Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species

Phase 1 Site Characterization

Technical Memorandum

September 25th, 2015
Prepared by HDR and Shannon and Wilson, Inc.
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<thead>
<tr>
<th>Acronym</th>
<th>Definition</th>
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<tbody>
<tr>
<td>°</td>
<td>degrees</td>
</tr>
<tr>
<td>3-D</td>
<td>three-dimensional</td>
</tr>
<tr>
<td>ASR</td>
<td>alkali-silica reactivity</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing Materials</td>
</tr>
<tr>
<td>BH</td>
<td>borehole</td>
</tr>
<tr>
<td>BP</td>
<td>before present</td>
</tr>
<tr>
<td>cal yr BP</td>
<td>calibrated years before present</td>
</tr>
<tr>
<td>CMCE</td>
<td>controlling maximum credible earthquake</td>
</tr>
<tr>
<td>CSZ</td>
<td>Cascadia Subduction Zone</td>
</tr>
<tr>
<td>D</td>
<td>dip-slip</td>
</tr>
<tr>
<td>DSHA</td>
<td>deterministic seismic hazard analysis</td>
</tr>
<tr>
<td>famsl</td>
<td>feet above mean sea level</td>
</tr>
<tr>
<td>fps</td>
<td>feet per second</td>
</tr>
<tr>
<td>FRO</td>
<td>flood retention only</td>
</tr>
<tr>
<td>FRFA</td>
<td>flood retention flow augmentation</td>
</tr>
<tr>
<td>FT</td>
<td>feet</td>
</tr>
<tr>
<td>g</td>
<td>acceleration as a percent of gravity</td>
</tr>
<tr>
<td>GCF</td>
<td>Gales Creek Fault</td>
</tr>
<tr>
<td>GPS</td>
<td>Global Positioning System</td>
</tr>
<tr>
<td>JS</td>
<td>joint set</td>
</tr>
<tr>
<td>ka</td>
<td>one thousand years ago</td>
</tr>
<tr>
<td>km</td>
<td>kilometer</td>
</tr>
<tr>
<td>LA</td>
<td>Los Angeles</td>
</tr>
<tr>
<td>LiDAR</td>
<td>Light Detection and Ranging</td>
</tr>
<tr>
<td>LL</td>
<td>liquid limit</td>
</tr>
<tr>
<td>Lu</td>
<td>Lugeon</td>
</tr>
<tr>
<td>m</td>
<td>meter</td>
</tr>
<tr>
<td>M</td>
<td>magnitude</td>
</tr>
<tr>
<td>mb</td>
<td>body wave magnitude</td>
</tr>
<tr>
<td>MCE</td>
<td>maximum considered earthquake</td>
</tr>
<tr>
<td>mm</td>
<td>millimeter</td>
</tr>
<tr>
<td>MMI</td>
<td>modified Mercalli intensity</td>
</tr>
<tr>
<td>Ms</td>
<td>surface wave magnitude</td>
</tr>
<tr>
<td>Mw</td>
<td>moment magnitude</td>
</tr>
<tr>
<td>NEHRP</td>
<td>National Earthquakes Hazard Reduction Program</td>
</tr>
<tr>
<td>NSHMP</td>
<td>National Seismic Hazard Mapping Project</td>
</tr>
<tr>
<td>O</td>
<td>oblique-slip</td>
</tr>
<tr>
<td>PGA</td>
<td>peak ground acceleration</td>
</tr>
<tr>
<td>PI</td>
<td>plasticity index</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------</td>
</tr>
<tr>
<td>PL</td>
<td>plastic limit</td>
</tr>
<tr>
<td>PSHA</td>
<td>probabilistic seismic hazard analysis</td>
</tr>
<tr>
<td>psi</td>
<td>pounds per square inch</td>
</tr>
<tr>
<td>PST</td>
<td>Pacific standard time</td>
</tr>
<tr>
<td>Qa</td>
<td>Quaternary alluvium</td>
</tr>
<tr>
<td>Qao</td>
<td>Quaternary alluvium older</td>
</tr>
<tr>
<td>Qc</td>
<td>Quaternary colluvium</td>
</tr>
<tr>
<td>Qls</td>
<td>Quaternary landslide deposit</td>
</tr>
<tr>
<td>Qos</td>
<td>Quaternary overburden soil</td>
</tr>
<tr>
<td>RCC</td>
<td>roller-compacted concrete</td>
</tr>
<tr>
<td>RMR</td>
<td>rock mass rating</td>
</tr>
<tr>
<td>RQD</td>
<td>rock quality designation</td>
</tr>
<tr>
<td>S</td>
<td>subsidence</td>
</tr>
<tr>
<td>SMFZ</td>
<td>Saddle Mountain Fault Zone</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard Penetration Test</td>
</tr>
<tr>
<td>SSD</td>
<td>saturated surface dry</td>
</tr>
<tr>
<td>Tig</td>
<td>intrusive igneous volcanics</td>
</tr>
<tr>
<td>Tcb</td>
<td>Crescent Formation basalt</td>
</tr>
<tr>
<td>Tcs</td>
<td>Crescent Formation siltstone</td>
</tr>
<tr>
<td>Tml</td>
<td>McIntosh Formation lower</td>
</tr>
<tr>
<td>U</td>
<td>uplift</td>
</tr>
<tr>
<td>UHS</td>
<td>uniform hazard spectrum</td>
</tr>
<tr>
<td>μm</td>
<td>micrometer</td>
</tr>
<tr>
<td>USCS</td>
<td>Unified Soil Classification System</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
</tr>
<tr>
<td>V&lt;sub&gt;30&lt;/sub&gt;</td>
<td>average shear velocity down to 30 meters</td>
</tr>
<tr>
<td>VWI</td>
<td>vibrating wire piezometer</td>
</tr>
<tr>
<td>WBZF</td>
<td>Willapa Bay fault zone</td>
</tr>
<tr>
<td>WC</td>
<td>water content</td>
</tr>
<tr>
<td>WSDOT</td>
<td>Washington State Department of Transportation</td>
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Executive Summary

1 Executive Summary

Characterizing the geologic and geotechnical conditions at a dam site is a critical component of the planning and design stages of a project. It is common that site characterization work is completed in multiple phases, with each phase building upon previous investigation in order to appropriately address the important issues related to 1) the section of a preferred dam type, 2) configuring the preferred dam type and related appurtenant structures such as outlet works and spillways, and 3) providing sufficient information to establish project costs and address related construction and cost risks. The purpose of the Phase 1 Site Characterization work presented in this report is to provide geotechnical engineering information that will better inform the conceptual design and cost estimate of the potential Chehalis Dam and confirm the feasibility of using a roller-compactled concrete (RCC) dam at the dam site. The full report summarizes the following tasks:

- Investigate the location, thickness, and characteristics of landslide materials on the left abutment,
- Provide data regarding foundation conditions within the proposed dam footprint, including left and right abutments and valley bottom,
- Perform a field evaluation of landslides around perimeter of reservoir,
- Perform additional seismic analyses to define earthquake risks, and
- Perform additional and more detailed investigations into aggregate sources for concrete that could be used to construct a RCC dam.

The work performed for this Phase 1 Site Characterization has confirmed that foundation conditions are suitable for construction of either a roller compacted concrete (RCC) or rockfill dam type at the potential dam site. Suitable aggregate is likely available in reasonable proximity to the site for production of RCC and conventional concrete materials at competitive unit prices. Further evaluations of potential aggregate sources should be completed as part of the next phase of work funded to occur in the current state biennium.

In addition to confirming the feasibility of each dam type, information from this report has also confirmed that the required excavation for an RCC dam would likely be similar to the amount of excavation assumed in the previous conceptual level designs and overall costs would still fall within the estimated range presented in the Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR 2014). Consequently, considering cost and other technical factors, the RCC dam type for either the flood retention only (FRO) or flood retention flow augmentation (FRFA) configurations is the preferred option for the site and it is recommended that future design work proceed under this basis.

Results of the Phase 1 Site Characterization program have identified important subsurface conditions that vary across the potential dam site. These variable conditions will present localized design and construction considerations and requirements. To address this local variability, a three-dimensional (3-
D) site characterization model has been initiated using the information and interpretations completed as part of this Phase 1 work. Additional site characterization information will be needed to complete this model and establish the overall excavation limits that will provide suitable strength and deformability characteristics to meet design requirements for an RCC dam. This model will also be utilized to establish other foundation treatments such as 1) the limits and details of foundation grouting, 2) locations for over-excavation and replacement of softer foundation materials, and 3) excavation shaping to limit changes in the excavated rock surface that could cause the dam to adversely crack. Future phases of site characterization should be performed to provide the information needed to complete this model, and finalize design and construction requirements.

The rock structure information gathered during Phase 1 suggests that there is jointing and fractures in the bedrock that create a small potential for sliding surfaces beneath the dam, and along temporary and permanent excavated slopes required for dam construction. Based on the study results, the rock jointing is typical for similar geology and dam sites. Dam safety and construction risks resulting from the rock jointing can be addressed through proven design and construction methods.

Some zones with highly fractured rock and large open fractures were also identified in the Phase 1 program. Such zones are commonly found in rock at dam sites, and if untreated, can act as preferential seepage pathways beneath the dam foundation and abutments. The Phase 1 borehole data have confirmed that foundation grouting can be used to effectively treat these zones, reduce seepage through them, resulting in a safe dam foundation under all reservoir operating conditions. A grouting program will be an important component of the dam design to meet safety requirements.

No active faults were identified at the dam site. However, there are active faults in the region that will be an important consideration in the dam design. Specifically, the earthquake ground motions that may occur at the potential dam site will be a primary consideration in the development of the dam cross-section configuration as well as the structural design of spillway and outlet works facilities. These structures will be designed to withstand peak ground accelerations (pga) at the site that could be as high as 0.8 g.[1]

The inventory of landslides confirms twenty-three landslides exist in the reservoir basin. If the project moves forward, the design will consider how these landslides may impact dam and reservoir safety and operation. For example, the outlet works must be able to effectively manage and pass sediment and debris load that may occur from the reservoir basin, including sediment and debris from landslides activated during normal operations and flood events. Landslide remediation/stabilization represents a potential significant cost and should be further evaluated in the next phase of work.

[1] g is acceleration as a percent of gravity.
2 Introduction

2.1 PROJECT DESCRIPTION

The Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project (Project) is a feasibility-level 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 Chehalis Basin Strategy include studies of alternative water retention structures (dams), options for protecting Interstate 5 with or without a dam, and other small flood reduction projects throughout the Chehalis Basin (basin) with or without a dam.

As part of this Chehalis Basin Strategy, HDR, with support from other Anchor QEA team members, prepared a report entitled *Combined Dam and Fish Passage Alternatives Technical Memorandum* (HDR 2014). That study 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 least 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 study 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 while a rockfill dam may have seismic advantages. The 2014 study included recommendations that several site characterization activities be conducted to allow for further assessing of site conditions, selecting the preferred dam type, and addressing several dam configuration issues.

Subsequent to the 2014 technical memorandum, the Anchor QEA team was asked to perform additional site investigations and geotechnical engineering analyses to better inform the design and cost estimate of the dam and confirm the feasibility of using an RCC dam at the potential dam site. That work is the subject of this technical memorandum.

The work performed for this report was:

1. Investigation of foundation conditions within the potential dam footprint, including left and right abutments and valley bottom,
2. Initial evaluation of the potential 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 perimeter of reservoir,

4. Additional seismic analyses to define earthquake hazards and related design criteria, and

5. Additional 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 subconsultants to Anchor QEA.

2.2 DAM TYPES

Four technically feasible dam configuration alternatives were recommended for further development and consideration in the 2014 Combined Dam and Fish Passage Alternatives Technical Memorandum:

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

The locations of the 2015 Phase 1 Site Characterization work at the Chehalis River dam site are shown in Figure 2.2-1; Chehalis Dam vicinity map.

2.3 PURPOSE AND OBJECTIVES

This Phase 1 Site Characterization program has been performed with the following objectives;

1. Identify fatal flaws in reservoir and dam foundation geologic conditions of the identified site that would limit or preclude construction of an RCC or rockfill water retention structure (dam).
2. Advance foundation excavation and preparation concepts including potential foundation grouting requirements, foundation excavation objective, and requirements to achieve excavation and permanent slope stability to the conceptual level of design.
3. Assess the suitability of local aggregate sources for use as RCC aggregate.
4. Further evaluate seismic, landslide, and debris hazards in the reservoir basin and at the dam site.

This Final Phase 1 Site Characterization Report (Final Report) presents results from the Phase 1 Site Characterization field work including the feasibility and preference for the type of dam at the site and initial concepts for foundation treatment and preparation. The information in this Final Report will form the basis for identifying additional site characterization work needed at the site, and for further developing more advanced conceptual/preliminary designs that will be carried forward to the Alternatives Comparison phase.

2.4 SCOPE OF WORK

The scope of work for the Chehalis Dam Phase 1 Site Characterization program included the tasks discussed below.
2.4.1 GEOPHYSICAL TESTING
Seismic refraction surveys were performed along five transects within and near the potential dam footprint, as shown in Figure 2.4-1. 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 thickness of weathered rock along the potential dam axis and on the left abutment.

Downhole geophysical testing was performed in each borehole. 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 all boreholes to generate a continuous oriented 360° image of the borehole wall and record orientations of discontinuities including joints, shear zones, and lithologic contacts.

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

2.4.2 BOREHOLE DRILLING, SAMPLING, TESTING, AND INSTRUMENTATION INSTALLATION
The Phase 1 borehole drilling and testing program included six boreholes ranging in depth from 120 to 350 feet. Borehole (BH)-1, BH-3, and BH-4 were drilled through landslide and foundation bedrock materials in the vicinity of the left abutment of the potential dam footprint area. The purposes of these boreholes were to assist in characterization of the landslides, correlate borehole results with geophysical testing result, and assist with estimating excavation and foundation requirements within and near the dam footprint. The other three boreholes (BH-2, BH-5, and BH-6) were drilled along the potential dam centerline near the anticipated maximum section of the dam and in the left and right abutments for the purpose of characterizing the foundation excavation objective along the dam axis. Borehole locations are shown in Figure 2.4-1.

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, then decreased to the initial pressure. Tests were performed over 10-foot intervals using a double packer assembly for the entire depth of bedrock penetration.

Rock core was logged according 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 the laboratory for final logging and selection of laboratory specimens for testing.

A summary of the boreholes including purpose, depth, and instrumentation installation is provided in Table 2-1.
2.4.3 LANDSLIDE FIELD EVALUATION

Twenty-five potential deep-seated landslides were previously identified in and near the potential dam footprint and within the potential reservoir pool area using Light Detection and Ranging (LiDAR) hillshades images and geologic maps. These 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 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 landslide reconnaissance are summarized in Appendix A. This report includes descriptions and conclusions about each visited landslide and photographs and sketches of significant geologic features.

2.4.4 SEISMIC HAZARD EVALUATION

A deterministic and probabilistic ground motion study has been performed for this Phase 1 Site Characterization using available United States Geological Survey (USGS) and Shannon & Wilson, Inc., geologic and seismicity information.

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Table 2-1
Summary of Phase 1 Borehole Explorations

<table>
<thead>
<tr>
<th>BOREHOLE NUMBER</th>
<th>DEPTH (FEET)</th>
<th>PURPOSE</th>
<th>INSTRUMENTATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>140</td>
<td>Excavation objective, foundation treatment, and landslide characterization</td>
<td>Vibrating Wire Piezometer (VWP)</td>
</tr>
<tr>
<td>BH-2</td>
<td>240</td>
<td>Maximum section, excavation objective and foundation treatment</td>
<td>VWP</td>
</tr>
<tr>
<td>BH-3</td>
<td>150</td>
<td>Upstream landslide characterization</td>
<td>VWP</td>
</tr>
<tr>
<td>BH-4</td>
<td>120</td>
<td>Upstream landslide characterization</td>
<td>VWP</td>
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<td>VWP</td>
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<tr>
<td>BH-6</td>
<td>350</td>
<td>Excavation objective and foundation treatment</td>
<td>VWP</td>
</tr>
</tbody>
</table>
2.4.5 RCC AGGREGATE AND FOUNDATION BEDROCK LABORATORY TESTING

Results of previous pre-feasibility evaluations of alternative RCC aggregate materials were expanded as part of the Phase 1 Site Characterization. Four potential sources were selected and evaluated, including two commercial quarries, an abandoned Weyerhaeuser quarry, and rock outcropping near the dam. The following testing was performed by the Lafarge North America concrete lab in Seattle, Washington, on additional samples obtained from the four source sites:

- Alkali-silica Reactivity (ASR) – Four 16-day tests and four 1-year tests.
- Specific Gravity – All four rock sources were analyzed for Specific Gravity of the Fine Aggregate.
- Los Angeles (LA) Abrasion Test – All four rock sources were tested for resistance to abrasion.

Petrographic analyses on the four potential aggregate sources were performed by Spectrum Petrographics, Inc., of Vancouver, Washington, to determine the mineral composition and rock type of the samples. Unconfined compressive strength tests, slake durability tests, and bulk density tests were performed by GeoTesting Express in Acton, Massachusetts.

A summary of the laboratory index testing methods for the potential RCC aggregates is presented in Table 2-2.

