GEOTECHNICAL ENGINEERING REPORT

CLEARVIEW SWITCHING STATION
SNOHOMISH COUNTY PARCEL NO. 27051400101400
184TH STREET SE
SNOHOMISH, WASHINGTON

ZGA Project No. 3035.01
May 7, 2026

Prepared for:

Snohomish County PUD No. 1



Prepared by:

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ZGA Project No. 3035.01
May 7, 2026

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Attention: Mr. Will Blanchard, PE, Professional Engineer

Subject: **Geotechnical Engineering Report DRAFT**
Clearview Switching Station
Snohomish County Parcel No. 27051400101400
Snohomish, Washington

Mr. Blanchard:

In accordance with your request and written authorization, Zipper Geo Associates, LLC (ZGA) has completed our geotechnical engineering report for the proposed Clearview Switching Station project. This report presents the findings of the subsurface evaluation and geotechnical recommendations for the project. Our work was completed in general accordance with the scope of services described in Contract No. CW2260599. Written authorization to proceed was provided by the District on March 12, 2026. We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further assistance, please contact us.

Sincerely,
Zipper Geo Associates, LLC

DRAFT DRAFT

Martin R. Cross, LG
Project Geologist
Signed: x/xx/2026

Robert A. Ross, PE
Managing Principal
Signed: x/xx/2026

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Figure 1 – Site and Exploration Plan

APPENDICES

Appendix A – Subsurface Exploration Procedures and Logs

Appendix B – Laboratory Testing Procedures and Results

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CLEARVIEW SWITCHING STATION
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SNOHOMISH, WASHINGTON

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1.0 INTRODUCTION

This report summarizes the surface and subsurface conditions encountered at the site and our geotechnical engineering recommendations for the proposed Clearview Switching Station project (the Project). Our scope of services included reviewing readily available geologic data, a site reconnaissance, subsurface evaluation, laboratory testing, geotechnical engineering analysis, and preparation of this report. The project description, site conditions, and our geotechnical conclusions and recommendations are presented in the text of this report. Supporting data including detailed exploration logs and field exploration procedures, the results of laboratory testing, and other analyses results are presented as appendices.

1.1 Site Description

The project stie consists of an inverted L-shaped 3.06-acre parcel, located on the northern side of 184th St SE, in Clearview, Washington (Snohomish County Parcel No. 27051400101400). The project site wraps around the eastern and northern sides of the existing Clearview Substation, located at the northeastern corner of the 184th St SE and HWY 9 intersection. The southwestern corner of the site is located about 315 feet east and the northwestern side of the site is located about 305 feet north of the 184th St SE and HWY 9 intersection. The site is bordered to the north by undeveloped property, to the east by an existing nursery, to the south by 184th St SE with residential and commercial property beyond, and to the west by the Clearview substation and HWY 9.

The site is surfaced on the southwestern and southern sides by a variety of mature conifers (primarily on the southwestern side), young alder trees, shrubs, and blackberries. The remaining portion of the site is surfaced with thick blackberries, shrubs, and scattered young alder trees. Limited ponding water at the ground surface was observed on the northeastern corner of the site. In general, the site slopes gently from the east to the west, increasing to moderate on the western side above the existing substation to the west. Total topographic relief ranges from about 30 to 35 feet, sloping downward to the west at about 8 to 10 percent, increasing to 20 to 34 percent on the steeper western edge of the site.

1.2 Project Understanding

We understand the District intends to construct a new switching station and supporting infrastructure on the project site. At the time this report was prepared, project plans were generally at a conceptual level, site improvements are expected to include, but may not be limited to the following:

- New slab-supported transformers with spill containment slabs.
- New slab-supported switchgear enclosures.
- New circuit switches, disconnect switches, neutral reactors, and bus supports.
- Below-grade conduits and pre-cast concrete vaults.

- New underground stormwater infrastructure.
- New transmission poles.
- New access roads extending from the northwestern corner of the Clearview Substation to the northwestern corner of the site.
- New concrete retaining wall along the western side of the substation.
- New tiered gravity block wall along the eastern side of the substation.

As currently proposed, finished ground surface elevation in the yard is proposed at about elevation 572 to 574 ft. Grading to achieved finished grade is expected to generally consist cuts in the eastern three quarters of the site with a maximum anticipated depth of about 19 ft and fills in the western quarter of the site with a maximum anticipated thickness of about 13 ft. Cuts along the eastern property boundary will be supported by multi-tiered retaining walls, and fills along the western project boundary will be supported by a new fill retaining wall.

Detailed information regarding the proposed improvements was not known at the time this report was prepared. Once more detailed plans become available, ZGA should be provided with an opportunity to review them and modify this report as necessary.

2.0 SUBSURFACE CONDITIONS

2.1 General

In order to characterize subsurface soil and groundwater conditions at the project site, we reviewed readily available published surficial geologic mapping, test pit logs from a previously completed geotechnical report on site (prepared by Terraphase Engineering Inc. (2024), completed five (5) geotechnical borings, and three (3) small pilot infiltration (PIT) tests, at the locations shown on the attached Figure 1, Site and Exploration Plan. The following sections describe our understanding of subsurface soil and groundwater conditions based on the mapping and our borings.

2.2 Published Surficial Geologic Mapping

The *Geologic Map of the Bothell Quadrangle, Snohomish and King Counties, Washington, U.S. Geological Survey, Miscellaneous Field Studies Map MF-1747, 1985*, describes the site as underlain by Vashon glacial till (Qvt). Glacial till is described as unsorted, unstratified, compact, sand, silt, and clay with gravel to very large boulders. Glacial till is typically dense to very dense in nature due to being overridden during historical glacial advance. Glacial till generally has a high fines content (the soil fraction passing the US No. 200 sieve) which contributes to the soil's relatively low permeability in combination with the soil density.

2.3 Soil Conditions

In addition to reviewing geologic mapping and test pit logs from the Terraphase Engineering geotechnical report, we completed five (5) geotechnical boings (B-1 through B-5) and three (3) small PITs at the site in April 2026. The borings were completed to depths ranging from about 20.5 to 31 feet below the existing ground surface (bgs) and the PITs were completed to about 8 feet bgs. Approximate locations of each exploration are presented on the attached Figure 1, Site and Exploration Plan.

Soils encountered in the borings were visually described during recovery in general accordance with the Explanation of Exploration Logs provided in Appendix A. Detailed descriptive logs of the subsurface explorations and the procedures utilized in the subsurface exploration program are presented in Appendix A. Generalized descriptions of subsurface soil conditions observed in specific areas of the site are presented below. Please refer to the exploration logs in Appendix A for a more detailed description of the conditions encountered at the exploration locations. Stratification boundaries on the boring logs represent the approximate depth of changes in soil types, although the transition between materials may have been gradual. If variations become apparent during construction, it may be necessary to reevaluate the recommendations of this report.

For purposes of describing soil conditions observed in the borings and for reference in other sections of this report, soils with similar engineering characteristics were grouped together into Engineering Stratigraphic Units or ESUs. The following paragraphs provide our interpretation of ESUs encountered in the borings. ESUs are ordered in a top-down stratigraphic sequence as encountered in the explorations. Shallow surficial conditions encountered in our borings such as asphalt, concrete, and gravel surfacing are not described as ESUs below, and the reader is referred to the boring logs attached in Appendix A for information regarding shallow surficial conditions.

ESU 1 – Weathered Glacial Till: Soils interpreted to be weathered glacial till were encountered in each exploration. ESU 1 soils were encountered below the topsoil, extending to between 1.5 to 4.5 feet bgs. ESU 1 soils observed in our explorations generally consisted of gray-brown to light brown, medium dense to dense, silty sand with varying gravel content. The 2024 Terraphase report indicates weathered glacial till soils extending to between 3.1 and 3.7 feet bgs.

ESU 2 – Unweathered Glacial Till: Soils interpreted to be unweathered glacial till were encountered in each exploration below the weathered glacial till (ESU 1), extending to the termination of each exploration between 20.5 to 31 feet bgs in B-1 through B-5 and to about 8 feet bgs in PIT 1 through PIT 3. In general, ESU 2 soils consisted of a combination of gray-brown to gray, dense to very dense, silty sand with gravel and hard sandy silt with trace to some gravel. In addition, we observed occasional thin wet to saturated lenses within the ESU 2 soils. These zones were typically associated with an increase in sand content. The 2024 Terraphase report indicates unweathered glacial till soils below the weathered glacial till, extending to the termination of each test pit between 3.6 and 4.5 feet bgs.

2.4 Groundwater Conditions

Groundwater was observed in borings B-2 through B-5 between about 16 to 21 feet below existing site grade at the time of drilling. Groundwater was not observed in boring B-1 and PIT-1 through PIT-3. Additionally, no indications of groundwater occurrence were noted in the 2024 Terraphase report. Groundwater observed in our borings is interpreted to be thin saturated zones within sandy layers interbedded in ESU 2 soils.

Groundwater monitoring wells were installed in B-3 through B-5. A summary of groundwater measurements for all of our borings is provided in Table 1 below. Groundwater was measured in B-3 through B-5 on April 22, 2026, at 5.3, 10.2, and 7.1 feet below existing grade, respectively. Given the presence of some interbedded, saturated, soil lenses within the ESU 2 soils, in particular near the measured groundwater elevations on April 22, it appears the increase in groundwater elevation after drilling is associated with slow seepage from perched groundwater within the ESU 2 soils. Considering these measurements were taken near the end of the wet season, we anticipate groundwater elevations observed in April to be near the wet season high.

Table 1: Summary of Groundwater Observations

Exploration Number	Approx. Surface Elevation (ft)[2]	At Time of Drilling 4/7-8/2026	4/22/2026
B-1	568	Dry	NA
B-2	592	576	NA
B-3[1]	584	568	578.7
B-4[1]	573	557	562.8
B-5[1]	593	577	585.9

[1]Groundwater monitoring well installed.
[2]Ground surface elevations at the exploration locations were interpolated from the preliminary site concept plan provided to us by the District

In addition to thin saturated zones within ESU 2 soils, we expect that perched groundwater may develop at the contact between ESU 1 and ESU 2 soils during the wet season or after periods of heavy rainfall. Groundwater levels should be expected to seasonably vary with higher groundwater levels in the winter and spring months and lower groundwater levels during drier summer month.

2.5 Summary of Laboratory Testing

Laboratory testing including moisture content and grain size analyses was completed on select soil samples obtained from our borings and test pits. Table 2 below provides a summary of laboratory testing results. Individual moisture contents are shown on the boring logs in Appendix A. Individual grain size analysis test results are provided in Appendix B.

Table 1: Summary of Laboratory Testing

Engineering Stratigraphic Unit	*In Situ* Moisture Content Range (% of dry weight)	Average *In Situ* Moisture Content (% of dry weight)	Percent Passing No. 200 Sieve Range (fines)	Average Percent Passing No. 200 Sieve (fines)
ESU-1	14 – 21	17	35 - 38	37
ESU-2	7 – 25	14	57 – 65	61

Our scope of services also included testing (completed by a subcontract lab) for the presence of identifiable asbestos fibers in select shallow samples. Asbestos fibers were not detected in any of the samples tested. Refer to Appendix B for additional information.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

Based on information gathered during field exploration, laboratory testing, and analysis, we conclude that construction of the proposed substation improvements is feasible from a geotechnical perspective provided that the recommendations presented herein are followed during design and construction. Selected aspects of the site conditions that should be considered during design and construction are summarized below.

- Soils observed in our borings are generally favorable with respect to site grading and foundation support. Although due to the relatively high fines content of ESU 1 and ESU 2 soils grading during extended periods of wet weather may be difficult.
- We anticipate that excavations for foundations, vaults, and conduits may encounter perched water during the winter and spring near the contact with ESU 1 and ESU 2 soils. Grading during the drier time of year would help reduce seepage and potential dewatering needs.
- Due to shallow perched groundwater within the underlying ESU 2 soils, the use of drilled shaft foundation types may require casing or slurry method if advanced during the wet season. Tremie methods will be required for placement of concrete within drilled shaft excavations unless water can be removed from the drilled shafts prior to placing concrete.
- Onsite, non-organic, native soils were observed to contain relatively high fines content. Reuse of these soils as structural fill during grading will be feasible provided that the soil moisture content can be adequately controlled prior to compaction. However, we expect reuse of native soils during periods of wet weather will be very difficult to functionally impossible.
- Slow groundwater seepage was observed in thin sandy lenses within the high density glacial till soils across the site, except for in the northwestern corner. Seepage was observed on the southeastern portion of the site above the proposed substation yard elevation of 574 feet. Installation of an interceptor drain along the eastern side of the substation yard may be necessary to address observed seepage conditions.

The following sections of this report present specific geotechnical recommendations for the project. Our recommendations are based on the observed soil and groundwater conditions at specific exploration locations. Differing conditions than those observed at boring locations may become evident during construction. If differing soil conditions are encountered, the recommendations presented herein may require modification.

3.2 Geologically Hazardous Areas

According to Chapter 30.91G.020 of the Snohomish County Unified Development Code (SCC), geologically hazardous areas are those that "*because of their susceptibility to erosion, sliding, earthquake, or other geologic events, may not be suited to the siting of commercial, residential, or industrial development consistent with public health or safety concerns. Geologically hazardous areas include erosion hazard areas, landslide hazard areas, seismic hazard areas, and mine hazard areas*. Geologically hazardous areas pertinent to the site, which are summarized below, include erosion hazard areas, landslide hazard areas, and seismic hazard areas.

3.2.1 Erosion Hazard Areas

Chapter 30.91E.160 of the SCC defines an erosion hazard as:

(1) Areas containing soils which are at high risk from water erosion according to the mapped description units of the United States Department of Agriculture Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service, National Soil Classification System,

(2) Channel migration zones; and

(3) The shorelines of water bodies subject to wind and wave erosion.

According to the Snohomish County Online PDS Map Portal the project site and the adjacent properties are not mapped as erosion hazard areas. The project site is not located within a channel migration zone or shoreline. According to the US Department of Agriculture National Resources Conservation Service (NRCS), the project stie is mapped as Alderwood gravelly sandy loam 0 to 8 percent on the eastern side of the site and 8 to 15 percent on the westerns side. Based on the soil conditions encountered on site, it is our opinion that the project site does not meet the definition for an erosion hazard area as outlined in the SCC.

Given the presence of the slope on the western side of the site, we recommend that the project incorporate temporary erosion and sedimentation control (TESC) measures required by the Snohomish County. Stripped surfaces and soil stockpiles are typically a source of runoff sediments. We recommend that silt fencing or straw wattles be installed downslope of work areas to ensure sediment laden water does not flow onto the steep slope or leave the site. For controlling erosion, we recommend that, if wet weather is expected, soils exposed as a result of construction be covered with plastic sheeting.

3.2.2 Landslide Hazard Areas

SCC 30.91L.040 defines landslide hazard areas as *areas potentially subject to mass earth movement based on a combination of geologic, topographic, and hydrologic factors, with a vertical height of 10 feet or more. These include the following:*

(1) Areas of historic landslides as evidenced by landslide deposits, avalanche tracks, and areas susceptible to basal undercutting by streams, rivers or waves;

(2) Areas with slopes steeper than 33 percent which intersect geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock, and which contain springs or ground water seeps;

(3) Areas located in a canyon or an active alluvial fan, susceptible to inundation by debris flows or catastrophic flooding.

According to the Snohomish County Online PDS Map Portal, a portion of the slope on the western side of the site, near the center of the site, is mapped as a steep slope (>33 percent slope). In addition, the PDS Map Portal shows the southern edge of a landslide hazard area extending onto the northwestern side of the property about 14 feet. The majority of the mapped landslide hazard area is located on the neighboring property to the north.

As noted above, slopes steeper than 33 percent are considered landslide hazard areas if they intersect geologic contacts with a relatively permeable soil over relatively impermeable soils. Topographic contours of the area provided to us by the District indicate the southern portion of the mapped steep slope area is less than 10 feet in height, while the northern portion ranges from about 10 to 12 feet in height. During our site reconnaissance of the slope we observed the slope ranged from about 20 to 34 percent. We did not observe any signs of springs or groundwater seeps emanating from the slope. Our explorations encountered high density glacial till soils (ESU 1 and ESU 2 soils) directly below the topsoil across the site. As described previously, glacial till soils are characterized by their very low permeability due to their high density and relatively high fines content. The northern portion of the western slope exceeds 33 percent with a vertical height of 10 feet or more, however, due to the lack of springs or groundwater seeps emanating from the slope and the presence of shallow impermeable glacial till soils, it is our opinion that the slope is not considered a landslide hazard area per the SCC definition above.

With respect to the mapped landslide hazard area on the neighboring property to the north, our site reconnaissance was limited to that portion which is located on the subject property. Topographic contours of the slope indicate that it exceeds 10 feet in height however, our site reconnaissance indicated the slope ranges from about 22 to 28 percent. In addition, we did not observe any signs of springs or groundwater seeps emanating from the slope. Given the slope does not exceed 33 percent, it is our opinion that the slope is not considered a landslide hazard area per the SCC definition above.

In addition, given the soil conditions encountered on site and the scope of the proposed project, it is our opinion that the project will not negatively impact slope stability, provided the project is designed and constructed in accordance with the recommendations detailed in this report.

