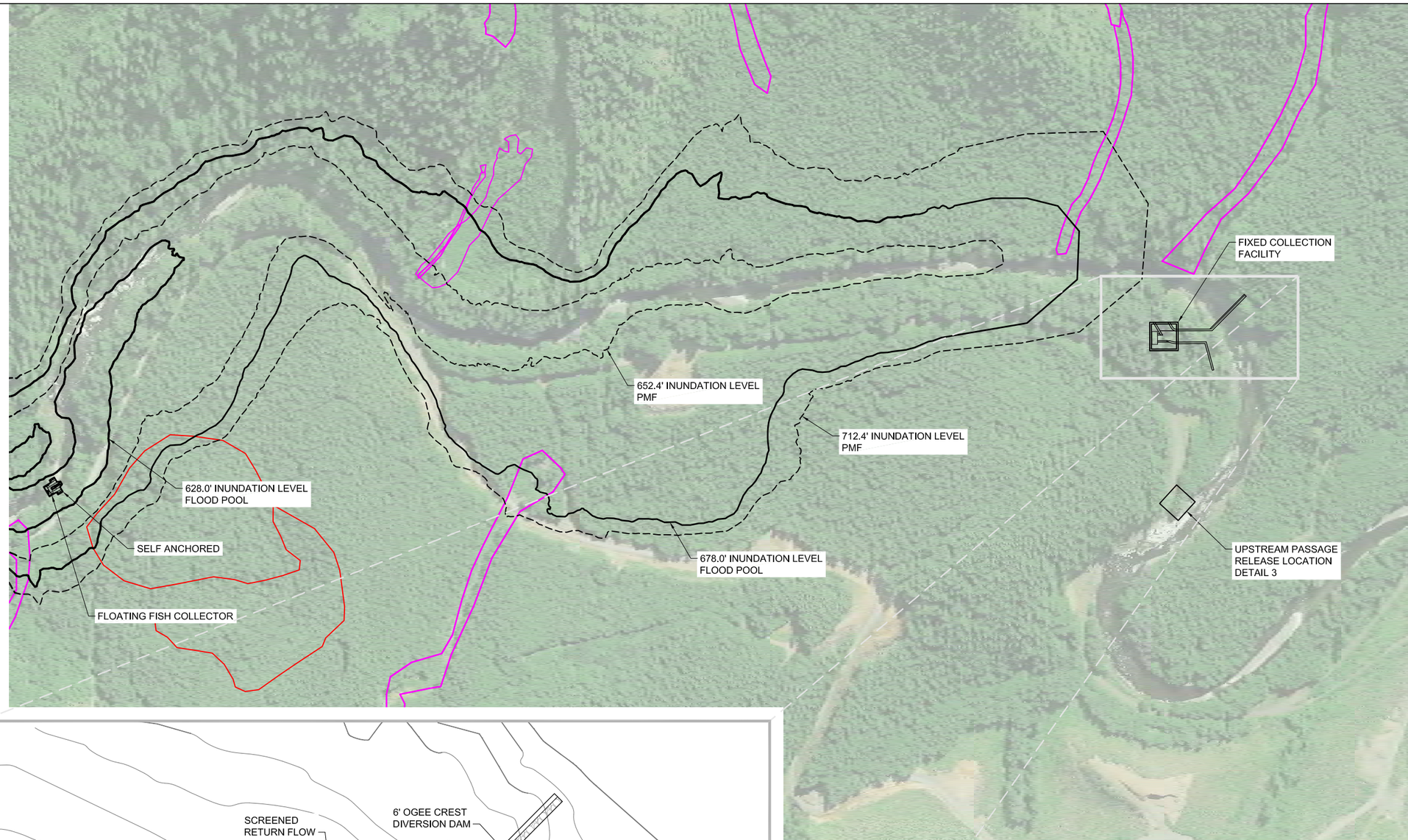


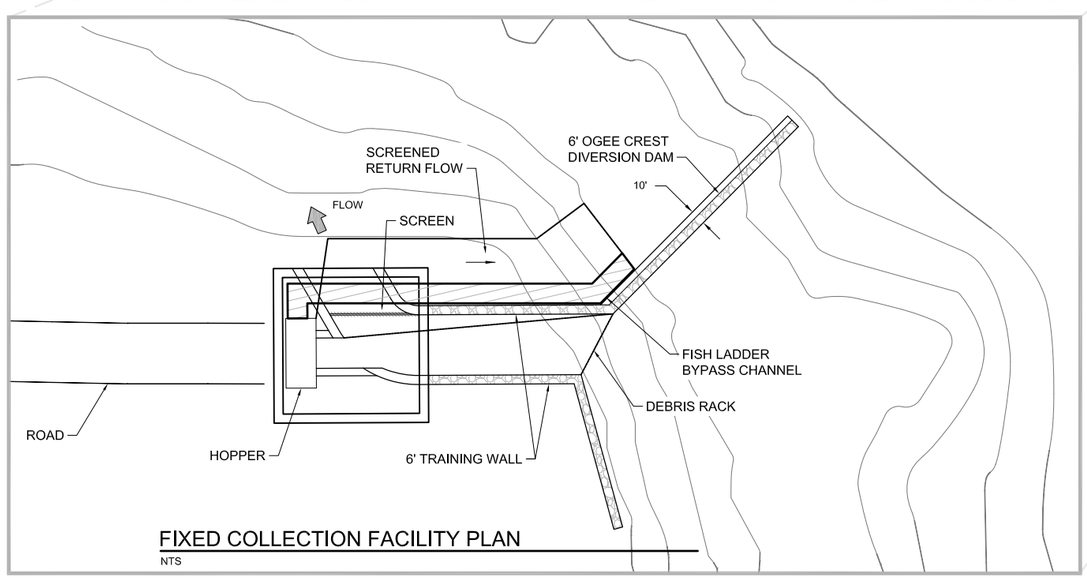
20 0 20 40
SCALE IN FEET

MULTIPURPOSE - RCC DAM
CHTR FACILITY PLAN

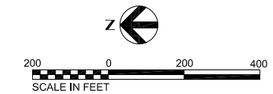
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
	JULY 2014
FIGURE	
	ALT-B-3





FIXED COLLECTION FACILITY PLAN
NTS



LEGEND:

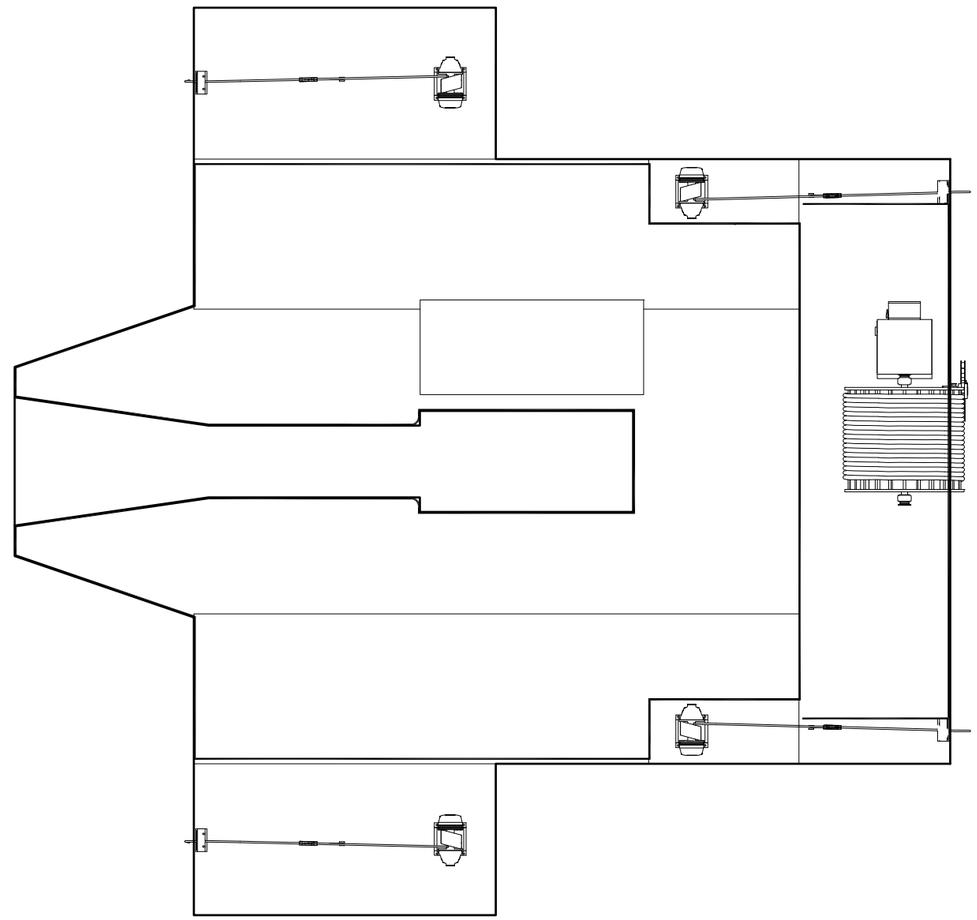
- 628.0' INUNDATION LEVEL - OPERATING POOL
- 678.0' INUNDATION LEVEL - FLOOD POOL
- 712.4' INUNDATION LEVEL - PMF
- GEODYNAMICS_LNDFRMREV
- S&W_LIDAR_LNDFRMREV
- HEADWATERS_WATERSHED_LS_INV

HDR

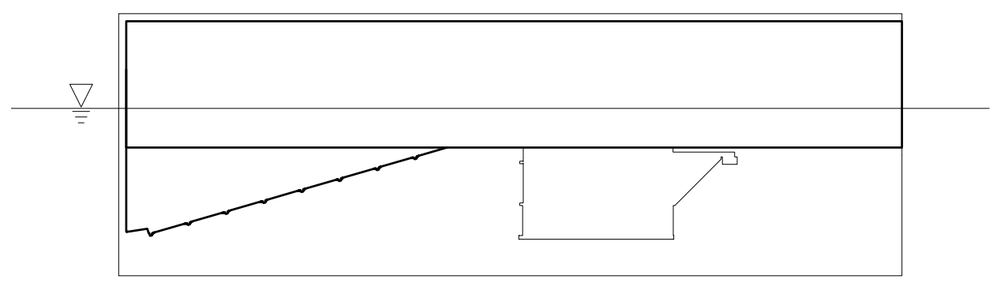
MULTIPURPOSE - RCC DAM
UPSTREAM FIXED COLLECTION
FACILITY PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
	JULY 2014
FIGURE	
	ALT-B-4



PLAN

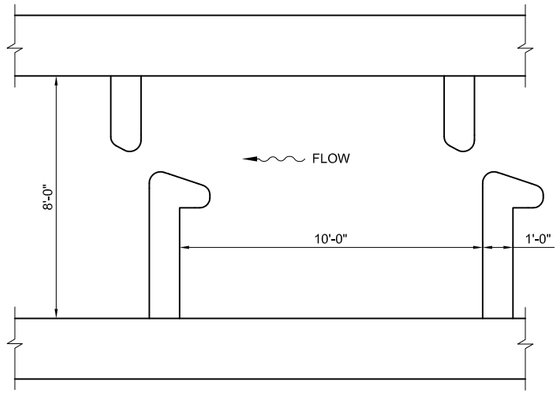


SECTION

MULTIPURPOSE - RCC DAM
PORTABLE FLOATING FOREBAY COLLECTOR
PLAN AND SECTION

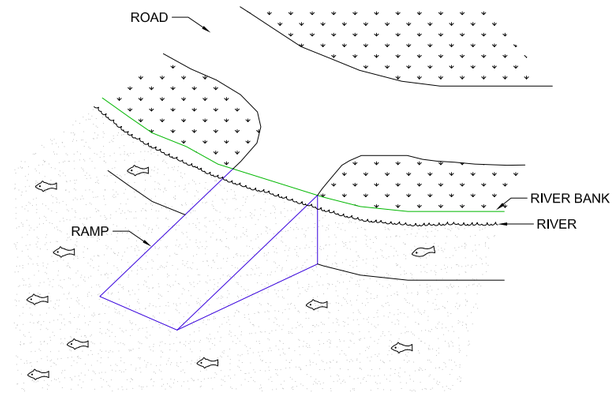
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	JULY 2014
FIGURE	ALT-B-5



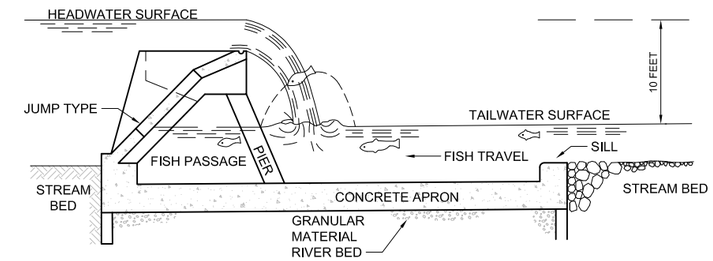
VERTICAL SLOT PLAN

1 ALT-B-3



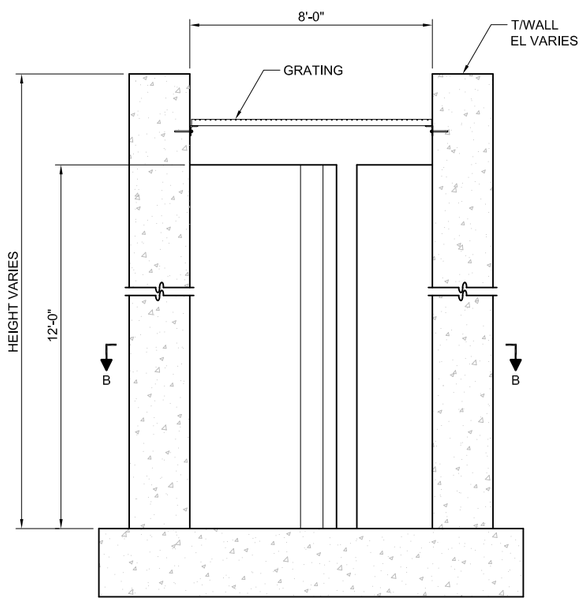
RAMP RELEASE DETAIL FOR
UPSTREAM & DOWNSTREAM PASSAGE

3 ALT-B-4



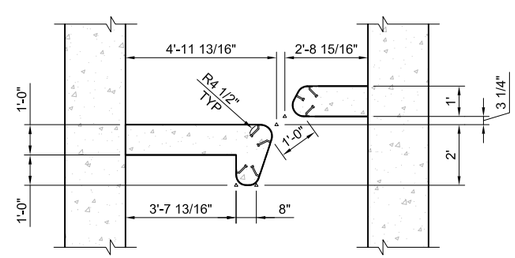
TYPICAL CROSS SECTION OF TYPICAL
BARRIER DAM

5 ALT-B-3



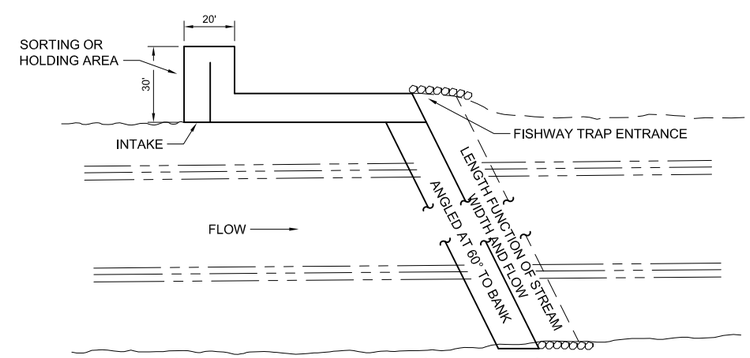
VERTICAL SLOT WALL ELEVATION

2 ALT-B-3



VERTICAL SLOT - SECTION B-B

4 ALT-B-3



TYPICAL PLAN VIEW OF TYPICAL BARRIER
DAM PLACED IN A RIVER

6 ALT-B-3

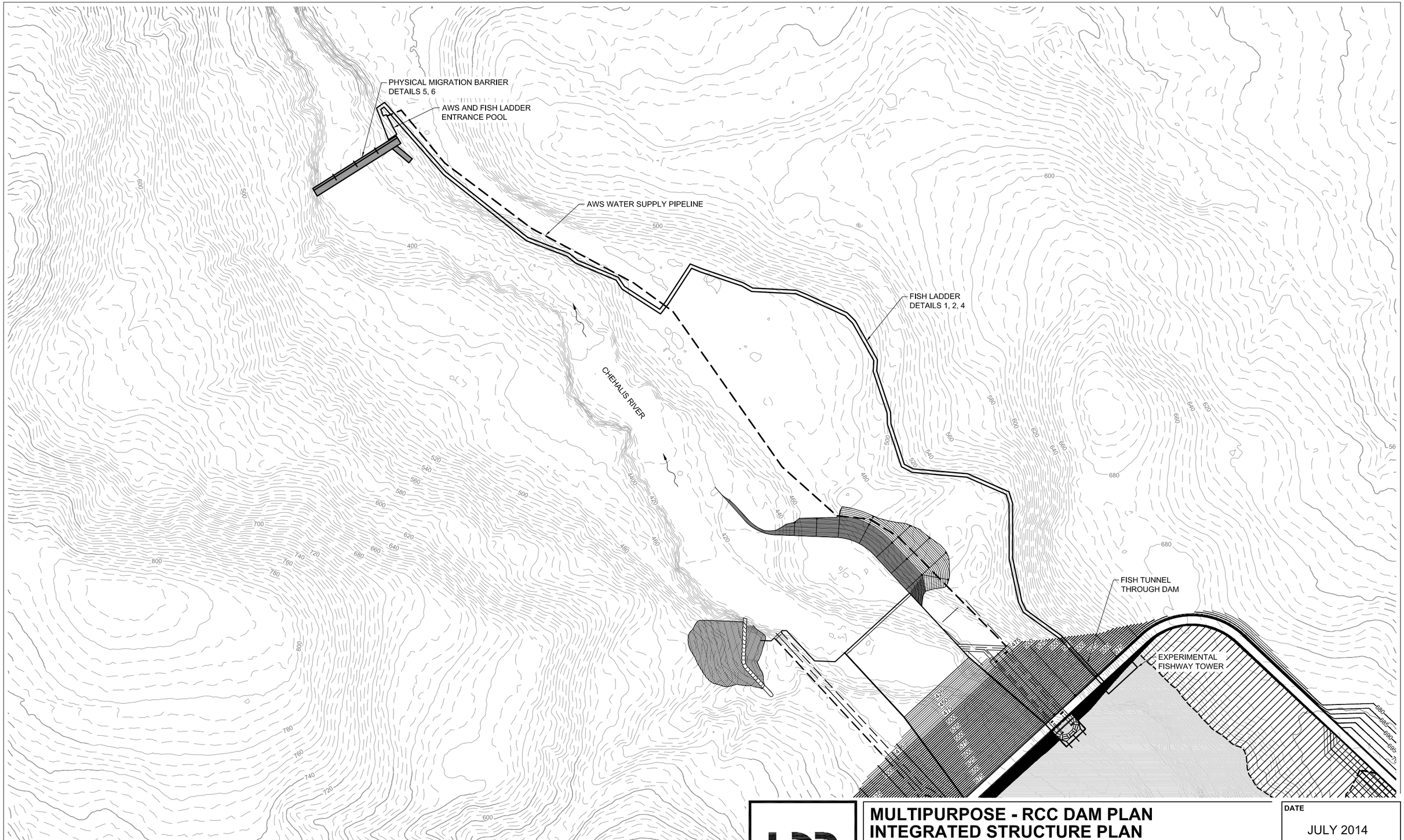
30 0 30 60
SCALE IN FEET

MULTIPURPOSE - RCC DAM
FISH PASSAGE DETAILS

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
ALT-B-6

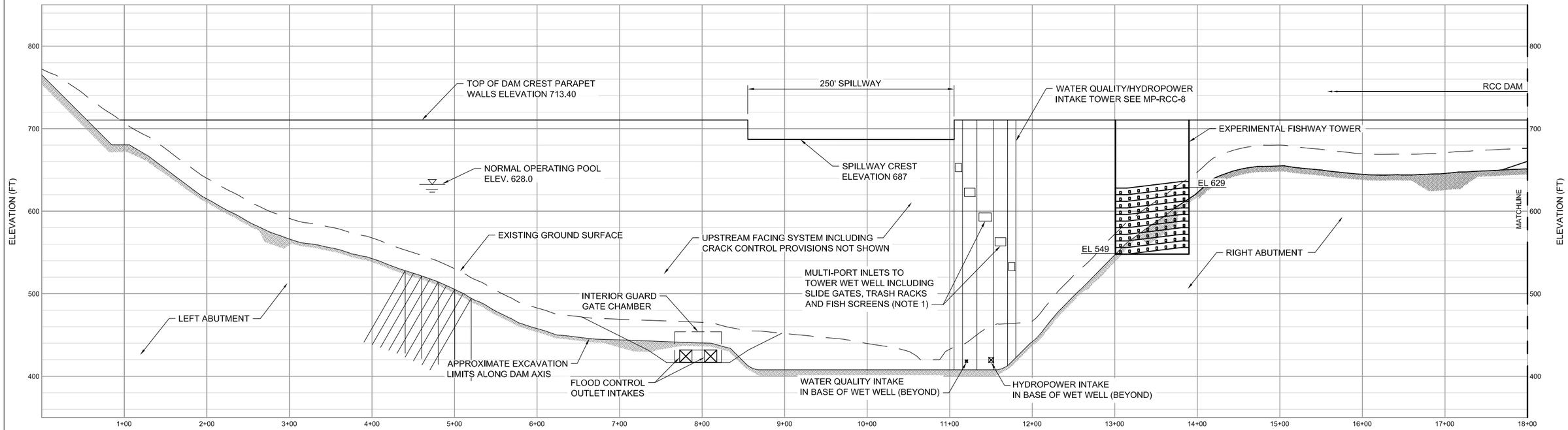
Appendix B.3 – Alternative C

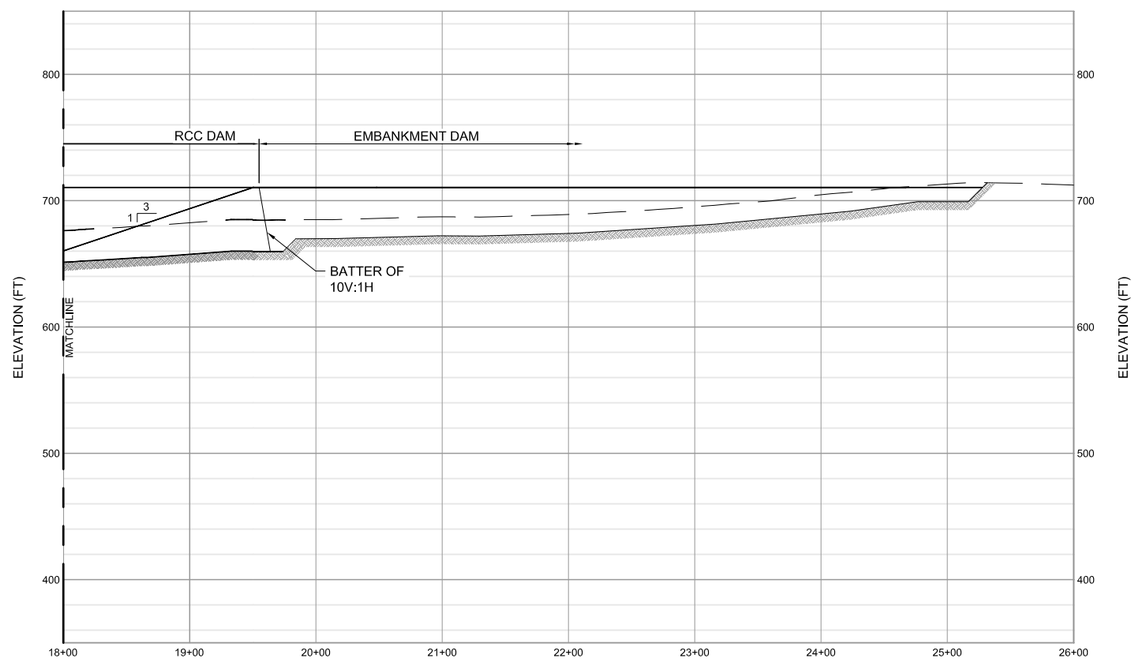


MULTIPURPOSE - RCC DAM PLAN
INTEGRATED STRUCTURE PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
ALT-C-1





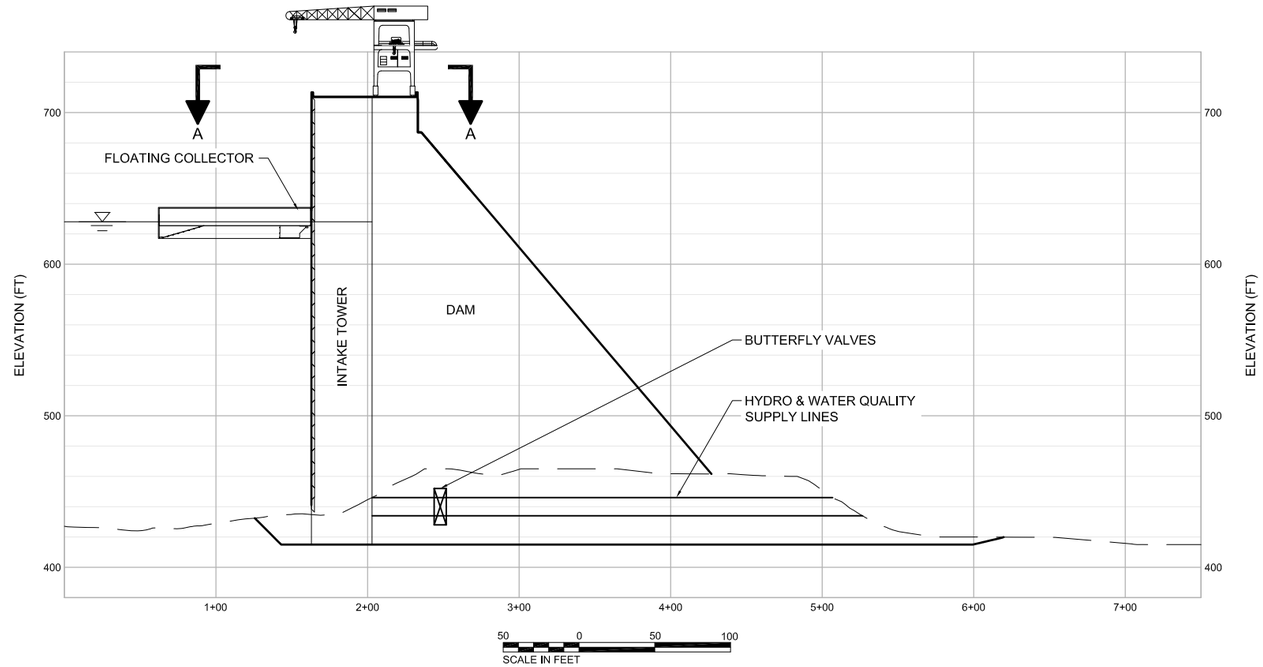
NOTE:
1. SIZE, LOCATION AND DETAILS OF INLET PORTS TO BE DETERMINED DURING FUTURE STUDIES.

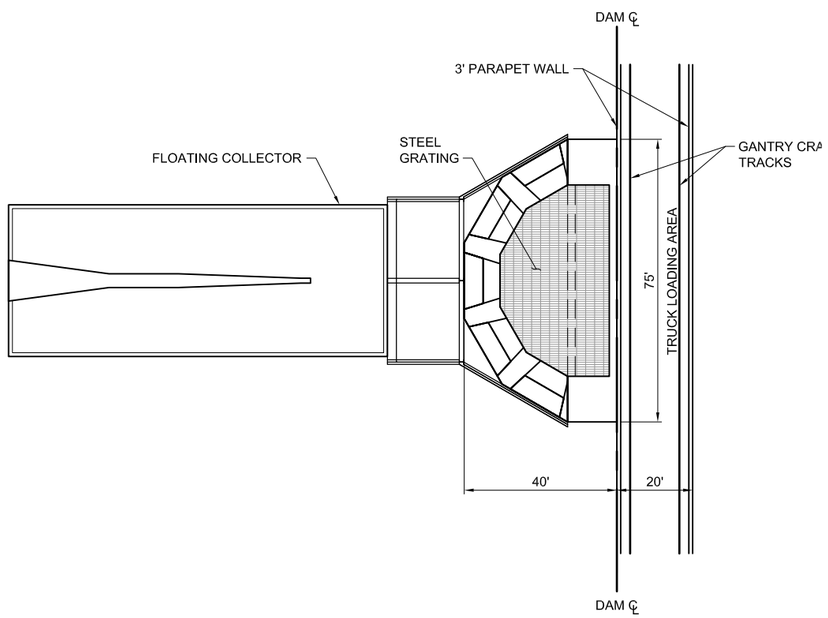
60 0 60 120
SCALE IN FEET

MULTIPURPOSE - RCC DAM PLAN
DAM CREST PROFILE
WITH EXPERIMENTAL FISHWAY TOWER

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

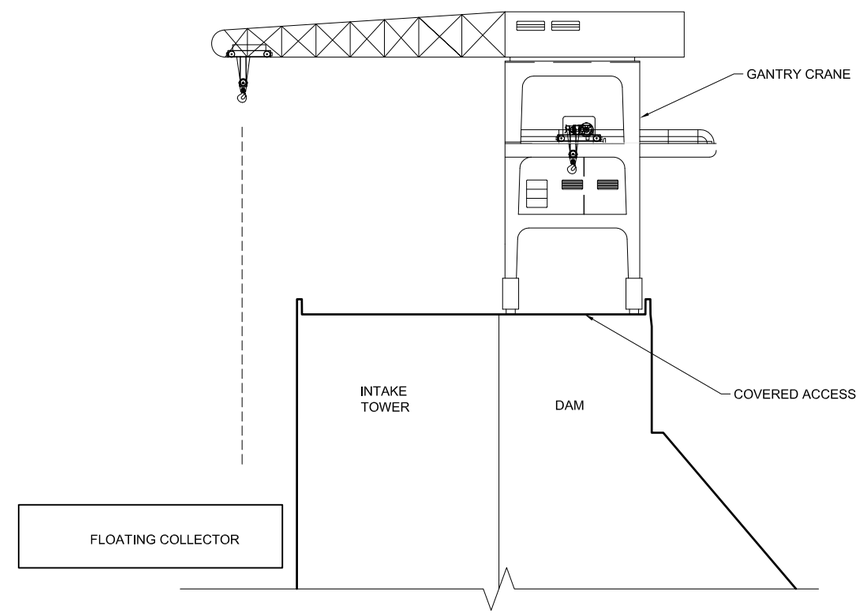
DATE
JULY 2014
FIGURE
ALT-C-2





SECTION A-A

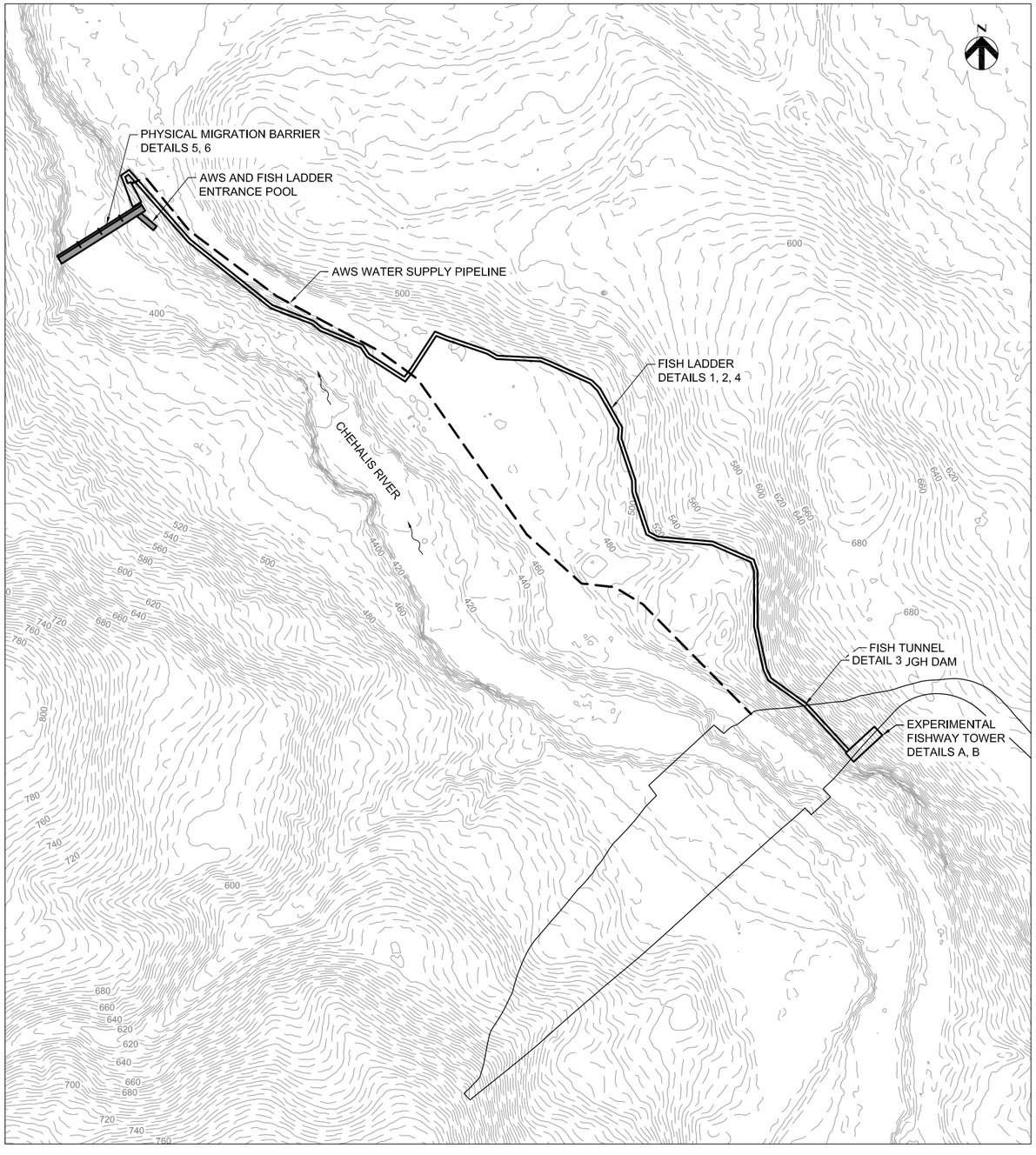
20 0 20 40
SCALE IN FEET



MULTIPURPOSE - RCC DAM PLAN
WATER QUALITY INTAKE TOWER
WITH FLOATING FOREBAY COLLECTOR

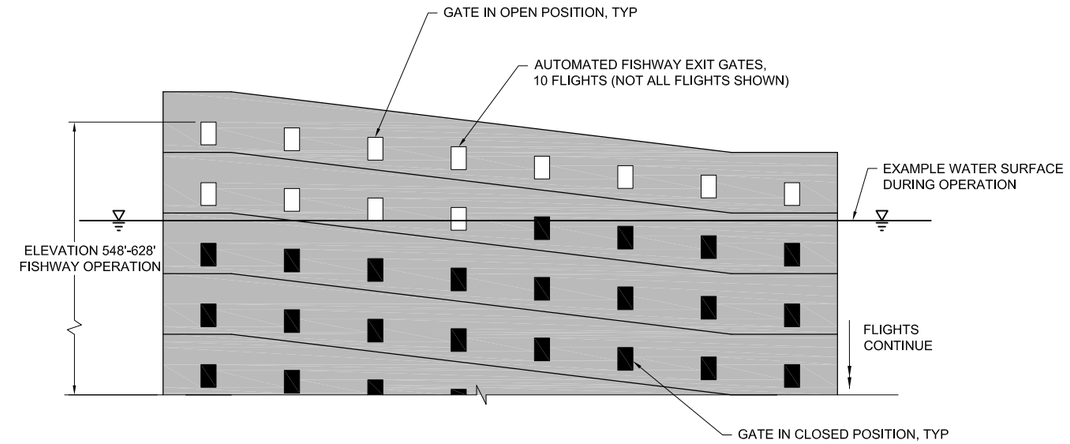
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	JULY 2014
FIGURE	ALT-C-3



