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GEOTECHNICAL SITE INVESTIGATION REPORT

SNOQUALMIE PASS UTILITY DISTRICT (SPUD)
WASTERWATER TREATMENT PLANT IMPROVEMENTS
HYAK DRIVE EAST, HYAK, SNOQUALMIE PASS, WA

GNN PROJECT NO. 219-1154

Snoqualmie Pass Utility District

NOVEMBER 2019

Prepared for

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GN Northern, Inc.

November 5, 2019

HLA Engineering & Land Surveying, Inc.
2803 River Road
Yakima, WA 98902

Attn: Justin Bellamy, PE

Subject: **Geotechnical Site Investigation Report**
Snoqualmie Pass Utility District (SPUD)
Wastewater Treatment Plant Improvements
Hyak Drive East, Hyak, Snoqualmie Pass, WA

GNN Project No. 219-1154

Dear Mr. Bellamy,

As requested, GN Northern (GNN) has completed a geotechnical site investigation for the proposed improvements at SPUD's wastewater treatment plant located in Hyak, on Snoqualmie Pass, in Kittitas County, Washington.

Based on the findings of our subsurface study, we conclude that the site is suitable for the proposed construction provided that our geotechnical recommendations presented in this report are followed during the design and construction phases of the project.

This report describes in detail the results of our investigation, summarizes our findings and presents our recommendations concerning earthwork and the design and construction of foundation for the proposed project. It is important that GN Northern provide consultation during the design phase as well as field compaction testing and geotechnical monitoring services during the earthwork phase to ensure implementation of the geotechnical recommendations.

If you have any questions regarding this report, please contact us at 509-248-9798.

Respectfully submitted,

GN Northern, Inc.

Karl A. Harmon

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1.0 PURPOSE AND SCOPE OF SERVICES

This report has been prepared for the proposed improvements at SPUD's wastewater treatment plant located in Hyak, on Snoqualmie Pass, in Kittitas County, Washington; site location is shown on the *Vicinity Map* (Figure 1, Appendix I). Our investigation was conducted to collect information regarding subsurface conditions and present recommendations for suitability of the subsurface materials to support the proposed structures and allowable bearing capacity for the proposed construction.

GN Northern, Inc. has prepared this report for use by the client and their design consultants in the design of the proposed development. Do not use or rely upon this report for other locations or purposes without the written consent of GN Northern, Inc.

Our study was conducted in general accordance with our *Proposal for Geotechnical Engineering Services* dated September 12, 2019. Notice to proceed was provided in the form of an authorized copy of the *Agreement for Subconsultant Services to HLA Engineering and Land Surveying, Inc.* signed by Michael T. Battle on September 17, 2019.

An annotated copy of a Google Earth image, showing the approximate location of the proposed improvements, was provided via email on October 10, 2019. A topographic survey of the site with surveyed exploration locations (dated 10/31/19) prepared by HLA was also provided. Field exploration and testing, consisting of three (3) borings and two (2) test-pits, was completed on October 15 & 16, 2019. Locations of the exploratory borings and test-pits are shown on the *Site Exploration Map* (Figure 2, Appendix I), and detailed boring and test-pit logs are presented in Appendix II. Additionally, a Refraction Microtremor (ReMi) survey was performed at the site on September 26, 2019.

This report has been prepared to summarize the data obtained during this study and to present our recommendations based on the proposed construction and the subsurface conditions encountered at the site. Results of the field exploration were analyzed to develop recommendations for site development, earthwork, and foundation bearing support. Design parameters and a discussion of the geotechnical engineering considerations related to construction are included in this report.

2.0 PROPOSED CONSTRUCTION

Based on the information you provided in an email on September 9, 2019 along with communications held onsite, we understand the SPUD plans to construct a new 0.35 MGD MBR treatment facility in two phases across vacant property east of the existing lagoon treatment system. The complete facility is anticipated to be housed in an approximate 6,000 square-foot building (or possibly two separate buildings), including piping, tanks, electrical and equipment rooms, office, and laboratory. While no structural plans are available, based on preliminary information provided by SPUD personnel, the building(s) may consist of a pre-engineered metal structure with concrete tilt-up panels or a masonry block structure.

Structural loading information was not available at the time of this report. Based on our experience with similar projects, we expect maximum wall loads to be on the order of 3,000 to 4,500 plf and maximum column loads to be less than 100 kips. It shall be noted that assumed loading is based on limited preliminary information provided at the time of this report. If loading conditions differ from those described herein, GNN should be given an opportunity to perform re-analysis. Settlement tolerances for structures are assumed to be limited to 1 inch, with differential settlement limited to ½ inch.

3.0 FIELD EXPLORATION & LABORATORY TESTING

Prior to our field investigation, a GNN engineer met with HLA & SPUD personnel on September 24, 2019 to perform site reconnaissance and check for drill-rig access. Field exploration, consisting of three (3) borings and two (2) test-pits was completed on October 15 & 16, 2019. A local public utility clearance was obtained prior to the field exploration. The exploratory test-pits were excavated by SPUD personnel using a John Deere 410C backhoe to depths ranging from approximately 10 to 11 feet below existing ground surface (BGS). Borings were drilled by Holt Services, Inc. using a track-mounted Terra Sonic TSi 150 Compact Crawler drill rig to depths of approximately 20 to 30 feet. Following completion, the borings were abandoned in accordance with WA-DOE guidelines and test-pit excavations were loosely backfilled with the excavated spoils. Locations of the test-pits and boring are shown on the *Site Exploration Map* (Figure 2).

The sonic drilling method includes continuous sampling by advancing a 4.75-inch core barrel followed by a 6-inch steel casing. Advancement of the 5-foot or 10-foot long barrels and casing equipment into the subsurface strata is by two oscillators attached to the drill head. The oscillation frequency to advance the equipment varied and was generally dependent on the subsurface conditions encountered. Continuous soil samples were collected during drilling of the borings and upon removal of the sample barrel from the borehole the soil was extruded into long plastic bags with minimal disturbance. A GNN geologist/engineer inspected the soil samples for classification and collected disturbed soil samples into sealed plastic bags.

In addition to continuous sampling, Standard Penetration Test (SPT) samples were obtained at various depths. The SPT sampler has a 2-inch outside diameter and a 1.38-inch inside diameter. Samples were obtained by driving the sampler with a 140-pound automatic hammer, dropping 30 inches in general accordance with ASTM D1586. The number of blows required to advance the samplers through each 6-inch increment is recorded in the field. The SPT resistance, or N-value, is defined as the number of blows required to drive the sampler from 6 inches to 18 inches below the auger tip, with the value reported as the number of blows per one foot of penetration. The SPT N-value, adjusted for hammer efficiency and sampler size, provides an indication of the relative density or consistency of the soil and is indicated on the boring logs.

The soils observed during our field exploration were classified according to the Unified Soil Classification System (USCS), utilizing the field classification procedures as outlined in ASTM D2488. A copy of the USCS Classification Chart is included in Appendix II. Photographs of the site and exploration are presented in Appendix IV. Depths referred to in this report are relative to the existing ground surface elevation at the time of our investigation. The surface and subsurface conditions described in this report are as observed at the time of our field investigation.

Representative samples of the subsurface soils obtained from the field exploration were selected for testing to determine the index properties of the soils in general accordance with ASTM procedures. The following laboratory tests were performed:

Table 1: Laboratory Tests Performed

Test	To determine
Particle Size Distribution (ASTM D6913)	Soil classification based on proportion of sand, silt, and clay-sized particles
Natural Moisture Content (ASTM D2216)	Soil moisture content indicative of in-situ condition at the time samples were taken

Atterberg Limits (ASTM D4318)	Liquid limit, plastic limit and plasticity index of soils
Organic Matter (ASTM D2974)	Organic matter in peats and other organic soils, such as organic clays, silts, and mucks.

Results of the laboratory test are included on the boring and test-pit logs and are also presented in graphic form in Appendix III attached to the end of the report.

4.0 REFRACTION MICROTREMOR TESTING

A Refraction Microtremor (ReMi) survey was completed at the site on September 26, 2019 along one line at the location shown in the attached report in Appendix V. ReMi is a non-destructive testing method that uses ambient noise and surface waves to generate a vertical shear wave velocity (Vs) profile of soil strata up to 300 feet in depth. The ReMi method complies with ASCE 7-10 for seismic site class determination. The ReMi survey characterizes the site for seismic design (Vs_{100}) by estimating shear-wave velocities beneath the project area. Testing was performed at the surface using a 24-channel seismograph and geophones. The data was processed to produce a 1-D image of the subsurface shear-wave layering below the center of the geophone array. See Section 8.0 for seismic design parameters based on site class determined from ReMi testing.

5.0 SITE CONDITIONS

The site of the proposed improvements is planned at SPUD’s existing Wastewater Treatment Plant located on the south side of Hyak Drive East, approximately 500 feet south of the intersection with Rampart Drive, in Hyak, on Snoqualmie Pass, Kittitas County, Washington. A majority of the existing WWTP facility, including the area of the proposed improvements, is located within a 6.21-acre parcel identified as Parcel ID 218335 by the Kittitas County Assessor, and is located within southern ¼ of Section 15, Township 22 North and Range 7 East, Willamette Meridian. The existing facility includes three to four buildings and two wastewater lagoons.

Based on our observations, the site currently includes a relatively flat area in the eastern portion of the site, with a total grade of less than 5 feet across the proposed footprint. Surface conditions across the site include a moderate to dense growth of native grasses and a row of trees along the east of the project area. The northern portion of the project area currently includes several stockpiles of miscellaneous fill soils and construction debris. Embankment slopes of the existing wastewater lagoons exist along the west side of proposed footprint. A recently constructed shop building is located north of the project area. Iron Horse Trail generally extends along the east side of the wastewater facility.

The history of past use and development of the property was not investigated as part of our scope of services for this geotechnical site investigation. However, based on a cursory review of available USGS topographic maps and historic aerial photographs, it appears that prior to construction of the existing WWTP, the site of the proposed improvements was developed with three unknown structures located along the west side of the pre-existing Chicago, Milwaukee, St. Paul and Pacific Railroad (current day Iron Horse Trail). An aerial image from August 1984 shows signs of site disturbance across the current proposed project area, likely associated with recent grading/construction of the existing wastewater lagoons. An aerial image from 2003 also shows apparent signs of secondary fill placement across the project area. Findings of our subsurface exploration across the site further confirm presence of fill soils across the site (see Section 7.0).

6.0 GEOLOGIC SETTING

The project area lies on the northern end of the South Cascades physiographic province, which extends from the Columbia River to the south to north of Interstate 90. The Cascade Range of Washington State, a western rampart of the North American Cordillera, comprises an older basement of accreted terranes and a cover of sedimentary and volcanic rocks. Regionally, the north-trending Straight Creek Fault cuts the mosaic of accreted terranes and mostly separates higher grade metamorphic rocks of the North Cascade crystalline core to the east from lower grade and unmetamorphosed rocks to the west. Rocks west of the fault are further partitioned by a Late Cretaceous and (or) Paleogene suture (the Helena-Haystack mélange) and coincident Paleogene, high-angle Darrington-Devils Mountain Fault Zone.

In the eastern part of the Green River-Cabin Creek Block, near the Straight Creek Fault Zone, the strongly deformed Naches Formation consists of middle Eocene to early Oligocene volcanic rocks and interbedded fluvial feldspathic subquartzose sandstone. In the project vicinity, the Guye Sedimentary Member and the Mount Catherine Rhyolite Member of the Naches Formation are interbedded. The Guye Sedimentary Member is characterized as light to dark-gray feldspathic sandstone, black slaty shale, and hard chert-pebble conglomerates, while the Mount Catherine

Rhyolite Member are commonly flowbanded, platy jointed, black, welded, crystal-lithic ash-flow tuff containing highly flattened pumice lapilli, some volcanic breccia, and minor thin feldspathic sandstone and shale interbeds (Tabor et. al, 2000).

