GEOTECHNICAL ENGINEERING REPORT
JENNINGS PARK SUBSTATION
7728 & 7808 – 47th Avenue NE
Marysville, Washington

Project No. 2494.01
10 February 2023

Prepared for:
Snohomish County PUD No. 1

Prepared by:

ZipperGeo

Zipper Geo Associates, LLC
19019 36th Avenue W., Suite E
Lynnwood, WA 98036

ZipperGeo
Geoprofessional Consultants

Project No. 2494.01
10 February 2023

Snohomish County PUD No. 1
Distribution & Engineering Services Division, PO Box 1107
Everett, Washington 98206-1107

Attention: Mr. Will Blanchard, PE, Professional Engineer

Subject: Geotechnical Engineering Report
Jennings Park Substation
7728 & 7808 – 47th Avenue NE
Marysville, Washington

Dear Mr. Blanchard:

In accordance with your request, Zipper Geo Associates, LLC (ZGA) has completed the subsurface exploration and geotechnical engineering evaluation for the proposed Jennings Park Substation. This report presents the findings of the subsurface exploration and geotechnical recommendations for the project. Our work was completed in general accordance with the scope of services described in Contract No. CW2245207. Written authorization to proceed was provided by the District on 23 August 2021. 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

Martin Cross, GIT
Staff Geologist

DAVID C. WILLIAMS

Signed 2.10.23

David C. Williams, LG, LEG
Principal Engineering Geologist

Signed 2.10.23

Robert A. Ross, P.E.
Managing Principal

Distribution: Addressee (1 electronic)
Cover photo courtesy Google Earth

19019 36th Avenue West, Suite E **Lynnwood, WA 98036** **(425) 582-9928**

TABLE OF CONTENTS

FIGURES

APPENDICES

GEOTECHNICAL ENGINEERING REPORT
JENNINGS PARK SUBSTATION
7728 & 7808 – 47th AVENUE NE
MARYSVILLE, WASHINGTON
Project No. 2494.01
10 February 2023

INTRODUCTION

This report summarizes the geotechnical engineering exploration and analysis completed for the proposed Jennings Park Substation project in Marysville, Washington. Seven borings (B-1 through B-7), six test pits (TP-1 through TP-6), and one cone penetrometer (CPT-1) were completed by ZGA to depths ranging from approximately 4.5 to 51.5 feet below the existing ground surface to evaluate subsurface conditions. Descriptive logs of the explorations are included in Appendix A while Appendix B contains a summary of laboratory testing procedures and results.

PROJECT INFORMATION

Site Location

The project property consists of two adjoining parcels located to the west of 47th Avenue NE. The new substation is proposed for construction on the undeveloped parcel at 7808 – 47th Avenue NE. This parcel, historically known as the Goetz parcel, has approximate dimensions of 385 feet east-west and 235 to 240 feet north-south (roughly 2.2 acres). The substation parcel is about 390 feet west of 47th Avenue NE and is accessed via a gravel-surfaced driveway in the north portion of an adjoining property known as the Jensen parcel at 7728 – 47th Avenue NE. Developed commercial and multi-family residential properties adjoin the parcels except to the south of the substation site which is currently undeveloped. The business Chet's Cabinets occupies about the southern half of the Jensen parcel. The primary site and the west portion of the driveway are illustrated on the *Site and Exploration Plan*, Figure 1. The eastern portion of the driveway and a portion of 47th Avenue NE to the north are illustrated on the Site and Exploration Plan, Figure 2.

Project Description

A new double bank substation is proposed for construction on the site. At the time this report was prepared, the proposed substation construction would not include the southern portion of the Jensen parcel. Site improvements on the Goetz parcel at the west are expected to include:

- Dead end towers (termination structures) in the eastern portion of the yard.
- Circuit switchers, disconnect switches, neutral reactors, termination structures, and bus supports.
- Two slab-supported switchgear enclosures.

- Two slab-supported transformers.

- Below-grade conduits and pre-cast concrete vaults in the yard and driveway.

- Structural fill placement to achieve a yard finished grade of 46 feet.

- New transmission poles are planned for construction at the southeast corner of the yard and along 47th Avenue NE.

SITE HISTORY

According to a Phase I Environmental Site Assessment report, dated 7 September 2012 and prepared by Terracon Consultants, Inc. (TCI), the Goetz parcel has been undeveloped since at least 1941. The Chet's Cabinets business was constructed on the Jensen parcel in 1973, and the other nearby commercial properties were developed starting in the 1980s. Two cellular communication compounds were constructed at the west end of the driveway circa the 1990s.

SITE CONDITIONS

Surface Conditions
The substation site is a relatively level area with ground surface elevations ranging from a low of 40 feet at the west to 44 feet at the east according to a topographic survey of the site provided for our review and our site observations. The slight elevation variation is likely due to limited historical grading as we observed fill material at some of the boring and test pit locations. The site is predominantly mantled with grasses and weeds, although trees are present along the east boundary. We did not observe standing or flowing surface water on site during our visits in September and October 2021, but we did observe several puddles on the substation site in November shortly following several days of significant rain.

The gravel driveway extending east of the substation site to 47th Avenue NE is approximately 30 feet wide and rises very gently toward the street with ground surface elevations of approximately 44 to 46 feet from the west to east, respectively. The driveway is surfaced with fine gravel-size crushed rock and contains underground electrical and communication utilities based on utility locate marks that we observed.

The east side of 47th Avenue NE, where the new transmission poles will be located, includes both paved and unpaved shoulder areas commonly used for parking. The shoulder area where we advanced boring B-7 near a proposed transmission pole location included underground storm sewer, natural gas, and water utilities according to locate marks that we observed.

ZipperGeo
Geoprofessional Consultants

Local Geologic Conditions

We assessed the geologic setting of site and the surrounding vicinity by reviewing the *Geologic Map of the Marysville Quadrangle, Snohomish County, Washington* (US Geological Survey, Map MF-1743, 1985). The published geologic mapping indicates the site is underlain by Vashon Recessional Outwash, Marysville Sand Member. The Marysville Sand is described as mostly well-drained, stratified to massive outwash sand, some fine gravel, and some areas of silt and clay. The sediments were deposited by melt water flowing south from the stagnating and receding Vashon glacier. The outwash is reported to have a maximum thickness of about 140 feet. Subsurface conditions disclosed by the explorations advanced by ZGA and others are consistent with the published mapping. Some of the borings and test pits disclosed undocumented fill material above the native soils.

Soil Conditions

The soil descriptions presented below have been generalized for ease of report interpretation. Please refer to the exploration logs for detailed soil descriptions at the exploration locations. Variations in subsurface conditions may exist between the exploration locations and the nature and extent of variations between the explorations may not become evident until additional explorations are completed or until construction. Undocumented fill material is present and it should be recognized that the nature of undocumented fill material is such that its composition and depth may vary over relatively short distances. Subsurface conditions at specific locations are summarized below.

Subsurface conditions were evaluated using a combination of six test pits, seven borings, and one cone penetrometer test (CPT). Borings B-1 through B-5 were advanced in the future substation yard. Boring B-6 was advanced through the driveway connecting the project site to 47th Avenue NE, and boring B-7 was advanced in 47th Avenue NE, located to the west of the existing dental office at 7825 - 47th Ave NE. The six test pits were excavated in the substation yard and cone CPT-1 was advanced approximately near the center of the yard. Approximate exploration locations, as well as pertinent surface features, are shown on Figures 1 and 2. Observed soil conditions are summarized below.

Surficial Organic Topsoil

The explorations disclosed about 2 to 10 inches of topsoil consisting of dark brown, silty sand with fine roots and fine organic matter. Fine roots were observed extending to about 1 foot below grade. The topsoil thickness should be expected to vary across the site.

Fill

We observed undocumented fill material consisting of brown to dark brown, silty sand with some gravel and trace cobbles to gravelly sand with some silt, and trace cobbles, extending to depths of approximately

1.75 to 2.25 feet at the test pit TP-5 and TP-6 locations, respectively. The coarse sand to cobble size material consisted of crushed rock. We observed undocumented fill material consisting of dark brown, brown and orange-brown, silty sand to sand with some silt, and a varying gravel content, extending to depths of approximately 2.5 to 3.3 feet at the boring B-1, B-2, B-4, and B-5 locations, respectively. The coarse sand and gravel size material observed in the upper 3.3 feet of boring B-5 consisted of crushed rock. We observed a thin relic topsoil horizon at approximately 2.5 feet in boring B-4. We observed undocumented fill material consisting of orange-brown to brown sand with some silt and a varying gravel content, extending to a depth of approximately 2.5 feet at boring B-6 in the crushed gravel driveway. We observed undocumented fill material consisting of crushed gravel over orange-brown sand with gravel and some silt, extending to a depth of approximately 2.5 feet at boring B-7.

Please note that the nature of undocumented fill is such that its composition and thickness can vary over relatively short distances. We submitted five samples of the fill material to an analytical laboratory in order to test for the presence of asbestos-containing material. The test results were negative.

Recessional Outwash

The test pits disclosed that the shallow native recessional outwash soils consisted of very loose to medium dense sand with a low silt and gravel content. The soils above the water table were generally in a moist condition. The test pits were terminated at relatively shallow depths of approximately 6.5 to 8.5 feet due to caving associated with the relatively low density and low fines content of the material in combination with shallow groundwater conditions.

The deeper recessional deposits as disclosed by CPT-1 consist of medium dense sand with a variable silt content to approximately 30 feet with dense sand, silty sand, and sandy silt to about 45 feet. Between about 45 and 50 feet (the CPT-1 termination depth), the density dropped off to medium dense and included a thin horizon of stiff sandy silt to clayey silt. Boring B-1 disclosed somewhat similar conditions, with medium dense sand with a variable silt content and discrete silt horizons to about 42 feet with very stiff sandy silt to the boring's approximately 51.5 foot termination depth.

Groundwater

We observed groundwater seepage at depths of approximately 4 to 6 feet while excavating the test pits and at approximately 3 feet while advancing boring B-1. We observed groundwater at depths of approximately 4.5 to 6 feet while advancing borings B-2 through B-5, and at approximately 6.5 feet while advancing boring B-7 along the east side of 47th Avenue NE, near the proposed location of a new transmission pole. We did not encounter groundwater while advancing boring B-6 in the driveway.

We installed a groundwater observation well at the boring B-3 location following completion of drilling and sampling. Groundwater measurements in the well subsequent to drilling and the well installation are summarized in the table below. It should be noted that groundwater conditions will likely vary seasonally and in response to precipitation events, land use, and other factors.

Table 1: Boring B-3 Groundwater Monitoring Well Observations			
Date	**10.27.21**	**11.11.21**	**11.17.21**
Groundwater Depth/Elevation (feet)	4.5 / 37.5	1.7 / 40.3	0.5 / 41.5
Date	**4.1.22**	**6.16.22**	**9.27.22**
Groundwater Depth/Elevation (feet)	1.8 / 40.2	1.86 / 40.14	4.3 / 37.7
*Groundwater depth measured relative to the rim of the flush-mount well monument. *Monument ground surface elevation approximately 42 feet			

CONCLUSIONS AND RECOMMENDATIONS

General Geotechnical Considerations

Based on information gathered during the field exploration, laboratory testing, and analysis, we conclude that construction of the proposed improvements is feasible from the 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.

- The native recessional outwash soils are generally favorable from the site grading and shallow foundation support perspectives. Selective removal of the existing undocumented fill material and underlying relic topsoil from below foundations, slabs, and vaults is recommended.

- Re-use of the existing non-organic native soil during grading will be feasible provided that the soil moisture content can be adequately controlled prior to compaction. The native soil has a low gravel content, and applications requiring a higher gravel content than typifies the native soils will necessitate selective import of aggregates.

- We anticipate that most excavations for foundations, vaults, and conduits will encounter groundwater, most likely necessitating dewatering during construction. Raising site grade to the extent feasible will help to reduce groundwater intrusion into the excavations and the dewatering magnitude.

- The granular nature of the shallow recessional outwash soils is favorable from the stormwater infiltration perspective.

- Our analysis indicates that the site soils between approximately 10 and 30 feet, and below about 45 feet, will likely liquefy during the IBC-defined seismic event. This may yield between about 3-1/2 and 4-1/2 inches of total settlement with differential settlement over 40 feet approximating half the total settlement. Based on our analysis, it appears that settlement associated with typical substation foundations due to liquefaction accompanying the design seismic event will likely be considered acceptable without the need for deep foundations or extensive ground improvement.