INTEGRATED STRUCTURE PLAN

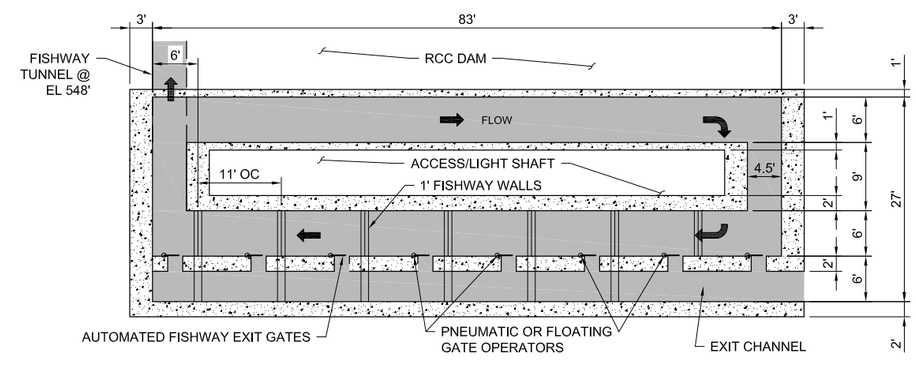
150 0 150 300
SCALE IN FEET



FISHWAY EXIT STRUCTURE - ELEVATION

A

10 0 10 20
SCALE IN FEET



FISHWAY EXIT STRUCTURE - PLAN

(DIMENSIONS APPROXIMATED)

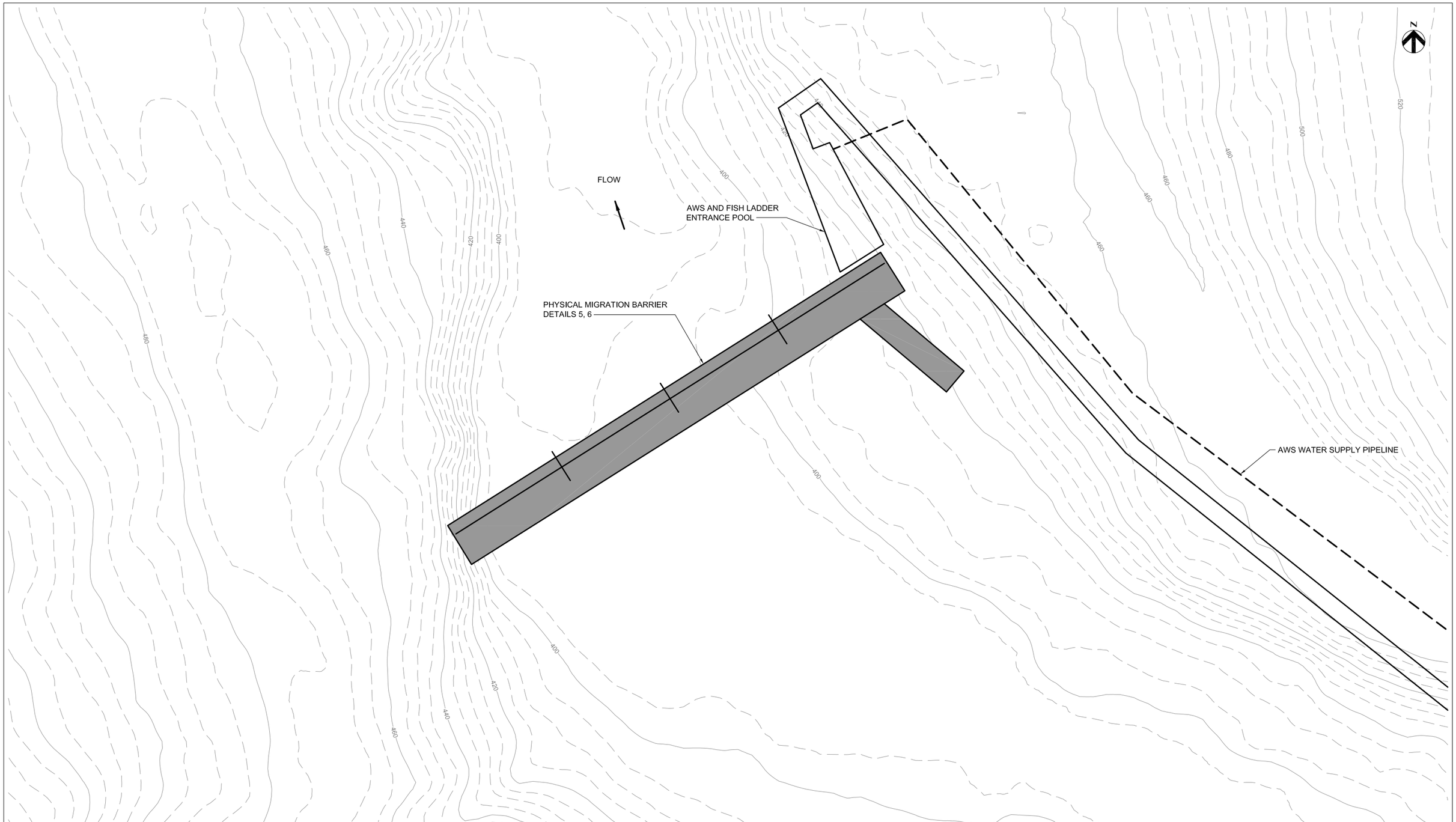
B

10 0 10 20
SCALE IN FEET

MULTIPURPOSE - RCC DAM PLAN
EXPERIMENTAL FISHWAY TOWER
EXIT STRUCTURE

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

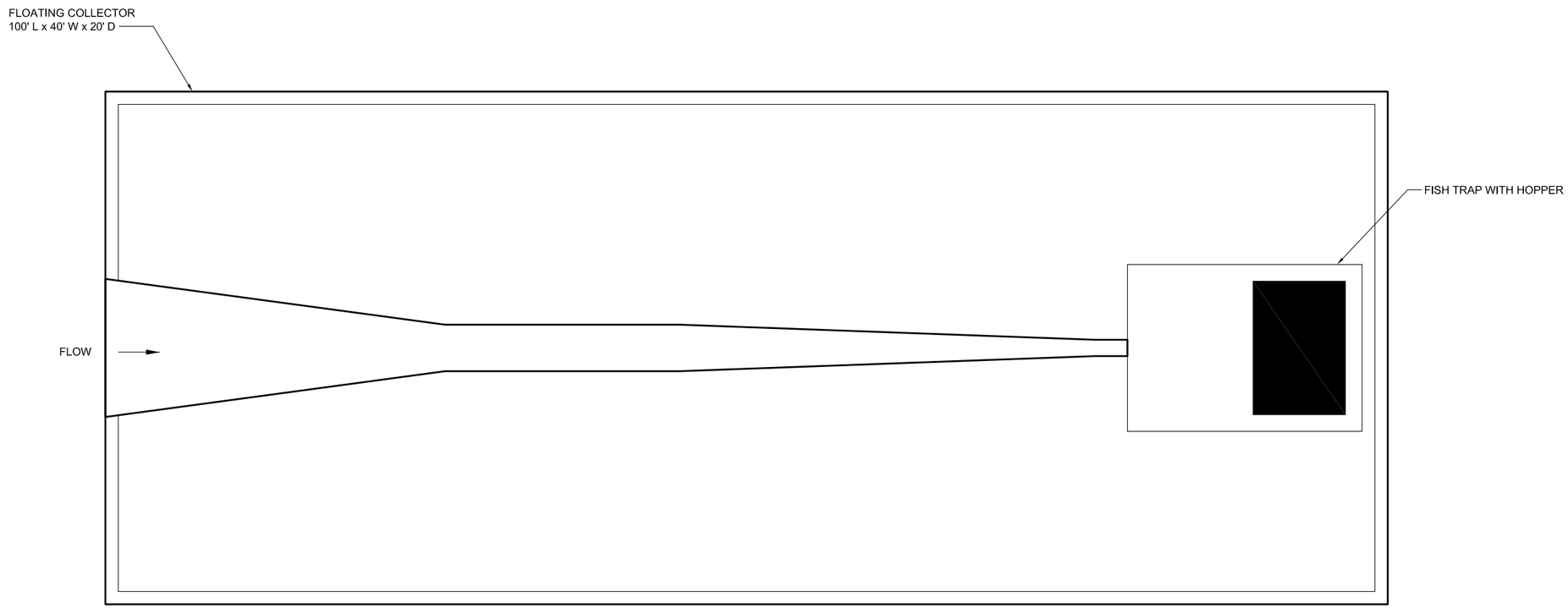
DATE
JULY 2014
FIGURE
ALT-C-4



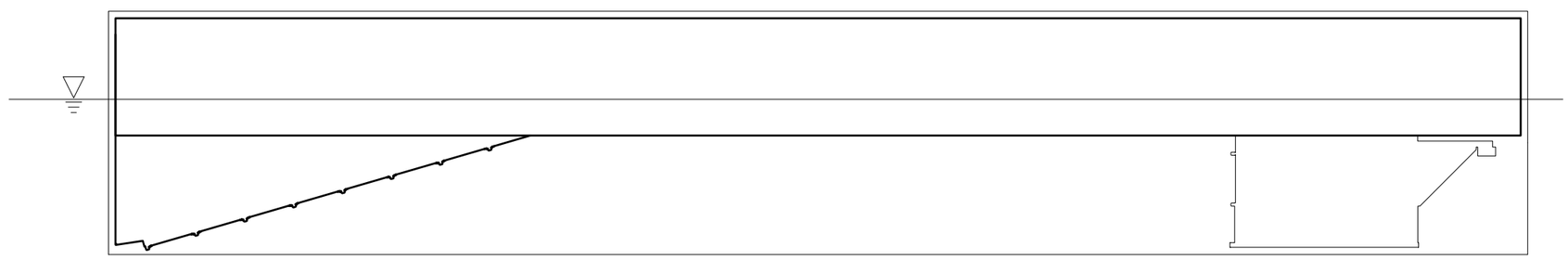
MULTIPURPOSE - RCC DAM PLAN
FISH LADDER ENTRANCE PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	JULY 2014
FIGURE	ALT-C-5



PLAN

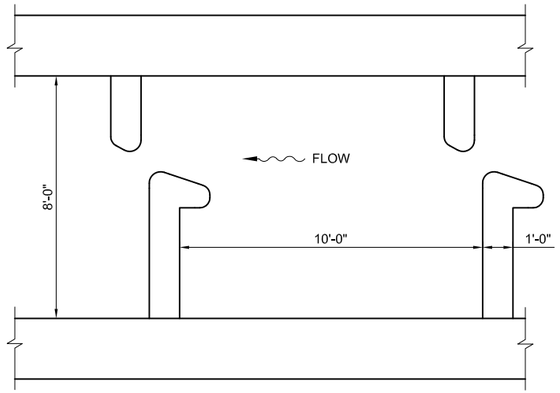


SECTION

MULTIPURPOSE - RCC DAM
FLOATING FOREBAY COLLECTOR
PLAN AND SECTION

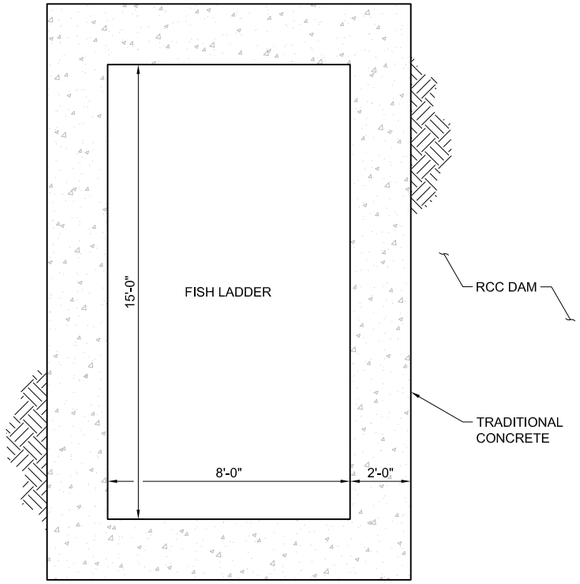
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
ALT-C-6



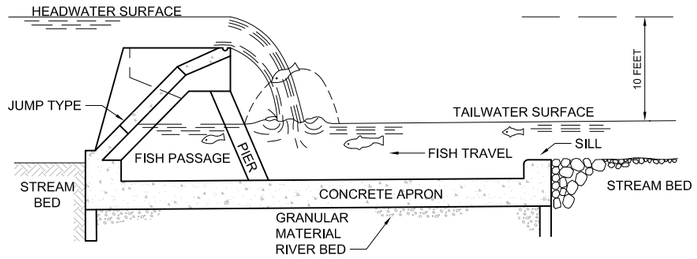
VERTICAL SLOT PLAN

1 ALT-B-3



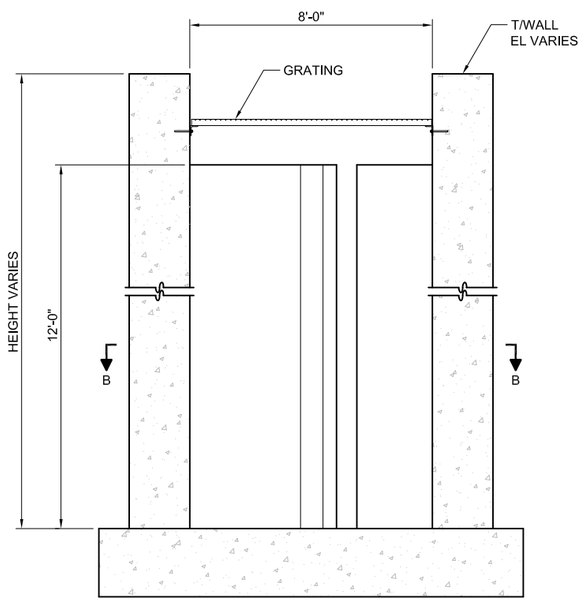
FISH LADDER PENETRATION DETAIL

3 ALT-C-4



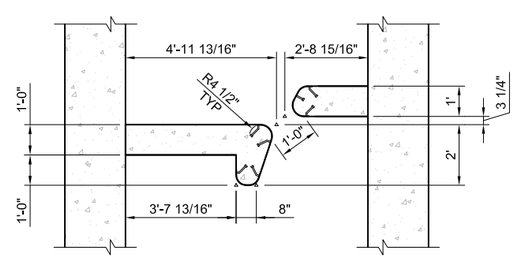
TYPICAL CROSS SECTION OF TYPICAL BARRIER DAM

5 ALT-B-3



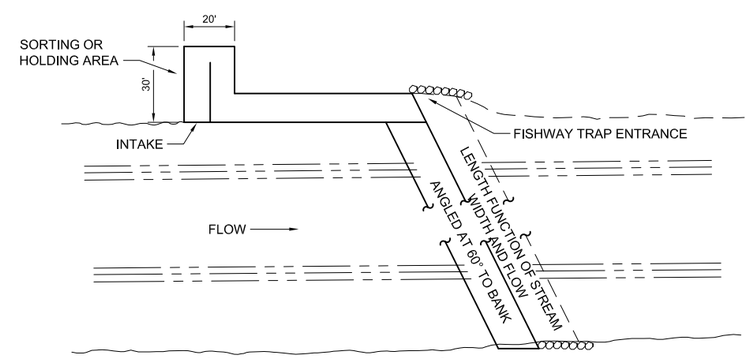
VERTICAL SLOT WALL ELEVATION

2 ALT-B-3



VERTICAL SLOT - SECTION B-B

4 ALT-B-3



PLAN VIEW OF TYPICAL BARRIER DAM PLACED IN A RIVER

6 ALT-B-3

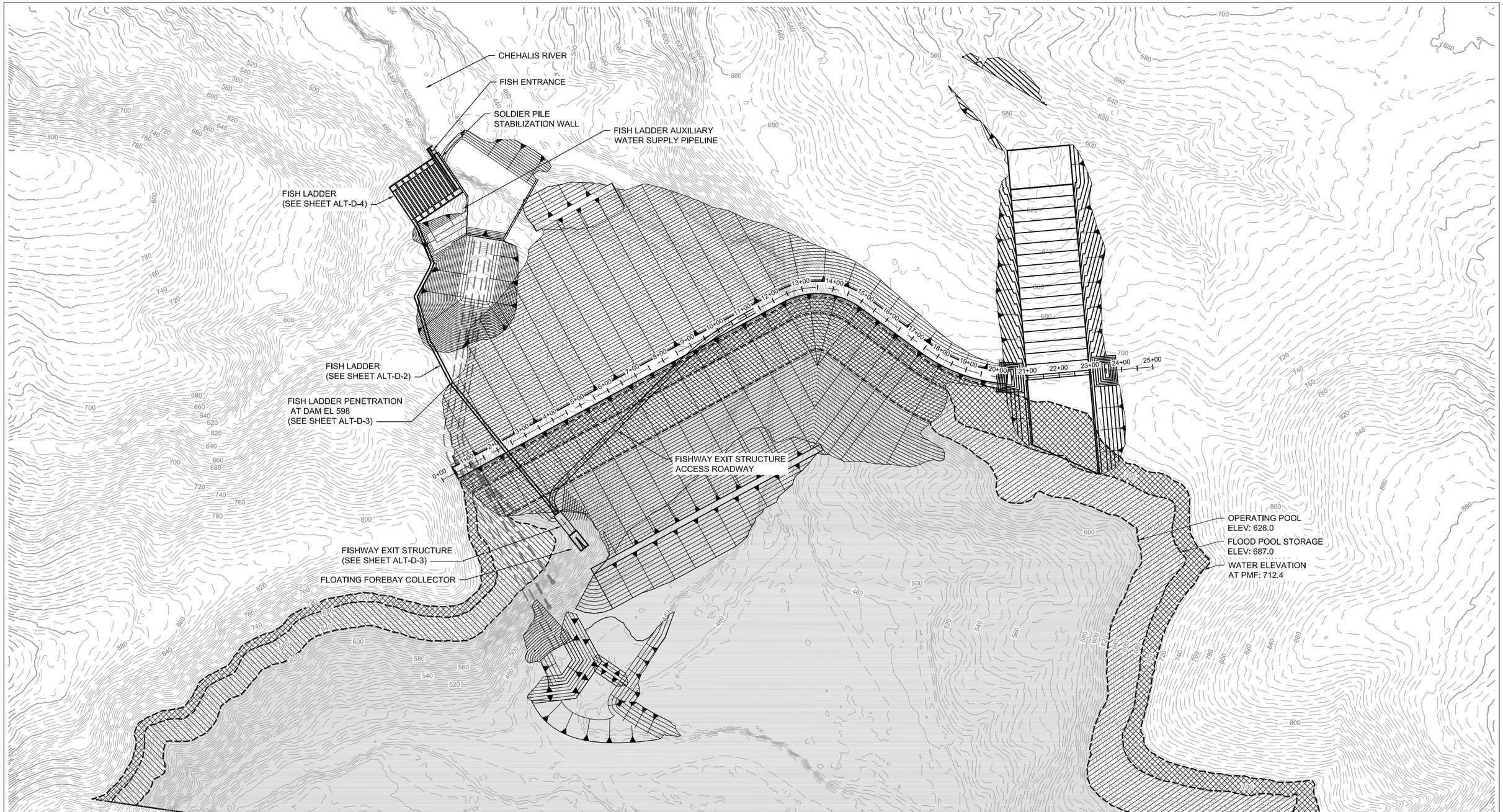
30 0 30 60
SCALE IN FEET

MULTIPURPOSE - RCC DAM
FISH PASSAGE DETAILS

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
	JULY 2014
FIGURE	
	ALT-C-7

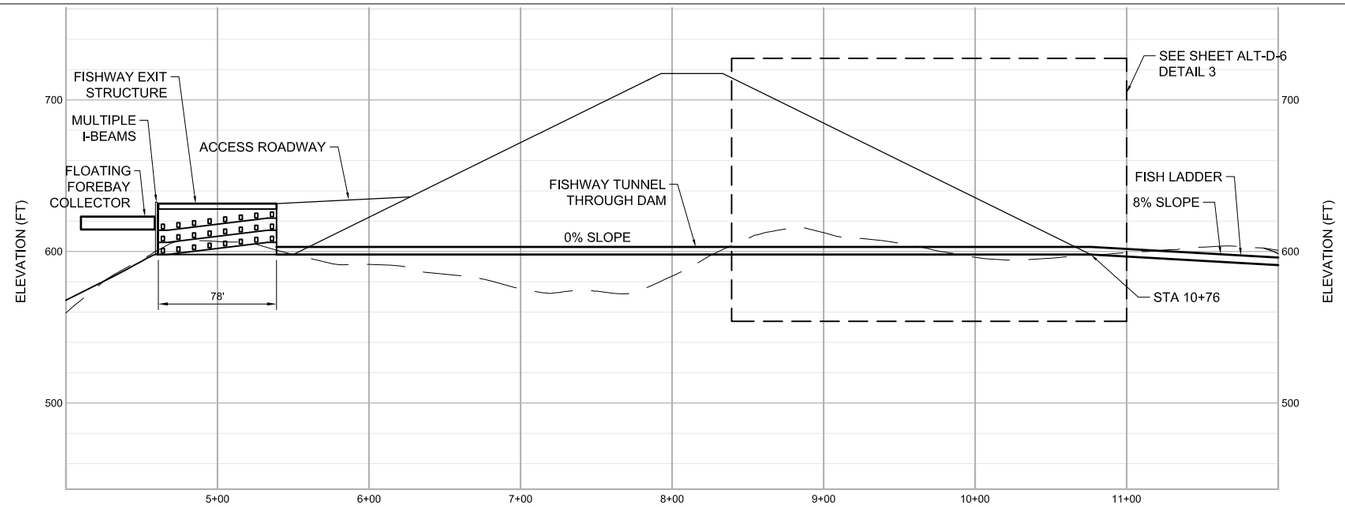
Appendix B.4 – Alternative D



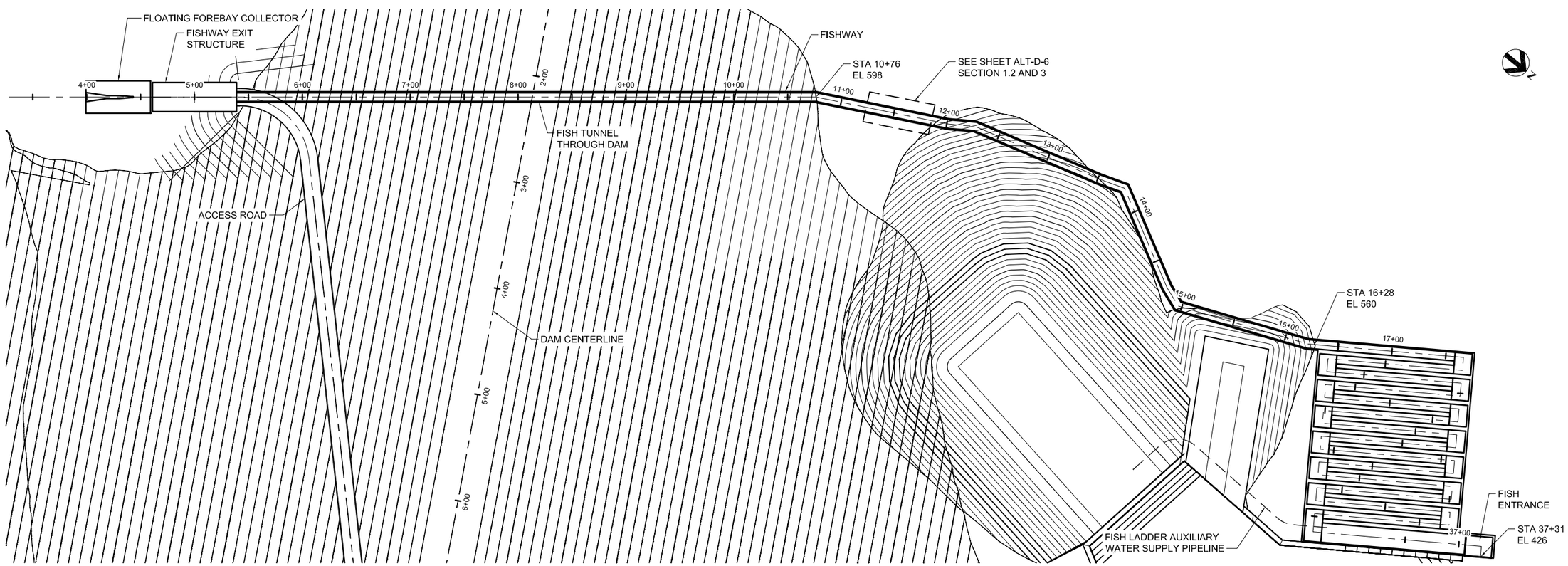
MULTI-PURPOSE - ROCKFILL DAM
INTEGRATED STRUCTURE PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014
FIGURE
ALT-D-1



FISHWAY EXIT STRUCTURE - PROFILE



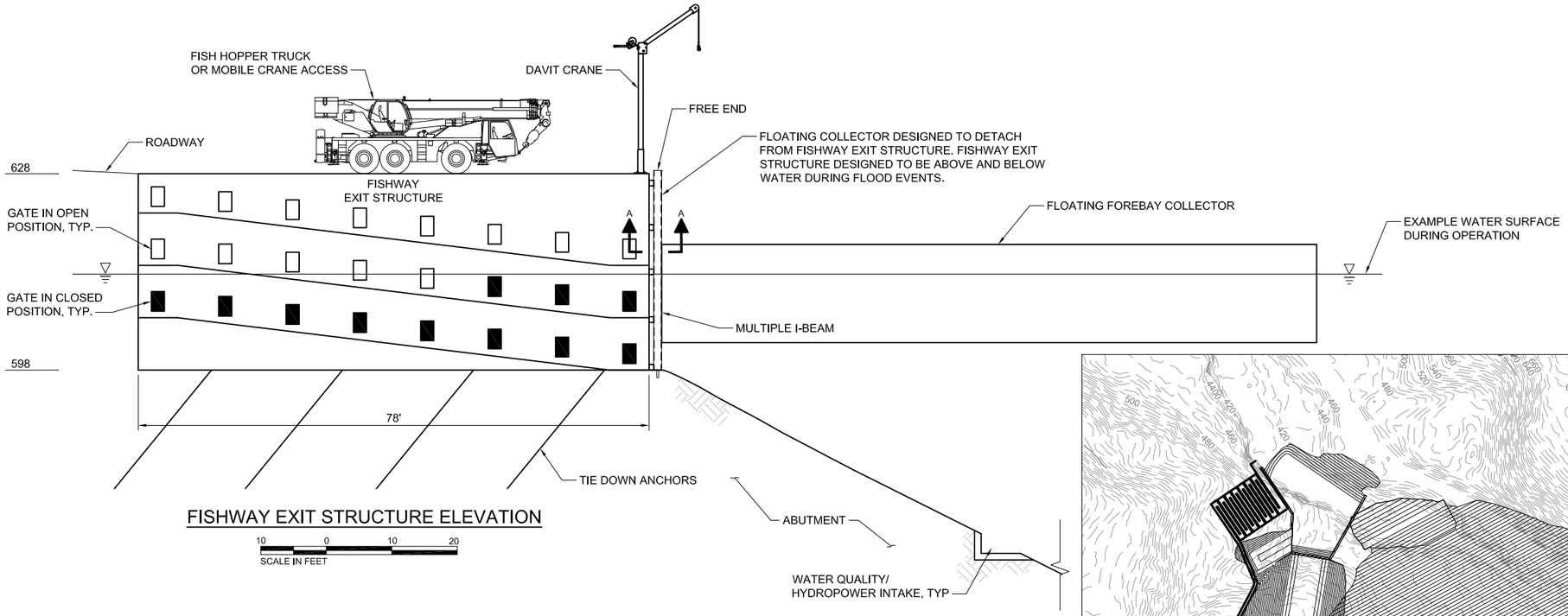
PLAN VIEW

50 0 50 100
SCALE IN FEET

MULTI-PURPOSE ROCKFILL DAM ROADWAY TO FOREBAY COLLECTOR AND FISH LADDER EXIT STRUCTURE

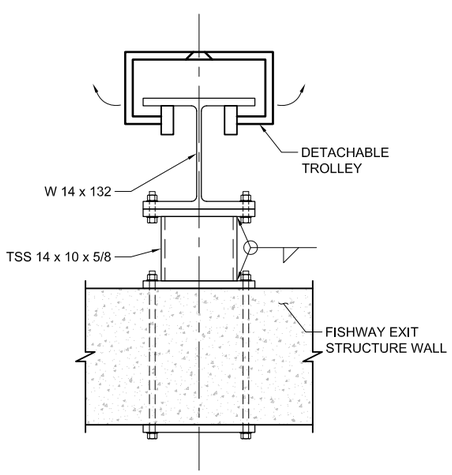
CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
	JULY 2014
FIGURE	
	ALT-D-2