Overlying the local bedrock formations, late Pleistocene and Holocene alpine glacial deposits occupy many of the higher mountain valleys and cirques. Valley floors throughout much of the higher parts of the Yakima River drainage basin are mantled with relatively unweathered and little-eroded drift that retains much of its original constructional morphology. The Hyak Member of the Lakedale drift, mapped across the project area, is a late stillstand or readvance of glaciers in the higher parts of the Cascade Range. Regarding the Hyak Member, Porter (1976) stated: “Stony bluish-gray till at the base of exposed sections is overlain by laminated silt, clay, and sand, and locally by gravel, deposited during the initial stages of ice recession. These sediments are overlain by peat, interstratified by [Mt. St. Helens & Mt. Mazama] tephra layers.”

7.0 SUBSURFACE CONDITIONS

Our understanding of the subsurface soil conditions is based on five points of exploration across the proposed project area. In general, site soils include an upper layer of artificial fill material atop native peat underlain by interbedded fine to coarse soils and glacial till. Boring and test-pit logs in Appendix II show detailed descriptions and stratification of the soils encountered. The following sections provide a detailed summary of the noted soil strata:

Historic Fill: Soils in the upper approximately 6 to 8 feet were classified as artificial fill material. These soils typically included two distinct layers; an upper primarily silty layer and a lower primarily gravelly layer. The upper layer was generally classified as silt with varying amounts of gravels and cobbles, and included organics and misc. trash (fabric, brick, wire, wood). The lower bluish-gray silty gravel layer included cobbles and boulders, and significant wood debris (apparently related to rail-road ties) was noted at the bottom of the fill layer.

Native Peat: Apparent native peat (highly organic soil) was encountered beneath the cover of fill materials. This peat unit included significant woody remnants and fibers. Based on laboratory testing of three samples collected from this unit, the organic matter ranged from 24% to 66%. Up to two distinct layers of volcanic ash were also observed within the peat stratum. SPT blow-count data suggests ‘very soft’ to ‘soft’ relative in-place consistency for these soils.

Interbedded Fine/Coarse Soils: While not observed within the test-pits, borings penetrating through the native peat soils encountered the underlying bluish-gray interbedded layers of Silt (ML), Sandy Silt (ML) and Silty Sand (SM) at approximately 9 to 13 feet BGS, deepest towards the north. Sand particles were noted to range from fine- to coarse-grained. At depths of approximately 12.5 to 18.5, these interbedded soils include subrounded to subangular gravels. These gravel-rich soils were classified as Silty Gravel with Sand (GM), Silty Sand with Gravel (SM) and Gravelly Silt with Sand (ML). SPT blow-counts generally suggest 'loose' relative density for the upper interbedded silts/sands and 'medium dense to dense' relative density for the lower gravelly soils. Although altered by introduction of water during drilling, these soils appeared moist to wet.

Glacial Till: Borings B-6 and B-7 encountered the deeper glacial till unit at approximately 27 and 17 feet BGS, respectively. The recovered soils were visually classified as bluish-gray Silty Gravel (GM) with cobbles and appeared relatively dry. Laboratory testing indicates that this material is friable to a Silty Sand (SM). The till typically resulted in slightly harder drilling conditions with sonic method. SPT blow-counts indicate 'very dense' relative in-place density of this unit.

7.1 NRCS Soil Survey

Although altered at the surface, the soil survey map of the site prepared by the Natural Resources Conservation Service (NRCS) identifies the site soils as *Chinkmin ashy sandy loam* and *Thetis ashy sandy loam* with parent materials described as *volcanic ash and pumice over dense basal till* and *glacial till with a mantle of volcanic ash*, respectively (Appendix VI).

7.2 Groundwater

While no groundwater was noted within test-pit TP-4 excavated on October 16th, groundwater seepage was encountered at approximately 7.5 to 8 feet BGS within test-pit TP-3 (excavated on October 15th) perching atop the peat layer. Due to introduction of water into the boreholes during the drilling process, groundwater measurements within boreholes were not possible at the time. A review of the Washington Department of Ecology's online water well log database and USGS Water Data portal revealed a lack of nearby water wells in the site vicinity.

Groundwater levels across the project site are believed to be primarily controlled by heavy precipitation and snowmelt in the region. Heavy rainfall in the Snoqualmie Pass area typically begins in October, gradually changing to snow in November which continues until April. Infiltrated water will tend to percolate through the near-surface fill soils and flow atop less permeable soil/rock strata, including peat, dense glacial till units and bedrock.

Nearby surface water bodies include the exiting wastewater lagoons west of the project area, and Keechelus Lake located approximately 800 feet east of the site. Water levels within the nearby Keechelus Lake are controlled by the Keechelus Dam and can reach a maximum pool elevation of 2517', which is approximately 50 feet below the lowest existing site elevations.

8.0 SEISMIC DESIGN CONSIDERATIONS

The seismic site class evaluation was conducted by means of Refraction Microtremor testing (ReMi). Based on the test results, the weighted-average shear-wave velocity of the upper 100 feet of site soil profile (Vs_{100}) from a sounding is 1193.6 feet/second (ft/s). This shear-wave velocity corresponds to Site Class 'D' as described in Chapter 20, Section 20.3 and Table 20.3-1, and sub-section 20.3.3 of ASCE 7-10. Based on Section 1613.3 of the 2015 International Building Code (IBC), we recommend the following site-specific seismic design values:

Table 2: 2015 IBC Design Response Spectra Parameters

2015 IBC Section	Parameter	Value (unit)
1613.3.1	S_s	0.907 (g)
	S_1	0.341 (g)
1613.3.3	F_a	1.137 (unitless)
	F_v	1.718 (unitless)
	SM_s	1.031 (g)
	SM_1	0.586 (g)
1613.3.4	SD_s	0.688 (g)
	SD_1	0.390 (g)
Design Peak Ground Acceleration (PGA)†		0.485 (g)

†Based on Maximum Considered Earthquake (MCE) with 2% probability of exceedance in 50-years using Probabilistic Seismic Hazard Analysis (PSHA)

Based on Section 1613.3.5 of the 2015 IBC, a Seismic Design Category of D is considered appropriate for the project. The following sections provide a discussion of potential geologic and seismic hazards in accordance with Sections 1803.5.11 & 1803.5.12 of the 2015 IBC.

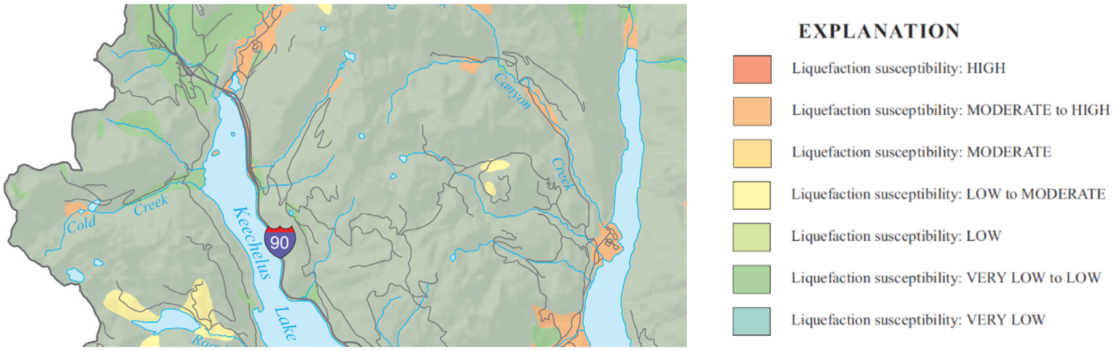
8.1 Slope Instability

The site of the proposed improvements is relatively flat with no significant slopes across the project area. Slopes along the west side of the proposed project footprint, associated with the existing lagoon embankments, are maximum ±10 feet tall at slope gradients of 20% or shallower. In our professional opinion, slope instability will not be a significant hazard provided the proposed improvements maintain an appropriate setback from the existing embankment slopes.

8.2 Soil Liquefaction & Lateral Spreading

Liquefaction is the loss of soil strength from sudden shock or vibration (usually earthquake shaking), causing the soil to become a fluid mass. Liquefaction results in a loss of soil strength and can cause building structures to settle if it occurs in the bearing zone. Soil liquefaction is a natural phenomenon that occurs when saturated granular soils (below the water table) are subjected to vibratory motions, causing an increase in the water pressure within soil pores, as the soil tends to reduce in volume. When the pore water pressure reaches the vertical effective stress, the soil particles become suspended in water causing a complete loss in soil strength.

The project site is currently mapped within a zone of 'very low to low' liquefaction susceptibility by the Washington State Department of Natural Resource, Division of Geology and Earth Resources (see excerpt below).



Excerpt from 'Liquefaction Susceptibility Map of Kittitas County, Washington' (Palmer, 2004)

In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. Liquefaction can cause excessive structural settlement, ground rupture, lateral spreading (movement), or failure of shallow bearing foundations. The following four conditions are generally required before liquefaction can occur:

- The soils must be saturated below a relatively shallow groundwater level.
- The soils must be loosely deposited (low to medium relative density).
- The soils must be relatively cohesionless (not clayey). Clean, poorly graded sands are the most susceptible. Silt (fines) content increase the liquefaction resistance in that more cycles of ground motions are required to fully develop pore pressures.
- Groundshaking must be of sufficient intensity to act as a trigger mechanism. Two important factors that affect the potential for soil liquefaction are duration as indicated by earthquake magnitude (M) and intensity as indicated by peak ground acceleration (PGA).

Based on the findings of our subsurface exploration, site soils include a cover of artificial fill atop native peat deposits overlying relatively loose interbedded silts/sands underlain the deeper glacial till unit. While peat soils are generally not considered to be susceptible to liquefaction, these deposits may be subject to permanent ground deformation caused by earthquake shaking. Furthermore, the loose sandy interbeds below the peat may be susceptible to liquefaction. However, geotechnical recommendations for foundation support presented in this report (Section 10.8) are intended to reduce the risk of soil liquefaction, and therefore, the potential for liquefaction at the site is considered low.

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when a layer of underlying soil loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present. Based on our understanding of the subsurface conditions and current site topography, it is our opinion that the risk of lateral spreading is low.

8.3 Surface Fault Rupture

The nearest regional faulting with Quaternary displacement (< 130,000 years) consists of the Rattlesnake Mt. Fault Zone located approximately 16 miles west of the project site (Czajkowski, 2014). For the purposes of this report, an active fault is defined as a fault that has had displacement within the Holocene epoch or last 11,700 years. Due to the lack of any known active fault traces in the immediate site vicinity, surface fault rupture is unlikely to occur at the subject property. While future fault rupture could occur at other locations, rupture would most likely occur along previously established fault traces.

9.0 SUMMARY OF FINDINGS & CONCLUSIONS

Conditions imposed by the proposed development have been evaluated on the basis of assumed elevations and engineering characteristics of the subsurface materials encountered in the exploratory borings and test-pits, and their anticipated behavior both during and after construction. The following is a summary of our findings, conclusions and professional opinions based on the data obtained from a review of selected technical literature and the site evaluation.

- Based on the findings of this geotechnical evaluation and our understanding of the proposed improvements, from a geotechnical perspective, it is our opinion that the site is suitable for the proposed construction, provided the soil design parameters and site-specific recommendations in this report are followed in the design and construction of the project.

