Chehalis Basin Strategy

Phase 3 Landslide Evaluation

May 10, 2019

This report prepared for the Office of Chehalis Basin

This report prepared by Shannon & Wilson, Inc.
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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
BC	boundary condition
CUTX	Consolidated undrained triaxial compression
FRE	Flood Retention Expandable
FRE Facility	The entire FRE facility, including reservoir, FRE structure, tunnels, gates and flow control, fish passage and handling facilities, plunge pool, operations and maintenance buildings, etc.
FRE Structure	The potential dam across the Chehalis River upstream of Pe Ell, Washington that would be part of the FRE facility.
FS	factor of safety
LRFD	load and resistance factor design
NGVD29	National Geodetic Vertical Datum of 1929
PGA	peak ground acceleration
Project	Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
VWP	vibrating wire piezometer
WSDOT	Washington State Department of Transportation

EXECUTIVE SUMMARY

This report discusses stability considerations or stability analyses results for nine landforms mapped within the Chehalis basin. These landforms include landslides, landslide deposits, and colluvium. These landforms, if they mobilize as landslides, have the potential to impact the proposed Flood Retention Expandable (FRE) facility, its operation, or reservoir impounded by the facility. Stability analyses were not conducted for three of the nine landforms: LS-18, LS-19, and LS-26.

- LS-18 and LS-19 are above the design FRE facility 100-year (return period) storm reservoir elevation. Therefore, stability of LS-18 and LS-19 would not be impacted by the reservoir.
- LS-26 was not analyzed because it is downstream of the FRE structure and will be excavated or stabilized during FRE structure and FRE facility construction.

Landforms LS-5 and LS-10, both upstream of the FRE structure, are classified as landslide deposits. Neither of these deposits show signs of movement or shearing. The stability analyses results, using representative soil properties for landslide deposits in the reservoir valley, indicate LS-5 and LS-10 should remain stable during reservoir inundation and drawdown.

Landform LS-11 is upstream of the FRE structure on the left side of the Chehalis River. The uppermost portion of LS-11, above about elevation 675 feet, is an active landslide. This active landslide is within the greater LS-11 mass. The greater LS-11 mass is a landslide deposit that extends to the Chehalis River. The failure surface for the active landslide within LS-11 is above the FRE facility 100-year reservoir elevation. Therefore, the calculated factors of safety (FSs) for this higher elevation active landslide do not change with reservoir inundation or drawdown. The stability analyses results indicate the lower elevation LS-11 material is currently stable and should remain stable during FRE facility 100-year reservoir inundation and drawdown.

Landform LS-13 is upstream of the FRE structure, on the left side of the Chehalis River. LS-13 is classified as a landslide deposit. It exhibits no signs of activity or movement. The stability analyses results indicate LS-13 is currently stable and should remain stable during FRE facility 100-year reservoir inundation and drawdown.

The stability analyses results also indicate that while the lower slopes of LS-11 and LS-13 will likely have an FS greater than 1.1 during reservoir inundation/drawdown, they may be susceptible to retrogressive failure should lower slope movement occur. Based on the geologic interpretation and analyses results, should movement of these four landforms occur, the movement is unlikely to be rapid or impact the FRE facility or FRE facility reservoir operations.

Landforms LS-3 and LS-4, previously analyzed for static conditions, were analyzed for seismic stability for the post-construction FRE facility condition, wherein drainage and toe buttresses were assumed to have

been constructed to improve stability of LS-3 and LS-4. LS-3 and LS-4 are upstream of and relatively near the FRE structure. Pseudo-static (i.e., seismic) stability analyses of these landforms were performed for this study to compute the yield acceleration, i.e., the seismic-event-induced horizontal ground acceleration that would reduce their FS to 1.0. Comparison of these computed yield accelerations to the estimated peak ground acceleration associated with different return period seismic events determined a minimum seismic event return period of 150 years to cause a yield acceleration for the lowest strength soil shear strength condition considered. Therefore, seismic-event-induced movement of these landslides is not deemed to present a significant risk to the FRE facility or structure.

Seismic analyses were not performed for other landforms in the reservoir evaluated for the current study presented in this report and previously evaluated for static stability. These landforms are at distances upstream of the FRE structure such that their movement or failure during a seismic event, should such movement occur, would not directly impact the FRE structure. An assumption was made that the reservoir would not be present during the seismic event because the probability is low for simultaneous occurrence of flooding resulting in an elevated reservoir and a strong seismic event.

1 INTRODUCTION

1.1 Project Background

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 restoration opportunities for aquatic species. One flood damage reduction measure under consideration for the Project is the construction of a flood retention structure (i.e., dam) across the Chehalis River about one mile south of Pe Ell in Lewis County, Washington (Figure 1).

One flood retention structure type is under consideration: the FRE dam (FRE structure). The FRE facility does not include a permanent reservoir. The FRE facility would operate without impounding water except during a major flood downstream in the Chehalis-Centralia area (Anchor QEA, 2017). When a major flood event in the Chehalis-Centralia area is projected, and as river flow increases, gates on openings through the FRE structure would close, resulting in impoundment of a reservoir. As the flood event passes, the gates would be opened and reservoir drawn down at a predetermined rate until the river was again free flowing through the FRE structure.

The 100-year flood event is used for landslide stability analyses because a 100-year flood is a standard used for evaluating flood reduction projects. Anchor QEA (2017) compares recent flood events and discusses flood impoundment and release procedures. During the design 100-year storm flood event, water would be stored in the reservoir to a maximum elevation of 604.4 feet National Geodetic Vertical Datum of 1929 (NGVD29). (Elevations in this report are referenced to NGVD29).

Twenty-seven landforms/landslides that have a potential to present a hazard to the FRE facility, its operation, or the FRE facility reservoir have been mapped within the Chehalis basin within or near the proposed reservoir that would be created by FRE facility construction (Shannon & Wilson, 2015, 2016, and 2017). Movement of these landforms/landslides could be triggered by reservoir inundation and drawdown. Table 1 presents a summary of the landforms.

1.2 Landslide Stability Evaluation Overview

Some of the landforms within the reservoir are near the FRE facility and/or large enough that, if fully mobilized, could threaten the FRE facilities and operation. Shannon & Wilson (2017a) previously analyzed stability of 10 of the 27 landforms. Based on the results of those analyses, Shannon & Wilson (2017b) developed concept alternatives for stability improvements for some of these landforms where reservoir operation could decrease stability and movement could impact FRE facility construction and operation.

For the current study, four primary sources of information are used to estimate subsurface conditions at each landslide: Shannon & Wilson, 2015, 2017a, 2017b, and 2019.

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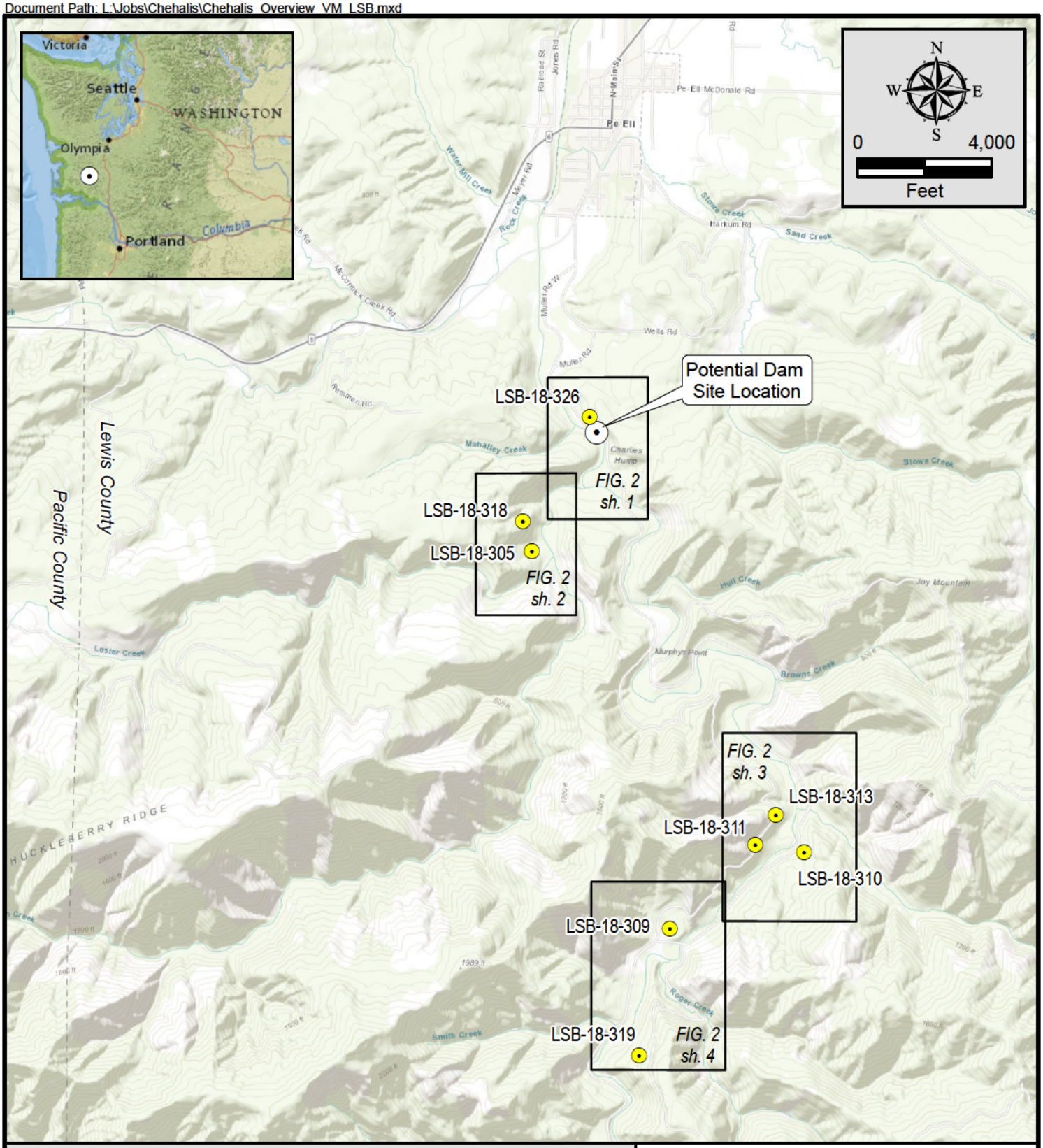

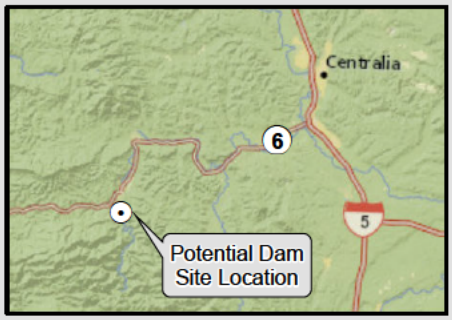

LEGEND

- 2018 Boring Locations
- Potential Dam Site Location
- Figure 2 Index

Chehalis Basin Strategy
Phase 3 Landslide Evaluation
Pe Ell, Washington

VICINITY MAP

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FIG. 1

This report discusses stability considerations or stability analyses results for seven landforms mapped within the Chehalis Basin that were not analyzed previously and have the potential to impact the proposed FRE facility, its operation, or the FRE facility reservoir. Each landslide for which stability analyses results are presented in this report was evaluated for:

- Stability under current free-flowing river conditions
- Stability changes resulting from inundation and drawdown

A consistent set of soil and rock engineering parameters was used for the analyses. The time steps used to model reservoir inundation and drawdown were consistent across the analyses.

2 OVERVIEW OF PREVIOUS SITE STUDIES

2.1 Previous Landslide Stability Reports

Results from previously performed stability analyses for some of the 27 landforms in the Chehalis River valley upstream of the potential FRE structure are presented in Shannon & Wilson 2014, 2017a, and 2017b. These reports present stability analyses results and potential stability improvement measures for some of the landforms.

2.2 Geology and Surface Conditions

The geologic setting, subsurface, and surface conditions for the Project are described in Shannon & Wilson 2015, 2017a, and 2017b. Geologic data collected through exploration programs for the Project are presented in Shannon & Wilson, 2019.

2.3 Phase 3 Landslide Explorations

Eight borings were drilled from June to August 2018 to assess subsurface conditions at eight identified landforms in and adjacent to the potential FRE facility reservoir. These borings were completed at landforms LS-5, LS-9, LS-10, LS-11, LS-13, LS-18, LS-19, and LS-26, at the locations shown in Figures 1, 2, and 3. The boring logs for these explorations can be found in the Phase 3 Geotechnical Data Report (Shannon & Wilson, 2019).

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LS-26

LSB-18-326

LS-27

Potential Dam
Site Location

Mahaffey Creek

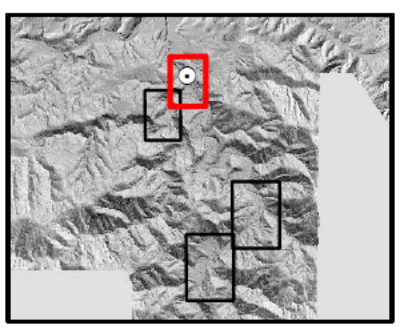
LS-3a

LS-3

Crim Creek

Chehalis River

LS-4



LEGEND

Landslide Location
and Designation

LS-26

Potential Dam
Site Location

2018 Boring Locations

NOTE:
Hillshade generated using ESRI
3D Analyst from LiDAR data provided
by HDR. NAVD 88, Illumina ion 315

N
W E
S

0 300
Feet

Chehalis Basin Strategy
Phase 3 Landslide Evaluation
Pe Ell, Washington

SITE PLAN

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FIG. 2
Page 1 of 4

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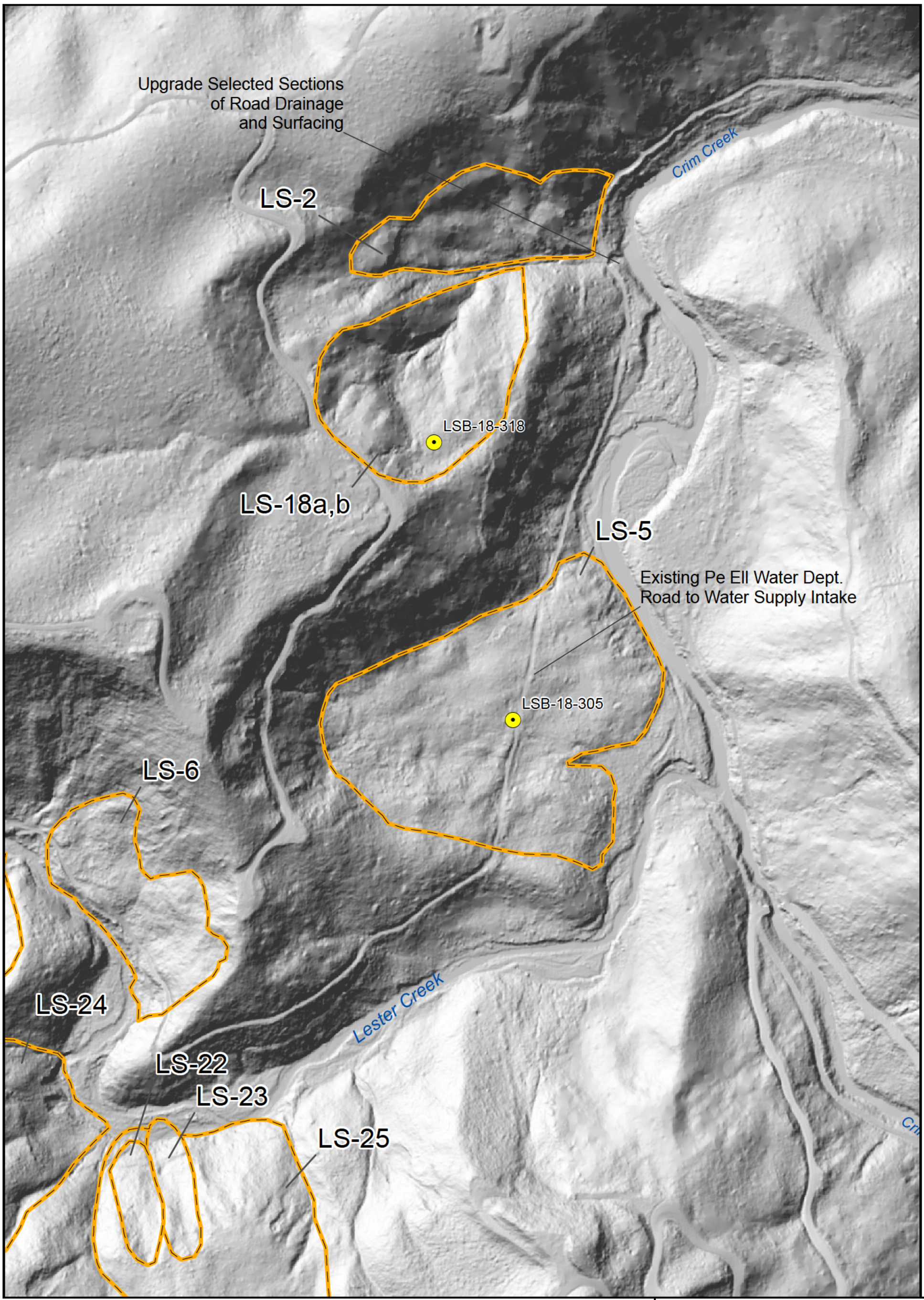