Geotechnical engineering recommendations for site grading, drainage, foundations, and other geotechnically-related aspects of the project are presented in the following sections. The recommendations contained in this report are based upon the results of and the field exploration, laboratory testing, engineering analyses, review of historical documents, and our current understanding of the proposed project design. ASTM and WSDOT specification codes cited herein refer to the current manual published by the American Society for Testing & Materials and the current edition of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (Publication M41-10).

Geologic Hazard Areas

Article IV of Chapter 22E of the Marysville Municipal Code (MMC) regulated geologic hazard areas as defined in Chapter 22A.020:

"Geologic hazard areas" means lands or areas characterized by geologic, hydrologic and topographic conditions that render them susceptible to potentially significant or severe risk of landslides, erosion, or seismic activity. It should be noted that the project site is not mapped as within, or near, any designated geologic hazard areas on the City of Marysville *Geologic Hazards* map, dated May 2014.

<u>Erosion Hazard Areas</u>

"Erosion hazard areas" means lands or areas that, based on a combination of slope inclination and the characteristics of the underlying soils, are susceptible to varying degrees of risk of erosion. Erosion hazard areas are classified as low hazard, moderate hazard and high hazard, based on the following criteria:

(1) Low Hazard. Areas sloping less than 15 percent.

(2) Moderate Hazard. Areas sloping between 15 and 40 percent and underlain by soils that consist predominantly of silt, clay, bedrock or glacial till.

(3) High Hazard. Areas sloping between 15 and 40 percent that are underlain by soils consisting largely of sand and gravel, and all areas sloping more steeply than 40 percent.

The project site is essentially level and lacks significant slopes, certainly lacking slopes 15 percent or steeper. It is our opinion that the site presents a low erosion hazard per the MMC definition.

<u>Landslide Hazard Areas</u>

"Landslide hazard areas" means areas that, due to a combination of slope inclination and relative soil permeability, are susceptible to varying degrees of risk of landsliding. Landslide hazard areas are classified as Classes I through IV based on the degree of risk as follows:

(1) Low Hazard. Areas with slopes of less than 15 percent.

(2) Moderate Hazard. Areas with slopes of between 15 and 40 percent and that are underlain by soils that consist largely of sand, gravel, bedrock or glacial till.

(3) High Hazard. Areas with slopes between 15 percent and 40 percent that are underlain by soils consisting largely of silt and clay, and all areas sloping more steeply than 40 percent.

(4) Very High Hazard. Areas with slopes over 40 percent and areas of known mappable landslide deposits.

As described above, the project site is essentially level and lacks significant slopes, including slopes 15 percent or steeper. It is our opinion that the site presents a low landslide hazard per the MMC definition.

Seismic Hazard Areas

"Seismic hazard areas" means areas that, due to a combination of soil and ground water conditions, are subject to severe risk of ground shaking, subsidence or liquefaction of soils during earthquakes. These areas are typically underlain by soft or loose saturated soils (such as alluvium), have a shallow ground water table and are typically located on the floors of river valleys. Seismic hazard areas are classified as follows:

(1) Low Hazard. Areas underlain by dense soils or bedrock.

(2) High Hazard. Areas underlain by soft or loose saturated soils.

Based upon our analysis, it appears that the site meets the MMC criteria for a High Hazard area due to the potential for liquefaction-induced settlement, as described in the following sections. We evaluated the seismic performance of the site relative to hazards resulting from ground shaking associated with a design seismic event with a 2,475-year return period determined in accordance with the 2018 International Building Code (IBC) and the American Society of Civil Engineers Standard 7-16 (ASCE 7-16). Conformance to the above criteria for seismic excitation does not constitute any kind of guarantee or assurance that significant structural damage will not occur if a maximum level earthquake occurs. The primary goal of the IBC seismic design procedure is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a building or structure may be damaged beyond repair, yet not collapse.

Ground Fault Rupture: The USGS Quaternary Fault Web Mapping Application indicates that the site is about 12 miles northeast of the South Whidbey Island Fault Zone and about 21 miles southeast of the Utsalady Point Fault. Based on the location of the mapped fault zones relative to the project site, it is our opinion that the risk of ground surface rupture at the site is low and does not require mitigation.

Landsliding: Based on the relatively level topography of the site and surrounding vicinity, it is our opinion that the risk of earthquake-induced landsliding is low and does not require mitigation.

Liquefaction: Liquefaction is a phenomenon wherein saturated cohesionless soils build up excess pore water pressures during earthquake loading. Liquefaction typically occurs in loose soils, but may occur in denser soils if the ground shaking is sufficiently strong. ZGA completed a liquefaction analysis in general accordance with Section 1803.5.12 of the 2018 IBC and Section 11.8.3 of ASCE 7-16. Specifically, our analysis used the following primary seismic ground motion parameters.

- A Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration of 0.472g, based on Figure 22-9 of ASCE 7-16.

- A Site Modified Peak Ground Acceleration (PGA_M) of 0.532g based on Site Class D, per Section 11.8.3 of ASCE7-16 (Site Class modification to MCE_G without regard to liquefaction in accordance with Sections 11.4.8 and 20.3.1 of ASCE 7-16).

- A Geometric Mean Magnitude of 7.08 based on 2014 USGS National Seismic Hazard Mapping Project deaggregation data for a seismic event with a 2% probability of exceedance in 50 years (2,475 year return period).

Our liquefaction analysis was completed using the computer program LiquefyPro Version 5.8 using the modified Robertson method for CPT data. Our analysis was based on CPT-1 completed to a depth of about 50 feet below existing grade. The approximate exploration location is shown on the enclosed *Site and Exploration Plan, Figure 1*. Our analysis indicates the potential for liquefaction at depths ranging from about 10 to 30 feet and greater than about 45 feet below grade.

Liquefaction Settlement: Based on our analyses, we estimate a total seismic settlement of approximately 3½ to 4½ inches. We estimate a differential seismic settlement of approximately 1¾ to 2¼ inches over a horizontal distance of 40 feet.

Lateral Spread: Lateral spreading is a phenomenon in which soil deposits which underlie a site can experience significant lateral displacements associated with the reduction in soil strength caused by soil liquefaction. This phenomenon tends to occur most commonly at sites where the soil deposits can flow toward a “free-face”, such as a water body. Our evaluation did not identify a nearby free face condition. We also evaluated the potential for lateral spread using the Liquefaction Severity Index (LSI) method developed by Youd and Perkins (1987). This method evaluates earthquake magnitude and the horizontal distance from the surface projective of the energy source to generate an LSI index value of 1 to 100, with 1 being a very low risk and 100 being a very high risk of lateral spread. Our evaluation indicates a site LSI value of about 1. Given the site LSI value and the lack of a free face condition, it is our opinion that the potential for lateral spread is low and does not require mitigation.

Earthwork

The following sections present recommendations for site preparation, subgrade preparation, and placement of engineered fills on the project. The recommendations presented in this report for design and construction of foundations and slabs are contingent upon following the recommendations outlined in this section.

Earthwork on the project should be observed and evaluated by a ZGA representative. Evaluation of earthwork should include observation and testing of structural fill, subgrade preparation, foundation bearing soils, deep foundations, and subsurface drainage installations.

Site Preparation

Stripping: In preparation for grading we recommend removal of all existing surficial vegetation and deleterious debris such as trash, small amounts of which we observed. These materials should be wasted away from the substation and access road areas.

Existing Fill Removal: Site preparation is recommended to include selective removal of existing undocumented fill material containing substantial organics or deleterious debris and any relic organic topsoil from within the yard below structure and conduit run locations.

Variation in the fill depth and composition, and the depth of relic topsoil below the fill, should be expected. These materials should be evaluated during construction and removed as necessary under the observation of a ZGA representative. Our representative will identify unsuitable materials that should be removed and possibly some that may be re-used as structural fill. The existing undocumented fill in the open areas of the yard (not below foundations, slabs, or conduit runs) and with no more than about 3 percent organic material and lacking deleterious material may be left in place.

The resultant excavations should be backfilled in accordance with the subsequent recommendations for structural fill placement and compaction. Specific recommendations regarding removal of existing fill material at foundation and slab locations are provided subsequently in association with foundation design and construction recommendations.

Site Preparation and Grading Scheduling: Most of the native soils likely to be exposed during grading consist of sand with a relatively low fines content. It will be feasible from the geotechnical perspective to grade these soils under a relatively wide weather band, although even with favorable granular soils it may be difficult or impossible to grade the site during very wet weather. If this is a concern with the District, we recommend that site preparation and grading take place in the drier summer and early fall months if possible. Completion of site preparation and grading under drier site and weather conditions will reduce the potential for disturbance of some of the moisture-sensitive soils and the need to replace disturbed soils with imported fill material. Completing the work during the drier summer and early fall months will also allow the grading to coincide with the seasonal low groundwater condition and this would reduce the extent of construction dewatering.

Structural Fill Placement and Compaction

Establishing a yard elevation of 46 feet will require placing about 3 to 5 feet structural fill. Structural fill will also be placed for conduit and vault installations, storm drainage piping and structures, 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, and we expect that this will locally be the case given the nature of the native recessional outwash soils, the soils below foundation, slab, vault, and entry drive 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 condition 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.

The suitability of soils for use as structural fill depends primarily on the gradation and moisture content of the soil when it is placed. As the amount of fines (that soil fraction passing the US No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult, or impossible, to achieve. Generally, soils containing more than about 5 percent fines by weight (based on that soil fraction passing the US No. 4 sieve) cannot be compacted to a firm, non-yielding condition when the moisture content is more than a few percent from optimum. The optimum moisture content is that which yields the greatest soil density under a given compactive effort.

Re-use of On-site Soils: Soil expected to be encountered in excavations include predominantly native soil typically consisting of sand with a variable silt content and some undocumented fill consisting of sand and gravelly sand with some cobbles and variable silt content with some organics. We collected seven native soils samples from depths of about 3 to 7 feet. Six of those samples had a fines content of less than 2 percent while the seventh has a fines content of about 9 percent. Overall, the native recessional outwash is well-suited for use as structural fill. Please note that some of the fill material contains a relatively high silt content. Using these materials as structural fill could be difficult due to the high fines content and moisture sensitivity.

Imported Structural Fill: We recommend that structural fill consist of a 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 Import Granular Fill Gradation	
US Standard Sieve Size	**Percent Passing by Dry Weight Basis**
2 inch	100
½ inch	56 - 100

Table 2: Snohomish County PUD No. 1 Substation Import Granular Fill Gradation	
US Standard Sieve Size	**Percent Passing by Dry Weight Basis**
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. Another advantage of using granular fill with a relatively low fines content is that it will drain better than fill with a higher fines content. 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 1.5-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 have 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 and yard entry to facilitate successful stormwater infiltration.

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 Soil Compaction Levels	
Location	**Minimum Percent Compaction***
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

* ASTM D 1557 Modified Proctor Maximum Dry Density

Earthwork may be difficult or impossible during periods of elevated soil moisture and wet weather. If soils are stockpiled for future use and wet weather is anticipated, the stockpile should be protected with plastic sheeting that is securely anchored.

Subgrade soils that become disturbed due to elevated moisture conditions should be overexcavated to expose firm, non-yielding, non-organic soils and backfilled with compacted structural fill. We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through June) it will be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork may require additional mitigative measures beyond that which would be expected during the drier summer and fall months. This could include diversion of surface runoff around exposed soils and draining of ponded water. Once subgrades are established, it will be necessary to protect the exposed subgrade soils from construction traffic during wet weather. Placing quarry spalls or crushed recycled concrete over these areas would further protect the soils from construction traffic.

If earthwork takes place during freezing conditions, we recommend allowing the exposed subgrade to thaw and then recompacting the subgrade prior to placing subsequent lifts of engineered fill. Frozen soil should not be used as structural fill.

We recommend that a ZGA representative be present during the construction phase of the project to observe earthwork operations and to perform necessary tests and observations during subgrade preparation, placement and compaction of structural fill, backfilling of excavations, and prior to construction of foundations and slabs.

<u>Drainage:</u> 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.