3.2.3 Seismic Hazard Areas

SCC 30.91S.121 defines a seismic hazard area as *areas that have been determined by the building official to have known or inferred faults, ground rupture potential, liquefaction potential, or seismically induced slope instability, where such information is provided to Snohomish County through any of the following means: geotechnical studies and reports prepared by licensed professionals pursuant ot chapter 10.27 RCW, SCC 30.62B.140 or 30.62B350; geotechnical studies and reports prepared by federal, state, or local agencies; and geotechnical reports or environmental impact statements prepared through the requirements of the State Environmental Policy Ace (SEPA) chapter 43.21C RCW.* These are addressed in the sections below.

Liquefaction: Liquefaction is a phenomenon wherein saturated cohesionless soils build up excess pore water pressure during earthquake loading. Liquefaction typically occurs in loose soils but may occur in dense soils if the ground shaking is sufficiently strong enough. Explorations advanced on site encountered dense to very dense glacial till soils (ESU 2), with a relatively high fines content, at shallow depths, and groundwater encountered was limited to thin saturated lenses observed within the high density glacial till soils. Shallow weathered glacial till soils (ESU 1) were observed to have high fines contents as well. The Snohomish County PDS Map Portal maps the site as having very low liquefaction susceptibility. Based on our current understanding of subsurface conditions at the site, it is our opinion that the risk of liquefaction occurring is low due to the presence of dense to very dense glacially consolidated soils at shallow depths.

Ground Fault Rupture: According to the Snohomish County PDS Map Portal, the site is located approximately 0.7 miles to the north and 0.9 miles south of inferred fault traces of the Southern Whidbey Island fault zone. Given the distance of the fault trace from the site, it is our opinion that the risk of ground surface rupture at the site is low.

Earthquake Induced Landsliding: Based on the very high density of glacial till soils on site and the topography of the site, it is our opinion that the risk of earthquake-induced landsliding is low.

3.3 Site Preparation

Stripping: The existing ground surface in the vicinity of the proposed improvements is generally covered with a mixture of blackberries, small shrubs, alders, and various mature conifers (primarily on the western side). Topsoil and forest duff generally ranged from about 3 to 12 inches thick. In preparation for grading, we recommend removal of all existing surficial vegetation and deleterious debris. Following clearing of surficial vegetation and deleterious debris, organic forest duff and organic-rich topsoil (soils containing more than about 4 percent organic material by weight) should be stripped from locations that are to receive structural fill, within the substation yard footprint, pavements, and new utility areas. These materials should be wasted off site or reused as topsoil for landscaping. Based on the nature of the project site and the site history, the thickness of surficial organic material should be expected to vary and may be deeper than observed at the exploration locations.

Site Preparation and Grading Schedule: ESU 1 and ESU 2 soils encountered below the surficial organic rich materials were observed to consist of a combination of silty sand to sandy silt with varying gravel content and trace cobbles. Laboratory testing of selected samples from these soils indicated fines contents ranging between 35 and 65 percent. These soils are considered to be highly moisture sensitive due to their relatively high fines content; therefore, it will be difficult or even functionally impossible to grade the site during wet weather. We recommend that major site grading take place in the drier summer and early fall months. Completion of site preparation and grading under drier site and weather conditions will reduce the potential for disturbance of the moisture-sensitive soils and the need to replace disturbed soils with imported fill material. Subgrade soils that become disturbed due to elevated moisture conditions should be over-excavated to expose firm, non-yielding, non-organic soils and backfilled with compacted structural fill.

3.4 Structural Fill Materials, Placement, and Compaction

Based on the site plan concept provided to us at the time of this report, the final yard grade is currently planned to range from about elevation 574 on the eastern side to 572 to the western side, which will require significant cuts on the eastern side and moderate to significant fill placement on the western side. To achieve a substation yard elevation of 572 on the western side will require fill placement of about 2 to 14 feet, with the thickest fill section on the southwestern side of the station yard.

In addition, structural fill will be used as backfill for conduit, vault, storm drainage piping and structures excavations, retaining walls, and adjacent to new slabs and shallow foundations. All fill material should be placed in accordance with the recommendations herein for structural fill. Prior to placement, the surfaces to receive structural fill should be observed by a ZGA representative in order to verify that at least medium dense properly prepared fill or native soil is present. In the event that soft or loose soils are present at the subgrade elevation the soils below foundation, slab, vault, and pavement locations should be compacted to a firm and non-yielding condition and to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557) prior to placing structural fill. In the event that the soils cannot be adequately compacted, they should be moisture conditioned as necessary or removed as necessary and replaced with other granular fill material at a moisture content that allows its compaction to the recommended density.

Reuse of On-site Soils: As described above, we understand that a moderate to significant cut along the western side will be necessary to achieve the proposed substation yard elevation of about 574 feet on the western side. Additional on-site material will likely be generated from footing excavations, utility trenches, and other excavations on site. It is our opinion that the non-organic soils encountered on the site are adequate for reuse as structural fill from a compositional standpoint, provided it is placed and compacted in accordance with the recommendations presented in this report. The feasibility of using the on-site soils as structural fill will depend greatly on the weather conditions at the time of placement and compaction. We recommend that the district include a contingency in the construction budget for the cost of exporting fines-laden soil and replacing them with imported granular fill material should construction occur during wet weather.

Soils encountered in our explorations generally contain a relatively high fines content (the portion passing the U.S. No. 200 sieve) ranging from about 35 to 65 percent. As the amount of fines increases, the soil becomes increasingly sensitive to small changes in moisture content. Soils containing more than about 5 percent fines cannot be consistently compacted to the appropriate levels when the moisture content is - more than approximately 2 percent above optimum moisture content (per ASTM D1557). Optimum moisture content is the moisture content which results in the greatest compacted dry density with a specified compactive effort. In situ moisture content of site soils is expected to be slightly to moderately above optimum moisture content for compaction. As a result, we expect that moisture conditioning (drying) of site soils during construction will be required to achieve suitable moisture contents for compaction.

During periods of dry weather, we anticipate site soils will be suitable for reuse as structural fill, although likely requiring moisture conditioning consisting of drying. During periods of wet weather, reuse of site soils as structural fill may become very difficult to functionally impossible. If site soils are planned for re-use, they should be protected from an increase in moisture content during periods of wet weather. At a minimum, we recommend stockpiles of excavated material to be used as structural fill be covered with plastic sheeting if rain is forecasted.

Imported Structural Fill: We recommend that structural fill consist of well-graded sand and gravel with a low fines content, such as the District's standard substation fill, the gradation of which is presented in the table below.

Table 2: Snohomish County PUD No. 1 Substation Imported Granular Fill Gradation

U.S. Standard Sieve Size	Percent Passing by Dry Weight
2 inch	100
½ inch	56 - 100
¼ inch	40 - 78
No. 10	22 - 57
No. 40	8 - 32
No. 200	< 5

This material may be considered slightly to moderately moisture-sensitive relative to placement and compaction. A means of reducing the moisture sensitivity of the imported fill would be to base the fines content to less than 5 percent based on the soil fraction passing the ½ inch sieve. It would be feasible to use other granular soils with a higher fines content as structural fill, but it should be recognized that soils with a higher fines content will be more moisture-sensitive and this may limit their use during wet weather or wet site conditions. The use of other fill types should be reviewed and approved by ZGA prior to their use on site.

It has been our experience that the District may specify the use of Crushed Surfacing, Base Course Gradation (CSBC) [WSDOT Specification 9-03.9(3)] as structural fill. It should be noted that the gradational criteria for crushed surfacing base course allows up to 7.5 percent fines for 1.25-inch minus material. Crushed surfacing base course with a fines content near the permissible upper limit should not be

considered select all-weather fill. Imported fill that is less moisture-sensitive could be achieved by specifying that the material has no more than 5 percent fines based on the soil fraction passing the 1/2-inch sieve. We recommend the use of 100 percent crushed CSBC with a low fines content at the base of fills in the yard to facilitate successful stormwater infiltration if necessary.

Compaction Recommendations: Structural fill should be placed in horizontal lifts and compacted to a firm and non-yielding condition using equipment and procedures that will produce the recommended moisture content and densities throughout the fill. Fill lifts should generally not exceed 10 inches in loose thickness, although the nature of the compaction equipment in use and its effectiveness will influence functional fill lift thicknesses. Recommended compaction criteria for structural fill materials, including trench backfill, are as follows:

Table 3: Recommended Compaction Criteria

Location	Minimum Compaction Criteria
Below foundations and slabs	95
Yard area and extending 5 feet beyond the fence	95
Under driveways, roadways, and sidewalks	95
Fill sections and berms in other areas of the site	90 – 95 (refer to report text)
Trenches, foundation, and slab backfill	95
All other areas	90

3.5 Other Grading Considerations

Freezing Conditions: If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be compacted prior to placing subsequent lifts of structural fill. Alternatively, the frozen material could be stripped from the subgrade to expose unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content; this may not be possible during winter months.

Drainage: Positive drainage should be provided during construction and maintained throughout the life of the project. Uncontrolled movement of water into trenches or foundation and slab excavations during construction should be prevented and it should be the responsibility of the contractor to implement measures to maintain positive drainage. Such measures may include, but may not be limited to placing fill berms or shallow trenches around foundation, vault/conduit, or storm sewer excavations.

Excavation Quantities: It has been our experience that grading calculations need to accommodate a "shrink or swell" factor when comparing in-place soil volumes to truck volumes. We recommend considering that the in-place volume of soil removed from excavations will increase by approximately 25 to 40 percent when measured on a loose cubic yard basis (truck yards). Likewise, loose truck yards delivered to the site will shrink on the order of 25 to 30 percent when compared to the in-place compacted

volume of the soil. Truck yards are also subject to other discrepancies when correlating to bank yards, including "rounding errors" that can be significant.

3.6 Temporary & Permanent Slopes

Temporary excavations may be required to install many of the planned improvements including foundations and underground utilities. Temporary excavation stability is a function of many factors, including the following:

- The presence and abundance of groundwater;
- The type and density of the various soil strata;
- The depth of cut;
- Surcharge loadings adjacent to the excavation;
- The length of time the excavation remains open.

It is difficult to pre-establish a safe and "maintenance-free" temporary cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe temporary excavation slope configurations since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered. It may be necessary to drape temporary slopes with plastic or to otherwise protect the slopes from the elements and minimize sloughing and erosion. Unsupported vertical slopes or cuts deeper than 4 feet are not recommended. The cuts should be adequately sloped, shored, or supported to prevent injury to personnel from local sloughing and spalling. In planning, completing, monitoring, and maintaining temporary excavations, the contractor must adhere to the requirements outlined in Chapter 296-155, Part N of the Washington Administrative Code (WAC) and any other applicable Federal, State, and local regulations.

Based upon our review of WAC Chapter 296-155-66401 (Appendix A – Soil Classification), we have interpreted ESU 1 soils as consistent with Type B and ESU 2 soils as consistent with Type A. The contractor should be responsible for determining soil types in all excavations at the time of construction and should be prepared to adequately shore or slope all excavations.

We recommend permanent cut and fill slopes be designed at an angle no steeper than 2H:1V (horizontal to vertical).

3.6.1 Construction Dewatering

Depending upon the time of year that the work takes place, water from precipitation or potentially from off site (if adequate perimeter control is not enforced), as well as seepage of perched water within ESU 2 soils (where excavations extend into native soil), may accumulate in excavations. The contractor should be prepared to pump water from excavations as necessary to maintain relatively dry conditions. We recommend that design, operation, and maintenance of dewatering systems should be the contractor's responsibility.

3.7 Below-grade Vaults

We anticipate below-grade vaults will be installed as part of this project. Based upon our experience with other District substation projects, we anticipate new vaults could range from approximately 4 to 8 feet below finished grade. Given the anticipated substation yard elevation of 572 to 574 feet, we anticipate that vault subgrades will generally consist of high-density ESU 2 soils on the majority of the site, with potentially some vault subgrades consisting of compacted structural fill, if located within the deeper section of fill on the western side of the site. For vault subgrades within native soil, some variation in soil type and density at vault subgrade locations should be expected. These soils are considered adequate for support of the new vaults.

The vaults will exert a relatively low bearing pressure on the existing soils, and we estimate that up to approximately ½ inch of settlement may take place soon after the vaults are installed and backfilled. In order to reduce the risk of differential vault settlement, we recommend placing a minimum 6-inch compacted thickness of crushed rock below the vault. The crushed rock should conform to the quality and graduation requirements for CSBC detailed in Specification 9-03.9(3) of the WSDOT *Standard Specifications*.

3.8 Underground Utility Installation

Below-grade utilities are expected to include conduits and storm sewer piping and structures. We recommend that utility trenching conform to all applicable federal, state, and local regulations, such as OSHA and WISHA, for open excavations. The existing non-organic native soils encountered on site and properly compacted structural fill (where applicable) are generally expected to be adequate for support of these utilities. Due to the high fines content of onsite soils, these soils may be easily disturbed by excavation and construction equipment, particularly during wet weather, and may need to be compacted prior to utility installation. Subgrade soils that cannot be compacted to a firm and non-yielding condition should be removed and replaced with structural fill that can be compacted to the density recommended in the *Structural Fill* section of this report or with clean crushed rock.

All trenches should be wide enough to allow for compaction around the haunches of the pipe. If water is encountered in the excavations, it should be removed prior to fill placement. Materials, placement and compaction of the utility trench backfill exclusive of Controlled Density Fill (CDF) should be in accordance with the recommendations presented in the *Structural Fill* section of this report. IN our opinion, the initial lift thickness should not exceed one foot unless recommended by the manufacturer to protect utilities from damage by compacting equipment. Light, hand operated compaction equipment may be utilized directly above utilities if damage resulting from heavier compaction equipment is of concern.

3.9 IBC Seismic Design Site Class

For development of IBC seismic ground motion parameters, we recommend Site Class C.

3.10 Foundations

We anticipate proposed structures for the project will be supported on conventional shallow foundations consisting of isolated footings and reinforced concrete slabs. The project may also include the use of

drilled piers for foundation support. Based on the anticipated substation yard grade of 572 to 574 feet and the conceptual layout we were provided, we anticipate shallow foundation subgrade soils will consist of primarily unweathered glacial till (ESU2 soils), with potentially a limited number supported by medium dense ESU 1 soils or compacted structural fill on the western edge of the site. These soils should provide adequate support for the anticipated foundation loads provided foundation subgrades are prepared in accordance with this report. At the time of this report, preliminary designs were not available regarding the presence and location of drilled pier foundations, however, given our past experience with similar projects, we anticipate drilled pier foundations will extend down to ESU 2 soils, which should provide adequate support. Our recommendations for both shallow and drilled pier foundations are provided in the following sections.

3.10.1 Shallow Foundations

Subgrade Preparation: For foundations bearing on structural fill, we recommend that all fill is placed and compacted in accordance with the *Structural Fill* section of this report. For footings bearing on medium dense ESU 1 soils we recommend that the exposed subgrade is compacted to a firm and non-yielding condition and to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557). In the event that the soils cannot be adequately compacted, they should be moisture conditioned as necessary or removed as necessary and replaced with compacted structural fill. Foundations may bear directly on very dense ESU 2 soils.

In order to reduce the risk of post-construction settlement, we recommend shallow foundations bearing on medium dense ESU 1 soils or site soils placed as structural fill be underlain by at least 12 inches of CSBC placed and compacted in accordance with the Structural Fill section of this report. For footings bearing on unweathered glacial till or quality imported granular material placed as compacted structural fil, the CSBC thickness may be reduced to the District's typical minimum thickness. If unsuitable soils, such as soils containing organics or deleterious materials, are encountered at CSBC subgrade elevation, they should be removed and replaced with compacted CSBC. The need for such removal should be determined by ZGA during construction.

New fill placed below shallow foundations should extend laterally beyond the edges of the foundations a distance equal to the new fill thickness. Excavations required to remove unsuitable soils or for placement of the recommended CSBC underlayment will need to consider the recommended lateral extents of new fill.

The base of all foundation excavations should be free of water, loose soil, or debris prior to placing concrete, and should be compacted as recommended in this report. Concrete should be placed soon after excavating and compaction of subgrade CSBC to reduce bearing soil disturbance. Should the bearing subgrade become excessively disturbed or frozen, the affected material should be removed prior to placing concrete. We recommend that a ZGA representative observe foundation subgrade conditions prior to form and reinforcing steel placement.

Shallow Foundation Geotechnical Design Parameter Recommendations: As described above, we anticipate foundation subgrades will consist of primarily very dense ESU 2 soils, with potentially a portion along the western side of the site supported by medium dense ESU 1 soils or structural fill. We have included two allowable bearing pressures, one for footings bearing on site compacted structural fill or compacted medium dense ESU 1 soils, and one for footings bearing on very dense ESU 2 soils. Geotechnical design parameter recommendations for shallow foundations are provided in Table 5 below.