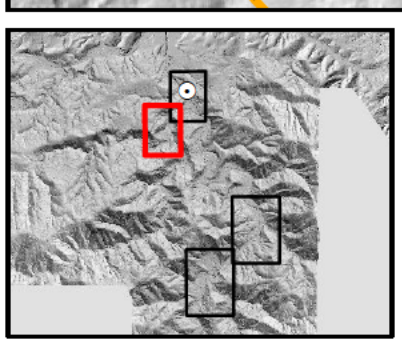

LEGEND

Landslide Location and Designation — LS-26

Potential Dam Site Location

2018 Boring Locations

NOTE:
Hillshade generated using ESRI 3D Analyst from LiDAR data provided by HDR. NAVD 88, Illumina ion 315

N W E S

0 300
Feet

Chehalis Basin Strategy
Phase 3 Landslide Evaluation
Pe Ell, Washington

SITE PLAN

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FIG. 2
Page 2 of 4

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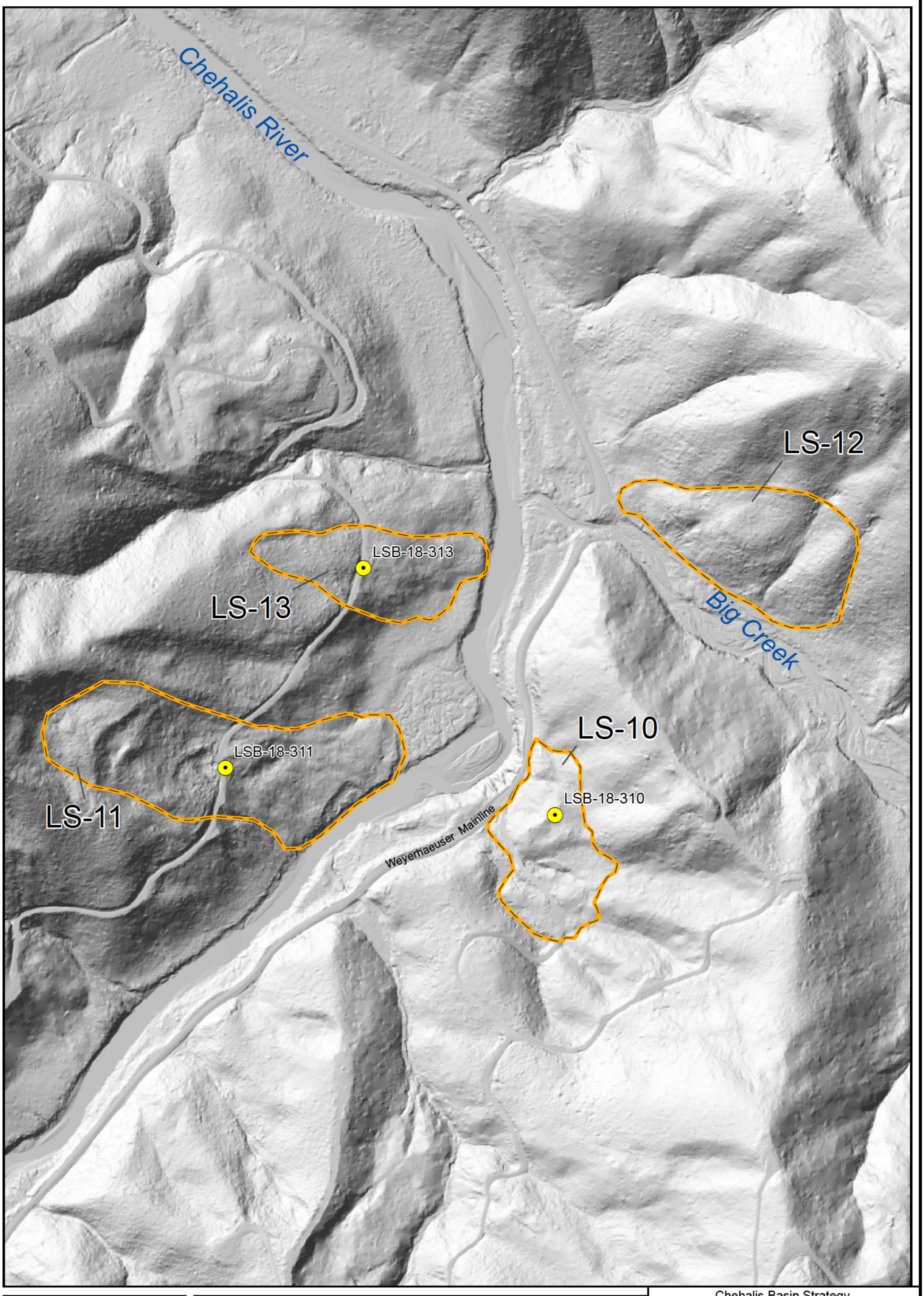

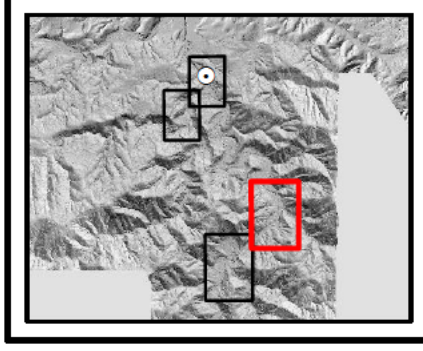

LEGEND

Landslide Location and Designation — LS-26

Potential Dam Site Location

2018 Boring Locations

NOTE:
Hillshade generated using ESRI 3D Analyst from LiDAR data provided by HDR. NAVD 88, Illumina ion 315

N W E S

0 300
Feet

Chehalis Basin Strategy
Phase 3 Landslide Evaluation
Pe Ell, Washington

SITE PLAN

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FIG. 2
Page 3 of 4

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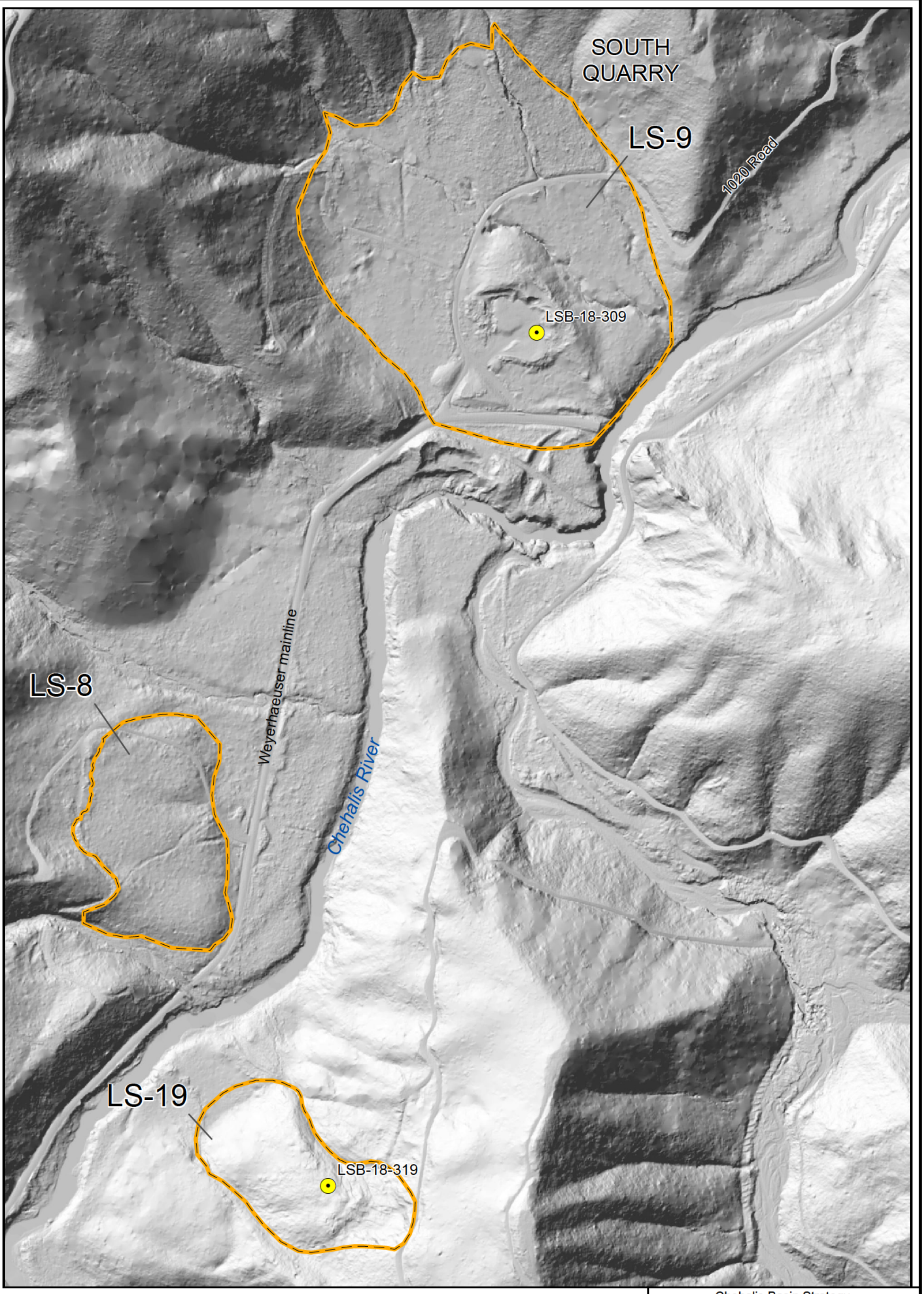

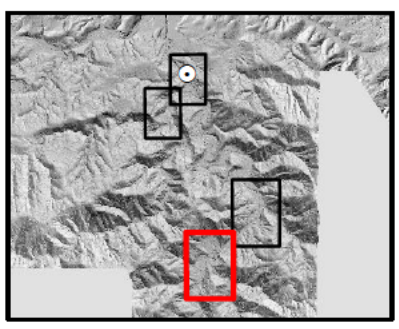

LEGEND

Landslide Location and Designation — LS-26

Potential Dam Site Location

2018 Boring Locations

NOTE:
Hillshade generated using ESRI 3D Analyst from LiDAR data provided by HDR. NAVD 88, Illumina ion 315

N W E S

0 300
Feet

Chehalis Basin Strategy
Phase 3 Landslide Evaluation
Pe Ell, Washington

SITE PLAN

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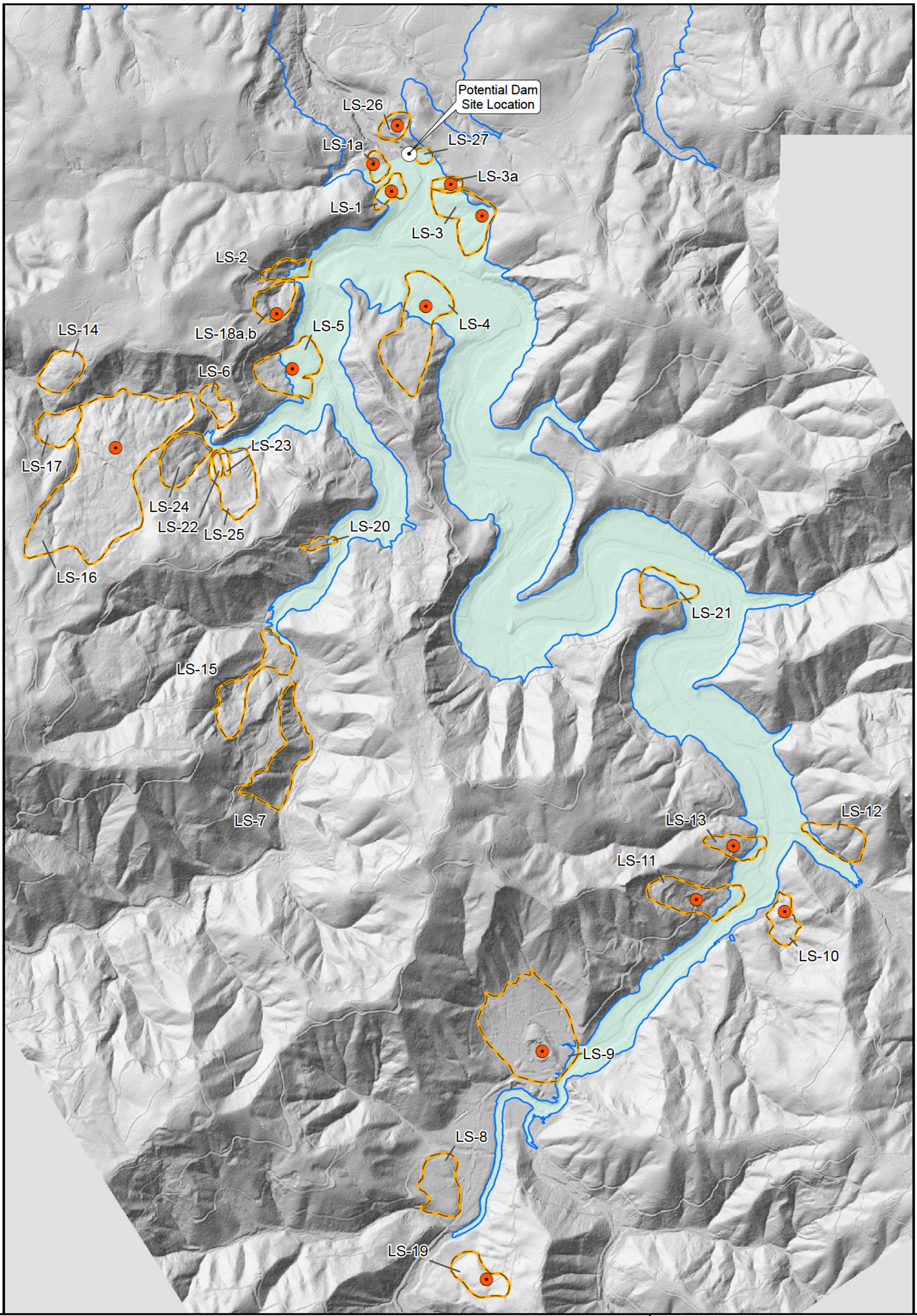
FIG. 2
Page 4 of 4



LEGEND

Exploration - Landslide Site

Shannon & Wilson, Inc.
Field Landform Review
Location & Designation

Area of Interest LS-19

Potential Dam Site Location

Dam Scenario Contour Elevation
604 Feet - NAVD 88

N W E S

0 1,500
Feet

Chehalis Phase 3
Landslide Evaluation
Pe Ell, Washington

SITE PLAN
WITH FRE DAM
100 YEAR INUNDATION

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FIG. 3

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2.4 Laboratory Testing

Laboratory testing was performed on selected samples retrieved from the 2018 explorations to determine index parameters, strength parameters, and hydraulic conductivity of soils encountered in the borings. Phase 3 laboratory testing included two types of shear strength tests: (1) torsional ring shear and (2) consolidated undrained triaxial compression (CUTX). Torsional ring shear tests were performed on soil samples from borings LSB-18-310, LSB-18-319, and LSB-18-326. Torsional ring shear tests were completed at two strength states:

1. Fully softened strength (intact colluvium or landslide deposit material)
2. Residual strength (soils on a potential landslide shear or failure surface)

CUTX tests and hydraulic conductivity tests were performed on the same samples. The soil used for the CUTX tests were from borings DB-1, DB5, DB-7, LSB-3, and TB-5. Shannon & Wilson (2019) contains laboratory test details and results.