<table>
<thead>
<tr>
<th>ASTM TEST DESIGNATION</th>
<th>TEST METHOD NAME</th>
<th>NUMBER OF TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>C 29</td>
<td>Standard test method for bulk density (unit weight) and voids in aggregates</td>
<td>4</td>
</tr>
<tr>
<td>C 131</td>
<td>Test method for resistance to degradation of small-size coarse aggregate by abrasion and impact in the LA machine</td>
<td>4</td>
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<tr>
<td>C 295</td>
<td>Petrographic examination of aggregates for concrete</td>
<td>7</td>
</tr>
<tr>
<td>C 535</td>
<td>Test method for resistance to degradation by abrasion and impact in the LA machine</td>
<td>4</td>
</tr>
<tr>
<td>C 1260</td>
<td>Test method for potential alkali reactivity of aggregates (mortar bar method)</td>
<td>4</td>
</tr>
<tr>
<td>D7012 Method D</td>
<td>Compressive strength and elastic moduli of intact rock core specimens</td>
<td>4</td>
</tr>
</tbody>
</table>

2.5 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 Geotechnical Specialist
Andrew Little  Project Geotechnical Specialist
Michael Woodward  Project Geotechnical Specialist
<table>
<thead>
<tr>
<th>Name</th>
<th>Position</th>
<th>Organization</th>
</tr>
</thead>
<tbody>
<tr>
<td>Garrett Harris</td>
<td>Project Geotechnical Specialist</td>
<td>Shannon &amp; Wilson, Inc.</td>
</tr>
<tr>
<td>Helen Regan</td>
<td>3-D Visualization Specialist</td>
<td>Shannon &amp; Wilson, Inc.</td>
</tr>
<tr>
<td>Stan Boyle, P.E., Ph.D</td>
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</tr>
<tr>
<td>Bill Laprade, L.E.G.</td>
<td>Senior Engineering Geologist</td>
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</tr>
<tr>
<td>Erik Scott, L.E.G.</td>
<td>Senior Geologist</td>
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</tr>
<tr>
<td>Ali Shahbazian, P.E., Ph.D</td>
<td>Seismic Hazards</td>
<td>Global Geophysics</td>
</tr>
<tr>
<td>Elizabeth Barret</td>
<td>Staff Geologist</td>
<td>Global Geophysics</td>
</tr>
<tr>
<td>John Liu, R.G., Ph.D.</td>
<td>Principal Geophysicist</td>
<td>Global Geophysics</td>
</tr>
</tbody>
</table>
Background Information

3 Background Information

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 USGS (Wells and Sawlan 2014). These two geologic studies form the basis of the background information for this Phase 1 Site Characterization program.

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 in the Figure 2.2-1 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 famsl. The potential axis of the main dam is shown in Figure 2.4-1 and is oriented approximately N47E. A 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 dam when reservoir storage is in excess of about elevation 670 to 680 famsl. The total crest length of this largest 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.
4 Phase 1 Site Characterization Program

4.1 INTRODUCTION
The scope of the Phase 1 Site Characterization program for the potential Chehalis Dam 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.

4.2 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 4-1).
Three pioneer roads were built for the project. Roads B and C were built off of the City of Pe Ell’s access road in the left (west) abutment area for access to BH-1 and to BH-4/BH-5/BH-3, respectively. Road B was about 400 feet long and Road C was about 1,100 feet long. In the upper right (east) abutment, Road A was constructed for access to BH-6 and extended 220 feet off the Weyerhaeuser main line road. The access road on the east abutment was graded by the excavator and did not require a corduroy surface. Approximate access road locations are shown in Figure 2.4-1.

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. During the subsurface exploration program the bearing capacity of the City of Pe Ell’s access road was damaged. To repair the road, about 1 foot of subbase was placed on the existing road surface and a thin drivable layer was placed on top of the subbase. Karnas finished the restoration of the Pe Ell access road and the pioneer roads after the subsurface explorations were completed.

4.3 RESERVOIR LANDSLIDE MAPPING

Landslides were previously identified using existing landslide inventories, aerial photographs, LiDAR hillshade images, and existing geologic maps. The purpose of the Phase 1 landslide study was to estimate the potential impacts of construction and fluctuating reservoir levels on the stability of the landslides.

The 25 landslides identified in the 2014 desktop study were visited by a two-geologist team between March 10 and 19, 2015. Permission to enter the land was granted by the Weyerhaeuser Company and Mr. Vincent Panesko.
Field reconnaissance included signs of unstable ground and other features characteristics of landslide masses:

- Earth cracks or fissures
- Springs
- Hummocky ground surface
- Toe bulging
- Diagnostic tree species
- Split trees
- Tree bowing or tilting
- River/creek undercutting
- Unusual and diagnostic drainage patterns
- Rock and soil exposures

The updated landslide study is discussed in more detail in Section 5 and presented in Appendix A.

4.4 DAM SITE SUBSURFACE CHARACTERIZATION

The Phase 1 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 Site Characterization program included both drilling and geophysical survey methods. Six boreholes were advanced during the exploration program to identify the subsurface stratigraphy, collect soil and rock samples for laboratory testing, perform in-situ water pressure tests to characterize the hydraulic conductivity in the bedrock, perform down-hole geophysical surveys, and install instrumentation. Three of these boreholes were located to characterize the potential alignment of the dam axis and three boreholes were designed to characterize the known landslides in the left abutment within and immediately upstream of the potential dam foundation area. Borehole locations are shown in Figure 2.4-1.

In addition to completing the boreholes, seismic refraction geophysical survey methods were used along five lines along the ground surface, shown on Figure 2.4-1. The seismic refraction surveys were performed to characterize the subsurface between boreholes and support estimates of the probable depth of excavation for the dam. The five seismic refraction lines had a total length of 4,600 feet.

Optical and acoustic down-hole televiwers were used to provide rock structure data in each borehole. In addition, sonic velocity data were obtained in each borehole via sonic suspension logging. 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 5.2 and 5.3.

4.4.1 BOREHOLES

The explorations were drilled by Holt Services using mud rotary drilling techniques in the overburden and HQ-3 core drilling techniques in rock. 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 triple-tube rock coring methods were used to advance the boreholes into bedrock. The results of the drilling and sampling are presented in the borehole logs in Appendix B and discussed in Section 5.2.1.

4.4.1.1 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.

4.4.1.2 HQ-3 Triple-tube Rock Coring
Rock core samples were obtained using a 5-foot-long, HQ-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 0.65 foot were required to improve recovery where low-rock-quality rock material was encountered.

4.4.1.3 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. The results of the SPTs are provided on the exploration logs included in Appendix B.

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. Photographs of the rock core are presented in Appendix B. Select core samples were designated for laboratory testing as is discussed in Section 4.6. Laboratory test results are presented in Appendix E.
4.4.2 WATER PRESSURE TESTING
Water pressure tests were performed in all six boreholes to estimate in-situ hydraulic conductivity and to evaluate the potential groutability of the subsurface bedrock stratigraphy beneath the potential dam alignment.

Water pressure tests were performed using a double packer test apparatus, a water pump, and clean water obtained from the City of Pe Ell. After drilling of each borehole was completed, water pressure testing was performed at 10-foot intervals from the 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 provide 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 presented in Appendix D and discussed in Section 5.2.2.

4.4.3 GEOPHYSICAL INVESTIGATIONS
The geophysical survey was conducted by Global Geophysics between February and April 2015. The methodology and instrumentation used are summarized in the following sections. A detailed report is presented in Appendix C. The results of the geophysical investigation are discussed in Section 5.3.

4.4.3.1 Seismic Refraction
The velocity with which seismic waves travel through rock indicates the competency and rippability of the rock and can also indicate the degree of fracturing and weathering present in the rock mass. Seismic refraction 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 survey was 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 were collected and saved in digital format to enable real-time quality assurance and quality control of the data. The details of the procedure are presented in Appendix C.

4.4.3.2 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. Accurate borehole deviation data were obtained during logging with a precision 3-axis magnetometer and two accelerometers. During post-processing, the video image is unwrapped and displayed as a simulated core sample. This image was analyzed for natural fractures, fracture type, and orientation and was compared with the actual core samples obtained from the boreholes. Where core loss and core damage occurred, the televiewer results were used to fill in information missing from the core logs.

4.4.3.3 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. The compressional and shear wave velocities are used to correlate the stratigraphy found in the boreholes with the surface seismic refraction survey results and assist with characterizing the rock mass properties and the subsurface lithology.
4.5 ROLLER-COMPACTED CONCRETE 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, including two commercial sources (Alderbrook and Hope Creek Quarries), an inactive Weyerhaeuser quarry (Rock Creek), and rock from the dam site. Those four sites were visited and initial laboratory testing was completed.

Because of mixed results in the 2014 study, the same four sites were revisited in March 2015 and additional samples were taken. In addition to conducting the same tests performed in 2014 (ASR 16-day test, specific gravity, and absorption), LA Abrasion testing and petrographic analysis were performed.

The ASR, specific gravity, absorption, and LA Abrasion tests were performed by the Lafarge North America concrete lab in Seattle, Washington. The ASR test results presented in this report are for 16-day tests. One-year ASR tests are underway and the results will be reported in March 2016. The petrographic analysis was carried out by Spectrum Petrographics, Inc., in Vancouver, Washington.

4.6 OTHER LABORATORY TESTING

Other laboratory testing was performed on selected samples from the overburden and rock core from the subsurface boreholes. Index testing consisting of six Atterberg limits determinations, and eight combined sieve analyses were performed to characterize the overburden material. The index testing was performed by Shannon & Wilson, Inc., in Seattle, Washington.

Strength testing was performed on four rock core samples of basalt lithology. Slake durability tests were performed on four siltstone samples. The strength testing and the slake testing were performed by GeoTesting Express of Acton, Massachusetts. Three rock core samples were sent to Spectrum Petrographics for petrographic analysis.

The laboratory test procedures are summarized in Section 5.4. The laboratory test results are presented in Appendix E.

4.7 SEISMIC HAZARDS

The Phase 1 Seismic Hazard Analysis was performed to identify potential ground motion and fault rupture hazards at the Chehalis Dam site. This analysis was based primarily on a review of published literature, inquiry of select authors of peer-reviewed literature, and preliminary deterministic seismic hazard analysis (DSHA) and probabilistic seismic hazard analysis (PSHA). Results of the USGS National Seismic Hazard Mapping Project (NSHMP) PSHA (Petersen et al. 2014) were reviewed. The NSHMP national study does not include some of the potentially active faults near the site. To assess the hazard posed by these faults, the recently peer-reviewed PSHA models by Shannon & Wilson, Inc. (e.g., Shannon & Wilson, Inc. 2009, 2013, and 2014), were modified.

Field explorations used in the seismic hazard analysis were limited to the rock geophysical shear wave velocity measurements (described in Section 4.4.2) that estimate the time-averaged shear wave velocity in the upper 30 meters of the site ($V_{s30}$ [average shear velocity down to 30 meters]) and corresponding National Earthquake Hazards Reduction Program (NEHRP) site class. No field investigations of potentially active faults were performed for Phase 1.
5 Exploration Program Results

5.1 GEOLOGIC MAPPING

During the Phase 1 Site Characterization program, two separate mapping efforts were made. Shannon & Wilson, Inc., performed landslide mapping in the dam footprint and reservoir area to refine the 2014 desktop study. HDR located several rock outcrops at the dam site and performed joint surveys to characterize the bedrock structure. In addition, HDR used subsurface data obtained from boreholes to further evaluate rock structure characteristics and refine the geologic map presented by Shannon & Wilson, Inc. (2009). The following subsections present updated geologic mapping information.

5.1.1 RESERVOIR AND DAM SITE LANDSLIDES

Twenty-five potential landslide areas were preliminarily identified during the 2014 desktop study. During the Phase 1 Site Characterization program, these locations were checked in the field by walking the landslides. Updated mapping information including the details of each landslide is presented in Appendix A. During the reconnaissance, Global Positioning System (GPS) points were recorded at significant locations, and photographs were taken to record observations. The GPS points and select illustrative photographs are also presented in Appendix A.

As a result of the 2015 field reconnaissance, many of the 2014 landslide polygon shapes were modified. Their revised shapes are shown in Figures 1 and 2 of Appendix A. Two of the landslides in the 2014 desktop study were found to be erosional landforms, not landslides. The other 23 landslides were classified as deep-seated (14), shallow rapid (8), or debris flow (1).

5.1.2 DAM SITE BEDROCK STRUCTURE

Two separate site visits were made at the end of March 2015 and the middle of April 2015 to collect discontinuity data at four different bedrock outcrops at or near the dam site. Detailed logging of each discontinuity found in the rock core obtained from boreholes was performed and used to calculate Rock Mass Rating (RMR) values as discussed in Section 5.2.1. Four outcrop locations were found for mapping of joint sets. These outcrop locations are indicated in Figure 2.4-1. All of the outcrops consisted of Crescent Formation basalt and are shown on Photographs 5-1 to 5-4 below. Joint strike measurements were converted to the equivalent dip direction and all discontinuity orientations were imported into the Dips 6.0 program from Rocscience to evaluate joint sets.

The stereonet produced in Dips is shown on Figure 5.1-1. Three prominent joint sets labeled Joint Set (JS)-1, JS-2, and JS-3 were identified in the surface outcrop areas and the strikes, dip direction, and dip for each joint set are shown in Table 5-1. 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. Contours of density concentrations shown in Figure 5.1-1 represent the dip poles that are oriented 90 degrees to the location where the dip vectors intersect the lower hemisphere of the stereo net. 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 shown on this figure relative to the orientation of the dam axis.
Table 5-1
Summary of Identified Joint Sets from Outcrop Mapping

<table>
<thead>
<tr>
<th>JOINT SET</th>
<th>STRIKE</th>
<th>DIP DIRECTION</th>
<th>DIP</th>
</tr>
</thead>
<tbody>
<tr>
<td>JS-1</td>
<td>75</td>
<td>165</td>
<td>70</td>
</tr>
<tr>
<td>JS-2</td>
<td>348</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td>JS-3</td>
<td>271</td>
<td>361</td>
<td>18</td>
</tr>
</tbody>
</table>

Photograph 5-1
Outcrop 1, upstream of the Dam and on the western bank of the Chehalis River
Photograph 5-2
Outcrop 2, along the western bank of the Chehalis River within the Dam footprint
Photograph 5-3
Outcrop 3, upstream of the Dam and on the eastern bank of the Chehalis River
5.1.3 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 by Shannon & Wilson, Inc., in 2009. The result is the geologic map shown in Figure 2.4-1 and associated geologic cross-sections shown in Figures 5.1-2 to 5.1-4.

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 110 feet at the toe of the largest landslide in the area, and are typically about 30 to 40 feet thick.

No evidence of active faults 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, see Sections 5.5 and 6.2 on seismic hazards.

5.1.4 DESCRIPTION OF GEOLOGIC UNITS

The following descriptions are based on previous studies of the area and geologic maps as mentioned in Section 3 and the material descriptions made on the borehole logs found in Appendix B. The two- to three-letter abbreviations of the geologic units are used on the geologic maps and cross sections shown in Figures 2.4-1 and 5.1-2 to 5.1-4. Soil classifications (e.g., MH) are according to the Unified Soil Classification System (USCS, ASTM D 2487-11).

5.1.4.1 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 (Shannon & Wilson, Inc. 2009). 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 1 study. Qa is present only in active stream channels.

5.1.4.2 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 (Shannon & Wilson, Inc. 2009). Colluvium was not directly observed in any of the boreholes during this Phase 1 study. Seismic refraction surveys suggest that these materials range from 0 to 12 feet thick along the upper slope of the right abutment near BH-6.
5.1.4.3 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 (Shannon & Wilson, Inc. 2009). 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 QIs 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 BH-1 to a depth of 35 feet, BH-4 to a depth of 62.5 feet, and the upper 18.5 feet of BH-3. 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 appear to be deep-seated failures with a maximum thickness of 40 and 110 feet, respectively.

5.1.4.4 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 in BH-3 between 18.5 and 35 feet, BH-5 to a depth of 47.5 feet, and BH-6 to a depth of 32.5 feet. Qos consists of medium dense to very dense, brown to dark gray silt with sand (ML), silty sand (SM), and silty gravel (GM). The fines in the Qos tend to be non-plastic to low plasticity; however, elastic silt occurs at a depth of 20 feet in BH-3 according to index lab testing discussed in Section 5.4.1. Overburden soils frequently contain highly weathered clasts of bedrock that are angular to subangular. Iron staining was also observed frequently in the Qos.

5.1.4.5 McIntosh Formation
The McIntosh Formation (Tml) represents a thick sequence of locally tuffaceous marine siltstone and claystone with interbedded arkosic sandstone and basaltic sandstone (Shannon & Wilson, Inc. 2009). In the vicinity of the dam site, only claystone was observed in BH-6. The claystone is very weak to weak, gray, very fine grained, rough to slickensided, with very close to medium-spaced low to high angle joints with mineral and clay infilling, slightly weathered with cross-bedded sandy siltstone interbeds. Two basalt intrusions ranging from 0.5 to 1.1 feet thick were observed within the McIntosh claystone in BH-6.

5.1.4.6 Intrusive Igneous Volcanics
Intrusive volcanic rocks (Tig) in the vicinity of the dam site consist primarily of gabbro; a 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 (Shannon & Wilson, Inc. 2009). During the Phase 1 site investigation, Tig was only encountered in BH-4. This material 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.

5.1.4.7 Crescent Formation Basalt
The Crescent Formation (Tcb) is characterized by massive basalt flows, pyroclastic flows, and tuffaceous sandstones. Crescent basalts are often in the form of pillow basalt flows but can also be locally intrusive. Several sequences of volcanism occurred during the deposition of Crescent basalts resulting in interbeds of siltstone and claystone as described in the next unit (Moothart 1992). The Crescent 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 clay and mineral 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.

5.1.4.8 Crescent Formation Siltstone

In between basalt flows, local volcanic rocks weathered and were eroded and deposited as silt and clay interbed lenses, ultimately becoming siltstone (claystone) within the Crescent Formation (Tcs). These materials have similar characteristics to the McIntosh formation. The Tcs (Tertiary Crescent Formation siltstone/claystone) encountered in the boreholes are very weak to moderately strong, dark gray to black, very fine to fine grained, with smooth to rough, closely to moderately spaced, low to high angle joints, and 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. By matching up the tops of siltstone layers within the nearby boreholes, a general orientation of the formation bedding was found to have about a 260 degree (N100W) strike and a dip of N14 degrees. Crescent Formation siltstone (claystone) occurred in every borehole. This material may have acted as a weak slide plane by which the large landslide upstream of left abutment failed as shown in Figure 5.1-4.

