DRAFT REPORT

Lords Lake East Dam

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Executive Summary

To address concerns by the Washington State Department of Ecology (Ecology) regarding the stability of the Lords Lake East Dam (Dam) operated and maintained by the Port Townsend Paper Company (PTPC), Golder Associates Inc. (Golder) has performed a geotechnical study to evaluate the stability of the Dam under the earthquake loading specified by Ecology. The geotechnical study consisted mainly of reviewing the previous geotechnical study completed in 1991, completing a subsurface exploration program and performing engineering analyses. The main conclusions of our geotechnical study are as follows:

- Our subsurface exploration program indicates the presence of a relatively weak layer within the Dam that would "liquefy" (i.e., lose its strength and stiffness) under earthquake loading as it becomes in a semi-liquid state. Theoretically the "liquefaction" of this layer could cause significant lateral movement of the Dam structure in the downstream direction, resulting in significant damage. This case of lateral movement is typically called "flow failure." This kind of soil movement typically happens toward the end of or after the end of earthquake shaking. Soil movement due to "flow failure" is difficult to estimate. The most known case history of "flow failure" occurred in February 1971. Pictures of the liquefaction-induced "flow failure" are presented in this website. https://research.engineering.ucdavis.edu/gpa/earthquake-hazards/liquefaction-lower-san-fernando-dam/

- Flow failure was not identified in the 2016 letter report prepared by Ecology. The main reason, in Golder's opinion, is the depth of the liquefiable layer encountered in the 1991 borings was assumed deeper than the depth assumed in the current study. The 1991 investigation identified a similar liquefiable layer at about 40 to 45 feet below the dam crest. The layer Golder identified, however, is at shallower depth (from about 25 to 35 feet). The 1991 borehole and Golder borehole, in both of which a liquefiable layer was encountered, are about only five to 10 feet apart horizontally from each other. Golder cannot speculate as to why this difference occurred over such a short horizontal distance.

- The Ecology letter report, however, identified a different kind of soil movement under the specified earthquake loading. This kind of soil movement occurs during earthquake shaking (i.e., before the onset of liquefaction) and can be reasonably estimated. Ecology estimated about 1.0 foot of displacement. Golder, using a similar approach to that used by Ecology, estimated about 1.5 feet of displacement.

In general, the results of Golder's study agree with Ecology's conclusions that the dam stability under seismic demands could be questionable, particularly the potential for flow failure. The analysis results for flow failure, however, highly depends on the extent of the liquefiable layer along the slope of the dam. There is no subsurface information on the slope of the dam available. As such, it is not feasible to draw a final conclusion regarding the potential for flow failure.

Golder would like to discuss the findings of this study with PTPC first and then with Ecology to evaluate risk and the next steps.

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1.0 INTRODUCTION

This report presents the findings of Golder Associates Inc.’s (Golder’s) geotechnical investigation and analysis to evaluate the performance of the Lords Lake East Dam owned by the City of Port Townsend and maintained by the Port Townsend Paper Company (PTPC).

1.1 Project Description

Lords Lake is approximately 6 miles northwest of Quilcene, Washington (Figure 1). The East Dam (Dam) is accessed from Lords Lake Loop Road and Little Quilcene Road. Golder understands that PTPC is proposing to raise the elevation of Lords Lake and increase the capacity of the reservoir. The Washington Department of Ecology (Ecology) issued a letter report dated March 28, 2016 expressing concerns of instability of the Dam in case of the proposed increase of capacity under seismic demands. Ecology’s letter report also recommended re-conducting the hydraulic analysis of the spillway next to the North Dam. Our scope does not include an evaluation of the North Dam nor spillway.

1.2 Site Description

The Lords Lake East Dam is on the eastern side of Lords Lake, directly bordering Lords Lake Loop Road. PTPC maintains the dam and the pipe system. The 36-inch reinforced concrete pipe and corresponding overflow box is the only utility at the site. There are also 4-inch PVC pipe drains serving as the curtain drain at the toe of the dam. Access to the crest of the dam is located off Little Quilcene Road and is monitored by PTPC.

2.0 REVIEW OF EXISTING INFORMATION

Golder has reviewed the available information that was provided by PTPC regarding the project. This information includes Ecology’s letter report dated March 28, 2016, as well two reports from Applied Geotechnology Inc. (AGI) dated February and March 1991, and a series of memorandums from Bechtel Corporation dated in 1957 and 1958. Those documents are the basis for the proposed field investigation and for establishing a general understanding of the project geology.

Golder was also provided with plans of the Dam by John W. Cunningham & Associates dated January 1956 and piezometer data. Golder used this information to help establish the inputs for the stability analysis, which is further discussed in Section 6.0.

3.0 GEOLOGIC CONDITIONS

Lords Lake lies in the Uncas quadrangle in the northeast Olympic Peninsula which is composed of the Crescent Formation – consisting of basaltic flows and breccias. The Crescent formation is thought to be the exposed mafic basement of the Coast Range which has been uplifted. Overlying the Tertiary bedrock units in the project area are consolidated surficial deposits related to the Vashon Stade of the Fraser glaciation.

The recent geologic history of the Puget Sound Lowland regions has been dominated by several glacial episodes. The most recent, being the Vashon Stade of the Fraser Glaciation and is responsible for most of the present-day geologic and topographic conditions in the Puget Sound area. As world-wide sea levels lowered and the Puget Lobe of the Cordilleran ice sheet advanced southward from British Columbia into the Puget Sound Lowland, deposits composed of proglacial lacustrine sediments, advance outwash, and lodgment till were deposited upon either bedrock or older pre-Vashon Sediments.

As the Puget lobe of the Cordilleran ice sheet glacier retreated northward, it deposited a discontinuous veneer of recessional outwash and local deposits of ablation till upon the glacial landscape. The sculpted landscape was characterized by elongate uplands, intervening valleys, and undulating outwash plains. Post glacial deposits include: alluvium deposits within active stream channels; modern lacustrine deposits; organic silt and local peat deposits within kettle depressions, drainages, and outwash channels; volcanic mudflow deposits; and landslide deposits.

Soils at the Lords Lake project area have been mapped and described as follows:

The Lidar-Revised Geologic Map of the Uncas 7.5’ Quadrangle, Washington, by Tabor et al. (2011), has the site underlain by Till (Qvt), described as a compact and firm, light- to dark-gray, nonstratified diamict containing subangular to well-rounded clast, glacially transported and deposited. Figure 2 presents the geologic map of the site.

Our observations were in general agreement with the mapped geology.

4.0 SUBSURFACE INVESTIGATION

Golder’s subsurface exploration investigation was performed on April 24 and 25, 2019 and consisted of advancing four boreholes. Shear wave velocity was measured in one of the borehole locations on April 29, 2019. The borehole locations are shown in Figure 3 and borehole records are presented in Appendix A.

Based on the soil conditions encountered in the field, the depths of proposed boreholes were modified from those presented in our proposal. Table 1 shows the final depths of the four boreholes compared to the proposed depths.

Table 1: Borehole Investigations

Borehole ID	**Location**	**Proposed depth [feet]**	**Actual depth [feet]**
GB-01	Dam crest	60	70
GB-02	Dam crest	50	41
GB-03	Dam toe	20	20
GB-04	Dam toe	20	30
Total Depth (feet)		150	161

Borehole GB-01 was extended to 70 feet to accommodate the shear wave velocity measurement equipment and ensure that these measurements could be performed for the entire thickness of the dam material. Borehole GB-02 was terminated at 41 feet because bedrock was encountered shallower than anticipated. Borehole GB-04 was extended to 30 feet due to an unexpected loose layer encountered at the top of the borehole.

The boreholes were advanced using a truck-mounted Mobile B-58 Drill operated by Holocene Drilling, Inc. under the full-time observation of a Golder geotechnical engineer. Hollow Stem Auger (HSA) method was used for three boreholes except for GB-01 which utilized the mud rotary method. Soil cuttings were drummed and removed from the site after completion of the investigation.

Geotechnical soil samples were collected at sampling intervals of either 2.5 feet or 5 feet and sealed in plastic containers and returned to our Redmond, Washington laboratory for further classification and geotechnical laboratory analysis. The boreholes were backfilled with bentonite chips in general accordance with Washington State regulations following drilling. In GB-01, a 2.5-inch Schedule 40 PVC casing was installed and grouted into place to serve as casing for shear wave velocity measurements. The borehole was completed with a flush-mounted surface monument.

4.1 Laboratory Testing

Geotechnical laboratory tests were conducted on selected samples in Golder's Redmond, Washington soil laboratory to characterize engineering and index properties of the encountered soils. Grain size distribution was tested in accordance with ASTM C136/C136M (sieve) and ASTM C117 (#200 wash). The #200 wash measures only the percentage of fines, while a sieve determines the percentage of gravel and sand in addition to fines. Atterberg limits tests were conducted per ASTM D4318 to evaluate the properties of the fine-grained material encountered. Moisture content for all samples was tested in accordance with ASTM D2216. Tables 2 and 3 present the results from the laboratory testing.