Shear strength parameters selected from the ring shear and CUTX test results for use in analyses are presented in Section 3.4.1.

3 LANDSLIDE ANALYSIS METHODOLOGY

This section summarizes the methodology for the Phase 3 FRE facility slope stability evaluation. Seven landforms were considered: LS-5, LS-10, LS-11, LS-13, LS-18, LS-19, and LS-26. Figures 2 and 3 show the landform locations.

These landforms were selected because they had not been previously analyzed or new subsurface information was obtained for them from the Phase 3 explorations. These landforms, if mobilized, have the potential to impact the FRE facility or its operation. Stability of landforms within the reservoir could be reduced by inundation and drawdown as reservoir water levels change.

To evaluate stability of each landform profile, a coupled seepage and global slope stability computer model was prepared using the software suite GeoStudio Version 9. The seepage module of GeoStudio, SEEP/W (Geo-Slope International, 2018a) is a two-dimensional, finite-element seepage analysis program that simulates fluid flow and pressure distribution in saturated and unsaturated materials such as soil and rock. The global stability module of GeoStudio, SLOPE/W (Geo-Slope International, 2018b) uses limit equilibrium analysis methods. The porewater pressures used in the SLOPE/W global stability analyses were derived from the results of SEEP/W analyses.

3.1 Flood Retention Expandable (FRE) 100-Year Inundation and Drawdown Curves

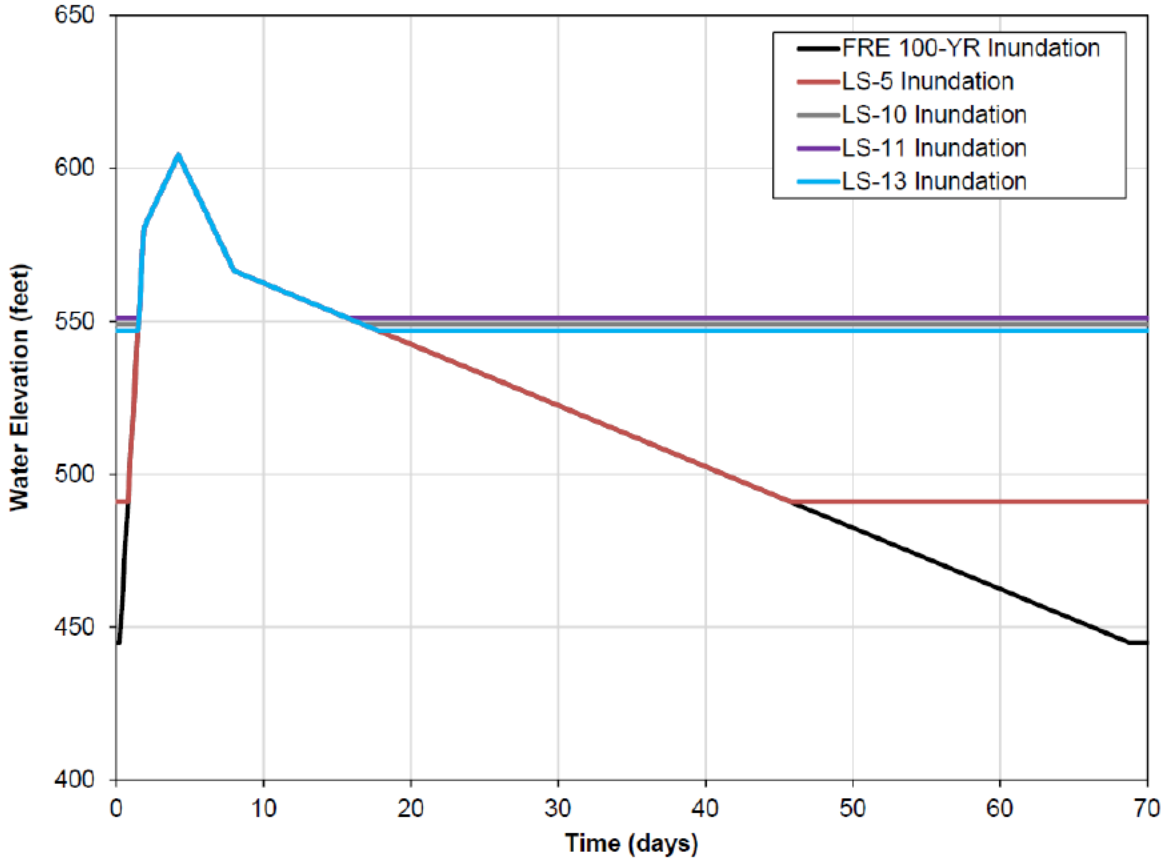
The FRE facility would allow the Chehalis River and various creeks within the flood reservoir footprint to flow unimpeded except during a major flood event. During a major flood event, the FRE facility would impede flow, resulting in a temporary reservoir. Impounded water would inundate reservoir slopes. After the flood event peaked, the reservoir would be drained over a period of days to weeks, to control the flow of released floodwater, until the Chehalis River again flows unimpeded.

The retention facilities will be designed for 10-, 20-, 100-, and 500-year flood events. The seepage and stability analyses were conducted using the 100-year flood event inundation/drawdown water level versus time curves developed by Anchor QEA, LLC. The inundation/ drawdown event curves used in the analyses were approximated as linear segments, as shown in Figure 4.

“Modified” inundation/drawdown event curves were developed for these analyses based on Phase 2 landslide stability analyses results (Shannon & Wilson, 2017a, 2017b). For the FRE facility inundation/drawdown curve, the analyses assumed that after the reservoir reaches the peak 100-year inundation elevation (604.4 feet), drawdown would begin at a rate of 10 feet/day for approximately 3.8 days to a reservoir elevation of 566.5 feet. Then the drawdown rate would decrease to 2 feet/day until the reservoir is drained.

As indicated in Figure 4, the inundation/drawdown curves at LS-5, LS-10, LS-11, and LS-13 consider the site topography. These landforms are thousands of feet upstream of the FRE structure. The inundation/drawdown curves used start and end at higher water elevations than the inundation/drawdown curves at the FRE structure, reflecting the delay in initiation of inundation relative to initial impoundment.

Figure 4
FRE Reservoir Inundation and Drawdown Curves Used for Stability Analysis, 100-Year Flood



3.2 Subsurface Profiles

The 2018 Phase 3 borings encountered various overburden soil units, including loose to very dense Holocene colluvium (Hc), loose to dense Holocene landslide deposits (Hls), and loose to very dense weathered bedrock. These soil units were underlain by several types of bedrock (e.g., claystone, siltstone, and basalt). For the stability analyses, all bedrock was treated as an impenetrable material type, referred to as “bedrock” in the analyses input and output.

The soils above the bedrock were divided into fine-grained colluvium, coarse-grained colluvium, and a weak soil layer. The division between materials was based on the soil descriptions presented in the boring logs. In general, the stability model soil layers were selected to be approximately parallel to the slope ground surface. To analyze the active landslide at the top of the greater LS-11 landform mass, the model included a weak soil layer that followed the assumed base of this active landslide.

Table 1
Landslide Character and Analysis

LANDSLIDE NO.	SIGNS OF ACTIVITY	LANDFORM CLASSIFICATION	PARTIAL INUNDATION BY FRE DAM?	EXPLORATIONS	LABORATORY STRENGTH TESTING	SLOPE STABILITY ANALYSIS	STABILITY IMPROVEMENT NECESSARY	COMMENTS
1		LS		LSB-1		Y	Y	Will be stabilized or removed as part of dam construction
1a		LS		LSB-2		Y	Y	Will be stabilized or removed as part of dam construction
2		D	Y			Y		
3		D	Y	LSB-3	TXCU	Y	Y	
3a	Y	LS	Y	LSB-4	TXCU		Y	Slope failed Jan. 2018
4		D	Y	SW-17-LS4		Y	Y	
5		D	Y	LSB-18-305		Y		
6		D		LSB-6				
7	Y	LS						Shallow failures only. Above maximum reservoir elevation
8		NL						
9		NL		LSB-18-309				
10		D		LSB-18-310	TRS	Y		Above maximum reservoir elevation
11	Y	LS	Y	LSB-18-311		Y		
12		NL	Y					
13		D	Y	LSB-18-313		Y		
14	Y	LS						Above maximum reservoir elevation
15	Y	D	Y					Shallow debris flows from head scarp area.
16		LS		SW-17-LS16a				Above maximum reservoir elevation
17	Y	D						Head scarp earth cracks. Above maximum reservoir elevation
18		D	Y	LSB-18-318		Y		Mostly above maximum reservoir elevation
19	Y	LS		LSB-18-319	TRS			Above maximum reservoir elevation
20	Y	D	Y					Shallow failures only
21		NL	Y					
22		LS	Y					Shallow failures. Mostly above max reservoir elevation
23		LS	Y					Shallow failures. Mostly above max reservoir elevation
24		LS	Y					Shallow failures. Mostly above max reservoir elevation
25		LS	Y					Shallow failures. Mostly above max reservoir elevation
26	Y	LS		LSB-18-326	TRS	Y	Y	Will be stabilized or removed as part of dam construction
27		LS				Y		Will be removed as part of dam construction

Notes:
Explorations completed in 2015, 2017, and 2018.
CUTX = Consolidated-Undrained Triaxial Compression; FRE = Flood Retention Expandable; LS = Landslide, D = Landslide Deposit, NL = Not Classified as a Landslide; TRS = Torsional Ring Shear
Y = Yes, No entry = no or not applicable where alternative column entry is yes (Y).

3.3 Failure Modes

3.3.1 Deep-Seated Failures

LS-5, LS-10, the lower slope of LS-11, and LS-13 are classified as landslide deposits with the potential to become mobilized during FRE facility reservoir inundation/drawdown. The predominant failure mode for these landslide deposits is sliding above the bedrock-colluvium interface. The analyses considered failure surfaces above this contact such that the entire landslide deposit could become mobilized during a failure.

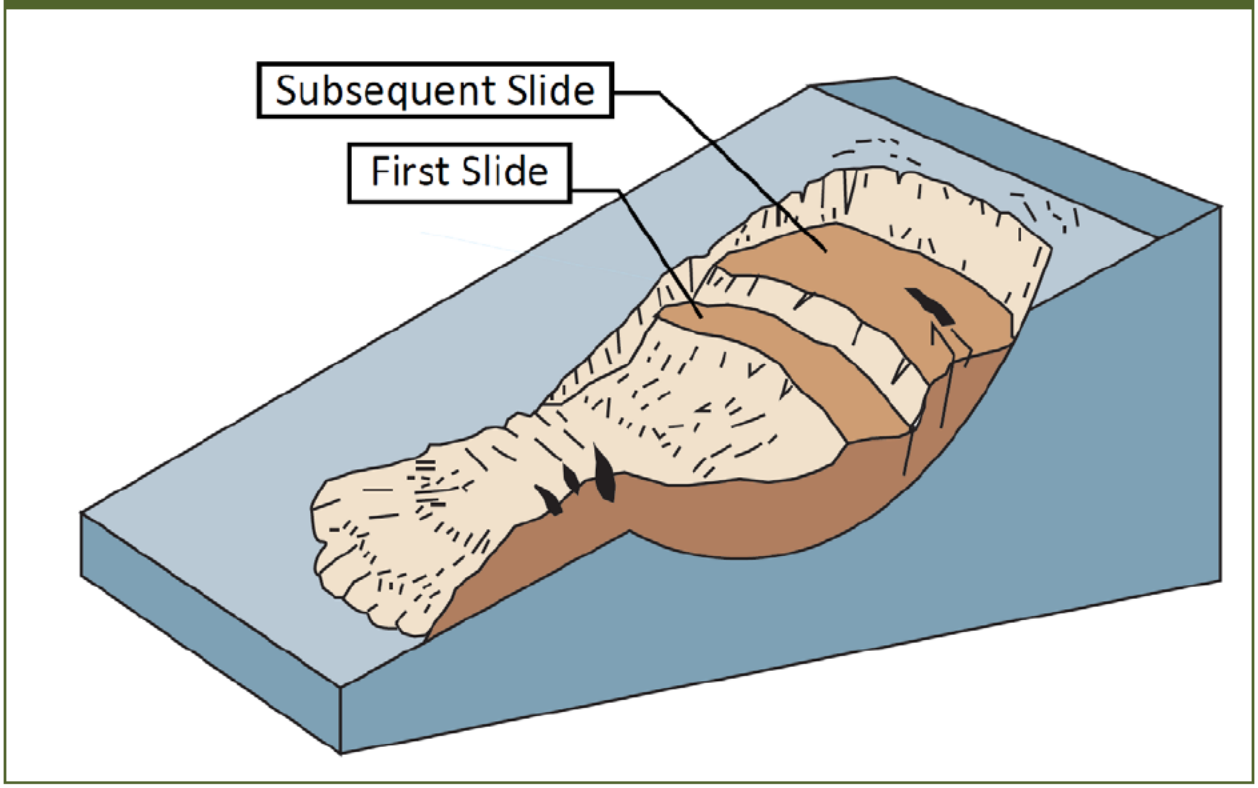
For LS-5, LS-10, and LS-13, the model included a 5-foot-thick layer of slide plane soil above the bedrock. For the lower slope of LS-11, the model assumes colluvium directly overlies the bedrock. For the active landslide at the top of LS-11, the model includes a 5-foot-thick layer of slide plane soil following the interpreted base of this active landslide.

3.3.2 Retrogressive Failures

Reservoir inundation, reservoir drawdown, and high river flow have the potential to decrease stability or erode the toe of LS-11 and LS-13 or create localized landslides near the river channel. Slope toe movement and removal of toe support could result in retrogressive failure of the LS-11 and LS-13 landforms. A retrogressive failure occurs when movement of a downslope portion of a slope results in reduction of support for and subsequent movement of material farther upslope. Repeat inundation and drawdown cycles and erosion could increase the upslope distance to which retrogression failure occurs. Figure 5 illustrates a retrogressive failure.

LS-11 and LS-13 were evaluated for retrogressive failure potential. To do so, three different failure surfaces were analyzed within the slope: (1) failures in the lower third of the slope, (2) failures extending from the toe to the middle third of the slope, and (3) failures extending from the toe to the upper third of the slope. If the FS increased as the failure surfaces become longer, e.g., extend farther uphill, the slope has the potential for retrogressive failure. Both LS-11 and LS-13 were modeled and analyzed the case where soil at the slope toe was removed to an elevation of 605 feet. This condition simulates retrogressive failure or soil removal by erosion to the FRE facility 100-year reservoir elevation.

Figure 5
Typical Retrogressive Failure Surfaces Considered for LS-11 and LS-13 (after U.S. Geological Survey, 2004)



3.4 Soil and Rock Properties

3.4.1 Shear Strength

Table 2 summarizes the soil parameters used in the seepage and slope stability analysis. Shear strength parameters were selected based on the results of ring shear and CUTX testing. Figure 6 summarizes the failure envelopes from these tests represented in shear-normal stress space.

The overburden soil above the bedrock was divided into three primary categories: (1) Fine-grained colluvium, (2) coarse-grained colluvium, and (3) slide plane soil along the soil-bedrock interface.

The two colluvium friction angle shear strength values were selected based on the fully softened ring shear and CUTX results. Samples from the borings used in CUTX testing were from reconditioned and recompacted to representative in situ densities and sheared at confining pressures representative of in situ stresses. Frictional shear strength estimates from CUTX were greater than the fully softened ring shear results. Representative frictional colluvium strengths were chosen based on soil grain size, resulting in friction angles of 31° and 32° for fine- and coarse-grained colluvium, respectively. Failure envelopes from these friction angles are shown in Figure 6 as dotted lines.

Where a clear slide plane was encountered in the borings, a thin layer of weaker soil was modeled along the interpreted slide plane (hereafter referred to as “slide plane soil”). This weaker soil layer was

assumed to be at a residual strength state. Three frictional shear strength values were used for the slide plane layer: 14°, 19°, and 25° (see dashed lines in Figure 6). The three values of slide plane soil bound the residual torsional ring shear results.