Table 4: Shallow Foundation Geotechnical Design Parameters

Parameter	Recommended Value
Net Allowable Bearing Pressure (Structural Fill or ESU 1 soils)	2,500 psf[2]
Net Allowable Bearing Pressure ESU 2	5,000 psf
Minimum Base Dimension	4 ft
Minimum Embedment Depth for Frost Protection	18 in
Estimated Total Settlement	½ in
Estimated Differential Settlement	½ total settlement over 40 ft
Ultimate Passive Resistance[1]	425 pcf
Ultimate Coefficient of Base Friction[1]	0.55

[1]Does not include a safety factor.
[2] Recommended allowable bearing pressure represents medium dense ESU 1 soils and site soil structural fill compacted to at least 95 percent per ASTM D 1557.

The recommended bearing pressure incorporates a safety factor of 3. A one-third increase may be applied for short-term wind or seismic loading. The recommended value of passive resistance assumes foundations are backfilled with structural fill placed and compacted in accordance with this report. The upper 18 inches of embedment should be ignored when calculating passive resistance. The recommended value for base friction assumes foundations are placed on CSBC placed and compacted as recommended in this report.

3.10.2 Drilled Pier Foundation Recommendations

We understand the proposed project will include some structures supported on drilled pier foundations. We understand that the District will complete lateral analysis of drilled piers using the computer program MFAD. Table 6 provides recommended MFAD geotechnical input parameters for in situ (native) site soils. Some of the drilled piers may be founded in structural fill, if located on the western side where fill material is used to raise site grades. Table 7 provides recommended MFAD geotechnical input parameters for piers founded in compacted structural fill.

Tables 8 and 9 provide geotechnical recommendations for drilled pier axial compressive side and tip resistance in native and structural fill soils, respectively. The values provided in Tables 8 and 9 are nominal ultimate values and do not include safety factors. The pressuremeter elastic modulus values are based

upon correlations with Standard Penetration Test values (N) published in "Estimating Foundation Settlements in Residual Soils", Journal of the Geotechnical Engineering Division, Vol. 103, No. 3, March 1977.

Table 5: Recommended Geotechnical Parameters for MFAD Input (Native Soils)

Depth Interval[1] (ft)	Layer Type	Depth to Bottom of Layer[1] (ft)	Total Unit Weight (pcf)	Deformation Modulus (kips/in^2)	Friction Angle, Ø (degrees)	Undrained Shear Strength (ksf)
0 – 4	Soil	4	120	2.71	34	0
4 – 8	Soil	8	125	3.31	36	0
8 – 25.5	Soil	25.5	135	4.45	38	0

[1]Depths refer to approximate depth below *existing ground surface*.

Table 7: Recommended Geotechnical Parameters for MFAD Input (Structural Fill)

Fill Type	Total Unit Weight (pcf)	Deformation Modulus (kips/in^2)	Friction Angle, Ø (degrees)	Undrained Shear Strength (ksf)
Compacted Structural Fill	125	2.00	32	0

Table 8: Drilled Pier Nominal Axial Compressive Resistance[1] (Native Soils)

Depth Interval[2] (ft)	Nominal Shaft Side Resistance, q_s[3] (psf)	Nominal Shaft Tip Resistance, q_t[4] (ksf)
0 – 4	239	NA[4]
4 – 8	600	15
8 – 25.5	1349	20

[1] This table is applicable to drilled piers constructed entirely in native soils. Resistance values are nominal ultimate values and do not include safety factors. For allowable stress design, use a minimum 2.5 safety factor.
[2] Depth interval refers to approximate depth below *finished site grade*.
[3] Ignore side resistance within the upper 1.5 ft (min.) from finished grade.
[4] We recommend a minimum tip elevation of 5 ft below finished grade..

Table 9: Drilled Pier Nominal Axial Compressive Resistance[1] (Structural Fill)		
Depth Interval[2] (ft)	**Nominal Shaft Side Resistance, q_s[3] (psf)**	**Nominal Shaft Tip Resistance, q_t (ksf)**
0 – 5	200	NA[4]
5 – 10	600	15

[1] This table is applicable to drilled piers constructed entirely in compacted structural fill. Resistance values are nominal ultimate values and do not include safety factors. For allowable stress design, use a minimum 2.5 safety factor.
[2] Depth interval refers to approximate depth below finished site grade.
[3] Ignore side resistance within the upper 1.5 ft (min.) from finished grade.
[4] We recommend a minimum tip elevation of 5 ft below finished grade.

3.10.3 Drilled Pier Construction Considerations

If constructed during the wet season, we anticipate that drilled pier excavations may encounter perched groundwater from saturated lenses within ESU 2 soils. As such, the use of casing, possibly in combination with drilling fluids may be required to maintain borehole stability. The contractor should develop means and methods including the use of casing, the use of drilling fluids or combinations thereof to maintain bore hole stability during construction. The contractor should be prepared to maintain an adequate head of drilling fluids in order to avoid bottom heave of drilled piers. Drilling fluid levels should be maintained to obtain hydrostatic equilibrium throughout the construction operation at a height required to provide and maintain a stable bore hole.

3.11 Cast-in-place Concrete Retaining Wall Recommendations

We understand the project will include cast-in-place concrete retaining walls along the western side of the station yard. At the time of this report, the site concept provided to us indicated the wall on the western side of the site will have level backfill. We recommend that conventional backfilled cast-in-place reinforced concrete walls be designed in accordance with the following recommendations. For recommended bearing capacities and lateral resistance parameters, refer to the *Shallow Foundations* section of this report. Retaining walls must be founded on at least medium dense native soil or compacted structural fill placed above at least medium dense native soils.

Lateral Earth Pressures: The lateral soil pressures acting on backfilled retaining walls will depend on the nature and density of the soil behind the wall, and the ability of the wall to yield in response to the earth loads. Yielding walls (i.e., walls that are free to translate or rotate) that are able to displace laterally at least 0.001H, where H is the height of the wall, may be designed for active earth pressures. Non-yielding walls (i.e., walls that are not free to translate or rotate) should be designed for at-rest earth pressures. Non-yielding walls include walls that are braced to another wall or structure, and wall corners.

For backfilled walls, assuming they are backfilled and drained in accordance with this report, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (active earth pressure). Non-yielding walls should be designed using an equivalent fluid density of 55 pcf (at-rest earth pressure). Design of permanent retaining walls should consider additional earth pressure resulting from the design seismic event. For the seismic case, walls supporting more than 6 feet of backfill should be designed for an additional uniform, total seismic earth pressure distribution of 8H. Surcharge pressures due to sloping backfill, adjacent footings, vehicles, construction equipment, etc. must be added to these lateral earth pressure values. For traffic loads, we recommend using an equivalent two-foot soil surcharge of 250 psf.

Drainage: Appropriate drainage measures must be installed to collect and direct subsurface water away from the back of retaining walls. All backfilled walls should include a drainage aggregate zone extending at least 1 foot from the back of wall for the full height of the wall. We recommend that the drainage aggregate consist of material meeting the requirements of WSDOT 9-03.12(2) *Gravel Backfill for Walls*. A minimum 4-inch diameter perforated PVC drainpipe should be provided at the base of the footing supporting the backfilled walls to collect and direct subsurface water to an appropriate discharge point.

3.12 Concrete Slab Subgrade Preparation Recommendations

We understand the new transformer and switchgear/control enclosures will be supported by reinforced concrete slabs and oil containment slabs will surround the transformer slab. Based on the cut and fill extents indicated by the site concept plan, provided to us at the time of this report, we anticipate that slab subgrade soils will consist of either medium dense ESU 1 soils, dense to very dense ESU 2 soils, or compacted structural fill, depending on where they are located. Slab subgrades should be prepared following the recommendations provided in the *Shallow Foundations* section of this report. We recommend placing a 12-inch compacted thickness CSBC leveling course below the slab. Provided that the slab subgrades are prepared as described herein, we anticipate that total settlement will be less than ½ inch.

3.13 Modular Block Retaining Wall Recommendations

Based on the plan provided to us at the time of this report, we understand the project proposes to construct a new gravity block wall retaining wall to support the cut on the eastern side of the site. The gravity block wall will consist of a tiered wall system, including three block walls with exposed heights of a maximum of 4 feet each. The area between each wall and the slope retained by the upper wall will be sloped at approximately 3H:1V (horizontal to vertical). We expect the wall to be designed as a gravity type retaining wall using pre-cast concrete segmental block units. We anticipate foundation subgrade soils at the retaining wall location will consist of very dense, unweathered glacial till (ESU 2 soils). Our recommendations regarding design and construction of gravity block walls are provided below.

Embedment Depth: 2 feet minimum.

Leveling Pad: The bottom block units should be underlain by a minimum 6-inch-thick compacted crushed rock leveling pad. The leveling pad should extend horizontally at least 6 inches from the front and back of

the block units. Leveling pad material should consist of crushed surfacing top course per Setcion 9-03.9(3) of the WSDOT Standard Specifications.

Wall Backfill: Backfill between the back of the wall and the temporary excavation line required to construct it should consist of compacted Gravel Backfill for Walls per Section 9-03.12(2) of the WSDOT Standard Specifications.

Underdrain: A minimum 4-inch diameter perforated drainpipe should be placed behind the bottom of each wall. The drainpipe should be surrounded by a minimum of 3-inches of Gravel Backfill for Drains per Section 9-03.12(4) of the WSDOT Standard Specifications. The gravel should be wrapped with Geotextile for Underground Drainage, Nonwoven, Moderate Survivability, Class A per Section 9-33.2(1) of the WSDOT Standard Specifications. The drainpipe should be connected to an appropriate discharge point such as the existing stormwater drain system.

Passive Resistance: Passive resistance at the toe of the wall should be ignored.

Block Depth: Each row of blocks should have a minimum depth into the soil perpendicular to the wall face of 40 percent of the wall height as measured from the bottom of the design row to the top of the wall.

Design of gravity walls should be completed using the following soil design parameters:

Table 10: Gravity Block Wall Soil Design Parameters

Parameter	Retained Soil	Foundation Soil
Unit Weight (pcf)	130	130
Friction Angle (deg)	36	40
Cohesion (psf)	0	0
Lateral Earth Pressure (pcf)	39[1]	NA
Allowable Bearing Pressure (psf)	NA	5,000
[1]Assumes backslope no steeper than 3:1 Horizontal:Vertical		

3.14 Driveway Flexible Pavement Section recommendations

We understand that the switching station improvements may include an asphalt-paved driveway. The District typically requires that the pavement section be able to accommodate H20 loading. Pavement subgrade and section recommendations are summarized below.

It should be realized that asphaltic pavements are not maintenance-free. The following pavement sections represent our minimum recommendations for an average level of performance during a 20-year design life; therefore, an average level of maintenance will likely be required. Thicker asphalt, base, and subbase courses would offer better long-term performance, but would cost more initially. Conversely, thinner courses would be more susceptible to “alligator” cracking and other failure modes. As such,

pavement design can be considered a compromise between a high initial cost and low maintenance costs versus a low initial cost and higher maintenance costs.

Based on the access drive contour lines in the plan provided to us at the time of this report, pavement subgrade soils are anticipated to consist of primarily a combination of ESU 1 and ESU 2 soils, with limited portions of compacted structural fill. While the unweathered native glacial till soil (ESU 2) is relatively dense, it has poor drainage characteristics. Our analysis assumes the pavement section subgrade will have a minimum California Bearing Ratio (CBR) value of 10.

We recommend a flexible pavement section consisting of 3 inches of asphalt concrete over 2 inches (compacted thickness) of crushed surfacing top course over 6 inches of crushed surfacing base course. This pavement section assumes that the upper 12 inches of subgrade soil will be prepared as recommended below. Washington State Department of Transportation (WSDOT) specification codes cited herein refer to the 2024 edition of the WSDOT *Standard Specifications*.

- Subgrade Preparation and Compaction: We recommend that subgrade preparation comply with WSDOT Specification 2-03.3(14)C Compacting Earth Embankments – Method C. If portions of the pavement subgrade soil cannot be compacted to the required level, the material should be removed to a depth of 12 inches and replaced with Crushed Surfacing Base Course compacted in accordance with WSDOT Specification 2-03.3(14)C Compacting Earth Embankments – Method C.

- HMA: We recommend that HMA conform to Section 9-02.1(4) for PG 58S-22 or PG 58H-22 Performance Graded Asphalt Binder as presented in the WSDOT *Standard Specifications*. We also recommend that the gradation of the asphalt aggregate conform to the aggregate gradation control points for a ½-inch mix as presented in WSDOT Specification 9-03.8(6), HMA Proportions of Materials.

- Crushed Surfacing Base Course: We recommend that Crushed Surfacing Base Course conform to Section 9-03.9(3) of the WSDOT *Standard Specifications*. Two inches of Crushed Surfacing Top Course per Section 9-03.9(3) may be substituted for the upper two inches of the crushed rock base layer if needed for fine grading purposes. This substitution will not adversely affect pavement performance.

- Compaction and Paving: All base material should be compacted in accordance with WSDOT Specification 2-03.3(14)C Compacting Earth Embankments – Method C. We recommend that HMA be compacted to a minimum of 92 percent and a maximum of 96 percent of the theoretical maximum density. Placement and compaction of HMA should conform to the requirements of Section 5-04 of the WSDOT *Standard Specifications*. This includes weather limitations as specified in Section 5-04.3(1) and maximum nominal asphalt lift thickness as specified in Section 5-04.3(7), Table 6.

3.15 Eastern Interceptor Drain

Slow groundwater seepage was observed in borings B-2 through B-5, associated with saturated sandy lenses within the ESU 2 soils. Groundwater elevations at the time of drilling ranged from 557 to 577 feet and were measured from 562.8 to 585.9 feet in the monitoring wells installed in B-3 through B-5 on April 22, 2026. In general, groundwater elevations were observed to be higher on the eastern side of the site (585.9 in B-5) and lower on the western side (562.8 in B-4). Boring B-5 was located outside of the substation yard footprint, to the east. Based on the substation yard elevation of 574 on the eastern side of the yard, groundwater seepage may be encountered at the time of excavation in this area (although given the cut necessary to achieve the yard elevation, the water bearing sand lense may be removed as part of that excavation). Given the potential for water seepage along the eastern side, in addition to installing a sufficient drainage system with the modular block wall, described above, we recommend that an intercept drain is installed along the eastern side of the substation yard.

We recommend the interceptor drain consist of a minimum 2-foot-deep French drain. The drain should include a 4-inch diameter, schedule 40, perforated PVC drainpipe located 4-inches above the bottom of the trench, sloped to drain to an appropriate discharge location. The trench should be lined with geotextile filter fabric such as Mirafi 140 non-woven geotextile filter fabric or approved equivalent. The trench should be backfilled using drainage aggregate consisting of material meeting the requirements of WSDOT 9-03.12(2) *Gravel Backfill for Walls*.

3.16 Stormwater Infiltration

We understand new stormwater facilities will be installed as part of the project. As part of our service, we performed small pilot infiltration tests (PITs) at three locations within the eastern side of the proposed substation yard. PITs were performed in general accordance with the 2021 Snohomish County Drainage Manual (*Manual*). Our stormwater evaluation was performed in general accordance with the *Manual,* the Snohomish County Public Works Engineering Design and Development Standards (EDDS), and the 2012 Low Impact Development Technical Guidance Manual for Puget Sound (TGM).

Our explorations performed on site encountered weathered and unweathered glacial till (ESU 1 and ESU 2 soils) at shallow depths across the site. As described previously, glacial till soils are overridden during glacial advance, and therefore are considered glacially consolidated. Per Section 3.3.6 of the Drainage Manual, the Soil Grain Size Analysis method for determining the design infiltration rate can only be used for recessional outwash or similar soils that have not been compacted by glacial advance. Therefore, design infiltration rates were determined via three small PITs (PIT-1 through PIT-3).

Glacial till soils typically have low to very low permeability due to their relatively high fines content and high density. Stormwater infiltration into unweathered glacial till is generally not feasible in many cases because of the soil's low permeability, although in some cases even a very low infiltration rate may be adequate given specific site circumstances and design requirements. Infiltration into the less dense weathered glacial till, albeit at low rates, can be feasible in some situations.

Similar soil and groundwater conditions, as related to stormwater infiltration, have been encountered at other substation sites during the design phase. At these sites, the District's approach to stormwater infiltration has been to use a layer of highly permeable substation rock directly below finished grade as an equivalent to permeable pavement. The concept is similar to plastic grid systems discussed in the TGM but without the grid system. The purpose of the grid system is to create a stable, uniform surface to prevent compaction of gravel infill material. However, we have demonstrated previously that very high infiltration rates and void ratios can be maintained in the substation rock after the material has been compacted. Additionally, the substations see little traffic after construction. As stated in the TGM, "Of all the permeable pavement systems, grid systems have the largest void space available for infiltration in relation to the solid support structure." Without the grid system, the void space available is greater. Additionally, as stated in the TGM, groundwater mounding is not a concern for permeable pavement installations that do not have stormwater run-on from adjacent impervious surface, which we anticipate is the case with this project. The permeable pavement surface only infiltrates precipitation falling directly on the permeable pavement surface. Additionally, subsurface flows reaching the infiltration receptor soils are attenuated as the infiltration precipitation flows through the aggregate before it reaches the natural ground below.