- Final design plans for the proposed development, including site grading, drainage and finished elevations, were not provided at the time of this report**.** Once the plans are finalized, GNN **must** be provided an opportunity to review final design plans to provide revised recommendations if/as necessary.

- Site soils include an upper approximately 6- to 8-foot thick layer of artificial fill material atop relatively soft native peat soils extending to depths of approximately 9 to 12 feet BGS. Underlying soils consist of loose interbedded silts/sands and medium dense gravels atop a relatively dense glacial till formation.

- Groundwater seepage was encountered within a test-pit at approximately 7.5 to 8 feet perching atop the apparent native peat soil. Groundwater levels across the project site are believed to be primarily controlled by heavy precipitation in the region. Infiltrated water will tend to percolate through the near-surface fill soils and flow atop less permeable soil/rock strata, including the native peat soil, dense glacial till units and bedrock.

- Due to the presence of unsuitable fill and compressible native soils to depths of approximately 12 to 18 feet BGS, with seasonal perched groundwater conditions, along with the anticipated loading conditions for the proposed structure(s), an engineered aggregate pier foundation system (also referred to as stone columns or rammed aggregate piers) is the preferred option for support of the new building structure foundations and floor slabs for this project.

- Ground improvement through over-excavation and replacement with import soils for support of a shallow foundation system is not considered feasible due to the unsuitable fill and native soils extending to depths greater than 10 feet BGS.
- The existing fill soils, consisting of silty and gravelly soils with organics and trash/debris, are unsuitable for reuse. The native peat and underlying silt/sand soils are also not suitable for re-use as fill. Engineered fill and utility trench backfill shall consist of imported granular fill material as recommended in this report.
- Upon completion, test-pit excavations were loosely backfilled with excavation spoils. The contractor is responsible to locate the test-pits to re-excavate the loose soils and re-place as compacted engineered fill.
- Based on results of site-specific ReMi testing, seismic Site Class 'D' is considered appropriate for the project site. The *minimum* seismic design should comply with the 2015 International Building Code (IBC) and ASCE 07-10, Minimum Design Loads for Buildings and Other Structures.
- The near-surface site soils are susceptible to wind and water erosion when exposed during grading operations. Preventative measures and appropriate BMPs to control runoff and reduce erosion should be incorporated into site grading plans.

10.0 GEOTECHNICAL RECOMMENDATIONS

The following geotechnical recommendations are based on our current understanding of the proposed project as described in Section 2.0 of this report. The report is prepared to comply with the Section 1803, Geotechnical Investigations, of the 2015 IBC. Please note that Soil Design Parameters and Recommendations presented in this report are predicated upon appropriate geotechnical monitoring and testing of the site preparation and foundation and building pad construction by a representative of GNN's Geotechnical-Engineer-of-Record (GER). Any deviation and nonconformity from this requirement may invalidate, partially or in whole, the following recommendations. We recommend that we be engaged to review grading and foundation plans in order to provide revised, augmented, and/or additional geotechnical recommendations as required.

10.1 Site Development – Grading

Site grading shall incorporate the requirements of IBC 2015 Appendix J. The project GER or a representative of the GER should observe site clearing, grading, and the bottoms of excavations before placing fills. Local variations in soil conditions may warrant increasing the depth of over-excavation and recompaction. Seasonal weather conditions may adversely affect grading operations. To improve compaction efforts and prevent potential pumping and unstable ground conditions, we suggest performing site grading during dryer periods of the year.

Soil conditions shall be evaluated by in-place density testing, visual evaluation, probing, and proof-rolling of the imported fill and re-compacted on-site soil as it is prepared to check for compliance with recommendations of this report. A moisture-density curve shall be established in accordance with the ASTM D1557 method for all onsite soils and imported fill materials used as structural fill.

10.2 Clearing and Grubbing

At the start of site grading, any vegetation, trees, large roots, stockpiles of artificial fill, trash and debris, and any abandoned underground utilities shall be removed from the proposed building and structural areas. The surface shall be stripped of all topsoil and/or organic growth (vegetation) that may exist within the proposed structural areas.

10.3 Temporary Excavations

It shall be the responsibility of the contractor to maintain safe temporary slope configurations since the contractor is at the job site, able to observe the nature and conditions of the slopes and be able to monitor the subsurface conditions encountered. Unsupported vertical cuts deeper than 4 feet are not recommended if worker access is necessary. The cuts shall be adequately sloped, shored or supported to prevent injury to personnel from caving and sloughing. The contractor and subcontractors shall be aware of and familiar with applicable local, state and federal safety regulation including the current OSHA Excavation and Trench Safety Standards, and OSHA Health and Safety Standards for Excavations, 29 CFR Part 1929, or successor regulations.

According to chapter 296-155 of the Washington Administrative Code (WAC), it is our opinion that the soil encountered at the site is classified as Type C soils. We recommend that temporary, unsupported, open cut slopes shall be no steeper than 1.5 feet horizontal to 1.0 feet vertical

(1.5H:1V) in Type C soils. No heavy equipment should be allowed near the top of temporary cut slopes unless the cut slopes are adequately braced. Final (permanent) fill slopes should be graded to an angle of 2H:1V or flatter. Where unstable soils are encountered, flatter slopes may be required.

10.4 Construction Dewatering

Depending upon the time of the year construction takes place, there is potential for perched groundwater conditions to develop atop the native peat soils. Consequently, dewatering will be required for excavations extending below the zone of groundwater seepage to facilitate construction. Dewatering should be accomplished in advance of construction, as necessary, so that excavation and placement of foundations, pipe, pipe bedding and backfill materials are completed in relatively dry conditions. Dewatering should be performed such that the groundwater level around nearby existing structures is unaffected, as lowering the water level around existing structures could induce settlements. Design and implementation of dewatering systems should be the responsibility of the contractor.

We recommend that the contract documents require the Contractor to prepare and submit a dewatering plan for review and approval by the geotechnical engineer. Contractor shall also be made responsible for the dewatering system installation and maintenance. In addition, the Contractor should be responsible for control of surface water and should employ sloping, slope protection, ditching, sumps, and other measures as necessary.

10.5 Utility Excavation, Pipe Bedding and Trench Backfill

To provide suitable support and bedding for the pipe, we recommend the utilities be founded on suitable bedding material consisting of clean sand and/or sand & gravel mixture. To minimize trench subgrade disturbance during excavation, the excavator should use a smooth-edged bucket rather than a toothed bucket. All excavations should be wide enough to allow for compaction around the haunches of pipes and underground tanks. We recommend that utility trenching, installation, and backfilling conform to all applicable federal, state, and local regulations such as OSHA and WISHA for open excavations.

Pipe bedding and pipe zone materials shall conform to Section 9-03.12(3) of the *2018 WSDOT Standard Specifications.* Pipe bedding should provide a firm uniform cradle for support of the pipes. A minimum 4-inch thickness of bedding material beneath the pipe should be provided. Prior to installation of the pipe, the pipe bedding should be shaped to fit the lower part of the pipe exterior with reasonable closeness to provide uniform support along the pipe. Pipe bedding material should be used as pipe zone backfill and placed in layers and tamped around the pipes to obtain complete contact. To protect the pipe, bedding material should extend at least 6 inches above the top of the pipe.

In areas where the subgrade soils in the trench excavation consist of fine-grained soils, such as silt/clay, a geotextile separator fabric should be placed over the native soils prior to placement of the pipe bedding. We recommend that the geotextile meet the requirements of 2018 WSDOT Standard Specifications Section 9-33.2(1), Table 3 for Separation.

Placement of bedding material is particularly critical where maintenance of precise grades is essential. Backfill placed within the first 12 inches above utility lines should be compacted to at least 90% of the maximum dry density (ASTM D1557), such that the utility lines are not damaged during backfill placement and compaction. In addition, rock fragments greater than 1 inch in maximum dimension should be excluded from this first lift. The remainder of the utility excavations should be backfilled and compacted to 95% of the maximum dry density as determined by ASTM D1557. Compaction of backfill material should be accomplished with soils within ±2% of their optimum moisture content in order to achieve the minimum specified compaction levels recommended in this report. However, initial lift thickness could be increased to levels recommended by the manufacturer to protect utilities from damage by compacting equipment.

10.6 Suitability of the Onsite Soils as Reuse

The existing fill soils, consisting of silty and gravelly soils with organics and trash/debris, are unsuitable for reuse. The native peat and underlying silt/sand soils are also not suitable for re-use as fill. Suitable onsite soils shall be placed in maximum 8-inch lifts (loose) and compacted to at least 95% relative compaction (ASTM D1557) near its optimum moisture content. Compaction of these soils shall be performed within a range of ±2% of optimum moisture to achieve the proper degree of compaction.

10.7 Imported Fill Soils

For the purposes of this report, material placed under structures, or used as backfill against or above below-grade structures, is classified as structural engineered fill. Structural fill placed as backfill around structures should consist of clean, non-plastic, freely-draining sand and gravel, which is free of organic matter or other deleterious materials. Such materials should contain particles no larger than 4 inches in maximum dimension, with less than 7 percent fines (based on the ¾-inch fraction) as described in Section 9-03.14(1) of the 2018 WSDOT Standard Specifications. Structural fill/engineered fill placed below structures should consist of crushed surfacing base course, meeting the requirements of Section 9-03.9(3) of the 2018 WSDOT Standard Specifications.

Structural fill should be placed in loose, horizontal, lifts of not more than 8 inches in thickness and each lift should be compacted to at least 95% of its maximum dry density as determined by ASTM D1557 (Modified Proctor). At the time of placement, the moisture content of structural fill should be at or near optimum. The procedure required to achieve the specified minimum relative compaction depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and the soil moisture-density properties.

10.8 Foundations & Floor Slabs Supported by Rammed Aggregate Piers

Due to the presence of unsuitable fill and compressible native soils to depths of approximately 12 to 18 feet BGS, with seasonal perched groundwater conditions, along with the anticipated loading conditions for the proposed structure(s), ground improvement through over-excavation and replacement with import soils for support of a shallow foundation system is not considered feasible. An engineered Rammed Aggregate Pier (RAP) foundation system, also referred to as stone columns, shall be utilized for support of the new building structure foundations and floor slabs for this project. Engineered aggregate piers for soil reinforcement is a design-build foundation system constructed by a specialty geotechnical contractor. The design shall utilize the subsurface information and soil parameters presented in this report and structural loading provided by the Structural Engineer-of-Record.

Based on the presence of undocumented fill soils, soft/compressible peat and loose (potentially liquefiable) sands, coupled with perched groundwater conditions, a “displacement process” is recommended for installation of the piers. Displacement process consists of a hollow mandrel with an internal compaction surface which is driven into the ground using a powerful static down force augmented by dynamic vertical impact energy. After driving to the design depth, the hollow mandrel serves as a conduit for aggregate placement. As the mandrel is raised and re-driven downward, thin lifts of compacted aggregate are formed and compacted both vertically and laterally. The process is repeated until the RAP is constructed. The displacement system can typically be installed to significant depths below the groundwater table, while eliminating the need for casing.

The aggregate pier elements shall extend to a minimum depth of 18 feet BGS to the dense gravels or glacial till unit, or refusal, whichever is reached first. Practical refusal is typically considered as less than 1 foot of mandrel advancement in 60 seconds. The RAP system shall be designed to reinforce and stiffen the upper portion of the soil profile beneath the planned subgrade elevation. The upper reinforced zone shall support the building loads, as well as reduce the potential for surface manifestations and lessen settlements caused by liquefaction and minimize differential settlement. The RAP elements will typically help reduce pore pressure generation during a seismic event due to increased lateral stress and the higher permeability of the RAP element, thereby reducing the potential for triggering liquefaction.