Soil shear strength used in previous reports issued by Shannon & Wilson (Shannon & Wilson, 2017a and 2017b) and in the updated LS-3 and LS-4 analyses presented in Section 4.6 of this report were estimated based on in situ Standard Penetration Test (SPT) blow count correlations, soil gradations, and classification. Correlations between SPT blow counts and soil friction angle were obtained from the Washington State Department of Transportation (WSDOT, 2015); Naval Facilities Engineering Command (1986); Peck, Hanson and Thornburn (1974); and American Association of State Highway and Transportation Officials (AASHTO, 2015).
The resulting friction angles were compiled by Unified Soil Classification System (USCS) soil classification (ASTM, 2017) and standard statistical methods were used to calculate the mean, median, standard deviation, and range of calculated friction angles by USCS classification. Three frictional strength values (25°, 30°, and 35°) were selected to bound the range of SPT-correlated friction angles and applied to the overburden soil in each stability analysis scenario completed in part of the 2017 studies (Shannon & Wilson, 2017a and 2017b). The three colluvium friction angles reflected the uncertainty and range of frictional soil strengths developed using the SPT correlations.

Table 2
Engineering Soil Parameters Used for Analysis

SOIL UNIT	USCS	NATURAL UNIT WEIGHT, g_n (pcf)	SATURATED UNIT WEIGHT, g_{sat} (pcf)	SHEAR STRENGTH PARAMETERS		HYDRAULIC PARAMETERS			
				EFFECTIVE STRESS FRICTION ANGLE, ϕ' (degrees)	COHESION INTERCEPT, c' (psf)	HORIZONTAL HYDRAULIC CONDUCTIVITY, k_H (ft/day)	SATURATED VOLUMETRIC WATER CONTENT, w_{VOL}	COEFFICIENT OF VOLUME COMPRESSIBILITY, M_v $(psf)^{-1}$	RATIO k_V/k_H (ft/day)
Fine-Grained Colluvium	ML, MH, CL, CH	115	115	31	0	0.001	0.50	7.E-06	0.2
Coarse-Grained Colluvium	SC, SM, GC, GM	120	120	32	0	0.1	0.46	4.E-06	0.2
Slide Plane Soil	SC, SM, MH, ML	115	115	14	0	0.001	0.50	7.E-06	0.2
				19	0				
				25	0				
Bedrock	N/A	N/A	N/A	*Impenetrable*		0	0.1	0	1

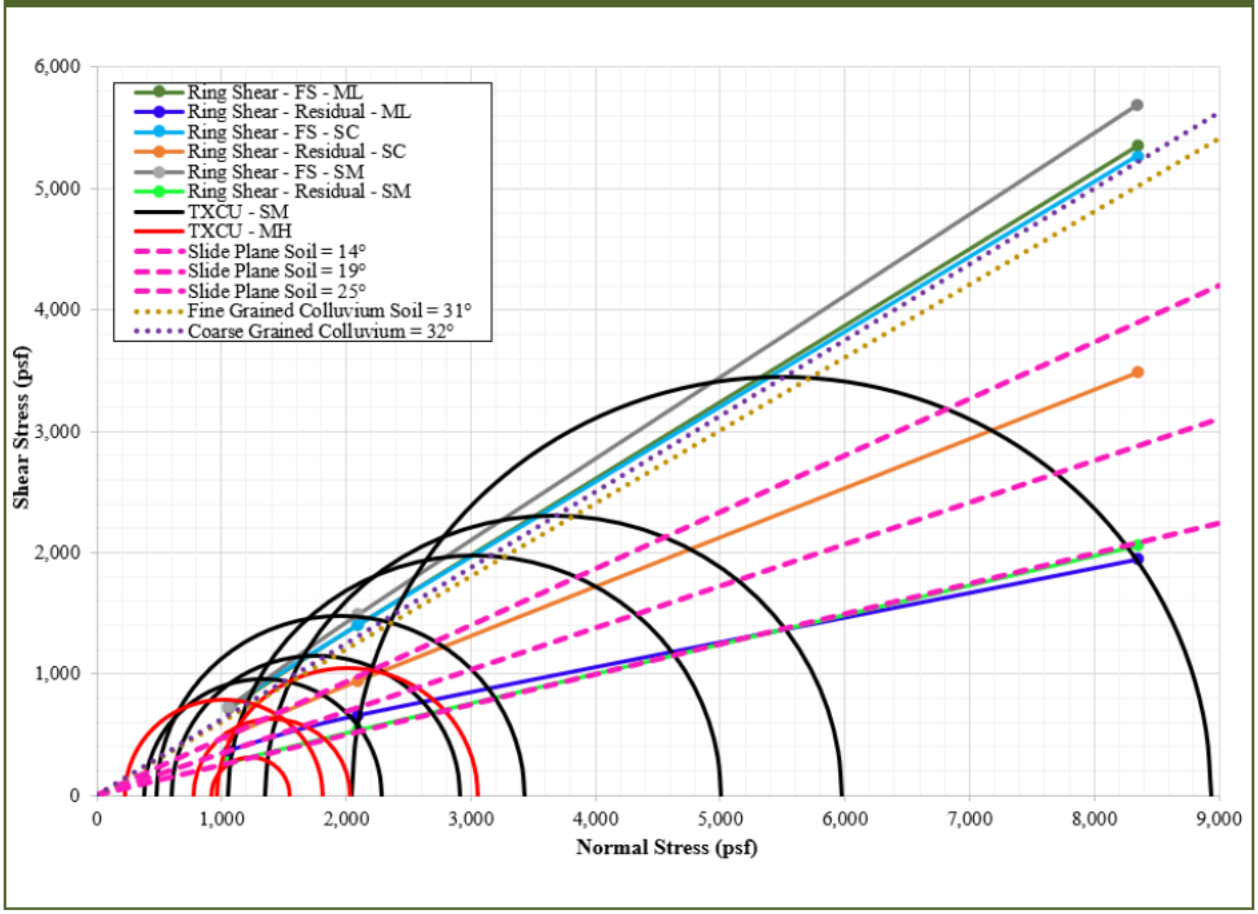
Notes:

The Unified Soil Classification System (USCS) definitions are as follows: SM = Silty Sand; SC = Clayey Sand; ML = Silt; MH = Elastic Silt; GP = Poorly Graded Gravel; GW = Well Graded Gravel

The parameters above were based on statistical distributions of index properties, laboratory tests, published correlations, and engineering judgment.

bpf = blows per foot; ft = feet; N/A = not applicable; pcf = pounds per cubic foot; psf = pounds per square foot

Figure 6
Ring Shear and CUTX Strength Testing Plot with Slide Plane Soil and Colluvium Values



3.4.2 Hydraulic Conductivity

Shannon & Wilson (2017a) describes how hydraulic conductivity parameters are used in the SEEP/W analysis. Hydraulic conductivity values selected for the current study are based on laboratory test results (Shannon & Wilson, 2019) and correlations to soil grain size distribution. Table 2 presents the hydraulic parameters used in the analyses.

Laboratory hydraulic conductivity testing was completed on the same samples used for CUTX testing prior to shearing (Shannon & Wilson, 2019). The hydraulic conductivities from these tests were compared to hydraulic conductivity values estimated from soil grain size and density correlations presented by Odong (2007) and Mesri and others (1994). Using these data, hydraulic conductivities were chosen for fine-grained colluvium and coarse-grained colluvium, respectively, from the distribution of the laboratory test results and hydraulic conductivity correlations. The slide plane soil was assumed to be fine-grained based on soils observed in samples retrieved from the explorations (Shannon & Wilson, 2019).

3.4.3 Groundwater Conditions

Groundwater level information is required to develop groundwater boundary conditions (BC) in SEEP/W. The highest groundwater elevation recorded by vibrating wire piezometers (VWPs) installed in borings LSB-18-305, LSB-18-310, LSB-18-311, and LSB-18-313 were used in the coupled seepage and stability analysis. Plots of groundwater elevation for the 2018 borings are presented in Shannon & Wilson (2018).

To perform finite element SEEP/W seepage analyses, groundwater BCs were imposed on the analyses' domains. These BCs fall into two primary categories:

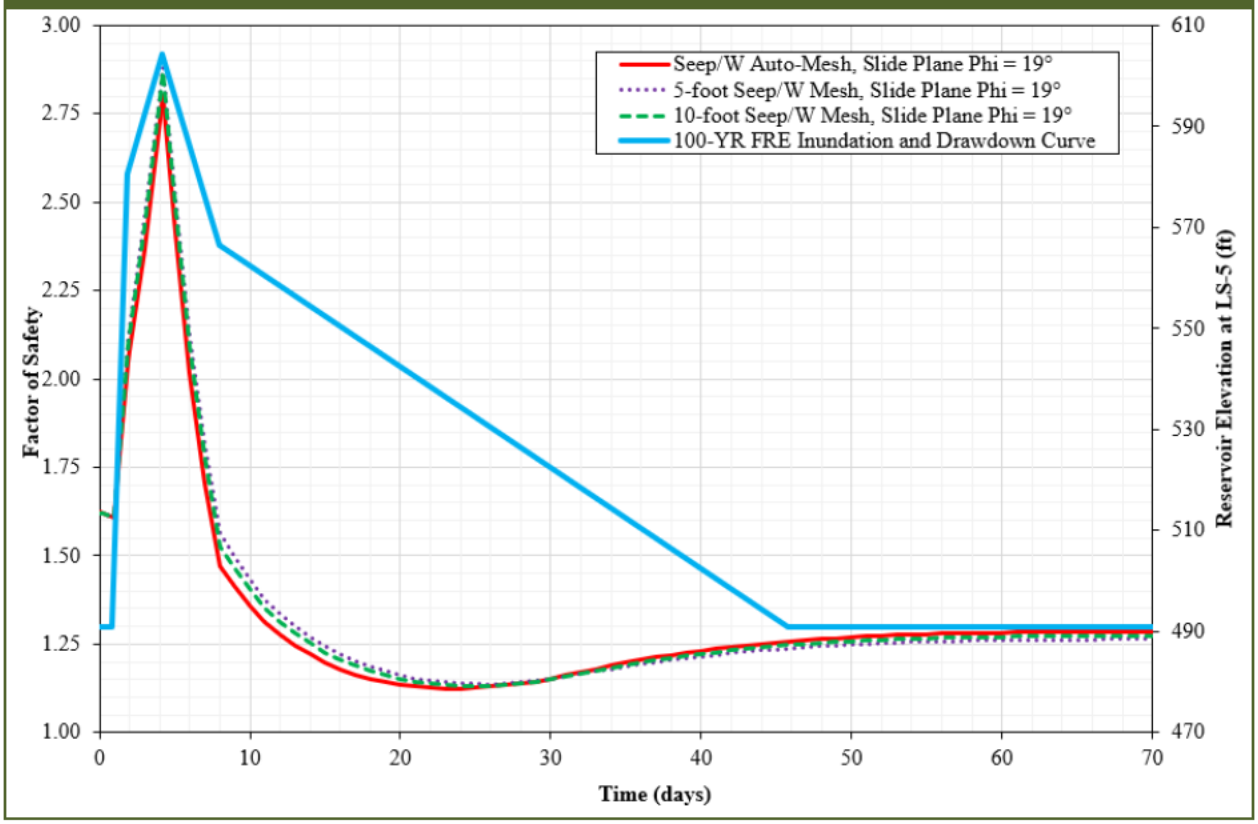
1. **Exterior BC.** These are applied to the left or right edges of the model domain to account for groundwater conditions outside the model. In the analyses, the upslope constant groundwater head BCs were applied to approximately match the observed high groundwater levels recorded by the VWPs. For pre-flood (i.e., no reservoir) conditions, a downslope total head BC equal to the elevation of the river, creek, or channel below the slope toe.
2. **Applied Surface BC.** Surface BC applied included potential seepage face boundaries for areas above the reservoir and a variable water level surface versus time boundary that models reservoir inundation/drawdown used to conduct the transient seepage analyses.

3.4.4 Seepage Analysis Mesh Size

SEEP/W uses a finite element mesh to discretize the model regions into smaller areas to calculate solutions at each finite element and node. The program automatically generates a finite element mesh size inside the regions that have assigned properties. Generally, the program defaults to use a "coarse" mesh size to decrease solving time. This coarse mesh is adequate for many situations (Geo-Slope International, 2018a). Using a finer mesh increases solving time and provides more elements and nodes where solutions are calculated. A sensitivity analysis was performed to evaluate the impact different mesh sizes used for the seepage analyses have on the computed FS against slope instability.

The resulting FS from the stability analysis conducted at each time step of the transient seepage analysis is influenced by the groundwater pressures and resulting effective stress distributions. To evaluate the sensitivity of the FS values to the finite element mesh size, the global mesh size of the LS-5 model was adjusted from the "coarse" default mesh (10 to 20 feet on the longest edge) to 5 feet and 10 feet mesh. FS results from these finer mesh results and the default mesh results are presented in Figure 7.

Figure 7
SEEP/W Seepage Mesh Size Influence on Calculated Factor of Safety for LS-5



Based on this sensitivity analysis for LS-5, the mesh size impacts the calculated FS by up to 6.5%. In general, the finer mesh results in a greater computed FS for a given time step during time steps where reservoir elevation is changing. The largest FS variation occurs during the time steps where the FRE reservoir is changing elevation rapidly from 0 to 8 days. Between approximately 8 days and 26 days, the FS decreases to a minimum value corresponding to the reservoir drawdown at the toe of LS-5 and the FS converge to be less than 1% difference at about 26. The drawdown of the reservoir continues to approximately 46 days, but the colluvium is no longer inundated. After about 26 days, the groundwater pressure in the colluvium gradually decreases over time as groundwater flows out of the soil and groundwater pressures dissipate .

The computed FS at the end of the analyses is up to 1.8% lower for the 5-foot mesh than is computed using the default coarse mesh. The reported FS for each landslide in this and past reports (Shannon & Wilson, 2017a and 2017b) is the minimum computed FS occurring during inundation and drawdown of the reservoir. Where the minimum FSs occur in the models, the FS computed using the coarse mesh is within 1.5% of the FS computed using the 5-foot mesh, corresponding to a difference in FS equal to approximately 0.01 to 0.02 depending on slide plane friction angle. This difference is insignificant for the slope stability analyses method being employed.

Based on this mesh-size sensitivity analyses, stability analyses results computed using the default coarse mesh are sufficiently accurate and use of a finer finite element mesh for seepage analyses is not warranted.

3.5 Global Stability Analysis Details

3.5.1 Factory of Safety (FS) Calculation

Limit equilibrium analyses methods were used for the slope stability analyses. Limit equilibrium global slope stability analyses produce an estimate of the FS for slope stability. The FS is the ratio of forces resisting sliding (i.e., stability) to forces driving sliding (i.e., instability). If the FS is less than 1.0, the driving forces are greater than the resisting forces and the slope is in a state of failure. If the FS is greater than 1.0, the resisting forces are greater than the driving forces and the slope is in a stable state. For reference, FS values ranging from 1.25 to 1.5 are typically targeted for new cut or fill slopes for highway projects or developments. FS values for existing landslides commonly range from less than 1.0 for active landslides to 1.2 for dormant landslides.

Limit equilibrium analyses treat a slide mass as a rigid body, subdivide the mass into slices, and calculate the forces acting on each slice. The Morgenstern-Price limit equilibrium method was used (Morgenstern and Price, 1965). For slopes where transient inundation/drawdown reservoir conditions were considered, the FS for the existing conditions and the minimum FS during reservoir drawdown were computed.

3.5.2 Critical Failure Surfaces

For each stability analysis, the computer program searched for the critical failure surface, i.e., the failure surface with the lowest FS. In addition to randomized searches, failure surface searches were forced to intersect or pass through landslide features and potential failure surfaces identified through geologic reconnaissance of the sites. For retrogressive failure analyses, the slope was divided into three sections, the failure surface was specified to exit near the landslide toe, and analyses conducted using three progressively longer failure surfaces.

4 LANDSLIDE ANALYSIS RESULTS

4.1 LS-5

Landform LS-5 is upstream of the FRE structure, on the left bank of Crim Creek. Landform LS-5 is classified as a landslide deposit that does not show signs of activity or movement. During inundation by the FRE facility 100-year reservoir, water will cover approximately the lower half of LS-5.

