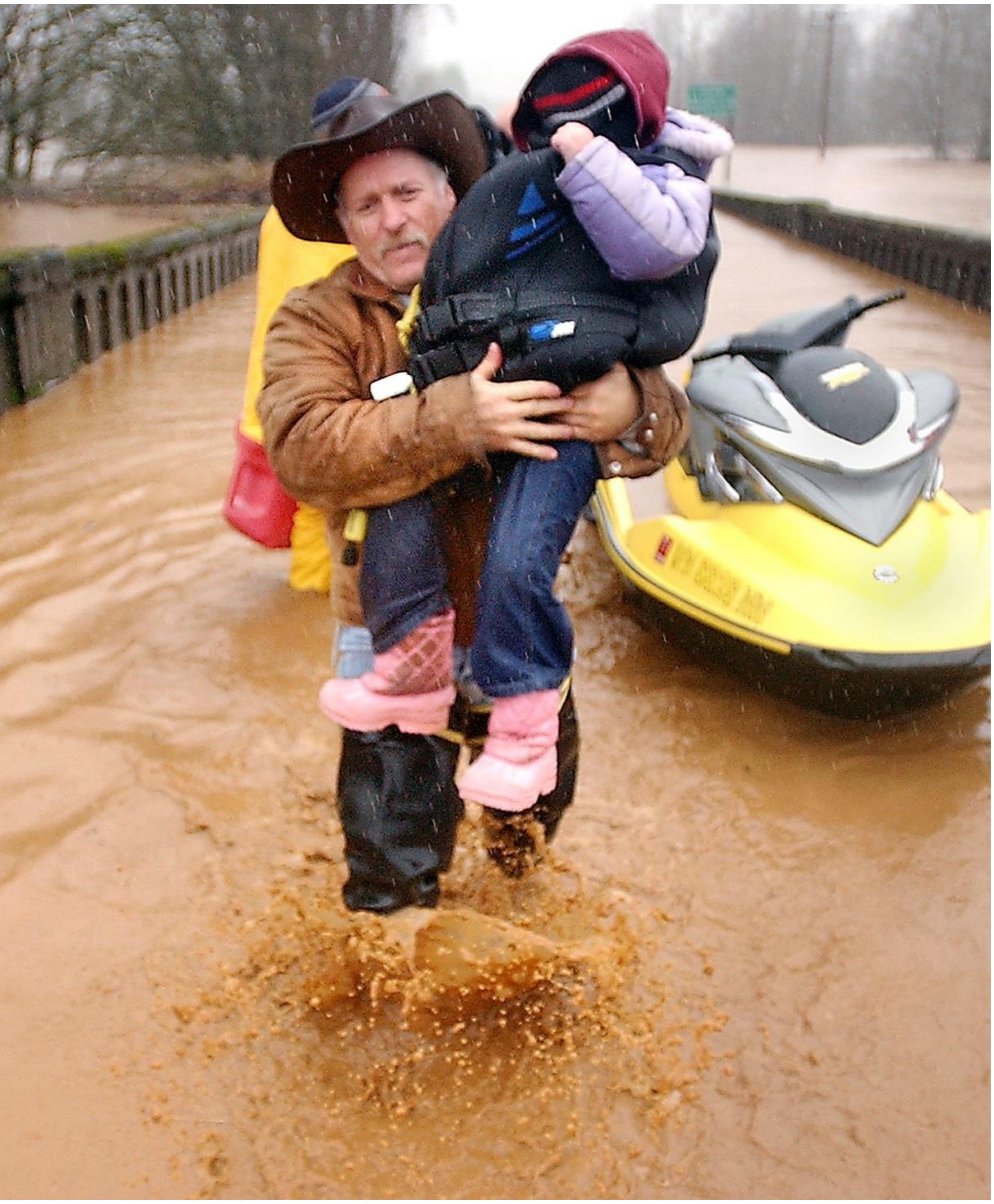
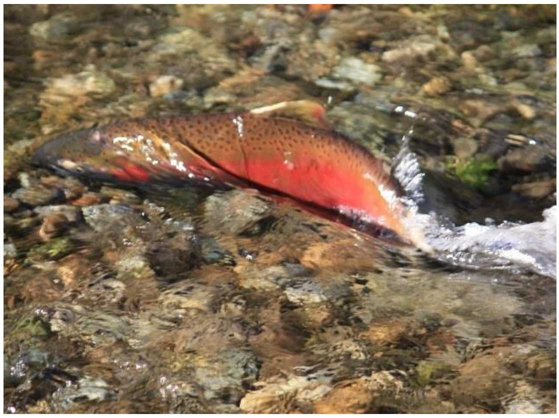
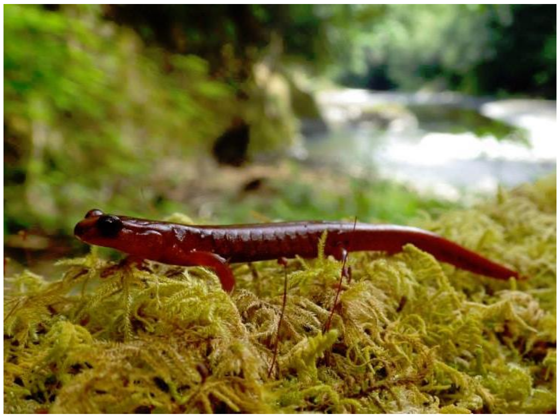
Chehalis Basin Strategy

Phase 2 Site Characterization Technical Memorandum







Reducing Flood Damage and
Restoring Aquatic Species Habitat

June 2017

TABLE OF CONTENTS

LIST OF TABLES

LIST OF PHOTOGRAPHS

LIST OF APPENDICES

ACRONYMS AND ABBREVIATIONS

°	degrees
3-D	three-dimensional
ASR	alkali-silica reactivity
ASTM	American Society for Testing Materials
BH	borehole
CMCE	controlling maximum credible earthquake
CSZ	Cascadia Subduction Zone
D	dip-slip
DB	Dam Boring
DSHA	deterministic seismic hazard analysis
famsl	feet above mean sea level
fps	feet per second
FRO	flood retention only
FRFA	flood retention flow augmentation
FS	Factor of safety
FT	feet
g	acceleration as a percent of gravity
GPS	Global Positioning System
JS	joint set
km	kilometer
LA	Los Angeles
LiDAR	Light Detection and Ranging
LL	liquid limit
LSB	Landslide boring
Lu	Lugeon

m	meter
M	magnitude
MCE	maximum considered earthquake
mm	millimeter
M_W	moment magnitude
NEHRP	National Earthquakes Hazard Reduction Program
OB	Outlet-works boring
PGA	peak ground acceleration
PI	plasticity index
PL	plastic limit
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
Qa	Quaternary alluvium
Qao	Quaternary alluvium older
Qc	Quaternary colluvium
Qls	Quaternary landslide deposit
Qos	Quaternary overburden soil
RCC	roller-compacted concrete
RMR	rock mass rating
RQD	rock quality designation
SPT	Standard Penetration Test
SSD	saturated surface dry
TB	Tunnel boring
Tig	intrusive igneous volcanics
Tcb	Crescent Formation basalt
Tcs	Crescent Formation siltstone
Tml	McIntosh Formation lower
UHS	uniform hazard spectrum

µm	micrometer
USCS	Unified Soil Classification System
USGS	United States Geological Survey
V_{s30}	average shear velocity down to 30 meters
VWP	vibrating wire piezometer
WC	water content
WSDOT	Washington State Department of Transportation

1 EXECUTIVE SUMMARY

The Phase 1 Site Characterization performed in 2015 provided valuable insight into the subsurface conditions at the proposed Chehalis Dam site through six boreholes, five seismic refraction survey lines and a laboratory testing program of dam site soils and rock in addition to preliminary aggregate source testing. Both dam vicinity and reservoir landslides identified in a previous desktop study were investigated. A comprehensive seismic analysis was performed and a preliminary geologic model was developed. The Phase 1 program identified additional data needs for further evaluation to advance the design to a full conceptual level through a Phase 2 Site Characterization program. The Phase 2 program was performed in 2016 and is the subject of this report.

The Phase 2 Site Characterization program has provided the additional geologic and geotechnical engineering information needed to complete conceptual designs and inform the cost estimates of the potential Chehalis Dam configurations. The Phase 2 Site Characterization program included the following tasks:

- Drilling 18 boreholes, including two angled boreholes, to refine the geologic model developed in Phase 1, provide samples for laboratory testing and geotechnical data such as; strength, rock mass rating (RMR) classification, weathering characteristics, and bedrock structure data.
- Water pressure testing was performed to evaluate the hydraulic conductivity of the subsurface in order to advance the grout curtain depth evaluation.
- Acoustic and Optical downhole televiewers were utilized in most of the boreholes to provide detailed joint structure data and fill in the gaps where drilling core loss occurred.
- Sonic Suspension logging was performed in most boreholes to identify weak zones and correlate the borehole data with the surface geophysical survey.
- Inclinometers were installed in four landslides that are in the immediate vicinity of the dam to monitor movement over time
- Piezometers were installed in most of the boreholes and data will be collected periodically to characterize the ground water regime.
- Laboratory testing on soil and rock to assist in the site characterization and develop material properties to be used as model parameters for structural analysis.
- An aggregate evaluation was advanced which included reconnaissance of several quarry options in the dam vicinity and based on visual evaluation of the outcrop quality, selecting three sites for further investigation. The Phase 2 program included boreholes to collect samples and assess the depth of quality rock, geophysical survey to assess the amount of overburden waste, and a laboratory lab program to assess the rock quality.

- A Landslide field evaluation was advanced to further characterize both dam site and reservoir landslides to provide insights on the requirements of removal, mitigation or stabilization.
- A landslide stability improvement evaluation was performed based on field investigations and analysis
- Both a 2-dimensioanl (2D) model utilizing geologic maps and sections and a 3-dimensional (3D) model utilizing visualization software was advanced based on data collected during the investigation.
- A kinematic analysis was performed on data collected during both Phase 1 and 2 to evaluate the potential for sliding surfaces beneath the dam and during temporary excavation cuts during construction.
- A full 3D excavation surface, based on a multi-attribute excavation objective model was developed effectively utilizing all site characterization data gathered to date. This excavation surface is a major component of the updated construction costs provided in the Conceptual Design Report.

These results continue to confirm the feasibility of a roller-compacted concrete (RCC) dam type for the dam site. Consequently, considering cost and other technical factors, the RCC dam type remains preferred for either the flood retention only (FRO) or flood retention flow augmentation (FRFA) project configurations.

The work performed for the combined Phase 1 and 2 site characterization program has expanded the understanding of foundation conditions for the FRO or FRFA dam and reservoir configurations. Three alternative quarry sites suitable to generate RCC and conventional concrete aggregate have been confirmed in reasonably close proximity to the site. The presence of multiple suitable sources will allow for efficient unit prices from a competitive bidding process.

Results of the combined Phase 1 and 2 Site Characterization programs continue to demonstrate that the site is complex from the standpoint of the design of a large and high-hazard RCC dam and the associated hydraulic structures (spillway, fish passage and flood control outlet works, water supply outlet works and construction diversion abutment tunnel).

The combined Phase 1 and 2 site characterization information has confirmed that the required excavation for an RCC dam would likely be similar to the amount of excavation estimated in the previous design report. The configuration of the saddle (embankment) dam has been updated to a composite earthfill/rockfill configuration to address issues and concerns related to foundation bedrock materials obtained as part of the Phase 2 program. Overall, the estimated dam construction costs (presented in the Conceptual Design Report) are consistent with the estimated cost ranges presented in the *Combined Dam and Fish Passage Alternatives Technical Memorandum* (HDR, 2014).

The rock structure information gathered during the combined Phase 1 and 2 site characterization work indicate joint and fracture orientations in the bedrock that produce a high risk of potential for sliding wedges and block surfaces along temporary and permanent excavated slopes.

Additional highly fractured zones in the bedrock were identified in the Phase 2 program. These zones will act as preferential seepage pathways beneath the dam foundation and abutment. The Phase 1 and 2 borehole data have confirmed that foundation grouting to seal these zones and reduce seepage will be an important component of the dam design.

Engineering properties of the foundation bedrock have been developed for input to structural models of the non-overflow and spillway overflow section of the dam and seismic response analyses. The results of the structural seismic response analyses are presented in a separate technical memorandum (TM). Based on structural analyses, the cross-sectional properties of the dam have been confirmed. The dam cross-sections have been utilized in development of the conceptual design level foundation excavation objectives for both the FRO and FRFA dam configurations previously described. The configuration of the hydraulic structures including the emergency spillway, fish passage conduits, flood control outlet works, construction diversion tunnel and the water supply outlet works have been updated based on the Phase 2 site characterization, development of the foundation excavation objectives, and concept level hydraulic analyses.

The inventory of landslides confirmed 27 landslides exist in the reservoir basin and at the dam site. Remediation/stabilization of some of these landslides in close proximity of the dam represents an important design and cost consideration.

- The Phase 2 Site Characterization program has provided valuable insight and advanced the engineering evaluations necessary for a large high hazard dam. The combined site characterization and engineering evaluations to date have provided a basis to identify further investigations needed to complete preliminary design, and to provide cost estimates that are suitable for establishing project funding. The following items should be considered as part of the preliminary design phase during the 2017 to 2018 biennium.
- Further evaluation of the three local potential aggregate sources including additional boreholes at identified quarry sites, a potential test quarry and feasibility level mix design studies.
- Additional boreholes and geophysical surveys to advance the dam site geologic model, fill in the data gaps and to finalize the foundation excavation limits and treatment requirements. Angled borings should be considered to advance the requirement for foundation excavation and evaluate the potential risks of slope failure during construction, particularly in the steep upper left and right abutment areas. Drilling methods that utilize helicopter access may be required in these areas.
- Additional water pressure testing in boreholes to fill in data gaps and refine the grout curtain design.

- Additional evaluation of the landslide hazards in the vicinity of the dam. These evaluations should include advancing seepage and stability analysis to support the development of remedial designs to stabilize these features for both construction and for long-term operation of the reservoir.
- Additional investigation and evaluation of the engineering characteristics of the foundation bedrock and landslide deposits along the diversion tunnel alignment. The additional investigation should consider both inclined and vertical boreholes that intersect the tunnel alignment to evaluate the localized rock properties and the need for temporary and permanent ground control improvement.
- Further evaluation of the saddle dam embankment including additional boreholes along representative cross-sections in order to characterize the extents of a large weak claystone/siltstone rock zone identified in Phase 2. Evaluations would include seepage and stability analyses of the saddle dam embankment to update the foundation treatment (including depth of cutoff) requirements for the saddle dam required for the FRFA configuration.

2 INTRODUCTION

2.1 Project Description

The Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project (Project) is a feasibility study of the benefits and effects of alternatives for flood reduction along the Chehalis River and a basin-wide assessment of enhancement opportunities for aquatic species. Flood damage reduction studies performed for the Project include studies of alternative water retention structures (dams) on tributary streams to the Chehalis River, options for protecting Interstate 5 with or without a tributary dam, and other small flood reduction projects throughout the Chehalis Basin (basin) with or without a dam. The location of the Chehalis River potential dam site is shown on Figure 2.1; Chehalis Dam Vicinity Map.

As part of this Project, HDR, with support from other Anchor QEA team members, prepared a report entitled Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR 2014). That report built upon earlier planning-level studies and included four alternative dam and fish passage configurations involving engineering solutions for a structure that would meet project needs and provide the desired benefits with the fewest adverse impacts. Alternative approaches to engineered water retention structures were evaluated with respect to water retention effectiveness and environmental sustainability to determine the appropriate structure: one that minimizes environmental impacts and provides the desired level of flood damage reduction.

The Combined Dam and Fish Passage Alternatives Technical Memorandum summarized key design considerations and provided recommendations to further advance concepts for flood retention, possible dam configurations for the alternative that would store water for low-flow augmentation, and development of fish passage. The objective was to move toward a preliminary design that would be used for environmental compliance documentation and support project funding and schedule development. The 2014 report did not find any site conditions that would preclude construction of a flood retention only (FRO) or a flood retention flow augmentation (FRFA) dam at the potential Chehalis River site and concluded that the identified site is suitable for either a roller-compacted concrete (RCC) or a composite earthfill/rockfill dam configuration. The RCC dam would have a smaller footprint and may be more advantageous for fish passage, spillway and outlet works configurations and lower total costs, while a rockfill dam may have seismic advantages. The 2014 report included recommendations that several site characterization activities be conducted to allow for further supporting design development, selecting the preferred dam type, and updating estimated costs.

Subsequent to the 2014 technical memorandum, the Anchor QEA team was asked to perform site investigations and geotechnical engineering analyses to move toward completed concept level designs and cost estimates of both the FRO and FRFA dam configurations. In addition, these investigations were targeted at verification of alternative sources of aggregate that can be used for both RCC and conventional concrete construction. An initial site investigation was performed in early 2015 and a Phase 1 Site Characterization Technical Memorandum (TM) (HDR AND S&W INC, 2015) presenting the results was submitted in August of 2015.

The work performed as part of the Phase 1 program included the following:

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

The work was performed by HDR and Shannon & Wilson, Inc., as sub-consultants to Anchor QEA.

A Phase 2 site characterization was performed in early 2016, the results of which are presented in this TM. The work performed for the Phase 2 Site Characterization included the following:

6. Additional borehole and geophysical investigations of foundation conditions within the dam footprint, and along potential dam structures such as outlet works and construction diversion tunnels
7. Advancement of the evaluation of the dam foundation investigation data for updating the conceptual-level design configuration of the dam, outlet works, and spillway structures,
8. Additional borehole and geophysical investigations of landslides in the dam abutment areas and within the reservoir, and
9. Additional boreholes, geophysical investigations, and laboratory testing of materials from three potential aggregate sources (quarries) that could be developed and used to construct an RCC dam. Boreholes, seismic refraction tomography survey lines, and laboratory testing were completed at 2 potential quarry sites. The third potential quarry site is an inactive quarry were representative samples were available and obtained for lab testing.

2.2 Background

In February 2009, the Chehalis Basin Flood Authority authorized the Lewis County Public Utility District No. 1 to contract with EES Consulting, Inc., and subconsultant Shannon & Wilson, Inc., to perform a preliminary evaluation of the geologic conditions of two potential dam sites in western Lewis County, Washington (Shannon & Wilson, Inc. 2009). One of these sites was on the Chehalis River about 2 miles south of the town of Pe Ell. This dam site is the current potential dam study area. The scope of the 2009 evaluation included a literature review, field reconnaissance and geologic mapping, and seismic refraction survey.

Subsequently, a regional preliminary geologic map that includes Lewis County was updated by Wells and Sawlan in 2014 for the United States Geological Survey (USGS) (Wells and Sawlan 2014). These two geologic studies form the basis of the background information for both the Phase 1 and 2 Site Characterization programs. The current potential dam site under investigation is about 2 miles south of Pe Ell in Lewis County, Washington, is slightly downstream of the site explored in the 2009 Shannon & Wilson, Inc., *Geologic Reconnaissance Study,* and is shown on the Figure 2.1 vicinity map. The dam site is on land owned by the Weyerhaeuser Company at a constriction in the valley known as Charlie's Hump. The elevation in the vicinity ranges from about 420 feet above mean sea level (famsl) at the river bed to a topographic high of over 940 famsl in the upper left abutment and a topographic high of 770 famsl in the upper right abutment. For purposes of the current site characterization work, the crest of the largest potential dam (FRFA with a total storage capacity of about 130,000 acre-feet at the spillway crest) is at an elevation of about 714 (feet above mean sea level, famsl). The potential alignment of the main RCC dam is approximately shown on Figure 2.1 and is oriented N47E. An embankment saddle dam is required across the topographic low point in the upper right abutment with an orientation of approximately S47E and perpendicular to the main RCC dam. Water will be stored against the embankment dam when the reservoir storage level is in excess of about elevation 670 to 680 famsl. The total crest length of this largest (FRFA) alternative dam is about 2,400 feet. The left abutment has about a 43 percent slope or 2.3H:1V ratio, whereas the lower right abutment is much steeper with about an 83 percent slope or 1.2H:1V ratio.

2.3 Dam Types

Four dam types and configuration alternatives were determined to be technically feasible as presented in the 2014 Combined Dam and Fish Passage Alternatives Technical Memorandum. These dam types and configurations include:

1. FRFA RCC dam
2. FRFA rockfill dam
3. FRO RCC dam
4. FRO rockfill dam

Due to estimated construction and total costs, as well as other technical considerations it was recommended that the RCC dam type options for both the FRFA and FRO configurations be advanced as part of updated conceptual designs.

2.4 Purpose and Objectives

This Phase 2 Site Characterization program has been performed with the following objectives:

1. Advance the design to include an updated concept level foundation excavation objective, foundation treatment requirements (grouting and drainage), and identify and address potential requirements to achieve excavation and permanent dam and slope stability.
2. Identify and evaluate the potential for development of on-site (or nearby) quarries to provide aggregate sources for use in the production of conventional and RCC materials.
3. Further evaluate landslide hazards in the reservoir basin and at the dam site.
4. Update dam related capital and total costs to support selection of a preferred project configuration for preliminary design.

This Phase 2 Site Characterization TM (Phase 2 TM) presents results from both the Phase 1 and Phase 2 Site Characterization field work. The information in this Phase 2 TM will form the basis for identifying additional site characterization work needed at the site, and for further developing more advanced preliminary designs that will be carried forward to the Alternatives Comparison phase.

2.5 Scope of Work

The scope of work for the Chehalis Dam Phase 2 Site Characterization program included the tasks discussed below. The site investigation plan is shown on Figure 2.2, Chehalis Dam Site Investigation Overview, and Figure 2.3, Site Investigation Program Layout.

2.5.1 Geotechnical Investigation

The Phase 2 geotechnical investigation consisted of subsurface borehole drilling, water pressure testing of the boreholes, and installation of select instrumentation in each borehole. A detailed Phase 2 Chehalis Dam Geotechnical Data Report (Data Report) has been prepared by Shannon and Wilson Inc. and is included as Appendix A.

2.5.1.1 Drilling

The Phase 2 borehole drilling program included eighteen boreholes ranging in depth from 50 to 240 feet. Borehole locations are shown on Figure 2.3, with the exception of the quarry boreholes. A summary of the combined Phase 1 and Phase 2 boreholes including depth, purpose and instrumentation installation is provided in Table 2.1.