<u>Additional Considerations:</u> It is anticipated that excavations for the proposed improvements can be accomplished with conventional earthmoving equipment.

<u>Excavation Quantities</u>: 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 yards 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.

Utility Installation Recommendations

Below-grade utilities are expected to include conduits and storm drain 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 shallow native and fill soils in the substation footprint are generally expected to be adequate for support of utilities.

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 utility trench backfill exclusive of 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 1 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.

Dewatering: Depending upon the time of year that the work takes place and the depth of the utilities, groundwater seepage should be expected in excavations and certainly during the wetter time of year. Seepage could be heavy enough to require temporary dewatering measures and flattening the sidewalls of excavations to reduce the risk of caving. The contractor should be prepared to pump water from excavations into either a nearby storm or sanitary sewer or Baker tank. Dewatering water discharged from the site will likely need to comply with permit requirements issued by the City of Marysville. We recommend that dewatering effectively lower the water table at least 2 feet below the bottoms of excavations until they are backfilled.

Temporary Excavation Slopes: We recommend that utility trenching, installation, and backfilling conform to all applicable Federal, State, and local regulations such as WISHA and OSHA regulations for open excavations. In order to maintain the function of any existing utilities that may be located near excavations, we recommend that temporary excavations not encroach upon the bearing splay of existing utilities, foundations, or slabs. The bearing splay of structures and utilities should be considered to begin at the edge of the utility, foundation, or slab and extend downward at a 1.5H:1V (Horizontal:Vertical) slope under fully drained conditions. Much shallower temporary slope inclinations will be required under saturated soil conditions. If, due to space constraints, an open excavation cannot be completed without encroaching on a utility, we recommend shoring the new utility excavation with a slip box or other suitable means that provide for protection of workers and that maintain excavation sidewall integrity to the depth of the excavation.

Temporary slope 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 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. We do not recommend vertical slopes or cuts deeper than 4 feet if worker access is necessary. The cuts should be adequately sloped or supported to prevent injury to personnel from local sloughing and spalling. The excavation should conform to applicable Federal, State, and local regulations.

Based upon our review of WAC Chapter 296-155-66401 (Appendix A – Soil Classification), we have interpreted the soils disclosed by the explorations and likely to be present in most excavations as consistent with the Type C definition. 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. Please note that the shallow granular soils have a low fines content and that unsupported excavation sidewalls in these soils may slough or cave readily.

Below-grade Vault Recommendations

Bearing Conditions: Below-grade conduit vaults will be installed as part of the project. Based upon our experience with other District substations, and depending on the orientation of the new conduit sweeps, the vault bases may be up to approximately 8 feet below grade, although due to the site’s shallow groundwater conditions, we recommend that consideration be given to using shallower vaults. Based upon conditions disclosed by the explorations, we anticipate that vault subgrades will consist of loose native sand with a low fines and gravel content.

The vaults will exert a relatively low bearing pressure on the existing soils, and we estimate that up to approximately 1 inch of settlement may take place soon after the vaults are installed and backfilled. Some subgrade improvement is recommended to reduce the potential for differential settlement. Placing a minimum 6-inch compacted thickness of crushed rock below the vaults will help to reduce the magnitude of differential settlement. The crushed rock should conform to the quality and gradation requirements for WSDOT CSBC. Moderate to rapid groundwater seepage should be expected for excavations that extend into groundwater. The contractor should be prepared to dewater excavations to the extent necessary to allow for installation of vaults, conduits, and bedding and backfill materials in accordance with the District’s requirements.

Buoyancy Considerations: The vaults will be subject to buoyant forces if they are water-tight. Potential buoyant forces acting on the vaults may be calculated by multiplying the volume of the portion of the vault below the water table (in cubic feet) by 62.4 pcf. Buoyant forces may be resisted by the weight of a vault and its contents. Additional resistance to buoyant forces may be achieved by installing flanges on the vault base. The weight of the soil backfill placed above the flanges will assist in counteracting buoyant forces. We recommend using a soil density of 125 pcf for backfill above the water table, and 60 pcf for backfill below the water table. Based on our observations, we recommend considering a seasonal high groundwater elevation of 41.5 feet.

IBC Seismic Design Parameters

Per the 2018 IBC seismic design procedures and ASCE 7-16, the presence of liquefiable soils requires a Site Class definition of F. However, through reference to Sections 11.4.8 and 20.3.1 of ASCE 7-16, the 2018 IBC allows site coefficients F_a and F_v to be determined assuming that liquefaction does not occur for structures with fundamental periods of vibration less than 0.5 seconds. Provided the buildings fundamental period of vibration is less than 0.5 seconds, Site Class D may be used to determine the values of F_a and F_v in accordance with Sections 11.4.8 and 20.3.1 of ASCE 7-16. If exceptions for Site Class D presented in Section 11.4.8 and 20.3.1 of ASCE 7-16 do not apply, a ground motion hazard analysis may be required.

Table 4: Recommended Seismic Parameters

Code Used	Site Classification
2018 International Building Code (IBC) [1]	F [2, 3]
Site Latitude/Longitude	48.0668/-122.1701
Peak Ground Acceleration, PGA	0.472g
Site Modified Peak Ground Acceleration, PGA_M	0.532g
S_s Spectral Acceleration for a Short Period	1.110g
S_1 Spectral Acceleration for a 1-Second Period	0.395g
F_a Site Coefficient for a Short Period	1.056 (Site Class D)
F_v Site Coefficient for a 1-Second Period	Null-See ASCE 7-16 Section 11.4.8

1. IBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile.
2. The explorations completed for this study extended to a maximum depth of approximately 50 feet below grade. ZGA therefore determined the Site Class assuming that medium dense normally consolidated soils extend to 100 feet as suggested by published geologic maps for the project area.
3. Per *ASCE 7-16*, Chapter 20, any profile containing soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils shall be classified as Site Class F.

ZipperGeo
Geoprofessional Consultants

Foundations

We anticipate that some of the new structures will be supported by drilled pier foundations, while others may be supported by slabs or conventional shallow foundations. The foundation net vertical bearing pressures are expected to be relatively low, and the slabs and foundations are typically about 2 to 5 feet deep, respectively, based upon our experience with other District facilities. The native granular soils and properly compacted structural fill are adequate for support of shallow foundations.

Based on conditions observed at the locations of borings and test pits completed at or near the proposed slab locations, we anticipate that foundation subgrade soils will largely consist of loose to medium dense sand with a low silt and gravel content. In order to reduce post-construction settlement, we recommend excavating 1 foot below the design foundation or slab subgrade elevation and replacing the existing soils with CSBC compacted to at least 95 percent per ASTM D 1557. In the event that loose soils or soils containing organics material or deleterious debris are encountered at the CSBC subgrade elevation, we recommend removing the organics and deleterious debris and compacting loose soils to a firm and non-yielding condition and to at least 95 percent density. The excavations made prior to CSBC placement and overexcavation of inadequate soils below footings should extend laterally beyond all edges of the footings a distance of 2 feet per 3 feet of overexcavation depth below footing base elevation. We recommend backfilling excavations made to remove unsuitable soils with CSBC placed in lifts of 10 inches or less in loose thickness and compacted to at least 95 percent density (ASTM D 1557). It would also be feasible to backfill the excavations with lean mix concrete or Controlled Density Fill (CDF). If excavations are backfilled with lean mix concrete or CDF, we recommend the material have a minimum compressive strength of 100 psi. When using CDF, the overexcavation need only be 1 foot wider than the foundation on all sides.

Recommended criteria for shallow foundations are summarized below.

<u>Net allowable bearing pressure:</u> 2,000 psf. This value incorporates a factor of safety of 3. A one-third increase may be applied for short-term wind or seismic loading.

<u>Minimum base dimension:</u> 4 feet

<u>Minimum embedment for frost protection:</u> 18 inches

<u>Approximate total settlement:</u> 1 inch

<u>Estimate differential settlement:</u> One half of total settlement over 40 feet

<u>Ultimate passive resistance:</u> 235 pcf. This value assumes that foundations are backfilled with native sand compacted to 95 percent density and does not include a factor of safety. Neglect the upper 18 inches of embedment when calculating passive resistance.

Ultimate coefficient of base friction: 0.55. This value assumes the foundations are formed above compacted CSBC.

Shallow Foundation Construction Considerations

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.

Drilled Pier Foundation / Direct Burial Recommendations

We anticipate that some of the structures in the substation, including the dead end (termination) structures, will be supported by drilled pier foundations, although the dead end structures may be installed via direct burial. Transmission poles are also proposed for construction in the southeastern portion of the substation and along 47th Avenue NE. Based upon conditions observed at the locations of the explorations, site conditions are generally favorable for support of drilled pier foundations or direct burial although the shallow groundwater condition will necessitate the use of casing during installation.

We understand that the District will complete the foundation designs in house. The tables below provide recommended soil values for incorporation into the District's *Caisson* design program. We have not incorporated factors of safety into the listed values. **The depth intervals referenced in the tables are relative to the existing ground surface elevation at the specific boring locations.** Non-cohesive soils were observed at the exploration locations, so soil cohesion values are not provided. 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.

We recommend incorporating the values listed in Table 5A and 5B for structures or poles at the substation site.

Table 5A: Recommended Soil Parameters Based on boring B-1

Depth interval in feet below existing grade	Soil Condition	Averaged Standard Penetration Resistance (N)	Correlated Pressuremeter Elastic Modulus (kips/in^2)[1]	Soil Wet Density (pcf)	Internal Friction Angle (Ø, in degrees)
0 – 3	Med. dense Sand and silty Sand, variable gravel, wood debris (Fill)	13	1.65	105[2]	31
3 – 9.5	Loose Sand, trace silt and gravel	10	1.39	100[2]	30
9.5 – 14.5	Med. Dense Sand and silty Sand	16	1.89	105[2]	32
14.5 – 42.5	Med. Dense Sand and silty Sand	25	2.52	107[2]	35
42.5 – 51.5	Very stiff sandy Silt	24	2.45	107[2]	34

1. The pressuremeter modulus values are based upon published correlations between Standard Penetration Test values (N) and the pressuremeter modulus; a factor of safety does not apply.
2. Soil Wet Density does not reflect buoyant unit density below the observed groundwater. Subtract 62.4 pcf for buoyant unit density.

ZipperGeo
Geoprofessional Consultants

Table 5B: Recommended Soil Parameters Based on boring B-1						
Depth interval in feet below existing grade	**Soil Condition**	**Relative Density (D_r as percent)**	**Ultimate Friction Factor[1]**	**Ultimate Friction Factor[2]**	**Moisture Content (percent by dry weight basis)[3]**	**Rankine Coefficient Passive[4] / Active**
0 – 3	Med. dense Sand and silty Sand, variable gravel, wood debris (Fill)	40	0.4	0.25	15	3.12 / 0.32
3 – 9.5	Loose Sand, trace silt and gravel	35	0.5	0.3	21[3]	3.0 / 0.33
9.5 – 14.5	Med. Dense Sand and silty Sand	45	0.5	0.3	23[3]	3.25 / 0.31
14.5 – 42.5	Med. Dense Sand and silty Sand	57	0.5	0.3	25[3]	3.69 / 0.27
42.5 – 51.5	Very stiff sandy Silt	57	0.35	0.2	26[3]	3.54 / 0.28

1. The ultimate friction factors are based upon published values for adhesion between concrete and the applicable soil type.
2. The ultimate friction factors are based upon published values for adhesion between steel and the applicable soil type.
3. Moisture contents are for saturated sand samples retrieved from below groundwater.
4. Passive resistance in the upper 1.5 feet should be neglected entirely.

We recommend incorporating the values listed in Table 6A and 6B for design of the proposed transmission poles along 47th Avenue NE.

Table 6A: Recommended Soil Parameters Based on boring B-7					
Depth interval in feet below existing grade	**Soil Condition**	**Averaged Standard Penetration Resistance (N)**	**Correlated Pressuremeter Elastic Modulus (kips/in²)[1]**	**Soil Wet Density (pcf)[2]**	**Internal Friction Angle (Ø, in degrees)**
0 – 4.5	Loose Sand with some gravel, trace silt	5	0.89	105[2]	28
4.5 – 17.5	Med. dense Sand, variable silt and gravel	19	2.11	106[2]	33
17.5 – 29	Loose Sand, trace silt	9	1.3	100[2]	29
29 – 36.5	Stiff to very stiff sandy Silt	16	1.89	105[2]	32

1. The pressuremeter modulus values are based upon published correlations between Standard Penetration Test values (N) and the pressuremeter modulus; a factor of safety does not apply.
2. Soil Wet Density does not reflect buoyant unit density below the observed groundwater. Subtract 62.4 pcf for buoyant unit density.