Due to the presence of wet, soft soils, open graded aggregate shall be placed and rammed to stabilize the pier bottom and may serve as the initial pier lift. The bottom of the pier excavation shall be rammed prior to the placement of aggregate. The engineered aggregate pier design shall meet the following criteria:

- Allowable End Bearing Pressure for Footings supported by Aggregate Pier Reinforced Soils: **3,000 psf** (increase by 1/3 for short duration loadings)
- Estimated Total Long-Term Settlement for Footings: ≤ 1-inch
- Estimated Long-Term Differential Settlement of Adjacent Footings: ≤ ½-inch

The layout, diameter, and spacing of the aggregate pier elements shall be dictated by the structural loading. The installation of the aggregate pier reinforcement including a downward modulus test shall be completed in accordance with the engineered aggregate pier design specifications.

The aggregate pier reinforcement design shall utilize a minimum 16-inch drill diameter. Aggregate pier elements shall be founded on the native dense gravel unit and socketed a minimum 12 inches into the dense gravel unit. Typically groups of 3 or 4 closely spaced aggregate piers would be required beneath column footings depending on the loading. Continuous spread footings may be supported by aggregate pier elements placed 6.5 to 8-feet center-to-center spacing, depending on the loading conditions. Aggregate piers used to support the floor slab are typically placed on a grid pattern at about 10-12 feet center-to-center spacing. It shall be noted that these spacings are preliminary.

Due to the noted soft/loose soil conditions, the installer may want to consider rigid inclusions against aggregate piers for settlement limitation. The final design submittal from the installer should include detailed calculations to support the installation for settlement limitation.

The final aggregate pier design should be completed by a qualified design/build contractor. The engineered aggregate pier installer shall provide design submittal package including pier layout, drawings, specifications and calculations, sealed by a Washington State Professional Engineer, to the GER and the Structural Engineer for review at least 2 weeks prior to mobilization.

The center of each constructed aggregate pier element shall be within 6 inches of the design location. Foundation elements installed outside of the above tolerance and deemed unacceptable, shall be either rebuilt or other remedial measures shall be taken as approved by the aggregate pier system designer.

The downward modulus test shall consist of loading the pier element in increments to 150% of the design load while measuring deflection to verify the design parameters. The modulus test shall also incorporate a creep test at 115% of the design load. The installation and the modulus test will be conducted under the supervision of an experienced geotechnical engineer.

Prior to installing production piers, the aggregate pier designer shall establish the required energy output for the rammer and terminal rammer-blow deflection criterion for the ramming of each lift. Rammer energy output shall be confirmed by the installer prior to construction of production piers. Rammer-blow deflection monitoring shall be performed periodically on installed piers to confirm that terminal rammer-blow deflections meet the acceptance design criterion. The installer shall provide verifiable data to confirm that the rammer design to be used for constructing the aggregate piers develops nearly full passive lateral pressure into the soil adjacent to the aggregate pier for a distance of at least 5 feet horizontally beyond the edge of the pier.

The aggregate pier installer shall provide a full time Quality Control (QC) representative on-site during pier construction to observe the drilling and construction of all engineered aggregate piers and document the pier installation. QC observations shall include confirmation that all aggregate lifts located 2 feet or more above the bottom of the pier have been constructed to the design criteria, as established by the aggregate pier design Professional Engineer. A daily aggregate pier installation report shall be completed by the installer's QC representative each day of installation and shall be furnished by the installer to the general contractor and the GER.

A representative of the GER shall be retained by the Owner to conduct Quality Assurance (QA) services during pier installation. Representative of the GER shall monitor installation procedures and confirm that subsurface conditions across the installation area as revealed by the pier drilling are in substantial conformance with the findings of the geotechnical explorations.

Aggregate for the piers shall consist of material meeting the gradation and quality requirements of WSDOT Standard Specifications 9-03.9(3), Crushed Surfacing. Wet weather and fine grained soil conditions (subgrade soil condition) at the site may require that the aggregate contain less than 5% fines (percent passing the No. 200 sieve size). We recommend a suitable washed, open graded aggregate be used as initial lifts due to wet conditions and soft soils present at the bottom of the aggregate pier. The aggregate pier system designer and installer shall make the final determination of acceptable materials to be used in pier construction.

The general contractor is responsible for the layout of the building pad, footings, grade beams, floor slabs and staking of all aggregate pier locations prior to aggregate pier installation. The layout and pier staking shall be conducted by a licensed surveyor. Information shall include existing ground surface elevations (±3 inches) within 50 feet of each aggregate pier element. All above and below ground utilities shall be located, clearly marked, and relocated as necessary prior to installation of aggregate pier elements. Proper grading, moisture migration control, surface drainage and BMPs shall be implemented by the general contractor.

Foundations shall be constructed at minimum 30 inches below finished exterior grade, or as prescribed by the local jurisdiction, for frost protection. Foundation excavations to expose the tops of aggregate piers shall be made with care and shall be protected until concrete placement, with procedures and equipment best suited to (1) preventing softening of the soil matrix between and around aggregate piers prior to pouring structural concrete, and (2) achieving suitable contact between the dense, undisturbed aggregate piers and the concrete footing.

10.9 Lateral Earth Pressures for Below-Grade Wall Design

Equivalent fluid pressures for lateral earth pressure design of below-grade structures are shown in Table 3 below. We assume that the structural wall backfill is adequately drained to avoid saturation and introduction of hydrostatic pressures. For calculation of active pressures, we assume that the wall can deflect in order to develop an active condition. Use at-rest pressures for restrained or braced walls. The horizontal resultant force (pressure x H/2, where H is height of buried wall) should be applied at an H/3 distance from the base of the wall.

Table 3: Lateral Earth Pressures

Lateral Earth Pressure Condition	Recommend Pressure for Imported Granular Backfill
Active Pressure (Use when wall is permitted to rotate 0.1% to 0.2% of wall height for granular backfill)	40 psf/foot
At-Rest Pressure	62 psf/foot

Should surcharge loads be closer than one-half of the wall height (horizontal distance) to the edge of the below-grade wall, increase the design wall pressure by *q*/2 over the whole area of the retaining wall. In this expression, *q* is the surface surcharge load in psf. GNN should review the actual surcharge loading to confirm that the appropriate design values are considered. The horizontal surcharge resultant force (pressure x H, where H is total height of buried wall) should be applied at an H/2 distance from the base of the wall.

<u>Dynamic Seismic Lateral Earth Pressure (Combination of Static & Seismic):</u> For the seismic loading condition, we have used the Seed & Whitman method based on Mononobe-Okabe equation with a horizontal coefficient K_h equal to 0.2425g (based on PGA of 0.485g). This method combines seismic lateral pressure with static active pressure for the total lateral pressure. We recommend a total lateral seismic earth pressure of $30H^2$ (where H is total height of buried wall).

The seismic loading uses the inverted triangular distribution with the resultant force located at 0.6H. This lateral pressure assumes horizontal backfill or level crest condition (β = 0°).

Drainage: Below-grade retaining structures should include adequate back drainage to avoid build-up of hydrostatic pressures. Positive drainage for retaining walls should consist of a vertical layer of permeable material (chimney drain), such as a pea gravel or crushed rock (typically ¼- to ¾-inch crushed), at least 18 inches thick, positioned between the retaining wall and the backfill. We recommend installing a non-woven filter fabric such as Mirafi® 140N between the drainage material and the general backfill to prevent fines from migrating into the drainage material.

10.10 Subgrade Protection

The degree to which construction grading problems develop is expected to be dependent, in part, on the time of year that construction proceeds and the precautions which are taken by the contractor to protect the subgrade. The fine-grained soils currently present on site are considered to be moisture and disturbance sensitive due to their fines content and may become unstable (pumping) if allowed to increase in moisture content and are disturbed (rutted) by construction traffic if wet. If necessary, the construction access road should be covered with a layer of gravel or quarry spalls course. The soils are also susceptible to erosion in the presence of moving water. The soils shall be stabilized to minimize the potential of erosion into the foundation excavation. The site shall be graded to prevent water from ponding within construction areas and/or flowing into excavations. Accumulated water must be removed immediately along with any unstable soil. Foundation concrete shall be placed and excavations backfilled as soon as possible to protect the bearing grade. We further recommend that soils that become unstable are to be either:

- Removed and replaced with structural compacted gravel fill, or
- Mechanically stabilized with a coarse crushed aggregate (possibly underlain with a geotextile) and compacted into the subgrade.

10.11 Surface Drainage

With respect to surface water drainage, we recommend that the ground surface be sloped to drain away from the structure. Final exterior site grades shall promote free and positive drainage from the building areas. Water shall not be allowed to pond or to collect adjacent to foundations or within the immediate building area. We recommend that a gradient of at least 5% for a minimum distance of 10 feet from the building perimeter be provided, except in paved locations. In paved areas, a minimum gradient of 1% should be provided unless provisions are included for collection/disposal of surface water adjacent to the structure. Catch basins, drainage swales, or other drainage facilities should be aptly located. All surface water such as that coming from roof downspouts and catch basins be collected in tight drain lines and carried to a suitable discharge point, such as a storm drain system. Surface water and downspout water should not discharge into a perforated or slotted subdrain, nor should such water discharge onto the ground surface adjacent to the building. Cleanouts should be provided at convenient locations along all drain lines.

10.12 Wet Weather Conditions

The near-surface site soils are fine-grained and sensitive to moisture during handling and compaction. Proceeding with site earthwork operations soils during wet weather could add project costs and/or delays. The stability of exposed soils may rapidly deteriorate due to a change in moisture content. Therefore, if at all possible, complete site clearing, preparation, and earthwork during periods of warm/dry weather when soil moisture can be controlled by aeration. During or subsequent to wet weather, drying/compacting the on-site soils will be difficult. It may be necessary to amend the on-site soils or import granular materials for use as structural fill. If earthwork takes place in wet weather, the following recommendations should be followed:

- Fill material should consist of clean, granular soil, and not more than 3% fines should pass the No. 200 sieve. Fines should be non-plastic. These soils would have to be imported to the site.
- Earthwork should be accomplished in small sections and carried through to completion to reduce exposure to wet weather. Soils that becomes too wet for compaction should be removed and replaced with clean, granular material.
- The construction area ground surface should be sloped and sealed to reduce water infiltration, to promote rapid runoff, and to prevent water ponding.
- To prevent soil disturbance, the size or type of equipment may have to be limited.
- Work areas and stockpiles should be covered with plastic. Straw bales, straw wattles, geotextile silt fences, and other measures should be used as appropriate to control soil erosion.
- Excavation and fill placement should be observed on a full-time basis by a representative of GER to determine that unsuitable materials are removed and that suitable compaction and site drainage is achieved.

11.0 REFERENCES

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Porter, S.C. (1976). Pleistocene glaciation in the southern part of the North Cascade Range, Washington. Geological Society of America Bulletin 87, 61–75.

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U.S. Department of the Interior, U.S. Geological Survey (USGS), (1961). Snoqualmie Pass Quadrangle, Washington, 15 Minute Series (Topographic). AMS 1778 IV-Series V791.