Table 2: Grain Size Distribution Testing Summary

Borehole ID	Sample Number	Sample depth [feet bgs]	% Gravel	% Sand	% Fines	USCS Soil Class	Description	Moisture Content [%]
Standard Sieve (ASTM C136/C136M)								
GB-01	S-5	22.5	39.4	46.3	14.3	SM/GM	Silty Sand and Silty Gravel	15.9
GB-02	S-4	20	39.8	45.6	14.6	SM/GM	Silty Sand and Silty Gravel	11.2
GB-02	S-5	22.5	33.0	52.1	14.9	SM	Gravelly Silty Sand	8.6
GB-02	S-7	27.5	33.4	52.2	14.4	SM	Gravelly Silty Sand	10.3
GB-04	S-2	7.5	27.0	52.6	20.4	SM	Gravelly Silty Sand	29.0
GB-04	S-3	10	41.9	43.6	14.5	SM/GM	Silty Sand and Silty Gravel	26.3
#200 Wash (ASTM C117)								
GB-01	S-4	20	-	-	13.6	-	-	15.1

Borehole ID	Sample Number	Sample depth [feet bgs]	% Gravel	% Sand	% Fines	USCS Soil Class	Description	Moisture Content [%]
GB-01	S-6	25	-	-	9.8	-	-	20.9
GB-02	S-3	15	-	-	16.0	-	-	10.7
GB-04	S-1	5	-	-	14.4	-	-	32.9

Notes: bgs=below the ground surface

Table 3: Atterberg Limits Testing Summary

Borehole ID	Sample Number	Sample Depth [feet bgs]	Liquid Limit	Plastic Limit	Plasticity Index	USCS Soil Class	Description	Moisture Content [%]
GB-01	S-2	10	25	20	5	CL-ML	Clayey Silt	15.5
GB-01	S-9	32.5	-	46	-	NP	Non Plastic	18.2
GB-02	S-6	25	27	23	4	ML	Silt	5.4

4.2 Sheave Wave Velocity Measurements

Sheave wave velocity (Vs) measurements were measured in borehole GB-01 on April 29, 2019. The Vs measurements were performed by Global Geophysics, LLC utilizing the suspension logging method. Appendix B presents the results. Based on our experience with similar soils in the region, Golder considers the measurements are questionable. While the “trend” of the data is consistent with the soils encountered in GB-01, the absolute values are not consistent with our experience. If advanced seismic analysis is required in a subsequent phase, Golder recommends new Vs measurements be done to confirm soil conditions, potentially using a method other than the suspension logging.

5.0 SUBSURFACE CONDITIONS

5.1 Soil Conditions

In general, the soil conditions encountered at the site are consistent with the geologic maps of the area. The subsurface soil units encountered include embankment fill, glacial till, and bedrock at varying degrees of weathering. Borehole GB-02 drilled at the crest of the southern half of the dam indicates that the embankment fill directly overlies bedrock. Weathered bedrock was encountered at the ground surface in borehole GB-03 drilled at the toe of the southern half of the dam. The embankment fill encountered in borehole GB-02 generally consists of medium dense (also described as “compact” in the borehole logs in Appendix A) gravelly sand. The bedrock appears to be volcanic in nature, which is consistent with geologic maps for the area.

In boreholes GB-01 and GB-04 drilled within the northern half of the site, the embankment fill overlies glacial till. The embankment fill encountered in GB-01 has a looser layer (compared to embankment fill in GB-02) from approximately 25 to 35 feet below the dam crest. The looser layer also includes a relatively thick (approximately 2 feet) organic seam. In borehole GB-04, a very loose clayey sand layer, which appears to be a fill or otherwise disturbed material, was encountered overlying a very dense till material. The very loose clayey sand layer, which is approximately 12-foot thick, is not similar to any other material encountered in the other boreholes and therefore, it is Golder's opinion that this layer is not a continuation of any material within the embankment.

The embankment fill was generally coarse with gravel contents measured from 30 to 40 percent. Such large amounts of gravel in a soil matrix may artificially elevate the Standard Penetration Test (SPT) blow counts.

Some of the soils encountered in our investigation are fundamentally different than those encountered in previous explorations at the site, which impacted the conclusion of our analysis. Golder's borehole GB-01 was drilled within several feet of AGI's Boring 1 (1991) on the crest of the dam. However, the soil conditions were significantly different from each other - most notable at a depth of approximately 40 feet. Where Golder encountered very dense glacially overridden soils, AGI encountered a very loose soil. The clouded area in Figure 4 demonstrates this discrepancy. This discrepancy and its impacts on the stability would be discussed further in Section 6.0.

The drill rig used in our investigation had a hammer efficiency of 96% whereas the drill rigs used in the 1990s typically had a hammer efficiency of less than 60%. SPT blow counts shown in Figure 4 are blow counts before hammer-efficiency correction. We expect the two curves in Figure 4 would be closer to each other after the correction. However, there is no available information of the hammer efficiency in the 1991 investigation and therefore the correction cannot be made on the 1991 SPT values to confirm.

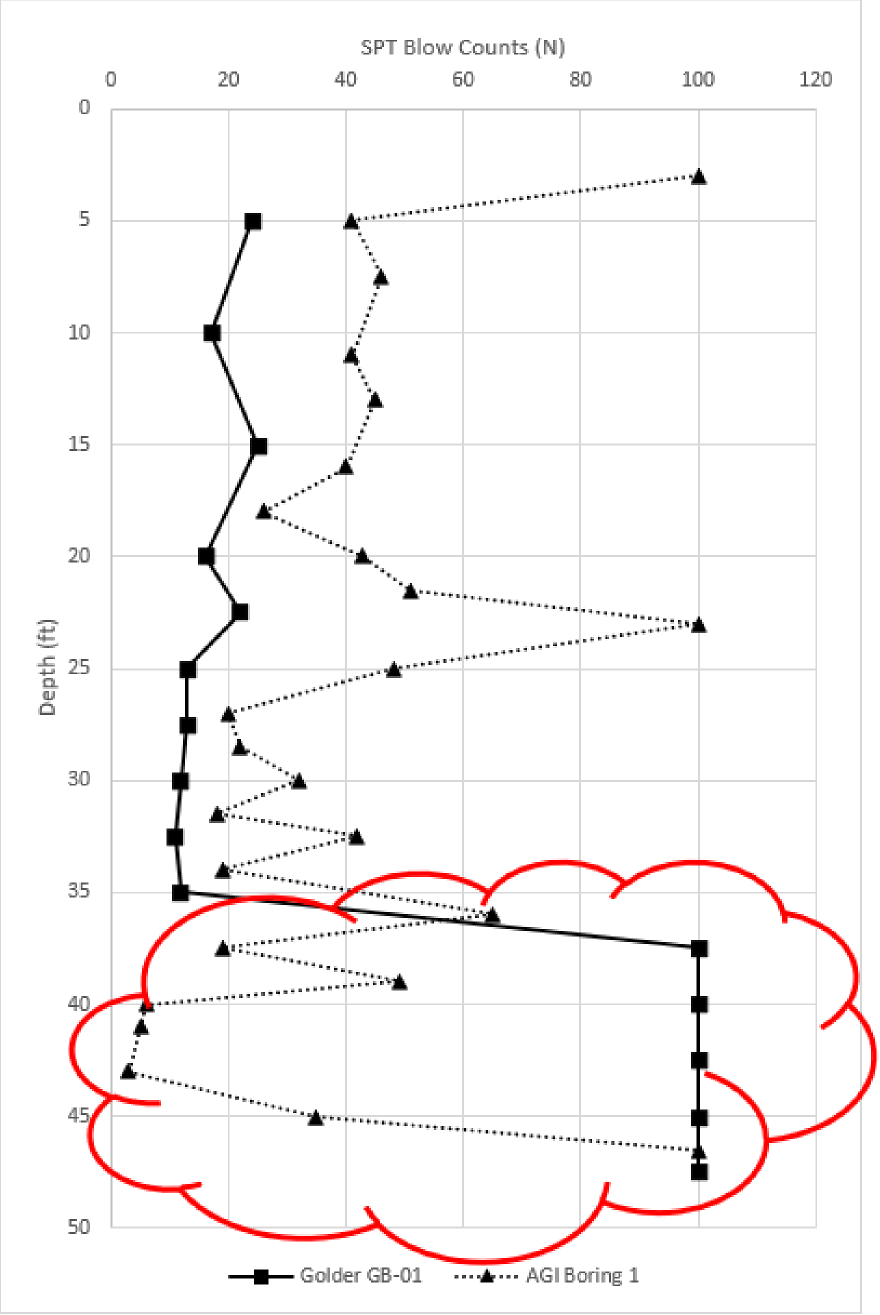

**Figure 4: Discrepancy between Golder 2019 investigation and AGI 1991 investigation.
Note the SPT blow counts are from field boring logs and are not corrected for hammer efficiency.**

Golder cannot speculate as to why this difference occurred, especially considering the close spacing of the boreholes as shown in Figure 3. However, the glacial contact we encountered in the field is consistent with all the other historical investigations in which bedrock or overridden material was encountered at depths less than 40 feet.

5.2 Groundwater Conditions

Groundwater conditions were assumed from field measurements, observations of soil samples, and piezometer readings provided to Golder by PTPC. Piezometer readings were interpreted assuming the same datum as was used in the plans by John W. Cunningham & Associates (1956). Based on readings from the piezometers at the crest of the dam, groundwater is on average approximately 15 feet below the crest of the dam. This approximate level was confirmed with observations of the soil samples from GB-01 and GB-02.

At the time of drilling, groundwater was measured at 17 feet below the ground surface in GB-03 and was not encountered in GB-04. Piezometers close to the toe of the dam, however, indicate that groundwater is approximately at the ground surface. We observed groundwater near the ground surface while drilling boreholes GB-03 and GB-04. This groundwater is likely perched on top of the relatively impervious weathered bedrock at borehole GB-03 and till material in GB-04. Samples in the upper 5-feet of both boreholes were wet, with high water contents, while deeper samples were dry or moist, further indicating that the groundwater measured in the piezometers is a perched water table.