It is our opinion that using the substation rock as an equivalent to permeable pavement at this site is feasible. However, some areas of the substation will be located below existing site grades. In these areas, additional excavation may be required to provide an adequate thickness of substation rock to serve as the permeable pavement. Additionally, we recommend areas outside of the main substation perimeter be graded to drain away from the main substation area. At the time of this report, the site concept plan indicates the site entrance will be located on the northwestern corner of the property, although we understand that an entrance connecting to 184th St SE may be evaluated. Consideration should be given to designing the station entrance/exit routes as permeable pavements in a similar manner as the substation.

Volume 5 of the Drainage Manual lists several infeasibility criteria as related to permeable pavements. A list of those criteria and our evaluation of them are provided below. The criteria are cited first in italics, and our evaluation is provided subsequently.

- *Where professional geotechnical evaluation recommends infiltration not be used due to reasonable concerns about erosion, slope failure, or down gradient flooding.*
    - It is our opinion that there is a very low risk of erosion, slope failure, or down gradient flooding if permeable pavement is used for this site.
- *Within an area whose ground water drains into an erosion hazard, or landslide hazard area.*
    - NA, groundwater at this site does not drain into an erosion hazard, or landslide hazard area.

- *Where infiltrating and ponded water below new permeable pavement area would compromise adjacent impervious pavements.*
    - NA
- *Where infiltrating water below a new permeable pavement area would threaten existing below grade basements.*
    - NA, there are not nearby below grade basements.
- *Where infiltrating water would threaten shoreline structures such as bulkheads.*
    - NA
- *Down slope of steep, erosion prone areas that are likely to deliver sediment.*
    - NA
- *Where fill soils are used that can become unstable when saturated.*
    - Fill used to raise site grades will be granular, free-draining material. The risk of instability due to saturation of fill soils is low, in our opinion.
- *Excessively steep slopes where water within the aggregate base layer or at the sub-grade surface cannot be controlled by detention structures and may cause erosion and structural failure, or where surface runoff velocities may preclude adequate infiltration at the pavement surface.*
    - The subgrade slopes gently. The risk of subsurface erosion and structural failure is low, in our opinion.
- *Where permeable pavements cannot provide sufficient strength to support heavy loads at industrial facilities such as ports.*
    - The pavement will be designed to support expected truck loads.
- *Where installation of permeable pavement would threaten the safety or reliability of preexisting underground utilities, pre-existing underground storage tanks, or pre-existing road sub-grades.*
    - NA, there are no pre-existing facilities.
- *Within 50 feet from the top of slopes that are greater than 20%.*
    - In its current condition, the slope on the western side of the site is greater than 20%, however, based on the site concept plan the slope area will be located within the station yard and will be reduced to a relatively level surface during construction. Therefore, it is our opinion that the use of permeable pavements will not increase the risk of erosion or landsliding on these slopes.
- *For properties with known soil or ground water contamination (typically federal Superfund sites or state cleanup sites under the Model Toxics Control Act (MTCA)).*

- Based on our review of information available from the Washington State Department of Ecology (DOE), the nearest site with known soil or groundwater contamination is approximately 900 ft north of the site and is upgradient.

- *Within 100 feet of a closed or active landfill*.
  - NA, we are unaware of a closed or active landfill near the site.
- *Within 100 feet of a drinking water well, or a spring used for drinking water supply, if the pavement is a pollution-generating surface.*
  - Based on our review of information available from DOE, the nearest drinking water well is more than 100 feet from the site.
- *Within 10 feet of a small on-site sewage disposal drainfield, including reserve areas, and grey water reuse systems. For separations from a "large on-site sewage disposal system", see Chapter 246-272B WAC.*
  - NA, no drainfields on the site.
- *Within 10 feet of any underground storage tank and connecting underground pipes, regardless of tank size. As used in these criteria, an underground storage tank means any tank used to store petroleum products, chemicals, or liquid hazardous wastes of which 10% or more of the storage volume (including volume in the connecting piping system) is beneath the ground surface.*
  - NA, no storage tanks planned for the project.
- *At multi-level parking garages, and over culverts and bridges.*
  - NA
- *Where the site design cannot avoid putting pavement in areas likely to have long-term excessive sediment deposition after construction (e.g., construction and landscaping material yards).*
  - NA
- *Where the site cannot reasonably be designed to have a porous asphalt surface at less than 5 percent slope, or a pervious concrete surface at less than 10 percent slope, or a permeable interlocking concrete pavement surface (where appropriate) at less than 12 percent slope. Grid systems upper slope limit can range from 6 to 12 percent; check with manufacturer and local supplier.*
  - We understand the site can be designed to meet a 6 to 12 percent slope limit.
- *Where seasonal high ground water or an underlying impermeable/low permeable layer would create saturated conditions within one foot of the bottom of the lowest gravel base course.*
  - We understand the permeable pavement will be designed to provide at least one foot of separation between the bottom of the gravel base course and underlying low permeability glacial till.

- *Where underlying soils are unsuitable for supporting traffic loads when saturated as determined by subgrade requirements set forth in Snohomish County EDDS Chapter 4.*
    - The system will be designed to support traffic loads.
- *Where appropriate field testing indicates soils have a measured (a.k.a., initial) native soil saturated hydraulic conductivity less than 0.3 inches per hour. (Note: In these instances, unless other infeasibility restrictions apply, roads and parking lots may be built with an underdrain, preferably elevated within the base course, if flow control benefits are desired.)*
    - Based on the results of our small PITs performed on site, initial saturated hydraulic conductivity of onsite ESU 1 soils ranged from 3.2 to 6.1 inches per hour. Please note that these rates reflect the INITIAL hydraulic conductivity and therefore are not the rates that we recommend for design infiltration. The design infiltration rates are detailed in the *Infiltration Rates* section of this report.
- *Roads that receive more than very low traffic volumes, and areas having more than very low truck traffic. Roads with a projected average daily traffic volume of 400 vehicles or less are very low volume roads (AASHTO, 2001)(U.S. Dept. of Transportation, 2013). Areas with very low truck traffic volumes are roads and other areas not subject to through truck traffic but may receive up to weekly use by utility trucks (e.g., garbage, recycling), daily school bus use, and multiple daily use by pick-up trucks, mail/parcel delivery trucks, and maintenance vehicles. Note: This infeasibility criterion does not extend to sidewalks and other non-traffic bearing surfaces.*
    - Traffic volume post-construction at the substation will be well below 400 vehicles per day.
- *Where replacing existing impervious surfaces unless the existing surface is a non-generating surface over an outwash soil with a saturated hydraulic conductivity of four inches per hour or greater.*
    - NA
- *At "high use sites" as described in Volume I, Chapter 4 of this manual.*
    - The site does not meet the definition of a "high use site".
- *In areas with "industrial activity" as identified in 40 CFR 122.26(b)(14)*
    - Based our review of the cited CFR, electrical substations do not meet the definition of "industrial activity".
    - The Standard Industrial Classification of a substation is 4911 and is not included in the cited CFR.
- *Where the risk of concentrated pollutant spills is more likely such as gas stations, truck stops, and industrial chemical storage sites.*
    - PUD substations are designed to prevent potential pollutant spills from intersecting with the ground surface through spill containment slabs.

- *Where routine, heavy applications of sand occur in frequent snow zones to maintain traction during weeks of snow and ice accumulation. Most lowland western Washington areas do not fit this criterion.*
    - NA

The Drainage Manual also refers to the EDDS for additional requirements related to permeable pavements. These additional requirements and our comments relating to them are provided below.

The EDDS specifies the wearing layer of all permeable pavement surface types shall have a minimum initial infiltration rate of 20 inches per hour. Project plans indicate that the substation yard will be mantled with a 4-inch compacted thickness of "substation rock" underlain by a 9-inch thickness of WSDOT Crushed Surfacing Base Course (CSBC) per Specification 9-03.9(3). The substation rock will effectively serve as the wearing layer. The substation rock is used for safety purposes as it has a very high void ratio and electrical resistivity, and its use reduces the likelihood of step potentials developing. The high void ratio of the substation rock and the CSBC are also beneficial from the stormwater management perspective because over the course of design and construction of numerous substations and switching stations it has been shown that these materials provide useful storage capacity.

As part of previous District substation projects, ZGA and others have tested crushed CSBC sourced from the Iron Mountain Quarry in Granite Falls, Washington. Samples of this material when compacted to approximately 95 percent density per ASTM D 1557 have been shown to have a permeability of 130 inches/hour and void ratio of over 40 percent. The Iron Mountain Quarry products are 100 percent crushed rock and no naturally occurring sand is blended with crushed rock to produce the finished product. Based on the testing, the crushed products from Iron Mountain Quarry tend to have a high permeability and void ratio compared to other products that combine crushed rock and sand and this is a function of the overall low fines content (the fraction of soil particles finer than the US No. 200 sieve) and angularity of the products. Below we have excerpted a section from the 30 November 2012 geotechnical engineering report prepared by Terracon Consultants, Inc. which summarizes testing completed on a sample of CSBC sourced from the Iron Mountain Quarry.

Geotechnical Engineering Report
Cedar Valley Substation ▪ Snohomish County, Washington
30 November 2012 ▪ Terracon Project No.: 81125096

We collected a sample of material meeting the criteria for WSDOT Specification 9-03.9(3) *Crushed Surfacing* (base course gradation). The sample was compacted to 95 percent of the modified Proctor maximum dry density (ASTM D 1557) and the permeability determined. Test results are summarized below.

Summary of Crushed Surfacing Laboratory Testing

Supplier / Location	Dry Density (ASTM D 1557)	Compaction (percent)	Specific Gravity (data provided by WSDOT)	Void Ratio	Permeability (inches/hour)
Iron Mountain Quarry / Granite Falls	120.6	95.0	2.75	0.424	130

It should be noted that the testing was completed on the sample fraction passing the US No. ¾-inch sieve for compliance with ASTM D 1557. Actually field values will vary slightly from the reported values due to the presence of aggregate larger than ¾-inch and also due to variations in loads. Material placement procedures can also result in aggregate segregation which can produce variable void ratio and permeability values.

It has been our experience that the crushed rock base course that is produced completely from crushed rock and not including any bank-run material is generally "clean" (lacking finer particles) and this is reflected in the test results.

In 2013, ZGA tested what Iron Mountain Quarry was selling as "substation rock" at the time. This was a 1.5-inch minus product, all crushed, and just slightly coarser than the 1.25-inch minus CSBC. The tested material had a void ratio of 45 percent. A photograph of this substation rock is shown below as a means to illustrate its angularity and obvious functional high void ratio even when compacted.



Iron Mountain Quarry "substation rock"

Clean imported crushed aggregate will also be quite permeable. As part of the design phase of the East Arlington-Oso Substation project that we completed for the District in 2016, we collected a sample of processed material meeting the gradation specification for CSBC as described in WSDOT 9-03.9(3) from Cal Portland of Everett, Washington and engaged the services of HWA Geosciences of Bothell, Washington to complete a permeability test in order to determine the CSBC's usefulness in terms of providing benefits for stormwater management. The sample was compacted to approximately 95 percent of the modified Proctor maximum dry density in order to replicate its condition at the substation and tested for permeability in general accordance with the ASTM D 2434 methodology. The testing indicated a value of 30.8 inches/hour.

The EDDS refers to the TGM for guidance on aggregate base/storage bed materials. The TGM recommends typical base aggregate as a sandy gravel material typical of road base construction. We expect the storage bed materials placed below the wearing course will consist of free draining imported gravel borrow type material typical of road base construction.

SCS Soil Type Classification: Section 2.2.3 *General Design Standards* of the Drainage Manual indicates that approved continuous runoff hydrologic models use one of three predominant soil types to represent soils in Western Washington: outwash, till, and saturated. According to the *Manual* the soil type for use with these models is determined based on the NRCS soil type. As described previously, proposed project area is mapped as Alderwood gravelly sandy loam. According to Table 3.1 *Hydrologic Soil Series for Selected Soils in Washington State* in the *Manual*, Alderwood soil type is considered as Hydrologic Soil Group C.

SSC-1 Setback and Separations for Infiltration Facilities

Given our understanding that the substation yard footprint will be considered permeable pavement, the substation yard meets the setback and separations criteria listed in SSC-1 in a similar manner as described above. Refer to section 3.13 for additional information.

SSC-2 Groundwater Protection Area

Based on information obtained from Snohomish County's online PDS Map Portal, the site is located within a "Sole Source Aquifer" area and a "Low Aquifer Sensitivty" area. Non-pollution generating surfaces with permeable pavements are not listed in Section 30.62C.140 of the SCC as activities or uses that require a hydrologic report and mitigation plan. Additionally, the aquifer from which domestic water wells derive their water in the area is capped by an impermeable layer of glacial till.

SSC-5 Depth to Bedrock, Water Table, or Impermeable Layer

SSC-5 states that the base of all infiltration basins or trench systems shall be ≥ 5 feet above the seasonal high-water mark, bedrock (or hardpan) or other low permeability layer. A separation down to 3 feet may be allowed if the ground water mounding analysis, volumetric receptor capacity, and the design of the overflow and/or bypass structures meet the site suitability criteria specified in this section and will prevent overtopping.

ESU 2 soils (unweathered glacial till) are considered a low permeability layer. ESU 2 soils were generally encountered at about 3 to 4.5 feet below existing site grade with the exception of B-4, located on the southwestern corner of the site, where ESU 2 was encountered at about 1.5 feet below grade. In general, the elevation of the ESU 2 contact ranged from about 564 to 589 feet. Groundwater was measured in the monitoring wells installed in B-3 through B-5 at elevations between about 562.8 and 585.9 feet on April 22, 2026. Based on the plans provided to us at the time of this report, finished yard elevation is anticipated to be at about 574 feet on the eastern side and 572 feet on the western side, with the bottom of permeable pavement section (infiltration system base elevation) planned at 1-foot below finished grade (573 to 571 feet). In general, groundwater elevations and ESU 2 contact elevations were observed on the western side of the site to be up to 8.2 and 7 feet below the infiltration system base elevation, respectively, while they were observed above the substation yard elevation on the eastern side of the site. Based on the proposed infiltration system base elevation, it appears that the western and northwestern sides of the project site meet the separation requirements for SCC-5 and the eastern side of the site does not meet the separation requirements.

3.16.1 Infiltration Rates

We determined stormwater infiltration rates via three small pilot infiltration tests (PITs), located within the proposed new substation yard, on April 13 and 14, 2026. The tests were performed between 2 to 3 feet below the existing grade within the weathered glacial till soils (ESU 1). The soil's design saturated hydraulic conductivity was determined using the measured field rates and the correction factors detailed

in the Drainage Manual. The results of PIT testing and recommended design infiltration rates are provided below.

Due to the presence of glacially consolidated soils, initial saturated hydraulic conductivity of receptor soils were determined via the small PIT method, in accordance with Section 3.3.6 of the Drainage Manual. The design saturated hydraulic conductivity is then determined by applying specific correction factors to the initial rate. According to the Drainage Manual, design infiltration rates for permeable pavement and infiltration trenches are determined using two separate sets of correction factors. We have provided design infiltration rates for both permeable pavement (in accordance with Section 3.3.10 of the Drainage Manual) and infiltration trenches (in accordance with Section 3.3.6 of the Drainage Manual) for each of our PITs. Our results for each are outlined below and summarized in Tables 11 and 12.

For permeable pavement, Section 3.3.10 of the Drainage Manual indicates design saturated hydraulic conductivity rates are determined by applying correction factors for site variability (CFv, from 0.33 to 1.0) and pavement base material (CFm, from 0.9 to 1.0) to the initial rate. Given the presence of glacial till on site, the variation in grain size distribution, and the initial rates observed in the field, we determined a CFv of 0.4 would be representative of the project site. According to the manual, a CFm of 1.0 can only be used if the permeable pavement aggregate base material is clean washed aggregate with 1 percent or less fines (material passing the U.S. 200 sieve). As noted previously in this report, we recommended that imported fill material placed below the substation rock contain a maximum of 5 percent or less fines, therefore a CFm value of 0.9 was used. The design saturated hydraulic conductivity rates for each test location using these correction factors are presented in Table 11.

Table 11: Infiltration Rate Summary – Permeable Pavement

Infiltration Test	Test Depth (ft)	Soil Type	Initial Saturated Hydraulic conductivity (in/hour)	Design Saturated Hydraulic conductivity (in/hour)[1]
PIT-1	2	ESU1: Weathered glacial till	3.2	1.2
PIT-2	2¾	ESU1: Weathered glacial till	3.7	1.3
PIT-3	2-½	ESU1: Weathered glacial till	6.1	2.2

[1]Includes *Manual* Correction Factors: CFv = 0.4 and CFm = 0.9

For permeable pavements, we recommend using an average of the three tests, or 1.6 inches per hour for design.

For infiltration trenches, Section 3.3.6 of the *Manual* indicates design saturated hydraulic conductivity rates are determined by applying correction factors for site variability (CFv, from 0.4 to 1.0), test method

(CFt, from 0.4 to 0.75), and long-term conductivity loss (0.9). Per table 3.5 of the *Manual*, the correction factor for site variability is divided into three categories; highly uniform soils (1.0), highly uniform soils with only one test performed for multiple facilities (0.7), and variable soil conditions with only one test performed for multiple facilities (0.4). Based on soil conditions encountered on site and the number of tests performed, we have used a CFv of 0.4. Per table 3.5 of the *Manual*, the correction factor for test method uncertainty for a small PIT is 0.5. The design saturated hydraulic conductivity rates for each test location using these correction factors are presented in Table 12.