Four boreholes, designated as Dam Borings (DB) DB-1, DB-5, DB-6 and DB-7, were drilled along either the RCC dam or embankment dam alignment for the purpose of characterizing the foundation geology, and setting a concept level excavation objective under the footprint of the dam and hydraulic structures. Three boreholes, designated as Outlet works Borings (OB) (OB-1, OB-2 and OB-3), were drilled in a sequence perpendicular to the dam alignment along the potential water quality outlet works. Five boreholes, designated as Tunnel Borings (TB) (TB-1, TB-2, TB-3, TB-4 and TB-5), were also drilled in a sequence perpendicular to the dam alignment along the potential flood control outlet tunnel/ construction diversion tunnel. The purpose of both OB and TB borings was to advance the general understanding of the subsurface geology and to characterize the rock through which the tunnels will be constructed. Four boreholes, designated Landslide Boring (LSB) (LSB-1, LSB-2, LSB-3 and LSB-4) were drilled through landslide and foundation bedrock materials in the vicinity of the left abutment and in the reservoir immediately up-stream of the dam site. The purpose of these boreholes was to assist in characterization of the landslide depth and limits, to support analyses of the landslide and to design landslide treatments that may be needed to achieve a desired level of stability during construction and normal reservoir operations. All boreholes were used to characterize the sub-surface conditions, assist with estimating excavation and foundation requirements within and near the dam footprint, and where applicable to correlate borehole results with geophysical testing results.

Two boreholes designated Quarry Borings (QB) QB-1 and QB-2, were drilled in two of the three primary quarry sites (Quarries 1 and 2, shown on Figure 2.2) identified as having suitable quality rock material to be used for production of aggregate for use in conventional concrete and RCC materials as well as for riprap and roadway surfacing. The third primary quarry site (Rock Creek quarry) is an old quarry site where the rock face is exposed for sample collection. Therefore a boring was not required to gather samples and assess the rock quality.

2.5.1.2 Water Pressure Testing

Once coring was completed, water pressure testing of the bedrock was performed to estimate hydraulic conductivity and treatability of the foundation rock by grouting. Water pressure testing was performed using a stepped Houlsby type testing method. As part of this method and when possible, the test pressures were increased for three 10-minute stages and then decreased to the initial testing pressure. Tests were performed over 10-foot intervals in the boreholes using a double packer assembly for the entire depth of bedrock penetration.

Rock core was logged according to International Society of Rock Mechanics and/or U.S. Bureau of Reclamation standards and photographed in the field. Rock core was stored in labeled wooden core boxes and transported to a warehouse for final logging and selection of specimens for laboratory testing.

2.5.1.3 Instrumentation

Instrumentation was installed in select boreholes after water pressure testing and any down-hole geophysics were completed. Instrumentation included vibrating wire piezometers and inclinometers. Fully grouted vibrating wire piezometers (VWP) were installed in fifteen boreholes, excluding LSB-3, QB-1 and QB-2. The model used was a Geokon standard VWP model 4500S and 4500 SH for the deeper holes with a Geokon LC-2 data logger to record continuous measurements of the groundwater depth. Four inclinometers were installed in the LSB boreholes to monitor movement of the hillside over time.

The inclinometer used was a Slope Indicator Digitilt AT System. After inclinometer installation initial readings were taken by Shannon and Wilson field personnel and are presented in the Data Report included as Appendix A of this TM.

Table 2.1
Summary of Phase 1 and 2 Borehole Explorations

PHASE	BOREHOLE NUMBER	DEPTH (FEET)	PURPOSE	INSTRUMENTATION		GEOPHYSICS	
				VWP1 DEPTH (FT)	INCLINO-METER	TELEVIEWER OT: OPTICAL AT: ACOUSTIC	SUSPENSION LOGGING
1	BH-1	140.4	Excavation objective, foundation treatment, and landslide characterization	136.8	NO	AT	Yes
	BH-2	241	Maximum section, excavation objective and foundation treatment	231.8	NO	OT	Yes
	BH-3	150	Upstream landslide characterization	143	NO	AT	Yes
	BH-4	120	Upstream landslide characterization	111.6	NO	AT	Yes
	BH-5	250	Excavation objective and foundation treatment	238	NO	OT/AT	Yes
	BH-6	320	Excavation objective and foundation treatment	342.2	NO	OT/AT	Yes
2	DB-1	240	RCC Dam excavation objective, foundation treatment, and investigate weak rock zone revealed by phase 1 geophysics	209	NO	AT	Yes
	DB-5	200	Embankment Dam excavation objective and foundation treatment	191	NO	OT	Yes
	DB-6	200	Embankment Dam excavation objective and foundation treatment	189.6	NO	OT	Yes
	DB-7	200	Embankment Dam excavation objective and foundation treatment	192	NO	AT	Yes
	OB-1	100	FRFA outlet works tunnel and intake area	97.8	NO	AT	Yes

PHASE	BOREHOLE NUMBER	DEPTH (FEET)	PURPOSE	INSTRUMENTATION		GEOPHYSICS	
				VWP1 DEPTH (FT)	INCLINO-METER	TELEVIEWER OT: OPTICAL AT: ACOUSTIC	SUSPENSION LOGGING
	OB-2	240	RCC Dam excavation objective, foundation treatment, FRFA outlet works tunnel and investigate weak rock zone revealed by phase 1 geophysics	234.1	NO	AT	Yes
	OB-3	100	FRFA outlet works tunnel and outlet area	97.2	NO	AT	Yes
	TB-1	100	FRO outlet works/construction diversion tunnel and intake area	97	NO	AT	Yes
	TB-2	200	RCC Dam excavation objective, foundation treatment, and FRO outlet works/construction diversion tunnel	197	NO	AT	No
	TB-3	200	FRO outlet works/construction diversion tunnel	197	NO	OT	No
	TB-4	100	FRO outlet works/construction diversion tunnel	98	NO	AT/OT	Yes
	TB-5	100	FRO outlet works/construction diversion tunnel and outlet area	99	NO	AT	Yes
	LSB-1	105	Investigate Landslide 1	50.2	YES	NA	No
	LSB-2	110	Investigate Landslide 1a	31.5	YES	NA	No
	LSB-3	100	Investigate Landslide 3	NA	YES	NA	No
	LSB-4	50	Investigate Landslide 3a	28.4	YES	NA	No
	QB-1	150	Investigate potential quarry site	NA	NO	NA	No
	QB-2	150	Investigate potential quarry site	NA	NO	NA	No
Vibrating Wire piezometer							

2.5.2 Geophysical Surveys

Seismic refraction tomography surveys were performed along fifteen transects as shown on Figures 2.2 and 2.3. Three transects designated Dam Seismic Lines (DSL) were run along potential dam structures such as the outlet works tunnels and stilling basin. Ten transects designated Landslide Seismic Lines (LSSL) were run along previously identified landslides both in the immediate vicinity of the dam and within the potential reservoir. Lastly, 2 transects designated Quarry Seismic Lines (QSL) were run in locations identified as primary potential quarry sites 1 and 2. The purpose of seismic refraction testing along each Seismic Line is to estimate:

1. The depth to the top of bedrock,
2. The depth of the top of bedrock suitable for supporting the RCC dam cross-section,
3. The thickness of alluvium, colluvium, and landslide materials, and
4. The amount of potential overburden/waste material at quarry locations.

Downhole geophysical testing (suspension logging) was performed in ten boreholes. The purpose of the down-hole geophysical tests is to measure compressional and shear wave (P and S) velocity of the rock penetrated by the boreholes, allowing comparison to and calibration of the surface geophysical refraction profiles.

Downhole optical/acoustic televiewer logging was completed in eleven boreholes to generate a continuous oriented 360° image of the borehole wall to support core logging activities, and to measure and record the location and orientation of joints and discontinuities along the boreholes. The televiewer is an excellent tool to identify and describe shear zones, zones of core loss, and lithological contacts that are subject to disturbance or destruction by the core drilling process.

The geophysical work was performed by Global Geophysics, Inc., of Monroe, Washington, under subcontract with Anchor QEA and with oversight from Shannon & Wilson, Inc., and HDR.

2.5.3 Laboratory Testing

The Phase 2 investigation laboratory testing included test on the foundation bedrock, overburden soil encountered in the boreholes and aggregate samples collected either from boreholes or the face of a potential quarry site.

2.5.3.1 Overburden Soil

Soil testing on the overburden in the boreholes was selectively tested by Shannon & Wilson Inc. to evaluate the overburden properties and accurately classify the soils found at the site. A summary of the soil testing is presented in Table 2.2. Discussion of the results of the overburden soil laboratory testing is presented in Section 4.2.1 below.

Table 2.2
Overburden Soil Laboratory Testing

LAB	TEST METHOD NAME	TEST DESIGNATION	NUMBER OF TESTS
Shannon & Wilson Inc. Seattle, Washington	Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates	ASTM C 136	13
	Standard Test Method for Particle-Size Analysis of Soils	ASTM D 422-63(2007)e2	16
	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	ASTM D 4318-10e1	24

2.5.3.2 Foundation Bedrock

To further advance the characterization of the bedrock foundation at the dam site and to evaluate modeling material property inputs a suite of testing was performed on core samples throughout the dam site area. A summary of the bedrock testing laboratories, test methods, and number of tests performed is presented in Table 2.3. Discussion of the results of the foundation bedrock laboratory testing is presented in Section 4.2.2 below.

Table 2.3
Foundation Bedrock Laboratory Testing

LAB	TEST METHOD NAME	TEST DESIGNATION	NUMBER OF TESTS
GeoTesting Express Acton, Massachusetts	Compressive strength and elastic moduli of intact rock core specimens	ASTM D 7012 Method C	18
	Slake Durability of Shales and Similar Weak Rocks	ASTM D 4644	8
	Unit Weight, Porosity and Specific Gravity of Rock	ISRM Method 2	5
	Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force	ASTM D 5607	9
Shannon & Wilson Inc. Seattle, Washington	Point Load Test	ASTM D 5731	14
American Engineering Testing St. Paul, Minnesota	Petrographic examination of aggregates for concrete	ASTM C 295	6

2.5.3.3 RCC Aggregate

Results of previous evaluations of alternative RCC aggregate materials were expanded as part of the Phase 2 Site Characterization program. Several potential aggregate quarry locations were considered; however based on visual classification of the rock quality three primary sites were selected for further evaluation as part of the Phase 2 scope of work. Locations of all three quarry sites are presented on Figure 2.2. Quarry 3 is the old Weyerhaeuser Rock Creek quarry considered in Phase 1 to have suitable test results. Preliminary quarry layouts were prepared of Quarry 1 and 2 in order to establish the best location and depth for borings QB-1 and QB-2 at each site.

Once the borings QB-1 (Quarry 1) and QB-2 (Quarry 2) were completed, boring logs were carefully reviewed and a total of six sample locations were identified for testing. Core samples were selected from QB-1 at 3 depth intervals, and QB-2 at 2 depth intervals. A grab sample from the face of the Rock Creek quarry was the final sample for the laboratory test program. A summary of the RCC aggregate testing and the laboratory that performed the tests is presented in Table 2.4.

Table 2.4
RCC Aggregate Laboratories, Testing Methods, and Number of Tests

LAB	TEST METHOD NAME	ASTM TEST DESIGNATION	NUMBER OF TESTS
Lafarge North America Seattle, Washington	Standard test method for bulk density (unit weight) and voids in aggregates	C 29	6
	Test method for resistance to degradation by abrasion and impact in the LA machine	C 535	6
	Test method for potential alkali reactivity of aggregates (mortar bar method)	C 1260	6
American Engineering Testing St. Paul, Minnesota	Petrographic examination of aggregates for concrete	C 295	4

2.5.4 Landslide Field Evaluation

Twenty-seven potential deep-seated landslides were previously identified in and near the dam site footprint and within the potential reservoir pool area using Light Detection and Ranging (LiDAR) hillshade images and geologic maps. These landslides were described in the *Preliminary Desktop Landslide Evaluation* by Shannon & Wilson, Inc. (2014). The intention of the Phase 1 Site Characterization program was to ground-truth the locations and extent of these landslides. A first-level reconnaissance of the identified potential landslides (reached by four-wheel-drive vehicle and by foot) was performed. During this work observations were made of: (1) signs of landslide activity, such as cracks, fissures, scarps, and bulging ground; (2) seepage conditions; (3) vegetational signs of slope movement; (4) undercutting of the landslide mass by the river or creek; (5) potential sites for subsurface exploration; and (6) geologic exposures that may aid in interpretation of the landslide. The results of the Phase 1 landslide reconnaissance are summarized in Appendix A of the Phase 1 Site Characterization TM (HDR AND S&W INC, 2015).

The Phase 2 site characterization program advanced the understanding of the previously identified landslides through additional reconnaissance, and completion of landslide seismic lines and boreholes. Such information will be used to further characterize the potential risks in order to aid in mitigation designs, when required. A Phase 2 Chehalis Dam Landslide Evaluation (Landslide Evaluation) was prepared by Shannon and Wilson Inc. and is included as Appendix B which presents the results of the Phase 2 landslide evaluation and includes descriptions and conclusions about each visited landslide.

Based on the results of the Landslide Evaluation Shannon and Wilson prepared a Landslide Stability Improvement Evaluation which considered the condition of the landslides after stability improvement measures and made recommendations for further study. The Phase 2 Chehalis Dam Site Characterization Landslide Stability Improvement Evaluation is included as Appendix C of this report.

Project Personnel

The following personnel assisted in the completion of the site characterization work and preparation of this report.

HDR

Keith A. Ferguson, P.E.	Principal Engineer
John Charlton	Senior Engineering Geology Specialist
Andrew Little	Project Geotechnical Specialist
Gokhan Inci	Project Geotechnical Specialist
Helen Regan	3-D Visualization Specialist

Shannon & Wilson, Inc.

Stan Boyle, P.E., Ph.D.	Senior Geotechnical Engineer
Bill Laprade, L.E.G.	Senior Engineering Geologist
Erik Scott, L.E.G.	Senior Geologist
Ali Shahbazian, P.E., Ph.D.	Seismic Hazards
William J. Perkins, P.E., L.E.G.	Seismic Hazard and Geologic Models
Elizabeth Barret	Staff Geologist
Stephen Newman	Staff Geologist
Jorge Avalos	Geotechnical Staff
Rex Whistler	Geotechnical Engineer

Global Geophysics

John Liu, R.G., Ph.D.	Principal Geophysicist

3 PHASE 2 SITE CHARACTERIZATION PROGRAM

3.1 Introduction

The scope of the Phase 2 Site Characterization program for the Chehalis Dam site consisted of the activities summarized in Section 2 of this report. Completion of this program required permitting and coordination with Weyerhaeuser, site access preparation involving vegetation clearing and temporary access road construction, delivery and management of drilling fluids, completing the explorations, and site restoration. Additional details related to the program are summarized in the following subsections.

3.2 Site Access

Located on Weyerhaeuser property, the borehole locations were accessed from Weyerhaeuser roads, the City of Pe Ell’s water access road, and temporary pioneer access roads. John J. Karnas Co. (Karnas) was subcontracted to construct the pioneer roads prior to the start of the subsurface exploration program activities. Prior to building the roads, the trees within the approved road right of way were purchased from Weyerhaeuser, cut by Karnas, and left on site. Karnas used an excavator to create the pioneer roads. To make the roads passable in wet areas, Karnas created a corduroy road surface made from the trees cleared for the road construction (Photograph 3.1).

Photograph 3.1
Corduroy Road Surface for Borehole Location Access

Ten pioneer roads were built for the project labeled Road D through Road M totaling approximately 1200 feet, the approximate access road locations are shown in Figure 2.3.

To restore the pioneer roads, Weyerhaeuser requested that the corduroy surface be left in place as erosion control. Water bars were placed every 100 feet in gentle slope area and every 50 feet on the steeper slopes. Straw and seed were spread over areas with exposed soil. Karnas finished the restoration of the pioneer roads after the subsurface explorations were completed.

3.3 Landslide Mapping

Landslides were previously identified using existing landslide inventories, aerial photographs, LiDAR hillshade images, and existing geologic maps (S&W, 2014). Subsequently during the Phase 1 Site Characterization a landslide study to estimate the potential impacts of construction and fluctuating reservoir levels on the stability of the landslides was performed by Shannon & Wilson Inc. and included with the Phase 1 Site Characterization as Appendix A. The purpose of the Phase 2 landslide study was to gather additional information on reservoir landslides through seismic refraction lines and advance the understanding of the dam site landslides through additional boreholes and seismic refraction lines.

The ten landslides that were investigated by seismic refraction lines including both dam site and reservoir landslides were evaluated to assess the impact they would have on construction and operation of either dam option. These ten landslides were chosen for investigation because they either contained a high enough potential failure volume to adversely affect the dam or were at elevations where a majority of the potential failure volume was located between the maximum pool and operating pool of the FRFA option that would experience elevated groundwater conditions and potential rapid drawdown failure modes. Finite element modeling was used to perform both seepage and stability analysis on all ten landslides under various conditions to evaluate the likelihood of failure and thus the need for stabilization. Results of the Phase 2 landslide analysis performed by Shannon and Wilson, Inc. are summarized below in Section 4.2.6 and a detailed report is included as Appendix B.

3.4 Subsurface Characterization

The Phase 2 Site Characterization program included multiple exploration methods to assess dam type feasibility issues, update anticipated foundation excavation and treatment requirements, and provide the basis to update conceptual level designs under a future phase of work. The program included both drilling and geophysical survey methods as discussed in sections 2.5.1 and 2.5.2. Boreholes locations and seismic refraction survey lines are shown on Figures 2.2 and 2.3.

Details of the methods and equipment used to perform the dam site subsurface characterization are discussed in the following sections. Note that borehole locations were scoped to coincide with the seismic refraction lines and the dam alignment; however, adverse field conditions due to steep slopes and heavy tree cover created drill rig access difficulty resulting in some boreholes being performed near, but not at the originally planned locations. Results of the subsurface investigation are discussed in Sections 4.1 and 4.2.

3.4.1 Boreholes

All the boreholes, with the exception of TB-2 and TB-3 were drilled by Holt Services using mud rotary drilling techniques in the overburden and either HQ-3 or PQ-3 core drilling techniques in rock. The PQ-3 drill bit creates a larger diameter borehole, 4.8 inches as opposed to 3.8 inches for HQ-3, to accommodate the inclinometer in the LSB boreholes. Standard Penetration Test (SPT) samples were collected every 5 feet in the overburden using a standard split-spoon sampler. Once bedrock had been reached, HQ-3 (or PQ-3) triple-tube rock coring methods were used to advance the boreholes into bedrock. TB-2 and TB-3 were angled boreholes and were drilled by Crux Subsurface, Inc. using HQ-3 core drilling directly into outcropping rock.

3.4.1.1 Mud-rotary Drilling

In the overburden soil Holt performed the mud-rotary drilling using a Mobile B-54 track-mounted drill rig, equipped with an approximately 3- to 6-inch-diameter tri-cone bit. Mud-rotary drilling uses bentonite drilling mud to carry soil cuttings up the borehole; the mud helped to maintain borehole stability and prevent heave at the borehole base. Soil samples were obtained by replacing the tri-cone bit with a split-spoon sampler and performing the SPT.

3.4.1.2 HQ-3 Triple-tube Rock Coring

Rock core samples were obtained using a 5-foot-long, HQ and PQ-size, triple-tube core barrel. The triple-tube core barrel consisted of inner and outer barrels and a split inner core tube. The outer barrel rotates while the inner barrel and the inner split tube remain stationary. This system protects the core from the drilling fluid and reduces the torsional forces transmitted to the core. In addition, the split inner tube allowed for detailed visual analysis of the relatively undisturbed core sample once it was extracted from the borehole.

Cuttings were removed from the borehole by circulating water or water and polymer through the drill casing. The water used for coring was obtained from the City of Pe Ell. Most core runs were 5 feet long, although runs as short as 0.3 feet were required to improve recovery where low-rock-quality rock material was encountered.

3.4.1.3 Sampling Methods

Samples were collected from each exploration for purposes of geologic evaluation and geotechnical testing. SPTs were performed in general accordance with the American Society for Testing Materials (ASTM) Designation: D1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils (2009). In the SPT, an 18-inch-long, 2-inch-outside-diameter, 1.375-inch-inside-diameter, split-spoon sampler is driven with an automated 140-pound hammer, falling freely from a height of 30 inches. The number of blows required to achieve each of three 6-inch increments of sampler penetration is recorded. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance, or N-value. When penetration resistances exceeded 50 blows for 6 inches or less of penetration, the test was generally terminated and the number of blows along with the penetration distance was recorded on the borehole log. The presence of gravels or cobbles larger than the sampler may impact measured penetration resistances and result in artificially high values.

Rock core recovered from the boreholes was logged and placed into wooden core boxes. In each core box, rock core was arranged in descending sequence beginning at the upper left end of the core box partition and continuing in the other partitions from left to right. Each core run was separated from the preceding run by blocks labeled with the run number, depth, run length, and core recovery. Zones of core loss were indicated with blocks labeled with the estimated depth interval where the loss occurred. If the zone of core loss was uncertain, the core loss was assigned to the bottom of the run. Each core box was photographed in the field when the box was filled with rock core. Select core samples were designated for laboratory testing as is discussed in Section 4.2 and 4.3.

A summary of the drilling and sampling results are found in Section 4.2 and interpretation of the results are presented in Section 5. A detailed Data Report prepared by Shannon and Wilson is attached as Appendix A and includes boring logs, core photos and laboratory results.

3.4.2 Water Pressure Testing

Water pressure tests were performed in twelve boreholes to estimate in-situ hydraulic conductivity and to evaluate the potential groutability of the subsurface bedrock stratigraphy beneath the proposed dam alignment. LSB and QB boreholes were excluded from the water pressure testing program since they will not be grouted or pertain to seepage analysis beneath the dam.

Water pressure tests were performed in completed borings using a double packer test apparatus, a water pump, and clean water obtained from the City of Pe Ell. The double packer test apparatus consisted of two inflatable packers connected by a 10-foot perforated pipe lowered into the borehole. The packers were connected to a nitrogen gas source at the ground surface and were expanded to conform to the borehole walls sealing a 10-foot-long test interval zone. Testing was performed upstage (bottom up), using the Houlsby (1976) method. As part of this method, the test pressure is stepped up to a maximum, calculated based on the estimated overburden pressure, and then stepped back down. The Houlsby method is used to characterize the type of flow occurring during the test and provides an estimate of the Lugeon value (Lu). The Lu value is a measure of bedrock permeability and hydraulic conductivity of fractures and is used to assess groutability and design grouting mixes and procedures. Results of the water pressure testing are discussed in Section 4.2.3.6.