The piezometer data show relatively consistent groundwater elevations despite varying water levels in the reservoir. This pattern indicates that the permeability of the dam embankment is relatively low, and that is why the fluctuation of groundwater within the dam embankment does not fluctuate with the temporary changes in the reservoir water level. Golder expects that if a higher water level elevation is held for a long period of time, groundwater in the embankment may stabilize to a higher level than the piezometers have recorded. To capture this expected behavior, Golder used the highest water level, approximately 14 feet below the crest of the dam, observed in the piezometers in this study. An advanced seepage analysis was not conducted for this stage of the analysis.

6.0 GEOTECHNICAL ANALYSIS

Golder evaluated the Dam’s stability under seismic loading. The seismic evaluation included the following:

- liquefaction analysis
- seismic slope stability analysis
- seismic displacement analysis

The above analyses were based on Golder’s subsurface investigation data. The analyses did not include data from the historical boreholes, given the associated uncertainties with those boreholes.

This section briefly presents input parameters, assumptions and results.

6.1 Seismic Parameters

Per Ecology’s report and previous analysis, the analyses listed above were based a design earthquake with a 2,500-year return period. The seismic Site Class was based on the average soil properties in the upper 100-feet of the site. Because none of the explorations extend to 100-feet, soil properties were extrapolated to estimate the Site Class. Based on measured blow counts, we assumed the dam embankment at the crest location is Site Class C.

Ground motion parameters were obtained using the USGS Unified Hazard Tool and from the USGS 2014 national seismic hazard maps. The accelerations for the Site Class B/C boundary were then adjusted using the site amplification factors for Site Class C. The following results were obtained for a latitude 47.8807°N and longitude 122.9313°W (a point located near the center of the site):

- PGA: 0.61 g
- Short (0.2 second) Spectral Acceleration
    - SS: 1.38 g
    - SMS: 1.66 g

6.2 Liquefaction Analysis

In general, loose to medium dense granular soil deposits below the groundwater table can be susceptible to liquefaction during earthquake shaking. The fill encountered at the site includes silty sands and sandy silts with gravel that are susceptible to liquefaction. Golder evaluated the liquefaction potential at the site using the simplified method presented in Boulanger and Idriss (2014). Liquefaction was evaluated assuming groundwater depths of 15 feet (from the crest of the dam) and 0 feet (from the toe). Depths of the groundwater change with seasonal variations as well as the reservoir water levels. Liquefaction analyses are often performed using an average groundwater depth of these variations. However due to the depth of the liquefiable layer and the properties of the soil, variations in the groundwater will have negligible impacts on the conclusions of the liquefaction analysis.

In addition to the PGA presented in Section 6.1, inputs for the liquefaction analysis are the corrected blow counts (N160 values) from the boreholes, and the magnitude of the design earthquake. Estimates for unit weight were also included in the inputs for the analysis.

Results from the analysis indicate liquefaction may be triggered at both boreholes along section A-A' (refer to Figure 3 for the cross-section location). The medium dense fill layer encountered in borehole GB-01 between 25 and 35 feet bgs is expected to liquefy under the design earthquake, and the shallow very loose sand in borehole GB-04 is also expected to liquefy. These units are not considered to be a continuous layer; though both liquefy, they are different soil units and are at different elevation. However, we assumed that the liquefiable layer encountered in borehole GB-01 is continuous across the dam and this assumption is critical for the stability of the Dam under seismic loading.

Liquefaction-induced settlements are estimated to range from approximately 4 to 6 inches.

No liquefaction is expected at boreholes GB-02 and GB-03 along section B-B'.

It is important to note that we did not perform liquefaction analysis for the conditions encountered in AGI boring 1. However, based on our experience, we expect that the soils encountered between 27 and 43 feet to liquefy under the design earthquake.

6.3 Stability Analysis

Stability of the Dam was evaluated using the slope stability program—SLIDE developed by RocScience. The Spencer method was used for the stability calculations. We considered four stability cases as follows:

- **Case 1 - Static Loading**. This case represents the steady state case for groundwater with drained soil properties.

- **Case 2 - Seismic Pseudo-Static Loading**. This case considers the stability of the Dam during earthquake shaking but assuming no soil liquefaction.
- **Case 3 - Seismic Flow Failure**. This case evaluates the stability of the Dam after the end of shaking (i.e. without seismic loading) but accounts for soil liquefaction. Liquefiable soils are assigned reduced strength (a.k.a. residual strength) compared to the strength value used in the other two cases presented above. This case is defined by the Bureau of Reclamation as “failure in which a soil mass moves over relatively long distances in a fluidlike manner” and would thus be qualified as a catastrophic failure of the dam.
- **Case 4 - Seismic Flow Failure during earthquake**. This case assumes that the triggering of liquefaction during rather than after earthquake. That case represents the worst-case scenario for seismic loading. The evaluation of that case was not performed at this point as Case 3 (seismic flow-failure case) indicates the Dam is not stable as discussed below.

Two cross sections were evaluated for stability. Section A-A’ extends from boreholes GB-01 to GB-04 at the north end of the dam. Section B-B’ extends from boreholes GB-02 to GB-03 at the south end of the dam. Figure 3 shows the cross sections.

6.3.1 Seismic Loading

For the pseudo-static analysis (Case 2), the seismic loading was calculated using the seismic parameters presented in Section 6.1 and adjustments based on the procedure for wave scattering effects presented in FHWA Seismic Design Manual (also used by Ecology in their 2016 analysis). This method resulted in a horizontal acceleration coefficient (k_h) of 0.26 g.

6.3.2 Slope Geometry

The geometry of the sections was developed using the borehole logs, the John W. Cunningham & Associates plans, and publicly available LiDAR data. The general geometry in the 1956 plans agreed with the conditions encountered in the explorations, indicating approximately 35 feet of embankment fill. Several features in the plans, however, were not observed in the field. For example, a culvert and headwall present on the plans was not observed in the field. Golder, therefore, utilized the Puget Sound LiDAR Consortium’s (2005) dataset to verify the geometry of the Dam. LiDAR data indicated that the height of the Dam was approximately 34 feet, which agreed with the assumptions made based on the borehole logs and 1956 plans.

We applied the same geometry to section A-A’ and B-B’ but assumed different material layering based on the conditions encountered in each borehole. For section A-A’, the potential liquefiable layer described in Section6.2 were included in the stability model. The extent of the liquefiable layer observed in GB-01 along the slope of the dam, however, is unknown, as there is no subsurface information on the slope. Golder assumed the liquefiable layer extends along the entire section. This assumption is consistent with the Ecology’s analysis in 2016. There is no liquefiable material observed in GB-02 or GB-03 and therefore no liquefiable layer in section B-B’.

6.3.3 Groundwater and Soil Conditions

Slope stability was evaluated for a reservoir elevation of 922.5 feet. Using piezometer data, the groundwater was modeled at a depth of 14 feet below the crest. Golder performed a sensitivity analysis on the stability considering groundwater depths of 13 and 15 feet and found that slight variations in the groundwater table did not have significant impacts on the results.

Soil parameters were estimated based on SPT values, laboratory testing, and experience with similar material in the area. Table 4 presents the soil parameters used in the stability analyses. The soil parameters are generally consistent with those recommended in Chapter 5 of the WSDOT Geotechnical Design Manual (2018).

Table 4: Soil Parameters for Stability Analyses

Soil Type	Unit Weight [pcf]	Friction Angle [°]	Cohesion [psf]
Embankment Fill	125	36	0
Liquefiable Fill	120	32 (Cases 1 and 2)	50 (Case 1 and 2)
		15 (Case 3)	0 (Case 3)
Glacial Till	130	40	50
Loose Sand	110	28	0
Weathered Bedrock	135	42	0
Rock Toe	135	40	0

6.3.3.1 Technical Discussion of Residual Strength Parameter Selection

This section presents a discussion of the development of soil parameters in case of liquefaction, which is applicable to Case 3. Chapter 6 of the WSDOT Geotechnical Design Manual (GDM) (2018) presents several methods based on case studies of historical failures for evaluating the residual strength of a soil after liquefaction occurs. Figures 6-4 through 6-7 in the GDM show plots for the various methods that estimate approximate residual strength based on the adjusted blow counts. The maximum value of adjusted blow counts in these plots based on the case studies (not extrapolated) is around 15 blows/foot. The adjusted blow counts of the liquefiable fill soil encountered in the dam embankment is approximately 18 to 20 (depending on whether the adjustment includes a clean sand correction). Therefore, none of the methods in the GDM extend to the blow counts of interest for the liquefiable fill at the site. Golder elected to assume a residual strength (in terms of a residual friction angle) that falls between two methods presented in the GDM. Based on these two methods, a residual friction angle of 15 degrees was estimated to be used in Case 3—seismic flow-failure analysis.

6.3.4 Stability Results

Ecology has established required factors of safety of 1.5 for static conditions and 1.1 for seismic conditions. According to the 2016 letter report, a factor of safety less than 1.1 for the seismic case does not indicate catastrophic failure, but rather the need for a displacement analysis. A factor of safety of less than 1.05 is considered to trigger a flow failure (Case 3), according to the GDM.

Table 5 shows a summary of the slope stability analysis results. The factors of safety listed in Table 5 are smaller than the required factors of safety for both static and seismic cases. Because the factor of safety against flow failure is less than 1.05, the other worst-case seismic analysis (Case 4) mentioned in Section 6.3 was not evaluated because flow failure will be triggered regardless of the timing of liquefaction (during or after shaking).