Washington State Department of Natural Resources (DNR), Washington Division of Geology and Earth Resources, on-line mapping tool, https://fortress.wa.gov/dnr/protectiongis/geology/

12.0 CONTINUING GEOTECHNICAL SERVICES

GNN recommends that the Client should maintain an adequate program of geotechnical consultation, construction monitoring, and soils testing during the final design and construction phases to monitor compliance with GNN's geotechnical recommendations. Maintaining GNN as the geotechnical consultant from beginning to end of the project will provide continuity of services. If GN Northern, Inc. is not retained by the owner/developer and/or the contractor to provide the recommended geotechnical inspections/observations and testing services, the geotechnical engineering firm or testing/inspection firm providing tests and observations shall assume the role and responsibilities of Geotechnical Engineer-of-Record.

GNN can provide construction monitoring and testing as additional services. The costs of these services are not included in our present fee arrangement, but can be obtained from our office. The recommended construction monitoring and testing includes, but is not necessarily limited to, the following:

- Consultation during the design stages of the project.
- Review of the grading and drainage plans to monitor compliance and proper implementation of the recommendations in GNN's Report.
- Observation and quality control testing during site preparation, grading, and placement of engineered fill as required by the local building ordinances.
- Geotechnical engineering consultation as needed during construction

13.0 LIMITATIONS OF THE GEOTECHNICAL SITE INVESTIGATION REPORT

This GEOTECHNICAL SITE INVESTIGATION REPORT ("Report") was prepared for the exclusive use of the Client. GN Northern, Inc.'s (GNN) findings, conclusions and recommendations in this Report are based on selected points of field exploration, and GNN's understanding of the proposed project at the time the Report is prepared. Furthermore, GNN's findings and recommendations are based on the assumption that soil, rock and/or groundwater conditions do not vary significantly from those found at specific exploratory locations at the project site. Variations in soil, bedrock and/or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until during or after construction. Variations in soil, bedrock and groundwater may require additional studies, consultation, and revisions to GNN's recommendations in the Report.

In many cases the scope of geotechnical exploration and the test locations are selected by others without consultation from the geotechnical engineer/consultant. GNN assumes no responsibility and, by preparing this Report, does not impliedly or expressly validate the scope of exploration and the test locations selected by others.

This Report's findings are valid as of the issued date of this Report. However, changes in conditions of the subject property or adjoining properties can occur due to passage of time, natural processes, or works of man. In addition, applicable building standards/codes may change over time. Accordingly, findings, conclusions, and recommendations of this Report may be invalidated, wholly or partially, by changes outside of GNN's control. Therefore, this Report is subject to review and shall not be relied upon after a period of **one (1) year** from the issued date of the Report.

In the event that any changes in the nature, design, or location of structures are planned, the findings, conclusions and recommendations contained in this Report shall not be considered valid unless the changes are reviewed by GNN and the findings, conclusions, and recommendations of this Report are modified or verified in writing.

This Report is issued with the understanding that the owner or the owner's representative has the responsibility to bring the findings, conclusions, and recommendations contained herein to the attention of the architect and design professional(s) for the project so that they are incorporated into the plans and construction specifications, and any follow-up addendum for the project. The owner or the owner's representative also has the responsibility to verify that the general contractor and all subcontractors follow such recommendations during construction. It is further understood that the owner or the owner's representative is responsible for submittal of this Report to the appropriate governing agencies. The foregoing notwithstanding, no party other than the Client shall have any right to rely on this Report and GNN shall have no liability to any third party who claims injury due to reliance upon this Report, which is prepared exclusively for Client's use and reliance.

GNN has provided geotechnical services in accordance with generally accepted geotechnical engineering practices in this locality at this time. GNN expressly disclaims all warranties and guarantees, express or implied.

Client shall provide GNN an opportunity to review the final design and specifications so that earthwork, drainage and foundation recommendations may be properly interpreted and implemented in the design and specifications. If GNN is not accorded the review opportunity, GNN shall have no responsibility for misinterpretation of GNN's recommendations.

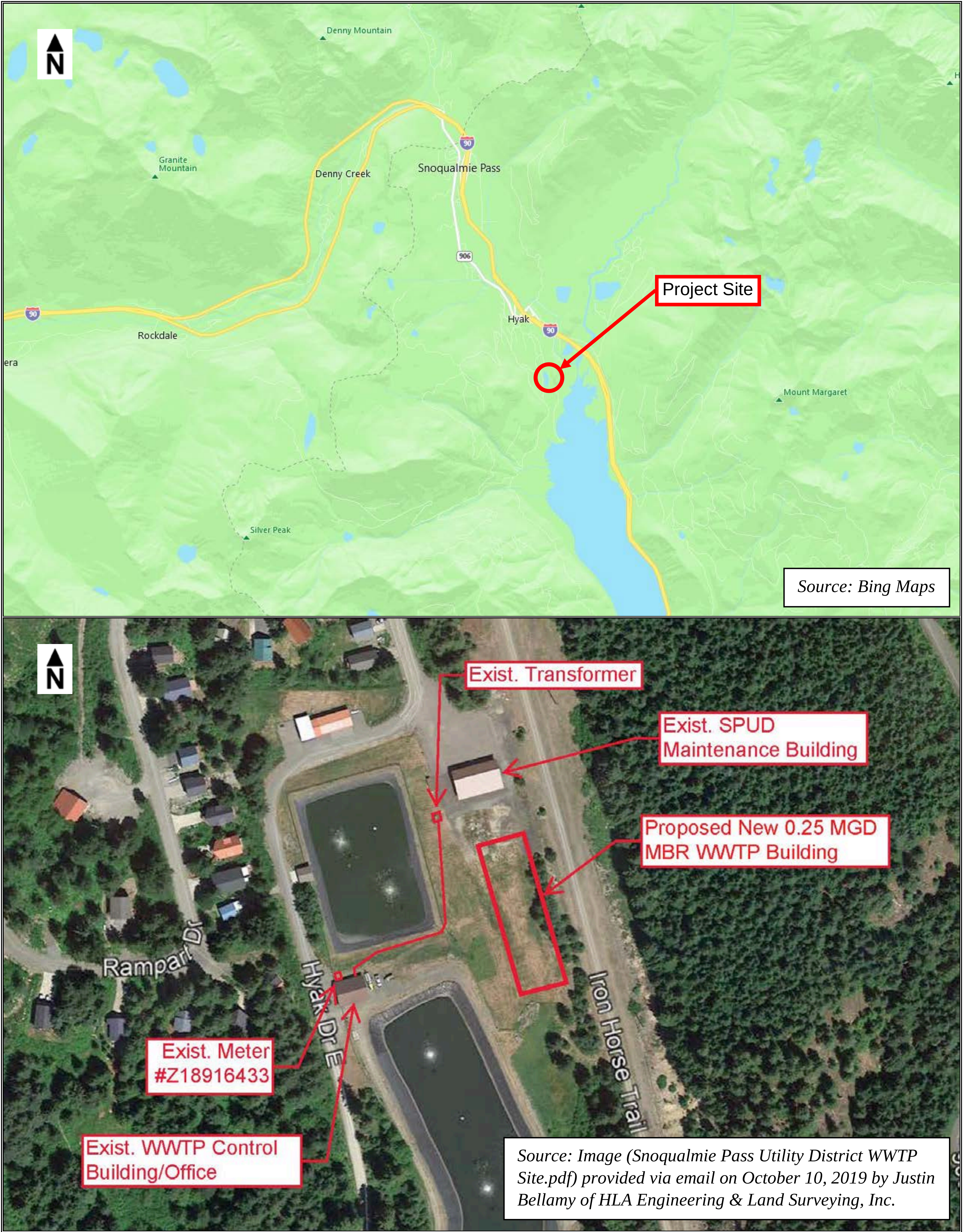
Although GNN can provide environmental assessment and investigation services for an additional cost, the current scope of GNN's services does not include an environmental assessment or an investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

APPENDICES

Appendix I

Vicinity Map (Figure 1)

Site Exploration Map (Figure 2)



Source: Bing Maps

Source: Image (Snoqualmie Pass Utility District WWTP Site.pdf) provided via email on October 10, 2019 by Justin Bellamy of HLA Engineering & Land Surveying, Inc.

FIGURE 1: VICINITY MAP **PROJECT NO. 219-1154**

LEGEND

🟥 = Exploratory test pit

⊕ = Exploratory borehole

NOTE

Image dated October 31, 2019 provided by HLA Engineering and Land Surveying, Inc.

FIGURE 2: SITE EXPLORATION MAP

PROJECT NO. 219-1154

Appendix II

Exploratory Boring & Test-Pit Logs

Key Chart (for Soil Classification)

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BORING NUMBER B-6

PAGE 1 OF 1

CLIENT HLA Engineering and Land Surveying, Inc.

PROJECT NAME SPUD Wastewater Treatment Plant Improvements

PROJECT NUMBER 219-1154

PROJECT LOCATION Hyak Drive E, Hyak, Snoqualmie Pass, WA

DATE STARTED 10/16/19 **COMPLETED** 10/16/19

GROUND ELEVATION 2558 ft **HOLE SIZE** 6 inches

DRILLING CONTRACTOR Holt Services Inc.

GROUND WATER LEVELS:

DRILLING METHOD Terra Sonic TSi 150 Compact Crawler

AT TIME OF DRILLING ---

LOGGED BY MYM **CHECKED BY** IM

AT END OF DRILLING ---

NOTES GPS Coords.: 47°23'14.82"N, 121°23'29.99"W

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0				ML		FILL: SILT WITH GRAVEL, (ML) orangeish brown, subrounded to subangular, moist, appears loose to medium dense	
						- becomes dark grayish blue, with a significant amount of wood debris	
5	SPT	3-5-11 (16)					
6.5							2551.5
7.0						ASH, tan, sandy	2551.0
	GB		MC = 30% LL = 32 PL = 28 Fines = 95%			PEAT, brown, moist to wet, very soft, with low plasticity silt, significant woody remnants and fibers, with pockets of ash and poorly graded sand (APPARENT NATIVE)	
10	SPT	0-0-0 (0)					
11.5	GB		MC = 63% Fines = 90%	SM		INTERBEDDED SILT (ML), SANDY SILT (ML) AND SILTY SAND, (SM) blueish gray, moist, loose, with fine to coarse grained sand	2546.5
15	SPT	2-2-3 (5)	MC = 20% Fines = 60%				
17.0				GM		SILTY GRAVEL WITH SAND, (GM) blueish gray, subangular, moist, medium dense, with fine to coarse grained sand	2541.0
20	SPT	7-10-19 (29)	Fines = 14%				
25	SPT	8-18-50/4"					
	GB		MC = 10% Fines = 50%				
27.0	GB		MC = 5% Fines = 34%	GM		SILTY GRAVEL, (GM) blueish gray, subangular, dry to damp, very dense, friable to silty sand [GLACIAL TILL]	2531.0
30	SPT	50/3"					
30.3							2527.8

- Referenced elevations are approximate and based on topography shown on survey plan provided by HLA Engineering and Land Surveying, Inc.

Bottom of borehole at 30.3 feet.

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BORING NUMBER B-7

CLIENT HLA Engineering and Land Surveying, Inc.

PROJECT NAME SPUD Wastewater Treatment Plant Improvements

PROJECT NUMBER 219-1154

PROJECT LOCATION Hyak Drive E, Hyak, Snoqualmie Pass, WA

DATE STARTED 10/16/19 **COMPLETED** 10/16/19

GROUND ELEVATION 2559.5 ft **HOLE SIZE** 6 inches

DRILLING CONTRACTOR Holt Services Inc.