In the slope stability model, a relatively weak 5-foot-thick soil layer was assumed to be present below the landslide deposit (see Figure A-1). The weak soil layer begins at an approximate elevation of 680 feet and daylights to Crim Creek at an approximate elevation of 520 feet. To evaluate potential sensitivity of the analyses results to estimated soil shear strength, the analyses were conducted using the three friction angles presented in Table 2 for "slide plane soil."

4.1.1 Current Factor of Safety (FS)

Based on the geologic reconnaissance (Shannon & Wilson, 2015) and subsequent observations, LS-5 does not currently exhibit signs of activity or movement. This suggests the current FS should be equal to or greater than 1.0. For the three friction angles considered for the weak soil layer, 14°, 19°, and 25°, FSs of 1.2, 1.6, and 2.2, respectively, were computed. These results indicate the LS-5 slope is stable under existing conditions.

4.1.2 Flood Retention Expandable (FRE) 100-Year Inundation and Drawdown

For the FRE facility 100-year inundation/drawdown scenario and weak soil layer friction angles of 14°, 19°, and 25°, the analyses results indicate FSs of less than 1.0, 1.1, and 1.5, respectively. Because LS-5 is a landslide deposit with no evidence of past movement or shearing in the weak soil layer, there is a low likelihood that the modeled weak soil layer is in a residual state with a friction angle of 14°. A friction angle of 25° or greater is reasonable for the LS-5 soil above the bedrock.

Therefore, LS-5 will likely remain stable during reservoir inundation and drawdown. Results for selected analyses are contained in Table 3 and Appendix A.

4.2 LS-10

Landform LS-10 is upstream of the FRE structure, on the right side of the Chehalis River valley. LS-10 is classified as a landslide deposit that does not show signs of activity or movement. During inundation by the FRE facility 100-year reservoir, water will cover approximately the lower quarter of LS-10.

In the slope stability model, a relatively weak 5-foot-thick soil layer was assumed to be present below the landslide deposit (see Figure A-2). This weak soil layer begins at an approximate elevation of

700 feet and daylights to the Chehalis River at an approximate elevation of 550 feet. To evaluate potential sensitivity of the analyses results to estimated soil shear strength, the analyses were conducted using the three friction angles presented in Table 2 for "slide plane soil."

4.2.1 Current Factor of Safety (FS)

Based on the geologic reconnaissance (Shannon & Wilson, 2015) and subsequent observations, LS-10 does not currently exhibit signs of activity or movement. This suggests the current FS should be equal to or greater than 1.0. For the three friction angles considered for the weak soil layer, 14°, 19°, and 25°, FSs of 0.8, 1.1, and 1.4, respectively, were computed. Because LS-10 is interpreted as being stable in its current condition, these analyses results indicate the soil in the landslide deposit has a friction angle above 19°.

4.2.2 Flood Retention Expandable (FRE) 100-Year Inundation and Drawdown

For the FRE facility 100-year inundation/drawdown scenario, and weak soil layer friction angles of 14°, 19°, and 25°, the analyses results indicate FSs of less than 1.0, 1.0, and 1.2, respectively. Based on the slope stability analyses results for LS-10 under current conditions, and because LS-10 is a landslide deposit with no evidence of past movement or shearing in the weak soil layer, there is a low likelihood that the modeled weak soil layer has a friction angle less than 19°. A friction angle of 25° or greater is reasonable for the LS-10 soil above the bedrock.

Therefore, LS-10 will likely remain stable during reservoir inundation and drawdown. Results for selected analyses are contained in Table 3 and Appendix A.

Table 3
Selected Global Stability Analysis Results and Figure Index

LANDSLIDE	FAILURE	SLIDE PLANE Φ'	COMMENTS	EXISTING CONDITION FACTOR OF SAFETY (FS)	MINIMUM FACTOR OF SAFETY DURING DRAWDOWN (FS)	APPENDIX A FIGURE NO.
LS-5	Whole Slope	14°	Critical Failure Surface	1.2	0.8	--
LS-5	Whole Slope	19°	Critical Failure Surface	1.6	1.1	A-1
LS-5	Whole Slope	25°	Critical Failure Surface	2.2	1.5	--
LS-10	Whole Slope	14°	Critical Failure Surface	0.8	0.7	--
LS-10	Whole Slope	19°	Critical Failure Surface	1.1	1.0	--
LS-10	Whole Slope	25°	Critical Failure Surface	1.4	1.2	A-2
LS-11	Rotational, El 700	14°	Critical Failure Surface	1.1	1.1	A-3
LS-11	Rotational, El 700	19°	Critical Failure Surface	1.5	1.5	--
LS-11	Rotational, El 700	25°	Critical Failure Surface	1.8	1.8	--
LS-11	Retrogressive	31°	Lower Slope Retrogressive	1.5	1.2	A-4
LS-11	Retrogressive	31°	Mid-Slope Retrogressive	1.5	1.2	--
LS-11	Retrogressive	31°	Upper Slope Retrogressive	1.5	1.3	--
LS-11	Retrogressive	31°	Soil Below El. 605-ft Removed - Mid-Slope Retrogressive	1.1	1.1	--
LS-11	Retrogressive	31°	Soil Below El. 605-ft Removed - Upper Slope Retrogressive	1.6	1.6	--
LS-13	Whole Slope	14°	Critical Failure Surface	0.8	0.8	--
LS-13	Whole Slope	19°	Critical Failure Surface	1.1	1.0	--
LS-13	Whole Slope	25°	Critical Failure Surface	1.5	1.4	A-5
LS-13	Retrogressive	31°	Lower Slope Retrogressive	1.3	1.2	A-6
LS-13	Retrogressive	31°	Mid-Slope Retrogressive	1.6	1.4	--
LS-13	Retrogressive	31°	Upper Slope Retrogressive	1.8	1.7	--
LS-13	Retrogressive	31°	Soil Below El. 605-ft Removed - Mid-Slope Retrogressive	1.1	1.1	--
LS-13	Retrogressive	31°	Soil Below El. 605-ft Removed - Upper Slope Retrogressive	1.5	1.5	--

Notes:

Figures in Appendix A show the minimum factor of safety during drawdown for the given analysis

LS = Landslide

4.3 LS-11

Landform LS-11 is upstream of the FRE structure, on the left side of the Chehalis River valley. The overall LS-11 landform is classified as a landslide deposit. The toe of this deposit is adjacent to the Chehalis River. An active landslide is present within LS-11. This active landslide is near the 1020 Road (Figure 2, Sheet 3). The lower slope, below an approximate elevation of 675 feet, consists of landslide debris mantling bedrock. LS-11 was analyzed for stability of the upper active landslide, effects on the overall LS-11 landform associated with FRE facility 100-year reservoir inundation and drawdown, and potential retrogressive failure. During inundation by the FRE facility 100-year reservoir, water will cover approximately the lower quarter of LS-11.

4.3.1 Active Portion of LS-11

4.3.1.1 Current Factor of Safety (FS)

Stability analyses were conducted on the active landslide that is within the greater LS-11 mass, near the 1020 Road elevation. In the slope stability model, a relatively weak soil layer was assumed to be present along the base of this active landslide (see Figure A-3). This soil was modeled as a 5-foot-thick weak soil layer above the bedrock. This layer begins at an approximate elevation of 745 feet and daylights to the slope face at an elevation of 675 feet. To evaluate potential sensitivity of the analyses results to soil shear strength, stability analyses were conducted for failure surfaces that followed this thin soil layer using the three friction angles presented in Table 2 for "slide plane soil." Below elevation 675 feet, higher-strength fine-grained colluvium was assumed to overlay the bedrock.

For the three friction angles considered for the weak soil layer, 14°, 19°, and 25°, FS greater than 1.0 were computed. This portion of LS-11 is an active landslide, suggesting the FS should be equal to or below 1.0, which would indicate marginal stability to instability. A possible explanation for these analyses results showing the FS to be greater than 1.0 is that the groundwater levels observed at LS-11 and used for the analyses may not be representative of groundwater levels present when LS-11 is moving. Groundwater level measurements were made from August to November 2018. Higher groundwater levels may occur during wetter periods.

4.3.1.2 Flood Retention Expandable (FRE) 100-Year Inundation and Drawdown

The failure surface for the active landslide near the 1020 Road is above elevation 675, about 70 feet above the FRE facility 100-year reservoir surface elevation of 604.4 feet. The seepage analyses results show groundwater levels in the active landslide are not impacted by the FRE facility reservoir. Therefore, reservoir impoundment would not change stability of this active landslide.

This active landslide within the greater LS-11 mass has relatively little volume, does not appear to move rapidly, and daylights to the slope about 70 feet above the FRE facility 100-year reservoir surface elevation. Continued movement is unlikely to result in rapid delivery of a significant volume of material to the intermittently present FRE facility reservoir. Therefore, continued instability and movement of

this portion of the LS-11 mass is unlikely to impact the FRE facility or its operation. Implementing stability improvement measures is likely not warranted.

Results for selected analyses are contained in Table 3 and Appendix A.

4.3.2 Lower Slope of LS-11

4.3.2.1 Current Factor of Safety (FS)

Based on the geologic reconnaissance (Shannon & Wilson, 2015) and subsequent observations, the lower slope of LS-11, below about elevation 675 feet, does not currently exhibit signs of activity or movement. This suggests the current FS for the lower slope of LS-11 should be equal to or greater than 1.0. For an assumed friction angle of 31° for colluvium above the bedrock, a FS of 1.5 was computed for current conditions and a potential failure surface extending from the middle third of the slope to the slope toe.

4.3.2.2 Flood Retention Expandable (FRE) 100-Year Inundation and Drawdown

For the FRE facility 100-year inundation/drawdown scenario, and colluvium friction angle of 31°, the analyses results indicate a FS of 1.2 for a potential failure surface extending from the slope toe to the middle third of the slope (see Figure A-4). This result suggests the lower slope of LS-11 will likely remain stable during reservoir inundation and drawdown.

The lower slope of LS-11 was further analyzed for potential retrogressive failure by evaluating lower slope stability and progressively longer potential slip surfaces. Retrogression could be initiated by toe of slope movement caused by reservoir inundation/drawdown or toe erosion. The analyses results show the FS increases with increasing failure surface length, which indicates LS-11 may be susceptible to retrogressive failure should the lower portion of the slope move or be eroded. Because the calculated FSs are above 1.0 for each failure surface considered, and the FS for a failure in the lower third of the slope is about 1.2 under inundation/drawdown conditions, the likelihood of retrogressive failure of LS-11 is low.

LS-11 was also analyzed assuming soil below elevation 605 feet had moved downslope or been eroded, and no longer supported soil above elevation 605 feet (maximum reservoir elevation = 604.4). For this condition, a FS of 1.1 is computed for LS-11 soil above elevation 605. This result indicates retrogression, should it occur, may stop near the maximum reservoir surface elevation.

Results for selected analyses are contained in Table 3 and Appendix A.

4.4 LS-13

Landform LS-13 is upstream of the FRE structure, on the left side of the Chehalis River valley. LS-13 is classified as a landslide deposit. No signs of recent activity or movement were observed during reconnaissance of LS-13. During inundation by the FRE facility 100-year reservoir, water will cover approximately the lower 35 vertical feet of LS-13.

4.4.1 Overall Slope Stability

4.4.1.1 Current Factor of Safety (FS)

To evaluate overall slope stability, in the slope stability model a relatively weak soil layer was assumed to be present below the landslide deposit (see Figure A-5). This soil was modeled as a 5-foot-thick layer over the bedrock that begins at an approximate elevation of 770 feet and daylights at the Chehalis River at an approximate elevation of 575 feet.

To evaluate potential sensitivity of the analyses results to estimated soil shear strength, analyses were conducted using the three friction angles presented in Table 2 for "slide plane soil." For the three friction angles considered for the weak soil layer, 14°, 19°, and 25°, a FS of less than 1.0, 1.1, and 1.4, respectively, was computed for a failure surface extending from the toe to the head of the landslide (whole slope). Because LS-13 is interpreted as being stable in its current condition, these analyses results indicate the soil in the landslide deposit has a friction angle above 19°.

4.4.1.2 Flood Retention Expandable (FRE) 100-Year Inundation and Drawdown

Because the FRE facility reservoir only rises approximately 35 feet vertically above the LS-13 slope toe, the FRE facility 100-year inundation and drawdown has little effect on slope FS, with inundation/drawdown reducing the FS by less than 0.1 from the current FS. For weak soil layer friction angle of 19° or above, LS-13 would remain stable during inundation and drawdown.

Results for selected analyses are contained in Table 3 and Appendix A.

4.4.2 Lower Slope of LS-13

4.4.2.1 Current Factor of Safety (FS)

Based on the geologic reconnaissance (Shannon & Wilson, 2015) and subsequent observations, the lower slope of LS-13, below about elevation 700 feet, does not currently exhibit signs of activity or movement. This suggests the current FS for the lower slope of LS-13 should be equal to or greater than 1.0. For these analyses, colluvium was assumed to be present above the bedrock. For an assumed friction angle of 31° for the colluvium, a FS of 1.3 was computed for current conditions and a potential failure surface extending from the middle third of the slope to the slope toe.

4.4.3 Flood Retention Expandable (FRE) 100-Year Inundation and Drawdown

LS-13 was analyzed for potential retrogressive failure by evaluating lower slope stability and progressively longer potential slip surfaces on the LS-13 slope (see Figure A-6). Retrogression could be initiated by toe of slope movement caused by reservoir inundation/drawdown or toe erosion. The analyses results show the FS increases with increasing failure surface length, which indicates LS-13 may be susceptible to retrogressive failure should the lower portion of the slope move or be eroded. Because the calculated FSs are above 1.0 for each failure surface considered, and the FS for a failure in the lower

third of the slope for inundation/drawdown conditions is about 1.2, the likelihood of retrogressive failure of LS-13 is low.

LS-13 was also analyzed assuming soil below elevation 605 feet had moved downslope or been eroded and no longer supported soil above elevation 605 feet (maximum reservoir elevation = 604.4). For this condition, a FS of 1.1 was computed for LS-13 soil above elevation 605. This result indicates retrogression, should it occur, may stop near the maximum reservoir surface elevation.

Results for selected analyses are contained in Table 3 and Appendix A.

4.5 LS-18, LS-19, and LS-26

Stability analyses were not conducted for landforms/landslides LS-18, LS-19, and LS-26.

- LS-18 and LS-19 are above the elevation that would be inundated by the design FRE facility 100-year storm reservoir. Therefore, stability of LS-18 and LS-19 would not be impacted by the reservoir.
- LS-26 was not analyzed because it is downstream of the FRE structure and will be excavated or stabilized during FRE facility construction. LS-26 stability will be considered in future design phases of the Project.

4.6 Landslide Stability Under Seismic Loading

Landslides 3 and 4 (LS-3 and LS-4) are near the FRE facility. Excessive movement of these landslides could directly impact the FRE facility operation. Prior analyses found these landslides in their existing condition do not meet static stability criteria for the FRE facility for rapid drawdown conditions (Shannon & Wilson, 2017a). Shannon & Wilson (2017b) presents concepts to improve stability of LS-3 and LS-4 by installing drainage and toe buttresses.

For the current study, LS-3 and LS-4 were evaluated for seismic stability for the assumed post-FRE facility construction configuration, wherein drainage and toe buttresses were assumed to be constructed (see Shannon & Wilson, 2017b). Pseudo-static stability evaluation was performed to evaluate the stability of these landslides for a seismic event. Yield accelerations for the slope for the three frictional strengths of colluvium and buttress materials found in Shannon & Wilson (2017b) were calculated and are presented in Table 4. The yield acceleration is the seismic-event-induced acceleration that would reduce the FS of the landslide to 1.0. The analyses assumed no reservoir during the seismic event because simultaneous occurrence of a seismic event and reservoir impoundment due to a storm are deemed unlikely.

The yield accelerations calculated from the stability analyses can be related to earthquake return periods or probability of exceedance in a time interval. A probabilistic seismic hazard analysis and time histories were completed for the proposed FRE facility and on-site facilities (Shannon & Wilson, 2015b). The horizontal mean uniform hazard spectra were concurrently developed for the FRE facility location.

The hazard spectra were developed for bedrock below the facility location not accounting for the overburden or landslide deposits found elsewhere in Chehalis River Basin.