Appendix C shows outputs from the slope stability software.

Table 5: Summary of Slope Stability Results

Section	Stability Factor of Safety		
	Static (Case 1)	**Pseudo-Static (Case 2)**	**Flow Failure (Case 3)**
A-A’	1.3	0.77	0.91
B-B’	1.4	0.81	Not Applicable because no liquefaction

6.4 Seismic Displacements

Seismic displacement of the Dam was evaluated using the Bray and Macedo (2018) method for subduction zones. The method utilizes a spreadsheet which returns a range of displacements corresponding to the 16th and 84th percentiles of potential deformations based on a statistical analysis. The estimated deformation is along the sliding direction of the mobilized mass and is a combination of horizontal and vertical deformation of the dam.

6.4.1 Displacement Inputs

Inputs for the Bray and Macedo methods are the yield coefficient, the initial fundamental period (based on dam geometry and shear wave velocity), earthquake magnitude, and spectral acceleration at a period 1.5 times the fundamental period. Both the earthquake magnitude and spectral acceleration were estimated based on the USGS 2014 national seismic hazard maps. The deaggregation results for peak ground acceleration (PGA) and for a 2,475-year return period show that the primary contributing earthquake scenario is a magnitude 7.1 earthquake from the Cascadia subduction zone intraslab. As such, we assumed an earthquake magnitude of 7.1 in the displacement analysis.

The yield coefficient used in the analysis was determined from the pseudo-static stability analysis performed using SLIDE. For section A-A’, the yield coefficient (k_y) is 0.125 g and for section B-B’ the yield coefficient is 0.15 g.

6.4.2 Displacement Results

Since, k_y values presented above are less than k_h (0.26g as presented in Section 6.3.1), the design earthquake would result in embankment displacements. Our displacement analyses indicated the following embankment displacements (16th to 84th percentile):

- Section A-A’: 0.3 to 1.4 feet
- Section B-B’: 0.2 to 1.0 feet

6.5 Comparison with Ecology

Golder received Ecology’s 2016 stability analysis via email on May 20, 2019. Golder used Ecology’s analysis to compare assumptions and results. Ecology’s stability section is based on the information from the 1991 reports completed by AGI and corresponds roughly with our section A-A’.

In Ecology's analysis, the liquefiable layer is modeled below the embankment fill, at a depth of approximately 40 feet below the crest of the dam. As discussed in Section 5.1, this is not consistent with what Golder encountered in our field investigation. Based on our investigation, a medium dense liquefiable layer is present within the embankment fill at depths of approximately 25 to 35 feet below the crest at the north end of the dam (GB-01), overlying glacial till.

These discrepancies between the Ecology's and Golder's models are attributed to the information available at the time of the analysis and the fundamental difference in subsurface conditions encountered in 1991 versus Golder's 2019 investigation. These differences have implications on stability results and conclusion. The stability analysis completed by the Ecology did not include the flow-failure case (Case 3). In the Ecology's model, the depth of the liquefiable material is below the grade of the road. The roadway, therefore, acts as a buttress against potential flow failure of the liquefiable material. Because Golder's model has liquefiable soil above the grade of the road, there is no buttress, and the potential for flow failure is much higher.

We are not certain, but it is likely that AGI based on its analysis on a design earthquake with a 475-year return period that is weaker that the current design earthquake with a 2,475 return period. As mentioned in Section 6.2, we expect that the soils encountered in AGI's boring 1 between 27 and 43 feet to liquefy under the current design earthquake.

Golder has also assumed slightly different soil parameters for the various units than those assumed by Ecology. These differences are not expected to have a large impact on the results of the stability analysis.

7.0 CONCLUSIONS AND NEXGT STEPS

Golder's conclusions are presented below.

- The dam embankment would experience displacements during shaking of the design earthquake and that is consistent with Ecology's conclusion in 2016, despite the differences in our models.
- A part of the dam may experience liquefaction and flow failure under the design earthquake.

The findings are most sensitivity to the assumption of the extent of the liquefiable layer along the slope of the embankment.

The proposed next steps are:

1) Meet with Ecology to present the findings of this study
2) Prior to resorting to a more sophisticated analysis, which is time-consuming and expensive, Golder recommends that additional subsurface investigation be done on the slope of the dam (as opposed to just at the crest and the toe) to evaluate the extent of the liquefiable soils.
3) The decision to resort to a more sophisticated analysis would depend on the results of the step above. In case the additional subsurface investigation shows extensive liquefaction, a more sophisticated analysis may not be warranted and for that case ground improvement of the liquefiable layer would be required to stabilize the dam against potential flow failure. In general, ground improvement could be designed based on simple analyses.
4) On the other hand, if the additional subsurface investigation shows that the liquefiable soil is limited and is not as extensive as assumed, a sophisticated analysis could be warranted.

8.0 CLOSING

This report has been prepared exclusively for the use of Port Townsend Paper Company and its consultants for the subject project. The conclusions and recommendations in this report are not intended, nor should they be construed to represent, a warranty regarding the project. Soil and groundwater conditions depicted are only for the specific dates and locations reported and, therefore, are not necessarily representative of other locations and times.

Judgement has been applied in interpreting and presenting the results. Variations in subsurface conditions outside the explorations are common in environments similar to those present at the site.

Signature Page

Golder Associates Inc.

DRAFT

DRAFT

Feng Li
Senior Project Engineer

Reda Mikhail
Principal/Geotechnical Practice Leader

FL/RM/em

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Figures

CLIENT
Port Townsend Paper Company (PTPC)

PROJECT
Lords Lake East Dam

CONSULTANT
GOLDER

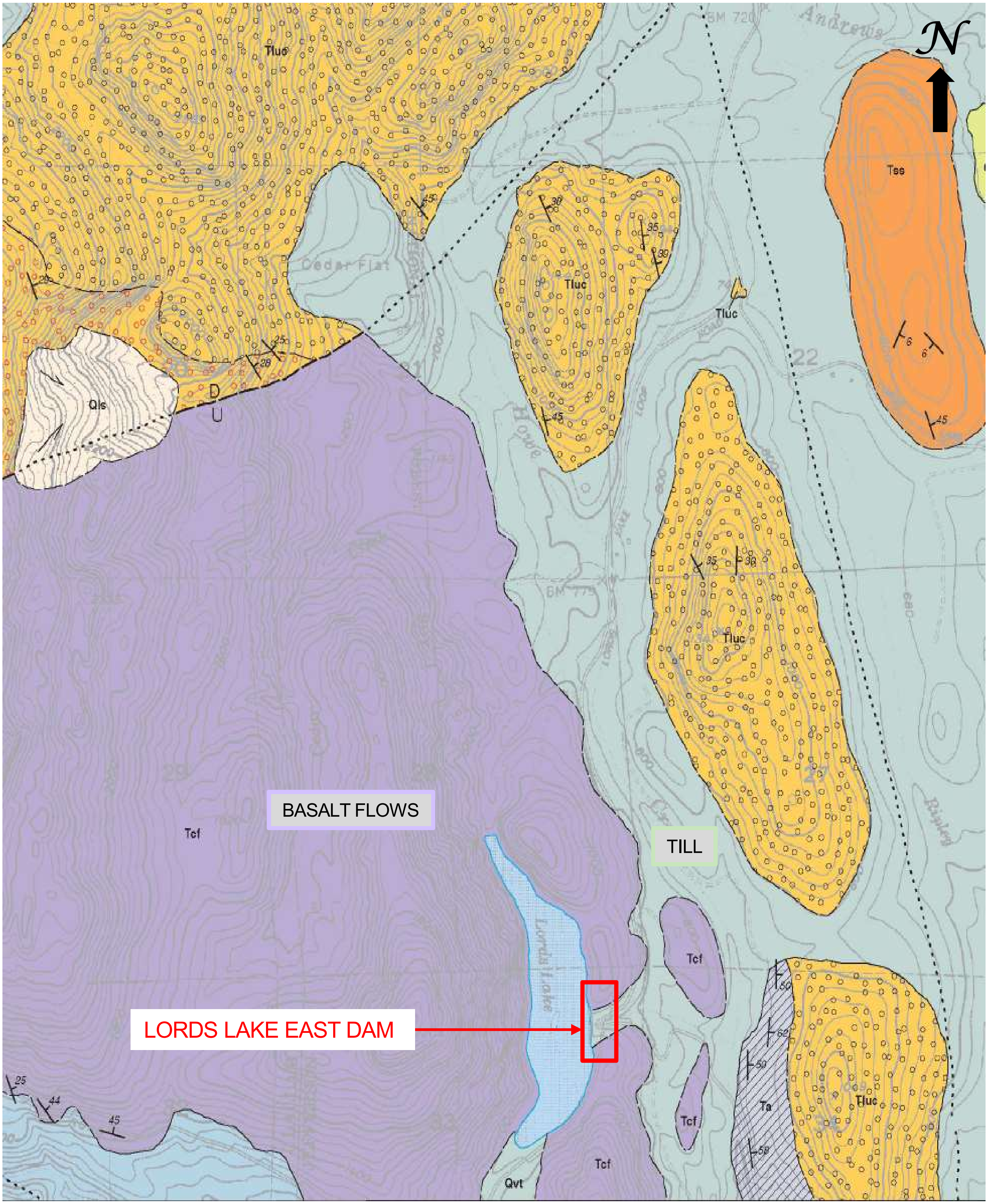
YYYY-MM-DD 2019-05-21
PREPARED CRS
DESIGNED -
REVIEWED FL
APPROVED -

TITLE
SITE VICINITY MAP

PROJECT NO.
19121213

REV.
DRAFT

1



CLIENT
Port Townsend Paper Company (PTPC)