3.4.3 Geophysical Investigations

The Phase 2 program geophysical survey was completed by Global Geophysics between February and April 2016. The methodology and instrumentation used are summarized in the following sections. A detailed report including results is presented in Appendix D. The results of the geophysical investigation are discussed in Section 4.

3.4.3.1 Seismic Refraction Tomography

The velocity of seismic (shear and compression) waves travelling through rock provides an indication of the competency and rippability of the rock and can also provide an indication of the degree of fracturing and weathering present in the rock mass. Seismic refraction tomography is a proven tool used to evaluate the subsurface conditions between boreholes once it is calibrated to information obtained from boreholes. Compression wave velocity profiles from these surveys help establish the depth to competent bedrock and foundation excavation limits, along with identifying highly fractured zones that may require foundation treatment.

The seismic refraction tomography surveys were conducted using a Geometrics Geode 24-channel digital seismograph and Mark Products 8-Hertz vertical geophones planted at 10- to 20-foot intervals.

Charges were set off at seven locations along the geophone array and data was collected and saved in digital format to enable real-time quality assurance and quality control of the data.

3.4.3.2 Optical/Acoustic Televiewer

The optical/acoustic televiewer used in the boreholes was made by Robertson Geologging. It provides a continuous 360° image of the borehole wall using an optical/acoustic imaging system. Televiewer data is collected by starting at the bottom of the borehole and continuing to a location near the top of rock where the borehole casing prevents further survey work. Optical surveys were performed when visibility conditions permitted. Otherwise, the surveys were completed using acoustical methods.

Accurate borehole deviation and image orientation data were obtained during logging with a precision 3-axis magnetometer and two accelerometers. During post-processing, the video image is unwrapped, analyzed, and then displayed. One of the displays shows an inverted image that simulates an intact core sample that can be compared to the extracted sample. These images were analyzed for natural fractures, fracture type, and orientation. When compared with the actual core samples obtained from the boreholes intervals of core loss and where core damage occurred were identified and used to fill in information missing from the core logs.

Televiewer data was obtained in all DB, OB and TB borings (except TB-3 due to collapsing rock in the angled boring) by Global Geophysics with the exception of TB-2 (an angled boring requiring exceptional skill with the equipment) which was obtained by Crux Subsurface Inc. after they performed the drilling.

3.4.3.3 Sonic Suspension Logging

Both compressional and shear wave velocities of the rock formations were evaluated in each borehole using a full-wave triple sonic probe made by Robertson Geologging. Sonic Suspension velocity logging measures compression (P) and shear (S) wave velocities in uncased relatively deep boreholes. The suspension logging system consists of a 27-foot-long probe, containing a source and two to three receivers spaced 1 meter apart, suspended by an information cable, and ultimately connected to a data logger.

Compression and shear wave velocities are measured by suspending the probe at a known depth within a borehole, whereupon the source produces a pressure wave into the borehole sidewall through the drilling fluid. The compression and shear wave components of the induced pressure wave are measured in the rock materials near the borehole. Test information is recorded in the data logger.

Borehole data, SPT and laboratory results, along with water pressure testing results, televiewer data, sonic logging results and other engineering evaluations have been assembled on “composite boring logs” for both Phase 1 and 2 and are included in Appendix E. The composite boring log information is described further in Section 4.2.3.

3.5 Bedrock and soil Laboratory Testing

Laboratory testing was performed on selected samples from the rock core and overburden soil from the subsurface boreholes. Compressive strength testing was performed on eighteen rock core samples of mostly basalt lithology with one claystone sample and one breccia sample. Point load strength testing was performed on fourteen samples of basalt. Slake durability tests were performed on seven claystone samples and one basalt sample. Direct Shear testing was performed on nine samples of basalt from depths where preliminary outlet tunnel locations have been proposed. Specific gravity tests were performed on four samples of basalt and one sample of claystone.

The laboratory test results are summarized in Section 4.2 and laboratory reports are included in Appendix F.

3.6 RCC Aggregate Investigation

Preliminary results of an RCC aggregate investigation were presented in a July 2014 report entitled Quarry Rock Desktop Study included in the Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR 2014). More than 20 potential aggregate sources were identified within a 25-mile radius of the dam site. Based on preliminary information available from Washington State Department of Transportation (WSDOT) Aggregate Source Approval reports, four of these sites were selected for further evaluation during the Phase 1 Site Characterization study, including two commercial sources (Alderbrook and Hope Creek Quarries), an inactive Weyerhaeuser quarry (Rock Creek), and rock from the dam site. The results of that study are presented in the Phase 1 Site Characterization TM (HDR AND S&W INC, 2015). Consequently, only the inactive Weyerhaeuser quarry (Rock Creek) displayed suitable 16 day ASR results and was carried over into the Phase 2 evaluation. In addition to the Rock Creek quarry two other primary locations were identified and lab testing was performed as discussed in Section 2.5.3.3 on the three primary quarry sites. The results of RCC aggregate lab testing program are presented in Section 4.3 below.

3.7 Seismic Hazards

The Seismic Hazard Analysis was performed during Phase 1 (HDR and S&W, 2015) to identify potential ground motion and fault rupture hazards at the Chehalis Dam site.

Subsequently a Preliminary Earthquake Time Histories TM was prepared by Shannon and Wilson in September, 2015 to be used in structural analysis of the conceptual design and is included as an appendix in the Conceptual Design Summary Report (HDR 2016). Additional analysis of the seismic hazards present at the dam site was not within the scope of work for the phase 2 site characterization.

4 EXPLORATION PROGRAM RESULTS

4.1 Geologic Overview

Subsurface borehole data and geophysical data from the following sections along with the Wells and Sawlan 2014 regional preliminary geologic map were used to refine the geologic interpretation presented in the Phase 1 TM (HDR AND S&W INC, 2015). The result is the geologic map shown on Figure 2.3 and associated geologic profile cross-sections shown on Figures 4.1 to 4.4 and 4.9.

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

Younger (last 2 million years) Quaternary surficial deposits of sediments such as stream alluvium, colluvium, and landslide deposits are found along the valley floor, and many slopes mask the underlying bedrock, except where overburden has been stripped by mass wasting or where slopes are too steep to develop soil cover (Shannon & Wilson, Inc. 2009). In addition to sedimentary deposits, most of the underlying bedrock is overlain by residual overburden soil that supports heavy vegetative cover. These Quaternary deposits range in thickness from 0 feet at outcrops and river channels to about 70 feet at the toe of the landslide 1 in the left abutment, and are typically about 30 to 40 feet thick.

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

4.1.1 Description of Geologic Units

The following descriptions are based on previous studies (Shannon and Wilson, 2009) of the area and geologic maps (Wells and Sawlan, 2014) as mentioned in Section 2.2 and the material descriptions made on the borehole logs found in Appendix A. The two- to three-letter abbreviations of the geologic units are used on the geologic map on Figures 2.3 and profiles and sections shown on Figures 4.1 to 4.4 and 4.9. Soil classifications (e.g., MH) are according to the Unified Soil Classification System (USCS, ASTM D 2487-11).

4.1.1.1 Stream Alluvium

Stream alluvium is located along the valley floors of the Chehalis River and its tributary streams and consists primarily of very loose to loose, light to dark brown, stratified slightly silty fine sand, gravelly sand and sandy gravel. Organics are present locally. Larger clasts range from pebbles to boulders, some as large as 3 to 4 feet. The clasts are predominantly subrounded. Modern Quaternary alluvium (Qa) is present in active stream channels and older Quaternary alluvium (Qao) is present in terraces more than 15 feet above the modern stream channel. Qao tends to be denser than Qa and contains subrounded to subangular gravel. Only Qao was observed directly in boreholes completed as part of this Phase 2 study as follows:

- Boring Depth of Qao (feet)
- DB-1 between Qc and Tcb from 7 to 30
- OB-1 20.4
- OB-2 27
- OB-3 34.6
- TB-1 24

4.1.1.2 Colluvium

Colluvium (Qc) consists of poorly sorted, loose to dense, light-brown to reddish-brown, sandy to gravelly clay or silt deposited on or at the base of hillslopes, primarily through gravity-driven transport of weathered rock and soil. These deposits may contain high percentages of subangular boulders consisting of basalt and gabbro ranging widely in size, and could be more than 2 feet in maximum dimension. Colluvium was directly observed only in boreholes at the base of steep slopes:

- Boring Depth of Qc (feet)
- DB-1 7
- TB-4 10

Seismic refraction surveys suggest that these materials range from 0 to 25 feet in vertical thickness along the upper slope of the right abutment near DB-5.

4.1.1.3 Landslide Deposit

Quaternary landslide deposits (QIs) are made up of heterogeneous, mostly unsorted and unstratified debris that is often characterized by hummocky topography, closed depressions, springs or seeps, and a lobate form. The soil in landslide deposits is highly variable. They were observed to consist of loose to very dense, reddish brown to dark gray sandy silt (MH) to clayey or silty sand (SC/SM) to gravel with silt and sand (GP-GM). Some iron staining exists among the Qls and the fines are typically low to medium plasticity. Clasts can range from gravel to boulders and be several feet in maximum direction. Landslide deposits were logged in the following boreholes:

- Boring Depth of Qls (feet)
- TB-5 47.5
- LSB-1 32.5
- LSB-2 40.8
- LSB-3 57.0
- LSB-4 30

Landslide thickness can vary considerably depending on the configuration and depth of the failure plane, ranging from relatively thin (less than 10 feet thick) or more than 100 feet thick at the toe in a deep-seated failure. The two landslides observed in the left abutment by BH-1 and BH-4 completed in Phase 1 and by LSB-1 and LSB-2 completed in Phase 2 appear to be deep-seated failures with a maximum thickness of about 70 feet.

4.1.1.4 Overburden Soil

Where soil has not been deposited by fluvial processes or gravity it is considered residual soil, developed in place from weathering of the bedrock beneath it. Quaternary overburden soil (Qos) was directly observed as follows:

- Boring Depth of Qos (feet)
- DB-5 40
- DB-6 31.5
- DB-7 37
- OB-1 beneath Qao from 20.4 to 29.5

- OB-2 beneath Qao from 27 to 38
- TB-5 beneath Qls from 47.5 to 60
- QB-1 35

Qos consists of medium dense to very dense, red-brown or yellow-brown to dark gray lean clay (CL), elastic silt (MH), silt (ML), silt with sand (ML), sandy silt (ML), sandy silt with gravel (ML), silty sand (SM), silty sand with gravel (SM) and silty gravel (GM). The fines in the Qos range from low to high plasticity except at about 30 feet depth in OB-2 where the sandy silt was non-plastic. Index soil laboratory testing is presented in Section 4.2.1. Overburden soils frequently contain highly weathered clasts of bedrock that are angular to subangular. Iron staining was also observed frequently in the Qos.

4.1.1.5 Intrusive Igneous Volcanics

Intrusive volcanic rocks (Tig) have been identified in the vicinity of the dam site by geologic maps and previous studies as primarily gabbro; which typically is high to very high strength, dark gray to black, occasionally white or black-speckled, aphanitic to medium grained and massive to columnar or block jointed rock. However; Tig was only encountered in BH-4 during the Phase 1 site investigation, and was not present in any of the Phase 2 boreholes. The material in BH-4 may be part of the disturbed landslide complex. At this location, the gabbro ranged from very weak to strong; black to green; medium grained; with rough, very close to moderately spaced, low to high angle joints with iron oxide staining, and mineral and clay infilling; and slightly to highly weathered.

4.1.1.6 McIntosh Formation

The McIntosh Formation (Tml) represents a thick sequence of locally tuffaceous marine siltstone and claystone with interbedded arkosic sandstone and basaltic sandstone. In the vicinity of the dam site, McIntosh claystone is interbedded with Crescent formation basalt (described in the next section), that creates the steep hill slope of the right abutment and underlies the embankment dam footprint. The McIntosh formation has only been observed in BH-6 and is locally very weak to weak, gray, very fine grained, and slightly to moderately weathered with completely weathered zones and cross-bedded sandy siltstone interbeds. Discontinuities are typically rough with trace slickensides and are very close to widely-spaced low to high angle joints with mineral and clay infilling, Crescent formation basalt intrusions ranging from 0.5 to 1.1 feet thick were observed within the McIntosh claystone in BH-6. Due to the contemporaneous depositions with the crescent formation it is difficult to determine if the claystone units found in the right abutment (DB-5, DB-6 and DB-7) are Mcintosh formation or Crescent formation claystones discussed in the following section.

4.1.1.7 Crescent Formation Basalt and Siltstone/Claystone

The Crescent Formation (Tcb) is characterized by massive basalt flows, pyroclastic flows, and tuffaceous sandstones. Crescent basalts are often in the form of pillow basalt flows but can also be locally intrusive. Several sequences of volcanism occurred during the deposition of Crescent formation basalts resulting in interbeds of siltstone and claystone (Moothart 1992). Specifically, these materials were encountered as alternating sequences of pillow basalt (Tcb) deposition, and weathering and erosional events of the pillow basalt to silts and clays deposited within depressions that were lithified to siltstone/claystone (Tcs) and occasionally claystone breccias consisting of basalt clasts in a claystone matrix by subsequent events of pillow basalt flow deposition. Both the Crescent and McIntosh Formations were deposited in the early to middle Eocene age contemporaneously. The Crescent Formation began developing in the early Eocene, and the McIntosh formation began developing later in the early to middle Eocene while the Crescent Formation continued to develop (Wells and Sawlan 2014).

The Crescent formation basalts were found in every borehole and ranged from weak to very strong. The strength of these materials increases with depth. They are dark gray to gray-green fine to medium grained, with smooth to rough, closely to widely spaced, high to low angle joints with occasional mineral and rare clay infilling. The basalt was typically fresh to slightly weathered with occasional moderately to highly weathered zones. Iron oxide staining occurs locally and the basalt is locally slightly vesicular. The Crescent Formation basalt makes up a large portion of the subsurface lithology at the potential dam site and was also found at all three Quarry locations. Initial laboratory assessment of the basalt at the quarry locations indicate it will be suitable for the development of both RCC and conventional concrete aggregate and also suitable for riprap and roadway surfacing materials that will be required.

In between basalt flows, local volcanic rocks weathered and were eroded and deposited as silt and clay interbedded units, ultimately becoming siltstone (claystone) within the Crescent Formation as a sub-unit (Tcs). These materials have some similar characteristics to the McIntosh formation since they were derived the same way however Tcs is older, more consolidated and generally less weathered than the McIntosh formation found in BH-6. The Tcs (Tertiary Crescent Formation siltstone/claystone) encountered in the boreholes range from locally very weak (strength index of 2 or less) to moderately strong, dark gray to black, very fine to fine grained, with smooth to rough, closely to moderately spaced, low to high angle joints, and with occasional clay infillings. Rock core samples were mostly fresh to slightly weathered with zones of moderate and high weathering. Low angle bedding planes were observed in the rock core. The geologic map of Wells and Sawlan (2014) shows a regional orientation of sedimentary beds dipping about 17 degrees to the north. However the Phase 2 borings petrographic analyses have revealed bedding planes that varied within the same unit from horizontal to a dip of about 17 degrees. This implies a variable depositional environment without a persistent horizontal bedding plane at deposition.

The top of a Tcs subunit of siltstone/claystone was found in borings DB-1, OB-2 and BH-2 that has a similar elevation. This suggests the possibility of a more persistent horizontal orientation of the top of this subunit. Based on the Phase 1 and 2 information gathered to date, a regional deposition/bedding orientation could not be confirmed based solely on boring data. The Tcs unit was not found in outcrop enough to measure/confirm a reliable regional orientation of these subunits.

The Tcs layers also are likely to be truncated by later intrusive volcanics. Hence, regular lenses as would occur in a quiet sedimentary environment may only occasionally be found. Evidence of this is shown in BH-5 which has a Tcs layer higher than the elevation of the Tcs in DB-1, OB-2 and BH-2. Such observations suggest that the Tcs in BH-5 is likely a separate unit. At this time, borings are just too far apart to establish lateral continuity.

Brecciated siltstone (and claystone) was observed in TB-5, LSB-4 and QB-2. These materials were likely created as angular clasts of basalt that were eroded from the host rock between volcanic events and accumulated in depressions that were overlain with silt and/or clay. Crescent Formation siltstone (claystone) occurred in every borehole.

4.2 Dam Site Investigation Results

The borehole and geophysical data presented in the following sections have been used to develop and subsequently update a geologic model of the dam site. Both two dimensional (2D) and 3D models have been developed. Examples of the 2D results of this interpretation are shown along dam profiles, cross-sections and seismic line transects. The locations of the profiles and cross-sections are shown on Figure 2.3. Figure 4.1 shows the RCC dam and the embankment dam centerline profile with the approximate excavation objective, discussed in Section 5.2.1, and approximate foundation grout curtain limits, discussed in Section 5.3.2. Figure 4.2 shows a profile along the construction diversion tunnel (1), a cross section through the spillway flip bucket that also corresponds with DSL-3 (2), and a section through the proposed FRFA outlet works and fish passage stilling basin (3). Figure 4.3 shows a cross section through the spillway that corresponds with DSL-2 (4), a cross section through the RCC dam to the left of the spillway showing the landslides upstream (Landslide 1) and downstream (Landslide 1a) of the left abutment (5), and a cross section through the embankment dam (6). Figure 4.4 shows a section through Landslide 1 upstream of the left abutment that corresponds with LSSL-6 and extends to the construction diversion tunnel near the intake (7), a section through landslide 3a upstream of the dam that corresponds with LSSL-7 (8), a section through what turned out to be an erosional feature and not actually a landslide that corresponds with LSSL-8 (9), and a section through the fish passage stilling basin and landslide 26 downstream of the dam that corresponds with LSSL-9 (10).

It should be noted that the profile and cross-sections presented represent the FRFA dam configuration. This is the largest possible dam configuration being considered. These sections are also applicable to the smaller FRO dam configuration. It should be noted that the FRO configuration does not include the upper right abutment embankment dam.

Details of the completed boreholes are summarized in Table 4.1. The subsurface conditions identified in each borehole are summarized in Table 4.2.

Table 4.1
Summary of Completed Borehole Information

PHASE	BOREHOLE NUMBER	LOCATION	EASTING[1]	NORTHING[1]	ELEVATION[1] (FT)	DEPTH (FT)	SOIL/ WEATHERED ROCK DEPTH (FT)	ANGLE OF HOLE FROM HORIZONTAL (DEGREES)	AZIMUTH (DEGREES)	WATER PRESSURE TESTS		GROUNDWATER[2]	
										HIGHEST LUGEON VALUE	DEPTH (FT)	DEPTH (FT)	ELEVATION (FT)
1	**BH-1**	Left abutment downstream landslide	936024.4	454038.5	507	140.4	35	90	NA	17.3	118	53	455
	BH-2	Dam alignment centerline - near maximum section	936325.6	454246.7	456	241	30	90	NA	75.4	178	23	432
	BH-3	Left abutment upstream landslide	935961.8	453694.0	577	150	35	90	NA	12.4	47	103	499
	BH-4	Left abutment upstream landslide	936303.2	453852.2	516	120	62.5	90	NA	22.7	87	60	452
	BH-5	Dam alignment centerline - left abutment	936012.7	453826.0	560	250	47.5	90	NA	67.5	57	127	432
	BH-6	Dam alignment centerline - right abutment	936660.0	454684.9	664	350	32.5	90	NA	165.8	137	243	422
2	**DB-1**	Alignment Sta. 6+50	936187.86	454055.83	470.74	240	30	90	NA	26.97	95	43	428
	DB-5	Alignment Sta. 15+36	936836.907	454641.444	686.794	200	40	90	NA	40.1	76	194	492.5
	DB-6	Embankment Dam Alignment Sta. 4+15	937040.256	454475.286	673.814	200	31.5	90	NA	22.3	186	192	482
	DB-7	Embankment Dam Alignment Sta. 9+00	937367.468	454137.034	691.829	200	37	90	NA	9.75	56	196	496
	OB-1	Upstream of Dam along FRFA dam flood control outlet	936516.79	453914.79	464.68	100	29.5	90	NA	12.29	65	36	429
	OB-2	Along Dam Alignment and FRFA dam flood control outlet	936273.08	454141.41	467.84	240	38	90	NA	2.71	55	40	428
	OB-3	Downstream of Dam along FRFA dam flood control outlet	935988.82	454415.80	466.20	100	34.3	90	NA	4.34	65	36	430
	TB-1	Upstream of Dam along FC dam flood control outlet	936842.9557	454093.6301	465.4385	100	25	90	NA	2.84	35	38	427.5
	TB-2	Alignment Sta. 11+68	936576.19	454445.04	467.85	200	0	22	N47E	50.56	176.6	33	435
	TB-3	Downstream of Dam Alignment along road	936377.8776	454573.3282	455.0369	200	0	18	N47E	213.96	42.5	25	430
	TB-4	Downstream of Dam Alignment along road	936397.08	454570.29	458.53	100	10	90	NA	65.71	45	40	419
	TB-5	Downstream of dam alignment in the landslide	936137.8354	454834.305	488.206	100	60	90	NA	5.4	75	70	418
	LSB-1	Landslide 1	936180.9439	453752.0784	524.6889	105	40	90	NA	NA	NA	16	509
	LSB-2	Landslide 1a	935874.80	454200.11	520.71	110	40	90	NA	NA	NA	21	500
	LSB-3	Landslide 3	937673.92	453352.60	556.59	100	57	90	NA	NA	NA	36	521
	LSB-4	Landslide 3a	937156.48	453873.02	586.29	50	30	90	NA	NA	NA	29	557.5
	QB-1	15NE1	939529.47	448782.15	804.20	150	35	90	NA	NA	NA	NA	NA
	QB-2	23SE1(P)	914366.39	439616.43	2564.25	150	0	90	NA	NA	NA	NA	NA

Notes:
1. Coordinate System: State Plane 4602 US Survey feet; Datum: WGS84. Coordinates and elevation obtained from survey data
2. Groundwater measurements were last taken for BH-2 and BH-6 taken in November, 2015 and BH-3, BH-4 and BH-5 in March, 2016 due to data logger failure; all other measurements taken in May, 2016