Table 12: Infiltration Rate Summary – Infiltration Trench

Infiltration Test	Test Depth (ft)	Soil Type	Initial Saturated Hydraulic conductivity (in/hour)	Design Saturated Hydraulic conductivity (in/hour)[1]
PIT-1	2	ESU1: Weathered glacial till	3.2	0.6
PIT-2	2¾	ESU1: Weathered glacial till	3.7	0.7
PIT-3	2-½	ESU1: Weathered glacial till	6.1	1.1

1. Includes *Manual* Correction Factors: CFv = 0.4, CFt = 0.5, and long-term conductivity loss = 0.9.

For an infiltration trench, we recommend using an average of the three tests, or 0.8 inches per hour for design.

NRCS Soil Type Classification and Hydrologic Soil Group: Volume III, Section 2.3.2 of the Drainage Manual indicates requires determination of soil types at the site based on NRCS classification and related Hydrologic Soil Groups. Based on our review of the NRCS Web Soil Survey, soils at the site are classified as Alderwood gravelly sandy loam, 0 to 8 percent slopes (57.3% of the site) and Alderwood gravelly sandy loam, 8 to 15 percent slopes (42.7% of the site). Table 3.1 of the Drainage Manual suggests Hydrologic Soil Group C for the Alderwood soil type.

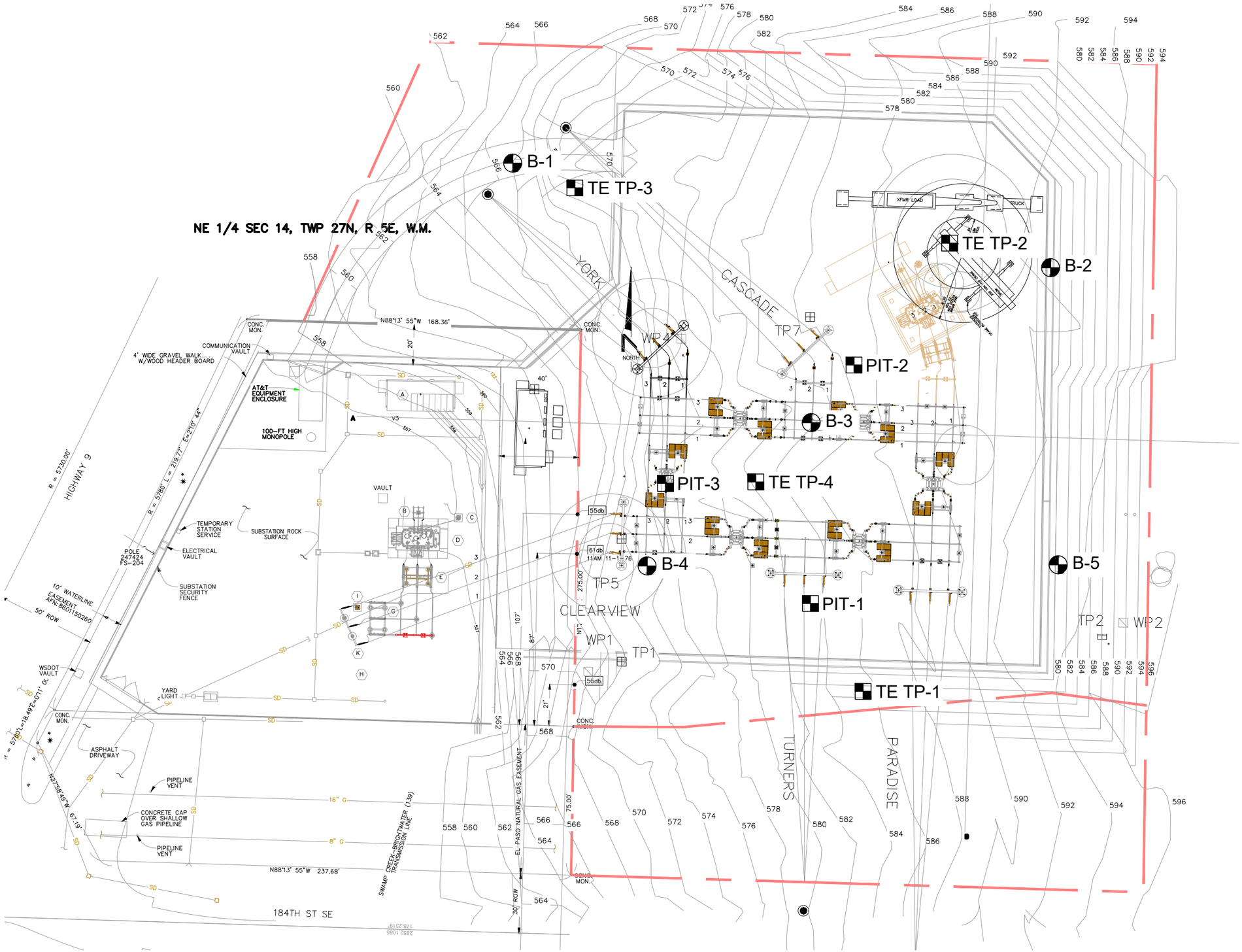
4.0 CLOSURE

The analysis and recommendations presented in this report are based, in part, on the explorations completed for this study. The number, location, and depth of the explorations were completed within the constraints of budget and site access so as to yield the information to formulate our geotechnical recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend that ZGA be provided an opportunity to review the final plans and specifications when they become available in order to assess that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design.

The performance of earthwork, structural fill, foundations, and temporary shoring depend greatly on proper site preparation and construction procedures. We recommend that Zipper Geo Associates, LLC be

retained to provide geotechnical engineering services during the earthwork-related construction phases of the project. If variations in subsurface conditions are observed at that time, a qualified geotechnical engineer could provide additional geotechnical recommendations to the contractor and design team in a timely manner as the project construction progresses.

This report has been prepared for the exclusive use of Snohomish County PUD No. 1 and their agents, for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety and excavation support are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Zipper Geo Associates, LLC reviews the changes and either verifies or modifies the conclusions of this report in writing



LEGEND

B-1	BORING NUMBER AND APPROXIMATE LOCATION
PIT-1	PILOT INFILTRATION TEST NUMBER AND APPROXIMATE LOCATION
TE TP-1	TERRAPHASE (2024) TEST PIT NUMBER AND APPROXIMATE LOCATION

REFERENCE: CLEARVIEW SWITCHING STATION SITE CONCEPT, PROVIDED APRIL 2026

CLEARVIEW SWITCHING STATION
SNO CO PARCEL NO. 27051400101400
CLEARVIEW, WASHINGTON

SITE AND EXPLORATION PLAN

Date: MAY 2026 Job No. 3035.01

Zipper Geo Associates, LLC
19019 36th Ave. W.,Suite E
Lynnwood, WA

FIGURE 1
SHT. 1 of 1

APPENDIX A

SUBSURFACE EXPLORATION PROCEDURES AND LOGS

APPENDIX A
SUBSURFACE EXPLORATION PROCEDURES AND LOGS

Subsurface Exploration Description

Our field exploration program for this project included completing a visual reconnaissance of the site, advancing five borings (B-1 through B-5) and three pilot infiltration test pits (PIT-1 through PIT-3). The approximate exploration locations are presented on Figure 1, the Site and Exploration Plan. Exploration locations were determined in the field using latitude and longitude coordinate from Google Earth. The ground surface elevation at the exploration location was interpolated from elevation contours shown on the preliminary site plan provided to us by the District. As such, the exploration locations and elevations should be considered accurate to the degree implied by the measurement methods. The following sections describe our procedures associated with the explorations. Descriptive logs of the explorations are enclosed in this appendix.

Boring Procedures

The borings were advanced using a track-mounted drill rig operated by an independent drilling company (Geologic Drill Partners) working under subcontract to ZGA. The borings were advanced using hollow stem auger drilling methods. A ZGA geologist continuously observed the borings, logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further evaluation and testing. Samples were generally obtained by means of the Standard Penetration Test at 2.5-foot intervals throughout the drilling operation.

The Standard Penetration Test (ASTM D 1586) procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows are struck within any 6-inch interval, the driving is stopped, and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

Test Pit Procedures

An independent contractor (Northwest Excavation & Trucking) working under subcontract to ZGA excavated the test pits through the use of a tracked excavator. An engineering geologist from ZGA continuously observed the test pit excavations, logged the subsurface conditions, and obtained representative soil samples. The samples were stored in moisture tight containers and transported to our laboratory for further visual classification and testing.

Soils were logged in general accordance with American Society for Testing and Materials (ASTM) Practice Designation D-2488, *Standard Practice for Description of Soils (Visual-Manual Procedure)*. The enclosed test pit logs indicate the vertical sequence of soils and materials encountered in each test pit, based primarily on our field classifications and supported by our subsequent laboratory testing. Where a soil contact was observed to be gradational or undulating, our logs indicate the average contact depth. We

estimated the relative density and consistency of *in situ* soils by means of the excavation characteristics and by the sidewall stability. Our logs also indicate the approximate depths of any sidewall caving or groundwater seepage observed in the test pits, as well as all sample numbers and sampling locations.

EXPLANATION OF EXPLORATION LOGS

SOIL DESCRIPTION: Soil descriptions presented on the borings logs are based on visual observations. Soil descriptions include density (coarse-grained soils) or consistency (fine-grained soils), moisture, color, major soil type, and grain size modifiers and should not be interpreted to suggest laboratory or field testing unless indicated on the logs. Soil descriptions include the following: Density/consistency, moisture, color, grain size modifier (adjective implying 31-49 percent), major soil type (CAPITALIZED implying 50+ percent), minor grain size modifier (some implying 6-12 percent, with implying 13-30 percent, and trace implying 0-5 percent), descriptive modifiers (i.e. roots, fill debris, cemented, etc.), and interpreted general geologic description. Descriptions may also include comments describing geologic properties such as dilatancy, toughness, structure, plasticity, and angularity of coarse-grained particles. Additional information regarding geologic properties is presented in the report text as applicable.

DENSITY/CONSISTENCY: Soil density/consistency in borings is related to the blow count number in blows per foot using the sampling method indicated on the logs. Soil density/consistency in test pits is related to a "Field Test" as described below. Soil consistency in test pits or borings may be augmented by field Torvane or Pocket Penetrometer testing.

Coarse-Grained Soils

Density Descriptor	SPT (# blows/ft)	Field Test
Very Loose	0 – 4	Easily penetrated with ½ -inch steel rod pushed by hand.
Loose	5 – 10	Difficult to penetrate with ½ - inch steel rod pushed by hand.
Medium Dense	11 – 30	Easily penetrated a foot with ½-inch steel rod driven with 5-lb hammer.
Dense	31 – 50	Difficult to penetrate a foot with ½-inch steel rod driven with 5-lb hammer.
Very Dense	>50	Penetrated only a few inches with ½-inch steel rod driven with 5-lb hammer.

Fine-Grained Soils

Consistency Descriptor	SPT (# blows/ft)	Torvane	Pocket Penetrometer	Field Test
		Undrained shear strength (tsf)	Unconfined Compressive Strength (tsf)	
Very Soft	0 – 2	<0.125	<0.25	Easily penetrates several inches by thumb.
Soft	3 – 4	0.125 – 0.25	0.25 – 0.5	Easily penetrates one inch by thumb.
Medium Stiff	5 – 8	0.25 – 0.5	0.5 – 1.0	Penetrated over ½ inch by thumb with moderate effort.
Stiff	9 – 15	0.5 – 1.0	1.0 – 2.0	Indented by thumb but penetrated only with great effort.
Very Stiff	16 – 30	1.0 – 2.0	2.0 – 4.0	Readily indented by thumbnail.
Hard	>30	>2.0	>4.0	Indented by thumbnail with difficult effort.

MOISTURE

Descriptor	Field Test
Dry	Absence of moisture, dusty, dry to the touch.
Damp	Too low to achieve compaction
Moist	Appears near optimum moisture content for compaction
Wet	Too wet to achieve compaction
Saturated	Below the groundwater table, visible free moisture.

MAJOR SOIL TYPE: Coarse-grained soils with over 50% of the material retained on the U.S. No. 200 sieve. Coarse-grained soils include boulders, cobbles, gravels and sands. Fine-grained soils with over 50% of the material passing the U.S. No. 200 sieve. Fine-grained soils include silts and clays.

GRAIN SIZE

Descriptor	Sieve Size	Grain Size
Boulder	>12"	>12"
Cobble	3 – 12"	3 – 12"
Gravel	3" – #4	3" – 0.19"
Sand	>#4 – #200	<0.19" – >0.0029"
Silt/Clay	Passing #200	<0.0029"

GRAIN SIZE MODIFIERS

Descriptor	Approximate Percentage
Trace	0 – 5
Some	6 – 12
With	13 – 30
Adjective (silty, clayey, sandy, gravelly)	31 – 50

BORING LOG SYMBOL LEGEND

SAMPLE LEGEND

 BULK SAMPLE

 2-INCH O.D. SPLIT SPOON SAMPLE

 3-INCH O.D. MODIFIED CALIFORNIA SAMPLE

 3-INCH I.D. SHELBY TUBE SAMPLE

TESTING KEY

GSA	GRAIN SIZE ANALYSIS
200W	200 WASH ANALYSIS
ATT.	ATTERBERG LIMITS

GROUNDWATER LEGEND

 CLEAN SAND

 BENTONITE

 GROUT/CONCRETE

 SCREENED CASING

 BLANK CASING

 GROUNDWATER LEVEL AT TIME OF DRILLING (ATD)

 GROUNDWATER LEVEL POST DRILLING

BORING LOG: B-1

B-1

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	568 ft			Reviewer:	MRC
Lattitude:	47.83130	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12700	Drill Rig:	EC 95 Track Drill	Hammer Type:	Cathead
Drill Date:	2026-04-07	Drilling Method:	HSA, SPT		

Depth (FT)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Samples No.	Recovery Length (in)	Type	Piezometer / Well	Standard Penetration Test (▲ NValue, ● % Fines, ○ % Moisture; 10 20 30 40 50)	SPT Number (N)	Testing
	Ground Surface EL 568 ft							
0	Blackberries over about 6-inches of topsoil, over dense, moist, gray-brown, silty SAND, some gravel (Weathered Glacial Till) (ESU 1)				No Data			
1								
2						○		
3		S-1	18	▽		▲	34	ACM
4	4 ft EL 564 ft							
5	Dense, moist, gray-brown, silty SAND, trace gravel (Unweathered Glacial Till) (ESU 2)					○		
6		S-2	18	▽		▲	43	
7								
8	Grades to hard, moist, gray-brown, sandy SILT, trace gravel	S-3	18	▽		○ ● ▲	67	GSA
9								
10	Gravel content increases to with to some					○		
11		S-4	18	▽		▲	57	
12								
13								
14								
15						○		
16		S-5	18	▽		▲	78	
17								
18								
19								
20								

ZipperGeo
Geoprofessional Consultants
19019 36th Ave. W, Suite E
Lynnwood, WA 98036

PROJECT TITLE: Clearview Switching Station

PROJECT NO.: 3035.01

CLIENT: Snohomish County PUD No. 1

SHEET: 1 of 2

RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT - 11.14.25 / zipper-geo-associates / admin / 2026-05-05 @ 02:27 PM

BORING LOG: B-1

B-1

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	568 ft			Reviewer:	MRC
Latitude:	47.83130	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12700	Drill Rig:	EC 95 Track Drill	Hammer Type:	Cathead
Drill Date:	2026-04-07	Drilling Method:	HSA, SPT		

Depth (FT)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Samples: No.	Samples: Recovery Length (in)	Samples: Type	Piezometer / Well	Standard Penetration Test: ▲ NValue 10 20 30 40 50 / ● % Fines (10 20 30 40 50 / ○ % Moisture 10 20 30 40 50	SPT Number (N)	Testing
	(continued)							
20	Dense, moist, gray-brown, silty SAND, trace gravel (Unweathered Glacial Till) (ESU 2)	S-6	18	▽			▲50/5"	
21	Very dense, moist, gray-brown, silty SAND with gravel							
22	21.5 ft EL 546.5 ft Boring completed at approximately 21.5 feet bgs.							
23	Groundwater was not observed during drilling.							
24								
25								
26								
27								
28								
29								
30								
31								
32								
33								
34								
35								
36								
37								
38								
39								
40								

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SHEET: 2 of 2

RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT - 11.14.25 / zipper-geo-associates / admin / 2026-05-05 @ 02:27 PM

BORING LOG: B-2

B-2

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	592 ft			Reviewer:	RAR
Lattitude:	47.83120	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12600	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-07	Drilling Method:	HSA, SPT		