Table 6B: Recommended Soil Parameters Based on boring B-7						
Depth interval in feet below existing grade	**Soil Condition**	**Relative Density (D_r as percent)**	**Ultimate Friction Factor[1]**	**Ultimate Friction Factor[2]**	**Moisture Content (percent by dry weight basis)[3]**	**Rankine Coefficient Passive[4] / Active**
0 – 4.5	Loose Sand with some gravel, trace silt	17	0.4	0.25	11	2.77 / 0.36
4.5 – 17.5	Med. dense Sand, variable silt and gravel	52	0.5	0.3	26[3]	3.39 / 0.29
17.5 – 29	Loose Sand, trace silt	30	0.4	0.3	30[3]	2.88 / 0.35
29 – 36.5	Stiff to very stiff sandy Silt	45	0.35	0.2	26[3]	3.25 / 0.31

1. The ultimate friction factors are based upon published values for adhesion between concrete and the applicable soil type.
2. The ultimate friction factors are based upon published values for adhesion between steel and the applicable soil type.
3. Moisture contents are for saturated sand samples retrieved from below groundwater.
4. Passive resistance in the upper 1.5 feet should be neglected entirely.

Drilled Shaft End Bearing Considerations

When calculating drilled pier end bearing values, it will be necessary to consider the density of the soils to a depth below the shaft that is a function of the shaft diameter. We can provide specific end bearing capacity recommendations once preliminary design efforts for the drilled pier foundations have identified likely drilled pier diameters and depths.

Open Shaft Construction Considerations

Given the soil conditions encountered at the exploration locations, we anticipate that construction of the shafts can be accomplished with standard drilling equipment. Although the exploratory drilling and probing processes did not suggest the presence of cobbles and potentially boulders or other possible drilling obstructions within the deposits encountered within our explorations, the contractor should be prepared to deal with the presence of oversize material and obstructions over the installation depth interval.

Casing / Sleeve Cleanout

We anticipate that the granular soils encountered over the drilled interval will cave in an open borehole condition. The contractor should be prepared to install full-depth casing or a sleeve through caving soil zones. The drilling contractor should be prepared to clean out the bottom of the shaft if loose soil is observed or suspected prior to placing the buried portion of the pole and surrounding concrete/crushed rock or prior to installing drilled pier reinforcing and concrete. We recommend that the drilling contractor have a cleanout bucket on site to remove loose soils and/or mud from the bottom of the drilled shafts.

Groundwater and Bore Hole Stability

The site is characterized by a groundwater table aquifer and groundwater will be encountered while drilling. We estimate that successful completion of drilled shafts may require dewatering or the use of drilling fluids. The contractor should develop means and methods such as dewatering, the use of casing, and 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 fluid in order to avoid bottom heave of the drilled shaft. Where drilling fluids are used, the slurry level used to maintain a stable bore hole should be maintained to obtain hydrostatic equilibrium throughout the construction operation at a height required to provide and maintain a stable bore hole.

Concrete Placement

Concrete for drilled piers should normally be placed via the free fall method. However, per the *Drilled Shaft Manual* published by the Federal Highway Administration, we recommend placing concrete by the tremie method if more than 3 inches of water has accumulated in the excavation as a means of displacing water and to reduce the risk of contaminating or segregating the concrete mix. A minimum 5-foot head of concrete should be maintained above the tremie.

IBC Non-constrained Pole Design Recommendations

Section 1805.7.2.1 of the 2003 the *International Building Code* (IBC) describes the methodology for determining a drilled pier foundation or pole depth of embedment in cases where no constraint is provided at the surface to resist lateral forces. We have evaluated the equivalent passive soil pressure per foot of depth for use in the IBC method. Recommended lateral bearing pressures as a function of pole depth are listed below in Table 7. We recommend neglecting resistance in the upper 1.5 feet of embedment. Please note that the values listed below are relative to the ground surface elevation at the boring locations.

ZipperGeo
Geoprofessional Consultants

Table 7: IBC Non-constrained Pole Lateral Bearing Pressure	
ZGA Boring	**Recommended Lateral Bearing Pressure (lbs/ft^2/ft) of Embedment Depth[1,2,3]**
B-1	1.5 to 3 feet: 130 3 to 9.5 feet: 120 9.5 to 14.5 feet: 135 14.5 to 42.5 feet: 158 42.5 to 51.5 feet: 150
B-7	1.5 to 4.5 feet: 115 4.5 to 17.5 feet: 145 17.5 to 29 feet: 115 29 to 36.5 feet: 135
1. Values incorporate a factor of safety = 2.5 2. Neglect upper 1.5 feet 3. Subtract 62.5 to determine effective value below groundwater	

In the event that structural fill compacted to 95 percent density per ASTM D 1557 is placed to raise grade at drilled pier locations, we recommend using a lateral bearing pressure of 200 lbs/ft^2/ft of embedment depth for compacted fill that extends below a depth of 1.5 feet. This value incorporates a factor of safety of 2.5. The upper 1.5 feet of embedment should be neglected.

Concrete Slab Subgrade Preparation Recommendations

The transformers and switchgear enclosures will be supported by reinforced concrete slabs, and oil containment slabs will surround the transformer slabs. Our previous recommendations regarding selective excavation and compaction of existing loose fill soils, and removal of organic materials and deleterious debris, should they be observed at the time of construction, are applicable to slab subgrades. Based on conditions observed at the locations of explorations completed at or near the proposed slab locations, we anticipate that slab subgrade soils will largely consist of loose to medium dense sand with a variable silt content. We recommend compacting the slab subgrades to a firm and non-yielding condition and to at least 95 percent of the modified Proctor maximum dry density prior to placing a 12-inch thick CSBC leveling course for the slabs. Provided that the slab subgrades are prepared as described herein, we anticipate that total settlement will be less than ½ inch.

Stormwater Management Analysis Considerations

The site is largely mantled by some uncontrolled fill material underlain by permeable native granular soil and is characterized by a relatively shallow seasonal groundwater condition. Conclusions regarding stormwater infiltration feasibility can be drawn from subsurface conditions disclosed by the subsurface explorations, groundwater observations, and laboratory testing completed to date.

We understand that stormwater management improvements will be designed in accordance with the Washington State Department of Ecology 2019 *Stormwater Management Manual for Western*

Washington (*Manual*). We collected representative samples of shallow soils and completed mechanical grain size tests as part of assessing the soils' saturated hydraulic conductivity, as summarized below.

Saturated Hydraulic Conductivity

The *Manual* allows a determination of soil saturated hydraulic conductivity to be estimated based on grain size distribution characteristics in accordance with the following formula:

$$\text{Log10}\,(K_{sat,\ initial}) = -1.57 + 1.9D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{fines} \text{ where:}$$

$K_{sat,\ initial}$ = initial saturated hydraulic conductivity in centimeters/second prior to the application of correction factors

D_{10} = grain size diameter (mm) for which 10 percent of the sample by weight is finer

D_{60} = grain size diameter (mm) for which 60 percent of the sample by weight is finer

D_{90} = grain size diameter (mm) for which 90 percent of the sample by weight is finer

f_{fines} = fraction of the sample by weight that passes the US No. 200 sieve.

The calculated hydraulic conductivity values for representative soils that we tested are listed in the table below. Grain size distribution curves for the samples are presented in Appendix B.

Table 8: Saturated Hydraulic Conductivity Summary		
Exploration / Sample	**Approximate sample depth (feet)**	**Unfactored Saturated Hydraulic Conductivity (inches per hour)**
TP-1 / S-3	5.5	83.9
TP-1 / S-4	7	59.6
TP-2 / S-2	3.5	67.3
TP-3 / S-2	3	83.9
TP-4 / S-3	3	77.4
TP-5 / S-3	4.5	78.1
TP-6 / S-3	3.3	28.7

Design Saturated Hydraulic Conductivity Rate

The *Manual* requires applying correction factors to the baseline saturated hydraulic conductivity rate. Table 3.3.1 *Correction Factors to be Used with In-Situ Saturated Hydraulic Conductivity Measurements to Estimate Design Rates* of the *Manual* calls for 40 percent reduction of the baseline rate. Table 3.3.1 also requires applying correction factors for site variability and number of locations tested (CF_M) and the degree of influent control to prevent siltation and bio-buildup (CF_v). Based upon the site conditions, testing, and our experience with projects of a similar nature, we applied values of 0.4, 0.4, and 0.9 for CF_v, CF_T, and CF_M, respectively. We recommend using a factored rate (K_{sat}) of 18 inches/hour for the *in situ* native outwash sand for purposes of stormwater infiltration analysis.

Construction of the substation will include selective removal of existing uncontrolled fill material prior to placing imported granular fill to foundation and slab subgrade elevations as necessary. This densification will reduce the site soil's infiltration rate compared to the underlying less dense *in situ* soils. However, this process is only recommended for below foundations and slabs; it is not recommended for the balance of the yard in order to promote stormwater infiltration.

Groundwater Considerations

We measured the depth to groundwater at approximately 5 feet while advancing boring B-3, and at 0.5 feet (approximately elevation 41.5 feet) on 17 November 2021 after several days of significant rain. This is the highest elevation at which we have measured groundwater, and we recommend considering elevation 41.5 feet as the seasonal high condition. This condition will yield approximately 4.5 feet of vertical separation between the seasonal high groundwater and the substation yard finished grade of elevation 46 feet. The yard will be constructed as an embankment of highly permeable granular fill and crushed rock and as described below it will essentially function as a permeable surface.

Storage Considerations

Project plans indicate that the substation yard will be mantled with a 4-inch compacted thickness of "substation rock" underlain by WSDOT CSBC per Specification 9-03.9(3). 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 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. In contrast to some other locally available CSBC, the Iron Mountain Quarry products are 100 percent crushed rock and no naturally occurring bank run sand is blended with the 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 some other locally available products that combine crushed rock and bank run sand and this is a function of the overall low fine to medium sand content and the 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" used at the Fitzgerald Substation (Bothell, Washington)

It is our understanding that the District will specify the use of CSBC in the substation yard that is composed of 100 percent crushed rock and not a product produced by blending crushed bank run rounded gravel with sand. The use of substation rock and CSBC as specified by the District and consistent with the gradation characteristics of these materials used over the past several years on multiple District substations will meet the performance standards described in the drainage report, in our opinion.

We recommend that imported crushed rock used for both structural fill in the yard and stormwater management purposes have the gradation show in the table below.

Table 9: Recommended Crushed Rock Fill Gradation

US Standard Sieve Size	Percent Passing by Dry Weight Basis
1.25 inch	100
1 inch	80 - 100
5/8 inch	50 - 80
No. 4	25 - 45
No. 40	3 - 18
No. 200	< 3

Groundwater Mounding Analysis

Plans available at the time this report was prepared indicate that the substation entry will include two bioretention features for stormwater management; their locations are illustrated on Figure 1. The bioretention features are proposed to have a bottom elevation of 43.25 feet (1.75 feet above the seasonal high groundwater elevation) and a design high water elevation of 45.5 feet. Stormwater management in the yard will rely upon the very high infiltration rate of the clear crushed rock and select granular fill materials that will be used to raise grade to the proposed elevation 46 feet. For modelling purposes, the base of the yard rock, elevation 45.7 feet, was considered the infiltration surface elevation.

The use of on-site infiltration depends on sizing the infiltration system such that the receptor soils below the system can accept the water without water backing up into the system to an unacceptable degree. The development of a groundwater mound, or a localized rise in the local groundwater table, can adversely affect an infiltration system if the mound rises too high. A groundwater mounding analysis was completed for the proposed storm water infiltration system per the requirements of the *Manual*.

The purpose of the mounding analysis was to evaluate if groundwater mounding below the proposed bioretention cells would adversely affect performance of the system, and in the case of the yard, adversely affect functional of the substation. We used the MODRET computer software program to model groundwater mounding at the yard entry and the yard itself.