GROUND WATER LEVELS:

DRILLING METHOD Terra Sonic TSi 150 Compact Crawler

AT TIME OF DRILLING ---

LOGGED BY MYM **CHECKED BY** IM

AT END OF DRILLING ---

NOTES GPS Coords.: 47°23'13.87"N, 121°23'28.42"W

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0				ML		FILL: SILT WITH GRAVEL, (ML) dark brown, moist, trace cobbles, with trash
					3.0	2556.5
				GM		FILL: SILTY GRAVEL, (GM) blueish gray, angular, wet, appears medium dense, organic odor, with cobbles
5	SPT	18-5-2 (7)				
					6.0	2553.5
	GB		Fines = 81%			PEAT, brown, moist to wet, very soft, with low plasticity silt, significant woody remnants and fibers, with pockets of ash and poorly graded sand (APPARENT NATIVE) - ~12" ash layer
					9.0	2550.5
				SM		INTERBEDDED SILT (ML), SANDY SILT (ML) AND SILTY SAND, (SM) blueish gray, moist to wet, loose, with fine to coarse grained sand
10	SPT	2-8-6 (14)				
					12.5	2547.0
				SM		SILTY SAND WITH GRAVEL, (SM) blueish gray, moist to wet, dense
	GB		MC = 9% Fines = 33%			
15	SPT	12-28-6 (34)				
					17.0	2542.5
				GM		SILTY GRAVEL, (GM) blueish gray, subangular, dry to damp, very dense, friable to silty sand [GLACIAL TILL]
20	SPT	50/4"				
					20.3	2539.2

- Referenced elevations are approximate and based on topography shown on survey plan provided by HLA Engineering and Land Surveying, Inc.

Bottom of borehole at 20.3 feet.

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BORING NUMBER B-8

PAGE 1 OF 1

CLIENT HLA Engineering and Land Surveying, Inc.
PROJECT NAME SPUD Wastewater Treatment Plant Improvements
PROJECT NUMBER 219-1154
PROJECT LOCATION Hyak Drive E, Hyak, Snoqualmie Pass, WA
DATE STARTED 10/16/19 **COMPLETED** 10/16/19
GROUND ELEVATION 2559 ft **HOLE SIZE** 6 inches
DRILLING CONTRACTOR Holt Services Inc.
GROUND WATER LEVELS:
DRILLING METHOD Terra Sonic TSi 150 Compact Crawler
AT TIME OF DRILLING ---
LOGGED BY MYM **CHECKED BY** IM
AT END OF DRILLING ---
NOTES GPS Coords.: 47°23'16.14"N, 121°23'29.33"W
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG		MATERIAL DESCRIPTION	
0				ML			FILL: SILT WITH GRAVEL, (ML) dark brown, moist, trace cobbles, with trash	
						4.0		2555.0
5	SPT	8-15-6 (21)		GM			FILL: SILTY GRAVEL, (GM) blueish gray, angular, wet, appears medium dense, organic odor, with cobbles	
						6.3		2552.8
	GB		Fines = 100%				PEAT, brown, moist to wet, very soft, with low plasticity silt, significant woody remnants and fibers, with pockets of ash and poorly graded sand (APPARENT NATIVE) - ash layer	
10	SPT	2-6-9 (15)						
						13.0		2546.0
15	SPT	3-2-4 (6)		SM			INTERBEDDED SILT (ML), SANDY SILT (ML) / SILTY SAND, (SM) blueish gray, moist, loose, with fine to coarse grained sand	
						18.5		2540.5
20	GB		MC = 11% LL = 20 PL = 13 Fines = 29%	SM			SILTY SAND WITH GRAVEL, (SM) blueish gray, moist to wet, very dense, some cobbles	
	SPT	13-10-50/4"						
						21.3		2537.7

- Referenced elevations are approximate and based on topography shown on survey plan provided by HLA Engineering and Land Surveying, Inc.

Bottom of borehole at 21.3 feet.

GN Northern Inc.
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Spokane Valley, WA, 99206
Telephone: (509) 248-9798
Fax: (509) 248-4220

TEST PIT NUMBER TP-3

PAGE 1 OF 1

CLIENT HLA Engineering and Land Surveying, Inc.

PROJECT NAME SPUD Wastewater Treatment Plant Improvements

PROJECT NUMBER 219-1154

PROJECT LOCATION Hyak Drive E, Hyak, Snoqualmie Pass, WA

DATE STARTED 10/15/19 **COMPLETED** 10/15/19

GROUND ELEVATION 2558.5 ft **TEST PIT SIZE** 30 x 72 inches

EXCAVATION CONTRACTOR Snoqualmie Pass Utility District

GROUND WATER LEVELS:

EXCAVATION METHOD John Deere 410C Backhoe

▽ **AT TIME OF EXCAVATION** 7.50 ft / Elev 2551.00 ft

LOGGED BY MBB **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES GPS Coords.: 47°23'15.66"N, 121°23'29.79"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG		MATERIAL DESCRIPTION	
0			GP-GM		1.0	FILL: POORLY GRADED GRAVEL WITH SILT AND SAND, (GP-GM) grayish brown, subrounded to subangular, moist, appears medium dense	2557.5
5	GB	MC = 18% Fines = 27%	SM		5.5	FILL: SILTY SAND WITH GRAVEL, (SM) dark brown, moist, appears medium dense, trace cobbles, with trash/debris (fabric, brick, wire, wood) - becomes dark blueish gray, with organics	2553.0
			GM		8.0	FILL: SILTY GRAVEL, (GM) blueish gray, angular, wet, appears medium dense, organic odor, with cobbles - appears medium dense, with boulders ▽ - railroad tie	2550.5
10					11.0	PEAT, brown, moist to wet, very soft, with low plasticity silt, significant woody remnants and fibers, with pockets of ash and poorly graded sand (APPARENT NATIVE)	2547.5

- Groundwater encountered at ~7.5' BGS at time of excavation
- Referenced elevations are approximate and based on topography shown on survey plan provided by HLA Engineering and Land Surveying, Inc.

Bottom of test pit at 11.0 feet.

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TEST PIT NUMBER TP-4

PAGE 1 OF 1

CLIENT HLA Engineering and Land Surveying, Inc.

PROJECT NAME SPUD Wastewater Treatment Plant Improvements

PROJECT NUMBER 219-1154

PROJECT LOCATION Hyak Drive E, Hyak, Snoqualmie Pass, WA

DATE STARTED 10/16/19 **COMPLETED** 10/16/19

GROUND ELEVATION 2559 ft **TEST PIT SIZE** 30 x 72 inches

EXCAVATION CONTRACTOR Snoqualmie Pass Utility District

GROUND WATER LEVELS:

EXCAVATION METHOD John Deere 410C Backhoe

AT TIME OF EXCAVATION ---

LOGGED BY MYM **CHECKED BY** IM

AT END OF EXCAVATION ---

NOTES GPS Coords.: 47°23'14.14"N, 121°23'29.44"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0		ML	▩	FILL: SILT WITH GRAVEL, (ML) dark brown, moist, appears medium dense, trace cobbles, with trash/debris (fabric, brick, wire, wood)	
4.0					2555.0
5		GM	▩	FILL: SILTY GRAVEL, (GM) blueish gray, angular, wet, appears medium dense, organic odor, with cobbles, some boulders	
6.5					2552.5
				PEAT, brown, moist to wet, very soft, with low plasticity silt, significant woody remnants and fibers, with pockets of ash and poorly graded sand (APPARENT NATIVE) - ~3" to 4" thick ash layer	
10 / 10.0					2549.0

- Referenced elevations are approximate and based on topography shown on survey plan provided by HLA Engineering and Land Surveying, Inc.

Bottom of test pit at 10.0 feet.

Northern, Inc.
Kennewick, Yakima, Spokane, Hermiston (OR)

KEY CHART

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COARSE-GRAINED SOILS			FINE-GRAINED SOILS		
DENSITY	**N (BLOWS/FT)**	**FIELD TEST**	**CONSISTENCY**	**N (BLOWS/FT)**	**FIELD TEST**
Very Loose	0 – 4	Easily penetrated with ½-inch reinforcing rod pushed by hand	Very Soft	0 – 2	Easily penetrated several inches by thumb
Loose	4 – 10	Difficult to penetrate with ½-inch reinforcing rod pushed by hand	Soft	2 – 4	Easily penetrated one inch by thumb
Medium -Dense	10 – 30	Easily penetrated with ½-inch rod driven with a 5-lb hammer	Medium-Stiff	4 – 8	Penetrated over ½-inch by thumb with moderate effort
Dense	30 – 50	Difficult to penetrate with ½-inch rod driven with a 5-lb hammer	Stiff	8 – 15	Indented about ½-inch by thumb but penetrated with great effort
Very Dense	> 50	penetrated only a few inches with ½-inch rod driven with a 5-lb hammer	Very Stiff	15 – 30	Readily indented by thumb
			Hard	> 30	Indented with difficulty by thumbnail

USCS SOIL CLASSIFICATION

MAJOR DIVISIONS			GROUP DESCRIPTION		
Coarse-Grained Soils <50% passes #200 sieve	**Gravel** and **Gravelly Soils** <50% coarse fraction passes #4 sieve	**Gravel** (with little or no fines)	▨	GW	Well-graded **Gravel**
			▨	GP	Poorly Graded **Gravel**
		Gravel (with >12% fines)	▨	GM	Silty **Gravel**
			▨	GC	Clayey **Gravel**
	Sand and **Sandy Soils** >50% coarse fraction passes #4 sieve	**Sand** (with little or no fines)	▨	SW	Well-graded **Sand**
			▨	SP	Poorly graded **Sand**
		Sand (with >12% fines)	▨	SM	Silty **Sand**
			▨	SC	Clayey **Sand**
Fine-Grained Soils >50% passes #200 sieve	**Silt** and **Clay** Liquid Limit < 50		▨	ML	**Silt**
			▨	CL	Lean **Clay**
			▨	OL	Organic **Silt** and **Clay** (low plasticity)
	Silt and **Clay** Liquid Limit > 50		▨	MH	Inorganic **Silt**
			▨	CH	Inorganic **Clay**
			▨	OH	Organic **Clay** and **Silt** (med. to high plasticity)
Highly Organic Soils			▨	PT	**Peat** ▨ **Top Soil**

LOG SYMBOLS

⧗	2S	2" OD Split Spoon (SPT)
■	3S	3" OD Split Spoon
⧄	NS	Non-Standard Split Spoon
◇	ST	Shelby Tube
◫	CR	Core Run
⊠	BG	Bag Sample
⧓	TV	Torvane Reading
⌶	PP	Penetrometer Reading
☐	NR	No Recovery
▽ ▼	GW	Groundwater Table

MODIFIERS

DESCRIPTION	RANGE
Trace	<5%
Little	5% – 12%
Some	>12%

MOISTURE CONTENT

DESCRIPTION	FIELD OBSERVATION
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but not visible water
Wet	Visible free water

SOIL CLASSIFICATION INCLUDES

1. Group Name
2. Group Symbol
3. Color
4. Moisture content
5. Density / consistency
6. Cementation
7. Particle size (if applicable)
8. Odor (if present)
9. Comments

MAJOR DIVISIONS WITH GRAIN SIZE

SIEVE SIZE

12" 3" 3/4" 4 10 40 200

GRAIN SIZE (INCHES)

12 3 0.75 0.19 0.079 0.0171 0.0029

Boulders	Cobbles	Gravel		Sand			Silt and Clay
		Coarse	Fine	Coarse	Medium	Fine	

Conditions shown on boring and testpit logs represent our observations at the time and location of the fieldwork, modifications based on lab test, analysis, and geological and engineering judgment. These conditions may not exist at other times and locations, even in close proximity thereof. This information was gathered as part of our investigation, and we are not responsible for any use or interpretation of the information by others.