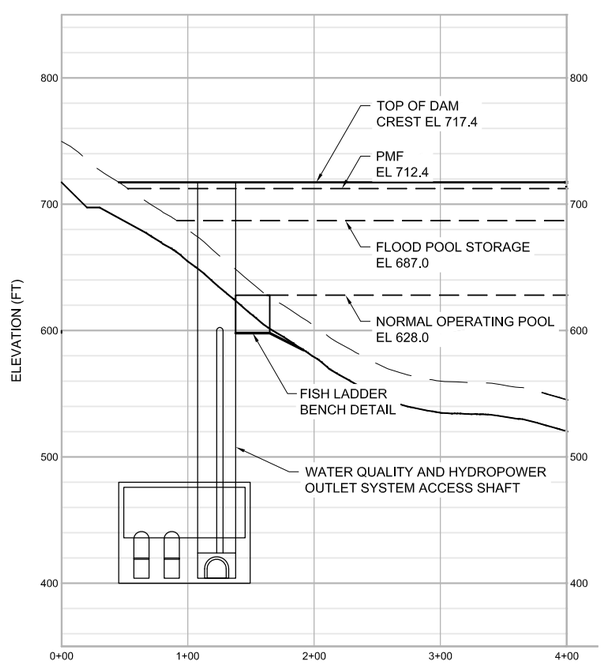


FISHWAY EXIT STRUCTURE ELEVATION

10 0 10 20
SCALE IN FEET

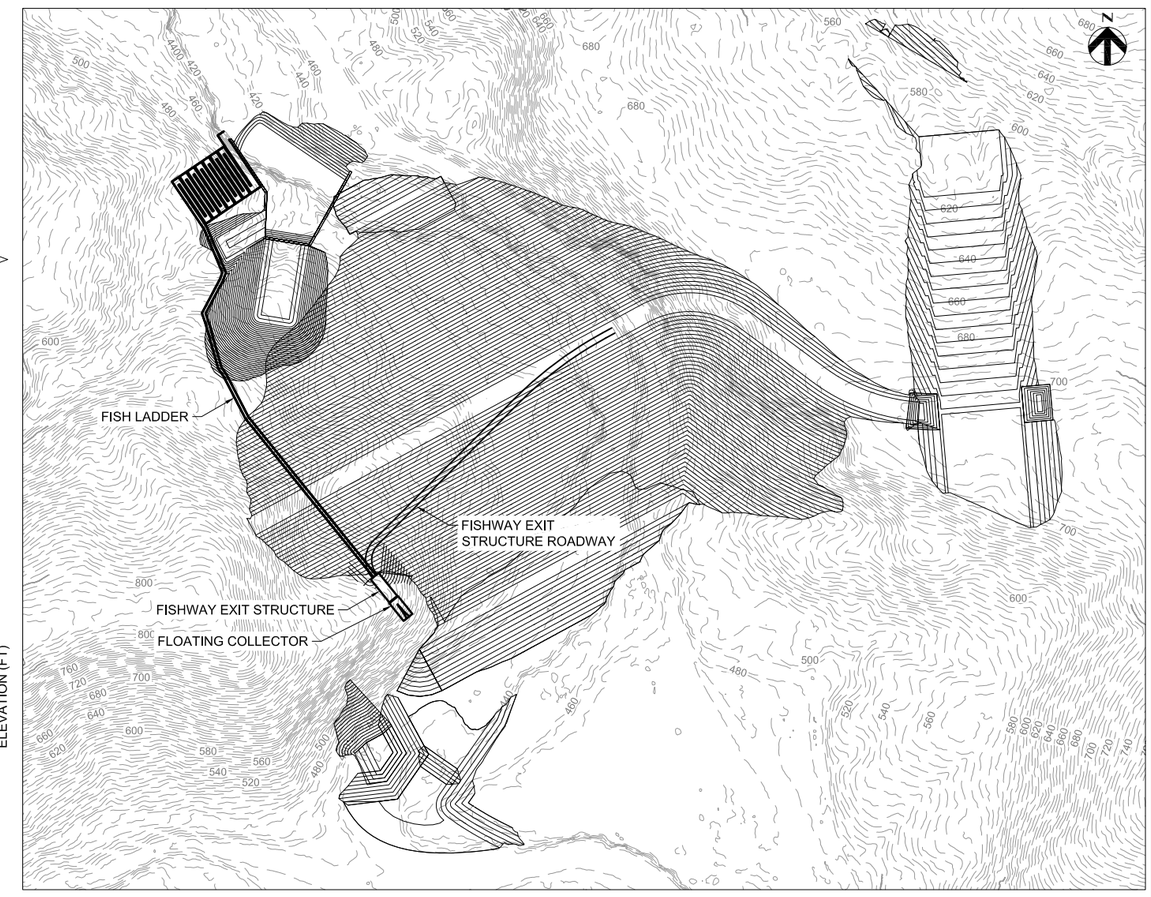


SECTION A-A
EXAMPLE ATTACHMENT
NTS



FISH LADDER DAM PENETRATION

60 0 60 120
SCALE IN FEET



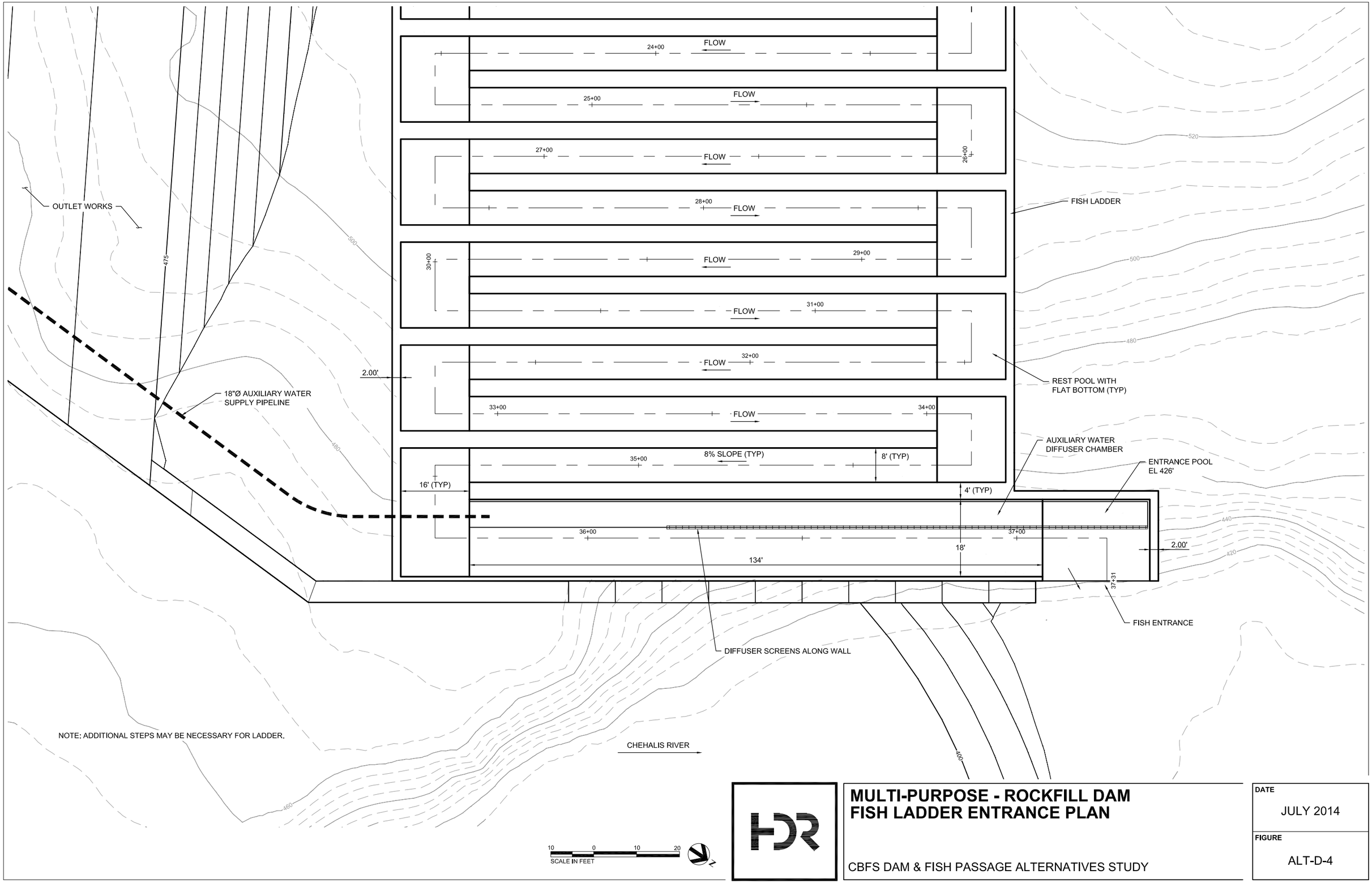
FISH LADDER - OPTION D

200 0 200 400
SCALE IN FEET

MULTI-PURPOSE ROCKFILL DAM
FISH LADDER DAM PENETRATION PROFILE AND FISHWAY EXIT STRUCTURE DETAILS

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

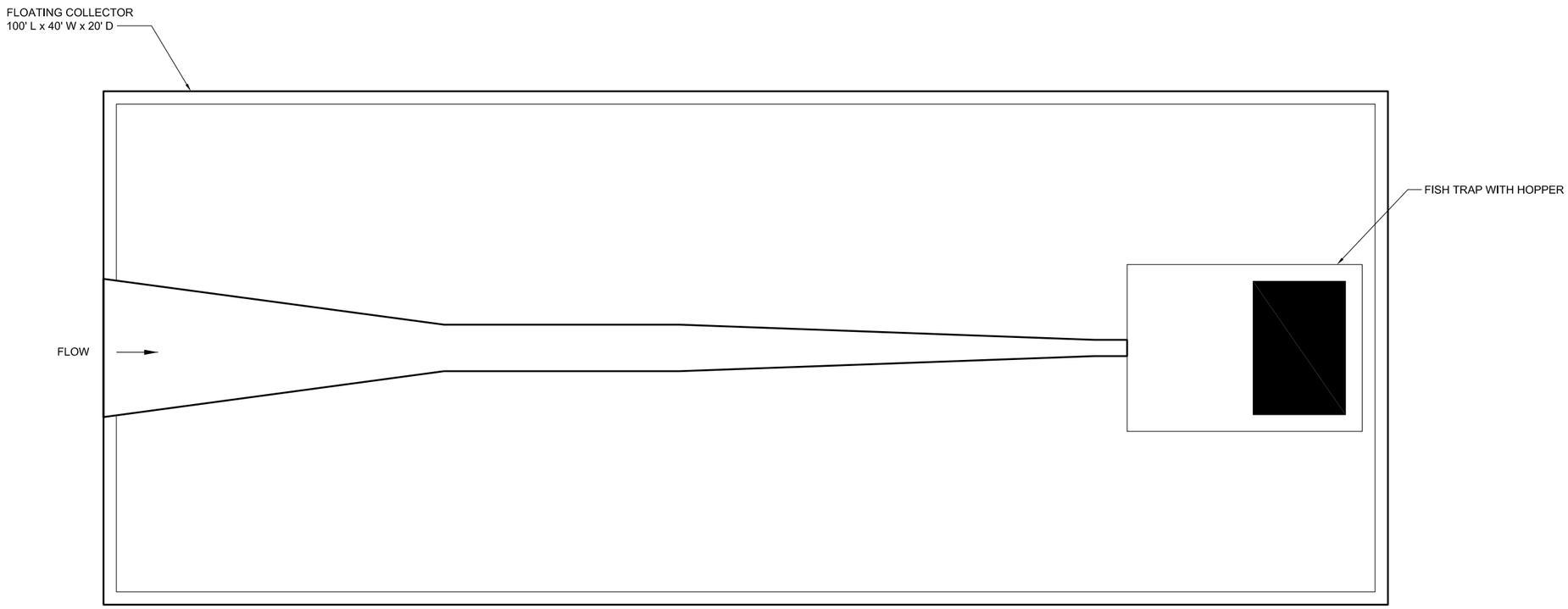
DATE	
	JULY 2014
FIGURE	
	ALT-D-3



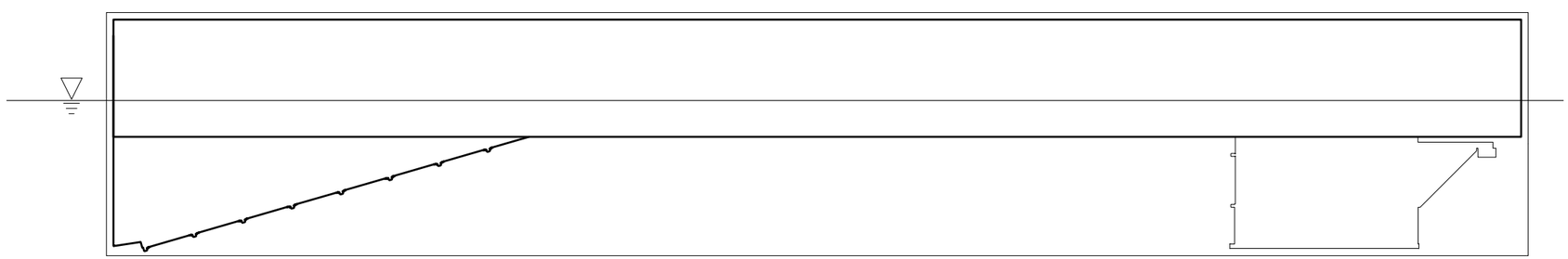
MULTI-PURPOSE - ROCKFILL DAM
FISH LADDER ENTRANCE PLAN

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE	
JULY 2014	
FIGURE	
ALT-D-4	



PLAN

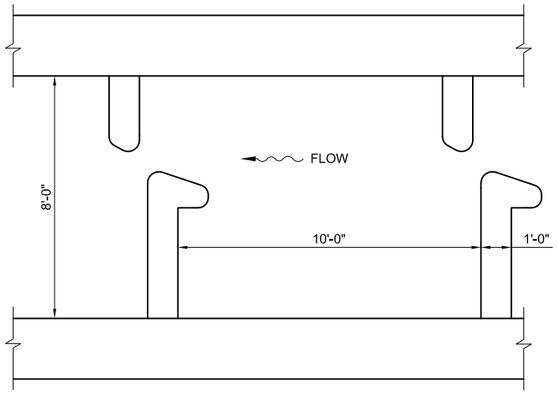


SECTION

MULTI-PURPOSE ROCKFILL DAM
FLOATING FOREBAY COLLECTOR
PLAN AND SECTION

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

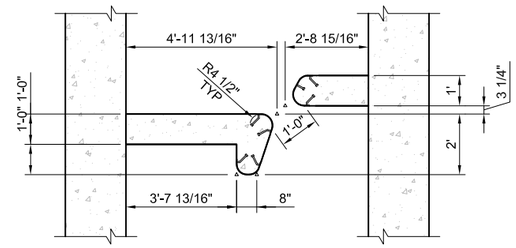
DATE
JULY 2014
FIGURE
ALT-D-5



VERTICAL SLOT PLAN

1
ALT-D-1

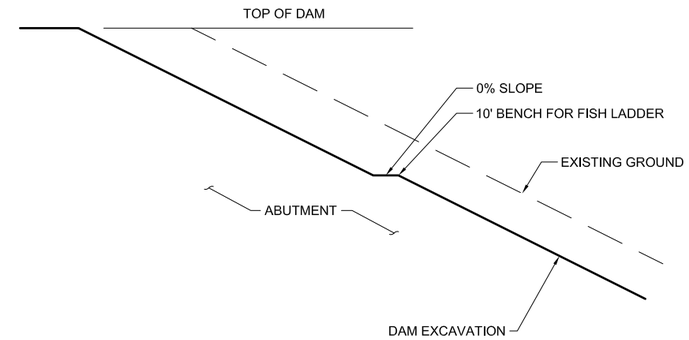
30 0 30 60
SCALE IN FEET



VERTICAL SLOT SECTION B-B

4
ALT-D-1

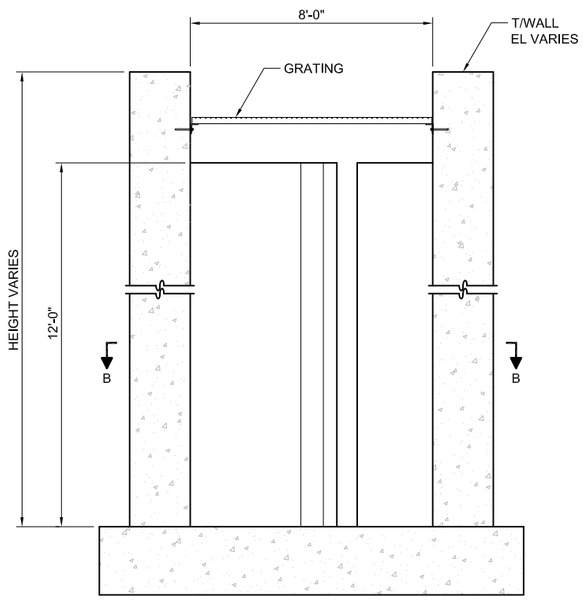
30 0 30 60
SCALE IN FEET



THROUGH DAM FISH LADDER BENCH DETAIL

NTS

3
ALT-D-1



VERTICAL SLOT WALL ELEVATION

2
ALT-D-1

30 0 30 60
SCALE IN FEET

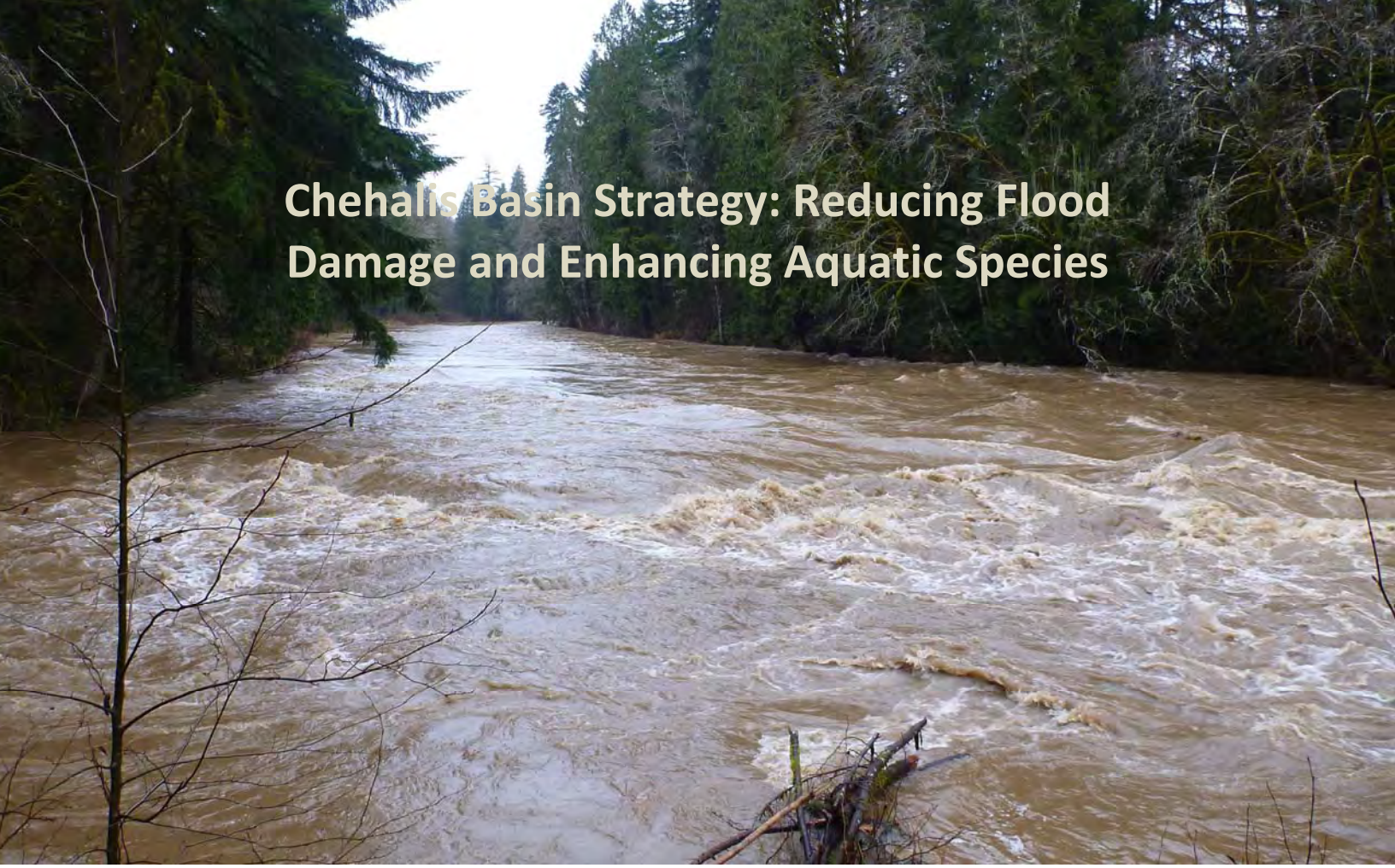
MULTI-PURPOSE ROCKFILL DAM
FISH PASSAGE DETAILS

CBFS DAM & FISH PASSAGE ALTERNATIVES STUDY

DATE
JULY 2014

FIGURE
ALT-D-6

Appendix C – Reservoir Landslide Evaluation TM



Preliminary Desktop Landslide Evaluation

July 30, 2014

Prepared by: Shannon & Wilson, Inc.

Prepared for: Chehalis Basin Workgroup

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

Introduction

This report presents the results of a desktop study of landslides in and adjacent to the proposed reservoir for the Chehalis River dam upstream of Pe Ell, Washington (Figure 1). The purposes of this study are as follows:

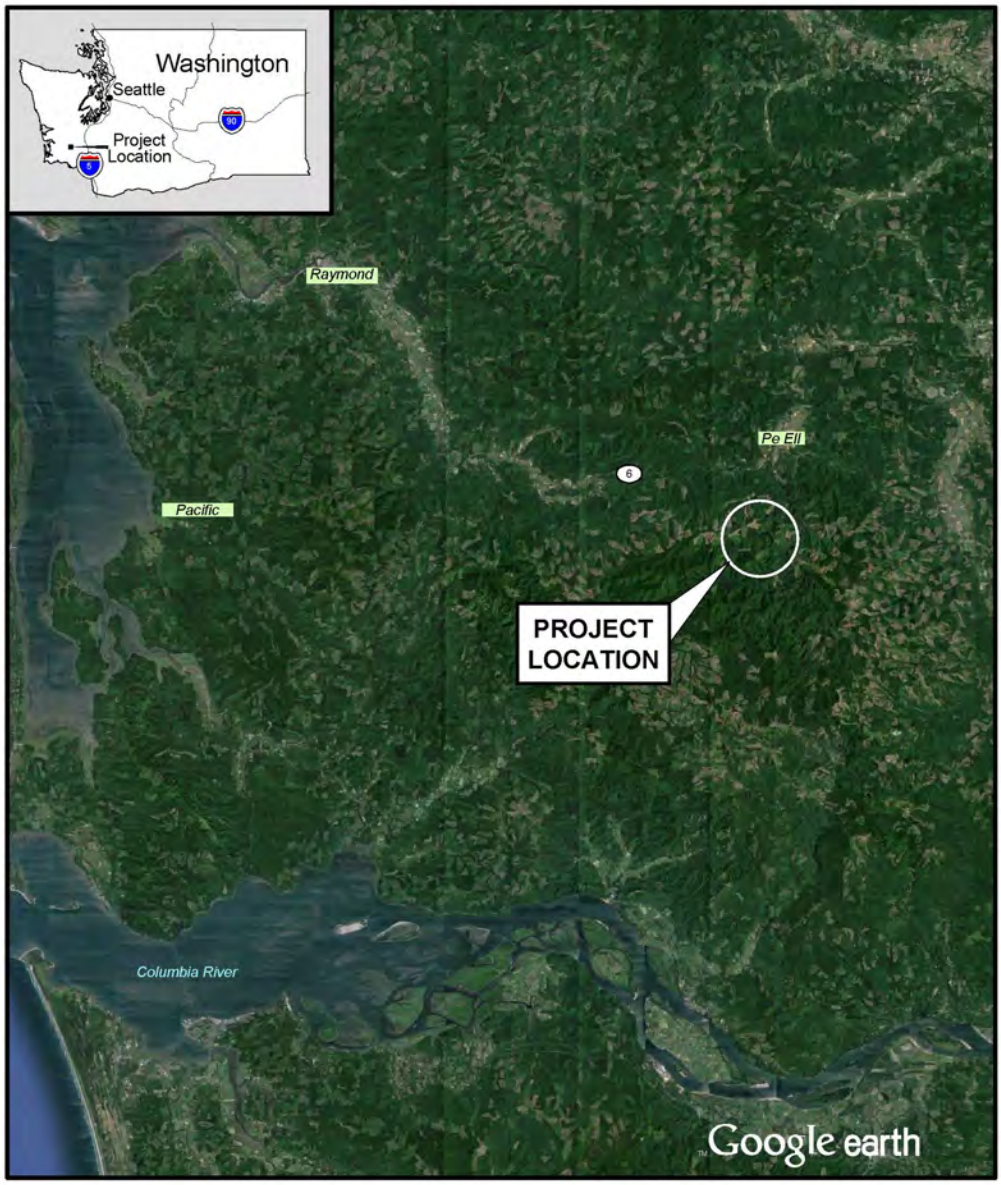
- Identify unstable slopes in the proposed reservoir area that could be affected by rising and falling of reservoir waters
- Assess the potential impact of the identified unstable slopes and other steeply sloping areas on the proposed reservoir

There are many examples in the literature of slopes adjoining reservoirs that were rendered unstable either by rising water that saturated the toe of the slope or by falling water levels that caused a rapid drawdown condition, resulting in slope failure. Identifying slopes that could be impacted by these rapid changes in reservoir level is a critical component of assessing the feasibility and cost of developing the Site.

In most cases of reservoir slope instability, movement of the landslide mass causes a temporary turbid condition, a small decrease in the reservoir capacity (relative to reservoir volume), or a small to negligible wave in the water surface. However, there are documented cases in which the reservoir was substantially filled in, or landslide movement into the reservoir created a large wave that damaged or destroyed the dam, or rendered the reservoir unusable.

On a smaller scale, rivers at the toes of landslides commonly remove soil and deliver it to the alluvial system annually or episodically. Additionally, small to medium volume shallow landslides occur on the sides of fluctuating reservoirs due to that fluctuation and to natural response to precipitation.

Figure 1
Vicinity Map



0 7 14

Approximate Scale in Miles

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

Chehalis Basin Flood Mitigation
Pe Ell, Washington

VICINITY MAP

July 2014 21-1-21897-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

Filename: J:\211\21897-001\21-1-21897-001 Fig 1.dwg Date: 07-23-2014 Login: lr

Methodology

A landslide inventory was performed to evaluate the potential for landslides to impact the dam, reservoir, and reservoir operations. Three data sources were used to identify landslides in the portion of the river basin that would be inundated by the proposed reservoir, including aerial photographs, Chehalis Headwaters Watershed Analysis landslide inventory, and Light Detection and Ranging (LiDAR).

PREVIOUS STUDIES

Watershed Geodynamics, Inc., used two sets of aerial photographs taken in 2008 and 2009 to locate landslides; primarily to document landslides that occurred in the 2007 storm that caused widespread flooding and slope instability in the Chehalis River basin. The landslides that Watershed Geodynamics, Inc., identified were classified as debris flows, debris slides, and debris avalanches. Debris torrents were noted, but they are not included on the combined map because they would be initiated outside of the influence of reservoir. However, debris torrents can contribute to the sediment budget in the reservoir and river.

The Chehalis Headwaters Watershed Analysis landslide inventory was completed by the Weyerhaeuser Company (Weyerhaeuser) in 1994. A mass wasting assessment was performed as part of that analysis by Weyerhaeuser geologists using six sets of aerial photographs taken between 1955 and 1993 (Weyerhaeuser 1994). There is no overlap in the photograph years used by Weyerhaeuser and Watershed Geodynamics, Inc. Because of the large number of landslides and time constraints on the completion of the watershed analysis, Weyerhaeuser geologists mapped only landslides greater than 150 cubic yards (CY). The percentage of landslides that were field-checked was not reported in the watershed analysis; however, it was common during such watershed studies in the 1990s for about half of the landslides identified on the aerial photographs to be field checked.

As a part of this study, Shannon & Wilson, Inc., used public-source LiDAR data to create two hillshade images. One image used illumination from 315 degrees azimuth; the other was from 45 degrees azimuth. The hillshade illumination process produces shadows on the LiDAR image that make landslide topography stand out from the rest of the landscape. Landslide features were identified on both maps because each illumination can highlight different topographic nuances on a map. The identified landslides were then compiled into one map. Fifteen landslides were identified by Shannon & Wilson, Inc., upstream from (south of) the dam site as it is presently envisioned. These are deep-seated landslides ranging from small (100,000 CY) to very large (more than 160 million CY), as shown in Table 1. Ten additional landslides were identified by the Washington State Department of Natural Resources ranging in size from 11,000 to 8,900,000 CY.

The sizes of the 25 landslides identified on the LiDAR images are shown in Table 1. The widths, lengths, and depths of these features were estimated using Geographic Information Systems methods.

Table 1
Volumes and Masses of Deep-Seated Landslide Deposits

LANDSLIDE ID		DEPTH OF DISPLACED MASS (D_D) [FEET]	WIDTH OF DISPLACED MASS (W_D) [FEET]	LENGTH OF DISPLACED MASS (L_D) [FEET]	$V_{DISPLACED}$ [CUBIC FEET]	$V_{DISPLACED}$ [CUBIC YARDS]	$MASS_{DISPLACED}$ [POUNDS]	$MASS_{DISPLACED}$ [SHORT TONS]
1		45	320	560	4.2E+06	1.6E+05	4.6E+08	**2.3E+05**
2		50	320	820	6.9E+06	2.5E+05	7.6E+08	**3.8E+05**
3		70	1130	680	2.8E+07	1.0E+06	3.1E+09	**1.5E+06**
4		25	850	1070	1.2E+07	4.4E+05	1.3E+09	**6.5E+05**
5		100	960	1090	5.5E+07	2.0E+06	6.0E+09	**3.0E+06**
6		100	710	270	1.0E+07	3.7E+05	1.1E+09	**5.5E+05**
7		250	2320	1460	4.4E+08	1.6E+07	4.9E+10	**2.4E+07**
8		70	1030	590	2.2E+07	8.2E+05	2.5E+09	**1.2E+06**
9		50	1310	1740	6.0E+07	2.2E+06	6.6E+09	**3.3E+06**
10		50	830	460	1.0E+07	3.7E+05	1.1E+09	**5.5E+05**
11		80	700	1590	4.7E+07	1.7E+06	5.1E+09	**2.6E+06**
12		110	850	560	2.7E+07	1.0E+06	3.0E+09	**1.5E+06**
13		100	400	1040	2.2E+07	8.1E+05	2.4E+09	**1.2E+06**
14		180	870	1020	8.4E+07	3.1E+06	9.2E+09	**4.6E+06**
15	upper zone	50	130	790	7.4E+06	2.7E+05	8.1E+08	**4.1E+05**
	lower zone	40	770	290				
16		80	1490	3830	2.4E+08	8.9E+06	2.6E+10	**1.3E+07**
17		40	700	530	7.8E+06	2.9E+05	8.5E+08	**4.3E+05**
18		60	490	780	1.2E+07	4.4E+05	1.3E+09	**6.6E+05**
19		30	440	660	4.6E+06	1.7E+05	5.0E+08	**2.5E+05**
20		10	170	610	5.4E+05	2.0E+04	6.0E+07	**3.0E+04**
21		60	750	710	1.7E+07	6.2E+05	1.8E+09	**9.2E+05**
22		10	140	410	3.0E+05	1.1E+04	3.3E+07	**1.7E+04**
23		10	120	470	3.0E+05	1.1E+04	3.2E+07	**1.6E+04**
24		50	610	940	1.5E+07	5.6E+05	1.7E+09	**8.3E+05**

25	60	670	1120	2.4E+07	8.7E+05	2.6E+09	**1.3E+06**
						Total:	**6.4E+07**

Notes:
Dimensions approximated from a LiDAR DEM in a Geographic Information System.
Sediment density (lbs./ft^3): 110
Volume of displaced mass (Vd) = (1/6)*π*Dd*Wd*Ld
Source:
The International Geotechnical Societies' UNESCO Working Party on World Landslide Inventory, 1990. A Suggested Method for Reporting a Landslide. *Bulletin of the International Association of Engineering Geology* 41: 5-12.
Cruden, D.M., and D.J. Varnes, 1996. Landslide Types and Processes. *Landslides: Investigation and Mitigation, Transportation Research Board, Special Report*, 247: 36-75.

Landslide Descriptions

A total of 25 deep-seated landslides were identified that could potentially have an effect on the proposed dam or reservoir (see Figure 2). The following section of this report presents the characteristics of the identified deep-seated landslides (see Figure 2). No field-checking has been performed.

Landslide No. 1 is in the left abutment of the proposed dam. It is about 320 feet wide, 560 feet long, and has a volume of about 160,000 CY. The headscarp area is subtle, and the debris deposit at the toe is hummocky. This landslide would be excavated as part of construction of the left abutment and foundation preparation of the dam.

Landslide No. 2 is upstream of the left abutment on the west slope of Crim Creek. It is about 320 feet wide, 820 feet long, and has a volume of about 250,000 CY. It is at the southern end of a steep convergent slope. The toe of the deposit is on the outside of a bend in the creek and is probably subject to toe erosion. Contours indicate that the body of the landslide deposit is benched. This landslide may be capable of moving quickly if saturated. Its subsurface should be explored to understand geologic conditions. The landslide debris could be excavated for fill or stabilized.