Table 4.2
Site Stratigraphy by Borehole

BOREHOLE NUMBER	LITHOLOGY	FORMATION	DEPTH (FT)		ELEVATION (FT)	
			TOP	BOTTOM	TOP	BOTTOM
BH-1	Soil	Qls	0	35	507	472
	Basalt	Tcb	35	102.4	472	404.6
	Claystone	Tcs	102.4	110.8	404.6	396.2
	Basalt	Tcb	110.8	124.9	396.2	382.1
	Claystone	Tcs	124.9	140.4	382.1	366.6
BH-2	Soil	Qao	0	30	456	426
	Basalt	Tcb	30	80.7	426	375.3
	Claystone	Tcs	80.7	131.7	375.3	324.3
	Basalt	Tcb	131.7	135.6	324.3	320.4
	Claystone-Siltstone	Tcs	135.6	138.1	320.4	317.9
	Basalt	Tcb	138.1	212.3	317.9	243.7
	Claystone	Tcs	212.3	219.3	243.7	236.7
	Siltstone-Sandstone	Tcs	219.3	241	236.7	215
BH-3	Soil	Qos	0	35	577	542
	Basalt	Tcb	35	133	542	444
	Claystone	Tcs	133	150	444	427
BH-4	Soil	Qls	0	62.5	516	453.5
	Gabbro	Tig	62.5	94.2	453.5	421.8
	Claystone	Tcs	94.2	120	421.8	396
BH-5	Soil	Qos	0	47.5	560	512.5
	Siltstone	Tcs	47.5	70.1	512.5	489.9
	Basalt	Tcb	70.1	146.1	489.9	413.9
	Claystone	Tcs	146.1	180.9	413.9	379.1
	Basalt	Tcb	180.9	250	379.1	310
BH-6	Soil	Qos	0	32.5	664	631.5
	Claystone	Tml	32.5	61.3	631.5	602.7
	Breccia	Tcb	61.3	75	602.7	589
	Basalt	Tcb	75	175.5	589	488.5
	Siltstone	Tcs	175.5	191.4	488.5	472.6
	basalt	Tcb	191.4	260	472.6	404
	Claystone	Tcs	260	320	404	344
	Basalt	Tcb	320	350	344	314

BOREHOLE NUMBER	LITHOLOGY	FORMATION	DEPTH (FT)		ELEVATION (FT)	
			TOP	BOTTOM	TOP	BOTTOM
DB-1	Soil	Qc	0	7	470.7	463.7
	Soil	Qao	7	30	463.7	440.7
	Basalt	Tcb	30	77	440.7	393.7
	Claystone	Tcs	77	85.6	393.7	385.1
	Basalt	Tcb	85.6	93.5	385.1	377.2
	Siltstone/Claystone	Tcs	93.5	108.3	377.2	362.4
	Claystone	Tcs	108.3	148.3	362.4	322.4
	Basalt	Tcb	148.3	219.6	322.4	251.1
	Claystone	Tcs	219.6	240	251.1	230.7
DB-5	Soil	Qos	0	40	686.8	646.8
	Basalt	Tcb	40	89.7	646.8	597.1
	Claystone	Tml	89.7	122.9	597.1	563.9
	Basalt	Tcb	122.9	200	563.9	486.8
DB-6	Soil	Qos	0	31.5	673.8	642.3
	Claystone	Tml	31.5	144	642.3	529.8
	Basalt	Tcb	144	200	529.8	473.8
DB-7	Soil	Qos	0	37	691.8	654.8
	Basalt	Tcb	37	81.9	654.8	609.9
	Claystone	Tml	81.9	150.2	609.9	541.6
	Basalt	Tcb	150.2	151.2	541.6	540.6
	Claystone	Tml	151.2	153.3	540.6	538.5
	Breccia	Tml	153.3	170	538.5	521.8
	Basalt	Tcb	170	174.6	521.8	517.2
	Breccia	Tml	174.6	185.6	517.2	506.2
	Basalt	Tcb	185.6	200	506.2	491.8
OB-1	Soil	Qao	0	20.4	464.7	444.3
	Soil	Qos (Tcb)	20.4	29.5	444.3	435.2
	Basalt	Tcb	29.5	59	435.2	405.7
	Claystone	Tcs	59	100	405.7	364.7
OB-2	Soil	Qao	0	27	467.8	440.8
	Soil	Qos (Tcb)	27	38	440.8	429.8
	Basalt	Tcb	38	94.9	429.8	372.9
	Claystone	Tcs	94.9	157.9	372.9	309.9
	Basalt	Tcb	157.9	224.1	309.9	243.7
	Claystone	Tcs	224.1	239.6	243.7	228.2
OB-3	Soil	Qao	0	34.6	466.2	431.6

BOREHOLE NUMBER	LITHOLOGY	FORMATION	DEPTH (FT)		ELEVATION (FT)	
			TOP	BOTTOM	TOP	BOTTOM
	Basalt	Tcb	34.6	69	431.6	397.2
	Claystone	Tcs	69	100	397.2	366.2
TB-1	Soil	Qao	0	25	465.4	440.4
	Claystone	Tcs	25	32.2	440.4	433.2
	Basalt	Tcb	32.2	83.1	433.2	382.3
	Siltstone	Tcs	83.1	100	382.3	365.4
TB-2	Basalt	Tcb	0	51	467.9	448.7
	Claystone	Tcs	51	62.7	448.7	444.4
	Basalt	Tcb	62.7	200	444.4	392.9
TB-3	Basalt	Tcb	0	109.2	455.0	421.3
	Claystone	Tcs	109.2	111.2	421.3	420.7
	Basalt	Tcb	111.2	113.1	420.7	420.1
	Claystone	Tcs	113.1	200	420.1	393.2
TB-4	Soil	Qc	0	10	458.5	448.5
	Basalt	Tcb	10	43.8	448.5	414.7
	Claystone	Tcs	43.8	88.1	414.7	370.4
	Basalt	Tcb	88.1	100	370.4	358.5
TB-5	Soil	Qls	0	47.5	488.2	440.7
	Soil	Qos (Tcs)	47.5	60	440.7	428.2
	Claystone	Tcs	60	88.5	428.2	399.7
	Breccia	Tcs	88.5	94.7	399.7	393.5
	Claystone	Tcs	94.7	100	393.5	388.2
LSB-1	Soil	Qls	0	32.5	524.7	492.2
	Claystone	Tcs	32.5	47	492.2	477.7
	Basalt	Tcb	47	105	477.7	419.7
LSB-2	Soil	Qls	0	40.8	520.7	479.9
	Basalt	Tcb	40.8	110	479.9	410.7
LSB-3	Soil	Qls	0	57	556.6	499.6
	Claystone	Tcs	57	97.9	499.6	458.7
	Basalt	Tcb	97.9	100	458.7	456.6
LSB-4	Soil	Qls	0	30	586.3	556.3
	Breccia	Tcs	30	37	556.3	549.3
	Claystone Breccia	Tcs	37	40	549.3	546.3
	Siltstone	Tcs	40	50	546.3	536.3
QB-1	Soil	Qos	0	35	804.2	769.2
	Claystone/Basalt	Tcb	35	37.8	769.2	766.4
	Basalt	Tcb	37.8	114.5	766.4	689.7
	Siltstone	Tcs	114.5	119	689.7	685.2
	Breccia	Tcb	119	150	685.2	654.2
QB-2	Basalt	Tcb	0	93.9	2564.3	2470.4
	Claystone	Tcs	93.9	109.7	2470.4	2454.6
	Basalt	Tcb	109.	112.1	2454.6	2452.2

BOREHOLE NUMBER	LITHOLOGY	FORMATION	DEPTH (FT)		ELEVATION (FT)	
			TOP	BOTTOM	TOP	BOTTOM
			7			
	Siltstone	Tcs	112.1	113	2452.2	2451.3
	Breccia	Tcs	113	117.6	2451.3	2446.7
	Siltstone	Tcs	117.6	131.5	2446.7	2432.8
	Basalt	Tcb	131.5	150	2432.8	2414.3

4.2.1 Overburden Soil Laboratory Results

Laboratory testing of materials from the dam foundation included representative overburden soil samples from the test boreholes as described in Section 2.5.3.1. Laboratory soil testing results are summarized in Table 4.1. Most of the soil located in the dam vicinity is residual soil or colluvium that ranges from elastic silt to silty sand. However; there is significant gravel at the surface of the alluvium terraces that flank the river found in the OB-1, OB-2, OB-3 and TB-1. A majority (19) of all samples (30) tested for plasticity (Atterberg Limits) were above 50% liquid limit (LL) hence are considered plastic materials. The natural water content ranged widely from 3.5% in a poorly graded gravel found at 15 feet depth in OB-3 to 107.5% in a silty sand found at 15 feet depth in DB-6. The overburden soil found in the landslide borings has a wide range of characteristics including gravel content ranging from 0 to 57% and fines content ranging from 9.9 to 58%. All the soil in the dam foundation excavation will be removed and further analysis should be carried out to evaluate suitability for reuse as fill material if needed.

Table 4.3
Phase 1 and 2 Laboratory Soil Testing Results

PHASE	BOREHOLE	SAMPLE NUMBER	DEPTH (FEET)	GRAIN SIZE DISTRIBUTION – ASTM D422							ATTERBERG LIMITS – ASTM D4318			
				USCS GROUP SYMBOL	USCS GROUP NAME	GRAVEL %	SAND %	FINES %	<0.02MM %	<2 µM %	NAT WC %	LL %	PL %	PI %
1	**BH-1**	S-2	10	MH	Sandy Elastic Silt	0	34	66	50	23	45.5	59	37	22
	BH-1	S-4*	20	SM	Silty Sand	7	45	48	36	15	36.5	54	32	22
	BH-2	S-2*	10	SM	Silty Sand	9	57	35	17	6	28.7	--	--	--
	BH-3	S-4*	20	MH	Elastic Silt	--	--	--	--	--	42.3	64	35	29
	BH-4	S-8*	40	SC	Clayey Sand with Gravel	28	44	28	16	6	19.9	48	26	22
	BH-4	S-10*	50	SM	Silty Sand	7	61	32	20	10	23.9	44	27	17
	BH-6	S-6	30	SM	Silty Sand	0	71	29	15	5	27	30	29	1
2	**DB-1**	S-2*	10	GP-GM	Poorly graded Gravel with Silt and Sand	70	20	10	7	3	12.9	--	--	--
	DB-1	S-3*	15	SM	Silty Sand	0	61	39	19	4	32	--	--	--
	DB-5	S-3	15	MH	Elastic Silt	--	--	--	--	--	37	73	42	31
	DB-5	S-4	20	ML	Silt with Sand	0	23	77	66	42	40.2	--	--	--
	DB-5	S-5	25	MH	Elastic Silt	--	--	--	--	--	46.5	77	50	27
	DB-5	S-7	35	MH	Elastic Silt with Sand	--	--	--	--	--	79.2	66	51	15
	DB-6	S-2	10	MH	Elastic Silt	--	--	--	--	--	56.8	82	62	20
	DB-6	S-3	15	ML	Silt with Sand	0	15	85	72	50	107.5	--	--	--
	DB-6	S-4	20	MH	Elastic Silt	--	--	--	--	--	60.5	76	42	34
	DB-6	S-5A	25	MH	Elastic Silt	--	--	--	--	--	71	84	57	27
	DB-7	S-3	15	MH	Elastic Silt	--	--	--	--	--	89.7	109	71	38
	DB-7	S-4	20	MH	Elastic Silt	--	--	--	--	--	66.9	65	59	6
	DB-7	S-7	35	ML	Silt	--	--	--	--	--	40.6	49	38	11
	LSB-1	S-2*	10	SM	Silty Sand with Gravel	26	48	25	15	6	19.8	--	--	--
	LSB-1	S-3*	15	SM	Silty Sand	1	66	33	20	9	24.2	--	--	--

PHASE	BOREHOLE	SAMPLE NUMBER	DEPTH (FEET)	GRAIN SIZE DISTRIBUTION – ASTM D422							ATTERBERG LIMITS – ASTM D4318			
				USCS GROUP SYMBOL	USCS GROUP NAME	GRAVEL %	SAND %	FINES %	<0.02MM %	<2 µM %	NAT WC %	LL %	PL %	PI %
	LSB-1	S-4	20	ML	Silt	--	--	--	--	--	14.5	29	23	6
	LSB-1	S-5*	25	GP-GM	Poorly graded Gravel with Silt and Sand	57	33	9.9	5	2	10.2	--	--	--
	LSB-1	S-8	40	CL	Sandy Lean Clay	--	--	--	--	--	19.2	46	26	20
	LSB-2	S-2*	10	SM	Silty Sand with Gravel	33	50	17	8	3	18.4	--	--	--
	LSB-2	S-4*	20	GM	Silty Gravel with Sand	53	35	12	6	2	11.4	--	--	--
	LSB-2	S-6*	30	GM	Silty Gravel with Sand	56	32	12	--	--	15.7	--	--	--
	LSB-3	S-1	5	MH	Gravelly Elastic Silt	--	--	--	--	--	33.9	55	33	22
	LSB-3	S-4	20	MH	Elastic Silt	--	--	--	--	--	46.9	68	34	34
	LSB-3	S-6	30	MH	Sandy Elastic Silt	0	42	58	41	14	67.1	66	47	19
	LSB-3	S-9	45	MH	Elastic Silt	--	--	--	--	--	54	63	42	21
	LSB-3	S-11	55	SM	Silty Sand	0	57	43	31	11	48.9	--	--	--
	LSB-4	S-2*	10	SM	Silty Sand	0	53	47	--	--	65	--	--	--
	LSB-4	S-4*	20	SM	Silty Sand with Gravel	19	64	17	--	--	15.2	--	--	--
	LSB-4	S-5*	25	SM	Silty Sand with Gravel	17	54	28	12	4	20.2	41	27	14
	OB-1	S-1*	5	GP-GM	Poorly graded Gravel with Silt and Sand	69	24	7.3	--	--	15	--	--	--
	OB-1	S-3*	15	GM	Silty Gravel with Sand	57	31	12	--	--	14.5	--	--	--
	OB-1	S-4*	20	SM	Silty Sand with Gravel	22	60	18	--	--	18.4	--	--	--
	OB-2	S-2*	10	GP-GM	Poorly graded Gravel with Silt and Sand	67	24	9	5	1	11.1	--	--	--
	OB-2	S-6	30	ML	Sandy Silt	--	--	--	--	--	36	32	33	NP
	OB-2	S-7	35	SM	Silty Sand	--	--	--	--	--	25.1	29	31	NP
	OB-3	S-1*	5	GW-GM	Well-graded Gravel with Silt and Sand	69	29	7.8	--	--	17.5	--	--	--
	OB-3	R-4*	15	GP	Poorly graded Gravel	99	1	0.2	--	--	3.2	--	--	--
	OB-3	S-5*	30	SM	Silty Sand with Gravel	36	52	12	--	--	17.6	--	--	--
	QB-1	S-2*	10	MH	Sandy Elastic Silt	0	47	53	--	--	73.1	69	61	8

PHASE	BOREHOLE	SAMPLE NUMBER	DEPTH (FEET)	GRAIN SIZE DISTRIBUTION – ASTM D422							ATTERBERG LIMITS – ASTM D4318			
				USCS GROUP SYMBOL	USCS GROUP NAME	GRAVEL %	SAND %	FINES %	<0.02MM %	<2 µM %	NAT WC %	LL %	PL %	PI %
	QB-1	S-4*	20	MH	Sandy Elastic Silt	--	38	62	--	--	65.5	62	58	4
	QB-1	S-6*	30	MH	Elastic Silt with Sand	--	23	77	--	--	45.8	69	40	29
	TB-1	S-2*	10	GP-GM	Poorly graded Gravel with Silt and Sand	67	24	9.1	5	3	20	--	--	--
	TB-1	S-4*	20	SM	Silty Sand with Gravel	37	40	24	--	--	24.4	--	--	--
	TB-5	S-2*	10	SM	Silty Sand with Gravel	27	56	17	13	6	33.4	--	--	--
	TB-5	S-3*	15	SM	Silty Sand with Gravel	17	49	34	23	9	46.5	--	--	--
	TB-5	S-6	30	ML	Sandy Silt	--	--	--	--	--	28.2	33	30	3
	TB-5	S-8	40	ML	Sandy Silt	--	--	--	--	--	20.5	31	27	4
	TB-5	S-10	50	CH	Sandy Fat Clay	--	--	--	--	--	22.2	51	27	24

Notes:
* = Sample specimen weight did not meet required minimum mass for ASTM method
USCS = Unified Soil Classification System
WC = water content
LL = liquid limit
PL = plastic limit
PI = plasticity index
mm = millimeters
µm = micrometers

4.2.2 Foundation Bedrock Laboratory Results

Lab testing was performed on selected samples according to the scope of work in section 2.5.3.2 and a brief discussion of the results is presented in the following sections. Detailed laboratory reports are included as Appendix F.

4.2.2.1 Strength and Density Testing

Three types of tests were performed to evaluate the strength, density, Young's modulus and Poisson's ratio of the foundation rock. The point load test is a quick method of testing several samples for relative strength however the test method is known to be imprecise. Imperial correlations exist to derive unconfined compressive strength (UCS) in pounds per square inch (psi) from point load results. However the UCS test (ASTM D7012) is necessary for correlation and accuracy. The results of successful point load tests are shown in Table 4.4. The UCS test can be run in two ways; method C and method D. Both methods provide compressive strength results and bulk density, shown in Table 4.5. However; method D of the UCS test also provides Young's Modulus and Poisson's ratio which are shown in Table 4.6.

Table 4.4
Phase 2 Rock Laboratory Testing Results – Point Load (ASTM 5731)

BOREHOLE	LITHOLOGY	DEPTH (FT)		UCS (PSI)
		FROM	TO	
OB-1	basalt	43.8	44.5	16,600
OB-2	basalt	48.3	48.7	2,450
OB-2	basalt	70.9	71.4	14,600
DB-5	basalt	43.3	43.6	19,600
DB-7	basalt	45.0	45.5	10,000
TB-2	basalt	77.9	78.6	14,300
TB-2	basalt	105.4	105.7	11,700
TB-3	basalt	80.8	81.2	14,700
TB-3	basalt	102.5	103.1	9,500
TB-5	claystone	72.8	73.2	540

Note: All tests performed as diameter tests. PSI – pounds per square inch.

Table 4.5
Phase 1 and 2 Rock Laboratory Testing Results - Bulk Density and Compressive Strength

PHASE	BOREHOLE	DEPTH (FT)		LITHOLOGY	BULK DENSITY (POUNDS/ CUBIC FEET)	COMPRESSIVE STRENGTH (PSI)	ASTM
		FROM	TO				
1	**BH-1**	45.76	46.2	Basalt - Tcb	171	15,683	D7012 - Method C
		76.22	76.66	Basalt - Tcb	174	20,663	D7012 - Method D
	BH-2	41.7	42.14	Basalt - Tcb	171	14,043	D7012 - Method C
		62.15	62.59	Basalt - Tcb	173	26,083	D7012 - Method D
	BH-5	96.5	96.94	Basalt - Tcb	170	12,728	D7012 - Method C
		123.5	123.94	Basalt - Tcb	170	15,345	D7012 - Method D
	BH-6	105.06	105.5	Basalt - Tcb	174	17,751	D7012 - Method D
		213.86	214.3	Basalt - Tcb	171	13,786	D7012 - Method C
2	**DB-1**	40.1	40.55	Basalt - Tcb	171	11,407	D7012 - Method C
		77.93	78.38	Claystone - Tcs	143	2,598	D7012 - Method C
	DB-5	75.04	75.54	Basalt - Tcb	171	13,883	D7012 - Method C
	DB-7	81.28	81.73	Basalt - Tcb	164	13,318	D7012 - Method C
	LSB-1	61.95	62.55	Basalt - Tcb	168	8,271	D7012 - Method C
	OB-1	41.0	41.6	Basalt - Tcb	168	14,175	D7012 - Method C
	OB-2	42.2	43.0	Basalt - Tcb	158	1,229	D7012 - Method C
	OB-3	45.6	46.8	Basalt - Tcb	170	18,359	D7012 - Method C
	QB-1	38.8	39.6	Basalt - Tcb	158	5,763	D7012 - Method C
		85.0	85.9	Basalt - Tcb	164	1,834	D7012 - Method C
		125.7	126.7	Breccia - Tcb	163	5,237	D7012 - Method C
	QB-2	6.0	6.8	Basalt - Tcb	168	13,210	D7012 - Method C
		71.1	72.2	Basalt - Tcb	167	9,655	D7012 - Method C
	TB-1	35.1	35.55	Basalt - Tcb	167	8,271	D7012 - Method C
	TB-2	77.3	77.75	Basalt - Tcb	167	8,805	D7012 - Method C
		103.1	103.55	Basalt - Tcb	166	15,282	D7012 - Method C
	TB-3	92.6	93.05	Basalt - Tcb	171	12,828	D7012 - Method C
	TB-4	36.0	36.8	Basalt - Tcb	169	5,093	D7012 - Method C

Note: psi = pounds per square inch

Table 4.6
Phase 1 Rock Laboratory Testing Results – Young's Modulus and Poisson's Ratio (ASTM D7012-Method D)

BORE-HOLE	DEPTH (FEET)		LITHOLOGY	STRESS RANGE (PSI)		YOUNG'S MODULUS ($X10^6$ PSI)	AVG. YOUNG'S MODULUS ($X10^6$ PSI)	POISSON'S RATIO	AVG. POISSON'S RATIO
				FROM	TO				
BH-1	76.22	76.66	Basalt - Tcb	2100	7600	9.410	8.933	0.26	0.273
				7600	13100	9.360		0.26	
				13100	18600	8.030		0.3	
BH-2	62.15	62.59	Basalt - Tcb	2600	9600	11.000	10.147	0.29	0.320
				9600	16500	10.300		0.29	
				16500	23500	9.140		0.38	
BH-5	123.5	123.9	Basalt - Tcb	1500	5600	8.370	8.100	0.26	0.267
				5600	9700	8.120		0.27	
				9700	13800	7.810		0.27	
BH-6	105.06	105.5	Basalt - Tcb	1800	6500	8.340	7.600	0.29	0.363
				6500	11200	7.900		0.31	
				11200	15800	6.560		0.49	

Note: psi = pounds per square inch

Only one claystone sample was selected for each point load testing and UCS testing resulting in 540 and 2,598 psi respectively which is considered very weak rock by Hoek and Bray (1997). A single sample of Breccia was tested from QB-1 and is also classified as weak rock. The strength of basalt ranged widely from 1,229 to 26,083 psi with an average of 12,452 psi. Although the basalt has significantly higher strength than the siltstone/claystone and breccia tested, it is slightly low for basalt which typically has an unconfined compressive strength in the range of 14,500 to 50,000 psi (Johnson and Degraff, 1988). The lower than typical strengths found in the dam vicinity may be due to vesicles which reduces rock strength locally and were observed in a few of the borings. However the rock mass will exhibit behavioral characteristics based on the average strength of the overall rock mass which is considered moderately strong according to Hoek and Bray (1997).