Table 4
Summary of Yield Accelerations For LS-3 and LS-4 Stability Improvements

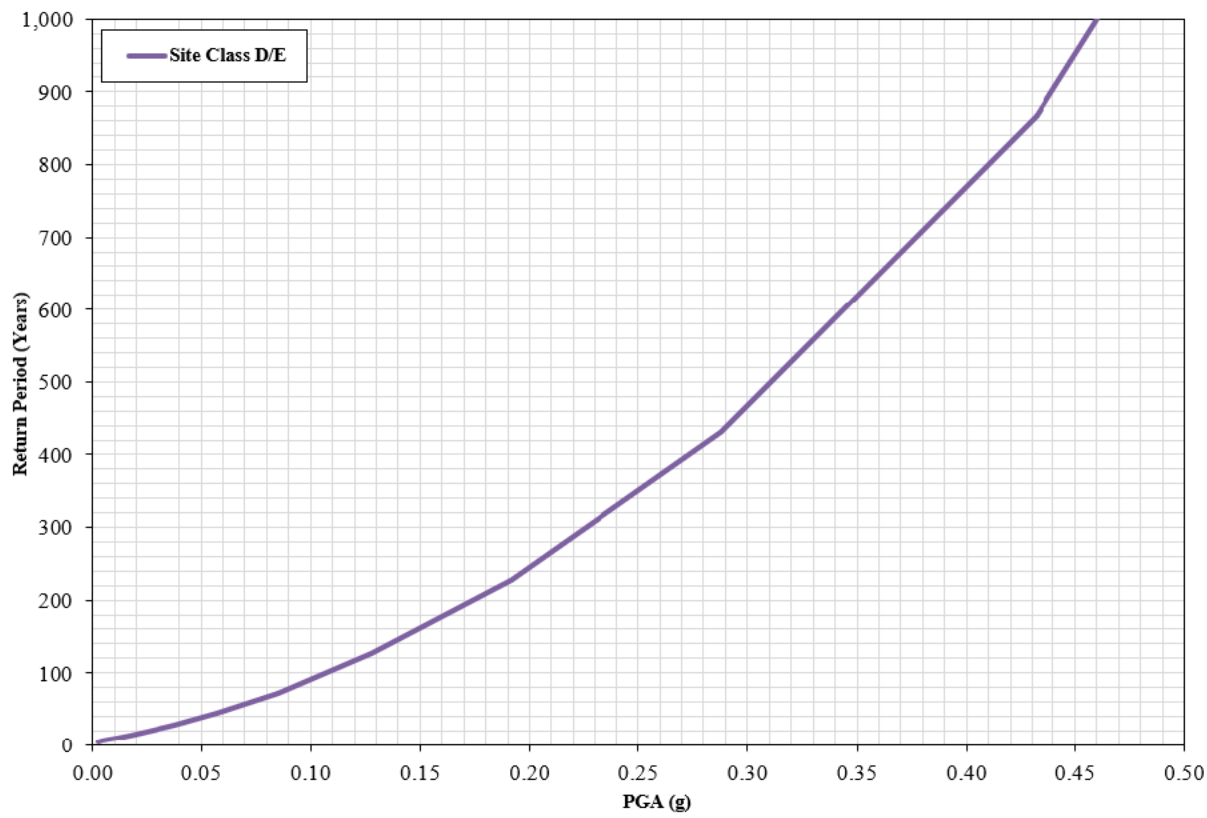
LANDSLIDE AREA	CROSS SECTION	ANALYSIS NAME	YIELD ACCELERATION (g)	E4STIMATED RETURN PERIOD (yr)	HORIZONTAL ACCELERATION(g)	ESTIMATED RETURN PERIOD (yr)
LS-3	MF-1	Lower Search, 4H:1V Φ = 25° Buttress, Drainage, Hc Φ = 25°	0.09	75	0.18	200
LS-3	MF-1	Lower Search, 4H:1V Φ = 25° Buttress, Drainage, Hc Φ = 30°	0.12	100	0.24	325
LS-3	MF-1	Lower Search, 4H:1V Φ = 25° Buttress, Drainage, Hc Φ = 35°	0.14	125	0.28	400
LS-3	MF-1	Lower Search, 4H:1V Φ = 30° Buttress, Drainage, Hc Φ = 25°	0.09	75	0.18	200
LS-3	MF-1	Lower Search, 4H:1V Φ = 30° Buttress, Drainage, Hc Φ = 30°	0.13	125	0.26	350
LS-3	MF-1	Lower Search, 4H:1V Φ = 30° Buttress, Drainage, Hc Φ = 35°	0.17	175	0.34	575
LS-3	MF-1	Lower Search, 4H:1V Φ = 35° Buttress, Drainage, Hc Φ = 25°	0.09	75	0.18	200
LS-3	MF-1	Lower Search, 4H:1V Φ = 35° Buttress, Drainage, Hc Φ = 30°	0.14	125	0.28	400
LS-3	MF-1	Lower Search, 4H:1V Φ = 35° Buttress, Drainage, Hc Φ = 35°	0.19	225	0.38	700
LS-4	2	Lower Search, 4H:1V Φ = 25° Buttress, Drainage, Hc Φ = 25°	0.08	50	0.16	150
LS-4	2	Lower Search, 4H:1V Φ = 25° Buttress, Drainage, Hc Φ = 30°	0.13	125	0.26	350
LS-4	2	Lower Search, 4H:1V Φ = 25° Buttress, Drainage, Hc Φ = 35°	0.17	175	0.34	575
LS-4	2	Lower Search, 4H:1V Φ = 30° Buttress, Drainage, Hc Φ = 25°	0.09	75	0.18	200
LS-4	2	Lower Search, 4H:1V Φ = 30° Buttress, Drainage, Hc Φ = 30°	0.14	125	0.28	400
LS-4	2	Lower Search, 4H:1V Φ = 30° Buttress, Drainage, Hc Φ = 35°	0.19	225	0.38	700
LS-4	2	Lower Search, 4H:1V Φ = 35° Buttress, Drainage, Hc Φ = 25°	0.10	75	0.20	225
LS-4	2	Lower Search, 4H:1V Φ = 35° Buttress, Drainage, Hc Φ = 30°	0.16	150	0.32	525
LS-4	2	Lower Search, 4H:1V Φ = 35° Buttress, Drainage, Hc Φ = 35°	0.21	250	0.42	850

Notes:

Analyses were conducted for steady-state seepage with a river elevation of 445 feet

The thickness of soil deposits that make up LS-3 and LS-4 vary. The total thickness and variation in thickness of the landslide deposits influence the ground response from seismic events. These deposits will amplify the bedrock ground motion, with the magnitude of amplification dependent broadly on peak ground acceleration (PGA), soil density, soil shear strength, and index properties. Guidance for the amplification factors can be found in AASHTO load and resistance factor design (LRFD) 8 (AASHTO, 2017) and the WSDOT Geotechnical Design Manual (WSDOT, 2015). Amplification factors, or site class coefficients, were calculated for borings LSB-3 and SW-17-LS4 (Shannon & Wilson, 2016) using guidance from AASHTO LRFD 8. Both sites are classified as Site Class "E.". Figure 8 displays the relationship between PGA and return periods for Site Class D/E. The U. S. Geological Survey (Highland and Johnson, 2004) provides data for estimated PGA at the Site Class D/E boundary but does not provide data for Site Class E conditions. The Site Class D/E boundary is representative of the upper-bound return periods for Site Class E.

Figure 8
Return Periods Corresponding to Peak Ground Accelerations For Site Class D/E



For landslides where the toe of the potentially unstable slope is at a structure or if slope movement cannot be tolerated (WSDOT, 2015), the PGA should be used as the horizontal seismic coefficient for slope stability analysis. For slopes allowed to displace 1 to 2 inches or more during an earthquake, the

horizontal ground acceleration is reduced by a factor of 2 in slope stability analyses (WSDOT, 2015). Because displacement of LS-3 and LS-4 by 2 inches or more during a seismic event would be acceptable, and not pose a hazard to the FRE facility, a factor of 2 was applied to the computed yield accelerations presented in Table 4 to compute the estimated return periods for seismic events that could cause the computed yield accelerations. These return periods are estimated to be between 150 and 800 years.

For these return periods, LS-3 and LS-4 would not be considered to pose an unacceptable hazard to the FRE facility or structure after implementation of stability improvement measures, such as the drainage and toe buttress concept presented in Shannon & Wilson (2017a).

Landforms other than LS-3 and LS-4 within the reservoir limits evaluated for this study and in previous studies (Shannon & Wilson, 2017b) were not evaluated for seismic-event-induced instability. It was assumed that a reservoir would not be present during the seismic event because simultaneous occurrence of a seismic event and reservoir impoundment due to a storm are deemed unlikely. Seismic-event-induced instability of other landforms/landslides upstream of the FRE structure when a reservoir is not present would not pose an immediate risk to the FRE facility because when a reservoir is not impounded, there would be no water with which to create a tsunami. Seismic-event-induced instability and excessive movement of landslides and slopes could result in a landslide dam across the Chehalis main stem or one of its tributaries upstream of the FRE structure. These instabilities could occur for landforms and landslides evaluated for the current study and Shannon & Wilson (2017b) or other slopes within the watershed. The FRE facility condition should be checked after a seismic event. Actions to protect the FRE facility should be taken if landslide dams occur upstream of the FRE structure.

5 CONCLUSIONS

This report discusses stability considerations or stability analyses results for seven landforms mapped within the Chehalis basin that have the potential to impact the proposed FRE facility, its operation, or the reservoir impounded by the FRE facility.

LS-18 and LS-19 are above the elevation that could be inundated by the design 100-year (return period) storm reservoir impounded behind the FRE structure. Therefore, stability of LS-18 and LS-19 would not be impacted by the reservoir.

LS-26 was not analyzed because it is downstream of the FRE structure and will be excavated or stabilized during FRE facility construction. LS-26 stability will be considered in future design phases of the Project.

The slope stability analyses results for LS-5, LS-10, LS-11 (overall landform), and LS-13 indicate these landslides/landforms will likely remain stable during FRE facility 100-year storm reservoir inundation and drawdown. The analyses results also indicate that while the lower slopes of LS-11 and LS-13 will likely have a FS greater than 1.1 during reservoir inundation/drawdown, they may be susceptible to retrogressive failure should lower slope movement occur. Based on the geologic interpretation and analyses results, should movement of these four landforms occur, the movement is unlikely to be rapid or impact the FRE facility or its operation.

Slope stability analyses were performed for the active landslide near the top of LS-11. This active landslide is more than 70 feet above the FRE facility 100-year storm reservoir surface elevation of 604.4 feet. This unstable zone impacts the 1020 Road. Continued movement of this active portion of LS-11 is unlikely to impact the FRE facility or its operation.

6 FUTURE STUDIES

The global slope stability analyses are for conceptual engineering purposes only and based on limited subsurface information. Limited laboratory strength testing was completed for the Phase 3 landslide evaluation. Depending on the Project owner's tolerance of uncertainty and risk associated with potential landform/landslide movement, additional explorations, strength testing, in situ hydrologic testing, and analyses may be required for final design.

7 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they existed at the time of the geologic reconnaissance, and further assume that the explorations are representative of the subsurface conditions at the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations and the visual observations. If there is a substantial lapse of time between the submission of this report, or if conditions have changed, the report and analyses performed should be reviewed to determine the applicability of the conclusions and recommendations.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. No other warranty, either express or implied, is made. These conclusions and recommendations were based on the understanding of the Project as described in this report and the observed site conditions.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples from test borings.

This report was prepared for the exclusive use of Anchor QEA, LLC; the Office of Chehalis Basin; and other members of the Project team. The report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions.

The scope of services did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on, below, or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Shannon & Wilson has prepared Appendix C, “Important Information About Your Geotechnical/ Environmental Report,” to assist you and others in understanding the use and limitations of this report.

8 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 2015, AASHTO LRFD bridge design specifications: U.S. customary units (7th ed. with 2015 interim revisions): Washington, D. C., AASHTO, 2 v.

American Association of State Highway and Transportation Officials (AASHTO), 2017, AASHTO LRFD bridge design specifications (8th ed.): Washington, D. C., AASHTO, 2 v

Anchor QEA, 2017, Operations Plan for Flood Retention Facilities, Report prepared by Anchor QEA, LLC., Seattle, WA, for State of Washington Office of Financial Management and Chehalis Basin Workgroup, June.

ASTM International, 2017, Standard practice for classification of soils for engineering purposes (unified soil classification system), D2487-17: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 10 p., available: www.astm.org.

Geo-Slope International, 2018a, SEEP/W v 8.0: Calgary, Alberta, Geo-Slope International.

Geo-Slope International, 2018b, SLOPE/W v 8.0: Calgary, Alberta, Geo-Slope International.

Geotechnical Design Manual (GDM), M 46-03.11, 2015, Prepared by the Washington State Department of Transportation, Environmental and Regions Operations, Construction Office, Geotechnical Office. Tumwater, Washington, May 2015.

HDR and Shannon & Wilson, 2015, Phase 1 Site Characterization. Technical Memorandum prepared by HDR Engineering, Inc., Seattle, WA, and Shannon & Wilson, Inc., Seattle, WA, for State of Washington Office of Financial Management and Chehalis Basin Workgroup, September 25.

Mesri, Gholamreza; Lo, D. O. K.; and Feng, T. W., 1994, Settlement of embankments on soft clays, in Yeung, A. T. and Felio, G. Y., eds., Vertical and Horizontal Deformations of Foundations and Embankments, Settlement '94, College Station, Texas, 1994, Proceedings: N. Y., American Society of Civil Engineers, v. 1, p. 8-56.

Morgenstern, N.R. and Price, V.E., 1965, The analysis of the stability of general slip surfaces: Geotechnique, v. 15. no. 1, p. 79-93.

Odong, Justine, 2007, Evaluation of empirical formulae for determination of hydraulic conductivity based on grain-size analysis: The Journal of American Science, v. 3, no. 3, p. 54-60, available: http://www.jofamericanscience.org/journals/am-sci/0303/.

Peck, R. B.; Hanson, W. E.; and Thornburn, T. H., 1974, Foundation engineering (2nd ed.): N. Y., Wiley, 514 p.

Shannon & Wilson, Inc., 2014, Preliminary Desktop Landslide Evaluation. Report prepared by Shannon & Wilson, Inc., Seattle, WA for Chehalis Basin Workgroup, July 23, 2014.

Shannon & Wilson, Inc., 2015, Landslide Reconnaissance Evaluation of the Chehalis Dam Reservoir. Report prepared by Shannon & Wilson, Inc., Seattle, WA, for Chehalis Basin Workgroup, September 22.

Shannon & Wilson, Inc., 2017a, Technical Memorandum Phase 2 Chehalis Dam Landslide Evaluation Chehalis Dam Basin Workgroup, Pe Ell, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-21897-021, for Chehalis Basin Workgroup, June 29.

Shannon & Wilson, Inc., 2017b, Technical Memorandum Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species Project Phase 2 Chehalis Dam Site Characterization Landslide Stability Improvement Evaluation, Pe Ell, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-21897-021, for Chehalis Basin Workgroup, June 28.

Shannon & Wilson, Inc., 2019, Phase 3 Chehalis Dam Geotechnical Data Report. Report prepared by Shannon & Wilson, Inc., Seattle, Wash., for the Office of the Chehalis Basin, March 5.

U. S. Naval Facilities Engineering Command, 1986, Soil mechanics: Alexandria, Va., U. S. Naval Facilities Engineering Command, NAVFAC DM-7.01, 1 v., September.

U.S. Geological Survey, 2004, Landslide Types and Processes Fact Sheet: Available: https://pubs.usgs.gov/fs/2004/3072/pdf/fs2004-3072.pdf, accessed April 2019.

Highland, Lynn and Johnson, Margo, 2004, Landslide types and processes: U.S. Geological Survey Fact Sheet 2004-3072, 4 p., available: https://pubs.usgs.gov/fs/2004/3072/.

Washington State Dept. of Transportation (WSDOT), 2015, Geotechnical design manual: Olympia, Wash., WSDOT, Manual M 46-03, 1 v., May, available: http://www.wsdot.wa.gov/Publications/Manuals/M46-03.htm.