Landslide No. 3 is upstream of the right abutment on the east slope of the main stem. It is a triangular area about 680 feet long and about 1,130 feet wide at its toe. Its estimated volume is about 1,000,000 CY. A seismic refraction survey performed through this feature in 2009 indicates the landslide debris may be 50 to 60 feet thick (Shannon & Wilson 2009). Much of this feature would be covered with water during high levels of the reservoir. This landslide is likely to reactivate under cycles of saturation and drawdown; however, it is likely to move slowly. Additional seismic refraction surveys should be performed here, and borings should be advanced to understand the subsurface conditions.

Landslide No. 4 is in the north-facing slope of the spine of land between Crim Creek and the main stem Chehalis River. It is about 850 feet wide and about 1,070 feet long. It has an estimated volume of about 440,000 CY. This feature is an enigma because of the flat slopes throughout most of the body and a small headscarp in relation to the debris deposit. Because the slopes are relatively flat, it is unlikely to fail catastrophically, but could continue to move in small increments frequently because it will be wetted often. Because it is close to the dam, movement could impact the dam or dam operations. It should be explored with borings and seismic refraction to understand the subsurface conditions. A detailed reconnaissance should be performed to look for typical landslide features, such as scarps, earth cracks, pressure ridges, and seeps, to have more confidence that this is a landslide and to understand the risks it may pose to the facility.

Landslide No. 5 is on the western slope of Crim Creek. It is about 960 wide, 1,090 feet long, and its estimated volume is about 2,000,000 CY. This feature was recognized during field reconnaissance in 2009 as a landslide owing to its steep headscarp and large debris deposit. It has two or three benches in its body, and the toe of the deposit causes a narrowing of Crim Creek. Most of the body of the landslide would be inundated by the reservoir. This landslide may have the potential to move quickly, pinching off Crim Creek and reducing the volume of the reservoir. Landslide movement could add significant sediment volume to the reservoir and river. It should be explored with reconnaissance, borings, and seismic refraction to understand the surface and subsurface conditions.

Landslide No. 6 is a debris deposit about 710 feet wide and 270 feet long at the toe of a steep headwall on the north side of Lester Creek. Its estimated volume is about 370,000 CY. It appears that the debris may have come off of the upper part of the headwall, although field reconnaissance needs to confirm this. Reservoir fluctuation would not likely affect the headwall, but could cause creep or local instability of the debris deposit. Such action could result in increased sediment production. Reconnaissance of this feature is recommended to determine if subsurface explorations are warranted.

Landslide No. 7 is about 2,320 feet wide and 1,460 feet long. It is on the western slope of Crim Creek. Its estimated volume is 160,000,000 CY. At least two smaller landslides lie within these boundaries; one of them is designated Landslide No. 15 and is discussed below. Landslide No. 7 was identified in the Chehalis Headwaters Watershed Analysis study in 1994. This may be a landslide complex, as the very large feature may be deep-seated and rotational, but the smaller landslides may be either shallower deep-seated or earthflows. The northern of the two internal landslides, i.e., Landslide No. 15, has characteristics of an earthflow such as a longitudinal wavy surface and a lobate toe. Reservoir high water levels will wet the toe of the large feature. It is unknown how its stability will be affected by the reservoir fluctuation. A reconnaissance should be performed to understand this feature after which a subsurface exploration plan can be implemented, if necessary.

Landslide No. 8 is a debris deposit about 1,030 feet wide and about 590 feet long at the toe of an old scar on the mountainside to its west. Its estimated volume is about 820,000 CY. Some of the debris could also have originated from debris flows from the adjacent creek. The high water mark of the reservoir for the flood control only alternative would lap at the toe of this gently sloping debris deposit. This feature would be inundated for the multi-purpose alternative design maximum pool elevation. This feature would be impacted by the reservoir in extreme flood events. Reconnaissance should be performed to assess the potential for reservoir operation-induced landslide movement.

Landslide No. 9 is a colluvial deposit about 1,310 feet long and 1,740 feet wide at the toe of a broad convergent bowl. Its volume is estimated to be about 2,200,000 CY. The source of sediment is several high-gradient creeks. The deposit gently slopes down from west to east, and appears to be undercut by the river at its toe. Some parts appear to be unstable, particularly at the sharp river bend on its south side. The reservoir high waters would cover most of the deposit. A reconnaissance should be performed to understand this feature, after which a subsurface exploration plan can be implemented, if necessary.

Landslide No. 10 is a colluvial deposit that is about 830 feet wide and 460 feet long at the toe of a bowl with several small creeks—the source of the sediment. The estimated volume of the deposit is about 370,000 CY. The ground slopes down from east to west. Reservoir high water will cover the deposit. Reconnaissance of this feature is recommended to determine if subsurface explorations are warranted.

Landslide No. 11 is a large landslide about 700 feet wide and 1,590 feet long. Its estimated volume is about 1,700,000 CY. It has a high, steep headscarp and a long body, sloping down from west to east to the edge of the river. The LiDAR imagery shows multiple vertical setdowns in the body of the landslide, indicative of an active feature. It appears to have pushed the river against the eastern side of the valley. The high water of the reservoir would cover approximately the eastern half of the body of the landslide. Reservoir elevation fluctuations could accelerate movement of this landslide. Landslide movement could block the river. A reconnaissance should be performed to understand this feature, after which a subsurface exploration plan can be implemented, if necessary.

Landslide No. 12 is a colluvial deposit about 850 feet wide and 560 feet long at the base of a steep convergent slope. Its estimated volume is about 1,000,000 CY. Several small channels likely contributed sediment to this

deposit. The deposit slopes moderately down from northeast to southwest and pushes Big Creek against the southwestern side of the valley. Movement of this landslide could block this tributary. The proposed reservoir high water would cover much of the deposit. Reconnaissance of this feature is recommended to determine if subsurface explorations are warranted.

Landslide No. 13 is a relatively small landslide on a divergent nose of a ridge on the eastern slope of the main stem of the Chehalis River. It is about 400 feet wide, 1,040 feet long, and has an estimated volume of about 800,000 CY. Most of this feature would be inundated by the reservoir waters. It is presently not being directly attacked by the river, but the river is running parallel to its toe. Reconnaissance of this feature is recommended to determine if subsurface explorations are warranted.

Landslide No. 14 is a moderately sized landslide on the northern slope of Lester Creek. It is about 870 feet wide, 1,020 feet long, and has an estimate volume of about 3,100,000 CY. It is at the upvalley end of the highest potential reservoir. Its failure could deliver sediment to the system. Reconnaissance of this feature is recommended to determine if subsurface explorations are warranted.

Landslide No. 15 is a landslide on the northern margin of Landslide No. 7 and is apparently younger than its bigger parent. Its upper zone is about 130 feet wide and 790 feet long, and the lower zone is about 770 feet wide and 290 feet long. Its combined volume is approximately 270,000 CY. This landslide may be an earthflow along a wet perimeter of the larger feature, and it may be more sensitive to toe undermining by Crim Creek than the larger feature. It should be reconnoitered at the same time as Landslide No. 7 to determine any interrelationship and the best way to explore its subsurface.

Landslide No. 16 is a very large, deep-seated landslide that appears to be of different ages. This feature is about 1,490 feet wide and 3,830 feet long with an estimated volume of about 8,900,000 CY. The upper part of the feature is subdued, gently sloping, hummocky on a large scale, and appears to very old. The lower part of the landslide mass, near Lester Creek, shows many signs of more recent activity, such as hummocky, uneven ground surfaces and narrow chutes that extend to the creek, likely due to undercutting by the creek and spring action. The toe of this landslide would be wetted during the highest water of the multi-purpose dam. A reconnaissance of this large feature is recommended, after which an exploration program can be implemented, if necessary.

Landslide No. 17 is a medium-size, deep-seated landslide along the southern bank of Lester Creek. This feature is about 700 feet wide and 530 feet long with an estimated volume of about 290,000 CY. It appears to be connected to the lower part of Landslide No. 16. The primary trigger of this landslide is probably Lester Creek, which is directly attacking the toe of the slope. Two levels of scarps are in close proximity to the creek. Only the northeastern corner of the landslide is touched by the highest water of the multi-purposed dam, so the probability of reactivation owing to the reservoir is low. Nevertheless, the lower part of this landslide should be observed to check for recent activity, as toe instability would contribute sediment directly to Lester Creek.

Landslide No. 18 is a medium-size landslide or a high alluvial terrace. It is about 490 feet wide and 780 feet long with a potential volume of about 440,000 CY. Two benches are prominent, and the ground surface is slightly hummocky. The toe of the slope is not presently being eroded or attacked by Lester Creek. This feature needs to be reconnoitered to determine its geologic origin, which may require subsurface exploration.

Landslide No. 19 is a medium-size landslide scar along the eastern bank of the mainstem. It is about 440 feet wide and 660 feet long, with an estimated volume of about 170,000 CY. The toe of the slope bulges into the valley, and small, non-incised creeks border the sides of the feature. There do not appear to be new or active aspects to this feature. Reconnaissance needs to be performed to determine the relative activity of this landslide area.

Landslide No. 20 is a relatively narrow chute below a logging road that extends down to Crim Creek. The feature is about 170 feet wide and about 610 feet long with an estimated volume of 20,000 CY. Steep side walls define the upper half of the feature. A narrow incision in the ground surface is indicative of a spring-headed gully. Half to all of this feature would be inundated by the two dam scenarios, so it would be sensitive to fluctuations of the reservoirs. A reconnaissance of this narrow feature is recommended, after which an exploration program can be implemented, if necessary.

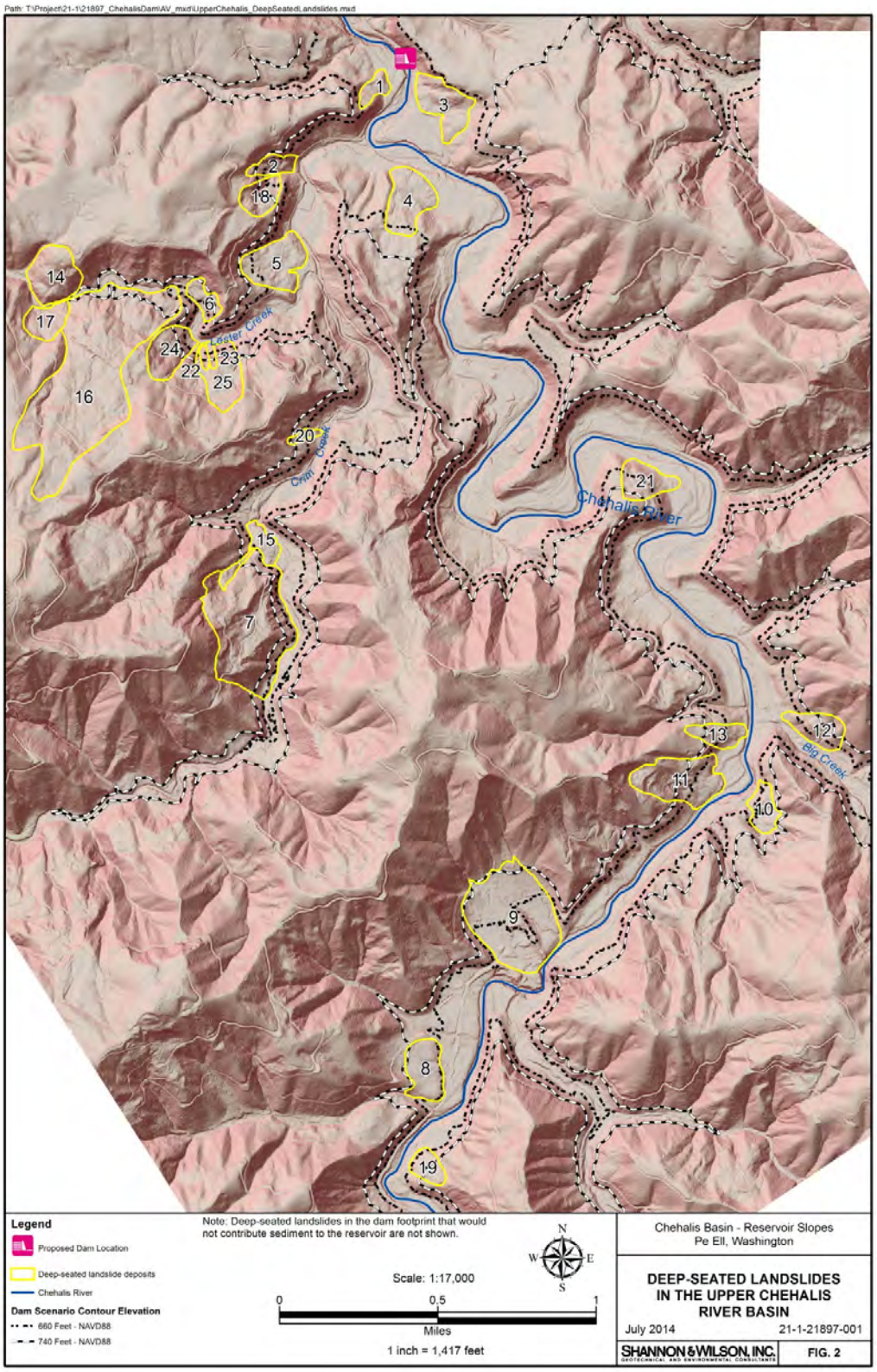
Landslide No. 21 is a steep, northeast-facing slope on the inside of a gooseneck section of the mainstem channel. This triangular landslide feature is about 750 feet wide and 710 feet long with an estimated volume of 620,000 CY. It has two convergent slopes oriented downslope and other smaller scale hummocky ground near the toe. It appears to be subdued and old, and debris that may have been deposited on the mainstem has been removed. Most of this landslide would be inundated under both dam alternatives. A reconnaissance of this narrow feature is recommended, after which an exploration program can be implemented, if necessary.

Landslide Nos. 22 and 23 are adjacent parasitic chutes within a large deep-seated landslide on the southern bank of Lester Creek. Landslide No. 22 is about 140 feet wide and 410 feet long with a volume of approximately 11,000 CY. Landslide No. 23 is about 120 feet wide and 470 feet long with an estimated volume of about 11,000 CY. Both features are located in the toe area of Landslide No. 25. They may have been triggered by undercutting by Lester Creek or by spring action in the toe of Landslide No. 25. About half of these features would be inundated by waters of both reservoir alternatives. Reconnaissance of these features at the same time is recommended as the other unstable landforms along Lester Creek. It is unlikely that a narrow feature such as this would be drilled and instrumented separately from the larger parent landslide, unless found to be presently active.

Landslide No. 24 is another discrete deep-seated landslide among the landslide complex along Lester Creek. It is about 610 feet wide and 940 feet long with an estimated volume of 560,000 CY. This landslide has a headscarp just below an existing road and is attacked at the toe by Lester Creek. The deposit is hummocky, and the convergent area in its middle may indicate the presence of seepage. The multi-purpose reservoir would fully inundate the toe of the landslide, but the flood retention option would barely wet the toe. A reconnaissance of this narrow feature is recommended, after which an exploration program can be implemented, if necessary.

Landslide No. 25 is a hummocky, deep-seated landslide on the south bank of Lester Creek. It is about 670 feet wide and 1,120 feet long with an estimated volume of 870,000 CY. The steep toe of this landslide contains several concavities with potential delivery directly to Lester Creek. The headscarp of this feature is ambiguous. The high water marks of both reservoir alternatives would wet the toe of this deep-seated landslide. A reconnaissance of this narrow feature is recommended, after which an exploration program can be implemented, if necessary.

Figure 2
Deep-Seated Landslides in the Upper Chehalis River Basin



Discussion

Not all landslides are equal. Different types of landslides have different triggers. Some of the landslides in the proposed reservoir basin would not be significantly affected by the reservoir, whereas others may be directly impacted by the rising and falling waters of the reservoir.

Shallow, rapid landslides at a higher elevation than reservoir waters and debris flows or debris torrents initiated from outside of the reservoir boundaries may add sediment to the reservoir; however, they will not be triggered by changes in reservoir water elevation. Such events may occur on a decade or several-decade frequency in a part or parts of the basin in response to an unusual storm event such as the 2007 storm. The tributary creeks could potentially form fans on the edge of the reservoir (outside the reservoir limit). This sediment could be mobilized during high water intervals of the reservoir or eroded when reservoir lowering exposes sediment.

Deep-seated landslides could contribute sediment to the reservoir in the following ways:

- The toe or a significant portion of the landslide is saturated, and the toe or a significant portion of the landslide moves in response to raising or lowering of the reservoir. Movement could be relatively slow or rapid, potentially causing a wave in the reservoir and reducing the reservoir storage volume.
- The landslide is saturated by the reservoir waters and moves slowly and incrementally into the reservoir.
- The landslide creeps very slowly downhill, causing entrainment of sediment at the toe of the deposit as it is eroded by the river.

For the first case in which the deep-seated landslide moves into the reservoir relatively rapidly, two large events over the 100-year life of the reservoir were assumed. The two most likely landslides, based on expert opinion, to act in this manner are Nos. 5 and 11. They are the two landforms that have the morphology of recently active, large land movements. If an assumption is made that half of the volume of the landslide mass could move into the reservoir when it is inundating the landslide (as opposed to a landslide moving into the running river and blocking the river), the two landslides could contribute a relatively quick sediment input of about 1,000,000 CY and 860,000 CY, respectively. The speed at which they would move is unknown.

Sediment flux to the reservoir from the 25 deep-seated landslides identified above may also occur in the following manner. When the toe of the debris deposit is wetted by the reservoir pool and then subjected to drawdown, the landslide deposit will either move suddenly a few to several feet or incur an accelerated creep rate of inches per day or week until the soil is drained. The increment of soil that moves will be entrained by the river or creek at that time or during subsequent floods. Because the rate of delivery cannot be predicted, range of movement and toe erosion of 1, 3, and 12 inches per year for all of the 25 identified landslide areas was assumed. The rate of sediment production was calculated by multiplying areas of the exposed face of the landslide deposit at river or creek level by the rate of landslide movement, as indicated in Table 2.

Table 2
Deep-Seated Landslide Sedimentation Rates

LANDSLIDE ID	HEIGHT OF LANDSLIDE TOE (H_{TOE}) [FEET]	WIDTH OF LANDSLIDE TOE (W_{TOE}) [FEET]	SEDIMENTATION RATE (= H_{TOE}*W_{TOE}*DX) [FT^3/YEAR]			SEDIMENTATION RATE (= H_{TOE}*W_{TOE}*DX) [YD^3/YEAR]			SEDIMENTATION RATE [POUNDS/YEAR]			SEDIMENTATION RATE [SHORT TONS/YEAR]		
			DX = 1"/YEAR	DX = 3"/YEAR	DX = 12"/YEAR	DX = 1"/YEAR	DX = 3"/YEAR	DX = 12"/YEAR	DX = 1"/YEAR	DX = 3"/YEAR	DX = 12"/YEAR	DX = 1"/YEAR	DX = 3"/YEAR	DX = 12"/YEAR
1	20	430	7.2E+02	2.2E+03	8.6E+03	2.7E+01	8.0E+01	3.2E+02	7.9E+04	2.4E+05	9.5E+05	**3.9E+01**	**1.2E+02**	**4.7E+02**
2	40	330	1.1E+03	3.3E+03	1.3E+04	4.1E+01	1.2E+02	4.9E+02	1.2E+05	3.6E+05	1.5E+06	**6.1E+01**	**1.8E+02**	**7.3E+02**
3	30	1300	3.3E+03	9.8E+03	3.9E+04	1.2E+02	3.6E+02	1.4E+03	3.6E+05	1.1E+06	4.3E+06	**1.8E+02**	**5.4E+02**	**2.1E+03**
4	25	960	2.0E+03	6.0E+03	2.4E+04	7.4E+01	2.2E+02	8.9E+02	2.2E+05	6.6E+05	2.6E+06	**1.1E+02**	**3.3E+02**	**1.3E+03**
5	40	1040	3.5E+03	1.0E+04	4.2E+04	1.3E+02	3.9E+02	1.5E+03	3.8E+05	1.1E+06	4.6E+06	**1.9E+02**	**5.7E+02**	**2.3E+03**
6	30	650	1.6E+03	4.9E+03	2.0E+04	6.0E+01	1.8E+02	7.2E+02	1.8E+05	5.4E+05	2.1E+06	**8.9E+01**	**2.7E+02**	**1.1E+03**
7	200	2500	4.2E+04	1.3E+05	5.0E+05	1.5E+03	4.6E+03	1.9E+04	4.6E+06	1.4E+07	5.5E+07	**2.3E+03**	**6.9E+03**	**2.8E+04**
8	40	1030	3.4E+03	1.0E+04	4.1E+04	1.3E+02	3.8E+02	1.5E+03	3.8E+05	1.1E+06	4.5E+06	**1.9E+02**	**5.7E+02**	**2.3E+03**
9	25	1170	2.4E+03	7.3E+03	2.9E+04	9.0E+01	2.7E+02	1.1E+03	2.7E+05	8.0E+05	3.2E+06	**1.3E+02**	**4.0E+02**	**1.6E+03**
10	50	770	3.2E+03	9.6E+03	3.9E+04	1.2E+02	3.6E+02	1.4E+03	3.5E+05	1.1E+06	4.2E+06	**1.8E+02**	**5.3E+02**	**2.1E+03**
11	20	950	1.6E+03	4.8E+03	1.9E+04	5.9E+01	1.8E+02	7.0E+02	1.7E+05	5.2E+05	2.1E+06	**8.7E+01**	**2.6E+02**	**1.0E+03**
12	90	1120	8.4E+03	2.5E+04	1.0E+05	3.1E+02	9.3E+02	3.7E+03	9.2E+05	2.8E+06	1.1E+07	**4.6E+02**	**1.4E+03**	**5.5E+03**
13	35	430	1.3E+03	3.8E+03	1.5E+04	4.6E+01	1.4E+02	5.6E+02	1.4E+05	4.1E+05	1.7E+06	**6.9E+01**	**2.1E+02**	**8.3E+02**
14	100	820	6.8E+03	2.1E+04	8.2E+04	2.5E+02	7.6E+02	3.0E+03	7.5E+05	2.3E+06	9.0E+06	**3.8E+02**	**1.1E+03**	**4.5E+03**
15	25	770	1.6E+03	4.8E+03	1.9E+04	5.9E+01	1.8E+02	7.1E+02	1.8E+05	5.3E+05	2.1E+06	**8.8E+01**	**2.6E+02**	**1.1E+03**
16	35	1860	5.4E+03	1.6E+04	6.5E+04	2.0E+02	6.0E+02	2.4E+03	6.0E+05	1.8E+06	7.2E+06	**3.0E+02**	**9.0E+02**	**3.6E+03**
17	25	580	1.2E+03	3.6E+03	1.5E+04	4.5E+01	1.3E+02	5.4E+02	1.3E+05	4.0E+05	1.6E+06	**6.6E+01**	**2.0E+02**	**8.0E+02**
18	70	170	9.9E+02	3.0E+03	1.2E+04	3.7E+01	1.1E+02	4.4E+02	1.1E+05	3.3E+05	1.3E+06	**5.5E+01**	**1.6E+02**	**6.5E+02**
19	30	490	1.2E+03	3.7E+03	1.5E+04	4.5E+01	1.4E+02	5.4E+02	1.3E+05	4.0E+05	1.6E+06	**6.7E+01**	**2.0E+02**	**8.1E+02**
20	30	100	2.5E+02	7.5E+02	3.0E+03	9.3E+00	2.8E+01	1.1E+02	2.8E+04	8.3E+04	3.3E+05	**1.4E+01**	**4.1E+01**	**1.7E+02**
21	100	810	6.8E+03	2.0E+04	8.1E+04	2.5E+02	7.5E+02	3.0E+03	7.4E+05	2.2E+06	8.9E+06	**3.7E+02**	**1.1E+03**	**4.5E+03**
22	10	130	1.1E+02	3.3E+02	1.3E+03	4.0E+00	1.2E+01	4.8E+01	1.2E+04	3.6E+04	1.4E+05	**6.0E+00**	**1.8E+01**	**7.2E+01**
23	10	110	9.2E+01	2.8E+02	1.1E+03	3.4E+00	1.0E+01	4.1E+01	1.0E+04	3.0E+04	1.2E+05	**5.0E+00**	**1.5E+01**	**6.1E+01**
24	35	310	9.0E+02	2.7E+03	1.1E+04	3.3E+01	1.0E+02	4.0E+02	9.9E+04	3.0E+05	1.2E+06	**5.0E+01**	**1.5E+02**	**6.0E+02**
25	200	530	8.8E+03	2.7E+04	1.1E+05	3.3E+02	9.8E+02	3.9E+03	9.7E+05	2.9E+06	1.2E+07	**4.9E+02**	**1.5E+03**	**5.8E+03**
Totals:			1.1E+05	3.3E+05	1.3E+06	4.0E+03	1.2E+04	4.8E+04	1.2E+07	3.6E+07	1.4E+08	6.0E+03	1.8E+04	7.2E+04

Notes:
Sediment density [lbs./ft^3]: 110
" = inch
DX = Annual erosion depth into landslide toe, assumed to be uniform across the landslide toe
aE+b = value expressed in scientific notation = a x 10^b
FT^3 = cubic feet
H_{TOE} = Height of landslide toe subject to local failure, erosion, and sediment production
W_{TOE} = Width of landslide measured across the landslide toe
YD^3 = cubic yard

Based on the above assumptions, annual rates of sediment production due to incremental creep of the 25 landslides would be as shown in Table 3:

Table 3
Annual Cumulative Sediment Production

RATE OF MOVEMENT (INCHES PER YEAR)	CUBIC YARDS PER YEAR	TONS PER YEAR
1	4,000	6,000
3	12,000	18,000
12	48,000	72,000

A total of 25 landslide sites have been identified that warrant additional evaluation. This evaluation will include field reconnaissance for all sites and then consideration of the need for and types of subsurface exploration and monitoring. This could include drilling and sampling, field testing, geophysics and packer testing, groundwater well installation and monitoring, laboratory testing, inclinometer installation and monitoring, and slope stability studies. Based on the results of geotechnical studies, informed decisions can be made regarding the need for and feasibility and cost of mitigating these landslides.

Landslide movement may also occur in response to surface water erosion of the toe, from direct ground saturation by precipitation, and by seismic event-induced ground motions. Shallow rapid landslides, sometimes known as skin slides, may be triggered around the perimeter of the reservoir owing to fluctuation of the pool water level. They are typically 3 to 10 feet thick and may range in volume from 10 to 10,000 CY. As is common for debris avalanches, they will likely contain a moderate to significant amount of vegetation, including mature trees. Because they initiate from very steep slopes and may free-fall from more than 10 to up to 200 feet, they may create a minor wave in pooled water or temporarily dam the river or creek.

The rate at which such mass movement will occur is impossible to estimate accurately. However, an estimate was made based on shallow rapid landslide rates in the Weyerhaeuser watershed analysis for the upper Chehalis River (Weyerhaeuser 1994). As shown in Table 4, the stream length of the river basin that will be inundated by the proposed reservoir is about 16 percent of the stream length of the entire upper Chehalis River basin. By taking 16 percent of the total of non-road-related landslides in the Weyerhaeuser report (for the whole basin), the potential number of natural landslides around the perimeter of the reservoir can be estimated. This yields a total of 55 landslides per 100 years or 0.55 shallow landslides per year. By using statistics from the Weyerhaeuser report for the delivery of sediment by landslides in the basin, it was determined that the average landslide size was about 6,500 CY. By multiplying the landslides per year (0.55) by the average landslide volume (6,500 CY), the annual production by shallow rapid landsliding is approximately 3,250 CY. This translates to 325,000 CY over a 100-year life of the dam.

As a check on the above estimated sediment production due to landsliding, it was compared to published representative sediment production rates (Roberts and Church 1986). In the Pacific Northwest forested environment, the quoted rate is 3 to 10 cubic meters per kilometer of stream channel per year. For the logged and roaded condition, that rate increases to 9 to 20 cubic meters per kilometer of stream channel per year. After conversion to English units, the rate is 250 to 900 CY per mile of stream channel per year for the forested condition and 800 to 1,700 CY per mile of stream channel per year for the logged and roaded condition. At its most conservative rate, it is still only about half of the rate estimated used for the Weyerhaeuser watershed analysis (Weyerhaeuser 1994). This difference is likely the result of intensive logging and roading in the time period of the aerial photographs (1955 to 1993) used by Weyerhaeuser. Therefore, if the reservoir is built and

the watershed is protected, it is likely that sediment production from areas above the reservoir and not affected by reservoir fluctuations will continue to decrease in the future approaching the Pacific Northwest average, as supported by Roberts and Church (1986).

Table 4
Spatial Frequency of Shallow Rapid and Small Sporadic Landslides

	CUMULATIVE STREAM LENGTH (STRAHLER STREAM ORDER ≥ 2)† [LINEAR MILES]	SHALLOW RAPID LANDSLIDES (NON-ROAD-RELATED)		SMALL SPORADIC LANDSLIDES (NON-ROAD-RELATED)		ALL NON-ROAD-RELATED LANDSLIDES	
		[LANDSLIDE PER 100 YEARS]	[LANDSLIDES / STREAM MILE]	[LANDSLIDES PER 100 YEARS]	[LANDSLIDES / STREAM MILE]	[LANDSLIDES PER 100 YEARS]	[LANDSLIDES / STREAM MILE]
Upper Chehalis River Watershed *	260	290	1.1	63	0.2	350	1.3
Inundation Zone of Proposed Upper Chehalis River Dam **	41	45	***	10	***	55	***

Notes:
† Stream lengths and Strahler stream-order values calculated using a geographic information system (GIS).
* Values from Weyerhaeuser (1994). *Chehalis Headwaters Watershed Analysis Mass Wasting Assessment.*
** Extrapolated values
*** Indicates the estimated number of landslides per stream mile is not significant.

Limitations

The conclusions and recommendations in this report are based on a review of existing information. No subsurface explorations were performed for this study. This work has been performed using practices consistent with geologic and geotechnical industry standards in the region for slope stability at a feasibility stage of the project. Prediction of slope movement with certainty is not possible with currently available scientific knowledge. As with any steep slope in this area, there are always risks of instability that present and future owners and operators must accept. Such risks include reservoir rise and fall, extreme or unusual storm events, and forest fire, among others. If conditions described in this report change, the authors of this report should be advised immediately so they can review those conditions and reconsider the conclusions and recommendations.

Shannon & Wilson, Inc., has included the enclosed Appendix 1, “Important Information About Your Geotechnical/ Environmental Report” to assist in understanding the use and limitations of the reports.

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Appendix A: Important Information About Your Geotechnical/Environmental Report

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-21897-001

Date: July 23, 2014
To: Chehalis Basin Workgroup

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Appendix D – Quarry Rock Materials TM

Quarry Rock Desktop Study

July 30, 2014

Prepared by: Shannon & Wilson, Inc.

Prepared for: Chehalis Basin Workgroup

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

LIST OF ACRONYMS AND ABBREVIATIONS

ASA	Aggregate Source Approval
ASR	alkali-silica reactivity
ASTM	American Society for Testing and Materials
CY	cubic yard
RCC	roller-compacted concrete
WDNR	Washington Department of Natural Resources
WSDOT	Washington State Department of Transportation

Executive Summary

Materials for the bulk of an earth and rockfill dam are available in the proposed reservoir area. Sand and gravel suitable for drainage and filter layers in an earth and rockfill dam are available within 40 miles of the dam site. Hard, durable rock for concrete aggregate for a roller-compacted concrete (RCC) dam is available in existing commercial and abandoned quarries within 25 miles of the dam site.

Three geologic formations are suitable for concrete aggregate, including Grande Ronde basalt, intrusive volcanics, and glacial outwash. Test results by the Washington State Department of Transportation and Northwest Testing (this study) indicate that concrete aggregate produced from these formations and deposit would be acceptable.

Both the Alderbrook and Hope Creek quarries can reportedly meet the volume requirements for the RCC dam alternatives within currently permitted reserves. Ample forested, undeveloped land is also available adjoining these existing quarries. The price of aggregate, crushed to size and delivered from the rock quarry to Pe Ell, Washington, was estimated by one rock pit owner at $12.95 per ton, but another owner estimated $20.00 to $23.00 per ton. For purposes of conversion to volume, the owner of the Alderbrook Quarry stated that 1 inch clean, loose basalt weighs 1.1 to 1.2 tons per cubic yard.

For rounded gravel and sand (concrete aggregate or drainage and filter materials), the only viable source is the glacial outwash deposit in the Centralia and Rochester area. One operator said that his pit could supply the required volume of materials for the drainage and filter layers, but another operator said that only a cooperative effort by all three local major producers could supply the projected volume for the earth and rockfill dam alternative. There is insufficient quantity of glacial outwash material in the Centralia and Rochester area pits to produce the aggregate volume required for the RCC dam alternatives. The price of rounded aggregate and sand, sorted to size, delivered from the pits in Centralia to Pe Ell was estimated by the two pit owners as $15.00 to $16.00 per ton.