Specific gravity is the ratio of the density of a material to the density of water. Four samples of basalt and one sample of claystone were tested for specific gravity as shown in Table 4.7. The basalt ranged from 2.63 to 3.03 with an average of 2.67. The average specific gravity, like average strength is also lower than typical for basalt which usually ranges from 2.8 to 3.0 (Edumine, 2016).

Table 4.7
Phase 2 Rock Laboratory Testing Results – Specific Gravity (ISRM Method 2)

BOREHOLE	DEPTH (FT)		LITHOLOGY	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SPECIFIC GRAVITY	POROSITY
	FROM	TO					
OB-1	43.6	43.9	Basalt - Tcb	2.84	160	2.66	0.05
OB-2	41	41.5	Basalt - Tcb	5.67	141	3.03	0.23
OB-3	47.2	47.6	Basalt - Tcb	2.65	163	2.67	0.06
TB-4	37.4	37.7	Basalt - Tcb	2.91	162	2.63	0.06
TB-4	47.8	48.2	Claystone - Tcs	13.6	119	2.66	0.26

4.2.2.2 Slake Durability Testing

The slake durability test is used to evaluate the reaction of clay bearing rocks to the weathering processes of air and water, the results of this test are presented in Table 4.8. The slake durability (%) represents the amount of material remaining after drying, wetting and rotating the sample in a drum for 20 minutes. Additional information about the sample is derived by describing the fragments left according to the note in Table 4.8. Basalt typically does not slake and this is confirmed with the only basalt sample having a slake durability % of 98. The slake durability % of the claystone/siltstone varied considerably between 5.0 and 88% with an average of 48.3%. The slake durability test results indicate the potential for these materials to degrade when exposed in excavations or foundation drain holes. Appropriate treatment precautions will be required when these materials are encountered.

Table 4.8
Phase 1 and 2 Rock Laboratory Testing Results - Slake Durability (ASTM D4644)

PHASE	BOREHOLE	DEPTH (FT)		LITHOLOGY	SLAKE DURABILITY %	WATER TEMP. AVG. °C	AS-RECEIVED WATER CONTENT %	DESCRIPTION OF FRAGMENTS
		FROM	TO					
1	**BH-1**	108.0	109.0	Siltstone - Tcs	84.7	22	11.8	Type II
	BH-2	84.0	85.2	Siltstone - Tcs	38.4	23	13.5	Type II
	BH-5	58.0	59.3	Siltstone - Tcs	56.0	22	11.2	Type III
	BH-6	56.5	57.5	Siltstone - Tml	10.7	22.0	13.6	Type III
2	**DB-1**	87.1	87.9	Basalt - Tcb	98.0	21	6.8	Type II
	DB-5	100.0	100.8	Claystone - Tcs	88.0	21	12.0	Type II
	DB-6	67.4	68.8	Claystone - Tcs	8.9	21	16.6	Type II
	TB-1	26.2	26.9	Claystone - Tcs	62.0	20	15.9	Type II
	TB-2	54.6	55.5	Claystone - Tcs	74.1	21	5.7	Type II
	TB-4	53.1	53.7	Claystone - Tcs	37.8	21	11.9	Type III
	TB-5	61.0	61.5	Claystone - Tcs	26.3	21	12.7	Type III
	TB-5	71.4	72.0	Claystone - Tcs	5.0	21	13.2	Type III

Notes: Description of appearance of fragments retained in drum:
Type I - Retained pieces remain virtually unchanged
Type II - Retained materials consist of large and small fragments
Type III - Retained material is exclusively small fragments

4.2.2.3 Direct Shear Testing

Ten Samples were selected for direct shear testing, however one sample did not remain intact during transportation to the lab, and results of the remaining nine samples are presented in Table 4.9. Samples were selected from near the maximum section of the RCC dam (DB-1), the maximum section of the embankment dam (DB-5) and from approximate depths related to the outlet works and diversion tunnel in OB-1 to OB-3 and TB-1 to TB-4. The middle confining stress was found by estimating the in-situ stress based on overburden in the boring. High and low confining stresses were assigned based on the location of the sample. If the sample was beneath a proposed structure or reservoir (DB-1, DB-5, OB-2, TB-1, TB-2 and TB-3) the maximum load was calculated based on the RCC, earth or water that would be above the sample location after the dam would be built and the minimum load was calculated as the overburden pressure at the time of excavation. If the sample location is outside of the footprint of the dam (OB-1, OB-3, TB-4) the high confining stress used was about double the estimated in-situ stress and the low confining stress used was about 33% of the estimated in-situ stress. The results of the direct shear test are integral to designing the outlet works and construction diversion tunnel through bedrock including material properties selected for analysis.

Table 4.9
Phase 2 Rock Laboratory Testing Results – Direct Shear (ASTM D5607)

BOREHOLE	DEPTH (FT) FROM	DEPTH (FT) TO	LITHOLOGY	CONFINING STRESS (PSI)			PEAK FRICTION ANGLE (DEG.)	PEAK COHESION (PSI)
DB-1	48.73	49.0	Basalt - Tcb	14	70	280	22.2	10
DB-5	51.07	51.36	Basalt - Tcb	7	48	85	31.1	2.2
OB-1	43.2	43.6	Basalt - Tcb	14	40	85	33.9	1.8
OB-2	41.8	42.2	Basalt - Tcb	70	175	350	33.7	6
OB-3	46.8	47.2	Basalt - Tcb	70	175	350	36.1	5
TB-1	36.0	37.0	Basalt - Tcb	14	55	110	25.5	9
TB-2	87.07	87.33	Basalt - Tcb	70	175	350	28.1	12
TB-3	107.07	107.34	Basalt - Tcb	70	140	210	31.9	11
TB-4	37.0	37.4	Basalt - Tcb	14	40	85	31.6	4

4.2.3 Data Evaluation Process

Phase 2 Site Characterization data is derived from many different sources including borehole drilling, surficial and borehole geophysical data, geologic mapping, water pressure testing, laboratory testing, and engineering evaluation. To facilitate interpretation HDR has developed a logging process that combines all the data onto a single composite log. This log includes several columns across the top of the long side of an 11x17 page and depth along the short side. Multiple pages are required to accommodate the depth of each boring. An example of this log is shown on Figure 4.5 for borehole DB-1 from depths of 70 to 105 feet. The derivation of each column is described in the sections below. Composite logs for all boreholes, including the Phase 1 boreholes that were retroactively processed are included as Appendix E.

It should be noted that all exploration methods and data may not have been obtained in some or all portions of each boring. Hence, when data was not obtained, the columns are blank. The heading of the composite logs contains basic project information and all pertinent borehole information such as; coordinates, elevation, orientation, depths, drill rig/method, logger, checker and groundwater information. The footer contains reference information to facilitate understanding of the data presented in each column. The log is bounded on both the left and the right with a column depicting the elevation and associated depth as a reference. For boreholes TB-2 and TB-3 which are angled from horizontal the elevation to depth ratio is not one to one and therefore the elevation is calculated based on the borehole angle from horizontal. Several columns for depth are included throughout the log as a quick reference.

4.2.3.1 Materials, Graphic, Sample Interval and Laboratory Test Data

The material description is the first data column on the left side of the log and comes from the final field logs created by Shannon and Wilson Inc. The material descriptions represent a standardized field method of describing soil and rock based on the ASTM D2487 (Standard Practice for Classification of Soils for Engineering Purposes) and the International Society of Rock Mechanics (ISRM) suggested methods (Brown, 1981). The material descriptions are an attempt to capture as much detailed information about the entire rock unit encountered and contains such information as soil/rock type, grain size, color, strength, weathering, plasticity (soil), qualitative discontinuity descriptions (rock), and any other useful information. Also included at the end of each description in parentheses is the geologic formation that the unit likely belongs to. Material descriptions are important for gaining an overall understanding of the rock or soil as more details are presented in the columns to the right. The graphic column presents a standardized graphic representing the lithology changes with depth as quick visual reference. A dynamic lithology key is included in the footer which only shows the lithologies represented in that particular boring to save space.

The next column to the right is for the Sample Interval which depicts the type, sample number and depth extents of each sample taken; a sample key is included in the footer. <The most common sample type in soil is the Standard Penetration Test (SPT) as described in Section 3.4.1, the results of which are shown in the SPT/Recovery/RQD column discussed in the next section. Tube samples are reserved for cohesive soil when it is important to collect an undisturbed sample for laboratory testing to evaluate engineering properties. Since all the soil will be removed within the dam footprint, there was no need to take any tube samples during these investigations. Core run samples are the rock core obtained by the triple tube rock core method described in Section 3.4.1. Rock cores are placed in core boxes, described in the field then photographed and transported to a warehouse for storage and laboratory testing sample selection.

The laboratory test data column is a space to report the results of any testing performed at the depth the sample was taken from. Many different tests can be run on both soil and rock core samples, they are depicted by various symbols in the laboratory testing column and a test data key is included in the footer for reference.

4.2.3.2 SPT, Recovery and RQD

The Standard Penetration Test (SPT) as described in Section 3.4.1 was performed on five foot intervals within the overburden soils and the resulting N-values in blows/foot are presented in graphical form in this column. The scale of the column only goes to 100 because a test that reaches 50 blows/foot without penetrating 6 inches is considered refusal when having an N-value greater than 100. This is represented by a symbol on the right side of the column with a notation showing the penetration in inches after 50 blows. For example 50/2”, read as 50 blows over 2 inches.

At the soil rock interface the drilling method must change and therefore the data collection method shifts from the SPT to rock core Recovery and RQD. Recovery represents the portion (reported as a percent) of the total core run length (typically 5 feet) that remains in the triple tube barrel when extracted from the boring and opened by the geologist for logging. The Recovery is affected by the quality of rock and the skill of the driller. High recoveries are desirable to log as much of the subsurface as possible and 100% recovery is typical in competent rock. The percent Recovery is shown on the graphic column by a black 45 degree hatching.

Rock Quality Designation (RQD) was developed by Deere (1963) and is widely used in geological engineering as an approximate measure of the jointing or fracture of the rock mass and overall rock quality. RQD is defined as the percentage of the length of intact core pieces longer than 100 millimeter (mm; 4 inches) in the total length of recovered core. RQD is represented in the graphic column with red cross hatching and is always equal to or lower than Recovery. The value of RQD indicates the rock mass quality according to the following from Bienawski (1989):

- RQD (%) Rock Mass Quality
- <25 completely weathered rock
- 25-50 weathered rock
- 50-75 moderately weathered rock
- 75-90 hard rock
- 90-100 fresh rock

4.2.3.3 Rock Mass Rating (RMR)

The Rock Mass Rating (RMR) system was developed by Bieniawski in 1976 and updated in 1989. The system is designed to provide a rating system of the geo-mechanical properties of a rock mass according to such factors as strength, RQD, and spacing and condition of discontinuities. The strength index discussed in the next section is used to rate the strength of the intact rock blocks while RQD described above provides a rating for the overall rock mass based on frequency of jointing and fractures. The spacing of discontinuities also provides an indication of the frequency of jointing but provides a broader scope of classification as RQD is limited to the length of the core run and pieces larger than 100mm. Many factors are considered when rating the discontinuity condition including; persistence (discontinuity length), separation (aperture), surface roughness, infilling amount and type, and the weathering rating of the joint surface. In order to facilitate data reduction all these factors are given a rating over the length of a core run and added up for an RMR value that ranges from 0 to 100. RMR indicates the rock mass class over the length of the core run according to the following from Bienawski (1989):

- RMR Class No. Description
- <20 I Very Poor Rock

- 21-40 II Poor Rock
- 41-60 III Fair Rock
- 61-80 IV Good Rock
- 81-100 V Very Good Rock

Since there are typically several discontinuities present within a single core run and each discontinuity has its own characteristics the condition of discontinuities is averaged over the length of a core run to derive the discontinuity condition rating. The discontinuity details required to derive the RMR classification are not typically included in the borehole logging process. Therefore HDR engineers observed the rock core either on site or at the warehouse to log additional discontinuity information to derive the RMR classification. The data collected for each borehole was input into a spreadsheet to evaluate RMR with depth for each borehole, the spreadsheet data is provided for reference in Appendix G. A summary of the results by arbitrary depth intervals and rock types is shown in Table 4.10. Information in this table provides a sense of the variability of the rock with depth and between rock types. For detailed interpretation the RMR classification for each core run is shown on the composite logs in Appendix E.

As can be seen in Table 4.10, the RMR rating of the sedimentary rocks are consistently lower than the basalt rock ratings. This is due to the inherent differences in strength between the two rock types but also in large part to the greater number and poorer condition of discontinuities found in the claystone/siltstone including slickensided and smooth surfaces. RMR and rock quality in general typically increase with depth. Therefore when the average RMR value decreases significantly with lower depths such as between 125 and 150 feet for basalt/gabbro and 250 to 275 feet for claystone/siltstone more evaluation is necessary to fully characterize the rock quality.

Landslide borings (LSB) and quarry borings (QB) were excluded from RMR analysis as the landslide borings were terminated shortly after the rock contact was encountered and the quarries are located too far from the dam site to have relevant rock mass mechanical properties.

Table 4.10
Average RQD and RMR vs. Depth and by Rock Type

DEPTH (FT)		BASALT/GABBRO			CLAYSTONE/SILTSTONE		
FROM	TO	AVERAGE RQD (%)	AVERAGE RMR	NO. OF CORE RUNS	AVERAGE RQD (%)	AVERAGE RMR	NO. OF CORE RUNS
0	25	67	46	15	--	--	--
25	50	72	55	53	41	44	9
50	75	82	58	63	56	44	25
75	100	80	57	49	66	50	45
100	125	95	64	21	61	51	46
125	150	84	58	19	73	54	39
150	175	93	67	35	81	52	20
175	200	97	69	36	87	55	14
200	225	94	66	22	77	54	5
225	250	97	65	10	79	57	12
250	275	100	62	2	71	46	3
275	300	--	--	--	93	55	5
300	325	98	63	1	98	56	5
325	350	100	67	5	--	--	--

4.2.3.4 Strength and Weathering Indices

The strength and weathering indices follow the ISRM suggested methods (Brown, 1981) and are evaluated in the field by a geologist or geotechnical engineer during material identification and borehole logging. The strength index follows the criteria shown in Table 4.11 (also included in the footer of the composite log for reference) by assigning an index number based on a qualitative analysis of the rock core such as the ability to be scratched or broken by a rock hammer. The strength index is used to evaluate RMR as discussed above and provides an indication of the variability of intact rock strength with depth.

The weathering index is a classification of the rock mass as opposed to a single discontinuity surface as is used in RMR evaluation and follows the criteria shown in Table 4.12 (also included in the footer of the composite log for reference) by assigning an index number based on a visual analysis of the rock core such as the extent of discoloration, mineral alteration (oxidization) and deterioration of rock structure present on the surface and within discontinuities. The weathering index is used to qualitatively evaluate the variability of the competence of rock with depth and aid in defining the soil bedrock contact. Weathering processes require water therefore a highly weathered zone at depth can indicate a location of high groundwater flow for further evaluation.

Table 4.11

Strength Index Criteria

TERM	INDEX	APPROXIMATE UCS (PSI)
Very Weak	1	<700
Weak	2	700-3,600
Medium Strong	3	3,600-7,200
Strong	4	7,200-14,500
Very Strong	5	14,500-36,250
Extremely Strong	6	>36,250

Source: Brown, 1981

Table 4.12

Weathering Index Criteria

TERM	INDEX	DESCRIPTION
Fresh	1	No evidence of alteration
Slightly Weathered	2	Slight discoloration on surface
Moderately Weathered	3	Discoloring evident, alteration well below rock surface
Highly Weathered	4	Entire mass discolored
Completely Weathered	5	Rock is reduced to soil with relict rock texture

Source: Brown, 1981

Both strength index and weathering index were recorded during the field logging process and are also included on the field logs presented in the Data Report provided as Appendix A. The data from the field logs was directly imported into the composite log development process.

4.2.3.5 Discontinuity Data

The discontinuity data shown on the composite log was collected in the field during the core logging process and is also included on the field logs presented in the Data Report provided as Appendix A. The data from the field logs was directly imported into the composite log process. The data includes listing any infilling material observed, the angle of the discontinuity relative to horizontal and a symbol indicating a fracture, healed joint, rubble, gouge or core loss zone. A discontinuity data key is included in the footer of the composite log as a reference. Additional detailed discontinuity data was collected by HDR to evaluate RMR as discussed above however since this data is captured in the RMR value only the basic discontinuity data is presented on the composite log for visualization. Groundwater flows mostly through discontinuities in bedrock therefore the ability to visualize the frequency of discontinuities is important to pinpoint the locations of high hydraulic conductivity within a ten foot water pressure test interval as discussed in the next section.

4.2.3.6 Water Pressure Test Data

Water pressure tests to evaluate in-situ hydraulic conductivity were performed as described in Section 3.4.2 after each borehole was completed. The tests were performed at continuous intervals from the bottom of the borehole to the surface. The water pressure test field logs are provided with the Data Report in Appendix A. The data from the field logs was entered in to a spreadsheet for processing and analysis which is included as Appendix H.

One-hundred and fifty-seven tests have been attempted throughout a total of eighteen boreholes completed during both Phase 1 and 2 explorations. Twenty-two of the attempted tests failed due to the following factors:

- Could not achieve required test pressure(s)
- Packer seal failure; no discernible water takes as water was coming out of top of casing instead of going into the formation
- Water bypassing the packers through highly fractured host rock or incomplete packer seal

Table 4.13 contains a summary of the combined Phase 1 and 2 average estimated hydraulic conductivities by depth interval and lithology. Figure 4.6 shows the same data in a scatter plot so the range and variability of the test results can be seen. Both the claystone/siltstone, and basalt tend to have decreasing hydraulic conductivity with depth. A wide range and large variability occurs for both basalt and claystone/siltstone materials, especially among the bedrock found at depths ranging from about 50 to 150 feet.

Table 4.13
Average In-Situ Hydraulic Conductivity vs. Depth and by Rock Type

DEPTH		AVERAGE IN-SITU HYDRAULIC CONDUCTIVITIES (CENTIMETERS PER SECOND)			
FROM	TO	BASALT/GABBRO	NO. OF TESTS	CLAYSTONE/SILTSTONE	NO. OF TESTS
0	30	4.90E-04	3	--	--
30	60	1.24E-04	30	1.68E-04	7
60	90	5.11E-05	33	4.64E-05	17
90	120	7.49E-05	16	5.35E-05	24
120	150	2.53E-05	10	4.69E-05	20
150	180	9.73E-05	10	1.19E-05	7
180	210	5.31E-05	8	2.10E-05	2
210	240	1.20E-05	2	8.18E-07	1
240	260	1.77E-05	3	--	--
Overall Average		8.51E-05	115	5.53E-05	78

Lugeon values (Lu) are commonly used when evaluating groutability of rock formations and to calculated hydraulic conductivity values. Lu values are empirically defined as the hydraulic conductivity required to achieve a flow rate of 1 liter/minute per meter of test interval under a reference water pressure equal to 1 megapascal (Houlsby 1976). Ewert (2003) used a comparative analysis of data from a number of projects, together with lab test data, and presented the following generalized correlations of Lu values and groutability. These "groutability" ranges are shown in Figure 4.6:

- Small Lu values (<2 to 5) usually indicates ungroutable rock.
- Moderate Lu values (5 to 10) usually indicate poor groutability.
- Large Lu values (>10) may indicate a groutable rock. However, joint frequency and fissure widths must be evaluated.

Weaver and Bruce (2007) provide commentary related to the Ewert groutability scale. Specifically, these values may correlate to "old" grouting technology and are not wholly valid for modern technology where it may be possible to grout to a 1-Lu closure standard, see Figure 4.6. Additional discussion related to groutability is provided in Section 5.2.3.

4.2.3.7 Televiewer Data

Optical and Acoustic televiewer data was obtained in most boreholes as described in Section 3.4.3.2. The results of the televiewer data are shown on the composite logs under the following columns:

- **Unwrapped Televiewer Image:** The walls of the boreholes unwrapped to a flat surface. The 360-degree unwrapped image begins with the zero degree representing magnetic north (azimuth)
- **3D Core View:** The data transformed (inverted) and plotted to represent an equivalent rock core sample for comparison with the actual core samples and detailed log of the borehole. A plane corresponding to the joint orientation is drawn through each identified joint.
- **Structure:** Joint structure data is presented as azimuth, dip and structure type on a "tadpole" plot. The azimuth is indicated by the orientation of the tick on each tadpole with up being 0/360 degrees. The dip is indicated by the x-axis location on the graphic column ranging from 0 (horizontal) to 90 (vertical) degrees. The type of joint is indicated by the color of the tadpole as shown in Table 4.14 which is also included in the footer of the composite log for reference.

The televiewer data is analyzed to identify rock type, joint structure, and other defects and characteristics that may influence excavation methods, stability, treatment requirements, and the overall excavation objective.

Review of the televiewer data and other structural characteristics indicates a wide range of joint conditions and orientations. The vast majority of fractures were type 2 in both basalt and claystone/siltstone with type 4 the next most prevalent but mostly in basalt. Type 1 fractures were relatively rare in both rock types and Type 3 was rare in the claystone/siltstone. A Type 5 feature was rare in the basalt as would be expected since any bedding feature in the basalt would indicate a boundary between basalt flows. Additional results of the structure data is discussed in Section 4.2.4 below along with the outcrop structure data obtained during field mapping.