5.2 BOREHOLE LITHOLOGY

5.2.1 DRILLING RESULTS

Details of the completed boreholes are summarized in Table 5-2. The subsurface conditions identified in each borehole are summarized in Table 5-3. This information was obtained from the borehole logs presented in Appendix B. The borehole stratigraphy and the geophysical data were used to develop a geologic interpretation of the dam site; the results of this interpretation are shown along cross-sections A-A’ through D-D’ in Figures 5.1-2 to 5.1-4. Figure 5.1-2 shows the main dam axis alignment in plan and profile with estimated excavation limits, discussed in Section 6.3.1, and approximate grout target zones, discussed in Section 6.3.2. Figure 5.1-3 shows the saddle dam axis alignment also with estimated excavation limits and approximate grout target zones. No boreholes were completed in the saddle dam alignment area during this phase of site characterization. Therefore, the subsurface interpretation along the saddle dam alignment was inferred from information obtained in nearby BH-6, the seismic refraction survey line performed along the saddle dam alignment, and our general understanding of the site geology. Figure 5.1-4 shows a cross section through the main dam (C-C’) along seismic refraction survey line 3 and across both landslides in the left abutment. Figure 5.1-4 also shows cross section (D-D’) through the large landslide in the left abutment along seismic refraction survey line 5. Data presented for each borehole on these figures has a graph showing RMR and Rock Quality Designation (RQD) percent along the upper abscissa and Lu values determined from Hydraulic Conductivity testing in the boreholes (0-70) along the lower abscissa.
## Table 5-2
**Summary of Completed Borehole Information**

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<th>BOREHOLE NUMBER</th>
<th>LOCATION</th>
<th>EASTING$^1$</th>
<th>NORTHING$^1$</th>
<th>ELEVATION$^2$ (FT)</th>
<th>DEPTH (FT)</th>
<th>SOIL/WEATHERED ROCK DEPTH (FT)</th>
<th>ANGLE OF HOLE FROM HORIZONTAL (DEGREES)</th>
<th>NUMBER OF WATER PRESSURE TESTS</th>
<th>GROUNDWATER$^4$ DEPTH (FT)</th>
<th>ELEVATION (FT)</th>
<th>VIBRATING WIRE PIEZOMETER DEPTH (FT)</th>
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<td>457.5</td>
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Notes:
2. Elevation from LiDAR topography.
3. Failed tests include: failed packer seal, by-pass, unachievable test pressures and no discernable takes (see Section 5.2.2).
4. Groundwater measurements were taken on May 1, 2015.
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Notes:
Datum: WGS84
Coordinate System: State Plane 4602 US Survey Feet
RMR and RQD for each borehole are discussed in the following sections, and Lu values for each borehole are discussed in Section 5.2.2.

The RMR system was developed by Bieniawski in 1976 and updated significantly in 1989. The system is designed to provide a rating system of the geomechanical properties of a rock mass according to such factors as strength, RQD, and spacing and condition of discontinuities. RQD was developed by Deere (Deere et al. 1967) to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100 millimeter (mm; 4 inches) in the total length of core (Bieniawski 1989). Data collected for each borehole by Shannon & Wilson, Inc., and HDR during the site investigation was input into a spreadsheet to evaluate RMR and RQD for each core, the results of which are shown in Figures 5.2-1 and 5.2-2.

The Intrusive Volcanics (Tig) Formation was found only in BH-4 and the McIntosh Formation (Tml) was found only in BH-6. The remainder of the bedrock materials encountered in the boreholes consisted of basalt and claystone that is part of the Crescent Formation basalt (Tcb). Specifically, these materials were encountered as alternating sequences of pillow basalt (Tcb) deposition, and weathering and erosional events of the pillow basalt to clays deposited within depressions that were lithified to claystone (Tcs) 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).

5.2.1.1 Main Dam Boreholes
BH-2, BH-5, and BH-6 were performed close to the main dam axis alignment and were used to interpret the subsurface stratigraphy along with seismic refraction survey line 1, spreads 1, 2, and 5. General lithology, along with RMR, RQD, and Lu value plots versus depth for the main dam boreholes, are shown in Figure 5.1-2. Detailed plots of the RMR, RQD, and Lu values are presented in Figure 5.2-1. All boreholes were projected to the main dam axis alignment and therefore do not match the ground surface and interpreted geology due to differences in topography and the bedding/contact dip and dip direction that were considered during the geologic interpretation. The general dip direction of the siltstone interbeds (Tcs) is N10W (strike 260 degrees) with a dip of N14 degrees. This results in an apparent dip of about 8 degrees along the main dam axis cross section toward the right abutment. With the exception of a small portion of the McIntosh Formation (Tml) at the top of the slope in the upper right abutment area, the subsurface lithology beneath the overburden consists entirely of Crescent Formation basalt (Tcb) and siltstone (Tcs).

5.2.1.2 BH-2
BH-2 was advanced near the potential maximum section of the main dam to a depth of 241 feet. As a result, this borehole reached the lowest elevation of the drilling program and represents the lower limit of the geologic interpretation. The overburden in BH-2 is 30 feet deep and interpreted as older alluvium (Qao) due to its deposition on a fluvial terrace more than 15 feet above the modern river channel. Low values for RMR and RQD in BH-2 are associated with the siltstone interbeds at depths of 81 to 131 feet, 136 to 138 feet, and 212 to 241 feet, since the siltsone is a weaker rock than the surrounding basalt. The RMR of the bedrock encountered in BH-2 ranges from poor to very good with the siltstone interbeds generally fair and the majority of the basalt as good.

5.2.1.3 BH-5
BH-5 was advanced on the slope of the left abutment of the main dam to a depth of 250 feet with 47.5 feet of overburden. As with BH-2, low values of RMR and RQD correlate with the weaker siltstone
beds found at depths of 48 to 70 feet and 146 to 181 feet. There is an exception, however, of a zone of low RMR/RQD between a depth of 90 and 92 feet within the basalt that has a slightly higher concentration of fractures than the surrounding basalt. The RMR of the rock core from BH-5 ranges from poor to very good with the siltstone interbeds ranging from poor to fair and all of the basalt as good with the exception of the 90- to 92-foot interval.

5.2.1.4 BH-6

BH-6 was advanced above the steep slope that makes up the right abutment to a depth of 350 feet. Claystone of the McIntosh Formation was encountered immediately beneath the overburden soil at a depth of 32.5 feet. This rock was so fractured and weak that rock coring did not commence until a depth of about 40 feet. Rock core data obtained from 40 to 61 feet in the McIntosh Formation demonstrates high RQD and fair RMR, suggesting the McIntosh claystone is slightly more competent than the siltstone interbeds of the Crescent Formation. The RMR below a depth of 61 feet is consistently fair with a few occasions of good condition without an appreciable difference in the siltstone interbeds. This suggests that the basalt beneath the hillslope in the right abutment is slightly less competent than the basalt encountered in BH-2 and BH-5. Note that BH-6 was projected a considerable distance (137 feet) to the main dam axis alignment.

5.2.1.5 Landslide Boreholes

BH-1 was completed in the center of the smaller landslide mapped in the left abutment dam footprint area to evaluate the depth of landslide deposits and possibly identify the failure plane. BH-3 and BH-4 were completed along the axis of the larger landslide upstream of the left abutment area. BH-1 and BH-4, in addition to obtaining landslide information, allow interpretation of a cross-section through the main dam as shown in Figure 2.4-1; cross-section C-C’. General lithology along with RMR, RQD, and Lu value summary plots versus depth for the landslide boreholes are shown in Figure 5.1-4. Detailed plots of the RMR, RQD, and Lu values are presented in Figure 5.2-2 and in Appendix D.

5.2.1.6 BH-1

BH-1 was advanced to a depth of 140 feet. The landslide plane was identified as the contact between soil and rock at a depth of 35 feet; however, no weak layer responsible for the landslide failure was identified in the core. Below the landslide materials, the RMR is relatively high, remaining mostly in the good range with a few exceptions as shown in Figures 5.2-2. There is no appreciable difference in the RMR between the siltstone layers encountered at depths of 102 to 112 feet and 125 to 140 feet and the surrounding basalt.

5.2.1.7 BH-3

BH-3 was advanced to a depth of 150 feet and had 35 feet of overburden soil. It is difficult to define the landslide failure plane within the overburden soil; however, the borehole log indicates the presence of cobbles and boulders in the overburden soil near a depth of 18.5 feet. The presence of cobbles and boulder are likely the result of the landslide. Beneath the landslide and overburden soil, the bedrock RMR consistently fall in the fair and good categories with no major difference shown in the siltstone lens found in the interval from 133 to 150 feet.

5.2.1.8 BH-4

BH-4 was advanced to a depth of 120 feet and had 62.5 feet of overburden soil. The slide plane is difficult to determine from the borehole information. However, a notable increase of both RMR and RQD values occurred at a depth of 110 feet as shown in Figure 5.2-2. Therefore, the failure plane is interpreted to pass through the relatively weak siltstone interbed as shown in Figure 5.1-4. BH-4 is the
only borehole where intrusive volcanic rocks (Tig) were logged; it is therefore interpreted that the intrusion approached from the south as indicated in the geologic map shown in Figure 2.4-1. Further, the northern margin of the intrusion may have created a plane of weakness between the surrounding basalt (Tcb) and intruding gabbro (Tig) on which the landslide failed. Additional characterization of this landslide will be required to better understand the limits of the landslide and influence on the dam design.

5.2.2 IN-SITU HYDRAULIC CONDUCTIVITY TESTING RESULTS

Water pressure tests to evaluate in-situ hydraulic conductivity were performed as summarized in Section 4.4.2 after each borehole was completed. Tests were performed using an apparatus consisting of double 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. The tests were performed at continuous intervals from the bottom of the borehole to the surface.

Ninety-eight tests were attempted throughout all six boreholes. However; the full 5-step Houlsby test method was only performed in 14, or 14 percent, of the total tested intervals. A number 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

Of the 98 attempted tests about 52 percent were not considered valid tests. Any other data gathered, including partially completed Houlsby sequences, were considered in the evaluation of hydraulic conductivity.

Table 5-4 contains a summary of the average estimated hydraulic conductivities by depth interval and lithology. Figure 5.2-3 shows the same data in a scatter plot so the range and variability of the test results can be seen. Both the siltstone and basalt tend to have higher hydraulic conductivity with depth. A wide range and large variability occurs for both basalt and siltstone, especially among the bedrock found at depths ranging from about 90 to 130 feet.

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>AVERAGE IN-SITU HYDRAULIC CONDUCTIVITIES (CENTIMETERS PER SECOND)</th>
</tr>
</thead>
<tbody>
<tr>
<td>From</td>
<td>To</td>
</tr>
<tr>
<td>0</td>
<td>25</td>
</tr>
<tr>
<td>25</td>
<td>52</td>
</tr>
<tr>
<td>52</td>
<td>72</td>
</tr>
<tr>
<td>72</td>
<td>92</td>
</tr>
<tr>
<td>92</td>
<td>112</td>
</tr>
<tr>
<td>112</td>
<td>132</td>
</tr>
<tr>
<td>132</td>
<td>152</td>
</tr>
<tr>
<td>DEPTH</td>
<td>AVERAGE IN-SITU HYDRAULIC CONDUCTIVITIES (CENTIMETERS PER SECOND)</td>
</tr>
<tr>
<td>-------</td>
<td>---------------------------------------------------------------</td>
</tr>
<tr>
<td>152</td>
<td>172 -- -- 1.51E-05 2</td>
</tr>
<tr>
<td>172</td>
<td>192 4.42E-04 1 4.95E-04 2</td>
</tr>
<tr>
<td>192</td>
<td>212 -- -- 2.22E-04 4</td>
</tr>
<tr>
<td>212</td>
<td>262 9.51E-04 1 7.48E-05 10</td>
</tr>
<tr>
<td>262</td>
<td>312 -- -- -- --</td>
</tr>
<tr>
<td>312</td>
<td>350 -- -- -- --</td>
</tr>
<tr>
<td>Overall Average</td>
<td>2.74E-04 18 1.09E-04 47</td>
</tr>
</tbody>
</table>

Lugeon values are more commonly used when evaluating groutability and were calculated from hydraulic conductivity measurement. Lugeon 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). Plots of Lu values are shown versus depth and elevation for each borehole in Figures 5.2-1 and 5.2-2 and also shown on the cross sections in Figures 5.1-2 to 5.1-4. 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, shown in Figure 5.2-3:

- 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 5.2-3. Additional discussion related to groutability is provided in Section 6.3.2.

5.3 GEOPHYSICAL INVESTIGATION RESULTS

5.3.1 SEISMIC REFRACTION SURVEYS

Seismic refraction survey lines were completed by Global Geophysics as discussed in Section 4.4.3.1, and the results are presented in Appendix C. The data were processed to show colored contours of compression wave velocity with depth beneath the ground surface. An example of the compression wave velocity measurements along spreads 1 and 2 of Seismic Line 1 across the left abutment axis of the dam is shown in Figure 5.3-1. Evaluation of the compression wave velocity with depths versus the measured depth of overburden soil and weak rock in the boreholes suggests that a compression wave velocity ranging from a low as 7,200 to about 10,000 feet per second (fps) represents the expected interval for a suitable excavation objective across the dam site. Velocities in the lower end of this range will likely be suitable in the upper abutment areas, whereas velocities in the upper end of this range will likely be required where the height of the dam exceeds 150 to 200 feet.

By establishing a compression wave velocity correlating to the approximate excavation limits, zones of weak or highly fractured rock that will require localized over-excavation or other foundation treatment methods can also be identified. Examples of two such possible locations are labeled in Figure 5.1-2. The seismic refraction survey data were also used to estimate depths of overburden between boreholes and
has been reflected in all cross sections presented in Figures 5.1-2 to 5.1-4. The overall average overburden thickness is interpreted as about 35 feet with the exception of the colluvium on the slopes of the right abutment, which is considerably thinner. The upper weak rock to be excavated has an average thickness of about 15 feet with considerable variation within the potential dam footprint. Additional discussion of locations where over-excavation or other treatments may be required are described in Section 6.3.1. The general depth of excavation is expected to be up to about 50 feet and locally deeper where significant lower-quality rock is found.

The Phase 1 site characterization results represent a first set of information to assess the suitability of the site for an RCC dam configuration and to make initial estimates of the depth of excavation required to reach a rock of sufficient strength and deformability for a large concrete structure. The seismic refraction results, combined with the rock conditions observed in the borehole, have confirmed the feasibility of the RCC dam type. The results have further indicated the variable nature of the rock and the potential for localized conditions that may require supplemental treatments to produce adequate bearing conditions for the dam.

5.3.2 OPTICAL/ACOUSTIC TELEVIEWER RESULTS

Borehole acoustic and digital optical televiewer data were collected by Global Geophysics between February 24 and April 23, 2015. A report presenting the methodology and results is provided in Appendix C.

Televiewer data were collected in BH-1, BH-2, BH-3, BH-4, BH-5, and BH-6 starting at the bottom of the borehole and continuing to a location near the top of rock where the borehole casing prevented further survey work. Borehole logs are presented in a format that begins at the top of the survey interval to the bottom of the hole. Optical surveys were performed when visibility conditions permitted. Otherwise, the surveys were completed using acoustical methods. The results of the acoustic and optical televiewer data were combined into a single log of each borehole. The information shows the walls of the boreholes unwrapped to a flat surface. The data has been transformed and plotted to represent an equivalent rock core sample for comparison with the actual core samples and detailed log of the borehole. An example of this information from a portion of BH-2 from a depth of about 84 to 88 feet in Crescent Formation siltstone/claystone material is shown on Figure 5.3-2.

The result of the televiewer data was 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. Joint structure data is presented as azimuth, dip and structure type on the televiewer logs. Azimuth and dips developed from the discontinuities observed in the data were plotted on Wulff Plots (example also shown on Figure 5.3-2). Structure type was presented on a scale from 0 to 6 to categorize fractures and joint types. The following Table 5-5 presents the correlating scale to joint description.
### Table 5-5

<table>
<thead>
<tr>
<th>SCALE</th>
<th>TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>No joint/fracture</td>
</tr>
<tr>
<td>1</td>
<td>Major open joint/fracture</td>
</tr>
<tr>
<td>2</td>
<td>Minor open joint/fracture</td>
</tr>
<tr>
<td>3</td>
<td>Partially open joint/fracture</td>
</tr>
<tr>
<td>4</td>
<td>Sealed joint/fracture</td>
</tr>
<tr>
<td>5</td>
<td>Bedding</td>
</tr>
</tbody>
</table>

Review of the Wulff Plots and other structural characteristics indicates a wide range of joint conditions and orientations. No specific trends or joint sets are discernable in comparison to the joint data collected from rock outcrops near the dam footprint shown in Figure 5.1-1. Fracture types were generally classified as Type 2 within the left abutment and in the maximum section BH-1, BH-2, BH-3, BH-4, and BH-5. BH-6 was completed in the upper right abutment area. Joints and fractures in the pillow basalt flows identified in BH-6 were generally classified as Type 4 to a depth of about 140 feet, transitioning to Type 2 at greater depths.

### 5.3.3 SONIC SUSPENSION LOGGING

Suspension logging was also performed by Global Geophysics between February 24 and April 23, 2015, as part of the overall Phase 1 geophysical survey program. Suspension S-P velocity logging measures compression and shear-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. Results of the sonic suspension logging are included in Global Geophysics report in Appendix C.

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 (P) and shear (S) wave components of the induced pressure wave are measured in the rock materials near the borehole. Test information is recorded in the data logger.

Global Geophysics completed suspension loggings in each of the boreholes (BH-1 through BH-6). Subsurface materials consisted of unconsolidated soil with shear wave velocities ranging from 1,000 to 1,860 fps, which overlie higher density soil/gravel or highly weathered rock with shear wave velocities ranging from 2,900 fps to 4,300 fps, which in turn overlie moderately weathered to unweathered competent rock with shear wave velocities ranging from 7,550 fps to 14,000 fps. The estimated depth to competent bedrock varied between outcropping to approximately 82 feet below the ground surface.

### 5.4 LABORATORY TESTING

A summary of the laboratory testing work completed as part of the Phase 1 program is presented in the following sections. Detailed test reports are included in Appendix E.