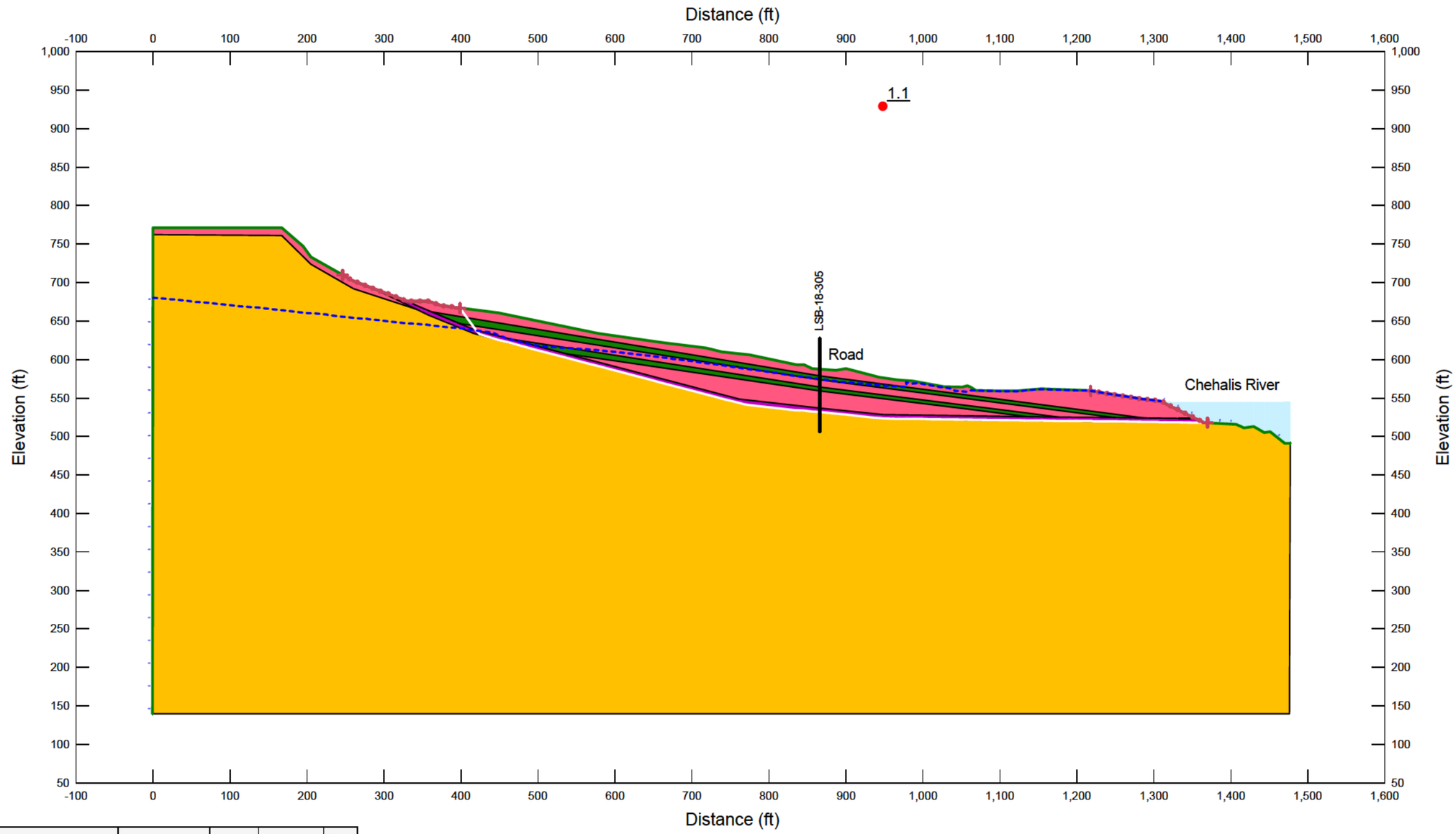
Xuecheng Gao, Hanlong Liu, Wengang Zhang, Wei Wang & Zhenyu Wang (2019) Influences of reservoir water level drawdown on slope stability and reliability analysis, Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards, 13:2, 145-153, DOI: 10.1080/17499518.2018.1516293

Appendix A
Slope Stability Analysis Results

SLOPE STABILITY ANALYSIS RESULTS

Table of Contents

Figures



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bed Rock	Bedrock (Impenetrable)			
	Coarse-Grained Colluvium	Mohr-Coulomb	120	0	32
	Fine-grained Colluvium	Mohr-Coulomb	115	0	31
	Slide Plane Soil (Φ = 19°)	Mohr-Coulomb	115	0	19

Chehalis Basin Strategy
Phase 3 Landslide Evaluation
Pe Ell, Washington

STABILITY ANALYSIS

LS-05 - Slide Plane (Φ = 19°)

May 2019 21-1-21897-025

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FIG. A-1

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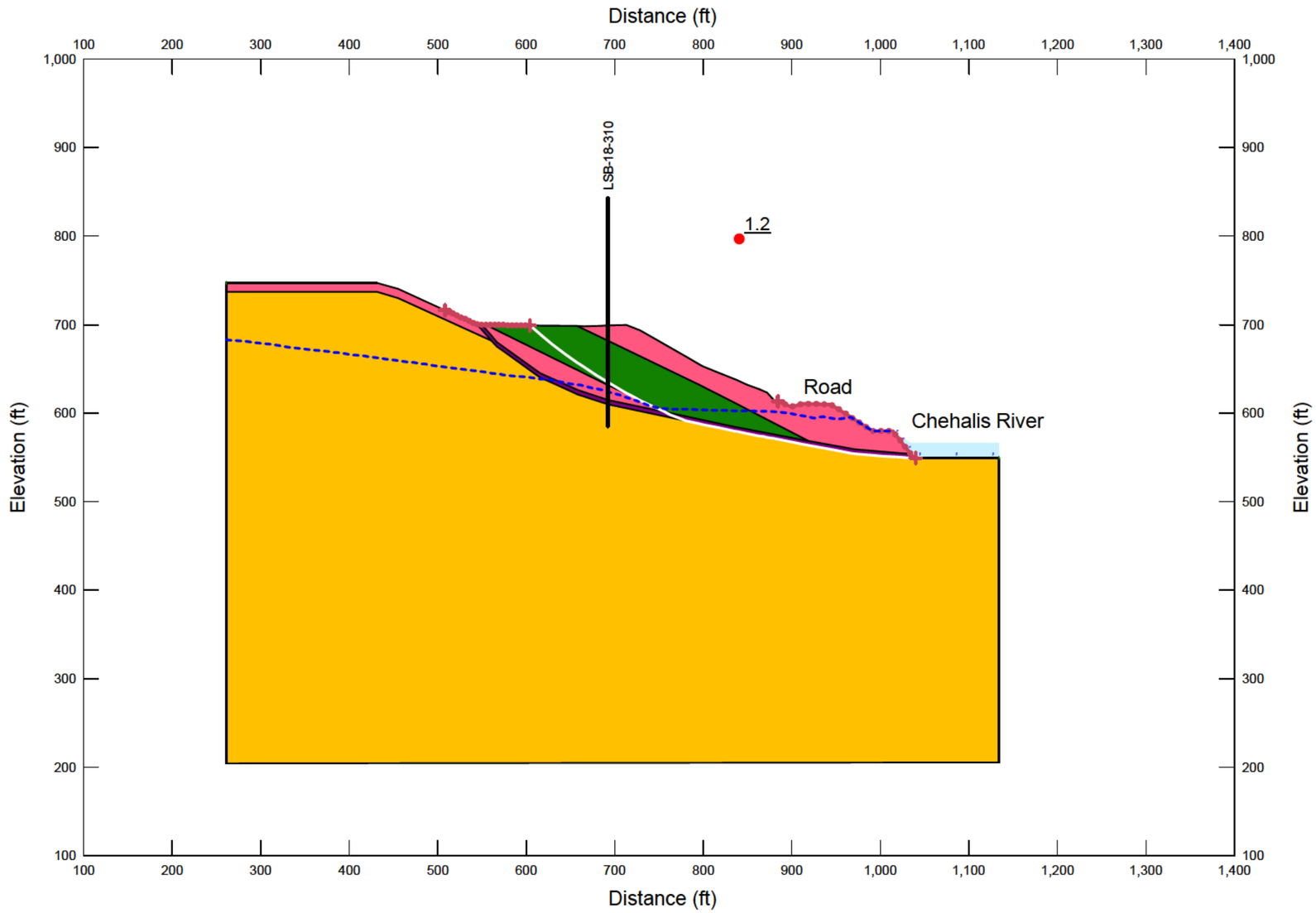

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bed Rock	Bedrock (Impenetrable)			
	Coarse-Grained Colluvium	Mohr-Coulomb	120	0	32
	Fine-grained Colluvium	Mohr-Coulomb	115	0	31
	Slide Plane Soil (Φ = 25°)	Mohr-Coulomb	115	0	25

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STABILITY ANALYSIS

LS-10 - Slide Plane (Φ = 25°)

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FIG. A-2

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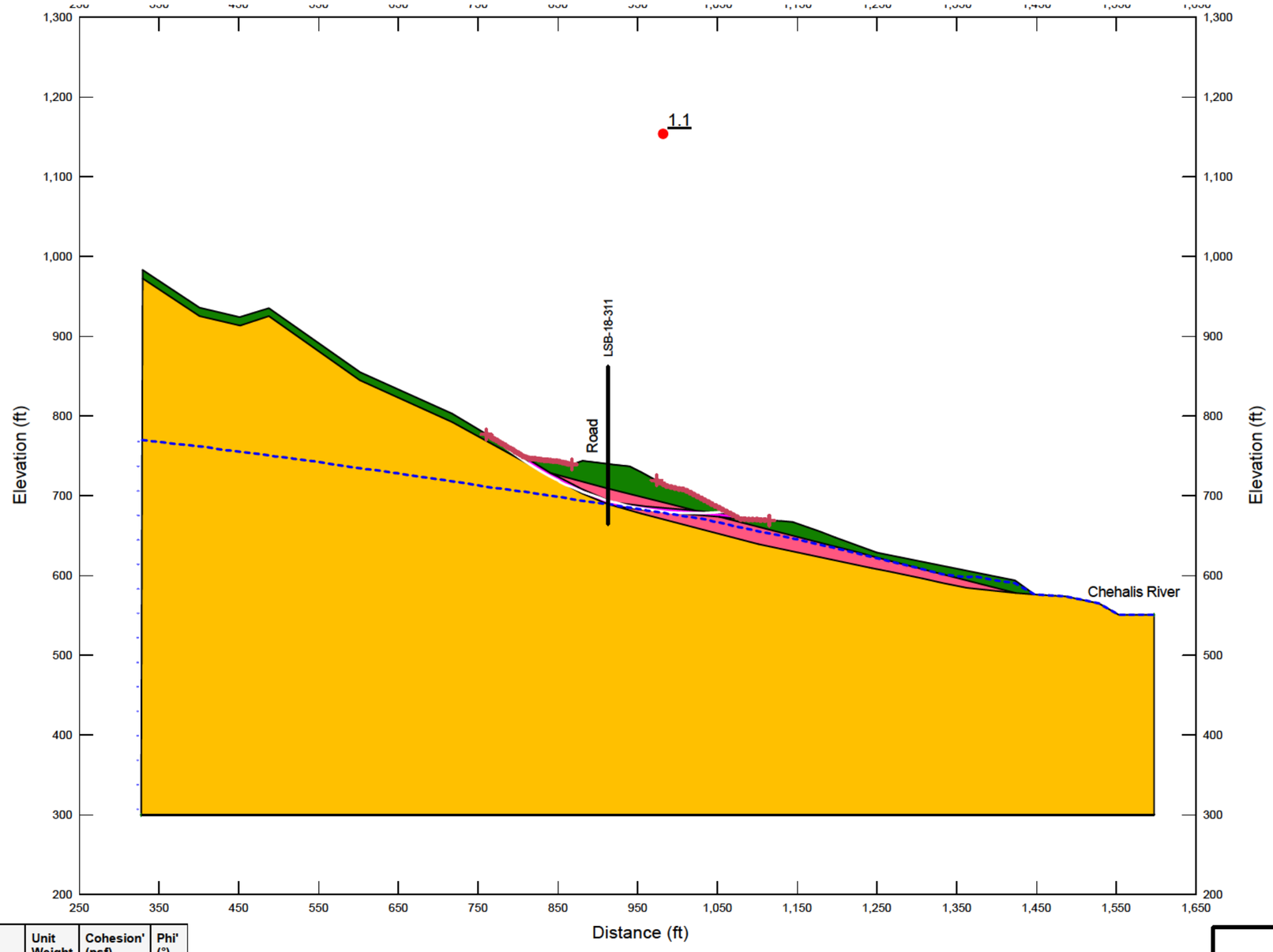

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bed Rock	Bedrock (Impenetrable)			
	Coarse-Grained Colluvium	Mohr-Coulomb	120	0	32
	Fine-grained Colluvium	Mohr-Coulomb	115	0	31
	Slide Plane Soil (Φ = 14°)	Mohr-Coulomb	115	0	14

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STABILITY ANALYSIS

LS-11 - Slide Plane (Φ = 14°)

January 2018 21-1-21897-025

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FIG. A-3

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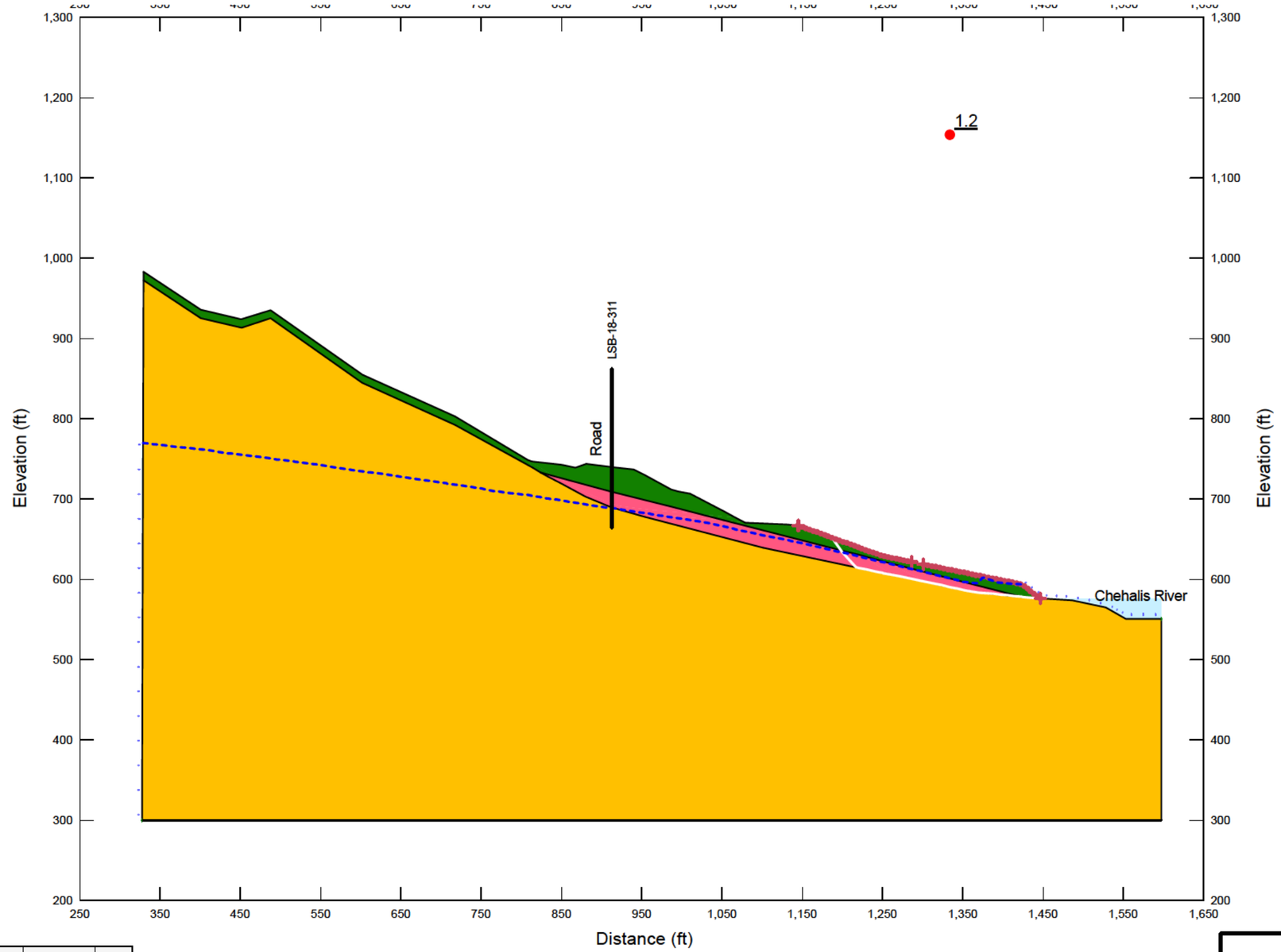