Introduction

This report presents the results of a desktop study of the potential imported hard rock sources for concrete aggregate for a roller-compacted concrete (RCC) dam and sand and gravel borrow for a rockfill dam near Pe Ell, Washington (Figure 1). Both types of dams are under consideration for construction on the main stem of the Chehalis River to the south of Pe Ell. The purpose of this study is as follows:

- To determine if suitable quarry rock is within a reasonable distance of the dam site and ascertain the qualities of that rock for the RCC option
- To evaluate the availability of sand and gravel borrow within a reasonable distance of the dam site for a rockfill dam

The scope of services for this task included the following:

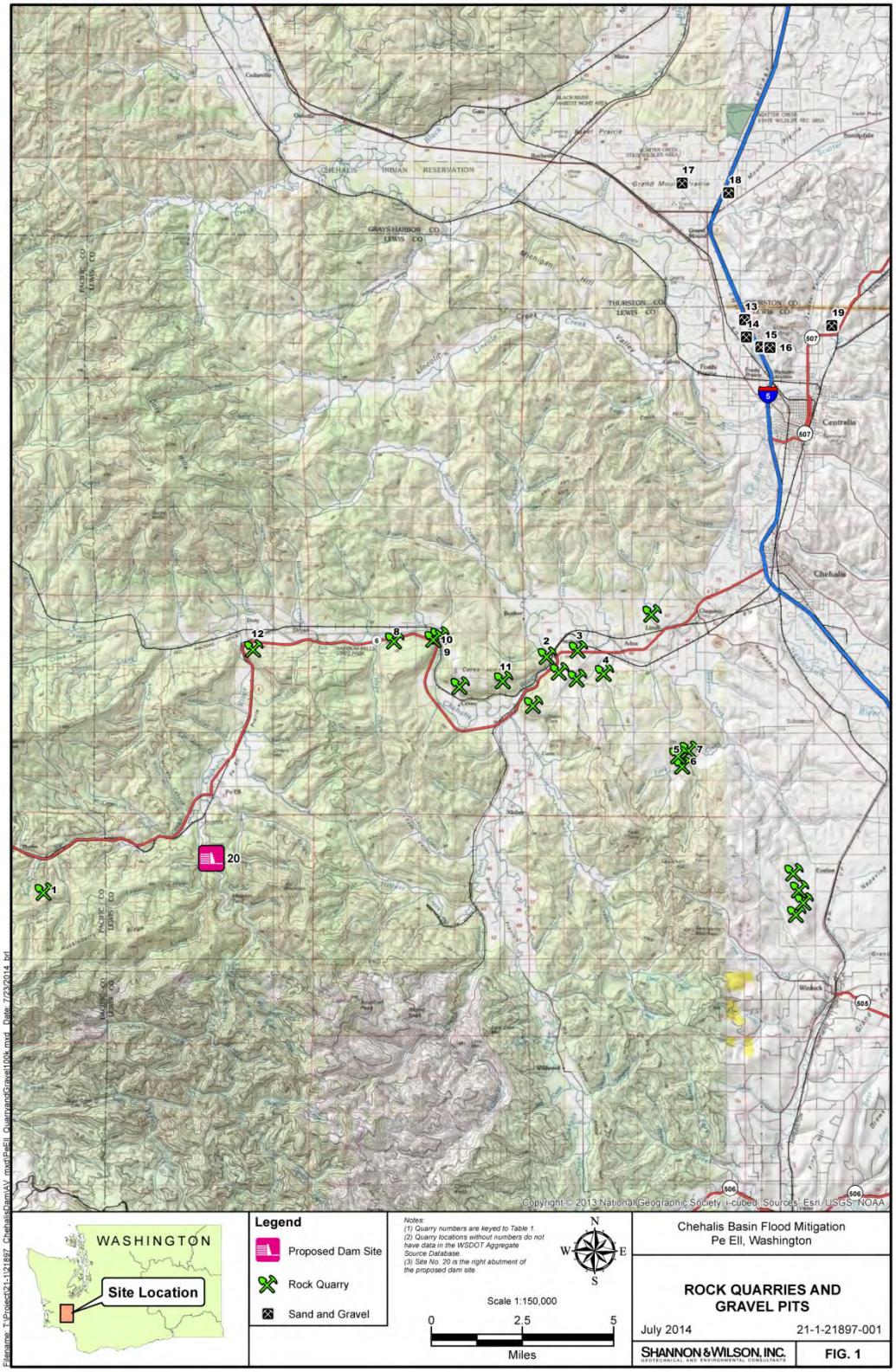
- Contacting the Washington Department of Natural Resources (WDNR) for locations of permitted and active rock pits in hard rock areas in the Pe Ell and Chehalis area
- Contacting the Washington State Department of Transportation (WSDOT) for information on rock in a WSDOT-owned pit
- Contacting owners of other public and private rock pits in the Pe Ell and Chehalis area
- Obtaining online information from WSDOT's Aggregate Source Approval (ASA) reports
- Synthesizing rock quarry test data and comparing the data to WSDOT testing standards of suitability
- Researching locations and suppliers of sand and gravel in the Pe Ell and Chehalis area
- Visiting rock quarries and sand and gravel suppliers in the Pe Ell and Chehalis area
- Performing alkali-silica reactivity (ASR) testing on three selected existing quarry rock samples
- Performing ASR, specific gravity, and absorption for rock exposed near the proposed dam site
- Discussing ASR test results with WSDOT materials engineers
- Preparing this technical memorandum

Based on HDR's presentation to the Water Retention Technical Committee on July 21, 2014, "Water Retention Structure Alternatives Cost Estimated Update and Climate Charge Cost Impacts," it is understood that the following approximate import quantities are required for the two types of dams:

- RCC – 1,400,000 cubic yards (CY) of quarry rock blasted and screened to size for coarse and fine concrete aggregate for multi-purpose dam
- RCC – 850,000 CY for quarry rock blasted and screened to size for coarse and fine concrete aggregate for flood control dam
- Rock Fill – 270,000 CY of rounded gravel and 270,000 CY of sand for drainage and filter zones in multi-purpose dam

The remainder of the materials (6,700,000 CY) for a rockfill dam would be produced from within the proposed reservoir area.

Figure 1
Rock Quarries and Gravel Pits



Geologic Considerations

The geologic landscape in the Pe Ell and Chehalis is diverse. It ranges from weak sedimentary to hard volcanic rock over very short distances. Glacial deposits are not present in close proximity to the proposed dam site; however, glacial outwash deposited south of the southernmost extent of repeated glaciations of the Puget Lowland is present in the Chehalis and Centralia valley.

Hard, durable rock or sand and gravel are required for concrete aggregate (RCC) and drainage and filter layers (rock fill) in the construction of the dam. Therefore, weak sedimentary rock, such as the Logan Hill, Lincoln Creek, McIntosh, Wilkes, and Montesano formations, has been eliminated from consideration for these construction materials in the dam. McIntosh sandstone and siltstone, which are close to the dam site, could be used for general fine-grained fill if judged suitable for such purpose.

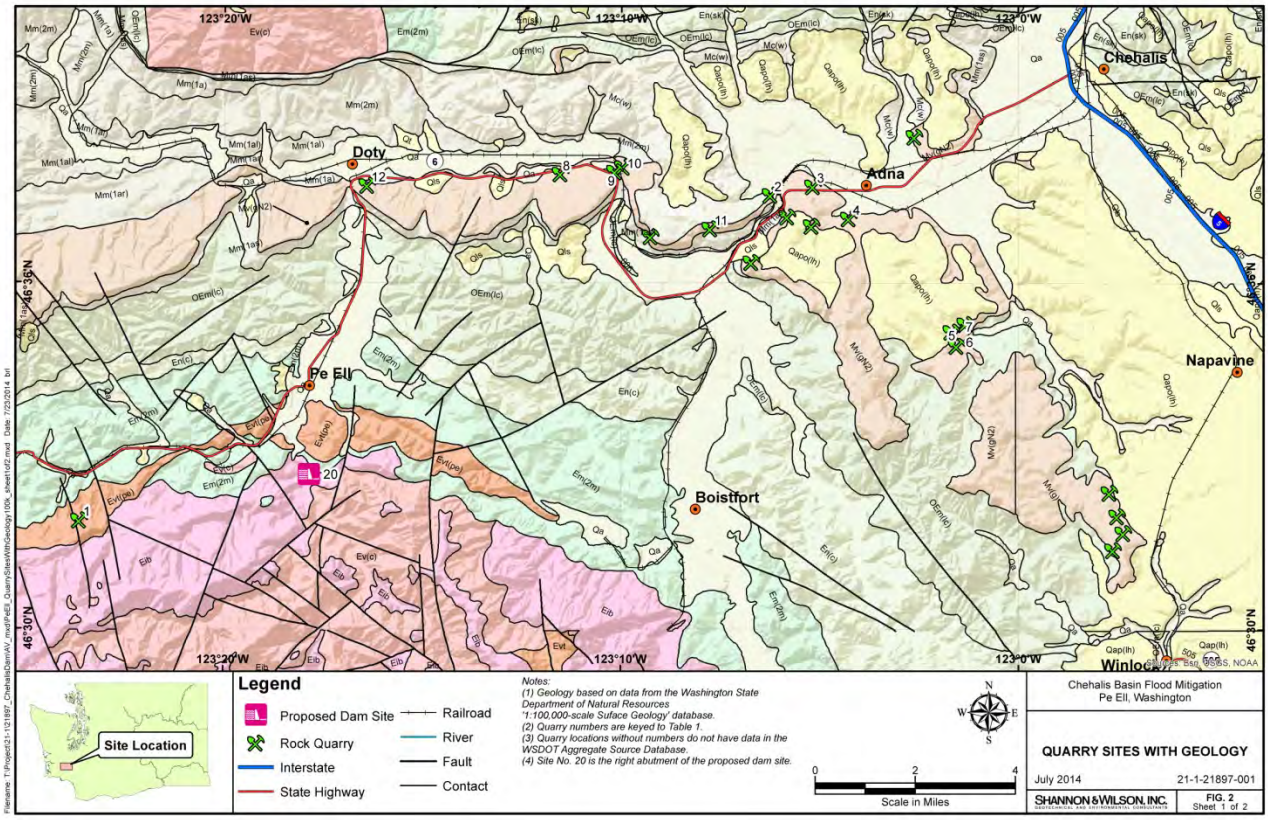
The rock formations in the project area and their general characteristics are presented below. They are named using two conventions. The older terminology, e.g., Crescent Formation, is based on mapping performed by Wells (1981). The newer terminology, e.g., Evt(pe), is based on recent compilation mapping by WDNR (2014). The geologic map using the newer terminology is presented in Figure 2.

The Crescent Formation [Ev(c) and Evt(pe)] comprises massive subaerial and submarine basalt flows, pyroclastic flows, and tuffaceous sedimentary rocks. It outcrops in the vicinity of the proposed dam site where it forms steep slopes. This basalt is high to very high strength, dark gray to black, and aphanitic to fine-grained. In the cliffs, it is massive but contains pillow structures as large as 10 feet in diameter. Owing to the unpredictability of the quality of the basalt in this formation, in part due to its marine origin, it may not be a good source for large quantities of high quality aggregate without an extensive exploratory program. It would be suitable as general rock fill for an earth or rockfill dam.

Intrusive volcanic rocks (Eib) consist primarily of gabbro outcrop at the dam site and the area to the west. The gabbro is a gray to black igneous rock that intrudes both the McIntosh and Crescent formations. It is high to very high strength, aphanitic to medium-grained, and has structure ranging from massive to columnar to blocky. Locally, quartz veins and quartz-filled cavities are present. This rock may be of use locally where it has to be excavated for dam or appurtenant structures. Because it is intrusive, it may not be consistently thick for the large volumes required for the concrete aggregate for the RCC dam option. Exploratory drilling would need to be performed to verify thicknesses and extents of the rock.

The Grande Ronde basalt, Mv(gN2), of the Columbia River Basalt Group is a dark gray to black subaerial extrusive rock that reaches western Washington from eastern Washington and Oregon. In the area near the proposed dam site, it outcrops in a narrow east to southeast-oriented band about 1 mile wide around the northern and northeastern edges of Willapa Hills. The closest outcrop of this geologic unit is about 4 miles north of the proposed dam site. Most of the commercial rock quarries near the dam site are developed in the Grande Ronde basalt.

Figure 2
Quarry Sites with Geology



Geologic Units

Unconsolidated Sediments

- Qa - Quaternary alluvium, dune sand, loess, and artificial fill
- Qt, Qls - Quaternary alluvial fans, beach deposits, undifferentiated sedimentary deposits, lacustrine deposits, landslides, peat, terraced deposits, and talus
- Qao; Qao(e) - Quaternary alpine glacial deposits, Fraser-age or younger
- Qgo; Qgog - Pleistocene continental glacial, glaciolacustrine, and outburst flood deposits, Fraser-age
- Qap(lh); Qapo(e); Qapo(h); Qapo(lh); Qapo(wh) - Pleistocene alpine glacial deposits, pre-Fraser

Sedimentary Rocks and Deposits

- Mc(w) - Quaternary-Oligocene continental sedimentary rocks
- OEn(t) - Oligocene-Eocene nearshore sedimentary rocks
- En(c); En(sk) - Eocene nearshore sedimentary rocks
- Mm(1a); Mm(1al); Mm(1ar); Mm(1as); Mm(2m) - Pliocene-Oligocene marine sedimentary rocks
- OEm(lc) - Oligocene-Eocene marine sedimentary rocks
- Em(1); Em(2); Em(2m); Em(2ms); Em(2s) - Eocene-Paleocene marine sedimentary rocks

Volcanic Rocks and Deposits

- Mv(sp) - Miocene Columbia River Basalt Group, Saddle Mountains Basalt
- Mv(g); Mv(gN2) - Miocene Columbia River Basalt Group, Grande Ronde Basalt
- Evt; Evt(pe) - Eocene volcaniclastic rocks, tuffs, and tuff breccias
- Ev(c); Eva(n); Evb(gr) - Eocene volcanic rocks (Crescent Basalt)

Intrusive Igneous Rocks

- Eib - Oligocene-Paleocene intrusive igneous rocks (gabbro)

Other Features

- Water

Chehalis Basin Flood Mitigation
Pe Ell, Washington

GEOLOGIC MAP

July 2014 21-1-21897-001

SHANNON & WILSON, INC. FIG. 2 Sheet 2 of 2

Alluvium (Qa) in the Chehalis River Valley between Pe Ell and Centralia is mostly fine-grained, having its origin in the mixed sedimentary and volcanic rocks to the west. It is not suitable for use as concrete aggregate or drainage and filter material. The alluvium in the Chehalis River Valley in and north of Centralia is glacial outwash originating in glacial ice that filled the Puget Lowland to the north several times during the Pleistocene Epoch. The clasts are rounded to subrounded and very high strength—a mixture of sand and gravel with scattered or layers of cobbles. Materials from these sources are likely suitable for use as concrete aggregate and drainage and filter material. Many gravel pits with this material are located in the Centralia, Grand Mound, and Rochester area.

Materials Information

Two public agencies track information on rock quarries and gravel pits: WDNR and WSDOT. The WDNR is responsible for permitting and regulating these facilities, and WSDOT requires periodic testing of the produced materials if the operator provides materials for public works projects.

Quarry and pit locations were obtained from the WDNR. The most up-to-date information regarding materials in the rock quarries and gravel pits is on a website maintained by WSDOT (Available from: www.wsdot.wa.gov/biz/mats/ASA). From this website and from discussions with the WSDOT Materials Quality Assurance Engineer (Rob Moholon), a table of test results for materials produced in the Pe Ell/Centralia/Chehalis area has been constructed (see Table1).

Categories obtained from the WSDOT website include location; WSDOT ID number; owner; and the most recent test results for absorption, specific gravity, degradation, ASR, and Los Angeles abrasion. Information, such as contact name, haul distance, price range, permit status, and activity, where known, were obtained from other sources. Included at the bottom of Table 1 are WSDOT criteria for absorption, apparent specific gravity, degradation, Los Angeles abrasion, and ASR.

Recent WSDOT test results are available for multiple quarry and pit sites for the tests discussed above. Only four test results were available for ASR (American Society for Testing and Materials International [ASTM] C1260) on glacial outwash gravel. Rock samples for ASR testing were obtained by visiting three rock quarries and the proposed dam site. Two active quarries (Alderbrook and Hope Creek, Photos 1 and 2 of Appendix 1) with many years of successful testing by WSDOT were visited. These two quarries are in areas mapped as Grande Ronde basalt. Rock samples were also taken at an inactive Weyerhaeuser Company rock pit about 4 miles west of the proposed dam site (Photo 3 of Appendix 1). This rock is Tertiary tuff and tuff breccia according to the geologic map rather than Grande Ronde basalt; however, it may be more representative of some of the rock at and in close proximity to the dam site. Rock samples were also obtained at the large outcrop of rock at the proposed dam site on the eastern side of the valley (Photo 4 of Appendix 1). This rock, mapped as intrusive gabbro, may be excavated during dam construction. A sand and gravel extraction site in Centralia, the Dulin pit (No. 14), was visited. This material is mined by a dragline excavating on the valley floor.

The rock samples obtained were tested by Northwest Laboratories, Inc., of Seattle, Washington. The test results are presented in Appendix 2. The samples for the quarry or potential quarry sites were tested for ASR using a 14-day test (ASTM C1260). The rock from the proposed dam site was also tested for specific gravity and absorption.

Table 1
Aggregate Source Database

OWNER	NO.	QUARRY TYPE	CONTACT NAME	HAUL DISTANCE (APPROXIMATE)	PRICE/TON	PERMITTED	ACTIVE	TOWNSHIP	RANGE	SECTION	WSDOT SOURCE ID	ABSORPTION	BULK SP. G (SSD)	DEG.	L.A. ABR.	ASR 14 DAY	ASR 1 YEAR
Weyerhaeuser Company	1	Rock	Eades	9		Y	N	12N	6W	11	V 28		2.73	11	11.6	0.01	
--	2	Rock		19		Y	N	13N	3W	7	L 72		2.72		17.4		
Parypa	3	Rock		20		Y	N	13N	3W	8	L 210		2.73	50	18.9		
O'Conner	4	Rock		22		Y	N	13N	3W	16	L 265		2.84	81	18.8		
--	5	Rock		24		Y	N	13N	3W	26	L 280	2.04	2.771	86	17		
Alderbrook	6	Rock	Moerke	24	$12.95	Y	Y	13N	3W	26	L 291	2.69	2.779	62	18	0.63	
--	7	Rock		24		Y	Y	13N	3W	26	L 258		2.79	59	17.9		
Hope Creek	8	Rock	Peterson	10	$20.00 - 23.00	Y	Y	13N	4W	9	L 298	2.8	2.718	67	18	0.70	
WSDOT Standard	9	Rock	Lowrey	11		Y	Y	13N	4W	10	L 107		2.83	86	18		
Lewis County	10	Rock	Jones	11		Y	N	13N	4W	10	L 300	0.9	2.836	79	17		
Halstrom	11	Rock		17		Y	Y	13N	4W	13	L 212		2.82	69	16		
Doty	12	Rock		6		Y	Y	13N	5W	11	L 21				13.8		
Dulin Construction	13	Sand and Gravel	Dulin	32		Y	N	15N	2W	30	L 249		2.65	66	21.8		
Dulin	14	Sand and Gravel	Dulin	32	$15.00	Y	Y	15N	2W	30	L 231	2.71	2.68	80	22	0.57	0.03
Breen	15	Sand and Gravel	Breen	32		Y	N	15N	2W	31	L 218		2.70	51	17.7		
--	16	Sand and Gravel		32		Y	N	15N	2W	31	L 114		2.65	63	20	0.31	
Dulin	17	Sand and Gravel	Dulin	37				15N	3W	2	J 143		2.68	70	24.1		
Pacific S&G	18	Sand and Gravel		37		Y	N	15N	3W	1	J 141		2.63	56	20.7		
Breen	19	Sand and Gravel	Breen	32	$16.00	Y	Y	15N	2W	28	L 268	1.9	2.69	70	12	0.57	
Weyerhaeuser Company	20	Rock	Eades	0		N	N	12N	5W	4	none	1.3	2.70			0.04	
WSDOT Criteria												3 maximum	2.55 minutes	30 minutes	35 maximum	See Note 4	0.04 maximum

Notes:
1) Haul Distance was measured with Google Earth.
2) Price per ton was obtained from owner/contact name.
3) Site No. 20, Weyerhaeuser property, is the proposed dam site.
4) For alkali-silica reactivity (ASR; 14-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.

Deg = Particle degradation test result determined in accordance with AASHTO T 104 test procedure.
L.A. ABR = Los Angeles Abrasion test result determined in accordance with AASHTO T 96 test procedure N = North, Township numbering for Township, Range, and Section property location
S&G = Sand and Gravel
SP. G = Specific Gravity, determined in accordance with AASTHO T 85 test procedure
SSD = Saturated surface dry
W = West, Range numbering for Township, Range, and Section property location
WSDOT = Washington State Department of Transportation

Conclusions

The quarry rock test results for use as concrete aggregate on the RCC dam are variable. Test results for absorption, specific gravity, and Los Angeles abrasion indicate the material would meet the WSDOT criteria. Test results for degradation indicate the rock meets WSDOT criteria except for a very low value for a test on rock from the Weyerhaeuser quarry (WSDOT Source I.D. V28).

The ASR test values for quarry rock in the pits that mine Grande Ronde basalt are high (0.63 and 0.70), whereas the tests for gabbro at the Weyerhaeuser rock pit and the proposed dam site are low (0.01 and 0.04) and considered innocuous. Therefore, the Grande Ronde basalt at the two commercial rock quarries (WSDOT Source I.D. L291 and L298) needs to be subjected to additional testing, such as petrographic analysis and longer ASR testing (ASTM C1567 and/or C1293), before determining if it would be suitable for concrete aggregate.

Both the Alderbrook (WSDOT Source I.D. L291) and Hope Creek (WSDOT Source I.D. L298) quarries can reportedly meet volume requirements for the RCC dam option within currently permitted reserves. Ample forested, undeveloped land is available adjoining existing pits to expand. The price of aggregate, crushed to size, from the pit to Pe Ell was estimated by one rock pit owner as $12.95 per ton, but the other pit owner estimated it to be $20.00 to $23.00 per ton. For purposes of conversion to volume, the owner of the Alderbrook Quarry stated that 1-inch clean, loose basalt weighs 1.1 to 1.2 tons per CY.

Sources for naturally occurring, hard, and durable gravel and sand are the glacial outwash borrow pits in the Centralia and Rochester area. All test values are acceptable, except for 14-day ASRs, which are high. The 1-year ASR tests on the sample from pit no. L231 in Centralia was acceptable. The 1-year test trumps the 14-day ASR test. Pit owners, Breen and Dulin, report that there are no known problems with ASR in the Centralia and Chehalis area. If these materials are to be used for concrete aggregate, additional testing will be necessary. One operator said that his pit could supply the required materials, but another operator said that only a cooperative effort by all three local major producers could supply the projected volume for the imported drainage and filter layers. The price of rounded aggregate and sand, sorted to size, from the pits in Centralia to Pe Ell was estimated by the two pit owners as $15.00 to $16.00 per ton delivered to Pe Ell.

It is recommended that additional testing be performed during the next phase of work, including, but not limited to:

- Additional source reconnaissance
- Additional laboratory testing of known and other rock and gravel sources
- Economic analysis of rock and gravel supplies
- Exploratory drilling and excavating of potential material supply sources

Limitations

The conclusions and recommendations in this report are based on existing public information and limited new test results, and the condition of the sites as they existed during the time of our field visits. No subsurface explorations were performed for this study. This work has been performed using practices consistent with geologic and geotechnical industry standards in the region for rock and gravel source evaluation during feasibility stages. If conditions or assumptions described in this technical memorandum change, the authors should be advised immediately so they can review those conditions/assumptions and reconsider the conclusions.

Shannon & Wilson, Inc., has included the enclosed Appendix 3, “Important Information About Your Geotechnical/Environmental Report” to assist y in understanding the use and limitations of the report.

References

WDNR (Washington State Department of Natural Resources), 2014. Surface geology database, 1:100,000 Scale. Available from: http://www.dnr.wa.gov/ResearchScience/Topics/GeosciencesData/Pages/gis_data.aspx.

Wells, R.E., 1981. Geologic map of the eastern Willapa Hills, Cowlitz, Lewis, Pacific, and Wahkiakum counties, Washington: U.S. Geological Survey Open File Report OF-81-674, scale 1:62500.

Appendix A: Photographs

Photo 1. Alderbrook Quarry.

Photo 2. Hope Creek Quarry.

June 2014
21-1-21897-001

SITE PHOTOGRAPHS

FIG. A-1
Sheet 1 of 2

Photo 3. Weyerhaeuser Quarry.

Photo 4. Proposed Dam Site, East Abutment.

June 2014
21-1-21897-001

SITE PHOTOGRAPHS

FIG. A-1
Sheet 2 of 2

Appendix B: Northwest Laboratories, Inc. Test Results

NORTHWEST LABORATORIES *of Seattle, Incorporated*
ESTABLISHED 1896
Technical Services for: Industry, commerce, Legal Profession & Insurance Industry
241 South Holden Street • Seattle, WA 98108-4360 • Phone: (206) 763-6252 • Fax: (206) 763-3949 www.nwlabs1896.com

Report To: Shannon & Wilson
Attention: William Laprade

Date: June 12, 2014

Report On: Concrete Aggregate

Lab No.: E88277

SUBMITTED:

Eight (8) Bags Of Aggregate

IDENTIFICATION:

1. Rock Creek A-Line Quarry
2. Hope Creek Quarry
3. Alderbrook Quarry
4. Chehalis Dam Sight

CHEMICAL ANALYSIS:

Four (4) submitted aggregate samples were crushed to specified size fractions and tested per ASTM C-1260-07 Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method).

LaFarge Portland cement grade I/II was used.
Amount of mixing water used was 0.52 fraction of cement by mass.

The following expansion values were obtained:

Rock Creek Quarry:	-0.01% - innocuous behavior
Hope Creek Quarry:	0.70% - potentially deleterious expansion
Alderbrook Quarry:	0.63% - potentially deleterious expansion
Dam Sight:	0.04% - innocuous behavior

Note:

Innocuous behavior -	Less than 0.10%
Potentially deleterious expansion -	More than 0.20%

Between 0.10% and 0.20% - includes both

Chehalis Dam Sight aggregate was tested per ASTM C 127 – 07 Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate.

NORTHWEST LABORATORIES *of Seattle, Incorporated*

Shannon & Wilson
Page – 2 –
E88277

The following results were obtained:

Relative Density (OD)	2.67 g/ml
Relative Density (SSD)	2.70 g/ml
Apparent Relative Density	2.72g/ml
Density (OD)	2660 kg/m3
Density (SSD)	2690 kg/m3
Apparent Density	2710kg/m3
Absorption	1.3%

This report applies only to the actual samples tested. Northwest Laboratories does not certify, warrant, or guarantee any products manufactured by others. Samples discarded within thirty **(30) days** unless otherwise requested in writing by you.

NORTHWEST LABORATORIES, INC.

Shapovalov

Anton Shapovalov, Chemist

wbm

Appendix C: Important Information About Your Geotechnical/Environmental Report

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-21897-001

Date: July 22, 2014
To: Chehalis Basin Workgroup

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Appendix E – Vegetation and Debris Management TM

Reservoir Vegetation and Debris Management, and Related Operational Considerations

July 30, 2014

Prepared by Shannon & Wilson, Inc.

Prepared for Chehalis Basin Workgroup

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PHOTOS

LIST OF ACRONYMS AND ABBREVIATIONS

AF	acre-feet
AWS	Additional Water Storage
BRD	Blue River Dam
cfs	cubic feet per second
HAHD	Howard A. Hanson Dam
LWD	large wood debris
MMD	Mud Mountain Dam
RCC	roller-compacted concrete
USACE	U.S. Army Corps of Engineers
WRB	Willamette River Basin

Introduction and Report Objectives

1 Introduction and Report Objectives

This report discusses some aspects of vegetation, large woody debris (LWD), and sediment management that may need consideration for the flood control only and multi-purpose dam alternatives being evaluated for the Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species study. The information presented regarding operations at existing northwest dams and reservoirs are based on vegetation, LWD, and sediment management and operations data for selected dams and reservoirs operated by the United States Army Corps of Engineers (USACE) Seattle and Portland Districts. This information was obtained through telephone discussions and email correspondence from January through mid-April 2014, and during a site visit to Mud Mountain Dam (MMD) and reservoir and Howard A. Hanson Dam (HAHD) and reservoir on April 16, 2014.

Shannon & Wilson, Inc., representatives Stan Boyle, P.E. (Geotechnical Engineer); Andy Caneday, L.E.G. (Engineering Geologist); and Brooke O'Neill (Biologist) participated in the April 16, 2014, site visit. USACE representatives participating in the April 16, 2014, site visit for USACE Seattle District were Richard Smith (Dam Safety Program Manager), Daniel Johnson (Operations Project Manager for both dams), Rick Emry (MMD Maintenance Supervisor), and Ellen Ingborg (Assistant Dam Safety Program Manager). Information on USACE Portland District dams and reservoirs was obtained through discussion with Mr. Dave Scofield, USACE Portland District Dam Safety Program Manager.

Some of the information presented herein was obtained from USACE websites.

1.1 REPORT OBJECTIVES

The information about operations of existing dams and reservoirs presented in this report was gathered to develop an understanding of the vegetation and debris management and operations issues that may be expected for the Chehalis Basin flood control only and multi-purpose dam alternatives being evaluated. Information was collected for one flood control only dam and 16 multi-purpose dams currently in operation in Western Washington and Western Oregon. Limited information was also obtained for the multi-purpose Libby Dam near Libby, Montana.

MMD, HAHD, and the USACE Portland District dams were selected because they are in environments similar to the environment near Pe Ell, Washington, where the Chehalis Basin dam being evaluated would be constructed. MMD is a flood control only dam, i.e., a permanent pool is not normally maintained. MMD is operated similar to that being evaluated for the current project flood control only scenario. HAHD and the USACE Portland District dams and reservoirs considered are operated as multi-purpose facilities, i.e., while providing flood flow storage, permanent pools are maintained for extended periods of time to provide other benefits such as river flow augmentation, drinking water, and hydroelectric power.

After information on operations at existing dams is presented, this report discusses implications of what was learned about existing facility operations regarding operational considerations and costs for vegetation, LWD, and debris management for the Chehalis Basin Strategy dam and reservoir alternatives.

Mud Mountain Dam

2 Mud Mountain Dam

2.1 DAM AND RESERVOIR DESCRIPTION

- MMD is a flood control only, run of river dam that holds a pool only during flood events, which generally occur several times a year.
- MMD data:
  - Completion date: 1948
  - Purpose: Flood control
  - Dam type: Rockfill with concrete cutoff wall in earth core
  - Structural height: 432 feet
  - Drainage area: 400 square miles
  - Reservoir capacity: 154,056 acre-feet (AF)
  - Low level outlet tunnel: 9-foot-diameter, 4,600 cubic feet per second (cfs) capacity
  - High outlet tunnel: 23-foot-diameter, 19,550 cfs maximum, 12,000 cfs flood control
- Since dam operation began, only five major floods have resulted in substantially high temporary reservoir pool elevations.
- Since beginning operations in 1948, the MMD reservoir has not filled to the elevation necessary to overtop the spillway.

2.2 RESERVOIR VEGETATION

- Most of the reservoir is vegetated.
- Areas without vegetation are limited to the active stream channel.
- Vegetation within the reservoir generally consists of willow shrubland on the lower slopes, transitioning to a multi-story broadleaf forest, and transitioning to a conifer forest on the highest reservoir slopes and above the maximum reservoir pool elevation.
- Shrubs observed on the lower slopes predominately consist of Sitka willow (*Salix sitchensis*) and black cottonwood (*Populus balsamifera*).
- Trees in the multi-story broadleaf forested slopes are dominated by alder (Alnus rubra) and black cottonwood (*Populus balsamifera*).
- Higher elevation slopes are dominated by western red cedar (*Thuja plicata*) and big leaf maple (*Acer macrophyllum*).
- Flood events bring water levels that over-top shrub vegetation on the lower slopes an estimated three times per year.
- Flood events bring water levels that reach into the cedar-dominated forested mid-elevation slopes an estimated one to two times per year.
- Invasive species, consisting predominantly of Himalayan blackberry (*Rubus armeniacus*), are prevalent surrounding the log storage fields where heavy equipment operates.

Photo 1
Vegetation on MMD reservoir slopes.