#### 5.4.1 DAM SITE FOUNDATION

Laboratory testing of materials from the dam foundation included representative overburden soil and bedrock samples from the test boreholes. Laboratory soil testing results are summarized in Table 5-6. Test results on samples of bedrock including bulk density, lithology classification, and unconfined
compressive strength are summarized in Table 5-7. Young’s modulus and Poisson’s ratio test results for the bedrock samples are summarized in Table 5-8. Slake durability test results on selected siltstone samples are summarized in Table 5-9. The slake durability test results indicate the potential for these materials to degrade when exposed in excavations. Appropriate treatment precautions will be required when these materials are encountered in excavations, grout holes, and foundation drainage holes.
<table>
<thead>
<tr>
<th>BOREHOLE</th>
<th>SAMPLE NUMBER</th>
<th>DEPTH (FEET)</th>
<th>GRAIN SIZE DISTRIBUTION – ASTM D422</th>
<th>ATTERBERG LIMITS – ASTM D4318</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>USCS GROUP SYMBOL</td>
<td>USCS GROUP NAME</td>
</tr>
<tr>
<td>BH-1</td>
<td>S-2</td>
<td>10</td>
<td>MH</td>
<td>Sandy Elastic Silt</td>
</tr>
<tr>
<td></td>
<td>S-4*</td>
<td>20</td>
<td>SM</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>BH-2</td>
<td>S-2*</td>
<td>10</td>
<td>SM</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>BH-3</td>
<td>S-4*</td>
<td>20</td>
<td>MH</td>
<td>Elastic Silt</td>
</tr>
<tr>
<td>BH-4</td>
<td>S-8*</td>
<td>40</td>
<td>SC</td>
<td>Clayey Sand with Gravel</td>
</tr>
<tr>
<td></td>
<td>S-10*</td>
<td>50</td>
<td>SM</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>BH-6</td>
<td>S-6</td>
<td>30</td>
<td>SM</td>
<td>Silty Sand</td>
</tr>
</tbody>
</table>

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
### Table 5-7
Rock Laboratory Testing Results - Bulk Density and Compressive Strength

<table>
<thead>
<tr>
<th>BOREHOLE</th>
<th>DEPTH (FT)</th>
<th>LITHOLOGY</th>
<th>BULK DENSITY (POUNDS/CUBIC FEET)</th>
<th>COMPRESSIVE STRENGTH (PSI)</th>
<th>ASTM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FROM</td>
<td>TO</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-1</td>
<td>45.76</td>
<td>46.2</td>
<td>Basalt - Tcb</td>
<td>171</td>
<td>15,683</td>
</tr>
<tr>
<td></td>
<td>76.22</td>
<td>76.66</td>
<td>Basalt - Tcb</td>
<td>174</td>
<td>20,663</td>
</tr>
<tr>
<td>BH-2</td>
<td>41.7</td>
<td>42.14</td>
<td>Basalt - Tcb</td>
<td>171</td>
<td>14,043</td>
</tr>
<tr>
<td></td>
<td>62.15</td>
<td>62.59</td>
<td>Basalt - Tcb</td>
<td>173</td>
<td>26,083</td>
</tr>
<tr>
<td>BH-5</td>
<td>96.5</td>
<td>96.94</td>
<td>Basalt - Tcb</td>
<td>170</td>
<td>12,728</td>
</tr>
<tr>
<td></td>
<td>123.5</td>
<td>123.94</td>
<td>Basalt - Tcb</td>
<td>170</td>
<td>15,345</td>
</tr>
<tr>
<td>BH-6</td>
<td>105.06</td>
<td>105.5</td>
<td>Basalt - Tcb</td>
<td>174</td>
<td>17,751</td>
</tr>
<tr>
<td></td>
<td>213.86</td>
<td>214.3</td>
<td>Basalt - Tcb</td>
<td>171</td>
<td>13,786</td>
</tr>
</tbody>
</table>

Note: psi = pounds per square inch

### Table 5-8
Rock Laboratory Testing Results – Young’s Modulus and Poisson’s Ratio (ASTM D7012-Method D)

<table>
<thead>
<tr>
<th>BOREHOLE</th>
<th>DEPTH (FEET)</th>
<th>LITHOLOGY</th>
<th>STRESS RANGE (PSI)</th>
<th>YOUNG’S MODULUS (X10^6 PSI)</th>
<th>AVG. YOUNG’S MODULUS (X10^6 PSI)</th>
<th>POISSON’S RATIO</th>
<th>AVG. POISSON’S RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FROM</td>
<td>TO</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-1</td>
<td>76.22</td>
<td>76.66</td>
<td>Basalt - Tcb</td>
<td>2100</td>
<td>9.410</td>
<td>8.933</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>7600</td>
<td>13100</td>
<td></td>
<td>9.360</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>13100</td>
<td>18600</td>
<td></td>
<td>8.030</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-2</td>
<td>62.15</td>
<td>62.59</td>
<td>Basalt - Tcb</td>
<td>2600</td>
<td>11.000</td>
<td>10.147</td>
<td>0.29</td>
</tr>
<tr>
<td></td>
<td>9600</td>
<td>16500</td>
<td></td>
<td>10.300</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16500</td>
<td>23500</td>
<td></td>
<td>9.140</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-5</td>
<td>123.5</td>
<td>123.9</td>
<td>Basalt - Tcb</td>
<td>1500</td>
<td>8.370</td>
<td>8.100</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>5600</td>
<td>9700</td>
<td></td>
<td>8.120</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>9700</td>
<td>13800</td>
<td></td>
<td>7.810</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-6</td>
<td>105.06</td>
<td>105.5</td>
<td>Basalt - Tcb</td>
<td>1800</td>
<td>8.340</td>
<td>7.600</td>
<td>0.29</td>
</tr>
<tr>
<td></td>
<td>6500</td>
<td>11200</td>
<td></td>
<td>7.900</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>11200</td>
<td>15800</td>
<td></td>
<td>6.560</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: psi = pounds per square inch
Table 5-9
Rock Laboratory Testing Results - Slake Durability (ASTM D4644)

<table>
<thead>
<tr>
<th>BOREHOLE</th>
<th>DEPTH (FT)</th>
<th>LITHOLOGY</th>
<th>SLAKE DURABILITY</th>
<th>WATER TEMP. AVG. °C</th>
<th>AS-RECEIVED WATER CONTENT</th>
<th>DESCRIPTION OF FRAGMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FROM</td>
<td>TO</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-1</td>
<td>108.0</td>
<td>109.0</td>
<td>Siltstone - Tcs</td>
<td>84.7</td>
<td>22</td>
<td>11.8</td>
</tr>
<tr>
<td>BH-2</td>
<td>84.0</td>
<td>85.2</td>
<td>Siltstone - Tcs</td>
<td>38.4</td>
<td>23</td>
<td>13.5</td>
</tr>
<tr>
<td>BH-5</td>
<td>58.0</td>
<td>59.3</td>
<td>Siltstone - Tcs</td>
<td>56.0</td>
<td>22</td>
<td>11.2</td>
</tr>
<tr>
<td>BH-6</td>
<td>56.5</td>
<td>57.5</td>
<td>Siltstone - Tml</td>
<td>10.7</td>
<td>22.0</td>
<td>13.6</td>
</tr>
</tbody>
</table>

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

5.4.2 RCC AGGREGATE TESTING

Laboratory testing was performed on representative samples obtained from three candidate quarries in the general vicinity of the potential Chehalis dam site and a sample of rock obtained from the east abutment of the dam site. Results of this testing are summarized in Table 5-10. This table includes criteria from WSDOT 2014 Standard Specifications Sections 9-03.1(1) for LA Abrasion, 9-13 for Bulk Specific Gravity, and 9-13.6 for Absorption, and Federal Highway Administration 2014 Standard Specifications for the Construction of Roads and Bridges on Federal Highway Projects criteria, Section 703.02 for ASR.

Table 5-10
Aggregate Source 2015 Test Results

<table>
<thead>
<tr>
<th>QUARRY NAME</th>
<th>QUARRY OWNER</th>
<th>CONTACT NAME</th>
<th>HAUL DISTANCE (MILES)</th>
<th>ACTIVE</th>
<th>ABSORPTION</th>
<th>BULK SPECIFIC GRAVITY (SSD)</th>
<th>LA ABRASION</th>
<th>ASR (16-DAY)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alderbrook</td>
<td>Alderbrook</td>
<td>Moerke</td>
<td>24</td>
<td>Y</td>
<td>2.7</td>
<td>2.71</td>
<td>16.5</td>
<td>0.212</td>
</tr>
<tr>
<td>Hope Creek</td>
<td>Hope Creek</td>
<td>Peterson</td>
<td>10</td>
<td>Y</td>
<td>6.75</td>
<td>2.71</td>
<td>17.9</td>
<td>0.215</td>
</tr>
<tr>
<td>Rock Creek A-Line</td>
<td>Weyerhaeuser</td>
<td>Schuh</td>
<td>9</td>
<td>N</td>
<td>1.08</td>
<td>2.73</td>
<td>18.9</td>
<td>0.011</td>
</tr>
<tr>
<td>Dam Site, E. Abut.</td>
<td>Weyerhaeuser</td>
<td>Schuh</td>
<td>0</td>
<td>N</td>
<td>1.21</td>
<td>2.78</td>
<td>22.3</td>
<td>0.155</td>
</tr>
<tr>
<td>WSDOT/FHWA Criteria</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>3 max.</td>
<td>2.55 min.</td>
<td>35 max.</td>
<td>See Note 4</td>
</tr>
</tbody>
</table>

Notes:
1. WSDOT = Washington State Department of Transportation
2. FHWA = Federal Highway Administration
3. Haul distance in miles measured with Google Earth
4. 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
The laboratory tests, including petrographic examination of the test samples, were completed to further evaluate the potential for ASR. All samples were classified as altered basalt. The test results indicate the samples from Alderbrook and Hope Creek have a high potential for reactivity. The test results indicated the samples from Rock Creek and the dam site east abutment to be approximately innocuous to acceptable. Based on these results and recent advances in mix designs to mitigate ASR concerns, there appears to be a good likelihood that acceptable RCC aggregate materials can be found. However, further testing and evaluations should be completed as part of the next phase of work.

### 5.5 SEISMIC HAZARD CHARACTERIZATION

Potential seismogenic sources that could generate fault rupture and associated ground motion hazards at the site are presented in this section of the report. This includes a description of the regional seismotectonics and seismicity, followed by a description of potentially seismogenic sources within approximately 100 kilometers (km), or 60 miles, of the site. A description of parameters for select crustal faults not incorporated in the USGS NSHMP PSHA but that were considered the preliminary site-specific DSHA and PSHA is also provided.

#### 5.5.1 SEISMOTECTONICS

The northern Cascadia fore arc is positioned within two tectonic convergence regimes that deform western Washington: east-west contraction across the Cascadia Subduction Zone (CSZ) and north-south shortening from the northward migration of fore-arc blocks (Figure 5.6-1). The combined effect of the two produces complex and diverse deformation within the northern edge of the Cascadia fore arc and triggers large, damaging earthquakes from multiple seismogenic sources in western Washington. Offshore, elastic release of strain accumulated in the locked plate interface of the CSZ produces mega-thrust earthquakes (greater than magnitude \([M]\) 8.0) about every 500 years (Atwater et al. 1997; Clague 1997; Goldfinger et al. 2003 and 2012). The most recent rupture is estimated to have occurred in AD 1700 (Satake et al. 1996; Atwater and Hemphill-Haley 1997; Clague, 1997; Yamaguchi et al. 1997; Goldfinger et al. 2003 and 2012). Onshore, north-south shortening across the Puget Lowland region is accommodated by likely a dozen or more shallow faults that have ruptured through the deep glacial and post-glacial sediments that blanket the Lowland (Figures 5.6-2 and 5.6-3). Paleoseismic studies have demonstrated that as recently as about 1,000 years ago, large, surface-rupturing earthquakes occurred along several of these crustal faults, including the Seattle fault zone, the Tacoma fault zone, the Saddle Mountain fault zone, and possibly the Olympia Structure (also called the Legislature fault; e.g., Atwater and Moore 1992; Bucknam et al. 1992; Sherrod 2001; Nelson et al. 2003a and b; Johnson et al. 2004; Sherrod et al. 2013; Nelson et al. 2014; Barnett et al. 2015).

#### 5.5.1.1 Cascadia Subduction Zone – Plate Interface Mega-thrust Earthquakes

Oblique subduction of the Juan de Fuca oceanic plate beneath the North American continental plate triggers earthquakes in three seismogenic sources: at the subduction plate interface, within the subducting slab (intraslab), and within the shallow, overriding continental crust (Figure 5.6-4). At the subduction plate interface, the two plates are locked together by friction. Great mega-thrust earthquakes rupture when the frictional strength of the fault is exceeded and the fault slips (Wang et al. 2003), potentially generating M 8 to M 9 earthquakes (Atwater et al. 1997; Goldfinger et al. 2003 and 2012). Along the coast, this fault slip can trigger sudden land subsidence, strong ground shaking, tsunami inundation, and submarine landsliding. Recent paleoseismic shoreline studies have uncovered over a 5,000-year paleoseismic record of rapid land level changes and tsunamis associated with plate interface ruptures along the 620-mile length of the CSZ from Northern California to Vancouver Island (e.g., Atwater 1987 and 1992; Grant 1989; Darienzo and Peterson 1990 and 1995; Clarke and Carver 1992; Obermeier 1995; Meyers et al. 1996; Nelson et al. 1996; Peterson and Darienzo 1996; Shennan...
et al. 1996; and Atwater and Hemphill-Haley 1997). Offshore, studies have temporally correlated shaking-induced submarine landsliding deposits (turbidites) found in deep-sea channels along the entire length of the CSZ (Adams 1990 and 1996; Goldfinger et al. 2003 and 2012). As documented in the 10,000-year turbidite record, this synchronicity of slip along the fault suggests that the CSZ has ruptured along its entire length and also along fault segments for a total of 41 great earthquakes (Goldfinger et al. 2012). Although the onshore and offshore paleoseismic studies provide compelling evidence for dozens of earthquakes along the CSZ over the past 10,000 years, one as recent as AD 1700, some disagreement exists about the recurrence rate and aspects of fault segmentation (Atwater et al. 2014).

5.5.1.2 Cascadia Subduction Zone - Intraplate Earthquakes

Despite the paleoseismic history of the M 8 to M 9 earthquakes rupturing at the CSZ plate interface, the only large, damaging CSZ earthquakes recorded during the past century have occurred deep within the subducting slab, 40 to 80 km beneath the Puget Lowland (seismicity shown in Figure 5.6-3). Recent, deep M 6.0 and greater earthquakes beneath western Washington include: the M 7.1 1949 Olympia earthquake, the M 6.5 1965 Seattle-Tacoma earthquake, and the M 6.8 2001 Nisqually earthquake (Ludwin et al. 1991; http://assets.pnsn.org/HISTCAT/isoseisms.html).

These intraslab earthquakes likely occurred as a result of physical changes within the subducting slab that may produce extension and high-angle normal to oblique faulting: (1) rock embrittlement that occurs when dehydration of hydrous minerals in the slab triggers an increase in pore fluid pressure to near-lithostatic values that might facilitate brittle failure, and (2) mineralogical phase changes to denser minerals in the rocks of the subducting oceanic crust (Kirby et al. 1996; Preston et al. 2003). In addition to the physical changes within the slab, McCrory et al. (2012) propose another source of extension: internal deformation where the down-going slab arches along a northeast-southwest axis beneath the Olympic Mountains. They suggest that the clustering of deep seismicity at the northern and southern ends of the CSZ occurs where there is flexure in the arched subducting slab.

5.5.1.3 Shallow Crust Seismogenic Sources

Within the shallow upper plate, active, crustal faults accommodate regional north-south shortening in the Puget Lowland. Western Washington is subject to tectonic compression as mobile fore-arc blocks migrate northward between the Cascade volcanic arc to the east and the convergent margin of the Cascadia Subduction Zone to the west (Figure 5.6-1; Wells et al. 1998). Northeast-directed subduction of the Juan de Fuca oceanic plate at a rate of 40 millimeters (mm) per year drives the Oregon Coast fore-arc block northward, compressing western Washington against the relatively stationary backstop of the British Columbia Coast Mountains (Wells et al. 1998; Wells and Simpson 2001). Geodetic studies show that between Oregon and Canada, the fore-arc blocks are translating northward at about 6 to 8 mm per year (Wells et al. 1998; Wells and Simpson 2001; Mazzotti et al. 2002), while within the Puget Lowland, the shortening rate is 3.0 to 4.4 ± 0.3 mm per year (Khazaradze et al. 1999; Mazzotti et al. 2002; McCaffrey et al. 2007; Pollitz et al. 2010).

This contraction in western Washington buckles the fore-arc basement and overlying Quaternary and older rocks into structural highs and deep basins that are bound by active folds and faults that traverse the Puget Lowland in particular and deform the ground surface (Figure 5.6-2; Gower et al. 1985; Pratt et al. 1997; Brocher et al. 2001; Blakely et al. 2002; Nelson et al. 2003; Johnson et al. 2004; Kelsey et al. 2004; Sherrod et al. 2004; Sherrod, Mazzotti, and Haugerud 2008). Earthquake histories for these faults have emerged from paleoseismic investigations of deformed shorelines, marsh subsidence or uplift, tsunami deposits, liquefaction and mass wasting features, and fault scarps (e.g., Atwater and Moore 1992; Bucknam et al. 1992; Jacoby et al. 1992; Nelson et al. 2003a and b, 2007; Kelsey et al. 2004; Sherrod et al. 2004, 2008, 2013; Witter et al. 2008; Nelson et al. 2014), while geophysical
investigations have mapped these faults in the subsurface (e.g., Gower et al. 1985; Pratt et al. 1997; Johnson et al. 1999, 2004; Brocher et al. 2001; Blakely et al. 2002; Liberty and Pratt 2008; Blakely et al. 2009). Many of these studies have determined that as recently as about 1,000 years ago, crustal faults throughout the Puget Lowland ruptured the ground surface in large earthquakes (larger in magnitude than M 6), including: the Seattle fault zone, the Olympia fault zone, the Tacoma fault zone, the Saddle Mountain fault zone, the South Whidbey Island fault zone, the Darrington-Devils Mountain fault zone, the Boulder Creek fault zone, and the Bellingham coastal fault zone (e.g., Atwater and Moore 1992; Bucknam et al. 1992; Nelson et al. 2003a and b; Johnson et al. 2004; Kelsey et al. 2004 and 2012; Sherrod et al. 2001, 2004, 2008 and 2013; Witter et al. 2008; Personius et al. 2009; Barnett et al. 2015).