Table 4.14
Rock Structure Scale

SCALE	TYPE	COLOR	NUMBER OF IDENTIFIED FRACTURES	
			BASALT/GABBRO	CLAYSTONE/SILTSTON
0	No joint/fracture	NA	--	--
1	Major open joint/fracture	Red	50	49
2	Minor open joint/fracture	Pink	523	410
3	Partially open joint/fracture	Orange	122	45
4	Sealed joint/fracture	Gray	248	88
5	Bedding	Green	43	80
		No Data	147	8
		Totals:	1,133	680

4.2.3.8 Sonic Suspension Logging

Sonic suspension logging was performed in the same boreholes as the televiewer as described in Section 3.4.3.3 with the exceptions of TB-2 and TB-3 because the method is not valid in angled boreholes. The results are shown in the Sonic Suspension Log Velocity column on the composite logs with the graphical x-axis representing a range of velocity ranging from 0 to 6,000 feet/second (ft/s).

The compressional (P) and shear (S) wave velocities are used to correlate the stratigraphy found in the boreholes with the surface seismic refraction tomography survey results and assist with characterizing the rock mass properties and the subsurface lithology. The shear wave velocities ranged from about 800 to 2,900 ft/s and the compression wave velocities ranged from about 1,500 to 5,600 ft/s. A relative change in both shear and compression wave velocities indicate a lithological transition with P and S wave velocities corresponding to those shown in Table 4.15 below.

Table 4.15
Weathering Index Criteria

LITHOLOGY P-WAVE VELOCITY	(FT/S) S-WAVE VELOCITY	(FT/S)
Basalt/Gabbro	4,800 – 5,600	2,000 – 2,900
Siltstone/Claystone	1,500 – 2,500	800 – 1,500

Source: Brown, 1981

In addition to identifying lithology transitions, weak or highly fractured zones in either rock type that warrant further evaluation are identified by a relative dip in both P and S wave velocities within a lithology.

4.2.3.9 Piezometer/Groundwater

The last column on the right of the composite log with the exception of depth and elevation is a graphic depicting the materials used to construct a piezometer after all data has been collected and testing completed and in LSB holes after inclinometer installation. In addition to the piezometer construction this column also contains water level indicators either during or after drilling with associated dates of the reading. A key of piezometer graphics is included in the footer of the composite log as a reference.

4.2.4 Dam Site Bedrock Structure

During the Phase 1 Site Characterization four outcrop locations were found for mapping of joint sets. These outcrop locations are indicated on Figure 2.3. All of the outcrops consisted of Crescent Formation basalt and representative examples of the outcrops are shown on Photographs 4.1 and 4.2 below. Anywhere a plane of a joint was exposed, a strike and dip of the plane was measured using a Brunton compass. Joint strike measurements were converted to the equivalent dip direction and a total of 44 joint orientations were imported into the Dips 6.0 software program from Rocscience (2014) to evaluate joint sets. Two prominent joint sets (JS)-1 and JS-2 were identified in the surface outcrop areas and the strikes, dip direction, and dip for each joint set are provided in Table 4.16. Set JS-1 was observed to have joint spacing of about 4 feet and JS-2 was observed to coincide with the slope of the outcrop.

A downhole televiewer was used in most borings as described in Section 3.4.3.2 and 4.2.3.7. The televiewer joint data was entered into Dips 6.0 the same way as the outcrop data for evaluation. The stereonet was initially kept separate so that the 44 outcrop data points do not get lost amongst the 462 data points from the televiewer surveys. Only discontinuity types that correspond with major joints, minor joints and bedding planes were analyzed. Partially open joints, sealed joints and contacts were removed from the data set since they are less likely to initiate a failure. Two prominent joint sets; JS-3 and JS-4 were identified in addition to the previously identified joint sets and the strikes, dip direction, and dip for these joint sets are also shown in Table 4.16.

The data sets were then combined using JS-1 and JS-2 from the outcrop data to inform selection of the 4 major joint sets so that JS-1 and JS-2 were accounted for amongst a larger data set. Outcrop data measured in the field is reliable despite the lower number of data points compared to the televiewer data and should not be ignored. The stereonet of combined data sets and identified joint sets is shown on Figure 4.7. Contours of density concentrations shown on Figure 4.7 represent the dip poles that are oriented 90 degrees to the location where the dip vectors intersect the lower hemisphere of the stereonet. This presentation aids in the visualization of the different joint sets. Great circles representing the average joint set dip direction and inclination from horizontal are also shown on this figure relative to the orientation of the dam axis.

The bedrock structure data from both Phase 1 and 2 was used to perform a kinematic analysis using the methods described by Goodman (1980) to evaluate the likelihood of unstable blocks during excavation of the foundation. The results of the kinematic analysis are presented in Section 5.1.

Table 4.16
Summary of Identified Joint Sets from Outcrop Mapping and Televiewer Data

JOINT SET	STRIKE	DIP DIRECTION	DIP	DATA SOURCE
JS-1	63	153	72	Outcrop Mapping
JS-2	0	90	74	
JS-3	343	73	21	Televiewer
JS-4	133	223	55	

Photograph 4.1
Outcrop 2, Along the Western Bank of the Chehalis River Within the Dam Footprint

Photograph 4.2
Outcrop 3, Upstream of the Dam and on the Eastern Bank of the Chehalis River

4.2.5 Seismic Refraction Tomography Results

Seismic refraction tomography (SRT) survey lines were completed by Global Geophysics as discussed in Section 3.4.3.1, and the results are presented in Appendix D. The data was processed to show estimated colored contours of compression wave velocity with depth beneath the ground surface. A total of seven SRT survey lines were completed in the vicinity of the dam; four of which were run across known or potential landslides and are discussed in the Section 4.2.6. The remaining three SRT surveys were completed along potential dam features. Dam Seismic Line 1 (DSL-1) was run along the lower right abutment access road and coincides approximately with the location of the water quality outlet works for the potential FRFA Dam option. Dam Seismic Line 2 (DSL-2) was run across the older alluvial terrace to the west of the river channel and approximately coincides with an alternative flood control outlet works or construction diversion channel for the FRO Dam configuration. Dam Seismic Line 3 (DSL-3) was run perpendicular to the river channel and approximately coincides with the location of the potential spillway flip bucket, and fish passage/flood control outlet stilling basin for both dam options. As an example of how the SRT survey data was used (as discussed further below) Figure 4.9 shows the SRT compression wave velocity contour profile for DSL-1 along with borehole data, interpreted top of rock and excavation objective. The SRT profiles were also used to interpret sections corresponding to DSL-2 (Figure 4.3), DSL-3 (Figure 4.2) and the LSSL's (Figure 4.4); however, they were removed from the sections for clarity but are provided with the geophysical report in Appendix D.

SRT surveys are a valuable tool for evaluating the depth of overburden and weak rock between boreholes where direct observations cannot be made; however, the value of SRT surveys is only as good as the correlations that can be made with the overburden and weak rock depths in borehole locations that the SRT survey passes through. Due to limited access many boreholes were moved off the SRT survey lines and had to be projected, increasing the uncertainty of the correlation. Additionally many boreholes that coincided with the SRT surveys showed either anomalously high or low compression wave velocities at the depth shown to be the soil rock contact in the borehole logs. Despite these challenges the data was reduced and a representative compression wave velocity was established for depth of both overburden soil and weak (rippable) rock.

4.2.5.1 Soil/Rock Contact Correlation

Table 4.17 presents information from each seismic line and borehole completed during Phase 1 and 2. Data includes the estimated projection distance of each borehole, the soil depth and the compression wave velocity at the soil depth taken from the SRT survey. The velocities at the bottom of the estimated soil depth varied greatly, ranging from 2,400 to 11,700 ft/s creating some significant uncertainty in the data at a number of locations. Furthermore the velocities at soil depth for projected values were relatively low. Six data points remain when high/low and projected velocities are removed from the average giving a reasonable average compression wave velocity at the base of the overburden (soil depth) of 6,200 ft/s. This compression wave velocity was used to estimate the soil/rock contact shown on Figures 4.1 to 4.4 as the top of rock where borehole data doesn't exist. Considerable geologic judgment had to be used where the SRT survey velocities did not match the borehole data and the borehole data always took precedence in subsurface interpretation and geologic model development.

4.2.5.2 Weaker (Rippable) Rock and Depth to Suitable Rock for Dam Foundation

To achieve proper contact between an RCC dam and bedrock foundation, weaker rock will need to be excavated during construction. From both a design and cost estimating perspective, it is critical to evaluate the volume of weaker rock to be removed. The extent of weaker rock will form the estimated excavation surface for the dam and related hydraulic structures. Several factors went into creating the excavation objective that is shown on Figures 4.1 to 4.3. Specifically, at each borehole location the following criteria (attributes) were used to evaluate the depth to suitable rock, and the corresponding depth of required weaker rock excavation:

- Increasing RQD (>50)
- Low weathering index
- RMR of about 55 or higher
- Presence of Basalt
- Engineering/Geologic judgment

Table 4.17

Overburden Soil Depth Correlation with SRT Compression Wave Velocity

PHASE	SEISMIC LINE	BOREHOLE	BOREHOLE PROJECTION DISTANCE (FT)	ESTIMATED SOIL DEPTH (FT)	ESTIMATED COMPRESSION WAVE VELOCITY (FT/S)
1	SL-1A	DB-1	-	30	6300
		OB-1	-	29.5	6600
		BH-2	21	30	5700
	SL-1B	DB-6	-	37	6600
		DB-7	40	37	5700
	SL-2	-	-	-	-
	SL-3	BH-1	38	35	6600
		BH-4	11	62.5	11700[1]
	SL-4	BH-1	27	35	8400[1]
	SL-5	BH-3	29	35	8700[1]
		BH-4	-	62.5	7500
		LSB-1	-	32.5	9750[1]
2	DSL-1	TB-1	-	25	5100
		TB-4	-	10	2400[1]
		TB-5	-	60	11100[1]
	DSL-2	OB-1	25	29.5	4800
		OB-2	-	38	5100
		OB-3	23	34.6	4800
	DSL-3	LSB-2	46	40.8	5550
	LSSL-6	LSB-1	36	32.5	11700[1]
		BH-4	13	62.5	5100
	LSSL-7	LSB-4	30	57	5100
	LSSL-8	-	-	-	-
	LSSL-9	TB-5	54	60	5850
				Minimum:	2400
				Maximum:	11700
				Average[2]:	6200

Notes:
1. Anomalously high or low values

2. Average does not include projected, high or low values

Once the depth to suitable rock was found at each borehole location, the depths were compared to the SRT compression wave velocities where they crossed the boreholes in a process similar to that used to estimate the depth of overburden soil correlation. Based on this evaluation, an average compression wave velocity of 9,000 ft/s was established as the location where suitable dam foundation rock could be found between boreholes. This criteria was then applied to the area beneath the entire footprint of the RCC dam to establish an excavation objective (surface).

By establishing a compression wave velocity correlating to the approximate base of the overburden soil and top of suitable rock, localized zones of weak or highly fractured rock that will require dental concrete, over-excavation or other foundation treatment methods can also be identified. The overall average overburden soil thickness is about 35 feet with the exception of the colluvium on the slopes of the right abutment, which is considerably thinner. The thickness of the upper weaker rock above the excavation objective to the top of suitable rock has an average thickness of about 15 feet below the overburden soil. Considerable variation of these surfaces occur within the potential dam footprint. Additional discussion of locations where over-excavation or other treatments may be required are described in Section 5.2.1. The general depth of excavation is expected to be up to about 50 feet and locally deeper where significant lower-quality rock is found.

4.2.6 Dam Site Landslides

There are four landslides (3a, 1, 1a, and 26) in the immediate vicinity of the dam as shown on Figure 2.3 that may pose potential hazards during construction and long term operation of the dam and reservoir. These landslides must be considered during the design process. The feature being investigated by LSSL-8 has subsequently been identified as an erosion feature and not a landslide therefore it is not discussed in this section. The Chehalis Dam Landslide Evaluation Report prepared by Shannon and Wilson (Landslide Evaluation), included as Appendix B, used finite element analyses to evaluate 10 landslides individually including the dam site landslides described below for the factor of safety (FS) against failure for various conditions. These conditions include the initial FS before drawdown and the minimum FS during drawdown for both the design drawdown rate and a slower, modified drawdown rate to the natural pool (628 feet for the FRFA and no pool for the FRO) after a 100 year flood event. When the landslides were very large such as landslide 1 both the upper and lower slopes were evaluated separately which could in turn lead to a whole slope failure. Effective stress friction angle was varied for both the FRO and FRFA configurations for all landslides, and a parametric analysis varying cohesion intercept shear strength and saturated horizontal hydraulic conductivity was run on the FRFA configuration of landslide 3 to evaluate the sensitivity of the models. The results of this analysis are summarized below for each of the dam vicinity landslides, and the hazards of the reservoir landslides are summarized Section 5.3, refer to Appendix B for details of the analysis. The results of the parametric analysis run on landslide 3 shows that the models are highly sensitive to variances in cohesion intercept shear strength and saturated horizontal hydraulic conductivity; indicating that more in-situ data to evaluate the actual soil parameters is needed to produce more precise model results.

Landslide 3a is about 450 feet upstream of the dam on the right bank of the Chehalis River and is shown in cross section 8 in Figure 4.4. Landslide seismic line 7 (LSSL-7) was completed across this landslide and LSB-4 was drilled through the upper portion to evaluate the depth of the slide plane. The soil depth in LSB-4 is 30 feet. This depth correlates well with the top of rock compression wave velocity established in the previous section of 6,200 ft/s. There is an anomalous zone between stations 8+00 and about 8+60 of the LSSL-7 data that show a reduction in compression wave velocity which is reflected in the interpreted top of rock. This landslide poses a risk during both construction and during reservoir operations as it is located directly above the diversion tunnel intake and construction coffer dam. If failure occurs, it could block the diversion intake causing the construction site to be flooded. Also the landslide complex could be frequently inundated during operation of the FRO configuration, and is entirely below the proposed operating pool of the FRFA RCC dam option at elevation 628 feet. It presents a risk of failure during reservoir filling/operation due to significant changes in the water table that could reactivate the landslide depositing debris on the upstream surface of the dam and possibly clogging fish passage and water quality outlet works. The results of the Landslide Evaluation for landslide 3a are shown in Table 4.18. The results indicate that the landslide will reactivate before drawdown during a 100 year flood event for all friction angles below 35 degrees for the FRO configuration and below 30 degrees for the FRFA configuration. If the landslide remains stable before drawdown it will fail in all situations during both the designed and modified drawdown.

Table 4.18
Global Stability Analysis Summary of Landslide 3a

DAM TYPE	EFFECTIVE STRESS FRICTION ANGLE, Φ' (DEGREES)	FACTOR OF SAFETY (FS) AGAINST GLOBAL INSTABILITY [RED = FS<1]		
		BEFORE DRAWDOWN	MINIMUM DURING DESIGN DRAWDOWN	MINIMUM DURING MODIFIED DRAWDOWN
FRO	25	0.78	0.18	0.51
	30	0.97	0.22	0.63
	35	1.18	0.26	0.77
FRFA	25	0.96	0.24	0.61
	30	1.19	0.29	0.75
	35	1.44	0.36	0.91

Landslide 1 is just upstream of the dam in the left abutment as shown on Figure 2.3 and has been assessed with two SRT survey lines (SL-5 and LSSL-6) and three boreholes (BH-3, BH-4 and LSB-1) over the course of the Phase 1 and Phase 2 investigations. It is shown in cross section 7 on Figure 4.4. Similar to 3a, the lower portion of this landslide could be frequently inundated during operation of the FRO configuration, and most of this landslide is also below the FRFA RCC dam operating pool of elevation 628 feet. This landslide has the possibility of reactivating when the reservoir is filled and/or operated. As with landslide 3a this landslide poses a risk to the alternative outlet works if reactivated, although the risk of this is reduced by the large flat alluvial terrace between the toe of the landslide and the river which currently buttresses the slide. Landslide 1 was evaluated for failure of the upper slope and lower slope separately due to its size. The results of the Landslide Evaluation are shown in Table 4.19. No failure occurs at any friction angle for either slope and either configuration before drawdown occurs during a 100 year event. However; during the design drawdown rate only the lower slope fails for the FRO configuration but both slopes fail for the FRFA configuration. With the modified drawdown rate only the lower slope fails for both configurations and all FS are improved suggesting that an even slower drawdown rate could prevent failure of the lower slope. Shannon & Wilson noted in the results that the failure plane was fairly small and is focused near the middle of the slope, just downhill of BH-4 where the topography becomes more steep. Given the large area downslope to allow for run-out of a small failure surface the risk to structures are greatly reduced, assuming the failure does not trigger failures up-slope at a larger scale.

Table 4.19
Global Stability Analysis Summary of Landslide 1

DAM TYPE	EFFICTIVE STRESS FRICTION ANGLE, Φ' (DEGREES)	FACTOR OF SAFETY (FS) AGAINST GLOBAL INSTABILITY [RED = FS<1]		
		BEFORE DRAWDOWN	MINIMUM DURING DESIGN DRAWDOWN	MINIMUM DURING MODIFIED DRAWDOWN
FRO (upper slope)	25	1.72	1.18	1.3
	30	2.12	1.46	1.61
	35	2.58	1.77	1.95
FRO (lower slope)	25	1.16	0.16	0.63
	30	1.44	0.23	0.77
	35	1.74	0.27	0.94
FRFA (upper slope)	25	1.73	0.78	1.41
	30	2.15	0.97	1.74
	35	2.60	1.18	2.11
FRFA (lower slope)	25	1.19	0.25	0.72
	30	1.50	0.31	0.89
	35	1.77	0.37	1.08

Landslide 1a is in the downstream left abutment area of the dam axis and partially falls within the dam footprint as shown on Figure 2.3. Since this landslide does not fall within the reservoir, a drawdown analysis was not performed in the Landslide Evaluation. However; a steady state seepage analysis was performed to evaluate the in-situ instability. This evaluation shows a FS less than one in the current state for angles of friction of at least 30 degrees or less. This landslide sits up slope from the proposed spillway flip bucket and fish passage stilling basin for both configurations and must be considered during the planning process.

Landslide 26 is located downstream of the right abutment of the dam as shown on Figure 2.3. It was not identified prior to the Phase 1 study and therefore added to the Phase 2 investigation due to its location and possible influence on the fish passage stilling basin, lower end of the construction diversion tunnel, and other possible fish passage facilities located in the downstream right abutment area for all configurations. During the Phase 2 investigation LSSL-9 was completed along the central axis of the landslide and DSL-1 was completed across the middle, perpendicular to LSSL-9. Borehole TB-5 was completed through the central portion of the mapped slide limits. The landslide is shown in cross section 10 on Figure 4.4 along with proposed dam features. As with landslide 1a this landslide is not in the reservoir therefore only a steady state seepage analysis was performed to estimate the FS and assess potential instability. The Landslide Evaluation reports FS above 1 and thus stable for all friction angles of 25 degrees and above. However; the excavation for the fish passage stilling basin will remove most of the toe of landslide posing a risk of instability for the reaming portion up slope of the excavation. Further evaluation of this landslide will be needed during preliminary design.

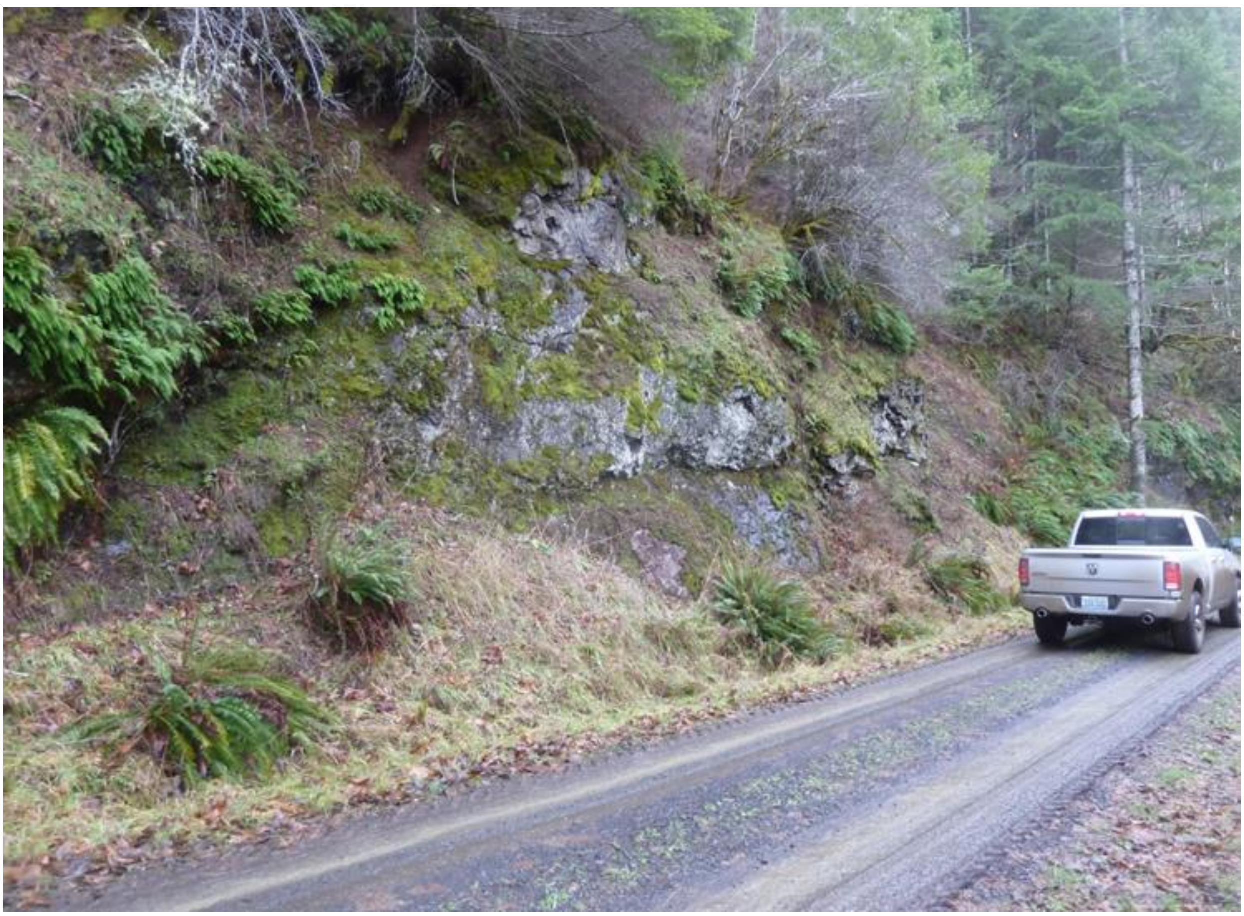
A detailed evaluation of all landslides, including those discussed above has been prepared by Shannon and Wilson Inc. and is included as Appendix B.