The simulations incorporated long-term surface water runoff data provided by the District, subsurface conditions as disclosed by the test pits, boring, and CPT, the results of laboratory testing, and measured aquifer properties described in the USGS report: *The Ground-Water System and Ground-Water Quality in Western Snohomish County, Washington* (USGS Water-Resources Investigations Report 96-4312, 1997), and ZGA site observations of other sites in the vicinity of the proposed substation. The groundwater mounding analyses for the entry and the yard incorporated the parameters listed on the data sheets included in Appendix D. Both models considered that at least 1 foot of select, clean, 100 percent crushed CSBC is placed above the existing ground surface.

The mounding analysis for the entry indicates that the high water elevation will extend to the bioretention cells' bottom elevation of 43.25 feet. Based upon our analysis, it is our opinion that the bioretention cells will function adequately relative to the groundwater conditions and the design inflow event.

The mounding analysis for the yard indicates that the high water elevation will extend to elevation 42.38 feet, or slightly less than 1 foot above the seasonal high groundwater elevation and 3.62 feet below the yard finished grade of 46 feet. Based upon our analysis, it is our opinion that the modeled design event will not adversely affect the substation functionality.

Driveway Flexible Pavement Section Recommendations

It is our understanding that the existing gravel and crushed rock surfacing of the access driveway will remain. However, we have provided the recommendations below in the event that the District elects to pave the entry drives. The District typically requires that the pavement section be able to accommodate H20 loading.

Pavement Life and Maintenance: It should be realized that asphaltic pavements such as hot mix asphalt (HMA) 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.

Soil Design Values: Pavement subgrade soils are anticipated to consist well-compacted gravelly sand and/or CSBC with a relatively low silt content. Our analysis assumes the pavement section subgrade will have a minimum California Bearing Ratio (CBR) value of 10.

Recommended Pavement Section: We recommend that the pavement section, at a minimum, consist of 3 inches of asphalt concrete over 2 inches (compacted thickness) of crushed surfacing top course over 8 inches of crushed surfacing base course.

We recommend the following regarding flexible pavement materials and pavement construction.

Subgrade Preparation and Compaction: The pavement subgrade will consist of structural fill and should be prepared in accordance with the recommendations presented in the *Subgrade Preparation* section of this report, and all fill should be compacted in accordance with the recommendations presented in the *Structural Fill* section of this report.

HMA: We recommend that the HMA conform to Section 9-02.1(4) for PG 58-22 or PG 64-22 Performance Graded Asphalt Binder as presented in the WSDOT *Standard Specifications*. We also recommend that the gradation of the HMA aggregate conform to the aggregate gradation control points for ½-inch mixes as presented in Section 9-03.8(6), HMA Proportions of Materials.

Base Course: We recommend that the CSBC conform to Section 9-03.9(3) of the WSDOT *Standard Specifications*.

Compaction and Paving: We recommend compacting the HMA to a minimum of 92 percent of the Rice (theoretical maximum) density per the 2021 WSDOT *Standard Specifications* is in effect. Placement and compaction of HMA should conform to requirements of Section 5-04 of the *Standard Specifications*.

Erosion Control

Construction phase erosion control activities are recommended to include measures intended to reduce erosion and subsequent sediment transport. We recommend that the project incorporate the following erosion and sedimentation control measures during construction:

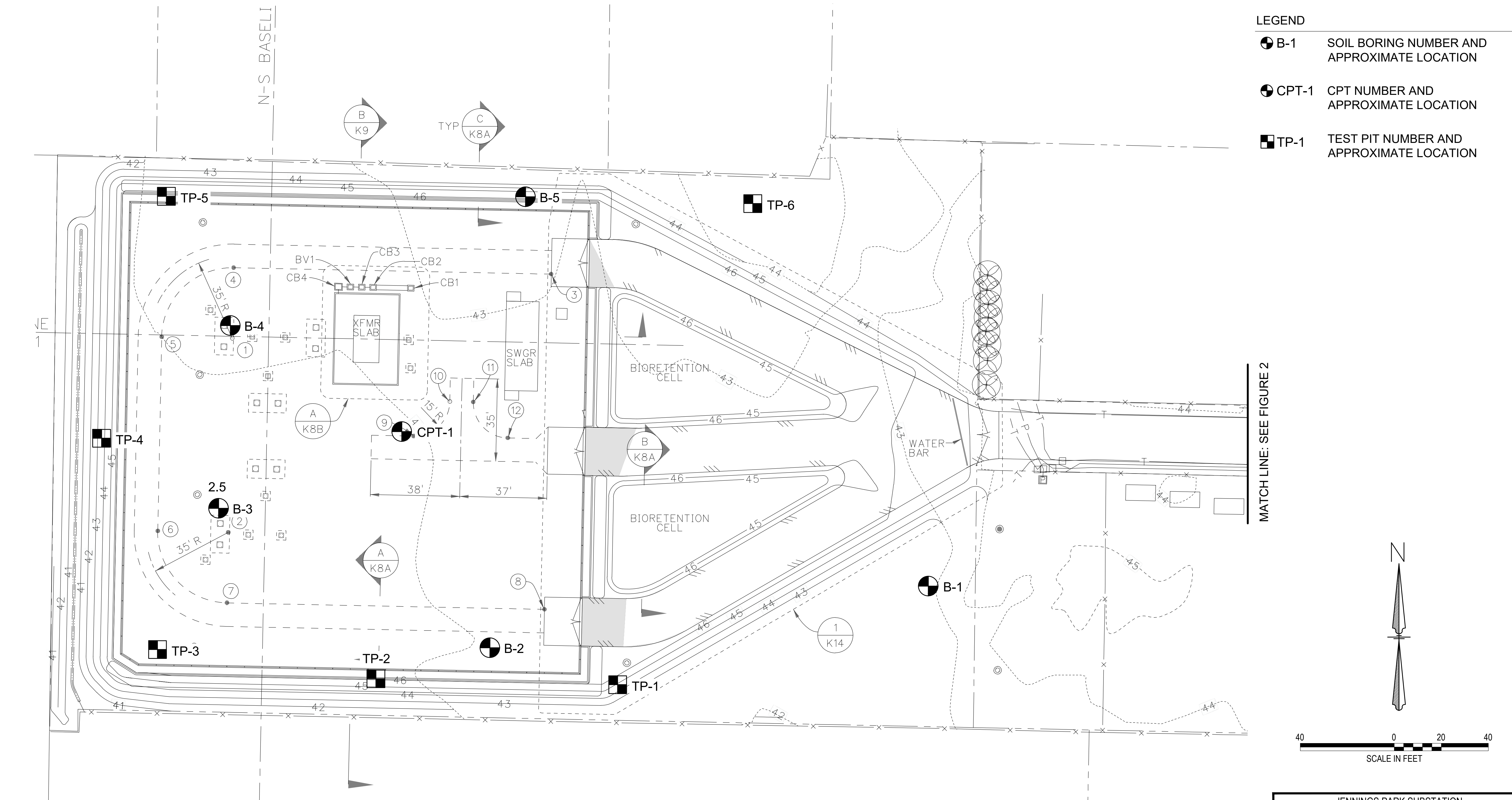
- Capturing water from low permeability surfaces and directing it away from bare soil exposures.
- Erosion control BMP inspection and maintenance: The contractor should be aware that inspection and maintenance of erosion control BMPs is critical toward their satisfactory performance. Repair and/or replacement of dysfunctional erosion control elements should be anticipated.
- Undertake site preparation, excavation, and filling during periods of little or no rainfall.
- Cover excavation surfaces with anchored plastic sheeting if surfaces will be left exposed during wet weather.
- Cover soil stockpiles with anchored plastic sheeting.
- Provide an all-weather quarry spall construction site entrance.
- Provide for street cleaning on an as-needed basis.
- Protect exposed soil surfaces that will be subject to vehicle traffic with crushed rock or crushed recycled concrete to reduce the likelihood of subgrade disturbance and sediment generation during wet weather or wet site conditions.
- Install siltation control fencing on the lower perimeter of work areas.
- If grounding wells are installed, containment of the cuttings produced during the drilling process will reduce the likelihood of off-site sediment migration. Cuttings with a high fines content should be removed from the site following completion of drilling.

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 recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend we 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 slabs depends 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 its agents, for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, express or implied, are intended or made. Site safety, excavation support, and dewatering requirements 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 ZGA reviews the changes and either verifies or modifies the conclusions of this report in writing.



LEGEND

B-1	SOIL BORING NUMBER AND APPROXIMATE LOCATION
CPT-1	CPT NUMBER AND APPROXIMATE LOCATION
TP-1	TEST PIT NUMBER AND APPROXIMATE LOCATION