5.5.2 SEISMICITY

Although the 15,000-year paleoseismic history of the CSZ and crustal faults records indicate many earthquakes greater than M 6 have affected western Washington, the 170-year historical record chiefly includes earthquakes of low to moderate magnitude, and only occasional stronger shocks. The following subsections review both the historical seismicity and Holocene paleoseismicity.

5.5.2.1 Paleoseismicity

Sherrod and Gomberg (2014) reviewed paleoseismicity studies of shallow crustal and compiled a list of paleoearthquakes that struck western Washington before there was a local written record. Those earthquakes are listed in Table 5-11. The faults on which these paleoseismic events occurred include: the Seattle fault zone, the South Whidbey Island fault zone, the Olympia fault zone, the Boulder Creek fault zone, the Bellingham Coastal faults, the Saddle Mountain fault zone, the Lake Creek-Boundary River fault zone, and the Darrington-Devils Mountain fault zone. Sherrod and Gomberg (2014) identify 30 large earthquakes that have occurred along these faults during the past 15,000 years.

5.5.2.2 Historical Seismicity

The largest historic earthquakes felt in Washington are listed in Table 5-12. We plot these and other magnitude 3.5 or larger earthquakes located in western Washington in Figure 5.6-3. In our discussion, we include both earthquake magnitude and intensity because prior to the 1940s historical events were primarily recorded using the Modified Mercalli intensity (MMI) scale (Wood and Neumann 1931). The MMI scale reflects ground shaking effects on people and objects using a closed-end scale with values ranging from I to XII to represent the severity of ground shaking. Roman numerals are used exclusively with the MMI scale. Magnitudes reported prior to the 1940s in the Pacific Northwest are typically estimated from the MMI.

Since the 1940s, earthquakes have generally been reported using magnitude scales. Earthquake magnitudes may correspond to several different scales including surface wave magnitude (Ms), body wave magnitude (mb), and “Richter” or local magnitude. The preferred scale is the moment magnitude (MW), which is a measure of the total energy (seismic moment) released by an earthquake. Unless otherwise noted in this report, use of moment magnitude is implied. All earthquake magnitude scales use Arabic numerals to represent the size of the event.

The largest historic earthquakes to affect the project site include (Figure 5.6-3):

- April 13, 1949, magnitude (Ms) 7.1 Olympia earthquake (approximate site epicentral distance 88 km, 55 miles)
- April 29, 1965, magnitude (mb) 6.5 Seattle-Tacoma earthquake (approximate site epicentral distance 46 km, 29 miles)
• February 28, 2001, magnitude (MW) 6.8 Nisqually earthquake (approximate site epicentral distance 82 km, 51 miles)

These intraslab earthquakes occurred beneath the Puget Lowland at depths of 53, 63, and 52 km (33, 39, 32 miles), respectively. The 1949 and 2001 events occurred at nearly the same location. Ground shaking in the project area reached intensity values of VII (1949), VII (1965), and IV to V (2001), respectively (U.S. Coast and Geodetic Survey 1949 and 1967; USGS 2009). The ground shaking levels that occurred during these three events are likely the maximum vibratory ground motions that would have occurred in project area during the 170 years of historical record.

Other large historic earthquakes in the extended region include the North Cascades earthquake of December 14, 1872. The location and magnitude of this event have been uncertain owing to the sparse population of the region at that time and, until recently, the lack of identified ground surface rupture. Bakun et al. (2002) suggest that it was likely a shallow, magnitude 6.8 event with an epicenter near the southeast end of Lake Chelan. Recently, a potential fault scarp located at the southeast end of Lake Chelan was identified on LiDAR, and initial paleoseismic trenching suggests that surface rupture during the 1872 event formed this scarp (Sherrod et al. 2014; Sherrod 2015). A location near the southeast end of Lake Chelan gives an epicentral distance from the project site of 270 km (168 miles), beyond the scope of this study.

5.5.3 SEISMIC SOURCE CHARACTERIZATION

Recent peer-reviewed PSHA models of Shannon & Wilson, Inc. (2009b, 2013), were modified to include the potentially active faults located near the project site but are not included in the latest USGS NSHMP PSHA. In the modified Shannon & Wilson, Inc., PSHA model, the sources were grouped into three broad seismogenic categories:

• The interface portion of the CSZ, which produces great mega-thrust events;
• The deep intraslab portion of the CSZ (i.e., the subducted portion of the Juan de Fuca Plate); and
• The overriding North American Plate (i.e., shallow crustal sources).

The CSZ interface source was modeled as a discrete, albeit large, fault. The intraslab portion of the CSZ was modeled as an areal source zone at depth. The shallow crustal sources were modeled using areal background zones (variable seismicity rates) and as discrete faults. A description of the models and parameters for these sources is provided in the Shannon & Wilson, Inc., reports (2009b, 2013).

The previously peer-reviewed models were modified for this report by incorporating discrete sources for known or suspected shallow crustal faults located within 100 km (60 miles) of the project site (Figure 5.6-2). Evidence or postulation that these faults have produced earthquakes greater than magnitude 6.5 since the Late Pleistocene or Holocene is documented in published reports. These faults include the Olympia, Saddle Mountain, Doty, Grays Harbor, Willapa Bay, and Gales Creek fault zones.

The Doty fault zone, located 15 km (9.3 miles) north of the project site, was included in the site characterization. No field evidence for recent deformation exists, but this fault is suspected to be active (described in the following sections) and is the subject of active research (Wells, personal communication 2015). The locations of these faults as modeled in the PSHA are shown in Figure 5.6-3.

The following subsections describe the shallow crustal faults and parameters used in the DSHA and PSHA (including weighting factors for different alternative parameters considered in the PSHA) for this project site.
5.5.3.1 Olympia Structure

The Olympia Structure is an 80-km-long (50-mile-long), prominent gravitational and aeromagnetic anomaly that separates the deep Tacoma Basin sediments from the basalt of the Black Hills uplift, and based on these and geological field studies, researchers consider the structure to be a potential active fault (Gower et al. 1985; Rogers et al. 1996; Blakely et al. 1999). Stanley et al. (1999) postulate that this structure dips steeply to the southwest and forms the southern boundary of the Seattle-Tacoma Basins. Just to the north of the fault trace, Sherrod (2001) mapped in marsh sediment cores rapid subsidence of a former forest. Based on stratigraphy and radiocarbon dating, Sherrod (2001) determined that this area experienced 1 meter (m) to 3 m of rapid subsidence 1,100 years ago. Sherrod (2001) infers that movement on the Olympia Fault could be an explanation for the observed subsidence and liquefaction.

Assuming a rupture length of 80 km (50 miles; i.e., no segmentation), depth of rupture between 15 and 25 km (9.3 and 15 miles) and dips between 80 and 50 degrees, the mean maximum magnitude is estimated to be about 7.0 to 7.4 based on the relationship between rupture area and magnitude by Wells and Coppersmith (1994). This fault may or may not be segmented.

Currently, no estimate of slip rate or recurrence interval has been reported for faulting on this structure. Slip rates are likely bounded by estimated rate on the active Seattle Fault within the Puget Sound Basin and the lower crustal shortening rates across southwest Washington south of the basin. The absence of evidence of multiple Holocene rupture or movement would suggest a slip rate lower than the Seattle Fault. Consequently, slip rates of 0.1 and 1 mm per year were used with corresponding weighting factors of 0.7 and 0.3, respectively.

5.5.3.2 Saddle Mountain Fault Zone

The Saddle Mountain fault zone (SMFZ) was the first active fault to be discovered in western Washington. Four decades ago, Wilson et al. (1979) excavated paleoseismic trenches across topographic scarps that traversed recently logged slopes of the southeastern Olympic foothills. Based on mapping of trench stratigraphy and radiocarbon dating of earthquake-related deposits, they determined that these faults were active during the late Holocene. Several follow-up studies confirmed and expanded on their results.

LiDAR imagery allowed researchers to better map the extent and geomorphology of the fault zone. Witter et al. (2008) mapped in detail a LiDAR-identified portion of the west strand of the SMFZ and found structural and stratigraphic evidence that thrust faulting and folding, with possible dextral movement, raised the fault scarp at least 1.7 m during one event between the end of the Vashon stade (about 15 thousand years ago [ka]) and about 7.7 ka and during another event soon after about 1.7 ka. Witter et al. (2008) estimate that the west fault strand could produce an M 7 earthquake, based on a rupture length of 38 km (24 miles; derived from LiDAR and aeromagnetic anomaly mapping of Blakely et al. [2009]).

A subsequent wetland coring study of earthquake-deformed wetland and trench excavations across fault scarps within two strands of the SMFZ confirmed that these faults also ruptured during the late Holocene (Barnett et al. 2015). Uplift of the Saddle Mountain east fault scarp impounded stream flow, forming Price Lake and submerging an existing forest, thereby leaving drowned stumps still rooted in place. Stratigraphy mapped in sediment cores collected from the lake and wetland reveals the former forest floor abruptly flooded and was buried by lake deposits. Radiocarbon dating of material from this buried deposit and from stumps submerged in the lake indicates that the east fault strand ruptured around 1,000 years ago. Stratigraphy mapped in two trenches, one across the Saddle Mountain east fault and the other across the sub-parallel Sund Creek fault, records one and two earthquakes, respectively, as faulting by around 4 m juxtaposed Miocene-age bedrock against glacial and postglacial
deposits. Both faults are steeply dipping reverse faults, but the east fault scarp is west-facing and the Sund Creek fault scarp faces east; Barnett et al. (2015) infer these faults bound an upthrown block of Crescent Basalt.

Although the stratigraphy demonstrates that reverse motion generated the scarps, slip indicators measured on fault surfaces suggest a component of left-lateral slip. From trench exposures, it is estimated that the postglacial slip rate is around 0.2 mm per year and between 0.7 and 3.2 mm per year during the past 3,000 years. Integrating radiocarbon data from this study with earlier Saddle Mountain fault studies into an OxCal Bayesian statistical chronology model constrains the most recent paleoearthquake age of rupture across all three Saddle Mountain faults to 1170 to 970 calibrated years before present (cal yr BP), which overlaps with the nearby moment magnitude (Mw) 7.5 1050 to 1020 cal yr BP Seattle fault earthquake. An earlier earthquake recorded in the Sund Creek trench exposure, dates to around 3,500 cal yr BP. The geometry of the Saddle Mountain faults and their near-synchronous rupture to nearby faults 1,000 years ago suggest that the Saddle Mountain fault zone forms a western boundary fault along which the fore-arc blocks migrate northward in response to margin-parallel shortening across the Puget Lowland.

The SMFZ was modeled as a single fault zone because the fault strands of the SMFZ are located within 1 to 2 km (0.6 mile to 1.2 mile) of each other and they ruptured within a century of each other (Witter et al. 2008; Nelson et al. 2014; Barnett et al. 2015). For consistency and for comparison to other Lowland faults, the post-glacial slip rate of 0.2 to 0.7 mm per year was used. Rupture length was estimated in several ways: based on the LiDAR-mapping of fault scarps, the length of magnetic anomalies that correspond to the fault traces (Blakely et al. 2009), and the inclusion of nearby mapped active faults that are inferred to be strands of the SMFZ (Walsh et al. 1997; Blakely et al. 2009; Barnett et al. 2015). Therefore, three lengths were included in the model: 18 km (11.2 miles; LiDAR only), 30 km (19 miles; LiDAR plus associated faults), and 45 km (28 miles; LiDAR, nearby faults and the magnetic anomaly-mapping). The third value was weighted the most because the location and trend of the anomaly align well with all of the mapped faults, implying that they are part of a single structure.

5.5.3.3 The Doty Fault Zone

Located about 15 km (9.3 miles) to the north of the site, the Doty fault zone is the closest mapped fault to the project site that is suspected of recent earthquake deformation (Figures 5.6-2 and 5.6-3). Located at the southern edge of the Puget Lowland, the Doty fault might be more accurately described as a 50-km-long (31-mile-long) zone of east-west and northwest trending fault strands that straddle the towns of Centralia and Chehalis. The west end of the Doty fault strand initiates about 5 km northwest of the town of Doty and extends east to just north of Chehalis, where it appears to join the Salzer Creek fault, a parallel fault located a few kilometers to the north. These two faults are included in the Doty fault zone. The eastern extension of both faults disappear beneath the Chehalis River valley fill and only the Salzer Creek fault discontinuously continues east for another 10 km (6.2 miles), possibly 20 km (12.4 miles; Snavely 1958), for a total length of 40 to 50 km (24 to 31 miles). These two steeply dipping thrust faults (Pease and Hoover 1957) bound an upthrown block that pinches off where they meet beneath the Chehalis valley (Snively 1958). The entire east-west structure is captured in aeromagnetic as a sharp, linear anomaly. Finn (1990 and 1999) compiled aeromagnetic data for much of western Washington and mapped fore-arc block boundaries and the faults that separate them. In this dataset, the Doty fault zone is imaged as a district linear anomaly that spatially corresponds with the mapped fault zone (Finn 1990 and 1999).

The Doty fault bounds the north side of the Chehalis basin. The major structural rise to the north is the Black Hills, which are separated from the basin by the Doty fault and the northwest-southeast trending
Scammon Creek fault, which intersects the Doty and Salzer faults beneath the Chehalis valley (Snavely 1958; Pratt 1997). The Scammon Creek fault is the southernmost fault in a set of several similarly trending structures that extend north toward the sub-parallel Olympia Structure. Gower et al. (1985) infer deformation along these northwest-trending faults into the Pleistocene, although they offer no explanation.

Post-glacial deformation along faults within the Doty fault zone has not been directly documented. However, indirect evidence of deformation does exist. Pease and Hoover (1958) cite reports of landsliding and faulting during a 1948 earthquake. No follow-up studies of these reports are known. More convincingly, recent studies of earthquake-induced liquefaction report field evidence for potential intense ground shaking around the towns of Oakville and Porter, which are located about 20 km (12.4 miles) northwest of the fault zone (Obermeier 1995; Obermeier and Dickenson 2000).

Obermeier (1995) and Obermeier and Dickenson (2000) surveyed the banks of the Chehalis River for earthquake-induced liquefaction features where they identified dike intrusions at 10 sites near Oakville and Centralia. The intent of the study was to survey liquefaction features that were produced by great interface earthquakes along the CSZ. The unweathered nature of most of the dikes near Centralia led Obermeier (1995) to conclude that they were results of the 1949 Olympia interslab earthquake (M 7.1). However, they identified an older set of gravelly sand dikes. Radiocarbon ages and soil weathering profiles from sites between Oakville and Porter suggest that the older liquefaction features could have originated from an earthquake dated to 1.8 to 2.5 ka (1995). Furthermore, Obermeier (1995) and Obermeier and Dickenson (2000) infer that this set of older features occurred as a result of local fault rupture during one large earthquake, based on: (1) sand dikes in these sites were radiocarbon-dated to the same age, (2) the dikes are most plentiful and largest centered around Oakville, 80 km (50 miles) from the coast, and (3) they are seemingly absent or rare along rivers within 60 to 70 km (38 to 44 miles) of the coast and also east of the Oakville area. Thus, Obermeier (1995) and Obermeier and Dickenson (2000) infer a possible local earthquake source for the 1.8 to 2.5 ka earthquake, as opposed to a coastal source, such as the CSZ. Another possible nearby source for the shaking could instead be the Olympia Structure, located 35 km (22 miles) northeast of the Oakville sites. Further work on both faults would be required to resolve the ground shaking source.

Together, considering the prominent geophysical signature of the Doty fault zone (Finn 1990 and 1999), the close proximity to and similar fault trends to other active Puget Lowland faults, and the nearby field evidence for earthquake-induced liquefaction (Obermeier 1995; Obermeier and Dickenson 2000), these lines of evidence suggest that the Doty fault zone is a candidate to be an active fault. In fact, future geologic and geophysical work along the fault zone is planned (Ray Wells, personal communication, 2015). Until then, fault and earthquake parameters remain uncertain.

The Doty fault zone was included due to its close proximity to the site and its suspected earthquake activity. However, because of the lack of field evidence supporting recent deformation, the activity was weighted as 0.3. In our model, we combine all fault strands into one 50-km-long fault zone and account for potential segmentation by giving it a 0.5 weight. Based on mapping, the potential rupture lengths were estimated to be 20, 40, or 50 km. The middle length was weighted the highest (0.6) because this is the length of the longest continuous fault strand with consistent geologically-mapped characteristics: the Doty-Salzer strand (Snavely 1958). A slip rate of 0.1 to 1.0 mm per year was used in the model. This rate is the same estimated rate for the nearby Olympia Structure, which could be kinematically linked to the Doty Fault Zone. The lower rate was weighted higher for the Doty fault zone. This fault may or may not be segmented.
Assuming a rupture length of 50 km (31 miles; i.e., no segmentation), the mean maximum magnitude is estimated to be about 6.9 based on the relationship between rupture area and magnitude by Wells and Coppersmith (1994).

5.5.3.4 Grays Harbor Fault Zone
The Grays Harbor fault zone consists of an approximately 22-km-long, 2.5-km-wide (14-mile-long, 1.5-mile-wide) set of reverse thrusts that trend east-west directly offshore the mouth of the Grays Harbor bay (Figure 5.6-2). McCrory et al. (2002) mapped these faults in seismic reflection profiles and determined the primary expression of surface deformation to be folding. The timing of deformation can be constrained only to the Quaternary to latest Quaternary as ages of the seafloor deposits vary, depending on location relative to the coastline (McCrory et al. 2002). Where the east end of the fault zone offsets active sedimentation, the fault is inferred to be latest Quaternary (less than 15 ka). Other strands offset seafloor deposits that range in age from Quaternary (less than 1.8 million years ago) to late Quaternary (less than 150 ka). McCrory limits vertical offset to between approximately 12 m to 40 m across the east and west ends of the fault zone, respectively. Seismic reflection imagery shows both north-side and south-side displacement across the dip-slip fault, with most inferred to be dipping to the north. McCrory (2002) infers that these are thrust faults, based on their association with anticlines, and thus have dips less than 45 degrees along the fault strands.

Similar to the Willapa Bay fault to the south, McCrory et al. (2002) surmise that these faults are the products of the north-verging Oregon Coast Range fore-arc block (Figure 5.6-1) impinging upon the Olympic Mountains; this is a location of expected north-south contraction and the orientation and slip sense of these faults conform to this tectonic model.