PROJECT
Lords Lake East Dam

CONSULTANT

GOLDER

YYYY-MM-DD	2019-05-21
PREPARED	CRS
DESIGNED	-
REVIEWED	FL
APPROVED	-

TITLE
GEOLOGIC MAP

PROJECT NO.
19121213

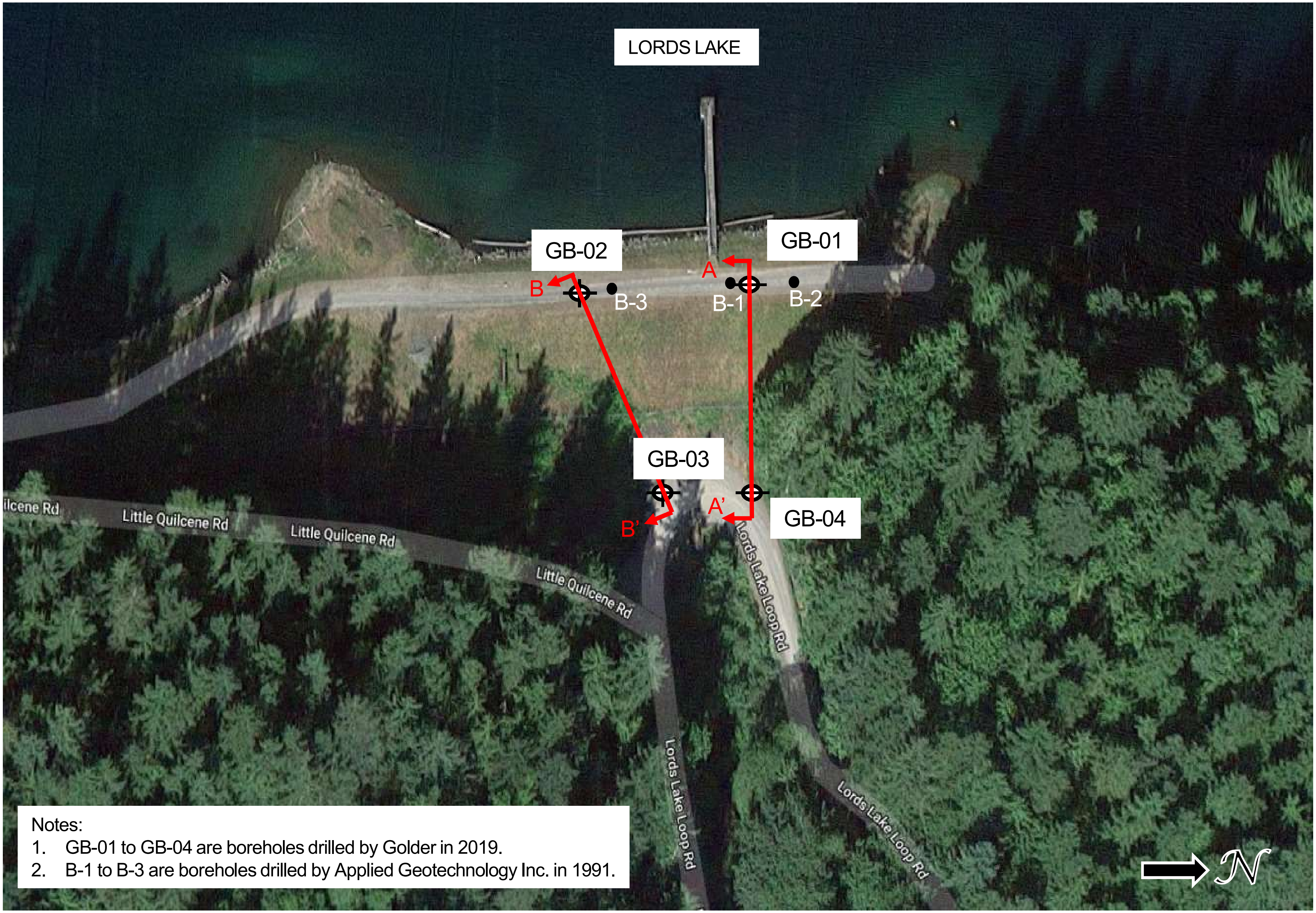
REV.
DRAFT

2

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

1 in

0



Notes:

1. GB-01 to GB-04 are boreholes drilled by Golder in 2019.
2. B-1 to B-3 are boreholes drilled by Applied Geotechnology Inc. in 1991.

CLIENT
Port Townsend Paper Company (PTPC)

PROJECT
Lords Lake East Dam

CONSULTANT

GOLDER

YYYY-MM-DD	2019-05-21
PREPARED	CRS
DESIGNED	-
REVIEWED	FL
APPROVED	-

TITLE
BOREHOLE LOCATIONS

PROJECT NO.
19121213

REV.
DRAFT

3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

1 in

0

APPENDIX A

Shear Wave Velocity Measurements

Appendix A

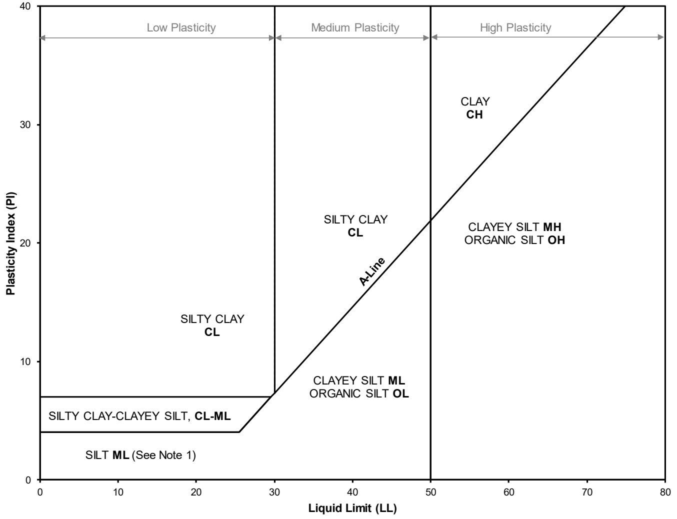
Drilling and sampling were performed in general accordance with Golder Technical Procedures. Standard penetration test (SPT) samples were taken at approximately 2.5-foot or 5-foot intervals. Soil samples were taken using a 2-inch diameter split-spoon sampler advanced with a 140-pound auto-hammer falling a distance of 30-inches for each strike in accordance with American Society for Testing and Materials (ASTM) D1586. The number of hammer blows for each 6 -inches of penetration was recorded. The standard penetration resistance (N-value) of the soil is calculated as the sum of the number of blows required for the final 12 inches of sampler penetration. The N-value is an indication of the relative density of cohesionless soils and the consistency of cohesive soils. If 50 blows are recorded for a single 6-inch interval, this is considered refusal and driving the sampler is stopped; the blow count is recorded as 50 blows for the total inches of penetration – such as 50 blows/5-inches.

METHOD OF SOIL CLASSIFICATION

The Golder Associates Inc. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10}xD_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content <30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with <12% * fines (by mass)	Poorly Graded	<4	≤1 or ≥3	<30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (>50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with <12% * fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content <30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below*)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3 mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3 mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3 mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below*)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to <50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CL	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous, or amorphous peat							75% to 100%		PEAT



Note 1 – Fine-grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are Non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with <5% organic content, include the descriptor “trace organics.” For soils with between 5% and 30% organic content include the prefix “organic” before the Primary name.

* Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC, and~~,~~ CL-ML. For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between “clean” and “dirty” sand or gravel).

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see plasticity chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, GM/SM, CL/ML. A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

SYMBOLS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimeters	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	> 300	> 12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	< 0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY CLAYEY" as applicable
> 5 to 12	Some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Test (DCPT), N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996) and correspond to typical N_{60} values. Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the compactness term. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	Undrained Shear Strength (tsf)	SPT 'N'[1,2] (blows/foot)
Very Soft	<12	<0.12	0 to 2
Soft	12 to 25	0.12 to 0.25	2 to 4
Firm	25 to 50	0.25 to 0.5	4 to 8
Stiff	50 to 100	0.5 to 1	8 to 15
Very Stiff	100 to 200	1 to 2	15 to 30
Hard	>200	>2	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does not apply. Rely on direct measurement of undrained shear strength or other manual observation.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
$\rho(\gamma)$	bulk density (bulk unit weight)* ($\gamma' = \gamma - \gamma_w$)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
p	mean total stress $(\sigma_1 + \sigma_3)/2$
τ_p, τ_r	peak and residual shear strength
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:

1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

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* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

DRAFT

RECORD OF BOREHOLE GB-01

SHEET 1 of 2

PROJECT: Lord's Lake East Dam
PROJECT NUMBER: 19121213
LOCATION: Lord's Lake East Dam - Crest
DRILLING METHOD: Mud Rotary
DRILLING DATE: 4/25/2019
DRILL RIG: Mobile B-58
DATUM:
COORDINATES: not surveyed
ELEVATION: 926
INCLINATION: -90