JENNINGS PARK SUBSTATION
7808 47TH AVE NE
Marysville, Washington

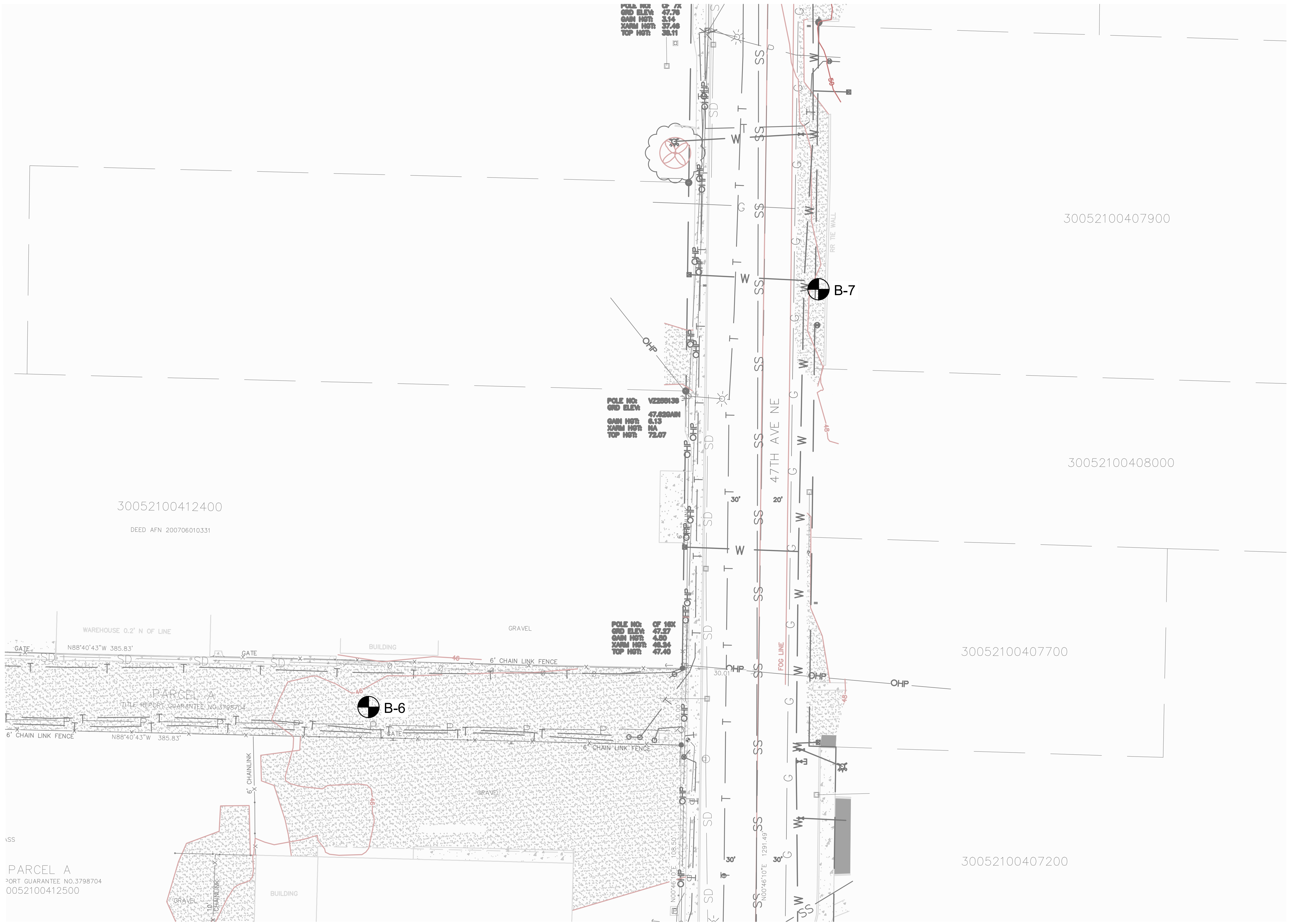
SITE AND EXPLORATION PLAN

DATE:FEBRUARY 2023 | Job No. 2494.01

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

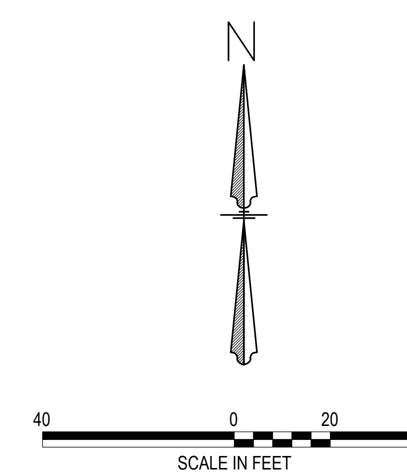
FIGURE 1
SHT.1 of 1

REFERENCE: JENNINGS PARK SUBSTATION GRADING AND DRAINAGE PLAN, DRAWING S-135-K8, SNOHOMISH COUNTY PUD



LEGEND

B-1	SOIL BORING NUMBER AND APPROXIMATE LOCATION



JENNINGS SUBSTATION
7808 47TH AVE NE
Marysville, Washington

SITE AND EXPLORATION PLAN

DATE: NOVEMBER 2021 | Job No. 2494.01

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

FIGURE 2
SHT.2 of 2

REFERENCE: CENTRAL MARYSVILLE REBUILD CONCEPT A, SHEET 1 OF 1, SNOHOMISH COUNTY PUD, AUG 6, 2021

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS

FIELD EXPLORATION AND TESTING PROCEDURES AND LOGS

Our field exploration program for this project included completing a visual reconnaissance of the site, advancing seven borings (B-1 through B-7), advancing one cone penetrometer test (CPT-1), and excavating six test pits (TP-1 through TP-6). The approximate exploration locations are presented on Figures 1 and 2, the *Site and Exploration Plans*. Exploration locations were determined in the field using steel and fiberglass tapes by measuring distances from existing site features shown on the *Central Marysville Rebuild Concept A* plan, dated 26 August 2021, provided by the District. The ground surface elevation at each exploration location was interpolated from the topography shown on an undated topographic survey prepared by ASPI, LLC and provided for our review. As such, the exploration locations and elevations should be considered accurate to the degree implied by the measurement method. 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 truck-mounted drill rig operated by an independent drilling company (Environmental Drilling) working under subcontract to ZGA. The borings were advanced using hollow stem auger drilling methods. An engineering geologist from our firm 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 to 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.

A groundwater observation well was installed at the boring B-3 location following completion of drilling and sampling. The well consists of a 10-foot long section of 2-inch inside-diameter PVC screen section with machined 0.020-inch wide slots. Washed silica sand was placed in the annular space between the screen and the borehole. A non-machined riser was installed to the ground surface, and bentonite clay was placed around the riser. The well as finished with a flush-mount metal monument set in concrete.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring. If groundwater was encountered in a borehole, the approximate groundwater depth and date of observation are depicted on the log.

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.

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.

Cone Penetrometer Testing

The cone penetrometer test was completed by a ZGA subcontractor (In Situ Engineering) using a truck-mounted rig. The testing was completed in general accordance with ASTM D 5778-12 procedures. The cone penetrometer testing involves advancing 35.7-millimeter diameter rods equipped with a friction sleeve, standard area cone, load cell, and pressure transducer. The apparatus is advanced via hydraulic pressure and the tip resistance and friction are recorded continuously. Pore pressure measurements and shear wave and compression wave testing may be taken at selected intervals.

The enclosed cone penetrometer test log indicate the recorded tip resistance, friction, friction ratio, pore pressure, correlation to the Standard Penetration Test, and a graphic representation of the soil type.

Sample Screening

The boring and test pit logs also include the results of sample container headspace measurements taken with a RAE Systems photoionization detector (PID). The measurements indicate the relative concentration of petroleum hydrocarbons in the headspace air, but do not identify the type of hydrocarbon. The sample headspace readings, recorded as hydrocarbon concentration in parts per million (ppm) are presented on the logs in this appendix. The sample screening did not detect hydrocarbon levels of concern.

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximately 43 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/27/2021	Drill Rig:	B-61	Logged by:	MRC

B-1

SOIL DESCRIPTION

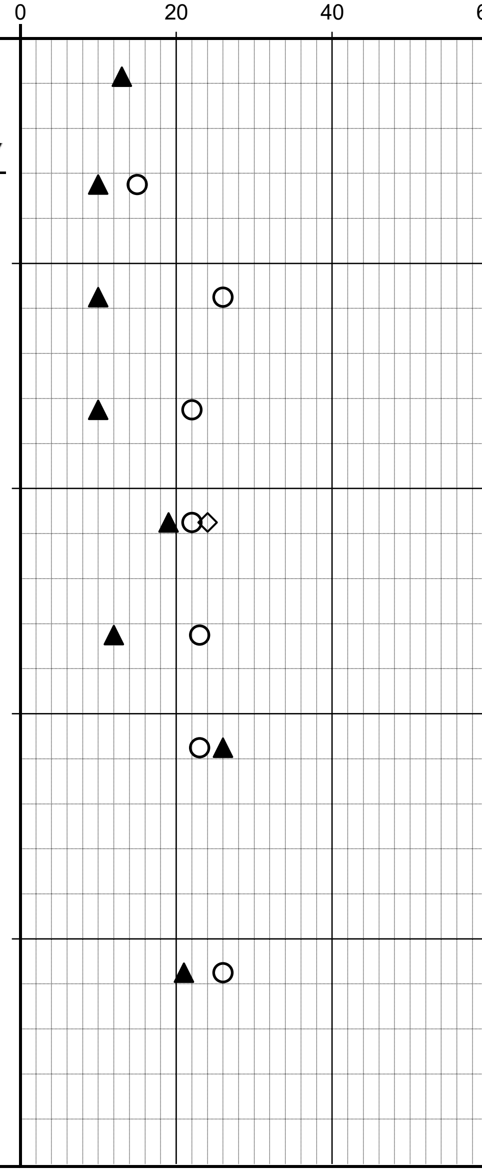
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
0	Grass over 6 inches of dark brown, silty SAND with fine roots (Topsoil)	S-1	0		13	< 1.0
	Loose, wet, dark brown, SAND, some silt, trace gravel, broken wood debris observed (Fill)	S-2	8	ATD	10	< 1.0
	Loose, wet to saturated, orange brown, fine to medium SAND, some to trace silt, trace gravel (Recessional Outwash)					
5		S-3	12		10	< 1.0
		S-4	12		10	< 1.0
10	Medium dense, saturated, light brown to gray, fine SAND and silty fine SAND	S-5	18		19	< 1.0
		S-6	10		12	< 1.0
15		S-7			26	< 1.0
20	2-inch thick fine sandy silt horizon	S-8	13		21	< 1.0
25						

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



SAMPLE LEGEND

- 2-inch O.D. split spoon sample
- 3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing
- Groundwater level at time of drilling (ATD) or on date of measurement. 11/2/12

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ├──○──┤ Liquid Limit

Natural Water Content

Jennings Substation

7808 47th Avenue NE

Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-1

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximately 43 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/27/2021	Drill Rig:	B-61	Logged by:	MRC

B-1

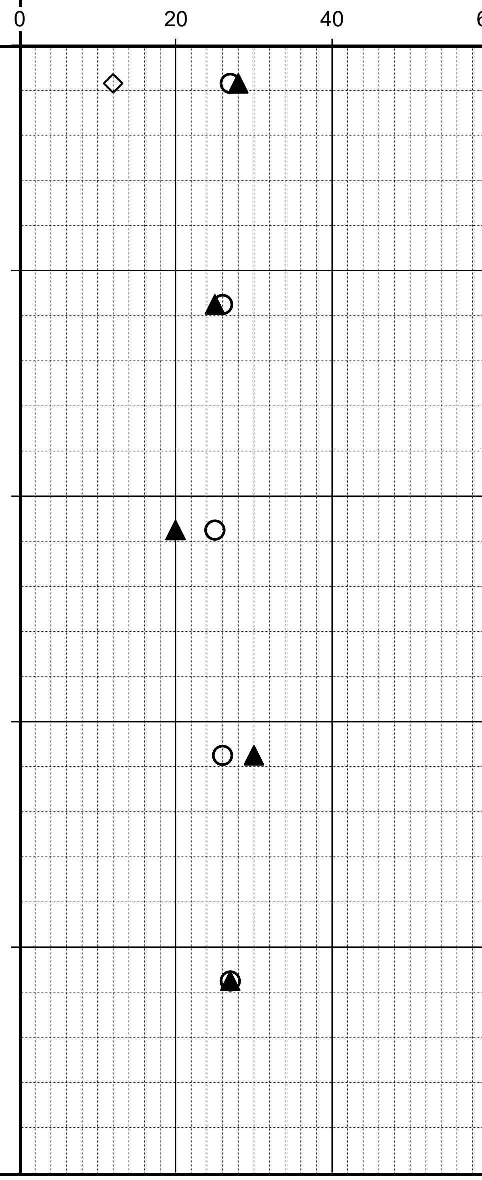
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	SOIL DESCRIPTION	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
25	Medium dense, saturated, gray brown, fine SAND, some silt	S-9	15		28	< 1.0
30	Fines content increases	S-10	14		25	< 1.0
35		S-11	12		20	< 1.0
40	Medium dense, saturated, gray, fine sand with silt to silty SAND	S-12	12		30	< 1.0
45	Very stiff, saturated to wet, gray, sandy SILT, with thin fine sand laminations	S-13	13		27	< 1.0
50						

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



SAMPLE LEGEND

2-inch O.D. split spoon sample

3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

Clean Sand

Bentonite

Grout/Concrete

Screened Casing

Blank Casing

Groundwater level at time of drilling (ATD) or on date of measurement.

11/2/12

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ├──○──┤ Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-1

Boring Location: See Figure 1, Site and Exploration Plan	Drilling Company: Environmental	Bore Hole Dia.: 8-inch	**B-1**
Top Elevation: Approximately 43 Feet	Drilling Method: Hollow Stem Auger	Hammer Type: Auto	
Date Drilled: 10/27/2021	Drill Rig: B-61	Logged by: MRC	

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.

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:

Depth (ft)	Soil Description	Sample Number	Groundwater	Penetration Resistance (0–60)	Blowcount	PID
50	Very stiff, saturated to wet, gray, sandy SILT, with thin fine sand laminations	S-14		▲ ○	22	< 1.0
	Boring completed at approximately 51.5 feet. Groundwater observed at approximately 3 feet ATD.					
55						
60						
65						
70						
75						

SAMPLE LEGEND

2-inch O.D. split spoon sample

3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

Clean Sand

Bentonite

Grout/Concrete

Screened Casing

Blank Casing

Groundwater level at time of drilling (ATD) or on date of measurement.