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bed Rock	Bedrock (Impenetrable)			
	Coarse-Grained Colluvium	Mohr-Coulomb	120	0	32
	Fine-grained Colluvium	Mohr-Coulomb	115	0	31

Chehalis Basin Strategy
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STABILITY ANALYSIS

LS-11 - Retrogressive - Lower

January 2018 21-1-21897-025

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FIG. A-4

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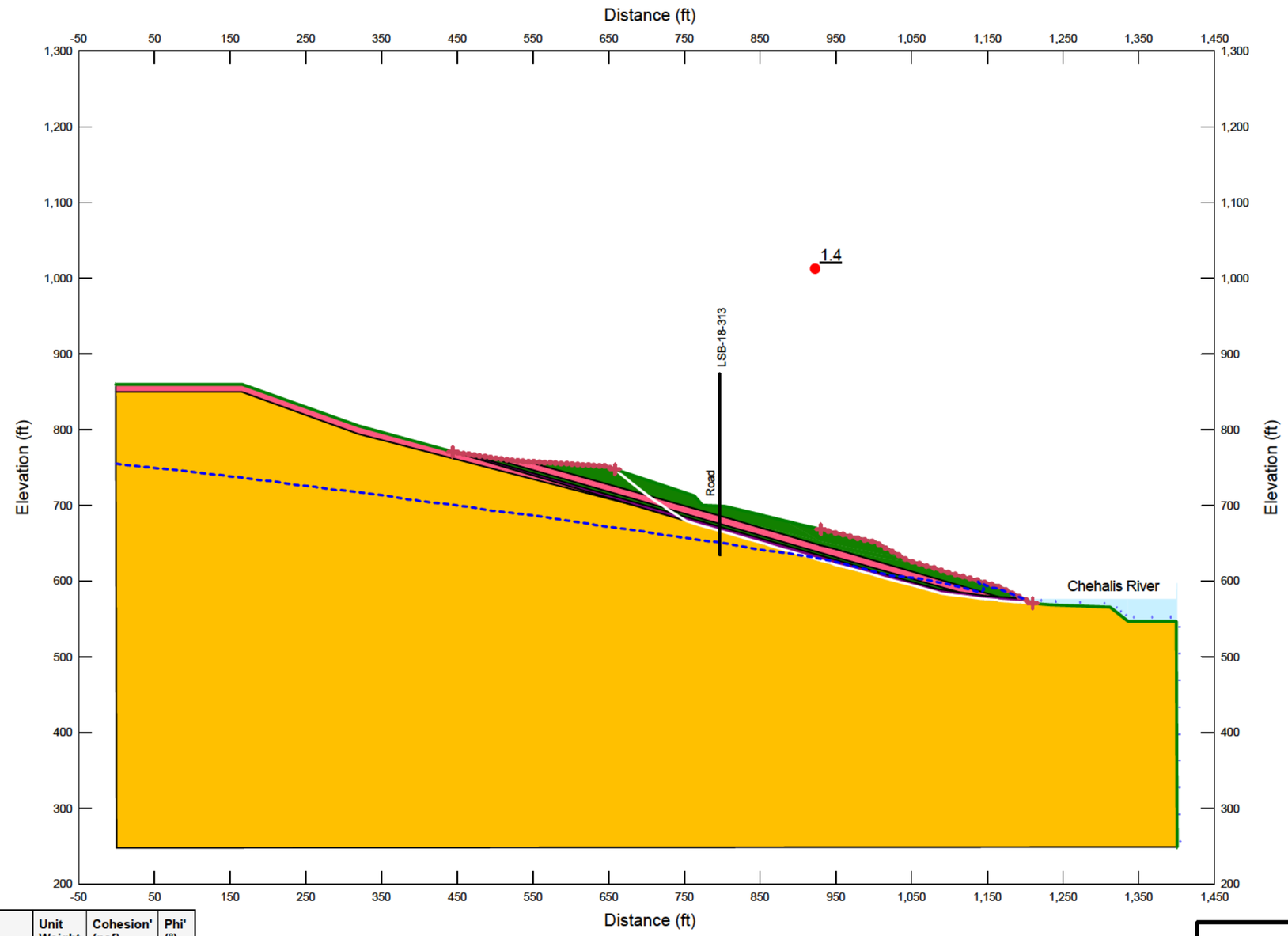

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bed Rock	Bedrock (Impenetrable)			
	Coarse-Grained Colluvium	Mohr-Coulomb	120	0	32
	Fine-grained Colluvium	Mohr-Coulomb	115	0	31
	Slide Plane Soil (Φ = 25°)	Mohr-Coulomb	115	0	25

Chehalis Basin Strategy
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STABILITY ANALYSIS

LS-13 - Slide Plane (Φ = 25°)

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FIG. A-5

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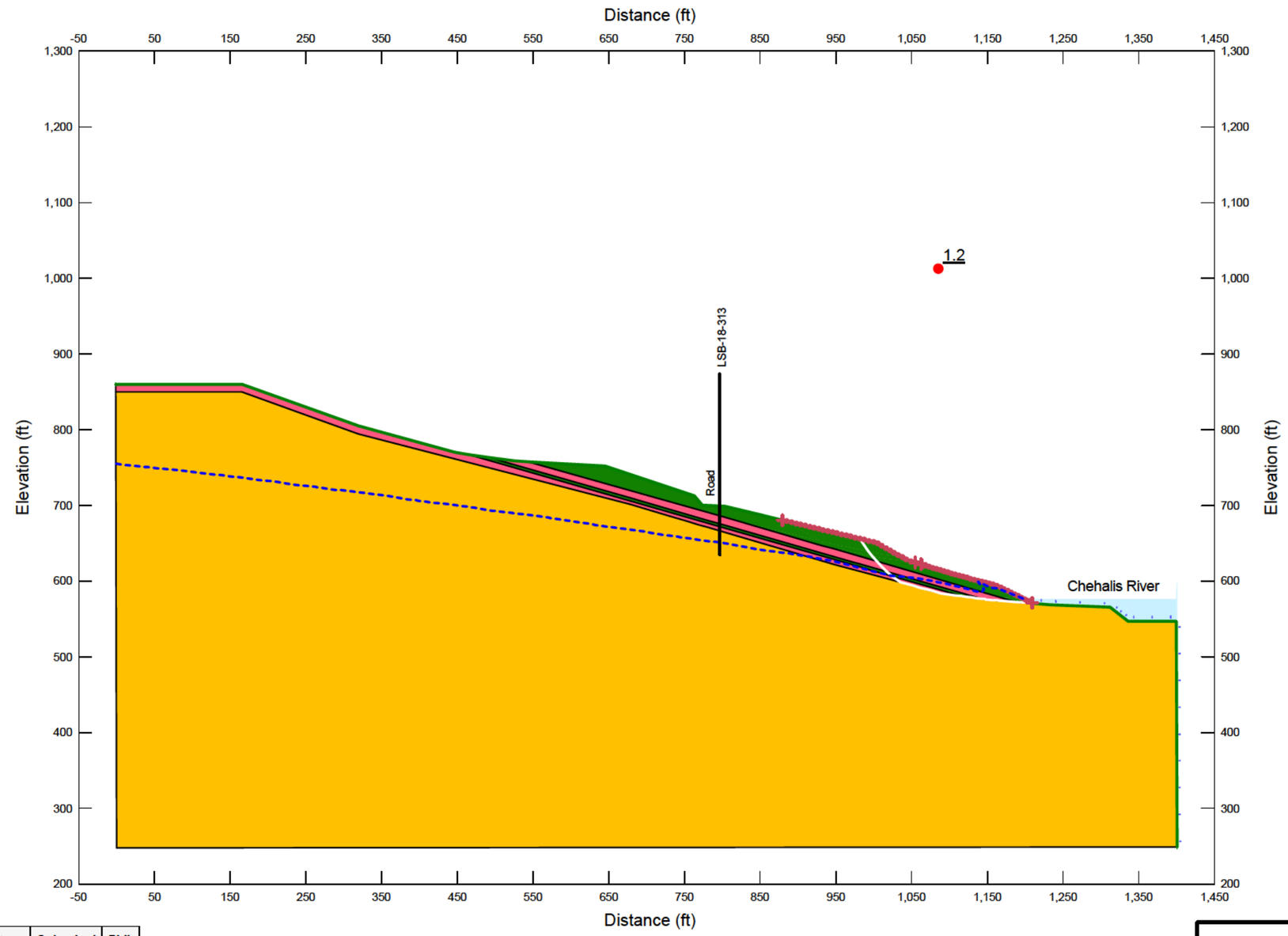

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bed Rock	Bedrock (Impenetrable)			
	Coarse-Grained Colluvium	Mohr-Coulomb	120	0	32
	Fine-grained Colluvium	Mohr-Coulomb	115	0	31

Chehalis Basin Strategy
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Pe Ell, Washington

STABILITY ANALYSIS

LS-13 - Retrogressive - Lower

May 2019 21-1-21897-025

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FIG. A-6

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Appendix B
Response to Comments Memorandum

TECHNICAL MEMORANDUM

Date: May 10, 2019

To: Heather Page, Robert Montgomery (Anchor QEA, LLC)

From: Stan Boyle (Shannon & Wilson, Inc.)

RE: Comments on Phase 3 Landslide Evaluation Draft Report from Department of Ecology Dam Safety Office – Shannon & Wilson response

The purpose of this memorandum is to respond to comments provided by Gus Ordonez, Washington State Dam Safety Office, on March 26, 2019, on a draft report titled "Phase 3 Landslide Evaluation."

Comment 1:

1) The analyses were conducted for the 100-year flood event. The report does not provide information explaining the reasons for selecting this specific flood event. The report should also explain how the 100-year flood event compares to other recent, significant flood events such as those of 1996, 2007, and 2009. This comparison should help to determine if a different flood event should also be considered in the stability analyses.

A 100-year flood event was selected because it is a standard used for evaluation of flood reduction projects. It has a remote possibility and short duration.

Comparisons of flood events are presented in the Operations Plan prepared by Anchor QEA (2017).

> Anchor QEA, 2017, *Operations Plan for Flood Retention Facilities*, Report prepared by Anchor QEA, LLC., Seattle, WA, for State of Washington Office of Financial Management and Chehalis Basin Workgroup, June.

The 100-year flood storage was not in the above-referenced 2017 document, but is it presented in the table below for comparison with the flood events evaluated.

Event Month	Max of FRE Storage - Existing (ac-ft)	Max of FRE Elevation - Existing (ft)
Jan 1990	31,440	578.4
Feb 1990	17,862	551.3
Nov 1990	26,292	569.0
Apr 1991	21,248	558.9
Feb 1996	45,984	601.4
Dec 2007	60,246	620.4
Jan 2009	35,674	585.5
100-year	48,149	604.4
10-year	25,576	567.6

The modeled 100-year flood event reservoir elevation is above the computed 1996 event reservoir elevation and below the computed 2007 event reservoir elevation. The 2007 event was somewhere between a 200- and 500-year flood in the watershed above the dam. The 2009 event is the next largest flood after the 1996 event, resulting in a computed reservoir elevation 19 feet below the 100-year elevation. Based on this comparison, the 100-year flood event reservoir impoundment was selected for landform and landslide stability evaluation. Floods resulting in a lower elevation reservoir would be expected to result in less reduction in stability of the landforms and landslides than would be expected for the 100-year flood event reservoir elevation.

Language has been added to the report stating why the 100-year flood event was selected. The hydraulic and operations plan reports prepared for the Project should be referred to understand the relationship between the 100-year flood and historic flood events.

Comment 2:

2) Landslide stability under seismic loading conditions was not evaluated.

Landslides LS-3 and LS-4 are near the FRE structure. Excessive movement of these landslides could directly impact the FRE structure or FRE facilities operation. Prior analyses found these landslides, in their current configuration, would not meet static stability criteria for the FRE facility for rapid drawdown conditions (Shannon & Wilson, 2017a). Shannon & Wilson (2017b) presents concepts to improve stability of LS-3 and LS-4 by installing drainage and toe buttresses.

To respond to this comment, stability analyses were conducted for the current study for LS-3 and LS-4 for a seismic event occurring when there is no reservoir, i.e., no impounded water. LS-3 and LS-4 were evaluated for using pseudo-static stability analyses assuming the post-FRE facility construction configuration, wherein drainage and toe buttresses may be constructed to improve their stability (see Shannon & Wilson, 2017b). The analyses assumed no reservoir during the seismic event because simultaneous occurrence of a seismic event and reservoir impoundment due to a storm are deemed unlikely. Discussion of these analyses results is included in the final report.

Stability analyses of Landslide LS-1, located immediately upstream and adjacent to the dam left abutment, was not evaluated for a seismic event. Evaluation of and development of stability improvements for LS-1 for both static and seismic conditions will be performed as part of future phases of FRE facility and FRE structure design.

Seismic-event-induced instability of other landforms/landslides upstream of the FRE structure when a reservoir is not present would not pose an immediate risk to the FRE facility, e.g., through creation of a tsunami in the reservoir, because no reservoir would be present. Seismic-event-induced instability and excessive movement of landslides and slopes could result in a landslide dam across the Chehalis main stem or one of its tributaries upstream of the FRE structure. These instabilities could occur for landforms and landslides evaluated in Shannon & Wilson (2017a) and the current report or other slopes within the watershed. The FRE facility condition should be checked after a seismic event. Actions to protect the FRE facility should be taken if landslide dams occur upstream of the FRE structure.

Comment 3:

3) The uncertainty and variability of strength parameters should be considered in a more thorough way.

Discussion on uncertainty and variability of soil strength and hydraulic parameters has been added to the report. Uncertainty and variability of these parameters were discussed in Shannon & Wilson (2017b). Soil strength testing was performed in 2018 on samples retrieved from 2018 explorations. These strength tests include residual and full-softened ring shear tests. These test results were used to develop upper and lower bounds and likely values for soil shear strength in the stability analyses.

Comment 4:

4) It is not clear how the strength parameters were obtained for some of the materials. In the Phase 2 landslide report, the shear strengths for some of the materials were based on correlations with SPT blow counts. The reference for the correlation was not provided. The reports should clearly indicate the source/reference/test/etc. of all the material properties used.

Discussion of how soil shear strength and hydraulic parameters were developed and sources relied on has been added to the report. To supplement strength parameter values selected based on correlations, as used in Shannon & Wilson (2017a, 2017b), soil strength testing was performed in 2018 on soil samples retrieved from the 2018 explorations. These tests included residual and full-softened ring shear tests. These test results were used to develop upper and lower bounds and likely values for soil shear strength in the stability analyses.

Comment 5:

5) In the GeoStudio models, a coarse mesh was used. The report does not indicate if the use of a finer mesh would influence the stability results.

To respond to this comment, we performed additional SEEP/W analyses of landslide LS-5 to compare stability analyses results obtained using the relatively coarse mesh automatically generated by the SEEP/W program (about 10 to 20 feet maximum dimension elements) with stability analyses results obtained using finer finite element meshes (5 and 10 feet maximum dimension). The automatically generated coarser mesh was used for seepage analyses of landforms and landslides evaluated in the draft of this report and Shannon & Wilson (2017a, 2017b).

Discussion and results of these analyses of the effect of mesh size on stability analyses results are included in the final report. The analyses results show the minimum computed FS during rapid drawdown, computed using the automatically generated coarse mesh, is within 2% of the minimum FS computed using the finer finite element meshes. This difference is not deemed significant. Therefore, the stability analyses results that rely on the seepage analyses performed using the coarser finite element mesh are applicable.

Appendix C
Important Information About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

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