Depth (FT)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Samples No.	Recovery Length (in)	Type	Piezometer / Well	Standard Penetration Test ▲ NValue 10 20 30 40 50 ● % Fines (10 20 30 40 50 ○ % Moisture 10 20 30 40 50	SPT Number (N)	Testing
	Ground Surface EL 592 ft							
0	Blackberries over about 3-inches organic rich topsoil, over medium dense, moist, gray-brown, silty SAND with gravel (Unweathered Glacial Till) (ESU 1)				No Data			
1								
2						○		
3		S-1	18	▽		▲	27	ACM
4	4.5 ft EL 587.5 ft							
5	Hard, moist, gray-brown, sandy SILT, trace gravel (Unweathered Glacial Till) (ESU 2)					○		
6		S-2	18	▽		▲	52	
7						○		
8		S-3	18	▽		▲	51	
9								
10	Grades to very dense, moist, gray-brown, silty SAND, with gravel	S-4	10	▽		○	▲50/4"	
11								
12								
13								
14								
15		S-5	10	▽		○	▲50/4"	
16	▽							
17								
18								
19								
20								

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CLIENT: Snohomish County PUD No. 1

SHEET: 1 of 2

RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT - 11.14.25 / zipper-geo-associates / admin / 2026-05-05 @ 02:27 PM

BORING LOG: B-2

B-2

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	592 ft			Reviewer:	RAR
Lattitude:	47.83120	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12600	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-07	Drilling Method:	HSA, SPT		

Depth (FT)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Samples No.	Samples Recovery Length (in)	Samples Type	Piezometer / Well	Standard Penetration Test ▲ NValue 10 20 30 40 50 ● % Fines (10 20 30 40 50 ○ % Moisture 10 20 30 40 50	SPT Number (N)	Testing
	(continued)							
20	Hard, moist, gray-brown, sandy SILT, trace gravel (Unweathered Glacial Till) (ESU 2)	S-6	6	▽			▲50/5"	
21								
22	Hard, moist, gray-brown, sandy SILT, trace gravel 20.5 ft EL 571.5 ft							
23	Boring completed at approximately 20.5 feet bgs. Groundwater was observed at approximately 16 to 17 feet during drilling.							
24								
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Geoprofessional Consultants
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PROJECT TITLE: Clearview Switching Station

PROJECT NO.: 3035.01

CLIENT: Snohomish County PUD No. 1

SHEET: 2 of 2

RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT - 11.14.25 / zipper-geo-associates / admin / 2026-05-05 @ 02:27 PM

BORING LOG: B-3

B-3

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	584 ft			Reviewer:	RAR
Lattitude:	47.83100	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12630	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-07	Drilling Method:	HSA, SPT		

Ground Surface EL 584 ft

Depth (FT)	SOIL DESCRIPTION	Samples No.	Recovery Length (in)	Type	Piezometer / Well	Standard Penetration Test (NValue / % Fines (/ % Moisture) 10 20 30 40 50	SPT Number (N)	Testing
0	Blackberries over about 4-inches organic rich topsoil, over medium dense, moist, gray-brown, silty SAND, trace gravel (Unweathered Glacial Till) (ESU 1)							
3		S-1	18				28	ACM
4	4 ft EL 580 ft							
5	Dense, moist, gray-brown, silty SAND, some gravel (Unweathered Glacial Till) (ESU 2)							
6		S-2	18				38	GSA
8	Grades to wet	S-3	18				50/5"	
10	Very dense, moist, gray-brown, silty SAND, with gravel	S-4	18				50/5"	
15		S-5	18				65	
16	Thin saturated zone observed between 16 and 17 feet							
20								

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PROJECT NO.: 3035.01

CLIENT: Snohomish County PUD No. 1

SHEET: 1 of 2

BORING LOG: B-3

B-3

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	584 ft			Reviewer:	RAR
Latitude:	47.83100	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12630	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-07	Drilling Method:	HSA, SPT		

Depth (FT)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Samples: No.	Samples: Recovery Length (in)	Samples: Type	Piezometer / Well	Standard Penetration Test: NValue 10 20 30 40 50 / % Fines (10 20 30 40 50 / % Moisture 10 20 30 40 50	SPT Number (N)	Testing
	(continued)							
20	Dense, moist, gray-brown, silty SAND, some gravel (Unweathered Glacial Till) (ESU 2)	S-6	18	▽		▲	39	
21	Hard, moist to wet, gray-brown, sandy SILT, trace gravel							
22								
23								
24								
25		S-7	18	▽				
26	26.5 ft EL 557.5 ft						▲ 63	
27	Boring completed at approximately 25.5 feet bgs. Perched groundwater was observed from approximately 16 to 17 feet during drilling. Well tag ID: BRE 324.							
28								
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33								
34								
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36								
37								
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40								

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PROJECT TITLE: Clearview Switching Station

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CLIENT: Snohomish County PUD No. 1

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RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT - 11.14.25 / zipper-geo-associates / admin / 2026-05-05 @ 02:27 PM

BORING LOG: B-4

B-4

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	573 ft			Reviewer:	RAR
Latitude:	47.83060	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12670	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-08	Drilling Method:	HSA, SPT		

Ground Surface EL 573 ft

Depth (FT)	SOIL DESCRIPTION	Sample No.	Recovery Length (in)	Sample Type	Standard Penetration Test (NValue / % Fines / % Moisture)	SPT Number (N)	Testing
0	about 12-inches forest duff over medium dense, moist, gray-brown, silty SAND, some gravel (Unweathered Glacial Till) (ESU 1)						
1.5 ft / EL 571.5 ft							
2	Very dense, moist, gray-brown, silty SAND, with gravel (Unweathered Glacial Till) (ESU 2)						
3		S-1	18			50/5	ACM
5	Grades to hard, moist, gray-brown, sandy SILT, trace gravel	S-2	18			51	GSA
8		S-3	18			50/6"	
10 ▼	Very dense, moist, gray-brown, silty SAND, with gravel	S-4	11			50/5	
15	Grades to wet, trace gravel						
16 ▽		S-5	17			50/5"	
20							

Piezometer / Well: #1

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PROJECT TITLE: Clearview Switching Station

PROJECT NO.: 3035.01

CLIENT: Snohomish County PUD No. 1

SHEET: 1 of 2

BORING LOG: B-4

B-4

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	573 ft			Reviewer:	RAR
Latitude:	47.83060	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12670	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-08	Drilling Method:	HSA, SPT		

Depth (FT)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Samples: No.	Samples: Recovery Length (in)	Samples: Type	Piezometer / Well	Standard Penetration Test: ▲ NValue 10 20 30 40 50 / ● % Fines (10 20 30 40 50 / ○ % Moisture 10 20 30 40 50	SPT Number (N)	Testing
	(continued)							
20	Very dense, moist, gray-brown, silty SAND, with gravel (Unweathered Glacial Till) (ESU 2)	S-6	10	▽			▲50/6"	
21	Very dense, wet to saturated, gray-brown, SAND, with silt							
22								
23								
24								
25	Very dense, moist, gray-brown, silty SAND, with gravel	S-7	5	▽			▲50/5"	
26	25.5 ft EL 547.5 ft Boring completed at approximately 25.5 feet bgs. Groundwater was observed from approximately 16 to 21 feet during drilling.							
27								
28	Well tag ID: BRE 325							
29								
30								
31								
32								
33								
34								
35								
36								
37								
38								
39								
40								

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PROJECT TITLE: Clearview Switching Station

PROJECT NO.: 3035.01

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BORING LOG: B-5

B-5

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	593 ft			Reviewer:	RAR
Lattitude:	47.83080	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12600	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-08	Drilling Method:			

Ground Surface EL 593 ft

Depth (FT)	SOIL DESCRIPTION	Sample No.	Recovery Length (in)	Type	Piezometer / Well	Standard Penetration Test (▲ NValue, ● % Fines (, ○ % Moisture; 10 20 30 40 50)	SPT Number (N)	Testing
0	Blackberries over about 6-inches organic rich topsoil, over medium dense, moist to wet, light brown with oxidation staining, silty SAND, trace to some gravel (Weathered Glacial Till) (ESU 1)				#1 Standpipe (JP)			
3		S-1	18				21	ACM
4	4.5 ft / EL 588.5 ft							
5	Dense, moist, gray, silty SAND with gravel (Unweathered Glacial Till) (ESU 2)	S-2	18				46	
7 ▼								
8	Grades to very dense	S-3	18				60	
10	Hard, moist, gray, sandy SILT, trace gravel	S-4	17				50/5"	
15	Moisture content increases to moist to wet	S-5	11				50/5"	
16 ▽								
20								

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SHEET: 1 of 2

BORING LOG: B-5

B-5

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	593 ft			Reviewer:	RAR
Lattitude:	47.83080	Drilling Company:	Boretec1, Inc	Borehole Diameter:	8 inch
Longitude:	-122.12600	Drill Rig:	EC 95 Track Drill	Hammer Type:	Rope and Cathead
Drill Date:	2026-04-08	Drilling Method:			

Depth (FT)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Samples: No.	Samples: Recovery Length (in)	Samples: Type	Piezometer / Well	Standard Penetration Test: NValue / % Fines (/ % Moisture (10 20 30 40 50)	SPT Number (N)	Testing
	(continued)							
20	Dense, moist, gray, silty SAND with gravel (Unweathered Glacial Till) (ESU 2)	S-6	18	▽			▲50/6"	
21								
22								
23								
24								
25		S-7	11	▽			▲50/5"	
26								
27								
28								
29								
30	Very dense, moist, gray, silty SAND, with gravel	S-8	11	▽			▲50/5"	
31	31 ft EL 562 ft Boring completed at approximately 31 feet bgs. Perched groundwater was observed from approximately 16 to 20 feet during drilling. Well tag ID: BRE 326							
32								
33								
34								
35								
36								
37								
38								
39								
40								

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PROJECT TITLE: Clearview Switching Station

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TEST PIT LOG: PIT-1

PIT-1

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	592 ft				
Latitude:	47.83070	Excavator Company:	NW Excavating and Trucking	Reviewer:	RAR
Longitude:	-122.12630				
Test Pit Date:	2026-04-13	Excavator:	Bobcat E42		

Depth (ft)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Sample	% Moisture	% Fines (<0.075mm)	Testing
	Ground Surface EL 592 ft				
0	About 6-inches organic rich topsoil over Medium dense, moist, light brown, silty SAND, some gravel. (Weathered Glacial Till) (ESU 1)				
3	3 ft EL 589 ft Very dense, moist, gray, silty SAND with gravel. (Unweathered Glacial Till) (ESU 2)	S-1	26	35	GSA
7		S-2	16		
8	8 ft EL 584 ft Boring completed at approximately 8 feet bgs. Groundwater was not observed during excavation.				

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PROJECT TITLE:	PROJECT NO.:
Clearview Switching Station	3035.01
CLIENT:	SHEET:
Snohomish County PUD No. 1	1 of 1

RSLog / 1ZGA Test Pit Log - CURRENT 11.14.25 / zipper-geo-associates / admin / 2026-05-05 @ 02:27 PM

TEST PIT LOG: PIT-2

PIT-2

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	592 ft				
Latitude:	47.83110	Excavator Company:	NW Excavating and Trucking	Reviewer:	RAR
Longitude:	-122.12620				
Test Pit Date:	2026-04-13	Excavator:	Bobcat E42		

Depth (ft)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Sample	% Moisture	% Fines (<0.075mm)	Testing
	Ground Surface EL 592 ft				
0	About 4-inches organic rich topsoil over Medium dense, moist, light brown, silty SAND with gravel. (Weathered Glacial Till) (ESU 1)				
1					
2					
3	3 ft EL 589 ft Very dense, moist, gray, silty SAND with gravel. (Unweathered Glacial Till) (ESU 2)				
4		S-1	21		
5					
6					
7		S-2	17		
8	8 ft EL 584 ft Boring completed at approximately 8 feet bgs. Groundwater was not observed during excavation.				
9					
10					
11					
12					
13					
14					
15					

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PROJECT TITLE: Clearview Switching Station	PROJECT NO.: 3035.01
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RSLog / 1ZGA Test Pit Log - CURRENT 11.14.25 / zipper-geo-associates / admin / 2026-05-05 @ 02:27 PM

TEST PIT LOG: PIT-3

PIT-3

Location:	See Figure 1.	Project Address:	184th St SE, Snohomish, WA	Logged By:	JAM
Elevation:	576 ft				
Latitude:	47.83080	Excavator Company:	NW Excavating and Trucking	Reviewer:	RAR
Longitude:	-122.12660				
Test Pit Date:	2026-04-14	Excavator:	Bobcat E42		

Depth (ft)	SOIL DESCRIPTION The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.	Sample	% Moisture	% Fines (<0.075mm)	Testing
	Ground Surface EL 576 ft				
0	Blackberries over about 6-inches of organic-rich topsoil, over medium dense, wet, orange-brown, silty SAND, some gravel (Weathered Glacial Till) (ESU 1)	S-1	51		
1					
2		S-2	28	38	GSA
3	3 ft EL 573 ft Dense, moist, gray-brown, silty SAND, with gravel (Unweathered Glacial Till) (ESU 2)	S-3			
4					
5					
6					
7					
8	8 ft EL 568 ft Boring completed at approximately 8 feet bgs. Groundwater was not observed during excavation. Infiltration testing was completed at about 2 feet bgs.	S-4	15		
9					
10					
11					
12					
13					
14					
15					

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PROJECT TITLE: Clearview Switching Station

PROJECT NO.: 3035.01

CLIENT: Snohomish County PUD No. 1

SHEET: 1 of 1

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APPENDIX B
LABORATORY TESTING PROCEDURES AND RESULTS

LABORATORY TESTING PROCEDURES

A series of laboratory tests were performed during the course of this study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of tests performed are given below.

Visual Classification

Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with the Unified Soil Classification system. Visual soil classification includes evaluation of color, relative moisture content, soil type based upon grain size, and accessory soil types included in the sample. Soil classifications are presented on the exploration logs in Appendix A.

Moisture Content Determinations

Moisture content determinations were performed on representative samples obtained from the exploration in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM: D-2216. The results are shown on the exploration logs in Appendix A.

Grain Size Analysis

A grain size analysis indicates the range in diameter of soil particles included in a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM D 6913. The results of the grain size determinations for the samples were used in classification of the soils, and are presented in this appendix.

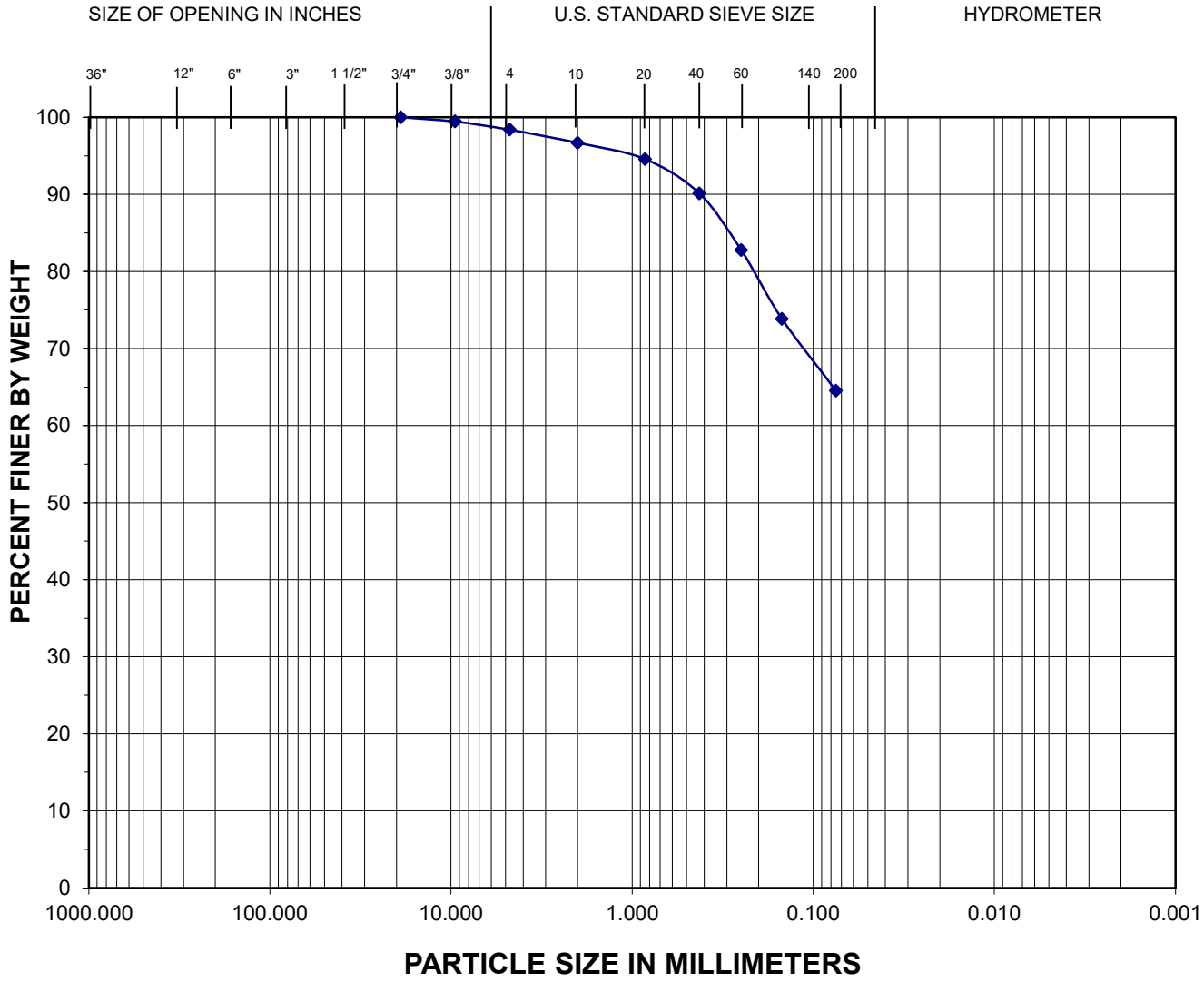
Asbestos Containing Material (ACM)

Five samples of near surface soils were collected from the borings in order to test for the presence of ACM. Examination of these samples was conducted for the presence of identifiable asbestos fibers using polarized light microscopy (PLM) with dispersion staining in accordance with both EPA 600/M4-82-020, Interim Method for the Determination of Asbestos in Bulk Insulation Samples and EPA 600/R-93/116 Method for the Determination of Asbestos in Bulk Building Materials. Results of the tests are presented in the attached NVL report in this appendix. ACM was not detected in any of the samples.