11/2/12

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit — Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-1

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximately 42 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/27/2021	Drill Rig:	Truck Rig	Logged by:	MRC

B-2

SOIL DESCRIPTION

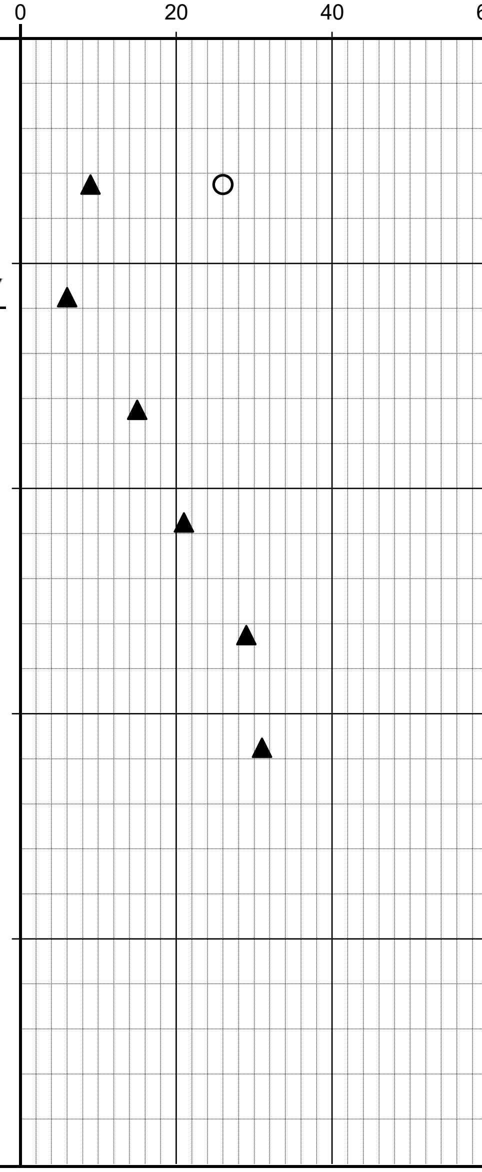
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
0	Grass over 6 inches of dark brown, silty SAND with fine roots (Topsoil), over orange brown SAND, with silt, trace gravel (Fill)					
	Loose, moist, orange-brown fine SAND, some to trace silt (Fill). Approximately 2-inch layer of topsoil encountered at approximately 3 feet	S-1	6		9	< 1.0
5	Loose, wet to saturated, light brown to gray, fine to medium SAND, trace silt (Recessional Outwash)	S-2	6	ATD	6	< 1.0
	Medium dense, saturated, gray, fine to medium SAND, trace silt	S-3	16		15	< 1.0
10		S-4	16		21	< 1.0
	Grades to light brown to gray-brown, predominantly fine sand	S-5	16		29	< 1.0
15	Dense, saturated, light brown, fine to medium SAND, trace silt	S-6	14		31	< 1.0
	Boring completed at approximately 16.5 feet. Groundwater was encountered at approximately 6 feet ATD.					
20						
25						

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



SAMPLE LEGEND

- 2-inch O.D. split spoon sample
- 3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing
- Groundwater level at time of drilling (ATD) or on date of measurement. 11/2/12

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit —○— Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-2

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximatey 42 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/27/2021	Drill Rig:	Truck Rig	Logged by:	MRC

B-3

SOIL DESCRIPTION

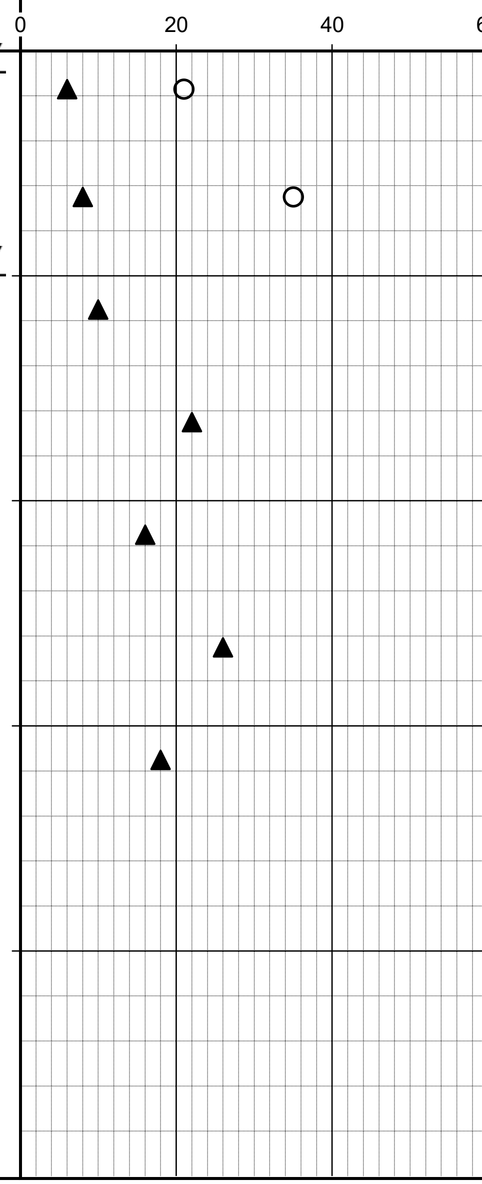
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
0	Grass over 6 inches of dark brown, silty SAND with fine roots (Topsoil), over loose, moist, orange-brown SAND, some silt, trace gravel (Fill)	S-1	12	11.17.21	6	< 1.0
	Loose, moist to saturated, dark brown, silty SAND grading to orange-brown, fine to medium SAND, trace silt	S-2	18		8	< 1.0
5		S-3	14	ATD	10	< 1.0
	Soil density increases to medium dense, trace coarse sand	S-4	18		22	< 1.0
10	Medium dense, saturated, gray, fine to medium SAND, trace silt, with approximately 1-inch thick interbedded silt layers	S-5	12		16	< 1.0
		S-6	10		26	< 1.0
15		S-7	18		18	< 1.0
	Boring completed at approximately 16.5 feet. Groundwater was encountered at approximately 5 feet ATD.					
20						
25						

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



SAMPLE LEGEND

- 2-inch O.D. split spoon sample
- 3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing
- Groundwater level at time of drilling (ATD) or on date of measurement. (11/2/12)

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ├──○──┤ Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-3

Boring Location: See Figure 1, Site and Exploration Plan
Top Elevation: Approximately 42 Feet
Date Drilled: 10/27/2021

Drilling Company: Environmental
Drilling Method: Hollow Stem Auger
Drill Rig: Truck Rig

Bore Hole Dia.: 8-inch
Hammer Type: Auto
Logged by: MRC

B-4

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.

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
0	Grass over 6 inches of dark brown, silty SAND with fine roots (Topsoil), over loose, moist, orange-brown fine SAND, some to trace silt (Fill)	S-1	12		5	< 1.0
	Loose, moist, dark brown, silty SAND, trace organics and fine roots (Relic Topsoil), over loose, moist, orange-brown grading to brownish gray, fine to medium SAND, trace silt (Recessional Outwash)	S-2	14	ATD	10	< 1.0
5		S-3	15		16	< 1.0
	Soil grades to gray	S-4	18		20	< 1.0
10	Medium dense, saturated, gray, fine to coarse SAND, trace gravel, trace silt	S-5	18		16	< 1.0
	Gravel content decreases, soil grades to light brown	S-6	10		17	< 1.0
15		S-7	6		13	< 1.0
	Boring completed at approximately 16.5 feet. Groundwater was encountered at approximately 4.5 feet ATD.					
20						
25						

SAMPLE LEGEND

- 2-inch O.D. split spoon sample
- 3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing
- Groundwater level at time of drilling (ATD) or on date of measurement. 11/2/12

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ⊢—○—⊣ Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-4

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximately 43 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/27/2021	Drill Rig:	Truck Rig	Logged by:	MRC

B-5

SOIL DESCRIPTION

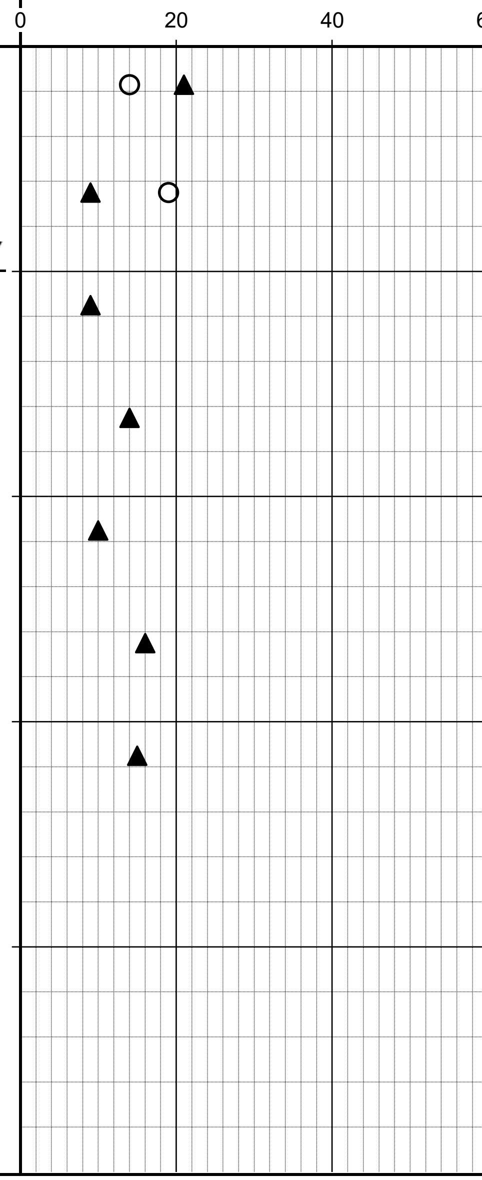
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
0	Grass over 2 to 3 inches of dark brown, gravelly SAND to sandy GRAVEL, some silt. Coarse sand and fine gravel are crushed rock (Fill)	S-1	9		21	< 1.0
	Loose, moist to saturated, light brown to brown-gray, fine SAND, trace silt (Recessional Outwash)	S-2	12		9	< 1.0
5		S-3	12	ATD	9	< 1.0
	Medium dense, saturated, light brown to gray, SAND, trace silt, with approximately 4-inch silty SAND interbed	S-4	15		14	< 1.0
10		S-5	18		10	< 1.0
		S-6	18		16	< 1.0
15		S-7	18		15	< 1.0
	Boring completed at approximately 16.5 feet. Groundwater was encountered at approximately 5 feet ATD.					
20						
25						

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



SAMPLE LEGEND

- 2-inch O.D. split spoon sample
- 3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing
- ▼ 11/2/12 Groundwater level at time of drilling (ATD) or on date of measurement.

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ├─○─┤ Liquid Limit

Natural Water Content

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-5

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximately 46 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/28/2021	Drill Rig:	Truck Rig	Logged by:	MRC

B-6

SOIL DESCRIPTION

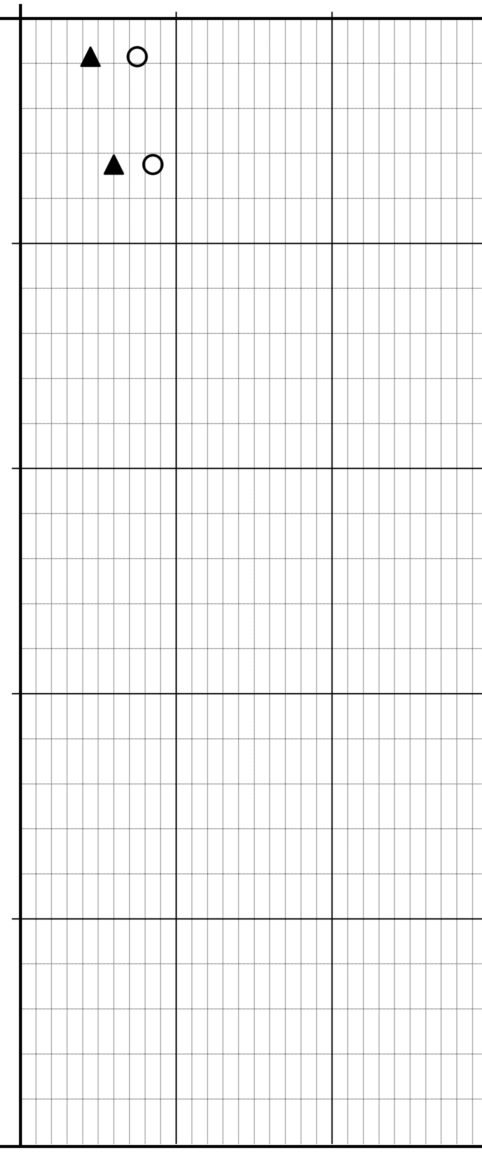
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
0	Approximately 6 inches of crushed rock over brown gravelly SAND (Fill) above loose, moist, orange-brown, SAND, some to trace silt, trace wood debris	S-1	9		9	< 1.0
	Medium dense, moist, brown, gravelly SAND, some silt, wood debris observed in SPT tip	S-2	3		12	< 1.0
5	Boring completed at approximately 4.5 feet. Groundwater was not encountered ATD.					
10						
15						
20						
25						



SAMPLE LEGEND

2-inch O.D. split spoon sample

3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

Clean Sand

Bentonite

Grout/Concrete

Screened Casing

Blank Casing

Groundwater level at time of drilling (ATD) or on date of measurement.