DEPTH (Ft)	BORING METHOD	SOIL PROFILE DESCRIPTION	USCS	ELEV. DEPTH (Ft)	SAMPLES NUMBER	TYPE	BLOWS per 6 in 140 lb hammer 30 inch drop	N	REC ATT	PENETRATION RESISTANCE BLOWS / ft	NOTES WATER LEVELS GRAPHIC
0	Mud Rotary	0.0 - 24.0 FILL (SM/GM) Gravelly SILTY SAND and sandy SILTY GRAVEL, fine to coarse gravel, fine to medium sand, trace organics; brown; non-cohesive; moist to wet, compact. Rig chatter	SM /GM								
5					S-1	SS	14-15-9	24	0.2 1.5		
10					S-2	SS	12-15-2	17	0.7 1.5		PI=5
		Rig chatter									
15		No Recovery			S-3	SS	7-10-15	25	0.0 1.5		
20					S-4	SS	4-6-10	16	0.4 1.5		FC=13.6%
					S-5	SS	10-12-10	22	1.0 1.5		FC=14.3%
25		24.0 - 27.0 FILL (SP-SM) POORLY GRADED SAND with SILT and gravel, fine to medium sand, fine to coarse gravel, trace wood and roots; gray and brown; non-cohesive; wet, compact.	SP-SM	902.0 24.0	S-6	SS	8-8-5	13	0.4 1.5		FC=9.8%
		27.0 - 32.0 FILL (SM) Gravelly SILTY SAND, fine to medium, fine to coarse gravel; mottled gray, iron oxide staining; non-cohesive; wet, compact.	SM	899.0 27.0	S-7	SS	9-8-5	13	0.5 1.5		
30		No Recovery			S-8	SS	6-7-5	12	0.0 1.5		
		32.0 - 35.0 FILL (OL) Sandy ORGANIC SILT, fine sand, wood chips and roots, some fine to medium gravel; dark brown and gray, non-cohesive; wet, stiff.	OL	894.0 32.0	S-9	SS	8-6-5	11	0.5 1.5		PI=NP
35		35.0 - 37.0 FILL (CL) Gravelly SILTY CLAY, fine gravel, some fine to medium sand, trace organics; dark brown and gray, cohesive; wet, stiff.	CL	891.0 35.0	S-10	SS	3-6-6	12	0.2 1.5		
		37.0 - 71.0 TILL (ML) Gravelly SILT, fine to coarse faceted gravel, some fine to medium sand; gray, cohesive; moist to wet, hard.	ML	889.0 37.0	S-11	SS	14-39-50	>50	1.0 1.5	>>	
40		Log continued on next page									

Penetration resistance scale: 10 20 30 40; PL MC LL; 20 40 60 80

1 in to 5 ft
DRILLING CONTRACTOR: Holocene
DRILLER: Matt Graham

LOGGED: Carly Schaeffer
CHECKED: Amy McMillin
DATE: 5/16/2019

GOLDER

DRAFT

RECORD OF BOREHOLE GB-01

SHEET 2 of 2

PROJECT: Lord's Lake East Dam
PROJECT NUMBER: 19121213
LOCATION: Lord's Lake East Dam - Crest
DRILLING METHOD: Mud Rotary
DRILLING DATE: 4/25/2019
DRILL RIG: Mobile B-58
DATUM:
COORDINATES: not surveyed
ELEVATION: 926
INCLINATION: -90

DEPTH (Ft)	BORING METHOD	SOIL PROFILE DESCRIPTION	USCS	ELEV. / DEPTH (Ft)	SAMPLES NUMBER	TYPE	BLOWS per 6 in (140 lb hammer 30 inch drop)	N	REC / ATT	PENETRATION RESISTANCE BLOWS / ft ◆ (10 20 30 40); PL MC LL (20 40 60 80)
40	Mud Rotary	37.0 - 71.0 TILL (ML) Gravelly SILT, fine to coarse faceted gravel, some fine to medium sand; gray, cohesive; moist to wet, hard.	ML		S-12	SS	50/6	>50	0.3 1.0	>>◆
		Increase in sand content			S-13	SS	50/3	>50	0.3 0.3	>>◆
45		Rig Chatter			S-14	SS	50/6	>50	0.5 1.5	>>◆
					S-15	SS	38-50/6	>50	1.0 1.0	>>◆
50					S-16	SS	35-50/4	>50	0.8 0.8	>>◆
		Increase in gravel content, decrease in sand content								
55					S-17	SS	50/5	>50	0.3 0.4	>>◆
60		Rock fragments in shoe, potential cobble/boulder present			S-18	SS	27-50/6	>50	1.0 1.0	>>◆
65					S-19	SS	45-45-50/5	>50	0.8 1.4	>>◆
70				855.0 71.0	S-20	SS	39-50/6	>50	0.3 1.0	>>◆
		2.5" Schedule 40 PVC installed to 71" No groundwater noted due to drilling method Boring completed at 71.0 ft.								
75										
80										

NOTES WATER LEVELS GRAPHIC

1 in to 5 ft
DRILLING CONTRACTOR: Holocene
DRILLER: Matt Graham

LOGGED: Carly Schaeffer
CHECKED: Amy McMillin
DATE: 5/16/2019

GOLDER

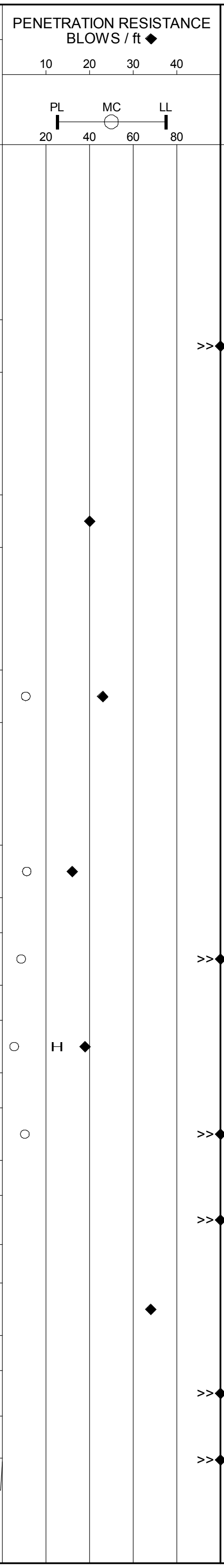
DRAFT

RECORD OF BOREHOLE GB-02

SHEET 1 of 2

PROJECT: Lord's Lake East Dam
PROJECT NUMBER: 19121213
LOCATION: Lord's Lake East Dam - Crest
DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 4/24/2019
DRILL RIG: Mobile B-58
DATUM:
COORDINATES: not surveyed
ELEVATION: 926
INCLINATION: -90

DEPTH (Ft)	BORING METHOD	DESCRIPTION	USCS	ELEV. / DEPTH (Ft)	NUMBER	TYPE	BLOWS per 6 in (140 lb hammer 30 inch drop)	N	REC / ATT	NOTES WATER LEVELS
0	Hollow Stem Auger	0.0 - 32.5 FILL (SM/ML) Gravelly SILT and SILTY SAND, fine to coarse gravel, fine to medium sand, trace organics; mottled gray and brown, iron oxide staining, non-cohesive; moist to wet, compact to very dense.	SM /ML							
5		No Recovery, rock fragments in sampler			S-1	SS	11-31-26	>50	0.0 / 1.5	
		Rig chatter								
10					S-2	SS	3-10-10	20	0.2 / 1.5	
15					S-3	SS	7-11-12	23	1.0 / 1.5	FC=16%
										▼ Estimate based on moisture in samples
20					S-4	SS	4-6-10	16	1.5 / 1.5	FC=14.6%
		Rock fragments in sample, potential cobbles/boulders, blow counts may be impacted			S-5	SS	25-18-36	>50	1.5 / 1.5	FC=14.9%
25					S-6	SS	10-8-11	19	1.5 / 1.5	PI=4
					S-7	SS	11-14-40	>50	1.5 / 1.5	FC=14.4%
30		Rock gragments in sample, potential cobbles/boulders, blow counts may be impacted			S-8	SS	7-33-50/5	>50	1.5 / 1.4	
		32.5 - 37.5 WEATHERED BEDROCK (BASALT), moderately to highly weathered with strength ranging from about R0 to R1, fine-grained, dark gray. Material is similar to a (ML) SILT WITH GRAVEL, fine to coarse angular gravel, some sand; dark gray, non-cohesive; moist to wet, dense to very dense.	ML	893.5 32.5	S-9	SS	20-16-18	34	1.0 / 1.5	
35					S-10	SS	7-13-50/4	>50	0.7 / 1.3	
		clean gravel seams noted in sample								
		37.5 - 41.0 BEDROCK (BASALT), slightly weathered to fresh with strength ranging from about R4 to R5.		888.5 37.5	S-11	SS	50/1	>50	0.0 / 0.1	
40		No Recovery								



1 in to 5 ft

Log continued on next page

DRILLING CONTRACTOR: Holocene
DRILLER: Matt Graham
LOGGED: Carly Schaeffer
CHECKED: Amy McMillin
DATE: 5/16/2019

GOLDER

DRAFT

RECORD OF BOREHOLE GB-02

SHEET 2 of 2

PROJECT: Lord's Lake East Dam
PROJECT NUMBER: 19121213
LOCATION: Lord's Lake East Dam - Crest
DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 4/24/2019
DRILL RIG: Mobile B-58
DATUM:
COORDINATES: not surveyed
ELEVATION: 926
INCLINATION: -90