Photo 2
Vegetation at lowest level of MMD reservoir. LWD stored on reservoir bottom. Log boom around perimeter of LWD to maintain containment of the LWD should reservoir water elevations rise.

2.3 VEGETATION MANAGEMENT

- Little vegetation management is currently conducted by USACE personnel.
- Vegetation and trees that grow on the reservoir slopes are not cut or removed.
- Hiking trails that extend below flood pool elevations are occasionally cleared for recreational access.
- Log storage areas are cleared to allow equipment to operate safely.

2.4 LARGE WOODY DEBRIS MANAGEMENT

- During flood events at MMD, LWD (from upstream sources) can be backed up for miles above the dam.
- More LWD is delivered to the reservoir during large flood events than during lower flow or more typical flood and elevated stream flow events.
- Floating booms are employed to collect a large portion of LWD that enters the reservoir before the LWD reaches the outlet works.
- LWD that is not collected in the floating booms either collects on the trash racks or is passed through the MMD outlets tunnels.
- Workers try to let as much LWD pass through the MMD outlet tunnels as possible.
- During flooding events, crews work from boats to turn debris to allow it to get through the trash rack and into the outlet tunnels.
- Sediment deposited upstream of the trash rack as a result of holding a pool in the reservoir is later removed by washing it through the lower 9-foot-diameter tunnel. Concentration of flow into the outlet pipe to lower the pool and headcutting of the sediment under the water surface can cause waves 2- to 3-feet-tall on the water surface.
- LWD that cannot pass through the low level trash rack (18-inch-wide opening) is gathered up with log booms by boat and stored in designated areas in the reservoir for later removal by trucks.
- Lowering of the MMD pool may be delayed for multiple days or weeks following flood events that deliver large amounts of LWD to the reservoir to facilitate LWD removal and storage.
- Lowering the reservoir prior to LWD removal could result in large volumes of LWD being deposited on reservoir slopes. During subsequent storm events, this LWD could refloat and potential block outlet work trash rack structures.
- LWD is made available to various stakeholders (e.g., USACE, Tribes, and King and Pierce County) for their use in habitat restoration projects.
- LWD not claimed by the various stakeholders is temporarily stored in the reservoir.
- Until about 2012, LWD that was not claimed by the various stakeholders was burned. Burning, while still allowed, is not practical for routine disposal of remaining woody debris because of restrictions on burning and air quality considerations. USACE is developing new management plans to address LWD management, including chipping, land application of chipped wood, and disposal outside the reservoir limits.

Photo 3
Tugboat with log boom collecting LWD for transport to upstream storage site. MMD trash rack structure around inlet of outlet tunnel in left-center of photo.

Photo 4
MMD work barge, tugboat, and support boat used for LWD and dam maintenance.

Photo 5
LWD stored in MMD reservoir basin about 0.5 mile upstream of the dam.

Photo 6
Log boom around LWD stored in MMD reservoir basin to maintain containment of the LWD should reservoir water elevations rise.

2.5 SEDIMENT MANAGEMENT

- MMD is operated as a run of river project, except during floods.
- Soil, gravel, cobbles, and boulders are allowed to be flushed through the low elevation 9-foot-diameter outlet tunnel that passes through MMD.
- To reduce damage to the higher elevation concrete-lined 23-foot-diameter tunnel, MMD is operated to pass the majority of the gravel, cobbles, and boulders through the steel-lined 9-foot-diameter tunnel.
- Recently, the USACE has been passing additional flow through the 23-foot-diameter tunnel to facilitate downstream fish passage.
- A boulder deposit downstream of the 9-foot-diameter outlet tunnel that included boulders about 3 feet dimension was observed.
- The material carried by water passing through the tunnel damage the steel liner through abrasion, hammering, and water cavitation.
  - The steel liner has been replaced subsequent to initial dam construction.
  - USACE has found that softer steel has less installation problems and does not crack upon installation as does harder, stronger steel.
  - USACE last relined the 9-foot tunnel in the mid- to late-1990s.
  - MMD operators anticipate the 9-foot tunnel will need to be relined in the next few years, at an estimated cost in excess of $15,000,000.
- Flood events deposit silt and sand on upstream dam structures and road surfaces, including access and work platforms around the trash racks and inlets to outlet works.
  - This sediment may be a couple of feet thick following a flood event.
  - This sediment is removed following flood events by excavating it from surfaces where access is required. The sediment is put in the river, which transports it through the dam and downstream.
  - A significant amount of sediment, ranging from silt to large cobbles and small boulders deposit in the reservoir when the pool is elevated. These materials may erode and pass though the reservoir when there is no pool. These materials source from glacial deposits originating on Mount Rainier.

2.6 RESERVOIR SLOPE STABILITY

- USACE does not take action to prevent landslides occurring in the reservoir.
- USACE has no operating procedures or reservoir slope stability guidance documents that establish maximum reservoir drawdown rates for the purpose of limiting potential for reservoir slope instability.
- USACE is aware that slumping and small-scale landslide movement occurs on reservoir slopes during drawdown. It is USACE's opinion that these small-scale slumps and landslides do not pose a threat to the dam or dam operations.
- In the 1970s, a significant landslide that posed a threat to dam operations occurred in the reservoir immediately upstream of the dam. Future landslides may pose a threat to the dam and dam operations.
- About once per year, a reconnaissance is made around the reservoir perimeter to assess reservoir slope conditions and document landslides and instability.
- There is a large ancient landslide downstream of the MMD left abutment. It is USACE's opinion that this landslide is not a threat to the dam.

2.7 RESERVOIR DRAWDOWN

- During the rainy season, following storm events, dam operators lower the reservoir as quickly as practical to the elevation necessary to provide storage capacity required for the next potential storm, with consideration for downstream impacts and predicted near-term weather.

- The reservoir drawdown rate, i.e., reservoir discharge flow, is constrained by downstream limitations associated with river stage, river channel capacity; levee conditions and ability to contain the flow; and potential impacts to bridges, roadways, other improvements, and people.
- USACE may reduce the discharge flow to reduce potential for, or actual, downstream impacts and to allow for in-reservoir debris management.
- Reservoir drawdown rates for MMD are about 10 feet per day average following flood events, but reservoirs are drawn down faster when the reservoir is near full.
- The maximum theoretical drawdown rate for the full pool condition is about 34 feet per day, based on published data for outlet gate and tunnel capacity and reservoir volume and surface area. This potential drawdown rate has not yet occurred. This maximum drawdown rate may have been built into the project to address dam safety considerations, and may not be appropriate for reservoir slopes or the dam for repeated rapid drawdown events.
- MMD is a flood control only structure, with no permanent or sustained pool. Lowering of the MMD pool may be delayed for multiple days or weeks following flood events to facilitate LWD removal and storage.
- Recently, tests have been conducted to hold a pool during some times of year to increase survivability of juvenile fish by allowing downstream fish passage through the higher elevation 23-foot-diameter outlet tunnel.

2.8 OPERATION COSTS AND PERSONNEL

See Operation Costs and Personnel section for HAHD.

Howard A. Hanson Dam

3 Howard A. Hanson Dam

3.1 DAM AND RESERVOIR DESCRIPTION

- HAHD is a flood control and multi-purpose facility, providing:
  - Flood flow storage
  - Summer stream flow augmentation for fish
  - Ecosystem restoration
  - Drinking water (City of Tacoma has water rights)
- HAHD data:
  - Completion date: 1962
  - Purpose: Multi-purpose
  - Dam type: Rockfill with earth core
  - Structural height: 235 feet
  - Drainage area: 221 square miles
  - Reservoir capacity: 105,463 AF (design flood pool elevation 1,206 feet)
  - Outlet tunnel: 19-foot-diameter, 12,000 cfs maximum, 10,000 cfs flood control
  - Bypass tunnel: 4-foot-diameter, 500 cfs maximum
- Since dam operation began, only five major floods have resulted in substantially high temporary reservoir pool elevations.
- Since beginning operations in 1962, the HAHD reservoir has not filled to the elevation necessary to overtop the spillway.
- HAHD is managed such that the (conservation pool) reservoir reaches maximum pool elevation at the start of summer.
- HAHD holds a summer conservation pool for low-flow augmentation. Filling of the conservation pool begins near the end of February to early March and reaches its maximum level in May or June.
- Maximum pool elevation is currently 1,167 feet. The reservoir is held at this elevation for about a month and then slowly released throughout the summer and fall for stream flow augmentation, salmon and steel head migrations starting in June or July, and to supply drinking water for the City of Tacoma.
- From the 1990s to 2007, the peak conservation pool elevation was 1,147 feet. Prior to that, peak conservation pool elevation was 1,141 feet.
- The Additional Water Storage (AWS) project raised the conservation pool to the current elevation of 1,167 feet. Future raises in the conservation pool elevation are planned.

3.2 RESERVOIR VEGETATION

- Vegetation is sparse below the maximum conservation pool elevation.
- Extensive vegetation does not grow on the reservoir slopes because water within the reservoirs is maintained high for months before lowering.
- During the April 16, 2014, site visit, reservoir slopes below the maximum pool elevation consisting of exposed soil and rock with dead trees were observed. Dead trees observed below the maximum pool

elevation are remnants from before the AWS project, when the maximum pool elevation was maintained 20 feet lower than today.

- Above the maximum pool level, slopes are forested with Douglas fir (*Pseudotsuga mensiezii*), western red cedar (*Thuja plicata*), western hemlock (*Tsuga heterophylla*), alder (*Alnus rubra*), and big leaf maple (*Acer macrophyllum*).
- Areas outside the reservoir were observed to consist of the same tree species observed within the reservoir (see above) as well as understory species, including western swordfern (*Polystichum munitum*) and salmonberry (*Rubus spectabilis*).
- Little understory could be observed from the vantage point on the dam.
- Invasive species were not observed from the vantage point on the dam.
- Prior to the AWS project, emergent wetland vegetation reportedly occurred in some areas of low gradient adjacent to the reservoir at and just below the pre-AWS maximum reservoir elevation of 1,147 feet.
- Vegetation growing near and just below the pre-AWS maximum reservoir elevation reportedly provided important elk forage habitat. USACE personnel expect that new wetland vegetation will establish at the new conservation pool elevation in suitable areas.

Photo 7: Vegetation and trees around the upper part of the HAHD reservoir killed after the conservation pool elevation was raised. Shallow failures of soil from slope have occurred and exposed soil and rock.

3.3 VEGETATION MANAGEMENT

- The HAHD reservoir was likely logged initially, during dam construction, up to elevation 1,206 feet.
- Within the reservoir, little vegetation management is currently conducted by USACE personnel.
- Vegetation and trees that grow on the reservoir slopes below maximum pool elevation are not cut or removed.
- The USACE does not have property rights to conduct vegetation management within most of the HAHD reservoir and watershed.
- When the conservation pool elevation at HAHD was recently raised to elevation 1,167 feet, transitional habitat was provided by not cutting brush or harvesting timber in the inundation zone.

3.4 LARGE WOODY DEBRIS MANAGEMENT

- LWD cannot pass through the dam.
- More LWD is delivered to the reservoir during large flood events than during lower flow or more typical flood and elevated stream flow events.
- Floating booms are employed to collect a large portion of LWD that enters the reservoir before the LWD reaches the outlet works.
- USACE has floating log-catchment booms anchored at different elevations to collect logs for different flood events.
- Relatively recently, USACE added a log-catchment boom that is anchored at an elevation suitable for the maximum flood elevation at HAHD. This boom was added because existing booms were anchored at too low of an elevation for the maximum flood elevation.
- The LWD collected is towed using a boat and floating boom to an upstream side channel for temporary or permanent storage.
- As part of habitat mitigation efforts, about 50 percent of the LWD captured at HAHD is hauled downstream by truck and reintroduced to the river under an adaptive management program.
- LWD reintroduced to the river downstream of HAHD has reportedly caused some issues, including log jams at downstream bridges, although information for specific instances were not obtained.
- LWD not hauled downstream of the dam is stockpiled and made available to various stakeholders (e.g., USACE, King and Pierce County, Tribes, and City of Tacoma) for their use in habitat restoration projects.
- LWD not hauled downstream of the dam or claimed by various stakeholders is stored in the reservoir to decompose.
- Until about 2012, LWD that was not claimed by the various stakeholders was burned. Burning, while still allowed, is not practical for routine disposal of remaining woody debris because of restrictions on burning and air quality considerations. USACE is developing new management plans to address LWD management, including chipping, land application of chipped wood, and disposal outside the reservoir limits.

Photo 8
Driftwood and LWD in HAHD reservoir.

3.5 SEDIMENT MANAGEMENT

- HAHD has experienced sediment buildup in the reservoir; however, the current sediment buildup is not enough to affect project operations.
- Surveys of the reservoir suggest that about 1,200 AF of storage has been lost to sediment buildup since the project was built in 1962. In 1962, the available storage at normal full pool (1,206 feet) was about 105,460 AF; today it is about 104,260 AF. So, in about 50 years, the project has lost about 1 percent of the originally available storage at normal full pool to sedimentation.
- Sediment management prior to 2008 was focused to determine if the project could be operated in such a way that USACE could actively sluice stored sediment out of the reservoir and into the downstream river. There were a lot of operational and environmental (stakeholder opposition) challenges to such an operation. Tests of this operation (i.e., reservoir drawdowns) indicated that this method didn't seem to be a very effective means of removing stored sediment from the reservoir.
- There is currently no plan for dealing with sediment at HAHD.
- Gravel, cobbles, and boulders are purchased, delivered, and placed on the riverbank downstream of HAHD as part of the downstream river adaptive management program. The streambed gravel, cobbles, and boulders, and LWD placed at the same location are eroded and moved downstream by the river during high flow. These actions are required to comply with the current Biological Opinions from National Marine Fisheries Services and the U.S. Fish and Wildlife Service for the continued operation and maintenance of the dam to mitigate for the dam blockage and capture of the natural downstream passage of material.

3.6 RESERVOIR SLOPE STABILITY

- About once per year, a reconnaissance is made around the reservoir perimeter to assess reservoir slope conditions and document landslides and instability. The USACE produces a summary document of the observations made during this reconnaissance and compares observations to observations made in prior years.
- USACE has no operating procedures or reservoir slope stability guidance documents that establish maximum reservoir drawdown rates for the purpose of limiting potential for rapid drawdown causing reservoir slope instability.
- The right abutment consists of a large landslide.
- Seepage through the right abutment has been problematic and a maintenance problem since the reservoir was first filled.
- Seepage and stability-related concerns about the right abutment resulted in USACE placing limits on the maximum reservoir pool elevation that could be maintained.
- Recent completion of subsurface groundwater drainage improvements in the right abutment will allow the maximum pool elevation to be raised.

Photo 9
Exposed soil where shallow failure occurred on HAHD reservoir slope.

3.7 RESERVOIR DRAWDOWN

- During the rainy season, following storm events, dam operators lower the reservoir as quickly as practical to the elevation necessary to provide storage capacity required for the next potential storm, with consideration for downstream impacts and predicted near-term weather.

- The reservoir drawdown rate, i.e., reservoir discharge flow, is constrained by downstream limitations associated with river stage; river channel capacity; levee conditions and ability to contain the flow; and potential impacts to bridges, roadways, other improvements, and people.
- USACE may reduce the discharge flow to reduce potential for or actual downstream impacts and to allow for in-reservoir debris management.
- Reservoir drawdown rates for HAHD are about 10 feet per day average following flood events, but reservoirs are drawn down faster when the reservoir is near full. HAHD maximum drawdown rate following a 1965 flood event was about 13 feet per day averaged over about 4 days, which occurred following the 1965 event. Another HAHD post-flood drawdown event (date not provided) had an average drawdown rate of about 13 feet per day over 7.5 days, with shorter term peak drawdown rate near about 20 feet per day occurring in that period (Dan Johnson, personal communication).
- The maximum theoretical drawdown rate for the full pool condition is about 30 feet per day, based on published data for outlet gate and tunnel capacity and reservoir volume and surface area. This potential drawdown rate has not yet occurred.

3.8 OPERATION COSTS AND PERSONNEL

- USACE employs an equivalent of eight full-time employees at MMD and eight employees at HAHD for operations, maintenance, and administration (i.e., about 16 full-time equivalents total).
  - Personnel consist of operators and maintenance staff (Dam Mechanics) at each dam, five administrative staff to support both dams, and an on-site Biologist who supports both dams.
  - Operators and maintenance personnel perform multiple functions such as dam operation, LWD removal, boat operations, trash rack clearing, inspections, road maintenance, and equipment operation and maintenance.
  - Other management and supplemental personnel that do not work at the project site are required for tasks such as dam operation, dam safety, capital improvement planning, and funding requests and authorization. These individuals, who may only spend a portion of their time working on tasks specific to the MMD and HAHD dams, are not included in the count of individuals associated with each dam. The cost for these individuals is included in the dam operating budget (see below).
- The annual operating budget is about $3,500,000 each for MMD and HAHD.
  - The operating budget does not include capital expenditures such as periodic repairs to outlet tunnel at MMD.
  - Operating budget includes an allowance for USACE Seattle District environmental support, dam safety personnel support and inspections, regulatory management, and personnel involved in operations decisions.
  - MMD and HAHD budgets include work barges, tugboats, and other boat- and land-based equipment required to corral LWD, to haul LWD to storage areas, and for clearing and cleaning trash racks and removing sediment from around outlet works.
  - The annual operating budget includes nearly $300,000 for the HAHD downstream river nourishment program for:
    - Purchasing, delivery, and placing streambed gravel, cobbles, and boulders on the riverbank downstream of HAHD.
    - Placing LWD removed from HAHD reservoir on the riverbank downstream of HAHD.
    - Monitoring and studies of the effectiveness of the nourishment program.

Other USACE Seattle District Dams

4 Other USACE Seattle District Dams

4.1 RESERVOIR SLOPE STABILITY

- For the Libby Dam, near Libby, Montana, which is also managed by the USACE Seattle District, the reservoir elevation does not go up and down as quickly as at MMD and HAHD, but USACE has had to locally buttress reservoir slopes to mitigate slope instability.
- No other information has been obtained regarding specifics of slope instability that has occurred or slope buttressing.

USACE Portland District Dams

5 USACE Portland District Dams

5.1 DAM AND RESERVOIR USES

- USACE Portland District operates 15 flood control dams in Western Oregon; 13 in the Willamette River Basin (WRB); and 2 in the Coast Basin.
- The dams are flood control and multi-purpose.
- For the WRB dams, during the rainy season flood waters may be stored in the dam reservoirs.
- The Portland District manages the reservoirs such that they are near full at the start of summer.
- Reservoirs are maintained high through the summer.
- Water is discharged throughout the summer and fall to augment stream flows to improve water quality and conditions for fish.
- Eight of the WRB dams include hydroelectric power generation.
- Many of the WRB reservoirs are accessible to the public and provide recreation opportunities, including boating, fishing, picnicking, walking, and biking.

5.2 VEGETATION

- Extensive vegetation does not grow on the WRB reservoir slopes because water within the reservoirs is maintained high for extended periods of time.
- Soil and rock are exposed on reservoir slopes when the reservoirs are low.

5.3 VEGETATION MANAGEMENT

- USACE Portland District has no vegetation management plan for reservoirs it operates.

5.4 LARGE WOODY DEBRIS MANAGEMENT

- LWD is removed from the reservoirs.
- No information was obtained regarding the disposal of LWD removed from reservoirs.

5.5 SEDIMENT MANAGEMENT

- No information on sediment management was obtained.

5.6 RESERVOIR SLOPE STABILITY

- USACE Portland District reports some issues with reservoir slope instability on first filling.
- Subsequent to first filling, where instability occurs, the reservoir slopes mostly experience creep movement.

- USACE has no operating procedures or reservoir slope stability guidance documents that establish maximum reservoir drawdown rates for the purpose of limiting potential for reservoir slope instability for the dams they operate.

5.7 RESERVOIR DRAWDOWN

- USACE reports typical reservoir drawdown rates of 4 to 6 feet per day following flood events for WRB reservoirs.
- The reservoir drawdown rate, i.e., reservoir discharge flow, is constrained by downstream limitations associated with river stage; river channel capacity; and potential impacts to bridges, roadways, other improvements, and people.
- Reservoir drawdown rates may be higher following smaller flood events or in summer when downstream river flows are not as high.
- During the rainy season, following storm events, dam operators lower reservoirs as quickly as practical to the elevation necessary to provide storage capacity required for the next potential storm, with consideration for downstream impacts and predicted near-term weather.
- Dam operators may reduce the discharge flow to reduce the potential for downstream impacts and to allow for in-reservoir debris management.
- The Blue River Dam (BRD) reservoir, east of Eugene, Oregon, is in a narrow canyon. BRD reservoir can rise nearly 100 feet per day during floods. Following one such event, the drawdown rate at BRD was about 5 feet per day.
- To address juvenile passage issues at Fall Creek dam, operations have recently been modified to increase fish passage efficiency and to transport gravel past the dam by lowering the multi-purpose reservoir water surface to below the design minimum conservation pool elevation in November and December (source: Bob Willis, Anchor QEA, August 11, 2014).

5.8 OPERATION COSTS AND PERSONNEL

- No dam or reservoir operating costs were provided.

Chehalis River Flood Control Only Dam Alternative

6 Chehalis River Flood Control Only Dam Alternative

6.1 INTRODUCTION

A roller-compacted concrete (RCC) dam is the only dam type being considered for the flood control only dam alternative. The dam configuration being considered would have multiple (possibly 8 to 10) channels that pass through the dam structure (low-level openings) near the stream channel elevation. Each low-level opening would be about 20 to 30 feet wide and about 10 feet high. The openings would be configured to allow fish to travel upstream and downstream past the dam for normal river flows, i.e., not during flood conditions. Each low-level opening would have a gate that could be closed during flood events.

When water is impounded by the dam, flood water would discharge through a bypass tunnel through one of the abutments. Low-level opening gate closure timing will be critical as flood events begin. Trash racks and intake tower structures would be constructed at the upstream end of the tunnel inlet. The bypass tunnel, intakes, and inlet gates would be sized such that the tunnel could pass the design flood flows and lower the reservoir at the design post-flood rate sufficient to reestablish fish passage through the dam within the desired interval.

The spillway would be integral to the RCC dam. Spilled water would pass over the central portion of the dam to the downstream channel.

6.2 DAM AND RESERVOIR VEGETATION INITIAL REMOVAL

Slopes would be cleared and grubbed (vegetation and roots removed) where the dam and dam appurtenances would be constructed and for staging and access areas and borrow and disposal sites. Clearing and grubbing would extend sufficient distances beyond the work zones to limit vegetation interference with the construction and as appropriate to reduce construction costs. It would not be necessary to strip and grub much of the reservoir for the RCC alternative. Stripping would be required at the RCC dam construction area, where RCC aggregate is mined from a project-developed quarry within the reservoir, and at excavated material disposal sites in the reservoir. Thus, for the RCC dam alternative, it is not anticipated that clearing and grubbing would be required throughout the reservoir.

It is anticipated that marketable timber within the reservoir, below the design maximum pool elevation would be harvested during dam and reservoir construction. An alternative to harvesting all marketable timber below the maximum pool elevation may be to limit initial timber harvesting to lower elevations, e.g., below the elevation of a selected flood event pool. It may be possible to consider harvesting trees containing harvestable timber that are left in the reservoir at the upper pool elevations at a later date, potentially after they are killed due to later inundation. The practicality of this alternative has not been verified.

Where timber harvesting occurs in the reservoir away from construction areas requiring clearing and grubbing, tree stumps and understory vegetation would likely be left in place to reduce erosion. Debris generated by timber harvest activities that could float, interfere with reservoir outlet works, plug gates, or require collection from the reservoir after dam and reservoir completion and water impoundment should be removed as part of the timber harvest activities. This debris would include branches, logs, and waste wood products.

6.3 VEGETATION MANAGEMENT

Vegetation capable of frequent to infrequent inundation would be expected to grow and become established outside the active channel and on reservoir slopes after the reservoir begins operating. Vegetation management would not likely include cutting, trimming, or removal of live vegetation that grows after initial reservoir clearing, unless hazards requiring removal are identified. Taking this approach would be similar to the vegetation management approach taken by USACE for MMD and other Pacific Northwest reservoirs they manage.

Vegetation would be expected to self-select for the inundation frequency it experiences. Vegetation that is infrequently inundated and becomes distressed, dies, or is floated or washed loose of the reservoir bottom and sides during storm events and elevated pool events would be flushed through the dam outlet tunnel or low-level openings through the dam, if possible, and small enough to pass the openings. Once downstream of the dam, the vegetation debris could be transported downstream by the river. Vegetation and debris too large to pass through the dam outlet tunnel or low-level openings would need to be collected and removed from the trash racks and reservoir.

6.4 LARGE WOODY DEBRIS MANAGEMENT

LWD management will likely be a significant aspect of the dam and reservoir operation. The multiple large dimension, low-level openings proposed for the flood control only RCC dam and the relatively large-diameter outlet works tunnel would likely be capable of passing a significant amount of the LWD that enters the reservoir. Woody debris may be captured on trash racks and held in the reservoir during flood events to protect low-level openings, the tunnel, and gates from damage or plugging. As flood flows decrease and the reservoir pool elevation lowered, gates on low-level openings would be opened. Some LWD would likely be allowed to pass through the low-level openings. Allowing some LWD to flow through the dam to the river downstream of the dam may provide some benefits to the downstream river environment, and may reduce costs associated with transporting LWD to downstream of the dam and LWD removal from the reservoir. A LWD management plan should be developed to establish criteria for the size of LWD to pass through the low-level openings and what size LWD should be collected in the reservoir.

Managing LWD will require anchored log booms to collect floating LWD and debris and labor, boats, and towable log booms to collect LWD and transport it to a permanent or temporary storage area. LWD management may require that access roads be constructed and maintained so that LWD can be cut up and hauled out of the reservoir. Agreements for project-related stakeholders or beneficiaries and non-stakeholders to remove some of the LWD that they could use may be part of reservoir operations, similar to USACE practice at MMD and HAHD.

USACE stated that the volume and diameter of LWD that they have had to manage and dispose of at MMD and HAHD has decreased over the decades since the dams initially came into operation. The reasons for this are not clear. The decrease in dimension of LWD may result in part from large old growth timbers having been harvested prior to reservoir construction and the greater volume and number of younger, i.e., non-old growth, timber in the watershed upstream of the MMD and HAHD reservoirs. Soon after dam and reservoir

construction, there may have been more dead, fallen, and non-marketable larger and old growth timber in the watershed. This material may have been transported into the river and reservoir since MMD and HAHD were constructed, and there is now much less of this material available. A similar decrease in volume and dimension of LWD may occur with time at the proposed dam as existing, dead, fallen, and older trees are delivered to the reservoir and the source of these materials decreases with time.

The volume of LWD available to be transported downstream of the dam site may increase after dam construction relative to the volume of LWD that is currently carried downstream past the dam site. This increase would likely result from the relative ease at which LWD that enters the reservoir can float on the reservoir surface to the dam. Whereas, under current conditions, LWD in various streams that flow into the river and LWD in the river itself may take years to be transported downstream to the dam site.

It is unclear if the river, riverbanks, bridges, and other features downstream of the proposed dam site could accommodate all LWD that could be delivered to the reservoir and possibly pass through the low-level openings and the tunnel. LWD management may include retaining some or significant volumes of LWD to reduce potential for impacts or damage to downstream areas. For smaller flood events where relatively little LWD is delivered to the dam and floats in the reservoir pool, the presence of this LWD may not delay reservoir lowering. For smaller flood events, limited delay to reservoir lowering and restoration of run-of-the-river flow, and thus fish passage, may be required to collect and haul the relatively low volumes of LWD by boat to a side channel for storage.

Infrequent, high volume, extended duration flood events are likely to deliver disproportionately higher volumes of LWD to the reservoir than frequent, lower volume, shorter duration flood events. Where the LWD that accumulates cannot be discharged through the dam, whether for reasons associated with dam operations or potential downstream impacts, it may be necessary to collect and temporarily store the LWD in the reservoir for later disposal. Log booms and boats would be used to collect the LWD and haul it upstream to side-channel storage areas. Following flood events that deliver large volumes of LWD, lowering the reservoir may need to be slowed or delayed by days to weeks to allow LWD to be collected and removed. Delays in lowering the reservoir would delay the restoration of run-of-the-river flow through the low-level outlets, and thus delay fish passage through the dam.

Funding should be allocated for labor and equipment to collect, store, and dispose of LWD; for post-flood LWD removal from gates and trash racks and the reservoir; and to repair LWD-related damage to gates and trash racks.

6.5 SEDIMENT MANAGEMENT

Sediment would be delivered to the reservoir area from streams and debris channels that flow into the reservoir. Erosion of reservoir slopes, slumps and shallow landslides occurring on reservoir slopes, and erosion at the toe of existing larger landslides would also deliver sediment to the reservoir area. Sediment could include clays, silt, sand, gravel, cobbles, and boulders. When there is no pool behind the reservoir, some of this material would remain mobilized or be temporarily deposited and later remobilized and transported downstream. Normal stream flow and lower volume flood events could create flow velocities sufficient to erode material from the stream and river bottoms, stream and river banks, and reservoir slopes. These flows would transport the sediment down the river channels to the dam.

When there is a pool behind the dam, fine-grained sediment may stay suspended and be transported to near the dam or pass through the outlet tunnel. Coarser sediment would likely be deposited near the location where it enters the reservoir pool as deltas and as alluvial fans where debris flows enter the reservoir pool. As the

reservoir pool is lowered, and flow velocities increase, some of the coarser sediment would be eroded and remobilized and transported toward the dam. Depending on stream, side-channel, and river flows, some of the sediment could be flushed out of the reservoir during a particular flood event, between flood events, or by future flood events.

Where sediment is transported to the dam, the sediment could be flushed out of the reservoir to downstream of the dam by arranging the low-level openings through the dam to encourage sediment transport. This might be accomplished by constructing one or more of the openings at lower elevation than the other openings so as to concentrate flow velocity and move larger particles, including boulders. Installing the inverts of the low-level outlets at slightly different elevations from one another and operating the gates in a manner that concentrates flow to erode and wash coarse sediment through the low-level openings would allow sediment to be delivered downstream of the dam to replenish stream aggregates. Steel armoring of the invert and walls of low-level outlet openings may be required, or prudent, to reduce damage and erosion of dam concrete by sediment as it moves through the dam. Periodic cleaning of the gate closure location as well as regular gate inspection and testing will be required to assure that coarse sediment does not impede gate closure during flood events.

Allowing coarse aggregate to wash through the tunnel could damage the tunnel lining and increase the frequency at which the tunnel lining must be repaired or replaced. To reduce potential damage to the outlet tunnel, the tunnel inlet could be constructed at an elevation that will reduce the volume of coarse sand, gravel, cobbles, and boulders that are flushed through the tunnel, while maintaining the design function of the tunnel.

Designing the flood control only dam such that sediment can pass through or by the dam would reduce operating costs related to removing trapped sediment and allow downstream augmentation of river bedload material, i.e., sand, gravel, cobbles, and boulders.

Funding should be allocated to develop an inspection schedule and protocol and periodically inspect the gates, trash racks, low-level outlets, and the tunnel and to repair sediment-related damage in a timely manner.

6.6 RESERVOIR SLOPE STABILITY – RAPID DRAWDOWN

Stability of reservoir slopes and existing deep-seated landslides are discussed in a separate report prepared for this project.

Shallow landslides are expected to occur around the reservoir perimeter in response to vegetation removal and reservoir elevation fluctuations. Reservoir pool elevation fluctuation-related shallow landslides, e.g., ranging from less than 1 foot thick to 3 to 6 feet thick, are expected to occur on steep slopes below the maximum pool elevation and just above the maximum pool elevation. The occurrence of these landslides will be related to steepness of slopes; weathering of the soil by freezing and thawing, wetting and drying, root penetration, and other factors; saturation by precipitation and snowmelt; and reduction in soil stability when the reservoir pool elevation rises and during reservoir pool drawdown. Estimated sediment volumes that could be delivered to the reservoir by debris flows and shallow and deep-seated landslides are discussed in separate reports prepared for this project.

Reservoir operation plans should include provisions for yearly, or more frequent, if appropriate, reconnaissance of the reservoir to observe and document slumping of reservoir slopes, shallow landslides, and movement of deep-seated landslides. Provisions should be included in operating plans for installing and monitoring survey points and instruments to measure and track landslide movement.