McCrory (2002) estimates a slip rate of 10 m per 20,000 years or 0.5 mm per year, for an upper limit, and derives a lower limit to be 0.2, based on offset along the southern fault strand around 150,000 years. More weight is given the latter because McCrory (2002) did not directly date the youngest-deformed deposits. They could be older. Based on the reflection data and shallower depths, McCrory (2002) infers these faults might extend only to less than 10 km (6.3 mile) depth, as noted in our model.

Assuming a rupture length of 22 km (14 miles; i.e., no segmentation) the mean maximum magnitude is estimated to be about 6.5 based on the relationship between rupture area and magnitude by Wells and Coppersmith (1994).

5.5.3.5 Willapa Bay Fault Zone
The Willapa Bay fault zone (WBFZ) encompasses several north-northwest trending fault strands that extend about 35 km (22 miles) the length of Willapa Bay. Displacement across these east- and west-side up structures is captured in seismic reflection imagery and in onshore exposures that define an uplifted late Pleistocene ridge (Wolf et al. 1998; McCrory et al. 2002). McCrory (2002) infers deformation timing based on the inferred age of the youngest deposit that is offset: a buried erosional surface radiocarbon-dated to about 20 ka at the northern end of the fault and about 150 ka at the southern end. At the northern end of the Long Beach Peninsula, onshore exposures reveal vertical offset of late Pleistocene deposits by around 2 to 8 m (6.5 to 26 feet), while offshore, McCrory (2002) maps offset between 10 to 12 m (33 to 39 feet). Seismic reflection data image steeply-dipping faults, although McCrory (2002) notes that faults mapped onshore have a shallower dip. At a minimum, all faults appear to have dips greater than 30 degrees, both east and west, but are probably steeper.

All motion observed in the seismic reflection data and onshore exposures reflects dip-slip motion. However, McCrory et al. (2002) invoke indirect evidence for the possibility of a component strike-slip
motion: the close proximity to the fault of young structures such as anticlines, closed depressions, and displaced channels mapped on the buried paleosurface. McCrory et al. (2002) note that given the location of these faults near the boundary of the north-verging Oregon Coast Range fore-arc block (Figure 5.6-1) and the Olympic Mountains overriding the northeast-driving CSZ, transpressional shear is expected. McCrory et al. (2002) interpret the WBFZ structures to be responding to strain in the upper plate of the thrust boundary between fore-arc blocks.

McCrory et al. (2002) estimate a slip rate of 10 m (33 feet) per 20,000 years (0.5 mm per year), for an upper limit, and derives a lower limit to be 0.2, based on offset along the southern fault strand around 150 ka. We model this zone as segmented based on its multi-strand nature.

Assuming a rupture length of 35 km (22 miles; i.e., no segmentation) the mean maximum magnitude is estimated to be about 6.6 based on the relationship between rupture area and magnitude by Wells and Coppersmith (1994).

### 5.5.3.6 Gales Creek Fault Zone

The northwest-trending Gales Creek fault (GCF), located about 40 km west of Portland, forms the structural boundary between the east edge of the Oregon Coast Range and the Tualatin and northern Willamette basins that lie to the east (Figure 5.6-2). The fault zone spans at least 70 km (25 miles) in length and consists of overlapping en echelon fault segments, identified and mapped by the juxtaposition of Miocene Columbia River Basalt Group rocks against Eocene volcanic rocks (e.g., Wells et al. 1994; Blakely et al. 2000). Although mapping of fault exposures shows a cumulative dextral offset of 12 km (7.5 miles) since 30 million years ago (Wells 2009), no surface deformation had been observed in Quaternary deposits until field mapping by Wells (2009) and Bemis et al. (2012) identified topographic scarps along the south-eastern end of the GCF.

Trenching across a 6-km-long (4-mile-long) scarp exposed a thrust fault that displaced Eocene Yamhill Formation and associated colluvium southwest over Pleistocene-age silt, inferred to be a loess-rich former ground surface (Bemis et al. 2012). Optically stimulated luminescence-dating of the paleosurface suggests that the fault was active after 200 to 250 ka; an overlying undeformed Missoula flood deposit (about 15 ka) indicates that this fault strand has not been active during the Holocene, providing a minimum age constraint (Bemis, personal communication, 2014). Bemis et al. (2012) were unable to determine total vertical offset within the trench, but they did map drainages right-laterally offset by 2 km (1.2 miles) along this portion of the GCF. On LiDAR, Wells (2009) mapped surface deformation features such as scarps, offset drainages, shutter ridges for at least 18 km (11.2 miles). Gravity anomaly-mapping extends the length of the GCF to 50 km (31 miles; McPhee et al. 2013); however, based on aeromagnetic anomaly data, Blakely et al. (2000) infer a kinematic link between the GCF and the Mount Angle fault located 25 km (16 miles) to the southeast. This similarly trending fault is the inferred source of the 1993 5.6 earthquake, and connecting the two faults would make the entire fault zone around 90 km (56 miles) in length. Considering the right-lateral offset mapped in the bedrock (Wells 2009), a minimum slip rate would be around 0.4 mm per year. Further work to better constrain the timing and amount of slip recorded by Quaternary surface deformation along the fault is scheduled (Wells, personal communication 2015).

Very little paleoseismic data are yet available for this fault, so we assigned a long-term slip rate of 0.4 mm per year, based on bedrock mapping by Wells (2009) and Wells et al. (1994). The rupture length is an estimate based on field and geophysical-anomaly mapping (Wells 2009; McPhee et al. 2013). The maximum length of 90 km (56 miles) is the GCF joined to the Mt. Angel fault (Blakely et al. 2000), but is assigned a lower weight because structural connects between the two fault zones is unclear. Assuming a rupture length of 90 km (56 miles; i.e., no segmentation) the mean maximum magnitude is estimated
to be about 6.8 based on the relationship between rupture area and magnitude by Wells and Coppersmith (1994).

5.5.4 SITE $V_{S30}$
The shear wave velocities measured in the rock at different locations are variable but consistent with Site Class A to B. Consequently, in the ground motion hazard analyses, the $V_{S30}$ ground motions were estimated to be approximately 5,000 fps, corresponding to the boundary between Site Class A and B.
### Table 5-11
Paleo-Earthquakes on Upper Plate Faults in Western Washington

<table>
<thead>
<tr>
<th>FAULT ZONE/STRANDS</th>
<th>NAME</th>
<th>EQ AGE (CAL YEARS BP)</th>
<th>SCARP HEIGHT (M)</th>
<th>MOTION</th>
<th>REFERENCES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Seattle Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1) Master</td>
<td>Raised Platform – 1,100 yr BP</td>
<td>1,050–1,020</td>
<td>U</td>
<td>D</td>
<td>Bucknam et al. (1992), Nelson et al. (2003a), and Atwater (1999)</td>
</tr>
<tr>
<td>(2) Vasa Park</td>
<td>Vasa</td>
<td>12,880–12,310</td>
<td>2</td>
<td>D</td>
<td>Sherrod (2002)</td>
</tr>
<tr>
<td>(3) Mac’s Pond</td>
<td>Spotted Frog</td>
<td>&lt;1,020</td>
<td>2.9</td>
<td>D</td>
<td>Nelson et al. (2003b)</td>
</tr>
<tr>
<td>(4) Waterman</td>
<td>Madrone</td>
<td>1,420–1,710</td>
<td>3.9</td>
<td>D</td>
<td>Nelson et al. (2003b)</td>
</tr>
<tr>
<td>(5) Toe Jam</td>
<td>Mossy Lane</td>
<td>~1,200</td>
<td>3.5</td>
<td>D</td>
<td>Nelson et al. (2003a)</td>
</tr>
<tr>
<td>(5) Toe Jam</td>
<td>Crane Lake</td>
<td>6,000–2,500</td>
<td>5.2</td>
<td>D</td>
<td>Nelson et al. (2003a)</td>
</tr>
<tr>
<td>(5) Toe Jam</td>
<td>Crane Lake</td>
<td>2,500–1,900</td>
<td>5.2</td>
<td>D</td>
<td>Nelson et al. (2003a)</td>
</tr>
<tr>
<td>(5) Toe Jam</td>
<td>Saddle et al.</td>
<td>~1,500</td>
<td>??</td>
<td>D</td>
<td>Nelson et al. (2003a)</td>
</tr>
<tr>
<td>(5) Toe Jam</td>
<td>Bear’s Lair</td>
<td>~15,000</td>
<td>??</td>
<td>D</td>
<td>Nelson et al. (2003a)</td>
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<td><strong>South Whidbey Island Fault Zone</strong></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>(1) Brightwater</td>
<td>Beef Barley</td>
<td>~15,000</td>
<td>&lt;1m</td>
<td>O?</td>
<td>Sherrod et al. (2008)</td>
</tr>
<tr>
<td>(2) Crystal Lake</td>
<td>Mtn Beaver</td>
<td>&lt;11,690</td>
<td>1.9</td>
<td>D</td>
<td>Sherrod et al. (2008)</td>
</tr>
<tr>
<td>(3) main</td>
<td>Whidbey Island</td>
<td>3,200–2,800</td>
<td>U/S</td>
<td>D</td>
<td>Kelsey et al. (2004)</td>
</tr>
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<td><strong>Olympia Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>Little Skookum</td>
<td>1,150–980</td>
<td>S</td>
<td></td>
<td>Sherrod (2003)</td>
</tr>
<tr>
<td><strong>Tacoma Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>Case Inlet</td>
<td>1,235–815</td>
<td>U</td>
<td>D</td>
<td>Sherrod et al. (2004)</td>
</tr>
<tr>
<td><strong>Boulder/Canyon Creek Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Boulder Creek</td>
<td>Hornet</td>
<td>1,165–699</td>
<td>3.4</td>
<td>D</td>
<td>Sherrod et al. (2014), Barnett (2007), and Seidlecki (2008)</td>
</tr>
<tr>
<td>FAULT ZONE/STRANDS</td>
<td>NAME</td>
<td>EQ AGE (CAL YEARS BP)</td>
<td>SCARP HEIGHT (M)</td>
<td>MOTION</td>
<td>REFERENCES</td>
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<td>------------------------------------------------</td>
</tr>
<tr>
<td>Boulder Creek</td>
<td>Hornet</td>
<td>3,344–2,990</td>
<td>3.4</td>
<td>D</td>
<td>Sherrod et al. (2014), Barnett (2007), and Seidlecki (2008)</td>
</tr>
<tr>
<td>Boulder Creek</td>
<td>Hornet</td>
<td>~7,700</td>
<td>3.4</td>
<td>D</td>
<td>Sherrod et al. (2014), Barnett (2007), and Seidlecki (2008)</td>
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<tr>
<td><strong>Bellingham Coastal Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1) Sandy Point</td>
<td>Sandy Point</td>
<td>&gt;2,100</td>
<td>U</td>
<td>D</td>
<td>Kelsey et al. (2012)</td>
</tr>
<tr>
<td>(1) Sandy Point</td>
<td>Sandy Point</td>
<td>~2,100</td>
<td>U</td>
<td>D</td>
<td>Kelsey et al. (2012)</td>
</tr>
<tr>
<td>(1) Sandy Point</td>
<td>Sandy Point</td>
<td>&lt;2,100</td>
<td>U</td>
<td>D</td>
<td>Kelsey et al. (2012)</td>
</tr>
<tr>
<td>(2) Birch Bay</td>
<td>Birch Bay</td>
<td>1,280–1,070</td>
<td>U/S</td>
<td>D</td>
<td>Kelsey et al. (2012)</td>
</tr>
<tr>
<td><strong>Saddle Mtn/Canyon River Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1) Frigid Creek</td>
<td>Hummingbird</td>
<td>3,800–415</td>
<td>2.9</td>
<td>D</td>
<td>Blakely et al. (2009)</td>
</tr>
<tr>
<td>(2) Saddle Mountain East</td>
<td>Quarry</td>
<td>~1,100</td>
<td>8</td>
<td>O</td>
<td>Wilson et al. (1979), Witter et al. (2008), and Barnett et al. (2014)</td>
</tr>
<tr>
<td>(3) Canyon River</td>
<td>Canyon River</td>
<td>&lt;1,630</td>
<td>~8</td>
<td>O</td>
<td>Walsh and Logan (2007)</td>
</tr>
<tr>
<td><strong>Boundary/Lake Creek Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>Nelson</td>
<td>2,000–600 (2 eqs)</td>
<td>1</td>
<td>O</td>
<td>Nelson et al. (2007)</td>
</tr>
<tr>
<td><strong>Darrington-Devils Mtn Fault Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1) Darrington-Devils Mtn</td>
<td>Lake Cavanaugh</td>
<td>&lt;2,200</td>
<td>1</td>
<td>O</td>
<td>Personius et al. (2009)</td>
</tr>
<tr>
<td>(2) Utsalady Pt.</td>
<td>Duffers</td>
<td>2,200–1,200</td>
<td>4.3</td>
<td>O</td>
<td>Johnson et al. (2004b)</td>
</tr>
<tr>
<td>(2) Utsalady Pt.</td>
<td>Duffers</td>
<td>500–100</td>
<td>4.3</td>
<td>O</td>
<td>Johnson et al. (2004b)</td>
</tr>
</tbody>
</table>

Notes:
- Strands with the same name and/or number may have multiple entries, corresponding to multiple earthquakes. In column “Scarp Height”, U and S indicate sites of coastal uplift and subsidence, respectively. Motion designations D and O refer to dip-slip or oblique-slip motion, respectively.
- Adapted from Sherrod and Gomberg (2014). References cited in this table can be found in Sherrod and Gomberg (2014).
Table 5-12
Largest Historic Earthquakes Felt in Washington