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

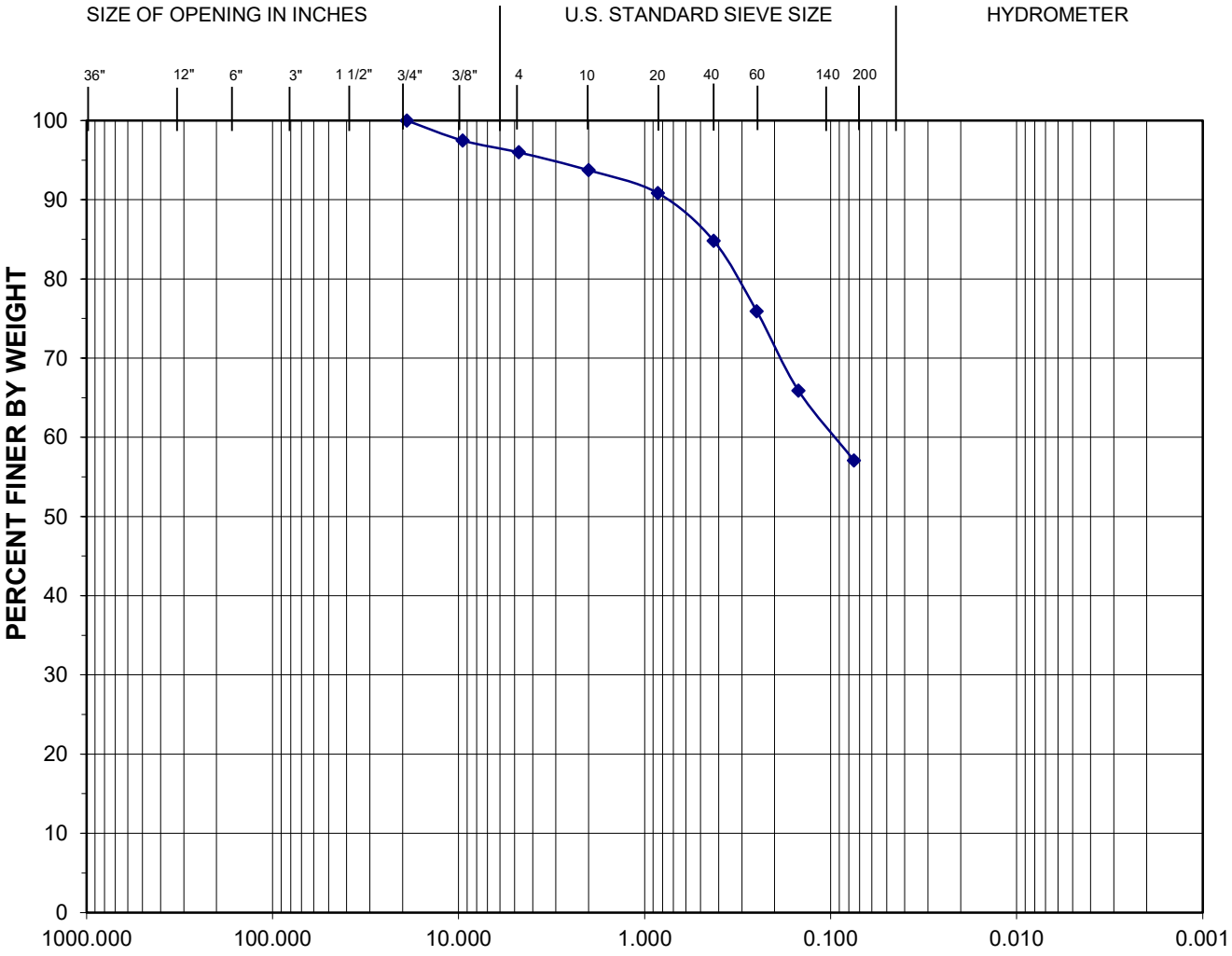
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-3	7.5'	13.0	64.5	sandy SILT, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 3035.01 DATE OF TESTING: 4/17/2026	PROJECT NAME: Clearview Switching Station

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

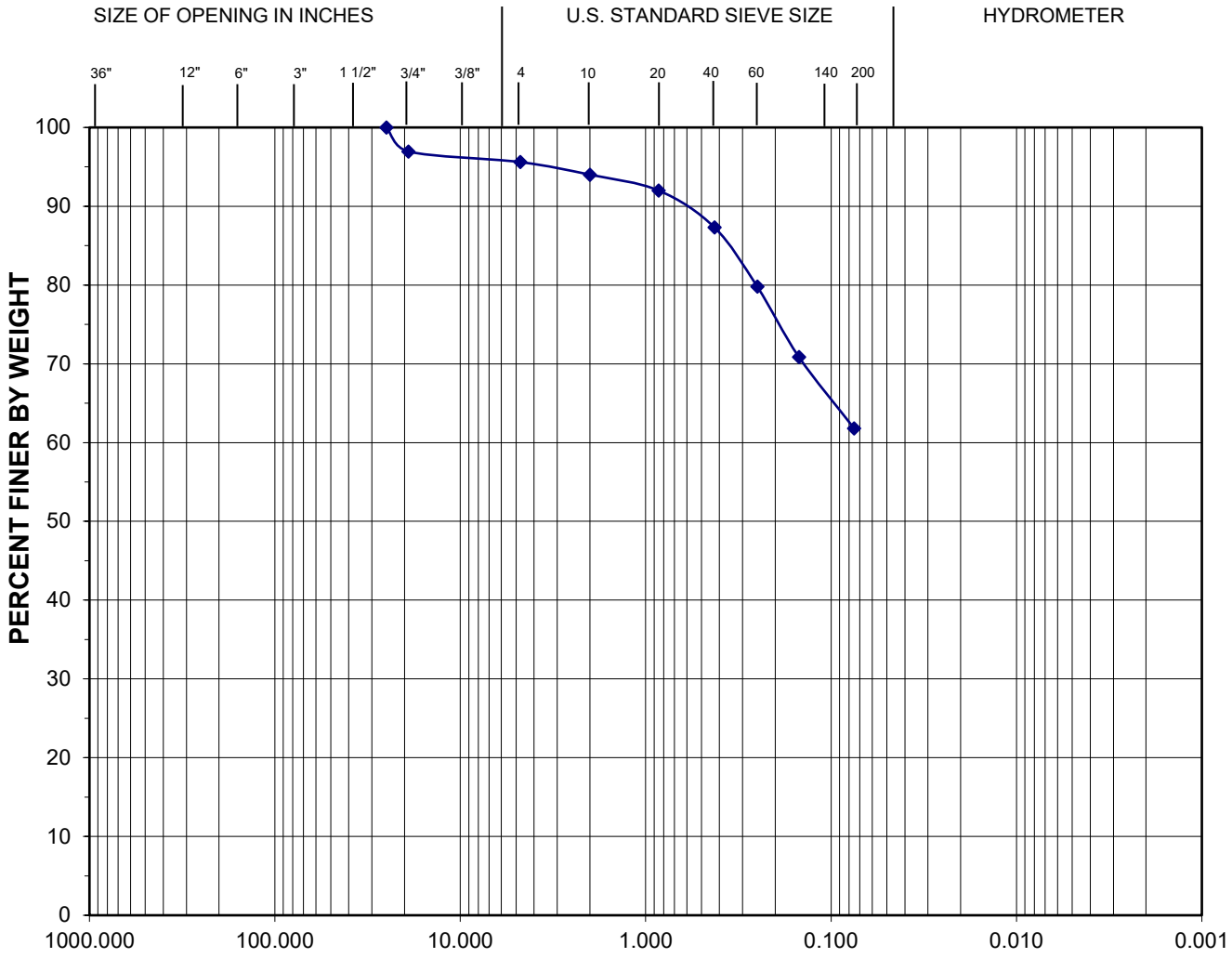
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-3	S-2	5'	14.3	57.1	sandy SILT, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 3035.01 DATE OF TESTING: 4/17/2026	PROJECT NAME: Clearview Switching Station

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

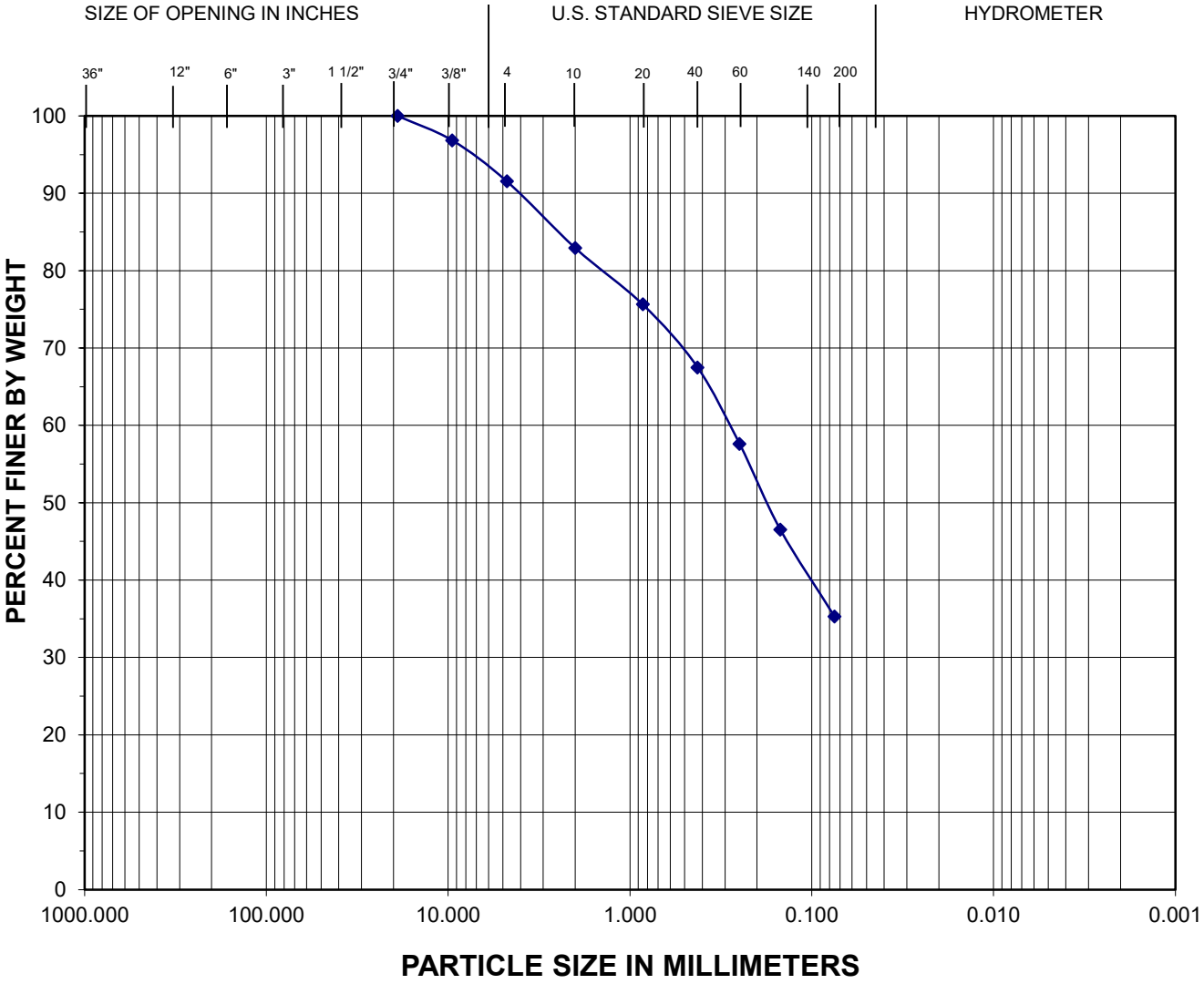
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-4	S-2	5'	13.1	61.8	sandy SILT, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 3035.01 DATE OF TESTING: 4/17/2026	PROJECT NAME: Clearview Switching Station

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

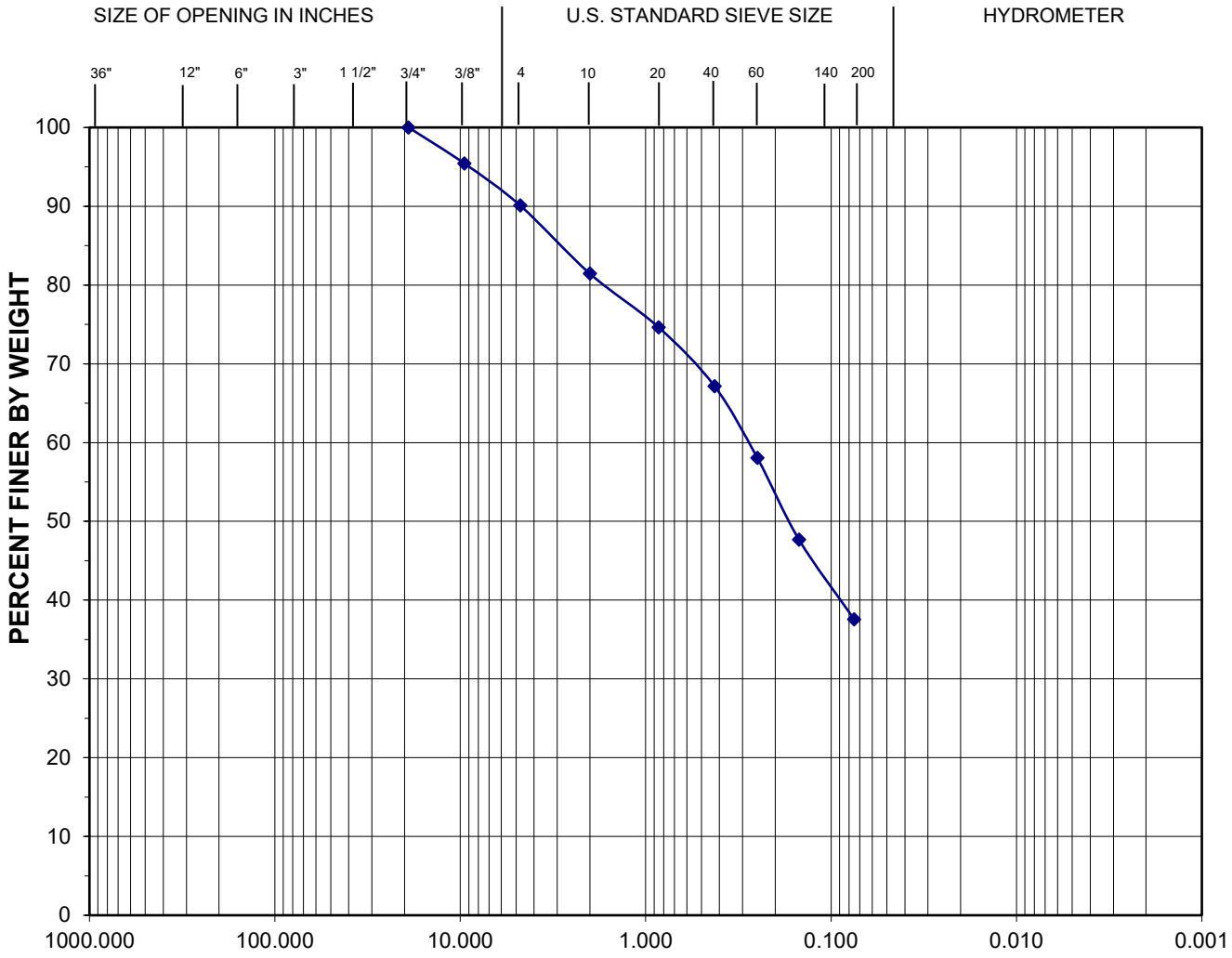
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
PIT-1	S-1	4	26.2	35.3	silty SAND, some gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 3035.01 DATE OF TESTING: 4/17/2026	PROJECT NAME: Clearview Switching Station

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
PIT-3	S-2	2'	28.0	37.6	silty SAND, some gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 3035.01 DATE OF TESTING: 4/17/2026	PROJECT NAME: Clearview Switching Station

April 21, 2026

NVL
INDUSTRIAL HYGIENE SERVICES
LABORATORY + MANAGEMENT + TRAINING

Martin Cross
Zipper Geo Associates, LLC
19019 36th Avenue West, Suite E
Lynnwood, WA 98036

RE: Bulk Asbestos Fiber Analysis; NVL Batch # 2606005.00

Client Project: 3035.01
Location: Clearview, WA

Dear Mr. Cross,

Enclosed please find test results for the 5 sample(s) submitted to our laboratory for analysis on 4/14/2026.