Appendix III

Laboratory Testing Results

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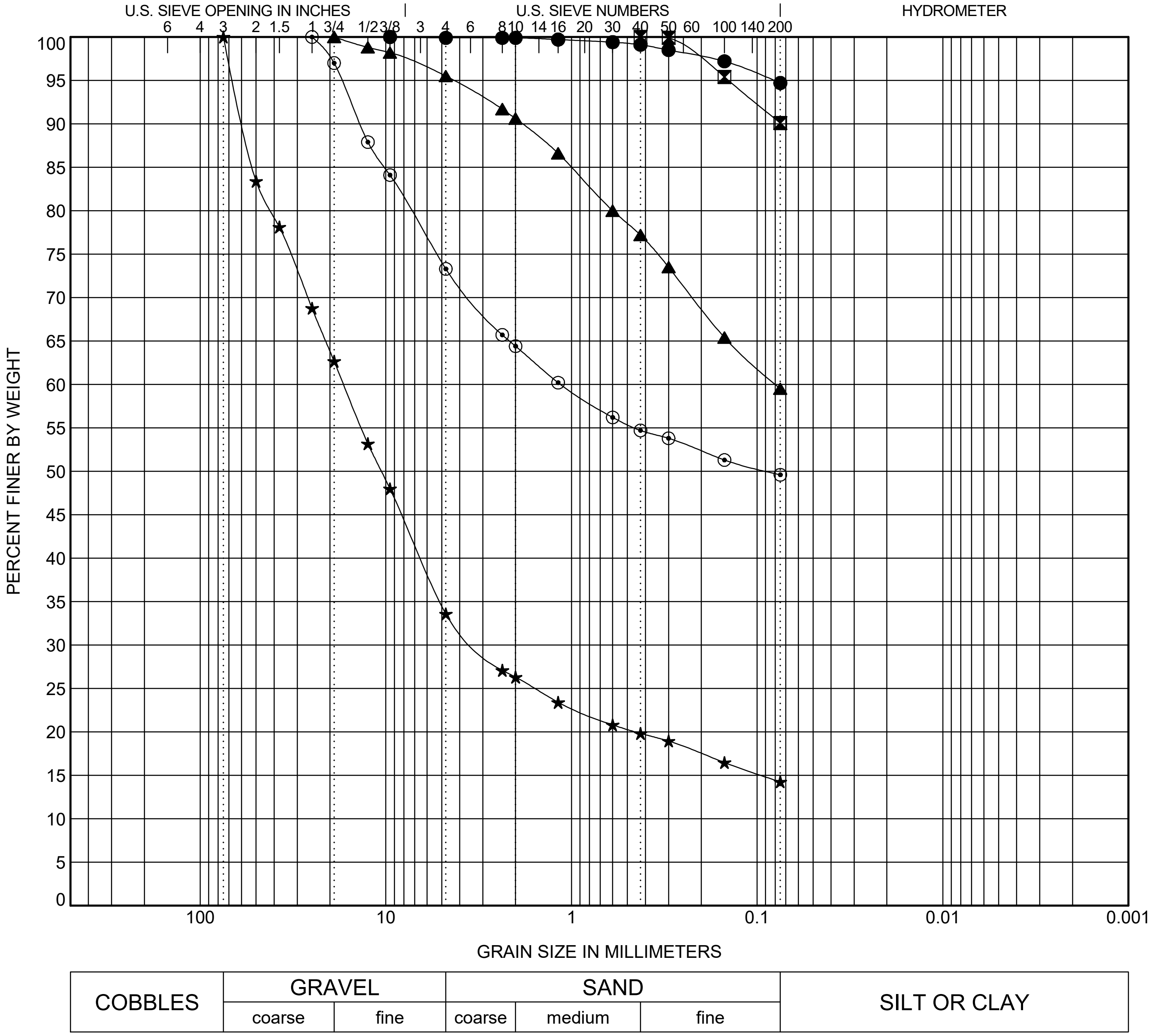
GRAIN SIZE DISTRIBUTION

CLIENT HLA Engineering and Land Surveying, Inc.

PROJECT NAME SPUD Wastewater Treatment Plant Improvements

PROJECT NUMBER 219-1154

PROJECT LOCATION Hyak Drive E, Hyak, Snoqualmie Pass, WA



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-6**	**8.0**	**ORGANIC SILT (ML)**	**32**	**28**	**4**		
☒	**B-6**	**11.5**	**SILT (ML)**					
▲	**B-6**	**15.0**	**SANDY SILT (ML)**					
★	**B-6**	**20.0**	**SILTY GRAVEL WITH SAND (GM)**					
⊙	**B-6**	**26.5**	**GRAVELLY SILT WITH SAND (ML)**					

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-6**	**8.0**	**9.5**				**0.1**	**5.2**	**94.7**	
☒	**B-6**	**11.5**	**0.425**				**0.0**	**9.9**	**90.1**	
▲	**B-6**	**15.0**	**19**	**0.08**			**4.5**	**36.0**	**59.5**	
★	**B-6**	**20.0**	**75**	**16.889**	**3.229**		**66.4**	**19.3**	**14.3**	
⊙	**B-6**	**26.5**	**25**	**1.141**			**26.7**	**23.7**	**49.6**	

GRAIN SIZE - TEMPLATE_JESSE.GDT - 11/1/19 16:41 - C:\USERS\GN NORTHERN\DROPBOX\5-ACTIVE PROJECTS\219-1154 SNOQUALMIE PASS UTILITY DISTRICT SITES\219-1154 LOGS.GPJ

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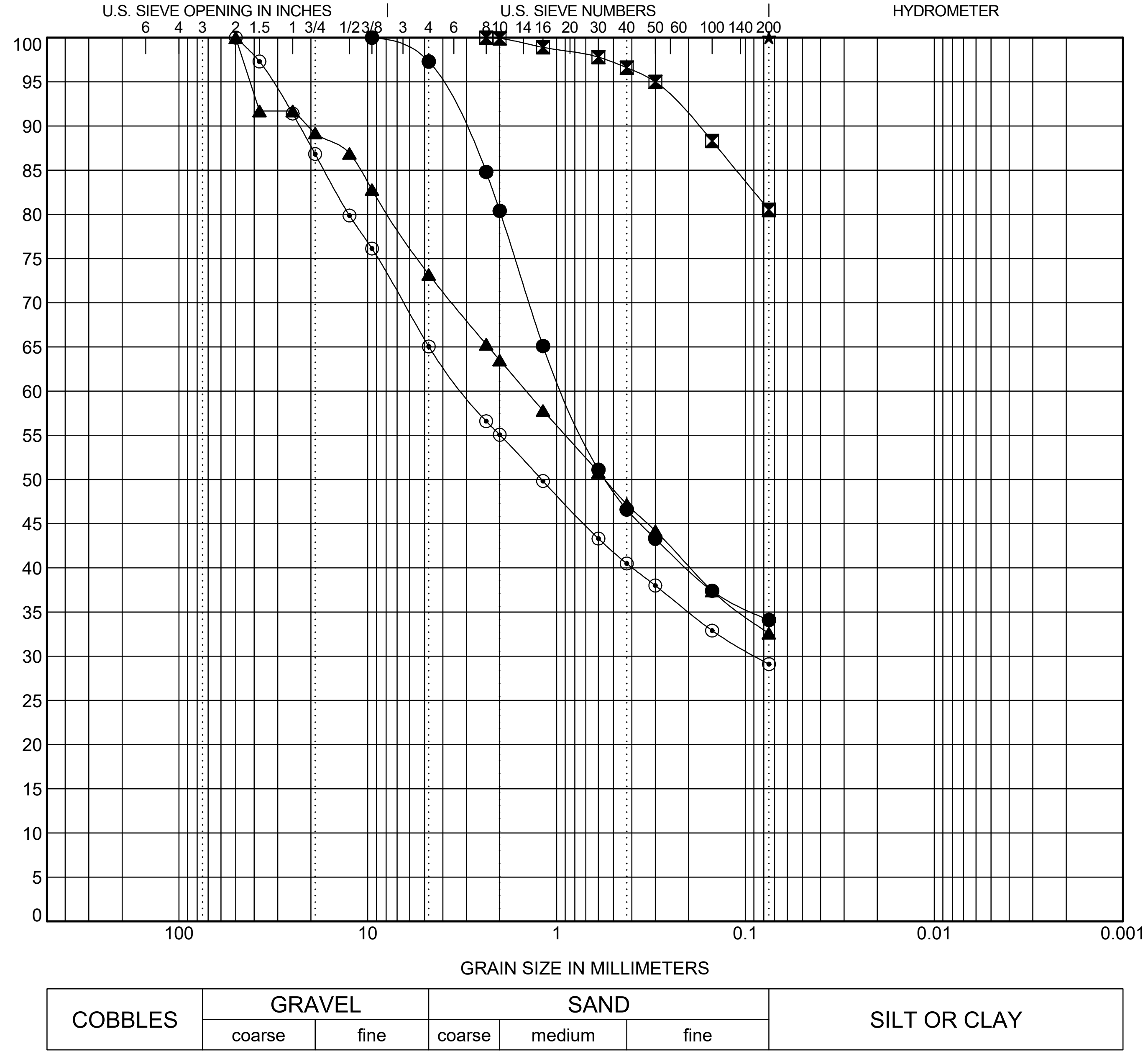
GRAIN SIZE DISTRIBUTION

CLIENT HLA Engineering and Land Surveying, Inc.

PROJECT NAME SPUD Wastewater Treatment Plant Improvements

PROJECT NUMBER 219-1154

PROJECT LOCATION Hyak Drive E, Hyak, Snoqualmie Pass, WA



BOREHOLE		DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-6**	**27.0**	**SILTY SAND (SM)**					
☒	**B-7**	**6.5**	**PEAT / SILT WITH SAND (ML)**					
▲	**B-7**	**13.0**	**SILTY SAND WITH GRAVEL (SM)**					
★	**B-8**	**7.0**	**PEAT / SILT (ML)**					
⊙	**B-8**	**19.0**	**CLAYEY SAND WITH GRAVEL (SC)**	**20**	**13**	**7**		

BOREHOLE		DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-6**	**27.0**	**9.5**	**0.922**			**2.7**	**63.2**	**34.1**	
☒	**B-7**	**6.5**	**2.36**				**0.0**	**19.5**	**80.5**	
▲	**B-7**	**13.0**	**50**	**1.447**			**26.8**	**40.6**	**32.6**	
★	**B-8**	**7.0**	**0.075**				**0.0**	**0.0**	**100.0**	
⊙	**B-8**	**19.0**	**50**	**3.127**	**0.088**		**34.9**	**36.0**	**29.1**	

Northwest Agricultural Consultants

2545 W Falls Avenue
Kennewick, WA 99336
509.783.7450
www.nwag.com
lab@nwag.com

PAP-Accredited
NAPT

GN NORTHERN INC
722 N. 16TH AVE #31
YAKIMA, WA 98902

Report: 49941-1-1
Date: October 26, 2019
Project No: 219-1154
Project Name:

Sample ID	Organic Matter
B-7 @ 8.0’	23.70%
B-8 @ 9.0’	53.17%
TP-3 @ 10.0’	66.07%
Method	**ASTM D2974**

Appendix IV

Site & Exploration Photographs

View of site conditions looking north

View of site conditions along fence on east side of existing lagoons, looking south

View of site conditions looking towards existing WWTP facility

View of conditions looking south

View of conditions looking west

Stockpiles of fill soils and construction debris in northern portion of the proposed improvements area