<table>
<thead>
<tr>
<th>YEAR</th>
<th>DATE</th>
<th>TIME (PST)</th>
<th>NORTH LATITUDE</th>
<th>WEST LONGITUDE</th>
<th>DEPTH (KM)</th>
<th>MAG (FELT)</th>
<th>MAG (INST)</th>
<th>MAXIMUM MODIFIED MERCALLI INTENSITY</th>
<th>FELT AREA (SQ KM)</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1872</td>
<td>Dec. 14</td>
<td>21:40</td>
<td>47° 45'00&quot;</td>
<td>119° 52'00&quot;</td>
<td>Shallow</td>
<td>6.8</td>
<td>None</td>
<td>IX</td>
<td>1,100</td>
<td>North Cascades</td>
</tr>
<tr>
<td>1877</td>
<td>Oct. 12</td>
<td>13:53</td>
<td>45° 30'00&quot;</td>
<td>122° 30'00&quot;</td>
<td>Shallow</td>
<td>5.3</td>
<td>None</td>
<td>VII</td>
<td>48,000</td>
<td>Portland, Oregon</td>
</tr>
<tr>
<td>1891</td>
<td>Nov. 29</td>
<td>15:21</td>
<td>48° 00'00&quot;</td>
<td>123° 30'00&quot;</td>
<td>Shallow</td>
<td>4.7</td>
<td>None</td>
<td>VII</td>
<td>21,000</td>
<td>Southeastern Washington</td>
</tr>
<tr>
<td>1893</td>
<td>Mar. 06</td>
<td>17:03</td>
<td>45° 54'00&quot;</td>
<td>119° 24'00&quot;</td>
<td>Shallow</td>
<td>?</td>
<td>None</td>
<td>VII</td>
<td>?</td>
<td>?</td>
</tr>
<tr>
<td>1896</td>
<td>Jan. 03</td>
<td>22:15</td>
<td>48° 30'00&quot;</td>
<td>122° 48'00&quot;</td>
<td>?</td>
<td>5.7</td>
<td>None</td>
<td>VII</td>
<td>?</td>
<td>Puget Sound</td>
</tr>
<tr>
<td>1904</td>
<td>Mar. 16</td>
<td>20:20</td>
<td>47° 48'00&quot;</td>
<td>123° 00'00&quot;</td>
<td>?</td>
<td>5.3</td>
<td>None</td>
<td>VII</td>
<td>50,000</td>
<td>Olympic Peninsula, eastside</td>
</tr>
<tr>
<td>1909</td>
<td>Jan. 11</td>
<td>15:49</td>
<td>48° 42'00&quot;</td>
<td>122° 48'00&quot;</td>
<td>Deep</td>
<td>6</td>
<td>None</td>
<td>VII</td>
<td>150,000</td>
<td>Puget Sound</td>
</tr>
<tr>
<td>1915</td>
<td>Aug. 18</td>
<td>6:05</td>
<td>48° 30'00&quot;</td>
<td>121° 24'00&quot;</td>
<td>?</td>
<td>5.6</td>
<td>None</td>
<td>VI</td>
<td>77,000</td>
<td>North Cascades</td>
</tr>
<tr>
<td>1918*</td>
<td>Dec. 06</td>
<td>0:41</td>
<td>49° 37'00&quot;</td>
<td>125° 55'00&quot;</td>
<td>?</td>
<td>7</td>
<td>7</td>
<td>VIII</td>
<td>650,000</td>
<td>Vancouver Island</td>
</tr>
<tr>
<td>1920</td>
<td>Jan. 23</td>
<td>23:09</td>
<td>48° 36'00&quot;</td>
<td>123° 00'00&quot;</td>
<td>?</td>
<td>5.5</td>
<td>None</td>
<td>VII</td>
<td>70,000</td>
<td>Puget Sound</td>
</tr>
<tr>
<td>1932</td>
<td>17-Jul</td>
<td>22:01</td>
<td>47° 45'00&quot;</td>
<td>121° 50'00&quot;</td>
<td>Shallow</td>
<td>5.2</td>
<td>None</td>
<td>VII</td>
<td>41,000</td>
<td>Central Cascades</td>
</tr>
<tr>
<td>1936</td>
<td>15-Jul</td>
<td>23:08</td>
<td>46° 00'00&quot;</td>
<td>118° 18'00&quot;</td>
<td>Shallow</td>
<td>6.4</td>
<td>5.75</td>
<td>VII</td>
<td>270,000</td>
<td>Southeastern Washington</td>
</tr>
<tr>
<td>1939</td>
<td>Nov. 12</td>
<td>23:46</td>
<td>47° 24'00&quot;</td>
<td>122° 36'00&quot;</td>
<td>Deep</td>
<td>6.2</td>
<td>5.75</td>
<td>VII</td>
<td>200,000</td>
<td>Puget Sound</td>
</tr>
<tr>
<td>1945</td>
<td>29-Apr</td>
<td>12:16</td>
<td>47° 24'00&quot;</td>
<td>121° 42'00&quot;</td>
<td>5.9</td>
<td>5.5</td>
<td>VII</td>
<td>128,000</td>
<td>Central Cascades</td>
<td></td>
</tr>
<tr>
<td>1946</td>
<td>Feb. 14</td>
<td>19:18</td>
<td>47° 18'00&quot;</td>
<td>122° 54'00&quot;</td>
<td>40 (Deep)</td>
<td>6.4</td>
<td>6.3</td>
<td>VII</td>
<td>270,000</td>
<td>Puget Sound</td>
</tr>
<tr>
<td>1946*</td>
<td>23-Jun</td>
<td>9:13</td>
<td>49° 48'00&quot;</td>
<td>125° 18'00&quot;</td>
<td>Deep</td>
<td>7.4</td>
<td>7.3</td>
<td>VIII</td>
<td>1,096,000</td>
<td>Vancouver Island</td>
</tr>
<tr>
<td>1949</td>
<td>13-Apr</td>
<td>11:55</td>
<td>47° 06'00&quot;</td>
<td>122° 42'00&quot;</td>
<td>54 (Deep)</td>
<td>7</td>
<td>7</td>
<td>VIII</td>
<td>594,000</td>
<td>Puget Sound</td>
</tr>
<tr>
<td>1949*</td>
<td>Aug. 21</td>
<td>20:01</td>
<td>53° 37'20&quot;</td>
<td>133° 16'20&quot;</td>
<td>7.8</td>
<td>8.1</td>
<td>VIII</td>
<td>2,220,000</td>
<td>Queen Charlotte Island, British Columbia</td>
<td></td>
</tr>
<tr>
<td>1959</td>
<td>Aug. 05</td>
<td>19:44</td>
<td>47° 48'00&quot;</td>
<td>120° 00'00&quot;</td>
<td>35 (Deep)</td>
<td>5.5</td>
<td>5</td>
<td>VI</td>
<td>64,000</td>
<td>North Cascades, east side</td>
</tr>
<tr>
<td>YEAR</td>
<td>DATE</td>
<td>TIME (PST)</td>
<td>NORTH LATITUDE</td>
<td>WEST LONGITUDE</td>
<td>DEPTH (KM)</td>
<td>MAG (FELT)</td>
<td>MAG (INST)</td>
<td>MAXIMUM MODIFIED MERCALLI INTENSITY</td>
<td>FELT AREA (SQ KM)</td>
<td>LOCATION</td>
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</tr>
<tr>
<td>1959*</td>
<td>Aug. 17</td>
<td>22:37</td>
<td>44° 49'59&quot;</td>
<td>111° 05'</td>
<td>10-12 (Shallow)</td>
<td>7.6</td>
<td>7.5</td>
<td>X</td>
<td>158,600</td>
<td>Hebgen Lake, Montana</td>
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<td>Nov. 05</td>
<td>19:36</td>
<td>45° 36'30&quot;</td>
<td>122° 35'54&quot;</td>
<td>18 (Shallow)</td>
<td>5.3</td>
<td>5.5</td>
<td>VII</td>
<td>51,000</td>
<td>Portland, Oregon</td>
</tr>
<tr>
<td>1965</td>
<td>29-Apr</td>
<td>7:28</td>
<td>47° 24'00&quot;</td>
<td>122° 24'00&quot;</td>
<td>63 (Deep)</td>
<td>6.8</td>
<td>6.5</td>
<td>VIII</td>
<td>500,000</td>
<td>Puget Sound</td>
</tr>
<tr>
<td>1976</td>
<td>16-May</td>
<td>0:35</td>
<td>48° 45'36&quot;</td>
<td>123° 19'48&quot;</td>
<td>Deep</td>
<td>5.1</td>
<td></td>
<td></td>
<td></td>
<td>Friday Harbor, San Juan Island, Washington</td>
</tr>
<tr>
<td>1981</td>
<td>Feb. 13</td>
<td>22:09</td>
<td>46° 21'01&quot;</td>
<td>122° 14'66&quot;</td>
<td>7 (Shallow)</td>
<td>5.8</td>
<td>5.5</td>
<td>VII</td>
<td>104,000</td>
<td>South Cascades</td>
</tr>
<tr>
<td>1981</td>
<td>28-May</td>
<td>1:11</td>
<td>46° 30'36&quot;</td>
<td>121° 22'48&quot;</td>
<td>Shallow</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>Goat Rocks</td>
</tr>
<tr>
<td>1983*</td>
<td>Oct. 28</td>
<td>6:06</td>
<td>44° 03'29&quot;</td>
<td>113° 51'25&quot;</td>
<td>14 (Shallow)</td>
<td>7.2</td>
<td>7.3</td>
<td>VII</td>
<td>800,000</td>
<td>Borah Peak, Idaho</td>
</tr>
<tr>
<td>1989</td>
<td>24-Dec</td>
<td>0:46</td>
<td>46° 39'00&quot;</td>
<td>122° 06'00&quot;</td>
<td>Shallow</td>
<td>4.9</td>
<td></td>
<td></td>
<td></td>
<td>Morton</td>
</tr>
<tr>
<td>1990</td>
<td>14-Apr</td>
<td>21:33</td>
<td>48° 49'48&quot;</td>
<td>122° 09'00&quot;</td>
<td>Shallow</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>Deming</td>
</tr>
<tr>
<td>1995</td>
<td>Jan. 28</td>
<td>7:11</td>
<td>47° 23'17&quot;</td>
<td>122° 21'54&quot;</td>
<td>16 (Shallow)</td>
<td>--</td>
<td>5</td>
<td>V</td>
<td>--</td>
<td>Robinson Point, Washington</td>
</tr>
<tr>
<td>1996</td>
<td>2-May</td>
<td>20:04</td>
<td>47° 45'36&quot;</td>
<td>121° 52'34&quot;</td>
<td>7 (Shallow)</td>
<td>--</td>
<td>5.1</td>
<td>V</td>
<td>--</td>
<td>Duvall, Washington</td>
</tr>
<tr>
<td>1997</td>
<td>23-Jun</td>
<td>11:13</td>
<td>47° 35'56&quot;</td>
<td>122° 32'26&quot;</td>
<td>7 (Shallow)</td>
<td>--</td>
<td>4.9</td>
<td>VI</td>
<td></td>
<td>Bremerton, Washington</td>
</tr>
<tr>
<td>1999</td>
<td>2-Jul</td>
<td>5:43</td>
<td>47° 04'33&quot;</td>
<td>123° 46'35&quot;</td>
<td>41 (Deep)</td>
<td>--</td>
<td>5.9</td>
<td>VII</td>
<td>--</td>
<td>Satsop, Washington</td>
</tr>
<tr>
<td>2001</td>
<td>28-Feb</td>
<td>10:55</td>
<td>47° 12'00&quot;</td>
<td>122° 42'00&quot;</td>
<td>52 (Deep)</td>
<td>--</td>
<td>6.8</td>
<td></td>
<td>--</td>
<td>Nisqually, Washington</td>
</tr>
</tbody>
</table>

Notes:
1. M (felt) = an estimate of magnitude, based on felt area (in local magnitude, M); unless otherwise indicated, it is calculated from M (felt) = -1.88+1.53 logA, where A is the total felt area; from Toppozada (1975).
2. M (inst) = instrumentally determined magnitude; refer to reference listed in the original Table 2 of Noson et al. (1988; or NGDC (1999) [post 1983]) for magnitude scale of the earthquakes.
* Located outside the state of Washington.
Interpretation of Site Characterization Results

6 Interpretation of Site Characterization Results

6.1 RESERVOIR LANDSLIDE HAZARDS
Twenty-three landslides have been confirmed in the reservoir basin. The project design must consider how these landslides may affect dam and 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.

A number of the landslides need further investigation and evaluation to determine whether remedial stabilization is needed. Landslide remediation/stabilization represents a potential significant cost and should be further evaluated in the next phase of work including additional site characterization.

6.2 SEISMIC HAZARDS
The results of the PSHA in terms of uniform hazard spectrum(UHS) for various return periods, hazard curves from which the UHS are derived, and ground motion hazard deaggregation by source, magnitude, and source-to-site distance are presented in this section. The results of the DSHA in terms of MCE spectra for the various potential controlling MCE sources are also summarized.

6.2.1 PROBABILISTIC SEISMIC HAZARD ANALYSIS RESULTS
The horizontal UHS from the PSHA analysis for 500-, 1,000-, 2,500-, 5,000- and 10,000-year return periods and NEHRP Site Class A/B boundary conditions (i.e., \( V_{390} \) of a about 5,000 fps) are plotted on Figure 6.2-1. These UHS are the sum of the hazards from the various seismic sources included in the seismic source characterization model.

The calculated hazard curves from the PSHA for horizontal ground motion versus mean annual rate of exceedance or return periods are presented in Figures 6.2-2 through 6.2-4 for periods of 0.0 (i.e., PGA), 0.2 and 1.0 second. The hazard curves indicate that the CSZ interface is the dominant contributor to the ground motion hazard for return periods of 500 to 10,000 years.

Deaggregation results for the mean 2,475-year return period are shown in Figures 6.2-5 through 6.2-7. These figures show the hazard contribution for different combinations of magnitude and source-to-site distance. The deaggregation results are presented in terms of magnitude and source-to-site distance versus ground motion hazard for periods of 0.0 (i.e., PGA), 0.2, and 1.0, seconds. Tabulated in each of these figures are the mean magnitude, distance, and ground motion epsilon. For a given set of fault parameters (e.g., fault type, magnitude, and source-to-site distance) and ground motion prediction equation, the ground motion epsilon is the number of standard deviations a particular ground motion is above the mean ground motion. Also tabulated in these figures is the relative hazard contribution by
source. On these figures, the height of the vertical columns corresponds to the proportion of the total ground motion hazard contributed by various combinations of earthquake magnitude and distance.

6.2.2 DETERMINISTIC SEISMIC HAZARD ANALYSIS RESULTS
A DSHA was completed to estimate the ground motions from a specific seismogenic source at a site regardless of the rate at which earthquakes are generated from the source. In a DSHA, the various seismic parameters (e.g., fault type, rupture dimensions, and maximum magnitude) for each potential earthquake source are evaluated and an MCE is determined for that source. Using the distance between the site and the source, the ground motions at the site for a given MCE are calculated, and the MCE source that produces the largest (strongest) ground motions at the site is the controlling maximum credible earthquake (CMCE).

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

The potential seismic sources (MCEs) evaluated for the current deterministic study are as follows:

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

Figure 6.2-8 compares the deterministic spectra calculated for the MCEs. As observed from this figure, the CMCE for the sites is the CSZ interface.

6.3 DAM FOUNDATION DESIGN CONSIDERATIONS

6.3.1 DAM TYPE
Results of the Phase 1 Site Characterization work have confirmed that foundation conditions at the potential site are suitable for construction of either an RCC or rockfill dam type. Suitable aggregate is likely available in reasonable proximity to the site for production of RCC and conventional concrete materials at competitive unit prices.

In addition to confirming the feasibility of each dam type, information from this report has also 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 with overall costs falling within the estimated range presented in the Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR 2014). Consequently, considering cost and other technical factors, the RCC dam type for either the FRO or FRFA project configurations should be the preferred option for the site, and future design work should proceed under this basis.
6.3.2 FOUNDATION EXCAVATION OBJECTIVE AND CONSIDERATIONS

Results of the Phase 1 site characterization program have demonstrated 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) as presented in the Combined Dam and Fish Passage Alternatives Technical Memorandum (HDR 2014). 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 objective (i.e., the estimated excavation limit that provides suitable strength and deformability characteristics to meet design requirements) will be a critical element of work during future design and construction activities. The Phase 1 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 and down-hole televiewer geophysical exploration methods will be suitable to establish this objective going forward.

The site characterization work indicates the potential for localized areas of decreased rock quality. 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. Additional site characterization work will better define the risk associated areas of lower quality rock and areas requiring additional treatment so that appropriate recommendations can be included in the preliminary design, and so that the costs to implement them can be included in final design configurations and cost estimates.

To that end, a 3-D site characterization model has been initiated using the information and interpretations completed as part of this Phase 1 work. An example of the model is illustrated on Figures 6.3-1. Sufficient information is not yet available to create an appropriate excavation objective surface for the dam footprint. Future phases of site characterization should be performed to provide the information needed to complete this model and finalize the foundation excavation limits and requirements.

Bedrock structure information obtained from surface and down-hole televiewer mapping has been evaluated and summarized for assessment of excavation and dam foundation stability concerns. The information gathered to date suggests jointing and fractures in the bedrock may create a potential for sliding surfaces beneath the dam and along temporary and permanent excavated slopes. These hazards require further characterization and analysis during future engineering design work.

Excavation with typical earthmoving equipment will be feasible in the overburden soils and weathered bedrock materials. The geophysical data and interpretations shown in Figures 5.1-2 to 5.1-4 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. An approximate excavation objective range is shown beneath the main dam alignment in Figure 5.1-2, the saddle dam alignment in Figure 5.1-3, and along a cross-section through the dam at station 5+44 feet in Figure 5.1-4.

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 the geophysical study as having a compression wave velocity range of 7,200 to 10,000 fps. These areas will likely require controlled blasting for practical removal. Considerable attention must be given to blasting
operations so that blasting does not induce additional fracturing and damage to the foundation rock that would require additional treatment. Detailed blasting requirements will be developed as part of the final design.

The large landslide in the left abutment shown in cross section D-D’ in Figure 5.1-4 is upstream of the dam axis and does not fall within the dam footprint. This landslide will likely require some combination of removal and stabilization as part of the dam construction work due to its proximity to the dam and outlet works facilities. Additional investigations and evaluations will be required to establish appropriate design and construction requirements. A smaller landslide in the left abutment, shown in cross section C-C’ in Figure 5.1-4, is within the dam footprint. This landslide should be completely removed to competent rock. Other dam foundation treatments may be needed in this area. Future investigations should be performed to support design of the dam foundation treatment requirements.

6.3.3 GROUTING

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 the Phase 1 program. 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 explorations, review of rock cores, down-hole geophysical tests, and water pressure tests have identified that foundation grouting to treat and reduce flow through fractures in the rock will be an important component of the dam design.

Zones encountered in the borehole that had significant hydraulic conductivity are summarized as follows:

- BH-2 elevation 274 to 284 famsl
- BH-5 above elevation 498 famsl
- BH-5 elevation 468 to 478 famsl
- BH-5 elevation 338 to 388 famsl
- BH-6 elevation 612 to 602 famsl
- BH-6 elevation 472 to 462 famsl

Overall, at least 35 percent, and possibly as high as 70 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 Figures 5.1-2 and 5.1-3 should be included in designs. Additional site characterization work along the dam axis, including the upper right abutment, should be performed.

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 a Lu closure criteria between 1 and 5.
6.3.4 FOUNDATION PREPARATION REQUIREMENTS

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.

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.

6.3.5 SEISMIC DESIGN CONSIDERATIONS

The estimated PGAs for the 2,475- and 10,000-year return period events from various sources shown in Figure 6.2-2 are summarized below in Table 6-1.

<table>
<thead>
<tr>
<th>SOURCE</th>
<th>ESTIMATED PGA, 2,475-YEAR RETURN PERIOD</th>
<th>ESTIMATED PGA, 10,000-YEAR RETURN PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crustal Faults</td>
<td>0.1</td>
<td>0.18</td>
</tr>
<tr>
<td>Crustal Background</td>
<td>0.2</td>
<td>0.3</td>
</tr>
<tr>
<td>CSZ Interface</td>
<td>0.25</td>
<td>0.39</td>
</tr>
<tr>
<td>CSZ Intraslab</td>
<td>0.4</td>
<td>0.79</td>
</tr>
<tr>
<td>Total Aggregated Hazard from PSHA</td>
<td>0.49</td>
<td>0.83</td>
</tr>
<tr>
<td>MCE Hazard from DSHA</td>
<td>more than 0.3</td>
<td></td>
</tr>
</tbody>
</table>

The ground motion hazard at the potential Chehalis dam site is significant and will be a primary consideration in the development of the RCC or rockfill dam cross-section as well as the structural design of spillway and outlet works facilities.

Based on the seismic hazard information presented in this report, the next phase of study should include appropriate model studies to evaluate the seismic response of an RCC dam.
6.3.6 TUNNEL DESIGN CONSIDERATIONS

Depending on the dam type and configuration ultimately selected for construction, appurtenant structures and/or fish passage solutions may require tunneling through the abutments of the dam. The Phase 1 Site Characterization program has provided important information for advancing tunnel design and construction requirements. Several important layout and design factors have been identified, including the following:

- Lithology and bedding/foliation orientation of each strata the 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
- Lithologic contacts that present abruptly varying rock mass conditions
- Soft zones, weathered zones, shear zones that may require additional tunnel excavation stabilization
- 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 outlet works systems

Additional site characterization activities should be performed along specific potential tunnel alignments to characterize the subsurface conditions and to identify and address design and construction challenges.
Conclusions and Recommendations

7 Conclusions and Recommendations

The subsurface beneath the potential Chehalis Dam consists entirely of early-middle Eocene-age Crescent Formation basalt with interbedded siltstone lenses, except at the top of the hill slope on the right abutment where the potential dam is underlain by McIntosh Formation Siltstone. The Crescent Formation basalt demonstrates characteristics suitable for foundation material, having a good RMR weathering and compression wave velocity characteristic. Siltstone interbeds demonstrate mostly fair RMR. However, the overall foundation strength is controlled by the overwhelming proportion of the basalt that has a “good” RMR. Results of the Phase 1 Site Characterization work at the potential Chehalis Dam site have confirmed that foundation conditions are suitable for construction of either an RCC or rockfill dam type. Suitable aggregate is likely available in reasonable proximity to the site for production of RCC and conventional concrete materials at competitive unit prices.

The rock structure information gathered during Phase 1 suggests jointing and fractures in the bedrock create the potential for sliding surfaces beneath the dam and along temporary and permanent excavated slopes. These hazards require further 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 and that overall dam construction costs should fall within the estimated range presented in the Combined Dam and Fish Passage Alternatives Memorandum (HDR 2014). Consequently, considering cost and other technical factors, the RCC dam type for either the FRO or FRFA project configurations is the preferred option for the site and future design work should proceed under this basis.