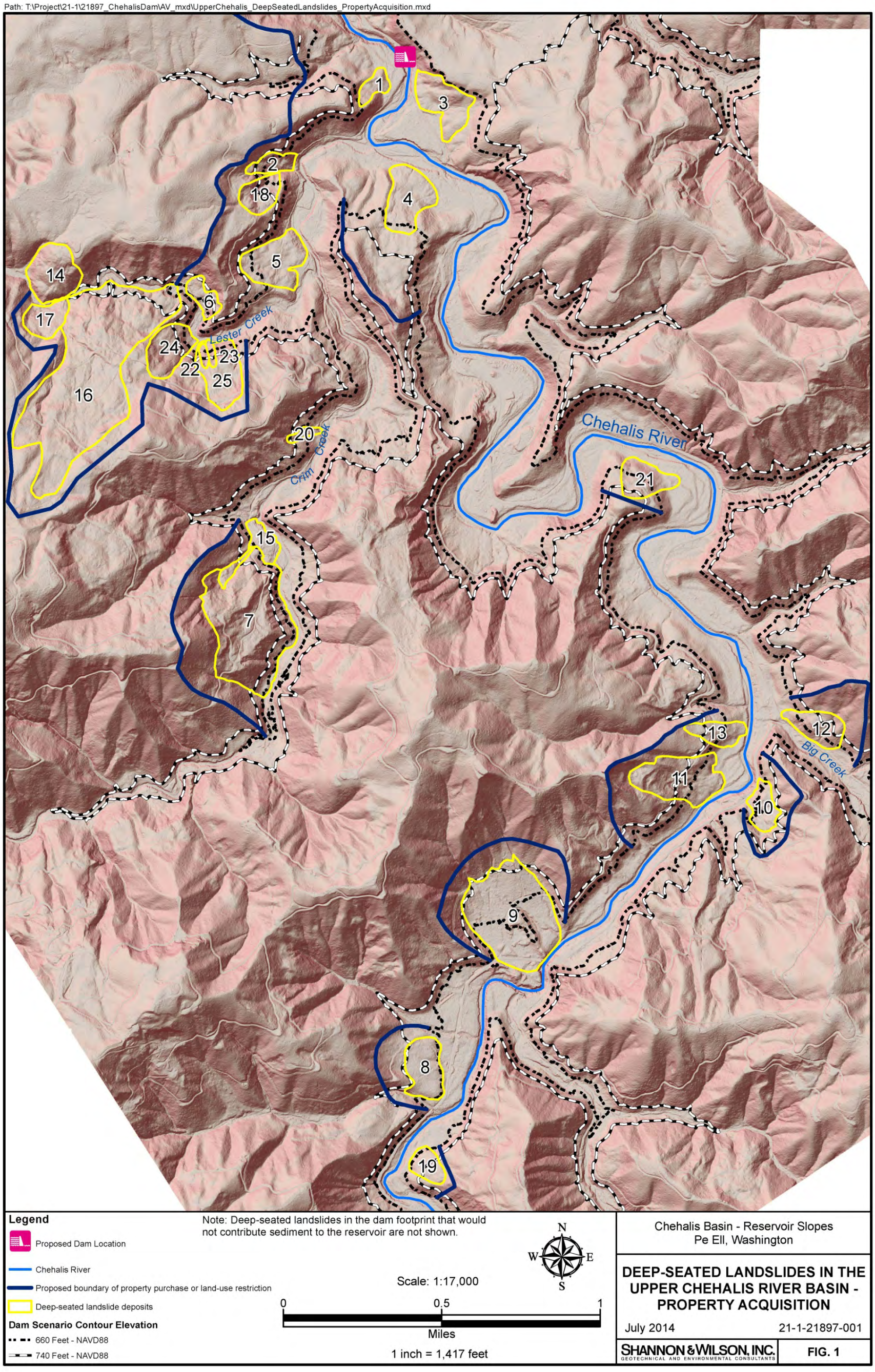
Some of the existing identified deep-seated landslides could move or be eroded, or behave differently than they historically have behaved, after the tree cover is removed during dam construction, and in response to saturation and reservoir raising and lowering. Changes in erosion pattern near landslide toes could accelerate landslide movement. Movement of these landslides could pose a hazard to the dam or reservoir. Movement of these landslides could pose a hazard to people or structures on the landslides or adjacent areas. Because of these potential impacts, consideration should be given to purchasing or putting use-restrictions on the properties or portions of properties that encompass these landslides beyond the reservoir limit. Property agreements, whether the land is acquired or the titles encumbered with use-restrictions, should include provisions for future access to the landslide surfaces and headscarps for monitoring, explorations, and implementation of potential stability improvement measures and the rights to perform these activities.

Figure 1 presents identified landslides that extend beyond the reservoir limits and the approximate limits of land around the landslides that might be considered for purchase or land use restriction as part of the project.

Landslides could occur or be present at locations other than those identified for this study. An allowance should be included in the long-term operating budget for future purchase or stability improvement of existing landslides not identified and landslides that occur in response to dam and reservoir construction and operation.

Figure 1
Deep-Seated Landslides in the Upper Chehalis River Basin – Property Acquisition

Path: T:\Project\21-1\21897_ChehalisDam\AV_mxd\UpperChehalis_DeepSeatedLandslides_PropertyAcquisition.mxd



6.7 RESERVOIR DRAWDOWN

A primary consideration for the project and dam design, including sizing of the low-elevation openings through the dam and reservoir outlet tunnel, is to provide the ability to drawdown the reservoir as quickly as possible to restore upstream and downstream fish passage through the dam. The outlet tunnel must be sized to lower the reservoir at a sufficient rate to provide storage for the next potential storm.

Reservoir drawdown rates could be limited by a number of factors. These factors include dam stability, dam abutment slope stability, reservoir slope stability, sediment and LWD management considerations and activities, and downstream flow constraints. Secondary to stability of the dam itself, downstream flow constraints are the primary factors limiting reservoir drawdown rates for existing USACE Seattle and Portland District flood control dams. Downstream flow constraints include life safety considerations; river stage; ability of rivers and levees to contain flows; downstream erosion and sediment deposition; and potential for flows to damage bridges, roads, structures, and other improvements. Sediment deposition and time and flow-variable changes in sediment deposits in the channel downstream of the dam could reduce river capacity to accept flows.

Rapid reservoir drawdown should not be a concern for stability of a properly founded, designed, and constructed RCC dam, where full or partial excavation of potentially unstable material occurs in conjunction with abutment and foundation stabilization measures. While RCC dams do develop internal pore-pressures during operation, impacts from excess pore pressure within RCC structures are significantly less critical to RCC dam operation than to embankment dam operation. Foundation grout curtains and foundation drains below the dam and in the abutments should be designed to limit pore pressures below the dam to acceptable levels.

Without implementation of slope stabilization or drainage measures, rapid reservoir drawdown could contribute to instability of the naturally occurring soil and rock in dam abutments and reservoir slopes. Abutment and reservoir slope instability, should it occur, could pose a hazard to dam operations. Geologic assessment, subsurface investigations, and geotechnical analyses of abutment and reservoir slopes and existing landslides would be performed as part of project design. Existing landslides and slopes found to be unstable under rapid drawdown conditions could result in the need to place constraints on the reservoir drawdown rate or require measures be implemented to improve stability of unstable landslides and slopes. The preferred approach would be to construct stability improvement measures for existing landslides near the dam and within the reservoir that are characterized as potentially posing unacceptable risk to the dam or dam outlet works operations should they mobilize. Stabilization measures may include full or partial removal of unstable material. Order of magnitude costs for design-phase subsurface explorations and to implement reservoir and abutment slope stability improvements during dam construction should be included in the estimate of dam and reservoir design and construction costs.

USACE Portland district reports typical post-flood reservoir drawdown rates of 4 to 6 feet per day. For MMD and HAHD, USACE reports typical post-flood drawdown rates on the order of 10 feet per day, maximum recorded multi-day sustained average drawdown rate of about 13 feet per day at both dams, and maximum recorded short-term (possibly for part of a single day) sustained drawdown rate of 20 feet per day on one occasion for HAHD. MMD and HAHD both are designed and constructed such that they could theoretically discharge water through their outlet tunnels to drawdown the reservoirs on the order of 30 feet per day, although this drawdown rate has not been experienced at either reservoir.

If stability measures are implemented during dam construction for landslides and slopes near the dam and in the reservoir that could pose a hazard to the dam, a typical multi-day sustained drawdown rate of 10 feet per day could be assumed for the current assessment of the dam and dam operations. A faster drawdown rate of 20 feet per day could be assumed for a short period (one to two days) during the drawdown cycle. A maximum

drawdown rate of 30 feet per day could be assumed. The maximum drawdown rate would apply to rapid-succession storm scenarios where it is important to re-establish reservoir storage capacity following a storm for dam safety considerations. The above assumptions apply to the Chehalis River flood-control only dam under consideration.

Slower drawdown rates are preferred from a slope stability perspective, but may not be preferred from an operational needs perspective. The sooner the reservoir is lowered after inundations, the less infiltration of water into the slopes, and thus reduced potential for deep seated landslides to be mobilized by rapid drawdown pore pressures. Drawdown soon after inundation may reduce the risk of saturating deep seated landslides and reduce potential for instability during drawdown. Drawdown rates provided above are based on historical operation of USACE Seattle and Portland District dams. Geologic reconnaissance, subsurface explorations, and engineering analyses should be performed as part of the next phase of the Chehalis Basin Strategy study to confirm the appropriateness of these assumed drawdown rates with regard to abutment and reservoir slope stability, develop recommendations for stability improvement measures, and assess risk and operation impacts associated with rapid drawdown.

6.8 OPERATION COSTS AND PERSONNEL

There is not sufficient information available to develop project-specific staffing, equipment, and operation requirements for the dam and reservoir being evaluated. In the absence of this information, it is assumed that the dam and reservoir would have staffing and equipment needs similar to MMD, with consequent similar operating costs. It is also assumed the dam and reservoir would be operated similar to MMD with regard to water impoundment; water release; and vegetation, LWD, and sediment management. Based on these assumptions, the following staff, equipment, and operating costs may be assumed:

Personnel:

- Equivalent of eight full-time personnel for dam operations and maintenance and administration
- A quarter-time equivalent Dam Safety Program Manager

Equipment:

- Tugboat, work barge, and inspection boat
- Mid-sized excavator
- Operator and maintenance personnel vehicles

Operating costs:

- $3,500,000 per year

Reservoir slope stability improvement costs:

- $200,000 to $10,000,000

Chehalis River Multi-Purpose Dam Alternative

7 Chehalis River Multi-Purpose Dam Alternative

7.1 INTRODUCTION

An RCC dam and a clay-core rockfill dam are being considered for the multi-purpose dam alternative. A permanent pool approximately 130 feet deep would be maintained behind the dam. Unlike the flood control only RCC dam alternative, there would be no stream or fish passage openings through the multi-purpose dam. For both the RCC and rockfill dam alternative, a rockfill dam embankment would extend for hundreds of feet from the main dam right abutment to raise the ground surface east of the right abutment to the desired top of dam elevation.

Water would be routinely impounded behind the dam and the reservoir water surface elevation maintained such that there is sufficient water storage volume available for potential precipitation and flood events. Water would be slowly released throughout the late spring, summer, and fall to augment downstream flows. Flood water would discharge through a bypass tunnel through one of the abutments. Trash racks and intake tower structures would be constructed at the upstream end of the tunnel inlet. This bypass tunnel, intakes, and inlet gates would be sized such that the tunnel could pass the design flood flows. Gates installed at multiple elevations in the intake tower would allow for mixing of water from different elevations to aid in controlling water temperature and air-entrainment quality of water discharged downstream.

A spillway would be integral to the RCC dam. Spilled water would pass over the central portion of the RCC dam to the downstream channel. For the rockfill dam alternative, a spillway structure would be constructed through one of the dam abutments to discharge water downstream of the dam or through the hill on the right abutment to discharge into the adjacent valley. Spillway discharge to the adjacent valley would flow into the Chehalis River some thousands of feet downstream of the dam.

7.2 DAM AND RESERVOIR VEGETATION INITIAL REMOVAL

For the RCC dam alternative, slopes would be cleared and grubbed (vegetation and roots removed) where the dam and dam appurtenances would be constructed and for staging and access areas and borrow and disposal sites. Clearing and grubbing would extend sufficient distances beyond the work zones to limit vegetation interference with the construction and as appropriate to reduce construction costs. It would not be necessary to strip and grub much of the reservoir for the RCC dam alternative. Stripping would be required at the RCC dam construction area, where RCC aggregate is mined from a project-developed quarry within the reservoir, and at excavated material disposal sites in the reservoir. Thus, for the RCC dam alternative, it is not anticipated that extensive clearing and grubbing would be required throughout the reservoir.

For the rockfill dam alternative, it is anticipated that materials to construct the rockfill shell of the dam, and possibly the dam core, would be derived from within the reservoir. Clearing, grubbing, and stump removal

would be required for a significant surface area within the reservoir to provide access to soil and rock that would be excavated and used in the rockfill dam. Clearing and grubbing would extend sufficient distances downstream of the dam, along the spillway, and around work zones to limit vegetation interference with the construction and as appropriate to reduce construction costs. For the rockfill dam alternative, it is anticipated that extensive clearing and grubbing would be required throughout the reservoir.

It is anticipated that marketable timber within the reservoir, below the design maximum pool elevation would be harvested during dam and reservoir construction. An alternative to harvesting all marketable timber below the maximum pool elevation may be to limit initial timber harvesting to lower elevations, e.g., below the elevation of a selected flood event pool. It may be possible to consider harvesting trees containing harvestable timber that are left in the reservoir at the upper pool elevations at a later date, potentially after they are killed due to later inundation. The practicality of this alternative has not been verified.

Where timber harvesting occurs in the reservoir away from construction areas requiring clearing and grubbing, tree stumps and understory vegetation would likely be left in place to reduce erosion. Debris generated by timber harvest activities that could float, interfere with reservoir outlet works, plug gates, or require collection from the reservoir after dam and reservoir completion and water impoundment should be removed as part of the timber harvest activities. This debris would include branches, small logs, and waste wood products.

7.3 VEGETATION MANAGEMENT

Reservoir slopes below the elevation at which the reservoir pool is maintained for extended periods of time would not be expected to have vegetative cover, i.e., soil and rock would be exposed in the reservoir below the sustained pool elevation. Vegetation capable of frequent to infrequent inundation would be expected to grow and become established on the upper reservoir slopes after the reservoir begins operating. Vegetation management would not likely include cutting, trimming, or removal of live vegetation that grows after initial reservoir clearing, unless hazards requiring removal are identified. Taking this approach would be similar to the vegetation management approach taken by USACE for HAHD and other Northwest reservoirs they manage.

Vegetation that does become established on higher elevation slopes in the reservoir would be expected to self-select for the inundation frequency it experiences. Vegetation that is infrequently inundated and becomes distressed, dies, or is floated or washed loose of the reservoir bottom and sides during storm events and elevated pool events and that cannot be flushed through the dam outlet tunnel may need to be removed from the reservoir.

7.4 LARGE WOODY DEBRIS MANAGEMENT

LWD management will likely be a significant aspect of the dam and reservoir operation. LWD that enters the reservoir area will remain in the reservoir. LWD would be captured on trash racks and held in the reservoir during flood events to protect the outlet tunnel and gates from getting damaged or plugged. A LWD management plan should be developed to establish criteria for the size of LWD to pass through the low-level openings and what size LWD should be collected in the reservoir.

Managing LWD will require anchored log booms to collect floating LWD and debris; labor, boats, and towable log booms to collect LWD and transport it to a permanent or temporary storage area; and that access roads be constructed and maintained so that LWD stored in the reservoir can be cut up and hauled out of the reservoir. LWD transport downstream of the dam and re-introduction of some volume of LWD to the river may be necessary as part of dam and reservoir operations to maintain or provide certain environmental conditions

downstream of the dam. Agreements for project-related stakeholders or beneficiaries and non-stakeholders to remove some of the LWD that they could use may be part of reservoir operations, similar to USACE practice at MMD and HAHD.

USACE stated that the volume and diameter of LWD that they have had to manage and dispose of at MMD and HAHD has decreased over the decades since the dams initially came into operation. The reasons for this are not clear. The decrease in dimension of LWD may result in part from large old growth timbers having been harvested prior to reservoir construction and the greater volume and number of younger, i.e., non-old growth, timber in the watershed upstream of the MMD and HAHD reservoirs. Soon after dam and reservoir construction, there may have been more dead, fallen, and non-marketable larger and old growth timber in the watershed. This material may have been transported into the river and reservoir since MMD and HAHD were constructed such that there is now less of this material available. A similar decrease in volume and dimension of LWD may occur with time at the proposed facility as existing, dead, fallen, and older trees are delivered to the reservoir and the source of these materials decreases with time.

The volume of LWD available to be transported to the dam site may increase after dam construction relative to the volume of LWD that is currently carried downstream past the dam site. This increase would likely result from the relative ease at which LWD that enters the reservoir can float on the reservoir surface to the dam. Whereas, under current conditions, LWD in various streams that flow into the river and LWD in the river itself may take years to be transported downstream to the dam site.

Infrequent, high volume, extended duration flood events are likely to deliver disproportionately higher volumes of LWD to the reservoir than frequent, lower volume, shorter duration flood events. Unlike the flood control only dam alternative, because there would be a permanent pool behind the dam, it will not likely be necessary to delay lowering the reservoir to manage and collect LWD following a flood event.

Funding should be allocated for labor and equipment to collect, store, and dispose of LWD; for post-flood LWD removal from gates and trash racks and the reservoir; and to repair LWD-related damage to gates and trash racks.

7.5 SEDIMENT MANAGEMENT

Sediment would be delivered to the reservoir area from streams and debris channels that flow into the reservoir. Erosion of reservoir slopes, slumps and shallow landslides occurring on reservoir slopes, and erosion at the toe of existing larger landslides would also deliver sediment to the reservoir area. Sediment could include clays, silt, sand, gravel, cobbles, and boulders. Because there would be a pool behind the dam, the majority of fine grained sediment and the coarse sediment would be deposited in the reservoir. Coarser sediment would likely be deposited as deltas near the location where it enters the pool and as alluvial fans where debris flows enter the reservoir pool. As the reservoir pool is lowered, and flow velocities increase, some of the coarser sediment would be eroded and remobilized and transported toward the dam, where it would be re-deposited in the lower, permanent pool area.

A sediment trap should be excavated within the reservoir, some hundreds of feet upstream of the dam, as part of dam and reservoir construction. This sediment trap should be sized to accommodate the estimated sediment volume associated with some pre-determined number of years of dam operation. A sediment trap was constructed in the HAHD reservoir. The HAHD sediment trap has not yet filled and has not been dredged to remove trapped sediment since HAHD began operations more than 50 years ago.

Excavation and removal of trapped sediment, and re-establishment of the sediment trap, should be planned and budgeted for as part of dam life-cycle operating costs.

Photo 10
Shallow slope failure on HAHD reservoir slopes.

Because sediment will be deposited and trapped within the reservoir, it will likely be necessary to import sand, gravel, cobbles, and boulders to downstream of the dam. Similar to the sediment augmentation program at HAHD, this imported streambed material could be placed along the river, within a mile or so downstream of the dam outlet works, to be eroded and transported downstream by the river. An adaptive management approach would likely be appropriate, wherein the volume and particle size distribution of imported material is adjusted depending on material behavior, downstream deposition and transport, storm frequency, and other factors. When available, sediment captured by and excavated from the sediment trap could be used for this purpose.

7.6 RESERVOIR SLOPE STABILITY – RAPID DRAWDOWN

Reservoir slope stability and existing deep-seated landslides are discussed in a separate report prepared for this project.

The discussion presented in Section 6.6 of this report for the flood control only dam alternative is also relevant to the multi-purpose dam alternative.

7.7 RESERVOIR DRAWDOWN

The reservoir outlet tunnel must have the capacity to lower the reservoir at a sufficient rate to provide storage for the next potential storm. The ability to drawdown the reservoir at a faster rate would not be a primary requirement because upstream and downstream fish passage would be handled primarily by transporting fish over or around the dam. Some provisions could be installed for downstream passage through tunnels or other system that pass through the dam or dam abutments. Operation of these features for downstream fish passage could be modified during when the reservoir is high, thus for the multi-purpose dam alternative there would likely be less need to drawdown the reservoir for fish passage considerations as rapidly as may be desired for the flood control only dam alternative.

Reservoir drawdown rates could be limited by a number of factors. These factors include dam stability, dam abutment slope stability, reservoir slope stability, sediment and LWD management considerations and activities, and downstream flow constraints. Secondary to stability of the dam itself, downstream flow constraints are the primary factors limiting reservoir drawdown rates for existing USACE Seattle and Portland District flood control dams. Downstream flow constraints include life safety considerations; river stage; ability of rivers and levees to contain flows; downstream erosion and sediment deposition; and potential for flows to damage bridges, roads, structures, and other improvements. Sediment deposition, and time and flow-variable changes in sediment deposits in the channel downstream of the dam could reduce river capacity to accept flows.

Rapid reservoir drawdown should not be a concern for stability of a properly founded, designed, and constructed RCC dam, where full or partial excavation of potentially unstable material occurs in conjunction with abutment and foundation stabilization measures. While RCC dams do develop internal pore-pressures during operation, impacts from excess pore pressure within RCC structures are significantly less critical to RCC dam operation than to embankment dam operation. Foundation grout curtains and foundation drains below the dam and in the abutments should be designed to limit pore pressures below the dam to acceptable levels. Rapid reservoir drawdown rate is a concern for rockfill dam stability. A rockfill dam would be constructed with upslope and downslope face inclinations and interior drainage features that provide for stability of the dam during rapid drawdown. The reservoir drawdown rate would be limited so that excess pore pressures can dissipate sufficiently to maintain dam stability. This criterion may be the controlling factor in setting the reservoir drawdown rate.

Without implementation of slope stabilization or drainage measures, rapid reservoir drawdown could contribute to instability of the naturally occurring soil and rock in dam abutments and reservoir slopes. Abutment and reservoir slope Instability, should it occur, could pose a hazard to dam operations. Geologic assessment, subsurface investigations, and geotechnical analyses of abutment and reservoir slopes and existing landslides would be performed as part of project design. Existing landslides and slopes found to be unstable under rapid drawdown conditions could result in the need to place constraints on the reservoir drawdown rate or require stability measures be implemented to improve stability of unstable landslides and slopes. The preferred approach would be to construct stability improvement measures for existing landslides near the dam and within the reservoir that are characterized as potentially posing unacceptable risk to the dam or dam outlet works operations should they mobilize. Stabilization measures may include full or partial removal of unstable material. For the rockfill dam alternative, significant quantities of material would be needed to construct the dam. Some of this material could be excavated from existing landslides and potentially unstable slopes within the reservoir, thereby providing material for dam construction and simultaneously improving stability of reservoir slopes. Order of magnitude costs for design-phase subsurface explorations and to implement reservoir and abutment slope stability improvements during dam construction should be included in the estimate of dam and reservoir design and construction costs.

USACE Portland District reports typical post-flood reservoir drawdown rates of 4 to 6 feet per day. For MMD and HAHD, USACE reports typical post-flood drawdown rates on the order of 10 feet per day, maximum recorded multi-day sustained average drawdown rate of about 13 feet per day at both dams, and maximum recorded short-term (possibly for part of a single day) sustained drawdown rate of 20 feet per day on one occasion for HAHD. MMD and HAHD both are designed and constructed such that they could theoretically discharge water through their outlet tunnels to drawdown the reservoirs on the order of 30 feet per day, although this drawdown rate has not been experienced at either reservoir.

If stability measures are implemented during dam construction for landslides and slopes near the dam and in the reservoir that could pose a hazard to the dam, a typical multi-day sustained drawdown rate of 10 feet per day could be assumed for the current assessment of the dam and dam operations. A faster drawdown rate of 20 feet per day could be assumed for a short period (one to two days) during the drawdown cycle. A maximum drawdown rate of 30 feet per day could be assumed. The maximum drawdown rate would apply to rapid-succession storm scenarios where it is important to re-establish reservoir storage capacity following a storm for dam safety considerations.

The above drawdown rates apply to drawdown of the active flood control pool above the elevation at which the reservoir had been maintained for multiple weeks or months before the latest flood event. Drawdown rates should be slower, about 1 to 5 feet per day, below the elevation at which a pool had been maintained for an extended period of time, i.e., where sustained infiltration from the reservoir pool would raise the groundwater elevation and increase the distance from the reservoir to which the ground is saturated. The slower drawdown rates provide more time for groundwater to flow to the lowered reservoir and for pore pressures in reservoir slopes to decrease.

Slower drawdown rates are preferred from a slope stability perspective, but may not be preferred from an operational needs perspective. The sooner the reservoir is lowered after inundations, the less infiltration of water into the slopes, and thus reduced potential for deep seated landslides to be mobilized by rapid drawdown pore pressures. Drawdown soon after inundation may reduce the risk of saturating deep seated landslides and reduce potential for instability during drawdown. Drawdown rates provided above are based on historical operation of USACE Seattle and Portland District dams. Geologic reconnaissance, subsurface explorations, and engineering analyses should be performed as part of the next phase of the Chehalis Basin Strategy study to confirm the appropriateness of these assumed drawdown rates with regard to abutment and reservoir slope stability, and to develop recommendations for stability improvement measures and assess risk and operation impacts associated with rapid drawdown.

7.8 OPERATION COSTS AND PERSONNEL

There is not sufficient information available to develop project-specific staffing, equipment, and operation requirements for the dam and reservoir being evaluated. In the absence of this information, it is assumed that the dam and reservoir would have staffing and equipment needs similar to HAHD, with consequent similar operating costs. It is also assumed the dam and reservoir would be operated similar to HAHD with regard to water impoundment; water release; and vegetation, LWD, and sediment management. Based on these assumptions, the following staff, equipment, and operating costs may be assumed:

Personnel:

- Equivalent of eight full-time personnel for dam operations and maintenance and administration
- A quarter-time equivalent Dam Safety Program Manager

Equipment:

- Tugboat, work barge, and inspection boat
- Mid-sized excavator
- Operator/maintenance personnel vehicles

Operating costs:

- $3,500,000 per year for staff and equipment
- $300,000 per year for downstream sediment and LWD river augmentation

Reservoir slope stability improvement costs:

- RCC Dam: $200,000 to $10,000,000
- Rockfill Dam: $200,000 to $6,000,000

Appendix F – Operations and Maintenance Cost Tables

Alternative A Flood Retention RCC Dam Annual Operations and Maintenance Costs ($2014)						
Dam and Reservoir						
Cost Category	**Cost Item**	**Cost Basis**	**Value**	**Unit $**	**Annual $**	**Comments/Assumptions**
Vegetation Management*	Part Time Labor	FTE	1.5	$65,000	$97,500	Harvesting potentially submerged trees by others at no cost
Debris Handling*	Part Time Labor	FTE	1	$65,000	$65,000	Reservoir only operated bi-annually
Debris Handling/Disposal*	Loaders/Trucks/Operators	LS	1	$50,000	$50,000	Short haul to disposal site for locals to collect
Fish/Environmental	Monitoring/Reporting	LS	1	$30,000	$30,000	Bi-annual consultant review/report
Operations	Dam Tender/Security	FTE	0.7	$85,000	$59,500	Manage/do routine maintenance/bi-annual operation
Administrative	Management	FTE	0.3	$120,000	$36,000	PT dam admin/staff management
Administrative	Reporting	FTE	0.3	$90,000	$27,000	
Administrative	Legal/Insurance	LS	1	$50,000	$50,000	Placeholder
Maintenance/Repairs	Part Time Labor	FTE	0.5	$80,000	$40,000	Special maintenance/repairs
Inspections	Safety Inspections	LS	1	$9,000	$9,000	Annual fund for 5-year inspection
Mechanical	Repair/Replace Fund	% Cap	0.4%	$21,952,000	$88,000	Mechanical Fund/Intermittent Use
Structural	Repair Fund	% Cap	0.1%	$50,850,000	$51,000	Structural repair fund/Intermittent Use
	Subtotal				**$603,000**	
Fish Passage						
Operations	Operator/Monitor	FTE	0.5	$90,000	$45,000	Part of dam tender FTE
Biological	Monitoring/Reporting	FTE	0.5	$90,000	$45,000	Special monitoring/reporting of bi-annual operation
Maintenance/Repairs	Repair/Replace Fund	LS		$60,000	$60,000	Annual funding
Trap and Haul	Loaders/Trucks	LS	1	$40,000	$40,000	Periodic T & H Expenses
	Subtotal				**$190,000**	
	Total Annual Cost				**$793,000**	

*Typical future annual average (2014 $) – does not include initial clearing and early stage vegetation management for the first few years of operation.

Alternative B Multi-purpose RCC Dam with CHTR and Combination Collector Annual Operations and Maintenance Costs (2014$)						
Dam and Reservoir						
Cost Category	**Cost Item**	**Cost Basis**	**Value**	**Unit $**	**Annual $**	**Comments/Assumptions**
Vegetation Management*	Part Time Labor	FTE	1	$65,000	$65,000	Harvesting potentially submerged trees by others at no cost
Debris Handling*	Part Time Labor	FTE	1.5	$65,000	$97,500	Special clearing/disposal of floating debris
Debris Handling*	Loaders/Trucks/Operators	LS	1.5	$50,000	$75,000	Short haul to disposal site for locals to collect
Fish/Environmental	Monitoring/Reporting	LS	1	$40,000	$40,000	Bi-annual consultant review/report
Operations	Dam Tender/Security	FTE	1.5	$85,000	$127,500	Manage/do routine maintenance
Administrative	Management	FTE	0.5	$120,000	$60,000	PT dam admin/staff management
Administrative	Reporting	FTE	0.3	$100,000	$30,000	
Administrative	Legal/Insurance	LS	1	$150,000	$150,000	Placeholder
Maintenance	Part Time Labor	FTE	1	$80,000	$80,000	Special maintenance/repairs
Inspections	Safety Inspections	LS	1	$10,000	$10,000	Annual fund for 5-year inspections
Mechanical	Repair/Replace Fund	% Cap	0.8%	$12,767,588	$102,000	Mechanical repair/replacement fund
Structural	Repair Fund	% Cap	0.2%	$60,432,000	$121,000	Structural repair fund
	Subtotal				**$958,000**	
Fish Passage						
Operations	Operator/Monitor	FTE	1.5	$90,000	$135,000	Part of dam tender FTE
Biological	Monitoring/Reporting	FTE	1	$90,000	$90,000	Special monitoring/reporting of bi-annual operation
Maintenance	Part Time Labor	FTE	1	$90,000	$90,000	Special maintenance/repairs
Mechanical	Repair/Replace Fund	% Cap	0.8%	$18,000,000	$144,000	Annual funding
Structural	Repair Fund	% Cap	0.2%	$20,000,000	$40,000	Annual funding
Trap and Haul	Loaders/Trucks	LS	1	$82,000	$82,000	T & H Expenses
	Subtotal				**$691,000**	
	Total Annual Cost				**$1,649,000**	

*Typical future annual average (2014 $) – does not include initial clearing and early stage vegetation management for the first few years of operation.

Alternative C Multi-purpose RCC Dam with Experimental Fishway and Floating Forebay Collector Annual Operations and Maintenance Costs (2014$)						
Dam and Reservoir						
Cost Category	**Cost Item**	**Cost Basis**	**Value**	**Unit $**	**Annual $**	**Comments/Assumptions**
Vegetation Management*	Part Time Labor	FTE	1	$65,000	$65,000	Harvesting potentially submerged trees by others at no cost
Debris Handling*	Part Time Labor	FTE	1.5	$65,000	$97,500	Special clearing/disposal of floating debris
Debris Handling*	Loaders/Trucks/Operators	LS	1.5	$50,000	$75,000	Short haul to disposal site for locals to collect
Fish/Environmental	Monitoring/Reporting	LS	1	$40,000	$40,000	Bi-annual consultant review/report
Operations	Dam Tender/Security	FTE	1.5	$85,000	$127,500	Manage/do routine maintenance
Administrative	Management	FTE	0.5	$120,000	$60,000	PT dam admin/staff management
Administrative	Reporting	FTE	0.3	$100,000	$30,000	
Administrative	Legal/Insurance	LS	1	$150,000	$150,000	Placeholder
Maintenance	Part Time Labor	FTE	1	$80,000	$80,000	Special maintenance/repairs
Inspections	Safety Inspections	LS	1	$10,000	$10,000	Annual fund for 5-year inspections
Mechanical	Repair/Replace Fund	% Cap	0.8%	$12,767,588	$102,000	Mechanical repair/replacement fund
Structural	Repair Fund	% Cap	0.2%	$60,432,000	$121,000	Structural repair fund
	Subtotal				**$958,000**	
Fish Passage						
Operations	Operator/Monitor	FTE	1	$90,000	$90,000	Part of dam tender FTE
Biological	Monitoring/Reporting	FTE	1	$90,000	$90,000	Special monitoring/reporting of bi-annual operation
Maintenance	Part Time Labor	FTE	0.5	$90,000	$45,000	Special maintenance/repairs
Mechanical	Repair/Replace Fund	% Cap	0.8%	$18,000,000	$144,000	Annual funding
Structural	Repair Fund	% Cap	0.2%	$20,000,000	$40,000	Annual funding
Fishway and Collector	Operational Costs	LS	1	$24,000	$24,000	Expenses
	Subtotal				**$433,000**	
	Total Annual Cost				**$1,391,000**	

*Typical future annual average (2014 $) – does not include initial clearing and early stage vegetation management for the first few years of operation.