PLATE 1: SITE & EXPLORATION PHOTOGRAPHS **PROJECT NO. 219-1154**

Exposed soil profile within test-pit TP-3

Exposed soil profile within test-pit TP-4

Sonic drilling recovery from boring B-6

Sonic drilling recovery from boring B-6

Sonic drilling recovery from boring B-6

Sonic drilling recovery from boring B-6

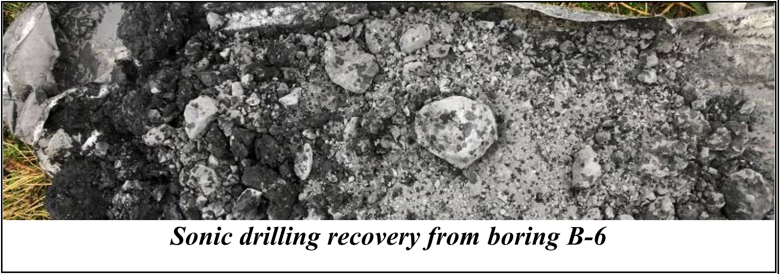
Sonic drilling recovery from boring B-6

Sonic drilling recovery from boring B-6

Sonic drilling recovery from boring B-6

Sonic drilling recovery from boring B-6

Sonic drilling recovery from boring B-6



Sonic drilling recovery from boring B-6

PLATE 2: SITE & EXPLORATION PHOTOGRAPHS **PROJECT NO. 219-1154**

Sonic drilling recovery from boring B-7

Sonic drilling recovery from boring B-7

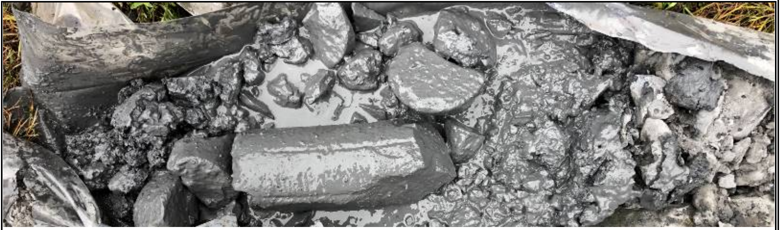
Sonic drilling recovery from boring B-7

Sonic drilling recovery from boring B-7

Sonic drilling recovery from boring B-7

Sonic drilling recovery from boring B-7

Sonic drilling recovery from boring B-7



Sonic drilling recovery from boring B-7

Sonic drilling recovery from boring B-8

Sonic drilling recovery from boring B-8

Sonic drilling recovery from boring B-8

Sonic drilling recovery from boring B-8

Sonic drilling recovery from boring B-8

Sonic drilling recovery from boring B-8

PLATE 3: SITE & EXPLORATION PHOTOGRAPHS **PROJECT NO. 219-1154**

Appendix V

Refraction Microtremor (ReMi) Survey Report

Global Geophysics

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

October 1, 2019 Our Ref.: 109-0926-001.000

GN Northern Inc.
722 North 16th Avenue, Suite 31
Yakima, WA 98902

Attention: Mr. Imran Magsi

RE: REPORT ON THE GEOPHYSICAL SURVEY AT WASTE WATER TREATMENT PLANT, SNOQUALMIE PASS, WA

Dear Mr. Magsi:

Global Geophysics conducted a refraction microtremor (ReMi) surveys at Waste Water Treatment Plant, Snoqualmie Pass, WA on September 26, 2019. The proposed objective of the geophysical investigation was to determine the average s-wave velocity of the 100 ft soil column below the ground surface.

METHODOLOGY AND INSTRUMENTATION

The ReMi method determines variations in surface wave velocities with increasing distances and wavelengths. The data from these measurements are used to model the shear wave velocities of the subsurface. This information can then be used to infer rock/soil types, stratigraphy and soil conditions.

The ReMi survey requires a seismic source, to generate surface-waves, and at least 24 geophones, to measure the ground response at increasing distances from the source. Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium. The depth of subsurface penetration of a surface-wave is directly proportional to its wavelength. In a non-homogeneous medium, surface-waves are dispersive, i.e. each wavelength has a characteristic velocity stemming from subsurface heterogeneities. The relationship between surface-wave velocity and wavelength is used to calculate the shear-wave velocity of the medium with increasing depth.

The seismic source can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, and vibrating pads. Examples of passive sources are drill rigs, road traffic, micro-tremors, and water-wave action (in near-shore environments). Geophone measures the arrival time of the various components of the surface wave-train traveling from the seismic source.

The surface-wave velocity with respect to frequency (called the ‘dispersion curve’) is determined by measuring the delay time in wave propagation between the geophones. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modeling procedure. The result is a profile of shear-wave velocity versus depth. This shear wave profile can be with used other parameters such as density, to estimate the dynamic shear modulus of the medium as a function of depth.

The ReMi survey was conducted using a Geometrics Geode 24-channel digital seismograph with acquisition software. The sensors were Mark Products 4.5-Hz vertical geophones placed at 5 ft spacing and the seismic energy source was from traffic.

RESULTS

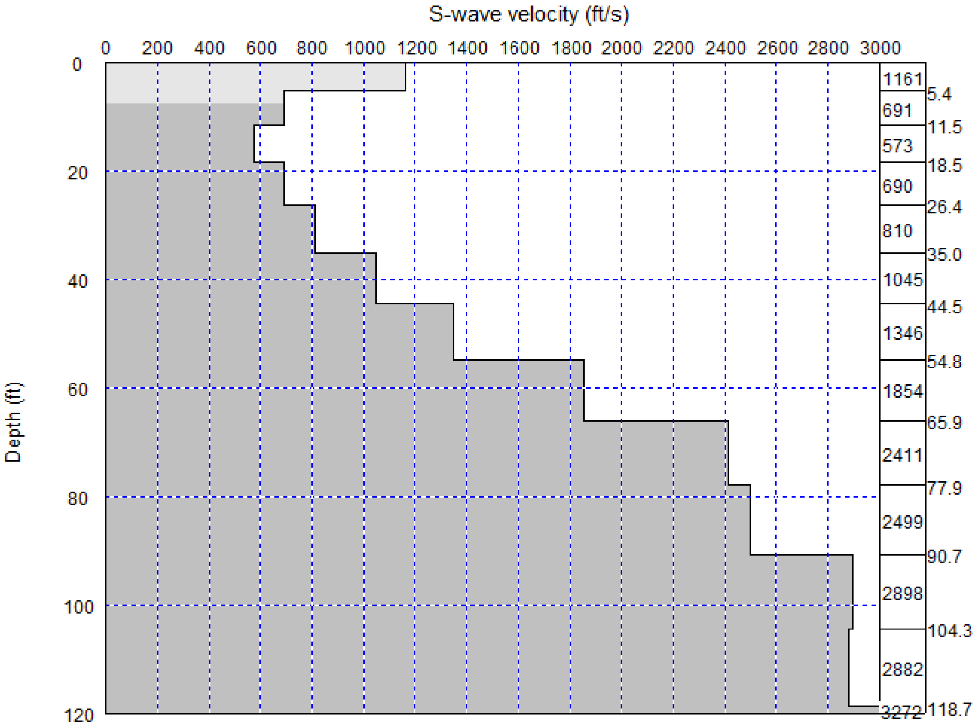
1D sounding data were collected at 1 location. The approximate location is shown below.

The s-wave models from the 1D soundings are shown below.

Table 1: S-wave velocity table

Depth(ft)	S-wave velocity(ft/s)
0	1162
5	691
12	574
19	691
26	811
35	1045
45	1346
55	1854
66	2411
78	2499
91	2898
104	2883
119	3273
134	3774

V100= 1193.6 ft/s



S-wave velocity model (inverted) : 21.dat
Average Vs 100ft = 1193.6 ft/sec

LIMITATIONS OF THE GEOPHYSICAL METHOD

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. ReMi is a remote sensing geophysical method that may not detect all subsurface conditions due to the limitations of the methods, soil conditions, size of the features and their depths.

Sincerely,

Global Geophysics, LLC.

John

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

Appendix VI

NRCS Soil Survey

United States
Department of
Agriculture

Natural
Resources
Conservation
Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for
Kittitas County Area, Washington

SPUD WWTP

October 28, 2019

Soil Map

Map Scale: 1:1,350 if printed on A portrait (8.5" x 11") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

Kittitas County Area, Washington

187—Chinkmin ashy sandy loam, 5 to 30 percent slopes

Map Unit Setting

National map unit symbol: 2ktj
Elevation: 2,500 to 5,900 feet
Mean annual precipitation: 40 to 120 inches
Mean annual air temperature: 35 to 41 degrees F
Frost-free period: 40 to 85 days
Farmland classification: Not prime farmland

Map Unit Composition

Chinkmin and similar soils: 80 percent
Minor components: 20 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Chinkmin

Setting

Landform: Lateral moraines, valley sides
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Volcanic ash and pumice over dense basal till

Typical profile

Oi - 0 to 1 inches: moderately decomposed plant material
Oa - 1 to 2 inches: highly decomposed plant material
H1 - 2 to 5 inches: ashy sandy loam
H2 - 5 to 11 inches: cobbly medial loam
H3 - 11 to 16 inches: cobbly medial loam
H4 - 16 to 23 inches: very cobbly medial sandy loam
H5 - 23 to 33 inches: very gravelly medial sandy loam
H6 - 33 to 41 inches: cemented material

Properties and qualities

Slope: 5 to 30 percent
Depth to restrictive feature: 20 to 40 inches to cemented horizon
Natural drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Low to moderately low (0.01 to 0.06 in/hr)
Depth to water table: About 18 to 36 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Low (about 4.3 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6e
Hydrologic Soil Group: C
Other vegetative classification: Pacific silver fir/rusty menziesia (CFS542)
Hydric soil rating: No

Minor Components

Nimue

Percent of map unit: 7 percent
Hydric soil rating: No

Thetis

Percent of map unit: 5 percent
Hydric soil rating: No

Vabus

Percent of map unit: 5 percent
Hydric soil rating: No

Cryaquepts

Percent of map unit: 3 percent
Landform: Troughs, depressions
Other vegetative classification: Sitka alder/alluvial bar (SWGR12)
Hydric soil rating: Yes

241—Thetis ashy sandy loam, 25 to 45 percent slopes

Map Unit Setting

National map unit symbol: 2kw1
Elevation: 2,400 to 4,000 feet
Mean annual precipitation: 65 to 80 inches
Mean annual air temperature: 39 to 42 degrees F
Frost-free period: 40 to 85 days
Farmland classification: Not prime farmland

Map Unit Composition

Thetis and similar soils: 80 percent
Minor components: 20 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Thetis

Setting

Landform: Mountain slopes, valley sides
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Glacial till with a mantle of volcanic ash

Typical profile

Oe - 0 to 2 inches: moderately decomposed plant material
H1 - 2 to 6 inches: ashy sandy loam
H2 - 6 to 12 inches: gravelly ashy sandy loam
H3 - 12 to 48 inches: very gravelly ashy sandy loam
H4 - 48 to 60 inches: very gravelly sandy loam

Properties and qualities

Slope: 25 to 45 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Low (about 5.9 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7e
Hydrologic Soil Group: A
Other vegetative classification: Pacific silver fir/rusty menziesia (CFS542)
Hydric soil rating: No

Minor Components

Gilpar

Percent of map unit: 5 percent
Hydric soil rating: No

Natkim

Percent of map unit: 5 percent
Hydric soil rating: No

Nimue

Percent of map unit: 5 percent
Hydric soil rating: No

Vabus

Percent of map unit: 5 percent
Hydric soil rating: No