DEPTH (Ft)	BORING METHOD	SOIL PROFILE			ELEV. / DEPTH (Ft)	SAMPLES					PENETRATION RESISTANCE BLOWS / ft ◆ (10 20 30 40) PL MC LL (20 40 60 80)	NOTES WATER LEVELS
		DESCRIPTION	USCS	GRAPHIC LOG		NUMBER	TYPE	BLOWS per 6 in (140 lb hammer 30 inch drop)	N	REC / ATT		
40		rough drilling, rig chatter		▨	885.0	S-12	SS	50/1	>50	0.0 / 0.1	>>◆	
		Boring completed at 41.0 ft. Refusal on bedrock			41.0							
45												
50												
55												
60												
65												
70												
75												
80												

1 in to 5 ft
DRILLING CONTRACTOR: Holocene
DRILLER: Matt Graham

LOGGED: Carly Schaeffer
CHECKED: Amy McMillin
DATE: 5/16/2019

GOLDER

DRAFT

RECORD OF BOREHOLE GB-03

SHEET 1 of 1

PROJECT: Lord's Lake East Dam
PROJECT NUMBER: 19121213
LOCATION: Lord's Lake East Dam - Toe

DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 4/24/2019
DRILL RIG: Mobile B-58

DATUM:
COORDINATES: not surveyed

ELEVATION: 892
INCLINATION: -90

DEPTH (Ft)	BORING METHOD	SOIL PROFILE DESCRIPTION	USCS	ELEV. DEPTH (Ft)	SAMPLES NUMBER	TYPE	BLOWS per 6 in 140 lb hammer 30 inch drop	N	REC ATT	PENETRATION RESISTANCE BLOWS / ft ◆ 10 20 30 40 PL MC LL 20 40 60 80	NOTES WATER LEVELS
0	Hollow Stem Auger	0.0 - 7.5 WEATHERED BEDROCK (BASALT) moderately to highly weathered with strength ranging from about R0 to R1, fine-grained, dark gray. Material is similar to a (SM), Gravelly SILTY SAND, fine to medium sand, fine to coarse angular gravel; gray, non-cohesive; dry to moist, very dense.	SM								Water at surface of hole
					S-1	SS	11-50/4	>50	0.5 0.8	>>◆	
5					S-2	SS	7-50/6	>50	0.3 1.0	>>◆	
		rough drilling 7.5 - 15.0 WEATHERED BEDROCK (BASALT) moderately to highly weathered with strength ranging from about R0 to R1, fine-grained, dark gray. Material is similar to a (ML), GRAVELLY SILT WITH SAND, fine to medium sand, fine to coarse angular gravel; brown and gray, non-cohesive; dry to moist, very dense.	ML	884.5 7.5	S-3	SS	13-50/5	>50	0.7 0.9	>>◆	
10					S-4	SS	18-50/5	>50	0.7 0.9	>>◆	
					S-5	SS	14-50/3	>50	0.5 0.8	>>◆	
15		15.0 - 21.0 WEATHERED BEDROCK (BASALT), moderately to highly weathered with strength ranging from about R0 to R1, fine-grained, dark gray. Material is similar to a (CL), Gravelly CLAY with SAND, fine to medium sand, coarse angular gravel (rock fragments), gray clasts within brownish matrix, non-cohesive; moist to wet, very dense.	CL	877.0 15.0	S-6	SS	39-50/2	>50	0.3 0.7	>>◆	
											▼ Measured 4/24/2019 10:00 am
					S-7	SS	7-50/6	>50	0.8 1.0	>>◆	
20					S-8	SS	20-50/6	>50	0.7 1.0	>>◆	
		Boring completed at 21.0 ft.		871.0 21.0							
25											
30											
35											
40											

1 in to 5 ft
DRILLING CONTRACTOR: Holocene
DRILLER: Matt Graham

LOGGED: Carly Schaeffer
CHECKED: Amy McMillin
DATE: 5/16/2019

GOLDER

DRAFT

RECORD OF BOREHOLE GB-04

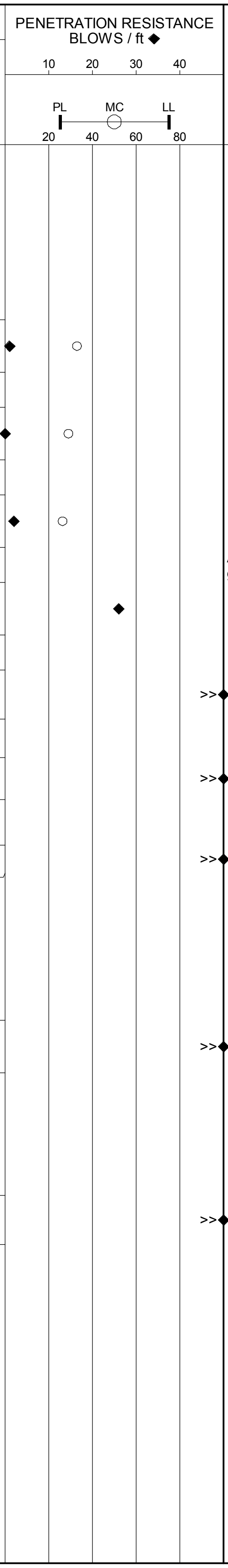
SHEET 1 of 1

PROJECT: Lord's Lake East Dam
PROJECT NUMBER: 19121213
LOCATION: Lord's Lake East Dam - Toe
DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 4/24/2019
DRILL RIG: Mobile B-58
DATUM:
COORDINATES: not surveyed
ELEVATION: 892
INCLINATION: -90

BORING METHOD: Hollow Stem Auger

DEPTH (Ft)	DESCRIPTION	USCS	ELEV. / DEPTH (Ft)
0	0.0 - 12.5 (SC) Gravelly CLAYEY SAND, medium to coarse sand, trace organics, fine to medium gravel, light to dark brown, non-cohesive; wet, very soft.	SC	879.5 12.5
12.5	12.5 - 31.4 TILL (ML) SANDY SILT with GRAVEL, fine sand, fine to coarse faceted gravel; light brown and gray, potential cobbles or boulders, non-cohesive; moist, compact to very dense. No Recovery (S-5) rig chatter (at approx. 23.5 ft)	ML	860.6 31.4
31.4	No groundwater measured Boring completed at 31.4 ft.		

NUMBER	TYPE	BLOWS per 6 in (140 lb hammer 30 inch drop)	N	REC / ATT	NOTES WATER LEVELS
S-1	SS	0-0-1	1	0.5 / 1.5	FC=14.1%
S-2	SS	0-0-0	0	1.5 / 1.5	FC=20.4%
S-3	SS	1-1-1	2	0.3 / 1.5	FC=14.5%
					Assumed perched groundwater on top of till
S-4	SS	5-10-16	26	1.5 / 1.5	
S-5	SS	5-21-50/5	>50	0.0 / 1.4	
S-6	SS	7-27-50/2	>50	1.0 / 1.2	
S-7	SS	27-50/4	>50	0.2 / 0.8	
S-8	SS	8-24-40	>50	1.5 / 1.5	
S-9	SS	10-33-50/5	>50	0.3 / 1.4	



1 in to 5 ft
DRILLING CONTRACTOR: Holocene
DRILLER: Matt Graham
LOGGED: Carly Schaeffer
CHECKED: Amy McMillin
DATE: 5/16/2019

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APPENDIX B

Slope Stability Outputs

Global Geophysics

P. O. Box 2229
Redmond, WA, 98073-2229
Tel: 425-890-4321
Fax: 360-805-0259

April 29, 2019

Our Ref.: 109-0423.000

Golder Associates, Inc.
18300 NE Union Hill Road, Suite 200
Redmond, WA 98052

Attention: Ms. Carly Schaeffer

RE: REPORT ON THE SUSPENSION LOGGING AT LORD'S DAM, WA

Dear Ms. Schaeffer:

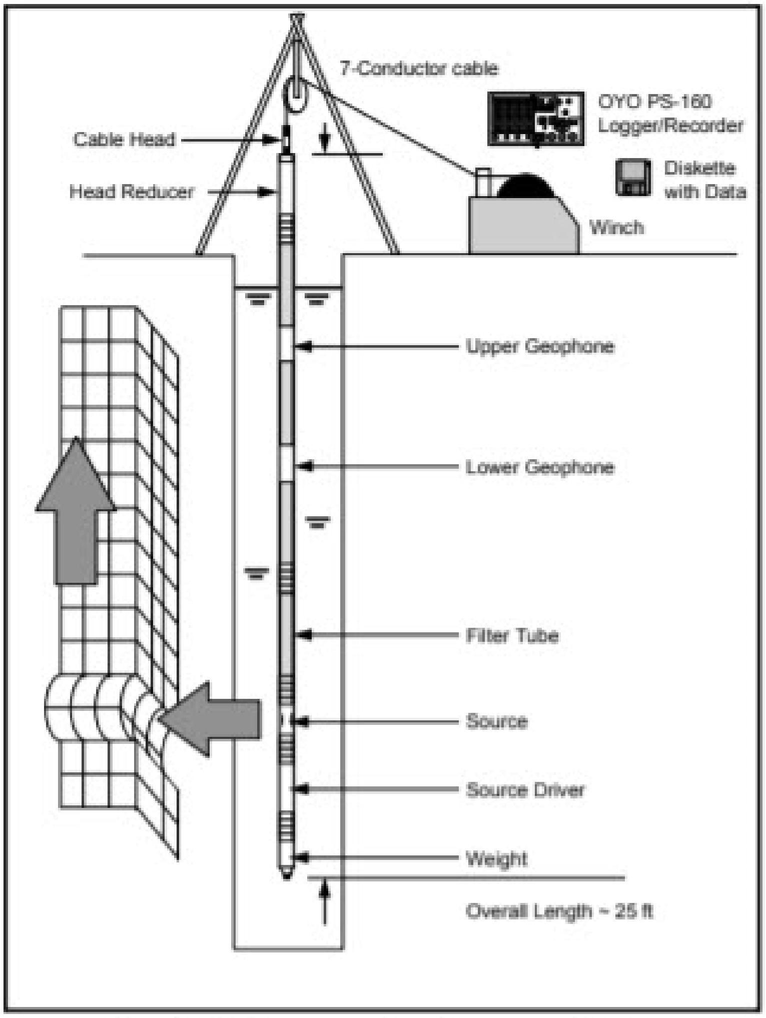
Global Geophysics conducted a suspension logging in borehole GB01 at Lord's Dam near Quilcene, WA on April 29, 2019. The proposed objective of the geophysical investigation was to determine the s-wave velocity of the soil column below the ground surface.