Highly fractured zones where large, open fractures occur in the bedrock were identified in the Phase 1 program. These zones will act as preferential seepage pathways beneath the dam foundation and abutment unless they are treated. The Phase 1 borehole data have confirmed that foundation grouting to treat the fractured zones and reduce seepage through them will be an important 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 weaknesses in the 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).

No active faults were identified at the dam site. The ground motion hazard at the Chehalis site is significant and will be a primary consideration in the development of the dam cross-section configuration as well as the structural design of spillway and outlet works facilities. From a design standpoint, PGA at the site could exceed 0.8 g.

Twenty-three landslides have been confirmed in the reservoir basin. The project design must consider how these landslides may affect dam and 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 from landslides activated by normal operations and floods. Landslide remediation/stabilization represents a potential significant cost and should be further evaluated in the next phase of work.
Establishing the appropriate foundation excavation objective (i.e., the estimated excavation limit that provides suitable strength and deformability characteristics to meet design requirements) will be a critical element of work during future design and construction activities. The Phase 1 program results suggest that a combined site characterization approach using both boreholes with RQD, RMR, and weathering parameters established from proper logging procedures along with a combination of surface seismic refraction and downhole televiwer geophysical exploration methods will be suitable to establish this objective going forward. Future Site Characterization Phases should be performed concentrating on boreholes and geophysical surveys to support completion of the 3-D subsurface model initiated as part of this study. Additional data will provide the ability to evaluate prominent joint sets, in-situ hydraulic conductivities, and excavation objective and limits along the entire dam foundation.
References

8 References

8.1 GENERAL

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8.2 SEISMIC HAZARD ANALYSIS


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NGDC (National Geophysical Data Center), 1999, National Geophysical Data Center, Geospatial Data and Services, available at: http://maps.ngdc.noaa.gov/viewers/hazards/


Wells, R.E., 2015, personal communication.


Figures
Transition from Main RCC Dam to Saddle Embankment Dam

Footprint of Proposed Multipurpose RCC Dam Shown

Embankment Saddle Dam Footprint

To Pe Ell

Proposed Dam Site

Chehalis Dam Vicinity Map

DATE 7-24-2015
FIGURE 2.2-1
Notes:
1) Dips 6.0 by Rocscience was used to create stereonet
2) Pole vectors (perpendicular to actual dip vectors) for discontinuities are used to aid in visualizing the data set
**Chehalis Dam**

Main Dam Axis

Cross Section A-A'

**Date:** 7-24-2015

**Figure:** 5.1-2

**Legend:**

- **BH-1** Borehole Location
- **SEISMIC LINE** Seismic Line Location
- **CROSS SECTION** Cross Section for Geologic Interpretation
- **APPROXIMATE GEOLOGIC CONTACT** Approximate Geologic Contact

**GEOLOGIC UNITS:**

- **Qls** - Landslide Deposits
- **Qc** - Colluvium
- **Qos** - Overburden Soil (undifferentiated - cross section only)
- **Qao** - Older Alluvium
- **Tml** - McIntosh Formation: Laminated Siltstone
- **Tig** - Intrusive Volcanic Rocks: Basalt/Gabbro
- **Tcb** - Crescent Formation: Pillow Basalt Flows
- **Tcs** - Crescent Formation: Siltstone (Interbedded with pillow basalt flows)

**Zones of Possible Supplemental Foundation Treatment Requirements:**

- See Figure 5.8

**Approximate Limits of Grout Curtain:**

- See Figure 5.2-1 for details

**Approximate Range of Foundation Excavation Objective:**

- See Figure 5.4

**Chehalis Dam**

Main Dam Axis

Cross Section A-A'

CONTOUR INTERVAL = 5 FEET

COORDINATES IN UTM

**CONTOUR**

**DATE**

**FIGURE**
Chehalis Dam
Saddle Dam Axis
Cross Section B-B'

LEGEND

BOREHOLE LOCATION
SEISMIC LINE LOCATION
CROSS SECTION FOR GEOLOGIC INTERPRETATION
APPROXIMATE GEOLOGIC CONTACT

APPROXIMATE LIMITS
LANDSLIDE DEPOSITS
COLLUVIUM
OVERBURDEN SOIL
(undifferentiated - cross section only)
SLIDER ALLUVIUM

SADDLE DAM FOUNDATION
EXCAVATION OBJECTIVE
APPROXIMATE RANGE OF RCC DAM FOUNDATION
APPROXIMATE LIMITS OF GROUT CURTAIN
ZONE OF POSSIBLE SUPPLEMENTAL FOUNDATION TREATMENT REQUIREMENTS
APPROXIMATE EMBANKMENT DAM FOUNDATION ELEVATION OBJECTIVE

CONTOUR INTERVAL = 5 FEET
COORDINATES IN UTM

DATE
7-24-2015
FIGURE
5.1-3

HOUR
60 0 60 120

STATION (FT)

ELEVATION (Ft Msl)

TRANSLATION FROM MAIN RCC DAM TO SADDLE EMBANKMENT DAM
SEISMIC LINE 1 SPREAD 3 & 4
EMBANKMENT SADDLE DAM FOOTPRINT

SEISMIC LINE 1 SPREAD 5

MAIN DAM AXIS ALIGNMENT
SADDLE DAM AXIS ALIGNMENT

Tml
Tcb
Qc

Embankment Saddle Dam Footprint

BH-1
SEISMIC LINE LOCATION
CROSS SECTION FOR GEOLOGIC INTERPRETATION
APPROXIMATE GEOLOGIC CONTACT

GEOLOGIC UNITS:

Qls  - LANDSLIDE DEPOSITS
Qc   - COLLUVIUM
Qos  - OVERBURDEN SOIL
(Qundifferentiated - cross section only)
Qao - OLDER ALLUVIUM
Tml - MCINTOSH FORMATION: Laminated Siltstone
Tig  - INTRUSIVE VOLCANIC ROCKS: Basalt/Gabbro
Tcb  - CRESCENT FORMATION: Pillow Basalt Flows
Tcs  - CRESCENT FORMATION: Siltstone
(Interbedded with pillow basalt flows)
LANDSLIDE COMPLEX

BOREHOLE LOCATION
SEISMIC LINE LOCATION
CROSS SECTION FOR GEOLOGIC INTERPRETATION
APPROXIMATE GEOLOGIC CONTACT

APPROXIMATE LIMITS
LANDSLIDE DEPOSITS
COLLUVIUM
OVERBURDEN SOIL
(undifferentiated - cross section only)
SLIDER ALLUVIUM

SADDLE DAM FOUNDATION
EXCAVATION OBJECTIVE
APPROXIMATE RANGE OF RCC DAM FOUNDATION
APPROXIMATE LIMITS OF GROUT CURTAIN
ZONE OF POSSIBLE SUPPLEMENTAL FOUNDATION TREATMENT REQUIREMENTS
APPROXIMATE EMBANKMENT DAM FOUNDATION ELEVATION OBJECTIVE

CONTOUR INTERVAL = 5 FEET
COORDINATES IN UTM

DATE
7-24-2015
FIGURE
5.1-3

HOUR
60 0 60 120

STATION (FT)

ELEVATION (Ft Msl)

TRANSLATION FROM MAIN RCC DAM TO SADDLE EMBANKMENT DAM
SEISMIC LINE 1 SPREAD 3 & 4
EMBANKMENT SADDLE DAM FOOTPRINT

SEISMIC LINE 1 SPREAD 5

MAIN DAM AXIS ALIGNMENT
SADDLE DAM AXIS ALIGNMENT

Tml
Tcb
Qc

Embankment Saddle Dam Footprint

BH-1
SEISMIC LINE LOCATION
CROSS SECTION FOR GEOLOGIC INTERPRETATION
APPROXIMATE GEOLOGIC CONTACT

GEOLOGIC UNITS:

Qls  - LANDSLIDE DEPOSITS
Qc   - COLLUVIUM
Qos  - OVERBURDEN SOIL
(Qundifferentiated - cross section only)
Qao - OLDER ALLUVIUM
Tml - MCINTOSH FORMATION: Laminated Siltstone
Tig  - INTRUSIVE VOLCANIC ROCKS: Basalt/Gabbro
Tcb  - CRESCENT FORMATION: Pillow Basalt Flows
Tcs  - CRESCENT FORMATION: Siltstone
(Interbedded with pillow basalt flows)
LANDSLIDE COMPLEX

BOREHOLE LOCATION
SEISMIC LINE LOCATION
CROSS SECTION FOR GEOLOGIC INTERPRETATION
APPROXIMATE GEOLOGIC CONTACT

APPROXIMATE LIMITS
LANDSLIDE DEPOSITS
COLLUVIUM
OVERBURDEN SOIL
(undifferentiated - cross section only)
SLIDER ALLUVIUM

SADDLE DAM FOUNDATION
EXCAVATION OBJECTIVE
APPROXIMATE RANGE OF RCC DAM FOUNDATION
APPROXIMATE LIMITS OF GROUT CURTAIN
ZONE OF POSSIBLE SUPPLEMENTAL FOUNDATION TREATMENT REQUIREMENTS
APPROXIMATE EMBANKMENT DAM FOUNDATION ELEVATION OBJECTIVE

CONTOUR INTERVAL = 5 FEET
COORDINATES IN UTM

DATE
7-24-2015
FIGURE
5.1-3

HOUR
60 0 60 120

STATION (FT)

ELEVATION (Ft Msl)

TRANSLATION FROM MAIN RCC DAM TO SADDLE EMBANKMENT DAM
SEISMIC LINE 1 SPREAD 3 & 4
EMBANKMENT SADDLE DAM FOOTPRINT

SEISMIC LINE 1 SPREAD 5

MAIN DAM AXIS ALIGNMENT
SADDLE DAM AXIS ALIGNMENT

Tml
Tcb
Qc

Embankment Saddle Dam Footprint

BH-1
SEISMIC LINE LOCATION
CROSS SECTION FOR GEOLOGIC INTERPRETATION
APPROXIMATE GEOLOGIC CONTACT

GEOLOGIC UNITS:

Qls  - LANDSLIDE DEPOSITS
Qc   - COLLUVIUM
Qos  - OVERBURDEN SOIL
(Qundifferentiated - cross section only)
Qao - OLDER ALLUVIUM
Tml - MCINTOSH FORMATION: Laminated Siltstone
Tig  - INTRUSIVE VOLCANIC ROCKS: Basalt/Gabbro
Tcb  - CRESCENT FORMATION: Pillow Basalt Flows
Tcs  - CRESCENT FORMATION: Siltstone
(Interbedded with pillow basalt flows)
LANDSLIDE COMPLEX

BOREHOLE LOCATION
SEISMIC LINE LOCATION
CROSS SECTION FOR GEOLOGIC INTERPRETATION
APPROXIMATE GEOLOGIC CONTACT

APPROXIMATE LIMITS
LANDSLIDE DEPOSITS
COLLUVIUM
OVERBURDEN SOIL
(undifferentiated - cross section only)
SLIDER ALLUVIUM

SADDLE DAM FOUNDATION
EXCAVATION OBJECTIVE
APPROXIMATE RANGE OF RCC DAM FOUNDATION
APPROXIMATE LIMITS OF GROUT CURTAIN
ZONE OF POSSIBLE SUPPLEMENTAL FOUNDATION TREATMENT REQUIREMENTS
APPROXIMATE EMBANKMENT DAM FOUNDATION ELEVATION OBJECTIVE

CONTOUR INTERVAL = 5 FEET
COORDINATES IN UTM

DATE
7-24-2015
FIGURE
5.1-3

HOUR
60 0 60 120

STATION (FT)

ELEVATION (Ft Msl)
Main Dam Axis Boreholes
RMR/RQD/Lugeon Values vs. Depth

7-24-2015

DATE
FIGURE 5.2-1
Scatter Plot of Hydraulic Conductivity vs. Depth

In-Situ Hydraulic Conductivity (cm/sec)

Notes:
1) Ewert, 2003
2) Weaver and Bruce, 2007
Compression Wave Velocity Contours, Seismic Line 1, Left Abutment Along the Dam Axis
BH-2 Optical Televiewer 84-88 ft

**Key**
- 0: No joint/fracture
- 1: Major open joint/fracture
- 2: Minor open joint/fracture
- 3: Partially open joint/fracture
- 4: Sealed joint/fracture
- 5: Bedding

**Lithology**

**BH-2 Wulf Plots**

**Example Optical Televiewer Results**

7-24-2015
NOTES:
1) Modified from Wells and Simpson (2001).
2) Red star shows project site location.
3) Gray arrows indicate relative plate motions.
4) Block motions (circled numbers) are in mm/yr.
NOTES: 1) Faults and folds from WA Dept. Natural Resources and USGS Quaternary Fold and Fault Database.
2) Earthquakes from Pacific Northwest Seismic Network, University of Washington

Chehalis River Dam Site: Pe Ell, Washington

SEISMICITY AND ACTIVE FAULTS IN WASHINGTON AND OREGON
CASCADIA SUBDUCTION ZONE

PLATE BOUNDARIES

LEGEND

A
Typical Geologic Cross Section Location

NOTES

2. Typical geologic cross section figure adapted from Wells and Others (2000).

Typical Geologic Cross Section

Not to Scale

Subduction Zone Mega-Thrust Earthquakes (1700)

Intraslab Earthquakes (1949, 1965, 2001)

Cascadia Subduction Zone Forearc

North American Plate

Canada

Washington

British Columbia

Vancouver

Sea Otter Fault

Seattle Fault

Crustal Earthquakes

TYPICAL GEOLOGIC CROSS SECTION

Chehalis River Dam Site
Pe Ell, Washington

CASCADIA SUBDUCTION ZONE

June 2015
21-1-21897-010

SHANNON & WILSON, INC.

FIG. 5.6-4
NOTE

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

Chehalis River Dam Site
Pe Ell, Washington

5% DAMPING HORIZONTAL UHS

June 2015 21-1-21897-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants FIG. 6.2-1
1. CSZ = Cascadia subduction zone
   g = acceleration due to gravity

NOTE

Chehalis River Dam Site
Pe Ell, Washington

TOTAL HAZARD CURVES
PEAK GROUND ACCELERATION

June 2015
21-1-21897-010

SHANNON & WILSON INC.
Geotechnical and Environmental Consultants

FIG. 6.2-2
CRD_Hazard Curves.xlsx 6/30/2015

FIG. 6.2-3

Chehalis River Dam Site
Pe Ell, Washington

TOTAL HAZARD CURVES
0.2 SECOND PERIOD

June 2015 21-1-21897-010
SHANNON & WILSON INC.
Geotechnical and Environmental Consultants FIG. 6.2-3

NOTE
1. CSZ = Cascadia subduction zone
g = acceleration due to gravity
1. CSZ = Cascadia subduction zone
2. g = acceleration due to gravity

Chehalis River Dam Site
Pe Ell, Washington

TOTAL HAZARD CURVES
1.0 SECOND PERIOD

June 2015
21-1-21897-010

SHANNON & WILSON INC.
Geotechnical and Environmental Consultants

FIG. 6.2-4
<table>
<thead>
<tr>
<th>Hazard Contribution</th>
<th>Contribution to Hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crustal Faults</td>
<td>1.4%</td>
</tr>
<tr>
<td>Crustal Background</td>
<td>7.1%</td>
</tr>
<tr>
<td>CSZ Interface</td>
<td>76.0%</td>
</tr>
<tr>
<td>CSZ Intraslab</td>
<td>15.5%</td>
</tr>
</tbody>
</table>

**NOTE**

1. Mw = moment magnitude  
   Rrup = source-to-site rupture distance  
   $\varepsilon_0$ = epsilon zero, a measure of the ground motion variation from the mean  
   km = kilometers  
   CSZ = Cascadia subduction zone
1. Mw = moment magnitude
Rrup = source-to-site rupture distance
$\varepsilon_0$ = epsilon zero, a measure of the ground motion variation from the mean
km = kilometers
CSZ = Cascadia subduction zone

<table>
<thead>
<tr>
<th>Hazard Contribution</th>
<th>Contribution to Hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crustal Faults</td>
<td>1.3%</td>
</tr>
<tr>
<td>Crustal Background</td>
<td>6.6%</td>
</tr>
<tr>
<td>CSZ Interface</td>
<td>72.9%</td>
</tr>
<tr>
<td>CSZ Intraslab</td>
<td>19.2%</td>
</tr>
</tbody>
</table>

**Chehalis River Dam Site**
Pe Ell, Washington

2,475-YEAR DEAGGREGATION
0.2-SECOND PERIOD HAZARD

June 2015 21-1-21897-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6.2-6
1. Mean Mw 8.7
Mean Rrup (km) 55
Mean $\varepsilon_0$ 0.98

<table>
<thead>
<tr>
<th>Hazard Contribution</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crustal Faults</td>
<td>0.4%</td>
</tr>
<tr>
<td>Crustal Background</td>
<td>1.5%</td>
</tr>
<tr>
<td>CSZ Interface</td>
<td>92.4%</td>
</tr>
<tr>
<td>CSZ Intraslab</td>
<td>5.7%</td>
</tr>
</tbody>
</table>

Hazard Contribution

- Crustal Faults: 0.4%
- Crustal Background: 1.5%
- CSZ Interface: 92.4%
- CSZ Intraslab: 5.7%

NOTE
1. Mw = moment magnitude
2. Rrup = source-to-site rupture distance
3. $\varepsilon_0$ = epsilon zero, a measure of the ground motion variation from the mean
4. km = kilometers
5. CSZ = Cascadia subduction zone

Chehalis River Dam Site
Pe Ell, Washington

2,475-YEAR DEAGGREGATION
1.0-SECOND PERIOD HAZARD

June 2015

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants
1. Spectra were calculated at the ground surface for rock outcrop condition (Vs30=1,500 meters per second).

2. Crustal faults and CSZ intraslab spectra were calculated for median (50th percentile) and CSZ interface spectrum was calculated for median plus one standard deviation (84th percentile).
Oblique Perspective of Geologic Sections Used to Develop 3D Model of the Chehalis Dam Site

DATE
7-24-2015

FIGURE
6.3-1