Alternative D Multi-purpose Rockfill Dam with Conventional Fishway and Floating Forebay Collector Annual Operations and Maintenance Costs (2014$)						
Dam and Reservoir						
Cost Category	**Cost Item**	**Cost Basis**	**Value**	**Unit $**	**Annual $**	**Comments/Assumptions**
Vegetation Management	Part Time Labor	FTE	1	$65,000	$65,000	Harvesting potentially submerged trees by others at no cost
Debris Handling	Part Time Labor	FTE	2	$65,000	$130,000	Special clearing/disposal of floating debris
Debris Handling	Loaders/Trucks/Operators	LS	2	$50,000	$100,000	Short haul to disposal site for locals to collect
Fish/Environmental	Monitoring/Reporting	LS	1	$40,000	$40,000	Bi-annual consultant review/report
Operations	Dam Tender	FTE	1.5	$84,000	$126,000	Manage/do routine maintenance
Administrative	Management	FTE	0.5	$120,000	$60,000	PT dam admin/staff management
Administrative	Reporting	FTE	0.3	$100,000	$30,000	
Administrative	Legal/Insurance	LS	1	$200,000	$200,000	Placeholder
Maintenance	Part Time Labor	FTE	1.5	$80,000	$120,000	Special maintenance/repairs
Inspections	Safety Inspections	LS	1	$14,000	$14,000	Annual fund for 5-year inspections
Mechanical	Repair/Replace Fund	% Cap	0.8%	$12,047,750	$96,000	Mechanical repair/replacement fund
Structural	Repair Fund	% Cap	0.2%	$60,921,250	$122,000	Structural repair fund
	Subtotal				**$1,103,000**	
Fish Passage						
Operations	Operators/Monitor	FTE	1	$90,000	$135,000	Part of dam tender FTE
Biological	Monitoring/Reporting	FTE	1	$90,000	$90,000	Special monitoring/reporting of bi-annual operation
Maintenance	Part Time Labor	FTE	0.5	$90,000	$45,000	Special maintenance/repairs
Mechanical	Repair/Replace Fund	% Cap	0.8%	$25,000,000	$200,000	Annual funding
Structural	Repair Fund	% Cap	0.2%	$40,000,000	$80,000	Annual funding
Fishway and Collector	Operational Costs	LS	1	$15000	$15,000	Expenses
	Subtotal				**$520,000**	
	Total Annual Cost				**$1,623,000**	

Appendix G – Fish Passage Assessment and Anticipated Operations

MEMORANDUM

Date: September 9, 2014
To: Water Retention Technical Committee
From: Mike Garello PE, Shaun Bevan EIT, and David Minner EIT, HDR Engineering, Inc.
Cc: Robert Montgomery, Anchor QEA; Jim Kramer, Kramer and Associates, Inc.
Re: Revised Assessment of Fish Passage and Anticipated Operations of Proposed Dam Structure Alternatives

Introduction

The fish passage design team has been tasked with assessing fish passage alternatives relative to the anticipated operations for both the multi-purpose and flood retention only structure alternatives. The purpose of this memorandum is to estimate the duration and timing that fish passage may be inhibited based on the preliminary assessment of fish passage of the potential dam structure alternatives using historical hydrographs routed through each of the concept alternatives. This document briefly describes the dam structure alternatives, proposed dam operations, the methodology and assumptions used in the fish passage assessment, and the results of the fish passage assessment.

Background

The dam design team has identified three dam alternatives that are recommended for further development and consideration:

- Flood retention roller compacted concrete (RCC) dam
- Multi-purpose roller compacted concrete (RCC) dam
- Multi-purpose rockfill dam.

The flood retention and multi-purpose alternatives are briefly described below. A more in-depth description of each alternative can be found in the Dam Study Team's Draft Design Technical Memorandum (HDR 2014a) and the future Dam Design Study, which is expected to be completed Spring of 2014. Since the multi-purpose RCC dam and rockfill dam are functionally the same with regard to operations, only one of the multi-purpose dam structures is referenced generically here.

Summary of Proposed Dam Alternative Structures

A flood retention only dam will provide temporary flood storage and not retain a permanent pool upstream of the dam. The current flood retention dam design has a reservoir storage capacity of 65,000 acre-feet, resulting in an estimated dam height of 232 feet. Under normal flow conditions, the dam will operate where inflow is equal to outflow, releasing water through nine 9-foot by 12-foot tunnels at the base of the dam. The tunnels will be designed to facilitate the range of expected fish passage flows and velocities. During flood flows, the tunnels will be closed. When the tunnels are closed, flood control releases will occur through a 25-foot-diameter tunnel or over the emergency spillway. The tunnels would be reopened once the stored water has been released and the inflows are equal to outflows once again.

A multi-purpose dam and reservoir will have a total storage capacity of 130,000 acre-feet. The reservoir storage will be comprised of a 65,000 acre-foot conservation (permanent) pool and a 65,000 acre-foot flood retention pool. The conservation pool will be used to augment flows during periods of low flow, water storage, and hydropower generation. The flood retention pool will be used to store water during large flood events. The estimated multi-purpose dam height is 292 feet. Elevations of the top conservation and flood retention pools are, respectively, 628 feet and 653 (spillway crest), which are over 200 feet above river channel immediately downstream of the dam. Releases from the permanent pool will be via an outlet at the bottom of the dam, while emergency flood control releases will occur over a 200 foot wide spillway.

Summary of Proposed Dam Operations

The Hydrology and Hydraulics (H&H) team has developed preliminary operating rules for both the flood retention only and multi-purpose dam alternatives, which are briefly summarized below. A more detailed description of anticipated dam operations can be found in the Draft Technical Memorandum titled Preliminary Dam Operations Plan Summary (Anchor QEA 2014a). This memo is expected to be finalized in August of 2014. Additionally, dam operations are anticipated to be influenced by targeted debris management activities which are to take place at each flood retention event. A detailed description of the anticipated debris management activities is presented in the Technical Memorandum titled Reservoir Vegetation and Debris Management, and Related Operational Considerations (Shannon & Wilson, Inc., 2014). An additional summary of the modeling and fish passage delay times anticipated for the flood retention only dam structure are provided in a Draft Memorandum titled Fish Passage Delay Times at Flood Retention Only Dam (Anchor QEA, 2014b).

In general, the flood retention only dam will be operated as follows:

- During normal conditions, the outflow of the reservoir will equal the inflow (natural hydraulic conditions) except during large floods. Reservoir outflow will occur through nine tunnels. Fish passage and sediment control will occur in the tunnels.
- When flows at USGS Gage #12027500 (Chehalis River near Grand Mound) are predicted to be above major flood stage (Flows greater 38,800 cubic feet per second (cfs)), the reservoir outflow will be reduced at a rate of 200 cfs per hour until the outflow is equal to 300 cfs. Reservoir outflow will be reduced by closing the gates on the tunnels and controlling outflow with the 25-foot-diameter flood control tunnel.
- Once the flow at the Grand Mound gage has dropped below major flood stage for 48 hours, reservoir outflow will increase by 1,000 cfs per hour, not to exceed a maximum drawdown of 30 feet per hour. Flood control releases will primarily be released through a 25-foot-diameter flood control tunnel.
- A pool elevation of 456 feet is maintained for an extended duration ranging from days to weeks as the reservoir empties and debris management activities occur.
- Once the reservoir is drawn down to stage 426 feet, the inflow is approximately equal to the outflow and the reservoir will be back to natural hydraulic conditions.

Below is an example flood retention operation (Figure 1). Fish passage will not occur while the gates on the tunnels are closed or while water is stored behind the reservoir for debris management operations.

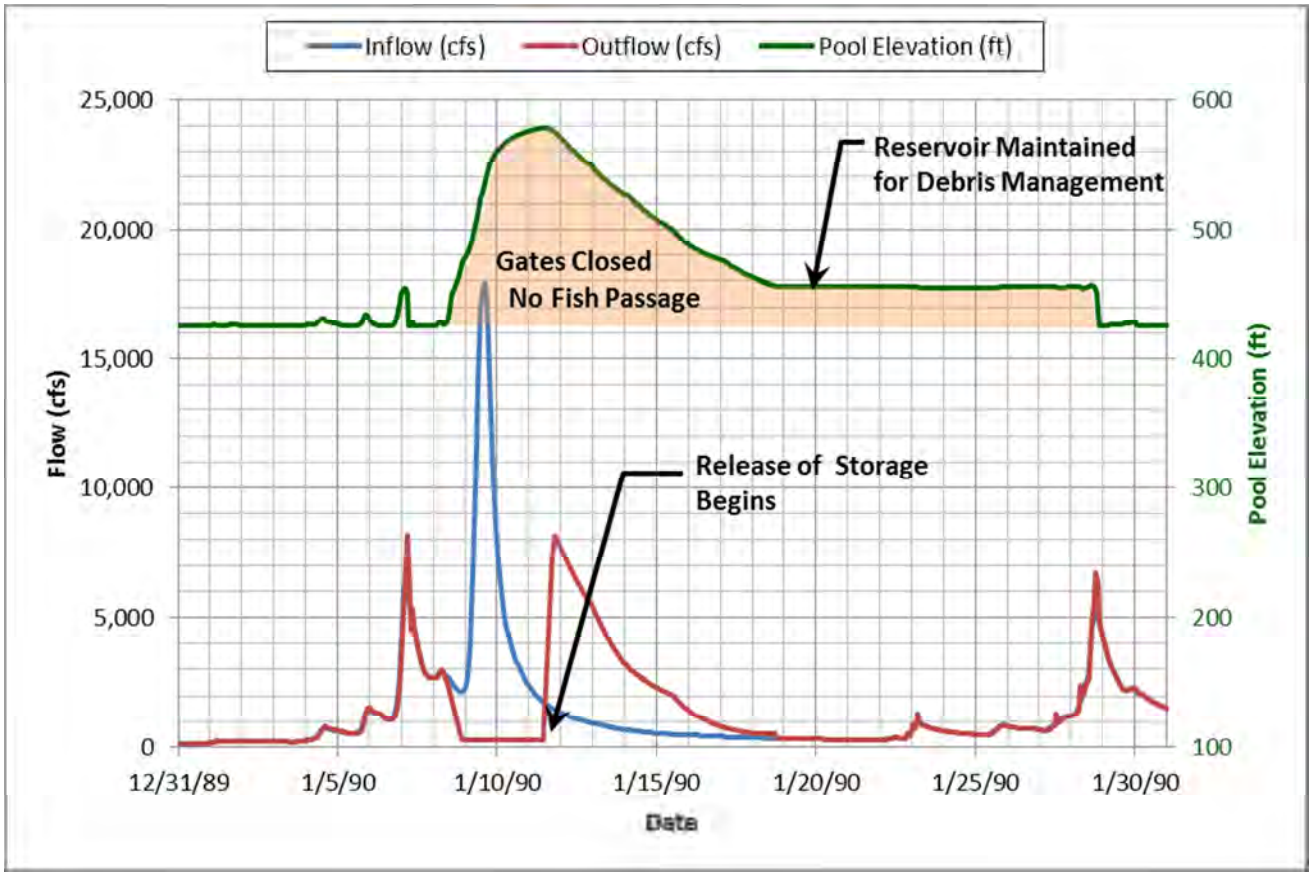


Figure 1. Reservoir inflow, outflow, and stage during 1990 flood event

The multi-purpose dam uses similar operation rules as the flood retention alternative to determine when to store water; however, there are slight operational differences for reservoir release depending on the reservoir storage at the start of flood control operations. During the release of impounded water, flow may be retained to replenish the conservation pool. If the reservoir is in the flood retention pool, releases will be increased by 1,000 cfs per hour up to a maximum of 11,000 cfs. During smaller events (inflows greater than or equal to 2,800 cfs), the flow will be allowed to pass through the multi-purpose reservoir assuming the flow at the Grand Mound gage is below flood stage.

In addition to providing flood control, the multi-purpose alternative will be operated to meet instream flow requirements ranging in magnitude from 160 to 290 cfs during the course of the year. Reservoir releases may be curtailed by 20% during a drought if the conservation pool is not filled. Figure 2 provides an example of reservoir operation.

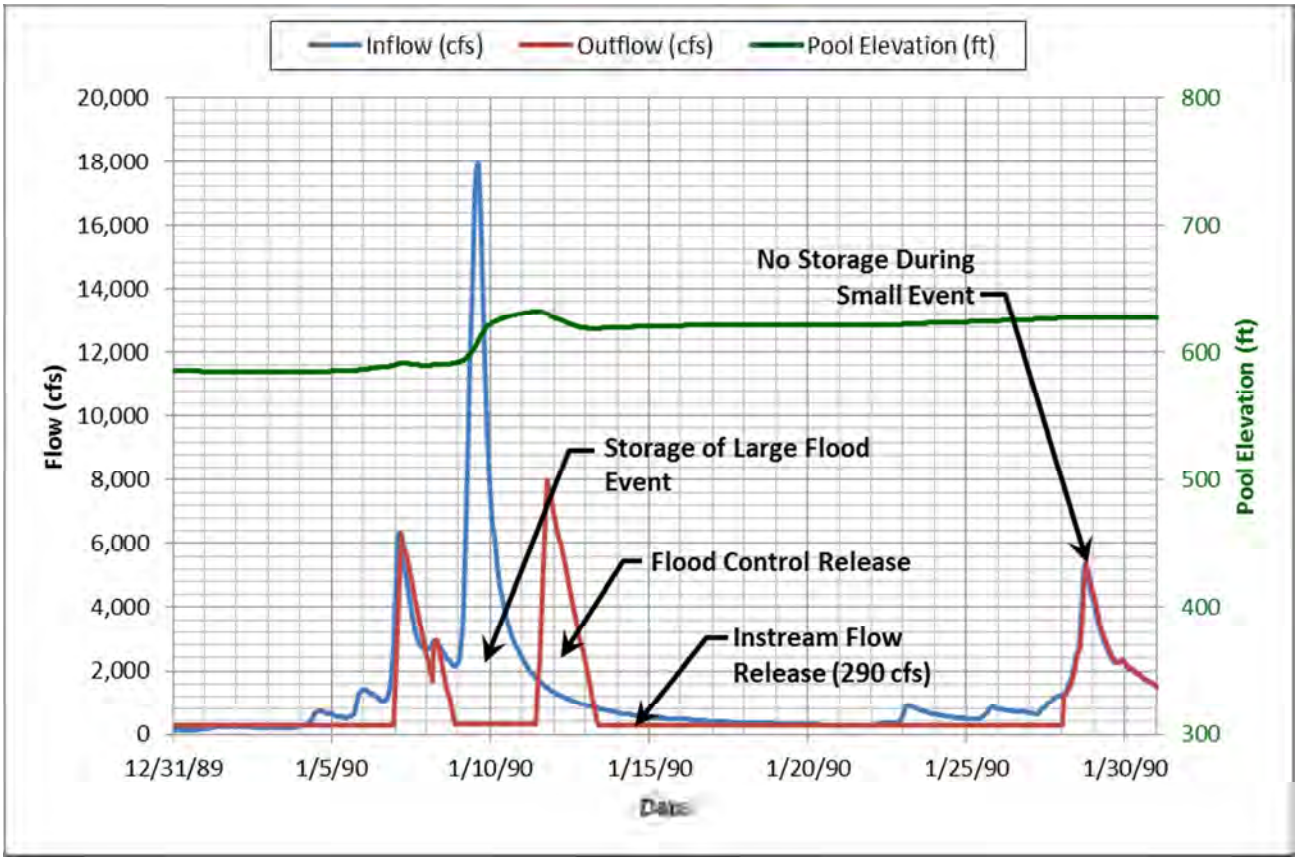


Figure 2. Reservoir inflow, outflow, and stage during 1990 flood event

Methods for Evaluating Fish Passage

Flood Retention Only Dam Structure

In order to assess the flood retention only alternative, a hydraulic model was created by the H&H team that simulated the dam operations using historical hydrographs. The H&H team results provided simulated inflow, outflow, stage, and storage for water years 1989 to 2012 on an hourly timestep. Fish passage was assessed at the flood retention only alternative by determining when flood operations were occurring and when the inflow to the reservoir exceeded the high fish passage flow (2,000 cfs). The results from the H&H analysis provided estimates for the duration and timing in which fish passage gates would be closed due to flood control operations using the historical data (1989 to 2012). The following assumptions were made during the analysis.

- Fish passage does not occur during flood retention operations (gates closed and/or water stored)
- Fish passage will be provided, at a minimum, up to reservoir inflows of 2,000 cfs (the high fish passage design flow).

For this analysis, it was assumed that fish passage would be limited when reservoir inflow exceeded 2,000 cfs (HDR 2013b). The term “limited” in this case refers to a condition where fish passage is likely still occurring but compliance with fish passage design criteria provided by resource agencies begins to diminish. For example, preliminary hydraulic modeling of the fish passage tunnels within the flood retention only alternative indicates velocities of 2 feet per second could be achieved at inflows of 2,000 cfs; however, higher flows were not modeled at this time.

Multi-Purpose Dam Structure

Similar to the flood retention only scenarios, the H&H team modeled the expected multi-purpose dam operations using historical hydrologic data for years 1989 to 2012. The model estimated hourly simulated stage, storage, inflow, and outflow for the multi-purpose alternative for water years 1989 to 2012. The multi-purpose alternative will retain a permanent pool behind the dam, and reservoir releases will be regulated. As a result, it was necessary to analyze reservoir stage as opposed to flow because a tunnel is not a possibility while maintaining a permanent pool. Stage duration curves were generated from the hourly simulated stage values on an annual basis and for each fish species migration period.

Fish Passage Considerations for Operation of Flood Retention Only Dam Structure

Assessment of Annual Fish Passage

Results of the fish passage assessment for the flood retention only scenario indicate that flood retention operations would take place in 6 water years (1990, 1991, 1996, 1997, 2008 and 2009) out of the 24 years included in the analysis. The model results indicate that the fish passage tunnels would only be closed 9 times over the 24 years of simulation. With the exception of two short 13 and 14 hour flood retention events, the fish passage tunnel closures ranged in duration from 10 to 26 days, with an average duration of approximately 18.4 days. As a result of flood retention operations, fish passage would not be provided for 5.4 days per year (1.5%) due to flood control and debris management operations when averaged over the 24 year modeling period.

Additionally, fish passage may be limited on average another 8.9 days a year (2.4%) due to river flows exceeding the high fish passage design flow of 2,000 cfs through the tunnels. High flows above 2,000 cfs ranged in duration from one hour to 4.7 days, with an average duration above 2,000 cfs of 1.3 days. Combined, fish passage is estimated to be inhibited an average of 14.3 days (3.9%) per year for the flood retention only dam alternative. Table 1 below presents the results on an annual basis.

Assessment of Fish Passage by Species

In general, fish passage would be most impacted during the winter months (November to February), when flood retention operations are more likely to occur, and flows are more likely to be above the high fish passage design flow (Table 2). January had the highest average total inhibited passage of 4.4 days, while December was an average of 3.7 days of total inhibited passage. Fish passage is not expected to be impeded by the flood retention only alternative May through September.

Upstream fish migration timing was overlaid on the average monthly inhibited passage duration (Figure 3). All species shown are present during months with expected limited fish passage. Winter steelhead, coastal cutthroat trout, and coho salmon are the most likely to impacted by the flood retention only alternative.

Table 1. Total duration where limited fish passage occurs by water year, WY 1989 -2012

WATER YEAR	FLOOD RETENTION OPERATIONS			HIGH FLOW (ABOVE 2,000 CFS)			TOTAL		
	HOURS	DAY	%	HOURS	DAY	%	HOURS	DAY	%
1989	0	0.0	0.0%	86	3.6	1.0%	86	3.6	1.0%
1990	733	30.5	8.4%	126	5.3	1.4%	859	35.8	9.8%
1991	535	22.3	6.1%	122	5.1	1.4%	657	27.4	7.5%
1992	0	0.0	0.0%	142	5.9	1.6%	142	5.9	1.6%
1993	0	0.0	0.0%	22	0.9	0.3%	22	0.9	0.3%
1994	0	0.0	0.0%	71	3.0	0.8%	71	3.0	0.8%
1995	0	0.0	0.0%	327	13.6	3.7%	327	13.6	3.7%
1996	627	26.1	7.1%	341	14.2	3.9%	968	40.3	11.0%
1997	14	0.6	0.2%	263	11.0	3.0%	277	11.5	3.2%
1998	0	0.0	0.0%	190	7.9	2.2%	190	7.9	2.2%
1999	0	0.0	0.0%	664	27.7	7.6%	664	27.7	7.6%
2000	0	0.0	0.0%	264	11.0	3.0%	264	11.0	3.0%
2001	0	0.0	0.0%	0	0.0	0.0%	0	0.0	0.0%
2002	0	0.0	0.0%	414	17.3	4.7%	414	17.3	4.7%
2003	0	0.0	0.0%	209	8.7	2.4%	209	8.7	2.4%
2004	0	0.0	0.0%	139	5.8	1.6%	139	5.8	1.6%
2005	0	0.0	0.0%	112	4.7	1.3%	112	4.7	1.3%
2006	0	0.0	0.0%	469	19.5	5.4%	469	19.5	5.4%
2007	0	0.0	0.0%	440	18.3	5.0%	440	18.3	5.0%
2008	586	24.4	6.7%	109	4.5	1.2%	695	29.0	7.9%
2009	619	25.8	7.1%	65	2.7	0.7%	684	28.5	7.8%
2010	0	0.0	0.0%	134	5.6	1.5%	134	5.6	1.5%
2011	0	0.0	0.0%	183	7.6	2.1%	183	7.6	2.1%
2012	0	0.0	0.0%	232	9.7	2.6%	232	9.7	2.6%
Total	3114	129.8	1.5%	5124	213.5	2.4%	8238	343.3	3.9%
Average	129.8	5.4	1.5%	213.5	8.9	2.4%	343.3	14.3	3.9%

Table 2. Mean monthly duration of limited fish passage, WY 1989 - 2012

MONTH	MEAN MONTHLY LIMITED PASSAGE DURATION (DAYS)		
	FLOOD RETENTION OPERATIONS	HIGH FLOW (ABOVE 2,000 CFS)	TOTAL
January	1.9	2.5	4.4
February	1.4	1.0	2.4
March	0.1	0.8	0.9
April	0.5	0.1	0.6
May	0.0	0.0	0.0
June	0.0	0.0	0.0
July	0.0	0.0	0.0
August	0.0	0.0	0.0
September	0.0	0.0	0.0
October	0.0	0.2	0.2
November	0.3	1.7	2.1
December	1.2	2.5	3.7
Total	1.9	8.9	*14.3*

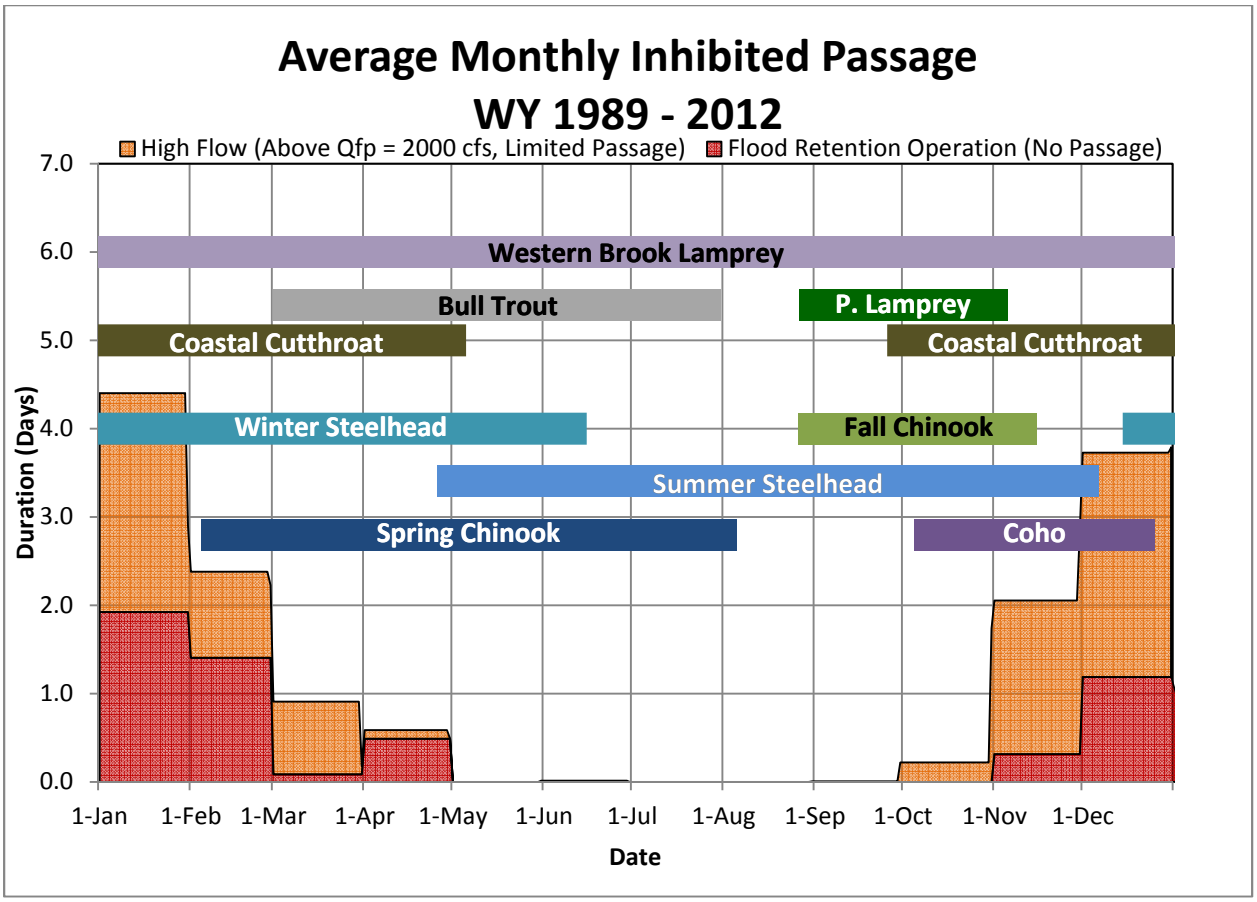


Figure 3. Average monthly duration of limited fish passage with upstream migration periodicity for flood retention only alternative

Fish Passage Considerations for Operation of Multi-Purpose Dam Structure

The model used to evaluate the multi-purpose dam operations simulated reservoir stage for the 24 years of available historical flow data. Simulated reservoir stage values for the multi-purpose alternative range from a minimum of 478.3 feet to a maximum of 671.5 feet (Table 3). The median simulated stage is 610 feet. The flood control pool is utilized less than 1% of the time.

Stage duration curves were created for each period of migration for each fish species (Figure 4). Reservoir levels were lowest during Pacific Lamprey and Fall Chinook migration periods and highest during bull trout, winter steelhead and spring-run Chinook salmon migration periods. Median stages range from about 569 feet (Pacific lamprey) to 627 feet (multiple species). In order to encompass the range of expected medians for all species, fish passage would have to at a minimum provide passage over a range of 58 feet, which is much larger than the assumed 30-foot vertical window that could be provided by a conventional fish ladder or similar facility. During design development the stage duration curves presented in Figure 4 are used to establish the range of forebay elevations in which both upstream and downstream fish passage facilities will need to operate.

Table 3. Annual stage duration anticipated for multi-purpose flood retention dam alternative, WY 1989 - 2012

EXCEEDANCE (%)	STAGE (FT)
0	671.5
0.1	646.5
1	627.1
5	627.0
10	627.0
20	627.0
30	626.0
40	619.1
50	610.0
60	600.8
70	588.4
80	577.7
90	563.3
100	478.3

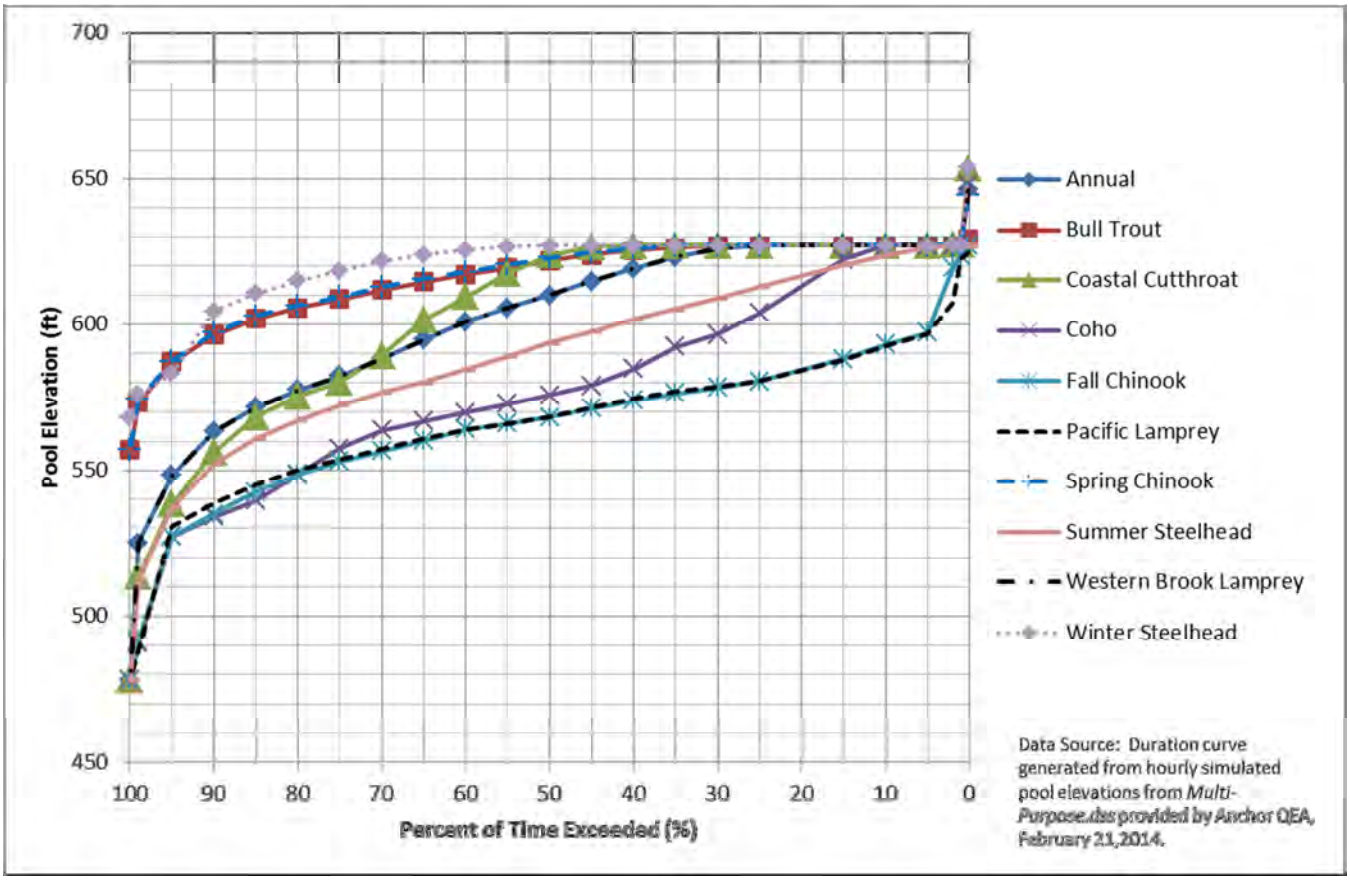


Figure 4. Upstream migration stage-duration curves for multi-purpose alternative, WY 1989 - 2012

Conclusions

The flood retention only alternative is projected to have an average of 5.4 days per year of no passage due to flood retention operations and 8.9 days of inhibited passage associated with high flows for an annual average of 14.3 days of limited passage per year. The majority of the no passage events would occur from November through February. While the average of 5.4 days per year appears minor, it is important to recognize that closures may not occur every year and may have typical closure durations ranging from 10 to 26 days.

The multi-purpose alternative also results show the potential variation in forebay water surface elevations. Over 99% of the time, the reservoir is within the conservation pool; however, the simulated reservoir stages in the conservation pool vary from 478.3 to 628 feet, a difference of about 150 feet. Additionally, the median stage associated with the migration window for each species varies from 569 to 627 feet, a difference of 58 feet, which presented a significant challenge in providing fish passage over the entire range of forebay conditions.

References

Anchor QEA, LLC. 2014a. Draft Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species - Preliminary Dam Operations Plan Summary. February 24, 2014.

Anchor QEA, LLC. 2014b. Draft Memorandum – Fish Passage Delay Times at Flood Retention Only Dam. August 21, 2014.

HDR. 2014a. Draft Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species: Dam Design Technical Memorandum.

HDR. 2014b. Interim Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species: Fish Passage Briefing Report.

Shannon & Wilson, Inc.. 2014. Technical Memorandum – Reservoir Vegetation and Debris Management, and Related Operational Considerations. July 30, 2014.