4.3 RCC Aggregate Investigation Results

The Rock Creek quarry site was the only location with acceptable aggregate quality results from the Phase 1 RCC aggregate investigation. It is over 8 miles from the dam site so additional potential quarry sites were identified to be investigated during the Phase 2 program. Preliminary reconnaissance of outcrops in the dam and reservoir vicinity on land owned by Weyerhaeuser was performed in the fall of 2015. Twenty potential sites were identified including nine within the reservoir and 11 outside of the reservoir. The Rock Creek quarry site was included in this evaluation. Based on visual inspection of the outcrop rock quality at each location, haul distance, and topographic conditions to yield the required quarry size the list was narrowed to seven potential sites. Two more sites were eliminated upon further inspection of the surrounding rock which turned out to be poor quality and cast doubt on the possibility of producing the required aggregate quantities. Of the five remaining sites two were moved to future phase investigations as backup locations due to limited access and environmental permitting difficulties given the proximity to the Chehalis River. The three sites that remained are the subject of the Phase 2 RCC aggregate investigation program.

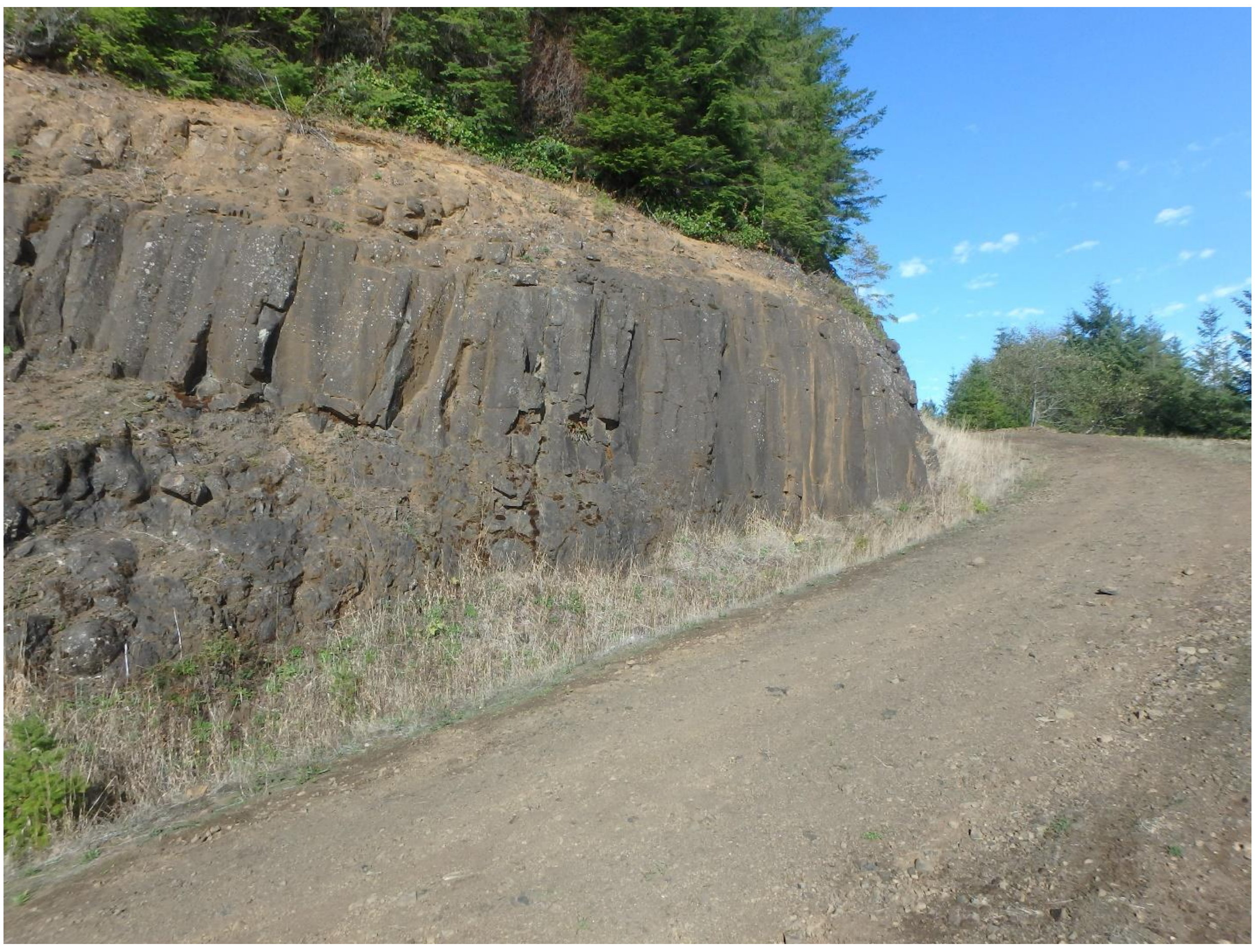
Quarry 1, shown on Figure 4.10, is located within the reservoir about 2 miles southeast of the dam site. It consists of a prominent NE-SW trending ridge of basalt at elevation 825 feet with outcrop along a haul road that leads up the side of the ridge with an old logging road along the top for easy drilling access. Photo 4.3 shows the rock outcrop along the haul road. The existing haul road may have to be re-located to accommodate the size of the quarry necessary for either the FRO or the FRFA RCC dam configurations. QB-1 was advanced on the ridge at about elevation 775 feet at the end of the old logging road and encountered a somewhat unexpected 35 feet of soil overlying solid Crescent formation basalt to a depth of 114.5 feet giving an estimated mineable thickness of about 80 feet as shown on the composite log found in Appendix E. Beneath the solid basalt was 4 feet of siltstone that would be unacceptable as aggregate material and beneath that to the bottom of the borehole at 150 feet depth was 31 feet of Breccia. The breccia at this location was a mix of basalt and claystone clasts ranging widely in size from 0.1 to 1.6 feet as opposed to the breccia encountered at the dam site where basalt clasts were found in a clear claystone matrix. Two representative sample intervals were selected for lab testing in the basalt from QB-1 and one interval was selected in the breccia to evaluate the suitability as aggregate. The results of the lab testing are discussed in Section 4.3.1 below. A single SRT survey line was completed along the crest of the ridge and passing through QB-1 for correlation and is discussed below in section 4.3.2.

Photograph 4.3
Basalt Outcrop Along Haul Road Route Bisecting Quarry 1



Quarry 2, shown on Figure 4.11, is located about 7 miles southwest of the dam site. It consists of a less prominent basalt ridge than Quarry 1 trending E-W at an elevation of 2,575 feet, with a steep southern face and easy access from the west on an old logging road off of a main haul road that bounds the north side of the site. Another haul road, where Photo 4.4 was taken from, bounds the east and southern boundaries of the site. The topography at this site provides plenty of space for the required size quarry without moving any haul roads and the current haul roads are positioned well for easy access. QB-2 was advanced about 10 feet below the ridge crest at about elevation 2,565 feet along the old logging road and encountered no soil overlying. Relatively high quality Crescent formation basalt was found to a depth 94 feet as shown on the composite log found in Appendix E. Beneath the quality basalt to a depth of 131.5 feet was about a 37 foot thick sequence of alternating claystone/siltstone with basalt and basalt breccia in a claystone matrix similar to the breccia encountered at the dam site. From 131.5 feet to the end of the boring at 150 feet was competent basalt. Two representative sample intervals were selected for lab testing in the basalt from QB-2 a. The results of the lab testing are discussed in Section 4.3.1 below. A single SRT survey line was completed from the top of the ridge along the old logging road to the northwest near QB-2 for correlation and discussed below in section 4.3.2.

Photograph 4.4
Rock Outcrop Along Eastern Boundary of Quarry 2



Quarry 3 (Rock Creek), shown in photo 4.5, is located about 8 miles west of the dam site down the A-line road off highway 6. It consists of about a 30 to 50-foot high rock face forming a rough semi-circle with a large flat access area. Rock Creek spills over the exposed rock face of this existing but inactive quarry and is shown in the photograph. The creek continues north through the site along the A-line road. To avoid the creek Quarry 3 would advance the existing rock face west into a N-S trending ridge that tops out at elevation 1,175 feet. An old logging road exists for easy access to the ridge from the south for future drilling. No borings or SRT survey lines were completed at the Rock Creek site because samples from the rock face were readily available for lab testing and the overburden soil above the face can be seen as minimal.

Photograph 4.5
Rock Creek Spilling over the Old Quarry Face at the Site of Quarry 3

4.3.1 Laboratory Testing Results

The results of the Phase 2 quarry laboratory testing program are summarized in Table 4.18. Quarry 2 and Rock Creek showed the most promising lab results with Rock Creek being slightly better having the same average 16 day ASR values as Quarry 2 but higher specific gravity and significantly lower absorption. Quarry 1 had acceptable but lower quality result. The breccia from a depth of 127 to 140 feet would be unsuitable for concrete aggregate materials. The two sample intervals of basalt from Quarry 1, showed acceptable ASR result. However, these materials would have significantly higher absorption rates, and lower specific gravity than either Quarry 2 or Rock Creek. The compressive strengths of the two basalt sample intervals from Quarry 1 are significantly lower than the basalt sampled at Quarry 2.

Table 4.20

Phase 2 Aggregate Source Test Results

QUARRY NAME	HAUL DISTANCE[1] (MILES)	SAMPLE DEPTH (FT) FROM	TO	UCS (PSI)	ABSORPTION	BULK SPECIFIC GRAVITY (SSD)	LA ABRASION	ASR (16-DAY)
Quarry 1	2	38	50	5,763	6.46	2.60	27.1	0.08
		84	95	1,834	4.69	2.65	26.8	0.076
		127	140	5,237	8.26	2.49	27.5	0.124
Quarry 2	7	15	27	13,2102	4.04	2.69	24.8	0.034
		45	55	9,6553	3.72	2.71	24.1	0.036
Quarry 3 (Rock Creek)	8.3	NA	NA	NA	1.37	2.74	18.9	0.035
WSDOT4/FHWA5 Criteria	--	--	--		3 max.	2.55 min.	35 max.	See Note 6

Notes:
1. Haul distance in miles measured with Google Earth
2. UCS sample depth was 6.0 to 6.8 ft
3. UCS sample depth was 71.1 to 72.2 ft
4. WSDOT = Washington State Department of Transportation
5. FHWA = Federal Highway Administration
6. For ASR (16-day test), a test value of 0 to 0.10 is innocuous, 0.11 to 0.20 is acceptable if supplemental testing confirms expansion is not due to ASR, and greater than 0.20 requires additional testing.
5. SSD = saturated surface dry

Based on these results and recent advances in mix designs to mitigate ASR concerns, all three potential aggregate quarries appear to be good sources of acceptable RCC aggregate materials. One year ASR tests will be received in the spring of 2017. Additional investigations, characterization and possible test quarry and mix design studies should be considered as part of the next phase of study.

4.3.2 Seismic Refraction Tomography

The SRT surveys completed at Quarry 1 (QSL-1) and Quarry 2 (QSL-2) are shown on Figures 4.10 and 4.11 respectively. The SRT compression wave velocity representing the soil and bedrock contact as discussed in Section 4.2.5 is 6,200 ft/s which corresponds to the transition from blue to green on the QSL-1 profile. The depth of the soil/bedrock contact as shown by QSL-1 is 85 feet whereas the QB-1 boring shows a depth of 35 feet to this interface. This discrepancy between the boring and seismic refraction tomography creates uncertainty at Quarry 1 and a need for further evaluation.

The SRT compression wave velocity shown on the profile of QSL-2 correlates well with the QB-2 boring data with rock at the surface. The bedrock from station 0+00 to 3+50 appears to be high velocity/competent rock very near the ground surface. The overburden soil depth likely increases significantly up-station to the northwest. However the SRT survey data shown on QSL-2 and the laboratory results from QB-2 show significant promise for large quantities of quality aggregate at the Quarry 2 site.

5 INTERPRETATION OF SITE CHARACTERIZATION RESULTS

5.1 Kinematic Analysis

A kinematic analysis was performed on joint data from both the dam site outcrops and the televiewer structure data by plotting the structure data on a stereo net using Dips 6.0 (Rocscience, 2014) as described in Section 4.2.4. Joint set data was used to perform two types of analysis to assist in the design process. The first is an analysis methodology described by Goodman (1980) which evaluates the potential for localized slope failures by three different mechanisms; toppling failure, planar sliding and wedge sliding. These mechanisms would be associated with excavation slopes along the right and left abutments of the proposed dam excavation. The second analysis pertains to the diversion tunnel alignment and indicates the factor of safety for potentially moveable blocks within the tunnel excavation along the ceiling or walls of the tunnel. Such moveable blocks may require stabilization and/or appropriately designed tunnel lining systems. The results of these analyses are presented in the following sections. A detailed calculation package for the Abutment Excavation Analysis is provided in Appendix I.

5.1.1 Abutment Excavation Analysis

Seven different excavated slope orientations in the bedrock were selected for evaluation including the bottom of each abutment excavation, and the downstream, upstream and the ends of each abutment excavation. Typically, each of the excavated slopes in rock are covered to some overburden soil before the excavation daylights to the ground surface. The four major joint sets that have been identified as described in Section 4.2.4 and shown in Table 4.16 were considered in the evaluation. The Goodman (1980) method of kinematic analysis considers three failure modes; planar sliding, wedge sliding and toppling failure by which blocks are created in an excavation slope. Using the four identified major joint sets, a friction angle of 25 degrees and the seven excavation slope orientations a critical zone for each failure type is identified. A risk rating was created based on the percentage of data points that fall within the critical zone and applied to each excavation slope for each failure type. In some cases the percentage of data points falling within the critical zone for a particular joint set was higher than the overall percentage of data points falling within the critical zone. In these cases the particular joint set controls the risk rating. The risk rating results are shown in Table 5.1 with controlling joint sets shown in parentheses for each excavation slope and potential failure mode.

The risk rating results for each failure mechanism are summarized below:

- **Planar Sliding:** The highest risk for planar sliding is along JS-4 in the right abutment excavation end. There is also moderate risk in the left abutment downstream slope and the right abutment upstream slope.
- **Wedge Sliding:** There is only a moderate risk of wedge sliding in the left abutment downstream slope and the right abutment upstream slope.
- **Toppling Failure:** There is a very high risk of a toppling failure in the right abutment downstream slope and the left abutment upstream slope. There is also a moderate risk of toppling failure at the end excavation slopes for both abutments.

Risk ratings that are above moderate are only present in the rock excavation slopes (0.5H:1V) that tie the bottom of the excavation to the soil excavation slope (1.5H:1V). Therefore it may be necessary to change the excavation slope specification to something less steep the 0.5H:1V to reduce the risk of blocks coming detached from the bedrock during excavation. The intent of this analysis is to identify locations were the risk was moderate or higher for further evaluation. Due to a vast majority of the boreholes from which the televiewer data was derived being drilled vertically there is some directional bias to the data that may skew the results. Additional angled boreholes should be performed during preliminary design to refine the data set and verify these results. Also further strength analysis along discontinuities including an evaluation of degree of weathering within the excavation zone should be performed to refine friction angle inputs that would improve accuracy of the analysis.

Table 5.1
Results of Abutment Excavation Analysis

ABUTMENT	RISK RATING EXCAVATION BOTTOM	EXCAVATION DOWNSTREAM SLOPE	EXCAVATION UPSTREAM SLOPE	EXCAVATION END
Lower Left Abutment	Planar Sliding: NCZ Wedge Sliding: NCZ Toppling: NCZ	Planar Sliding: M (JS-3) Wedge Sliding: M Toppling: VL	Planar Sliding: L (JS-3) Wedge Sliding: L Toppling: VH (JS-1)3	NA
Upper Left Abutment	Planar Sliding: VL Wedge Sliding: VL Toppling: VL			Planar Sliding: M (JS-3) Wedge Sliding: L Toppling: M (JS-4)
Right Abutment	Planar Sliding: L (JS-4) Wedge Sliding: VL Toppling: L (JS-2)			Planar Sliding: H (JS-4) Wedge Sliding: L Toppling: M (JS-2)

Notes:
Risk Rating (Percent of data points within the Critical Zone)

Very Low:	VL	0-12.5%
Low:	L	12.5-25%
Moderate:	M	25-50%
High:	H	50-75%
Very High:	VH	75-100%

NCZ: No Critical Zone Exists
1. Joint Set listed when the percentage of data points within the critical zone for a particular joint set was higher than the percentage of all data points within the critical zone
2. Bold format used when risk rating is moderate or higher
3. JS-2 has a moderate risk rating

5.1.2 Tunnel Excavation Analysis

A construction diversion tunnel will be necessary to route the Chehalis River through the right abutment during construction. The proposed tunnel will be 20 feet wide by 20 feet high with a horseshoe shaped roof, and an invert slope of 1%. The Diversion Tunnel is shown in plan view on Figure 2.3 and in profile 1 on Figure 4.2. This tunnel will remain open for the entire duration of construction activities to reduce risks of flooding of the construction site. The tunnel will be advanced mostly through strong basalt but may require some support measures if siltstone materials are encountered, or the joint sets are aligned in such a way that blocks could come out of the tunnel walls creating instability. Unwedge software created by Rocscience (2013) was used to evaluate the support required to maintain the tunnel opening. Discontinuity data was used from the tunnel borings; TB-1, TB-2, TB-4 and TB-5; major joint sets from this data set were found using Dips 6.0 (Rocscience, 2014) and are shown in Table 5.2 and on Figure 4.8. TB-3 after drilling was too unstable and the televiewer equipment was not able to obtain discontinuity data from that boring.

The tunnel was broken down into 3 segments that have a straight orientations. The indicated orientation is with respect to north and represent the angle when moving in a clockwise direction around a circle with 0/360 degrees representing north:

- Segment 1 is the upstream intake with an orientation 032 degrees
- Segment 2 is through the right abutment, perpendicular to the dam axis with an orientation of 317 degrees
- Segment 3 is the downstream outlet with an orientation of 278 degrees

To analyze the curved segments of the tunnel between the straight segments the orientation was sequentially varied by 10 degrees between the adjoining straight section orientations to identify required support pressures at various orientations. The results of the analysis are shown in Table 5.3 and show that some but minimal support will be required in segments 1 and 3.

Segment 2 will require more support along with both curved sections that approach segment 2. The greatest required support will be at the downstream end of segment 2 as it curves to the west towards segment 3. More data in the location of the proposed construction diversion tunnel including angled borings is needed to validate these results and to complete the preliminary and final design of the tunnel including temporary and permanent support systems.

Table 5.2
Summary of Identified Joint Sets from the Diversion Tunnel Area

JOINT SET	STRIKE	DIP DIRECTION	DIP	DATA SOURCE
JS-1	120	210	12	TB-1, TB-2. TB-4, TB-5
JS-2	134	224	77	
JS-3	308	38	77	

Table 5.3
Summary of Identified Joint Sets from the Diversion Tunnel Area

SEGMENT	ORIENTATION (DEGREES)	WEDGE LOCATION	WEIGHT (KIPS)	SUPPORT PRESSURE REQUIRED (KSF)
1	32	Roof	0.001	0.01
Curve between segments 1 and 2	22	Roof	0.001	0.01
	12	Roof	0.001	0.01
	2	Roof	0.002	0.01
	352	Roof	0.003	0.01
	342	Roof	0.013	0.02
	332	Roof	0.093	0.04
	322	Roof	1.995	0.11
	312	Roof	59.806	0.25
2	317	Roof	22.176	0.24
Curve between segments 2 and 3	307	Roof	210.619	0.46
	297	Roof	1.487	0.07
	287	Roof	0.072	0.03
3	278	Roof	0.001	0.01

Note: Tunnel Slope is 1% or 0.6 degrees

5.2 Design Considerations

5.2.1 Foundation Excavation Objective

Results of the Phase 2 site characterization program have reinforced that the site is moderately complex from the standpoint of the design for a large and high hazard RCC dam and associated hydraulic structures (spillway and outlet works including abutment tunnels). Proper characterization of the site will be critical to establishing designs with appropriate risk management strategies that result in qualified contractors submitting competitive costs, and establishing design and construction contingencies that support successful completion of the project construction. Establishing the appropriate foundation excavation objectives (i.e., the estimated excavation limit that provides suitable strength and deformability characteristics to meet design requirements) is a critical element of the design. Criteria presented in Section 4.2.5 have been utilized to develop a 3D excavation objective surface meeting the rock quality objectives presented in this report as shown on Figures 4.1 to 4.4 Evaluating the excavation objective indicated the potential for localized areas of decreased rock quality which have been identified on Figure 4.1. These areas may require treatment, such as over-excavation and replacement, use of shaping block(s), consolidation grouting, or combinations of these methods to create suitable conditions for dam construction. Subsequent site characterization work will be targeted to address these areas in order to better define the risks associated with lower quality rock and to further identify the extent and methods for additional treatment. Such evaluations will be necessary to complete the preliminary design, and develop cost estimates meeting a suitable level of accuracy for final decision making and budgeting.

To that end, a 3-D site characterization model has been updated using the information and interpretations completed as part of this Phase 2 work. An example of the updated model is illustrated on Figure 4.12. The excavation objective surface for the FRFA dam footprint has been inserted to the 3D visualization model in order to demonstrate the amount and types of materials that will be excavated. Such information will be used to confirm excavation definitions and unit pricing for the excavation. Materials that may be suitable for remediation of landslides (rockfill buttresses), aggregate production, riprap, roadway surfacing, etc. that may be derived from the dam, hydraulic structure and tunnel excavations and are described further in the Design summary report.

Excavation with typical earthmoving equipment will be feasible in the overburden soils and weathered bedrock materials. The SRT data and interpretations indicate that a large portion of the upper bedrock excavation required to reach the foundation excavation objective limits can be achieved with a single ripper D-9R Caterpillar bulldozer (or equivalent) to depths ranging from 20 to over 50 feet along the left and right abutments.

A number of areas near the Chehalis River waterline and in the right and left abutments will have relatively shallow and/or modestly weathered to competent rock that must be removed as part of the required foundation excavation. Modestly weathered to competent rock was identified in Section 4.2.5 as having a compression wave velocity range of 6,200 to 9,000 fps. Portions of these areas will likely require controlled blasting for practical removal. Considerable attention must be given to blasting operations so that blasting produces a properly shaped foundation surface and to prevent damage of the rock beyond the specified limits of the excavation that would require additional removal and/or treatment. Localized shaping of the foundation excavation will also likely require controlled blasting operations including presplitting. Detailed blasting requirements will be developed as part of the final design.