METHODOLOGY AND INSTRUMENTATION

Suspension logging

Soil velocity measurements were obtained using a suspension PS logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geologging. Data obtained with this system was used to calculate the average velocity of the soil column surrounding the boring by measuring the elapsed time of a wave propagating upward through the soil column from the transmitter to the receivers over a distance of 3.3 feet. The receivers that detect the wave, and the source that generates the wave, are moved as a unit down the boring at fixed intervals producing relatively constant amplitude signals at each depth where measurements are obtained.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear wave source (SH) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder. The separation of the two receivers is 3.28 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in these surveys is 21 feet. The probe receives control signals from, and sends the receiver signals to, instrumentation on the surface via an armored 4-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data, using a 1.3-foot circumference sheave fitted with a digital rotary encoder. The entire probe is suspended in the boring by the cable, therefore, source motion is not coupled directly to the boring walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the boring and surrounding the source. This pressure wave is converted to P and SH-waves in the surrounding soil and rock as it passes through the casing and grout annulus and impinges upon the wall of the boring. These waves propagate through the soil and rock surrounding the boring, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location.

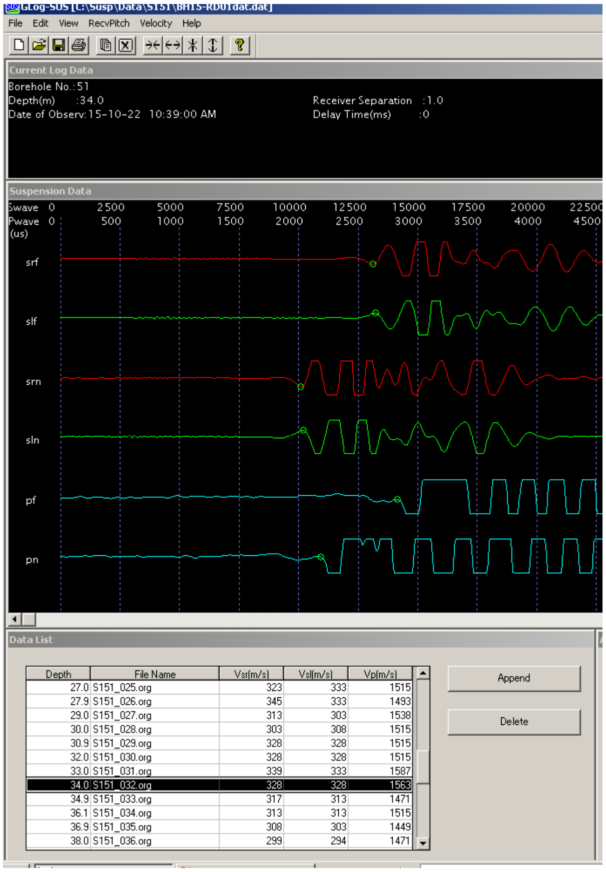


Oyo PS Suspension Logger Setup

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:
1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.
3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and SH-wave arrivals; reversal of the source changes the polarity of the SH-wave pattern but not the P-wave pattern.

QUALITY ASSESMENT ON SITE

The data quality was carefully monitored during acquisition. Polarity reversal for shear wave and strong s-wave data must be observed and confirmed. An example data is shown below.



Borehole No.:51
Depth(m) :34.0
Date of Observ:15-10-22 10:39:00 AM
Receiver Separation :1.0
Delay Time(ms) :0

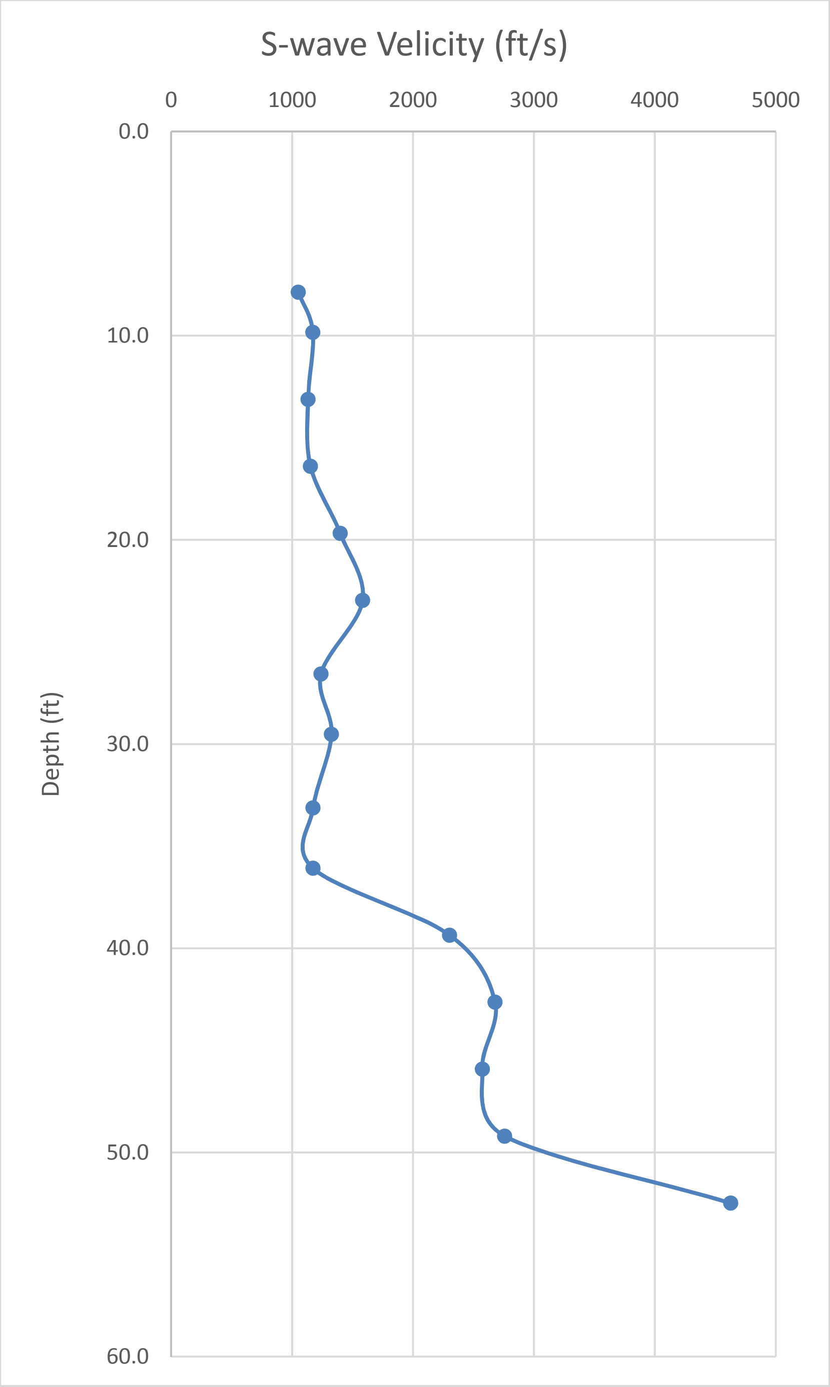
Depth	File Name	Vsr(m/s)	Vsl(m/s)	Vp(m/s)
27.0	S151_025.org	323	333	1515
27.9	S151_026.org	345	333	1493
29.0	S151_027.org	313	303	1538
30.0	S151_028.org	303	308	1515
30.9	S151_029.org	328	328	1515
32.0	S151_030.org	328	328	1515
33.0	S151_031.org	339	333	1587
34.0	S151_032.org	328	328	1563
34.9	S151_033.org	317	313	1471
36.1	S151_034.org	313	313	1515
36.9	S151_035.org	308	303	1449
38.0	S151_036.org	299	294	1471

RESULTS

The s-wave velocities from suspension logging are direct measurement of s-wave speed between the two geophones.

Table 1: S-wave velocity from Borehole GB01

Depth (ft)	S-wave velocity (ft/s)
7.9	1051
9.8	1171
13.1	1132
16.4	1151
19.7	1397
23.0	1583
26.6	1238
29.5	1325
33.1	1173
36.1	1173
39.4	2303
42.6	2678
45.9	2573
49.2	2758
52.5	4628


S-wave Velicity (ft/s)
0
1000
2000
3000
4000
5000
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Depth (ft)

LIMITATIONS OF THE GEOPHYSICAL METHODS

Global geophysics services are conducted in a manner consistent with the level of care and skill ordinarily exercised by other members of the geophysical community currently practicing under similar conditions subject to the time limits and financial and physical constraints applicable to the services. Suspension logging is a remote sensing geophysical method that may not detect all subsurface conditions due to the limitations of the methods, soil conditions, and size of the features.

Sincerely,

Global Geophysics, LLC.

John

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

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