Examination of these samples was conducted for the presence of identifiable asbestos fibers using polarized light microscopy (PLM) with dispersion staining in accordance with **U. S. EPA 40 CFR Appendix E to Subpart E of Part 763**, Interim Method for the Determination of Asbestos in Bulk Insulation Samples and **EPA 600/R-93/116**, Method for the Determination of Asbestos in Bulk Building Materials.

For samples containing more than one separable layer of materials, the report will include findings for each layer (labeled Layer 1 and Layer 2, etc. for each individual layer). The asbestos concentration in the sample is determined by calibrated visual estimation.

For those samples with asbestos concentrations between 1 and 10 percent based on visual estimation, the EPA recommends a procedure known as point counting (NESHAPS, 40 CFR Part 61). Point counting is a statistically more accurate means of quantification for samples with low concentrations of asbestos.

The detection limit for the calibrated visual estimation is <1%, 400 point counts is 0.25% and 1000 point counts is 0.1%

Samples are archived for two weeks following analysis. Samples that are not retrieved by the client are discarded after two weeks.

Thank you for using our laboratory services. Please do not hesitate to call if there is anything further we can assist you with.

Sincerely,

Hilary Crumley, Manager Asbestos Laboratory

NVLAP
Testing
Lab Code: 102063-0

Enc.: Sample Results

Phone: 206 547.0100 | Fax: 206 634.1936 | Toll Free: 1.888.NVL.LABS (685.5227)
4708 Aurora Avenue North | Seattle, WA 98103-6516

Bulk Asbestos Fibers Analysis

By Polarized Light Microscopy

Client: Zipper Geo Associates, LLC
Address: 19019 36th Avenue West, Suite E
Lynnwood, WA 98036

Attention: Mr. Martin Cross
Project Location: Clearview, WA

Batch #: 2606005.00
Client Project #: 3035.01
Date Received: 4/14/2026
Samples Received: 5
Samples Analyzed: 5
Method: EPA/600/R-93/116

Lab ID: 260035486 **Client Sample #: B-1, S-1, 2.5'**
Location: Clearview, WA
Comments: Qualitative analysis was conducted for the presence of asbestos fibers in this sample.

Layer 1 of 1	Description: Loose brown crumbly material		
	Non-Fibrous Materials:	Other Fibrous Materials:%	**Asbestos Type: %**
	Binder/Filler, Fine particles, Mineral grains	Wood fibers	**None Detected ND**
	Gravel	Cellulose	

Lab ID: 260035487 **Client Sample #: B-2, S-1, 2.5'**
Location: Clearview, WA
Comments: Qualitative analysis was conducted for the presence of asbestos fibers in this sample.

Layer 1 of 1	Description: Loose brown crumbly material		
	Non-Fibrous Materials:	Other Fibrous Materials:%	**Asbestos Type: %**
	Binder/Filler, Fine particles, Mineral grains	Cellulose	**None Detected ND**
	Gravel		

Lab ID: 260035488 **Client Sample #: B-3, S-1, 2.5'**
Location: Clearview, WA
Comments: Qualitative analysis was conducted for the presence of asbestos fibers in this sample.

Layer 1 of 1	Description: Loose brown crumbly material		
	Non-Fibrous Materials:	Other Fibrous Materials:%	**Asbestos Type: %**
	Binder/Filler, Fine particles, Mineral grains	Cellulose	**None Detected ND**
	Gravel		

Lab ID: 260035489 **Client Sample #: B-4, S-1, 2.5'**
Location: Clearview, WA
Comments: Qualitative analysis was conducted for the presence of asbestos fibers in this sample.

Sampled by: Client
Analyzed by: Urooj Yousuf **Date:** 04/21/2026
Reviewed by: Hilary Crumley **Date:** 04/21/2026

Hilary Crumley, Manager Asbestos Laboratory

Note: If samples are not homogeneous, then subsamples of the components were analyzed separately. All bulk samples are analyzed using both EPA 600/R-93/116 and EPA 40 CFR Appendix E to Subpart E of Part 763 with the following measurement uncertainties for the reported % Asbestos (1%=0-3%, 5%=1-9%, 10%=5-15%, 20%=10-30%, 50%=40-60%). This report relates only to the items tested. If sample was not collected by NVL personnel, then the accuracy of the results is limited by the methodology and acuity of the sample collector. This report shall not be reproduced except in full, without written approval of NVL Laboratories, Inc. It shall not be used to claim product endorsement by NVLAP or any other agency of the US Government

Bulk Asbestos Fibers Analysis

By Polarized Light Microscopy

Client: Zipper Geo Associates, LLC
Address: 19019 36th Avenue West, Suite E
Lynnwood, WA 98036

Attention: Mr. Martin Cross
Project Location: Clearview, WA

Batch #: 2606005.00
Client Project #: 3035.01
Date Received: 4/14/2026
Samples Received: 5
Samples Analyzed: 5
Method: EPA/600/R-93/116

Layer 1 of 1 **Description:** Loose brown crumbly material

Non-Fibrous Materials:	Other Fibrous Materials:%	**Asbestos Type: %**
Binder/Filler, Fine particles, Mineral grains	Cellulose	**None Detected ND**
Gravel		

Lab ID: 260035490 **Client Sample #: B-5, S-1, 2.5'**

Location: Clearview, WA

Comments: Qualitative analysis was conducted for the presence of asbestos fibers in this sample.

Layer 1 of 1 **Description:** Loose brown crumbly material

Non-Fibrous Materials:	Other Fibrous Materials:%	**Asbestos Type: %**
Binder/Filler, Fine particles, Mineral grains	None Detected ND	**None Detected ND**
Gravel		

Sampled by: Client
Analyzed by: Urooj Yousuf **Date:** 04/21/2026
Reviewed by: Hilary Crumley **Date:** 04/21/2026

Hilary Crumley, Manager Asbestos Laboratory

Note: If samples are not homogeneous, then subsamples of the components were analyzed separately. All bulk samples are analyzed using both EPA 600/R-93/116 and EPA 40 CFR Appendix E to Subpart E of Part 763 with the following measurement uncertainties for the reported % Asbestos (1%=0-3%, 5%=1-9%, 10%=5-15%, 20%=10-30%, 50%=40-60%). This report relates only to the items tested. If sample was not collected by NVL personnel, then the accuracy of the results is limited by the methodology and acuity of the sample collector. This report shall not be reproduced except in full, without written approval of NVL Laboratories, Inc. It shall not be used to claim product endorsement by NVLAP or any other agency of the US Government

ASBESTOS LABORATORY SERVICES

Company	Zipper Geo Associates, LLC	**NVL Batch Number**	**2606005.00**
Address	19019 36th Avenue West, Suite E Lynnwood, WA 98036	**TAT** 5 Days	**AH** No
		Rush TAT	
Project Manager	Mr. Martin Cross	**Due Date** 4/21/2026	**Time** 1:15 PM
Phone	(425) 582-9928	**Email**	mcross@zippergeo.com
Cell	(425) 231-1856	**Fax**	(425) 582-9930

Project Name/Number: 3035.01 **Project Location:** Clearview, WA

Subcategory PLM Bulk

Item Code ASB-02 EPA 600/R-93-116 Asbestos by PLM <bulk>

Total Number of Samples 5 Rush Samples

	Lab ID	Sample ID	Description	A/R
1	260035486	B-1, S-1, 2.5'		A
2	260035487	B-2, S-1, 2.5'		A
3	260035488	B-3, S-1, 2.5'		A
4	260035489	B-4, S-1, 2.5'		A
5	260035490	B-5, S-1, 2.5'		A

	Print Name	Signature	Company	Date	Time
Sampled by	Client				
Relinquished by	Client				

Office Use Only	Print Name	Signature	Company	Date	Time
Received by	Fatima Khan		NVL	4/14/26	1315
Analyzed by	Urooj Yousuf		NVL	4/21/26	
Results Called by					
☐ **Faxed** ☐ **Emailed**					

Special Instructions:

Date: 4/14/2026
Time: 3:33 PM
Entered By: Michelle Seidl

2606005

NVL
INDUSTRIAL HYGIENE SERVICES
LABORATORY • MANAGEMENT • TRAINING

ASBESTOS CHAIN OF CUSTODY

Turn Around Time: Standard
- ☐ 1 Hour
- ☐ 2 Hours
- ☐ 4 Hours
- ☐ 24 Hours
- ☐ 2 Days
- ☐ 3 Days
- ☐ 4 Days
- ☒ 5 Days
- ☐ 10 Days

Please call for TAT less than 24 Hours

Company: ZGA
Address:
Phone:
Project Manager: Marty Cross
Cell: (425) 231-1856
Email: mcross@zippergeo.com
Fax: ( ) -

Project Name/Number: 3035.01
Project Location: Clearview, WA

- ☐ PCM Air (NIOSH 7400)
- ☒ PLM (EPA 600/R-93-116)
- ☐ PLM Gravimetry (600/R-93-116)
- ☐ Asbestos Friable/Non-Friable (EPA 600/R-93/116)
- ☐ TEM (NIOSH 7402)
- ☐ EPA 400 Points (600/R-93-116)
- ☐ Asbestos in Vermiculite (EPA 600/R-04/004)
- ☐ TEM (AHERA)
- ☐ Other
- ☐ TEM (EPA Level II Modified)
- ☐ EPA 1000Points (600/R-93-116)
- ☐ Asbestos in Sediment (EPA 1900 Points)

Reporting Instructions
☐ Call ( ) - ☐ Fax ( ) - ☐ Email mcross@zippergeo.com

Total Number of Samples

	Sample ID	Description	A/R
1	B-1, S-1, 2.5'	Brown silty sand	
2	B-2, S-1, 2.5'	"	
3	B-3, S-1, 2.5'	"	
4	B-4, S-1, 2.5'	"	
5	B-5, S-1, 2.5'	"	
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			

	Print Name	Signature	Company	Date	Time
Sampled by	Kaelin Newman	KN	ZGA	4/14/26	13:15
Relinquish by	Kaelin Newman	KN	ZGA	4/14/26	13:15

Office Use Only

	Print Name	Signature	Company	Date	Time
Received by	Ahmadian		NVL Labs	4/14/26	1:15p
Analyzed by					
Called by					
Faxed/Email by					

APPENDIX C
TERRAPHASE ENGINEERING INC. TEST PIT LOGS

Test Pit 1

Date: 3/21/2024
Logged By: JHA/GHK

Location: Parcel No. 27051400101400
Snohomish County, WA

Depth (ft.)	Soil Description	USC	View of Test Pit 1
0.0 - 0.5	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM	
0.5 - 3.3	Light brown silty fine to medium sand with trace gravel (medium dense, moist) (**Weathered Till**)	SM	
3.3 - 4.5	Gray silty fine to coarse sand with gravel (very dense, moist) (**Glacial Till**)	SM	

Notes

- Test pit completed at 4.5 feet
- Groundwater was not observed
- Samples collected at 2.5 and 4.5 feet

Woodinville
17625 - 130th Avenue NE, Suite 102
Woodinville, Washington 98072
425.488.0599

terraphase engineering

Clearview Property
W159.001.001
Figure 4

Test Pit 2

Date: 3/21/2024
Logged By: JHA/GHK

Location: Parcel No. 27051400101400 Snohomish County, WA

Depth (ft.)	Soil Description	USC
0.0 - 0.5	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM
0.5 - 3.1	Light brown silty fine to medium sand with trace gravel (medium dense, moist) (Moisture Content = 33% (**Weathered Till**)	SM
3.1 - 3.6	Gray silty fine to coarse sand with gravel (very dense, moist) (**Glacial Till**)	SM

View of Test Pit 2



Notes

- Test pit completed at 3.6 feet
- Groundwater was not observed
- Samples collected at 2.5 feet

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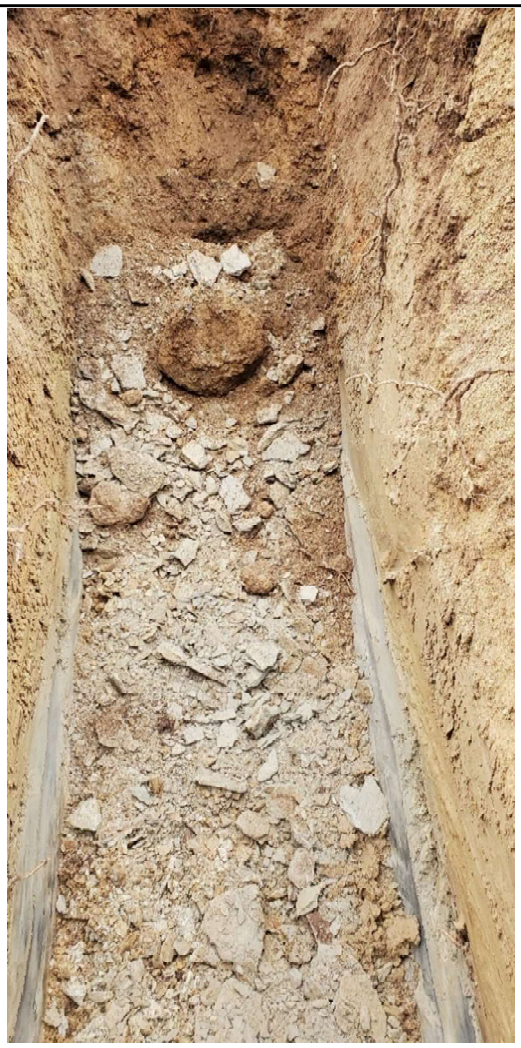
terraphase engineering

Clearview Property
W159.001.001
Figure 5

Test Pit 3

Date: 3/21/2024
Logged By: JHA/GHK

Location: Parcel No. 27051400101400
Snohomish County, WA

Depth (ft.)	Soil Description	USC	View of Test Pit 3
0.0 - 0.6	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM	
0.6 - 3.7	Light brown silty fine to medium sand with trace gravel (medium dense, moist) (**Weathered Till**)	SM	
3.7 - 4.3	Gray silty fine to coarse sand with gravel (very dense, moist) (**Glacial Till**)	SM	

Notes

- Test pit completed at 4.3 feet
- Groundwater was not observed
- Samples collected at 2.0, and 4.3 feet

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Clearview Property
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Figure 6

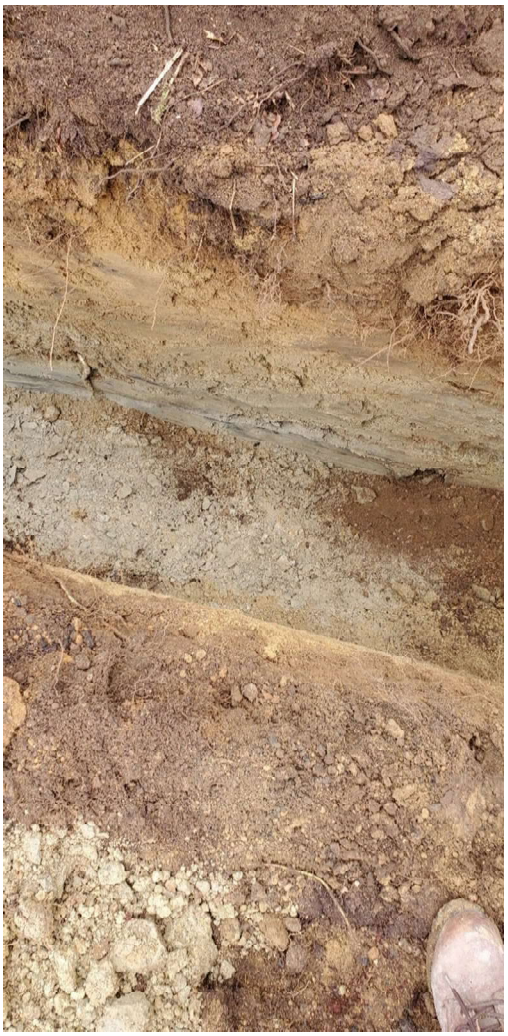
Test Pit 4

Date: 3/21/2024
Logged By: JHA/GHK

Location: Parcel No. 27051400101400
Snohomish County, WA

Depth (ft.)	Soil Description	USC
0.0 - 0.4	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM
0.4 - 3.4	Light brown silty fine to medium sand with trace gravel (medium dense, moist) (**Weathered Till**)	SM
3.4 - 3.7	Gray silty fine to coarse sand with gravel (very dense, moist) (**Glacial Till**)	SM

View of Test Pit 4



Notes

- Test pit completed at 3.7 feet
- Groundwater was not observed
- Samples collected at 1.8 and 3.7 feet

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Clearview Property
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Figure 7