5.2.2 Foundation Preparation and Treatment

After the initial excavation is completed, the exposed foundation rock should be cleaned, carefully examined, and mapped to verify that the excavation objective has been reached.

The initial excavation may reveal fractures, defects, and zones of unsuitable rock that will have to be selectively removed and/or treated. Replacing portions of the foundation that exhibit less favorable conditions with dental concrete, consolidation grouting, or construction of shaping blocks will improve the stability and performance of the dam and will be an important part of foundation preparation. Dental concrete will be required to complete the shaping of the foundation surface and reduce damage and/or deterioration of exposed foundation rock during construction activities, particularly any claystone/siltstone materials that are exposed.

The final excavated surface for the dam footprint will be variable across the site. It is important that the foundation be shaped so that a gradually varying surface is created along the dam axis, and from the upstream to the downstream toe of the dam. The prepared surface should be free of offsets or sharp breaks. Sharp breaks in the excavation can cause marked changes in stresses in both the dam and the foundation and have the potential to lead to adverse cracking and seepage from the RCC structure.

Once an acceptable foundation excavation and treatment are confirmed, the surface should undergo a final cleaning to remove remaining loose material. Cleaning will involve hand removal of unsuitable loose material, washing of the foundation rock surface, and removal of wash water and debris.

Foundation drains will be required to control uplift pressures on the base of the dam and within the foundation rock mass in order to achieve the stability and seismic design criteria for the project. Oversized drain-holes that are properly developed with well screens and sand packs across intervals of claystone/siltstone will be required to provide for long-term stability of the drain holes.

5.2.3 Grout Curtain and Foundation Drainage

Hydraulic conductivity is a measure of the rate that water flows through interconnected joint and fracture systems within a rock mass. Based on rock types at the site, movement of water through pore space in the rock is expected to be negligible. Highly fractured zones or locations where large open fractures occur in the bedrock were identified in both the Phase 1 and 2 site characterization programs. Unless treated, these fractures could act as preferential seepage pathways beneath the dam foundation and abutment. This may result in excessive water pressures acting on the base of the dam or other structure/tunnel elements, or result in unwanted loss of stored water. The Phase 1 and 2 explorations, review of rock cores, down-hole geophysical tests, and water pressure tests have identified that foundation grouting to treat and reduce flow through fractures in the rock will be an important and effective component of the dam design.

Zones encountered in non-landslide boreholes that had significant hydraulic conductivity are summarized in Table 5.4 and called out on Figure 4.1.

Table 5.4
Zones of High Hydraulic Conductivity

BOREHOLE	ELEVATION (FT) FROM	TO	LITHOLOGY	LUGEON VALUES >10	LOCATION
BH-2	362	352	Claystone	23	RCC Dam Alignment
BH-5	507	457	Claystone - Basalt	8-68	RCC Dam Alignment
DB-1	391	381	Claystone - Basalt	20-27	RCC Dam Alignment
DB-5	646	606	Basalt	19-40	Embankment Dam Alignment
DB-6	493	483	Basalt	22	Embankment Dam Alignment
TB-2	434	430	Basalt	25	RCC Dam Alignment
TB-2	404	393	Basalt	20-50	RCC Dam Alignment
TB-3	450	434	Basalt	20-214	Diversion Tunnel
TB-3	428	416	Claystone	43-52	Diversion Tunnel
TB-4	429	389	Basalt - Claystone	14-66	Diversion Tunnel

A number of zones of uncertainty at significant depth occurred due to failed hydraulic conductivity tests and are summarized in Table 5.5. They are also called out on Figure 4.1. BH-4 is not within either dam alignment and is of minimal concern for the grout curtain design; however the remaining zones of uncertainty fall within the RCC or embankment dam alignment and should be further addressed as the designs advance. Table 5.5 includes the highest Lu value that was found either above or below the zone of uncertainty. The criticality and size of the zone of uncertainty was considered when evaluating the depth of the grout curtain included in the conceptual designs.

Table 5.5
Deep Zones of Uncertain Hydraulic Conductivity and Need for Grouting Treatment

BOREHOLE	ELEVATION (FT)		HIGHEST LU VALUE ABOVE OR BELOW ZONE	LITHOLOGY	LOCATION
	FROM	TO			
BH-2	214	312	3	Basalt - Claystone	RCC Dam Alignment
BH-4	430	440	44	Gabbro	Landslide 1
BH-5	302	407	13	Claystone - Basalt	RCC Dam Alignment
BH-6	448	553	5	Basalt - Claystone	Outside of Dam footprint
BH-6	320	403	4	Claystone - Basalt	Outside of Dam footprint
DB-1	231	271	0	Basalt - Claystone	RCC Dam Alignment
DB-5	556	576	11	Claystone - Basalt	RCC Dam Alignment
DB-5	496	516	0.8	Basalt	RCC Dam Alignment
DB-6	503	523	6	Basalt	Embankment Dam Alignment
DB-6	474	483	22	Basalt	Embankment Dam Alignment
OB-2	249	288	2	Basalt	RCC Dam Alignment

Overall, at least 30 percent of the water pressure tests and corresponding estimates of hydraulic conductivity measurements suggest groutable bedrock zones.

Based on these test results, evaluation of core samples from the boreholes, and our general experience on other similar project sites, a multi-line grout curtain extending to the approximate limits shown on Figure 4.1 has been included in designs. The uncertainty in BH-5 is driving the large depth of the grout curtain in the left abutment. This limit should be refined with further investigations. The high Lu values and failed hydraulic conductivity tests at the bottom of DB-6 are the primary consideration related to the depth of the grout curtain in that area beneath the embankment dam. Additional site characterization work along the RCC dam axis, including the left abutment and the embankment dam axis, should be performed to further develop the grouting program requirements and estimates of grouting program costs.

Based on initial structural information from the boreholes, it is anticipated that grout holes should be inclined (15 to 30 degrees) in an orientation along the curtain alignment to intercept the maximum number of sub-vertical fractures in the basalt of the Crescent Formation. The number of grout lines, spacing and inclination of the grout holes, and sequence of grouting operations may vary along the dam based on results from supplemental site characterization studies. Grouting procedures should be based on achieving a Lu value closure criteria between 1 and 5.

Downstream of the grout curtain, a system of foundation drain holes has been included in the design. The combined grout curtain and drain hole system provides a proven strategy for controlling seepage from the reservoir and limiting the uplift pressures that can act on the base of the dam. Drain holes are expected to be installed following grouting operations and be located about 10 to 20 feet downstream of the curtain. They will extend into the dam drainage gallery in order to discharge any foundation seepage that by passes the grout curtain. These drains will be accessible for routine maintenance and monitoring in order to confirm the safe performance of the structure over its design life. The orientation of the drain holes will likely be similar to the orientation of the grouting holes in order to maximize the potential for intercepting foundation seepage and to maximize the net reduction in foundation water pressures under the dam.

5.2.4 Structural Model Parameters

The design process includes a structural analysis of both the FRFA RCC dam at both the spillway overflow and maximum non-overflow sections of the dam. This analysis requires model parameters derived from the site investigation data including; depth to the top of rock, rock strength (cohesion and friction angle) and rock modulus from Rock Mass Rating (RMR). Three deformation moduli were estimated based on RMR, lower bound (siltstone/claystone), upper bound (basalt) and a weighted average of the moduli of the siltstone and basalt. The lower bound value (E_D) of 1.39 x 10^6 psi used was representative of a siltstone/claystone bedrock stratigraphy below the dam. The upper bound was 3.15 x 10^6 psi and represents a basalt stratigraphy below the dam. A weighted average modulus of 2.61 x 10^6 psi was used to represent an interbedded siltstone/claystone and basalt stratigraphy below the dam. A Poisson's ratio of 0.3 was used for the bedrock in the model.

5.2.5 RCC Aggregate Source

Three Quarry sources were evaluated during the Phase 2 investigation and all three sites show potential for acceptable aggregate. Quarry 1 has the shortest haul distance (2 miles) and is within the reservoir but had the lowest quality laboratory testing results of the three sites. The seismic refraction tomography data (QSL-1) does not correlate well with soil depth shown QB-1 creating uncertainty with regard to the amount of overburden material that will have to be removed to reach a suitable quality of rock for aggregate production. Quarry 2 has the longest haul distance (over 8 miles) over logging haul roads but has relatively good laboratory results and potential for high quantities of aggregate. Quarry 3 (Rock Creek) indicate the highest quality laboratory results and from a surface reconnaissance standpoint has potential for large quantities with little overburden. However, the haul distance is about 7 miles, the majority of which would be over highway 6 which may be more favorable than hauls over the logging haul roads. Further testing and evaluations of all three of these quarries should be completed as part of the next phase of work including additional boreholes at all three sites, an SRT survey at Quarry 3 and perhaps additional SRT surveys at Quarries 1 and 2. For a more complete discussion of the RCC aggregate potential refer to the Conceptual Design Report (HDR 2016).

5.2.6 Dam Site Landslides

As discussed in Section 4.2.6 there are four landslides within the immediate vicinity of the dam that will likely require some combination of removal and stabilization as part of the dam construction work due to proximity to the dam and appurtenant features. Additional investigations and evaluations will be required to update and ultimately finalize design and construction requirements for these landslides.

Landslides 3a and 1 are both upstream of the proposed dam alignment and may have a high likelihood of reactivation during a 100 year flood event and the subsequent drawdown whether it is at the designed or modified drawdown rate. Movement of landslide 3a could threaten the outlet works intake during normal operations of the FRFA configuration but also poses a threat to both configurations during construction as discussed in Section 4.2.6. The results of the Landslide Evaluation indicate very low stability FS during and therefore this landslide will require extra effort during construction to be removed, or mitigated to bring the FS of instability within a reasonable level so as not to pose a risk to dam construction or operations. During construction a portion of Landslide 1 will need to be excavated to achieve the required excavation for the dam. Excavating into a landslide complex may pose a risk of reactivation and may require shallower than normal cut slopes increasing excavation volumes. Since the likely failure plane shown in the Landslide Evaluation is small and located mid-slope it is possible that removal of a portion of the landslide during excavation may result in increased stability and decrease the likelihood of failure during drawdown after a flood event. Furthermore the results of the Landslide Evaluation on Landslide 1 indicate that decreasing the drawdown rate could increase stability to a point where no mitigation is required. Further evaluation considering the planned excavation limits and possible decreased drawdown rate should be performed in the next phase of work to evaluate the likelihood of instability during construction and drawdown.

Due to the excavation plan, significant portions of both Landslides 1a and 26 will be removed which could either increase or decrease stability. Both of these landslides pose a risk to the spillway flip bucket and fish passage structures for all dam and fish passage system configurations. Further evaluation of these landslides considering the excavation plan should be performed during the next phase of work to evaluate the necessity and magnitude of removal or mitigation.

5.2.7 Construction Diversion Tunnel

The Phase 2 Site Characterization program has provided some important information for advancing tunnel design and construction requirements for either the FRO or FRFA project construction diversion channel configurations. Several important layout and design factors have been identified, including the following:

- Lithology and bedding/foliation orientation of each strata the proposed tunnel passes through
- Joint set orientations and variability along the tunnel alignment
- Hydraulic conductivity through the various strata and the potential for water pressures acting on tunnel linings
- Lithological contacts that present the potential for abruptly varying rock mass conditions and potential mixed-face tunnel excavation conditions

- Soft (basalt or siltstone/claystone) zones, weathered zones, and shear zones that may require additional tunnel excavation, stabilization, treatment and lining.
- Sequencing of dam excavation and construction activities with tunnel construction work to anticipate all loading conditions that may affect the design and long-term performance of the tunnel diversion system.

Additional borehole and laboratory testing data will be needed along the proposed tunnel alignment to evaluate the above factors if the construction diversion tunnel concept is advanced in preliminary design. Due to the steepness of the right abutment drilling vertical boreholes to intersect the proposed tunnel alignment will be difficult and expensive as it may require helicopter drilling access. The road through the lower right abutment area provides a suitable location to drill angled boreholes as was done in Phase 2 with TB-2 and TB-3.

5.2.8 Embankment Dam

A topographic ridge exists in the upper right abutment of the dam site. An embankment dam will be needed in this area to achieve the FRFA dam crest elevation of 713.4 feet. A composite rockfill dam section with a central zone of lower permeability embankment materials attaches to the RCC dam section at about dam axis station 18+00 as shown on Figure 2.3. The Phase 2 investigation added three boreholes (DB-5, DB-6 and DB-7) to the ridge as shown in the centerline profile on Figure 4.1. The embankment dam embankment will have side slopes of 2(H):1(V) and could have a maximum structural height of up to 80 feet in the vicinity of DB-6 depending on the final foundation excavation and treatment requirements. The overburden soil and weathered bedrock on the top of the ridge tends to be thicker than across the valley of the RCC dam and averages about 50 feet. As seen in DB-6 a weaker claystone locally extends to a depth of 144 feet and may create some challenges related to excavation stability and possible seepage through the ridge. This large zone of weaker material is unsuitable for RCC foundation therefore the transition from RCC to embankment has been shifted closer to the RCC dam area. Future work should include a seepage analysis through the ridge and stability analysis of the proposed composite dam section. The Phase 2 investigation results have added significantly to characterization of the subsurface beneath the embankment dam embankment. Additional subsurface information is required to fully evaluate the foundation conditions beneath the embankment dam.

5.3 Reservoir Landslide Hazard

Twenty-seven landslides have been confirmed in the reservoir basin as described in Section 3.3. The project design must consider how these landslides may affect reservoir safety and operation. For example, the outlet works must be able to effectively manage and pass sediment and debris load that may result from the reservoir basin, including sediment and debris associated with landslide movement that may occur during normal and flood operation conditions.

An updated Landslide Evaluation has been prepared by Shannon and Wilson Inc. included as Appendix B. Of the ten landslides evaluated two were downstream of the proposed dam alignment (Landslides 1a and 26) and two have been discussed as dam site landslides (1 and 3a) in Sections 4.2.6 and 5.2.6. The remaining six landslides evaluated almost all display unstable factors of safety values during both design and modified drawdown rates after a 100 year flood event. The only exception is the upper slope of Landslide 4 which shows stability at higher friction angles for the design drawdown rate, and all friction angles for the modified drawdown rate. The lower slope of landslide 4 also shows stability for the FRFA only and only at modified drawdown rates. It was noted that the failure surface of the landslide is relatively small compared to the overall mass of the landslide. For details of the Landslide Evaluation results refer to Appendix B.

Landslide remediation/stabilization of reservoir landslides in addition to the dam site landslides previously discussed represents potential costs that should be further evaluated in the next phases of work.

6 CONCLUSIONS AND RECOMMENDATIONS

The results of the Phase 1 and 2 subsurface investigations beneath the Chehalis Dam site have been presented in this report. These results indicate that the foundation at the proposed dam site consists primarily of early-middle Eocene-age Crescent Formation basalt with interbedded claystone/siltstone units. McIntosh formation siltstone was identified in the upper portion of BH-6 just outside of the dam footprint to the north. The McIntosh formation siltstone and Crescent Formation siltstone are very similar due to contemporaneous deposition. It is possible that McIntosh formation siltstone materials will be found at other locations in the upper right abutment area beneath the proposed embankment dam. The Crescent Formation basalt demonstrates characteristics that are suitable for the RCC dam foundation including good Rock Mass Rating (RMR), weathering and compression wave velocity characteristics. Claystone/siltstone units, where they occur, demonstrate mostly fair RMR and other RCC dam foundation characteristics. The overall foundation strength will be controlled by the overwhelming proportion of the basalt. Results of the Phase 2 Site Characterization work at the potential Chehalis Dam site have reinforced that foundation conditions are suitable for construction of either an RCC or rockfill dam type with the RCC dam type being preferred. Laboratory test results indicate that basalt bedrock that is suitable for the generation of competitively priced RCC and conventional concrete aggregate is available in reasonable proximity to the dam site.

The rock structure information gathered during Phase 2 suggests jointing and fractures in the bedrock that have a low potential for sliding surfaces beneath the dam and along temporary and permanent excavated slopes. However, these hazards require continued characterization and analysis during future engineering design work. Information from the study has confirmed that the required excavation for an RCC dam would likely be similar to the amount of excavation assumed in the previous conceptual-level design studies and that the overall dam construction costs should fall within the estimated range presented in the *Combined Dam and Fish Passage Alternatives Memorandum* (HDR 2014).

Highly fractured zones of bedrock were identified in the Phase 2 program. These zones will act as preferential seepage pathways beneath the dam foundation and abutment unless they are treated. The Phase 2 borehole data have confirmed that a combination of foundation grouting to treat the fractured zones and reduce seepage, and foundation drain holes downstream of the grout curtain will be an will be an important and effective component of the dam design.

Over-excavating highly fractured zones, shear zones, and/or highly weathered zones, use of dental concrete and installing a grout curtain to sufficient depth will address area of concern in the dam foundation, provide a consistent foundation surface for the dam structure, and sufficiently extend and mitigate seepage pathways beneath the dam foundation (Rizzo and Charlton 2008).

Twenty-three landslides have been confirmed in the immediate vicinity of the dam and in the reservoir basin. The project design has been updated to consider how these landslides may affect dam and reservoir safety and operation, and remediation requirements, particularly in the immediate vicinity of the dam site. Landslide remediation/stabilization represents a potential significant cost and further evaluation and remedial design development should be included during future design phases.

Establishing the appropriate foundation excavation objective (i.e., the estimated excavation limit that provides suitable strength and deformability characteristics to meet design requirements) is a critical element of work for the dam. A comprehensive excavation surface meeting design objectives identified during the Phase 2 evaluations has been developed. This excavation surface and the related shaping and treatment requirement should continue to be evaluated and updated during future design phases.

The Phase 2 program results suggest that a combined site characterization approach using boreholes with RQD, RMR, and weathering parameters established from proper logging procedures along with a combination of surface seismic refraction tomography and downhole televiewer geophysical exploration methods will be suitable to advance these excavation objectives going forward. Future site characterization phases should be performed concentrating on boreholes and geophysical surveys to support updating of the 3D subsurface model initiated as part of the Phase 1 study. Additional data will provide the ability to evaluate prominent joint sets, in-situ hydraulic conductivities, and updated excavation objective and limits along the entire dam foundation consistent with the desired level of cost estimating accuracy desired for preliminary design, and for appropriate risk management during final design and construction.

7 REFERENCES

ASTM D 2487-11, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

Brown, E.T. (Editor), 1981, Rock Characterization, Testing and Monitoring: ISRM Suggested Methods, Pergamon Press

Bieniawski, Z.T, 1989, Engineering Rock Mass Classifications, New York, NY, Wiley.

Deere, D.U., 1963, Technical Description of Rock Cores for Engineering Purpose, Rock Mechanics and Engineering Geology, Vol. 1, No. 1

Edumine, 2016, Rock Types and Specific Gravity, http://www.edumine.com/xtoolkit/tables/sgtables.htm, accessed November 21st, 2016

Ewert, F.-K., 2003, "Discussion of Rock Type Related to Criteria for Curtain Grouting," Proceedings of the Third International Conference on Grouting and Ground Improvement, ASCE Special Publication No. 120,

Goodman, R.E., 1980, Introduction to Rock Mechanics (Chapter 8), Toronto, Canada, Wiley.

HDR, 2014, Combined Dam and fish Passage Alternatives Technical Memorandum, Report Prepared for the State of Washington, October,

HDR and Shannon & Wilson Inc., 2015, Phase 1 Site Characterization Technical Memorandum, Report Prepared for the State of Washington, August, 2015.

HDR, 2016, Chehalis Dam Project Quarry Evaluation Technical Memorandum, Report Prepared for the State of Washington, September, 2016.

HDR, 2016b Cost Estimate for Use in EIS Dam Design and Fish Passage Cost Estimate, Memo Prepared for EES Consulting, August, 2016.

Hoek, E. and Bray, J.W., 1997 (first edition 1981), Rock Slope Engineering: Institution of Mining and Metallurgy, London

Houlsby, A., 1976, Routine Interpretation of the Lugeon Water-Test. Q. J. Eng. Geol. Vol. 9, pp. 303-313.

Johnson, R.B. and DeGraff, J.V., 1988, Principles of Engineering Geology, Wiley

Moothart, S.R., 1992, Geology of the Middle and Upper Eocene McIntosh Formation and Adjacent Volcanic and Sedimentary Rock Units, Willapa Hills, Pacific County, Southwest Washington, Master of Science Thesis, Oregon State University.

Rizzo, P.C., and Charlton, J., 2008, Foundation Preparation for RCC Dams Founded on Difficult Foundation Conditions, Presented at Hydro2008 in Lubljana, Slovenia, October 6-8, 2008.

Rocscience, Inc., 2013, Unwedge, version 3.023, October 16,2013

Rocscience, Inc., 2014, Dips, version 6.015, April 17, 2014

Shannon & Wilson Inc., 2009, *Geologic Reconnaissance Study, Proposed Chehalis River and South Fork Dam Sites, Lewis County, Washington,* Prepared for EES Consulting, Inc., Kirkland, Washington, Oct. 2009.

Shannon & Wilson, Inc., 2014, Preliminary Desktop Landslide Evaluation: Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-21897-010, for Chehalis Basin Workgroup., July 30, 2014.

Shannon & Wilson, Inc., 2015, Preliminary Design Earthquake Time Histories: Letter Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-21897-010, for HDR, Sept. 30, 2015.

Weaver, K. D., and D. A. Bruce, Dam Foundation Grouting, ASCE Press, 2007.

Wells, R.E., and Sawlan, M.G.,2014, Preliminary Geologic Map of the Eastern Willapa Hills, Cowlitz, Lewis, and Wahkiakum Counties, Washington, United States Department of the Interior, U.S. Geological Survey, Denver, CO.

Figures

Appendix A
Phase 2 Chehalis Dam Geotechnical Data Report

Appendix B
Phase 2 Chehalis Dam Landslide Evaluation

Appendix C
Phase 2 Chehalis Dam Site Characterization Landslide Stability Improvement Evaluation

Appendix D
Report on the Geophysical Surveys for Chehalis Dam – Phase 2, Pe Ell, WA

Appendix E
Composite Boring Logs

Appendix F
Laboratory Testing Data Reports

Appendix G
Rock Mass Rating Data

Appendix H
Water Pressure Testing Data

Appendix I
Abutment Excavation Analysis