11/2/12

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ├──⊖──┤ Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-6

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximately 48 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/28/2021	Drill Rig:	Truck Rig	Logged by:	MRC

B-7

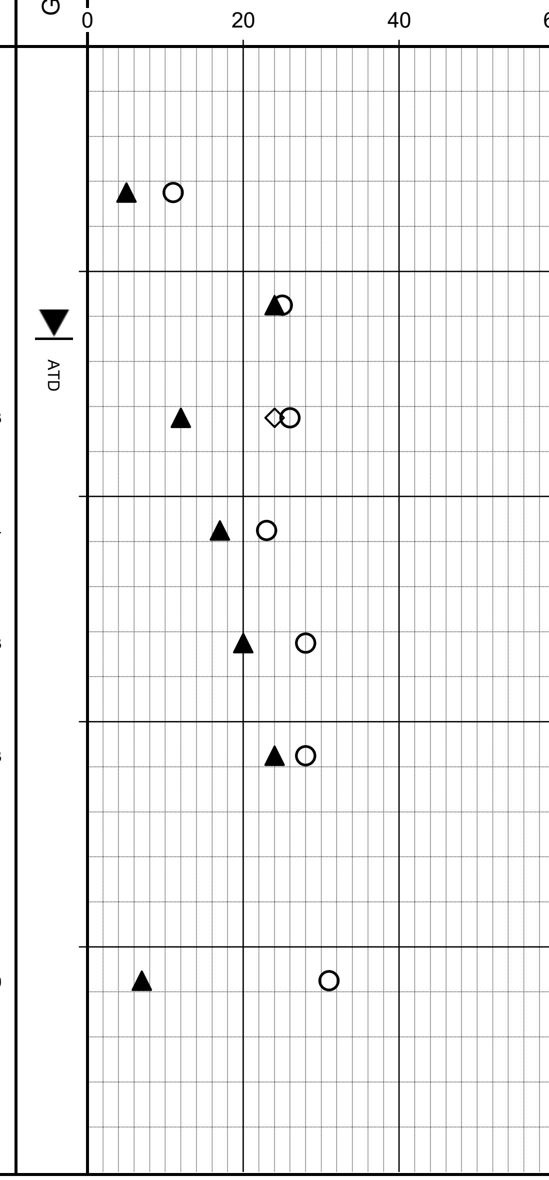
SOIL DESCRIPTION

The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Blowcount	PID
0	4 inches of crushed rock over orange-brown fine to coarse SAND, with gravel, some silt (Fill)				
	Loose, moist, light brown, fine to medium SAND, some to trace gravel, trace silt (Recessional Outwash)	S-1	11	5	< 1.0
5	Medium dense, moist to wet, fine to medium SAND, trace gravel and silt	S-2		24	< 1.0
	Medium dense, saturated, light brown to gray, fine to medium silty SAND, and trace gravel	S-3	13	12	< 1.0
10	Gravel content decreases	S-4	14	17	< 1.0
	Medium dense, saturated, gray, fine to medium SAND, trace silt	S-5	13	20	< 1.0
15	Grades to predominately fine sand	S-6	13	24	< 1.0
20	Soil density decreases to loose	S-7	10	7	< 1.0
25					

SAMPLE LEGEND

- 2-inch O.D. split spoon sample
- 3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing
- Groundwater level at time of drilling (ATD) or on date of measurement. (11/2/12)

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ├──○──┤ Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-7

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Environmental	Bore Hole Dia.:	8-inch
Top Elevation:	Approximately 48 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Auto
Date Drilled:	10/28/2021	Drill Rig:	Truck Rig	Logged by:	MRC

B-7

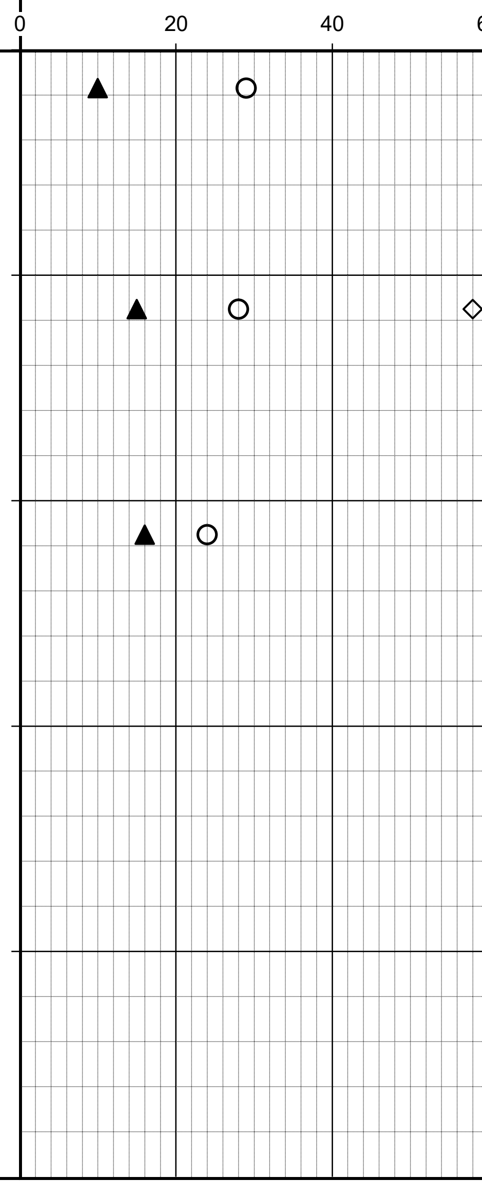
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	SOIL DESCRIPTION	Sample Number	Recovery (In.)	Groundwater	Blowcount	PID
25	Loose, saturated, brown to gray, SAND, some to trace silt, with silt interbeds approximately 1 inch thick	S-8	12		10	< 1.0
30	Stiff to very stiff, saturated to wet, gray, sandy SILT	S-9	18		15	1.3
35		S-10	12		16	1.1
	Boring completed at approximately 36.5 feet. Groundwater was encountered at approximately 6.5 feet ATD.					
40						
45						
50						

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



SAMPLE LEGEND

- 2-inch O.D. split spoon sample
- 3-inch I.D. Shelby tube sample

GROUNDWATER LEGEND

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing
- Groundwater level at time of drilling (ATD) or on date of measurement. 11/2/12

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ⊢—○—⊣ Liquid Limit

Natural Water Content

Jennings Substation
7808 47th Avenue NE
Marysville, Washington

Project No.: 2494.01

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

BORING LOG: B-7

ZIPPER GEO ASSOCIATES, LLC

19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

Test Pit TP-1

Location: See Site and Exploration Plan, Figure 1
Approx. Ground Surface Elevation: Approximately 42 Feet

Project: Jennings Substation
Project No: 2494.01
Date Excavated: September 21, 2021

Depth (ft)	Material Description	Sample	PID	%M	Testing
	Grass over 6 to 8 inches of dark brown, silty sand, some organics, with fine roots (Topsoil) Fine roots extend to approximately 1 foot				
1	Loose, moist, orange-brown, SAND, some silt, trace gravel				
		S-1 @ 1.3 feet	<1		
2					
	Loose, moist, gray-brown, fine to medium SAND, trace gravel and silt	S-2 @ 2 feet	<1	8	
3					
4					
5					
	Soil density increases to medium dense				
6		S-3 @ 5.5 feet	<1	18	GSA
	Moderate to strong seepage observed at approximately 6 feet				
7					
		S-4 @ 7 feet	<1	29	GSA
8	Test pit TP-1 completed at approximately 7.5 feet. Groundwater observed at approximately 6 feet. Test pit was terminated due to severe caving from approximately 6 to 7.5 feet				

ZIPPER GEO ASSOCIATES, LLC

19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

	Test Pit TP-2 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: Approximately 42 Feet	**Project:** Jennings Substation **Project No:** 2494.01 **Date Excavated:** September 21, 2021			
Depth (ft)	**Material Description**	**Sample**	**PID**	**%M**	**Testing**
	Grass over 6 to 8 inches of dark brown, silty sand, some organics, with fine roots (Topsoil) Fine roots extend to approximately 1 foot				
1					
	Loose, moist, orange-brown, SAND, trace silt, trace gravel				
2					
		S-1 @ 2 feet	<1	12	
3					
	Loose, wet, gray to gray-brown, fine SAND, trace gravel, trace silt				
4		S-2 @ 3.5 feet	<1	25	GSA
	Moderate seepage observed at approximately 4.3 feet.				
5					
	Soil density increases to medium dense.	S-3 @ 5.3 feet	<1		
6	Grades to medium sand				
7	Test pit TP-2 completed at approximately 6.5 feet. Groundwater observed at approximately 4.3 feet. Test pit was terminated due to severe caving from approximately 5 to 6.5 feet.				
8					

ZIPPER GEO ASSOCIATES, LLC

19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

Test Pit TP-3

Location: See Site and Exploration Plan, Figure 1
Approx. Ground Surface Elevation: Approximately 41 Feet

Project: Jennings Substation
Project No: 2494.01
Date Excavated: September 21, 2021

Depth (ft)	Material Description	Sample	PID	%M	Testing
	Grass over 8 to 10 inches of dark brown, silty sand, some organics, with fine roots (Topsoil) Fine roots extend to approximately 1 foot				
1	Loose, moist, orange-brown, SAND, trace gravel				
2		S-1 @ 1.5 feet	<1	8	
3	Loose, moist, light brown to gray-brown, fine to medium SAND, trace gravel, trace silt				
		S-2 @ 3 feet	<1	19	GSA
4	Grades to gray at approximately 3.5 feet				
5	Moderate seepage observed at approximately 5 feet				
6	Medium dense, saturated, gray, medium SAND, trace gravel	S-3 @ 5.5 feet	<1		
7					
	Test pit TP-3 completed at approximately 6.8 feet. Groundwater observed at approximately 5 feet. Test pit was terminated due to severe caving from approximately 5.5 to 6.8 feet.				
8					

ZIPPER GEO ASSOCIATES, LLC

19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

	Test Pit TP-4 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: Approximately 41 Feet	**Project:** Jennings Substation **Project No:** 2494.01 **Date Excavated:** September 21, 2021			
Depth (ft)	**Material Description**	**Sample**	**PID**	**%M**	**Testing**
	Grass and blackberries over 6 to 10 inches of dark brown, silty sand, some organics, with fine roots (Topsoil) Fine roots extend to approximately 1 foot				
1	Loose, moist, orange-brown, SAND, trace silt, trace gravel				
2					
		S-1 @ 2 feet	<1	20	
3					
	Loose, wet, gray-brown to gray, fine to medium SAND, trace gravel				
4	Grades to gray at approximately 3.8 feet Moderate seepage observed at approximately 4.3 feet	S-2 @ 3.5 feet	<1	19	
5					
6	Loose, wet, gray-brown to gray, gravelly SAND, trace silt	S-3 @ 5.8 feet	<1	24	GSA
	Seepage rate increases at approximately 6 feet				
7	Mild caving soil conditions observed				
8		S-4 @ 7.5 feet	<1		
	Test pit TP-4 completed at approximately 8 feet. Groundwater observed at approximately 4.3 feet.				

ZIPPER GEO ASSOCIATES, LLC
19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

	Test Pit TP-5 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: Approximately 42 Feet	**Project:** Jennings Substation **Project No:** 2494.01 **Date Excavated:** September 21, 2021			
Depth (ft)	**Material Description**	**Sample**	**PID**	**%M**	**Testing**
	Grass over 2 to 3 inches of dark brown, silty sand, some organics, with fine roots (Topsoil), over loose, moist, dark brown, silty sand, some gravel, trace cobbles, trace organics. Cobbles consist of quarry spalls (Fill)				
1		S-1 @ 0.5 feet	<1		ACM
2	Loose, moist, orange-brown, SAND, trace silt, trace gravel				
3					
		S-2 @ 3.3 feet	<1	10	
4	Loose, moist, light brown to gray, fine SAND, trace silt, trace gravel				
5		S-3 @ 4.5 feet	<1	23	GSA
6	Moderate seepage observed at approximately 5.8 feet				
7	Mild caving soil conditions observed at approximately 6.8 feet				
8	Medium dense, saturated, gray, fine to coarse SAND, trace gravel	S-4 @ 7.5 feet	<1		
	Test pit TP-5 completed at approximately 8 feet. Groundwater was encountered at approximately 5.8 feet.				

ZIPPER GEO ASSOCIATES, LLC

19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

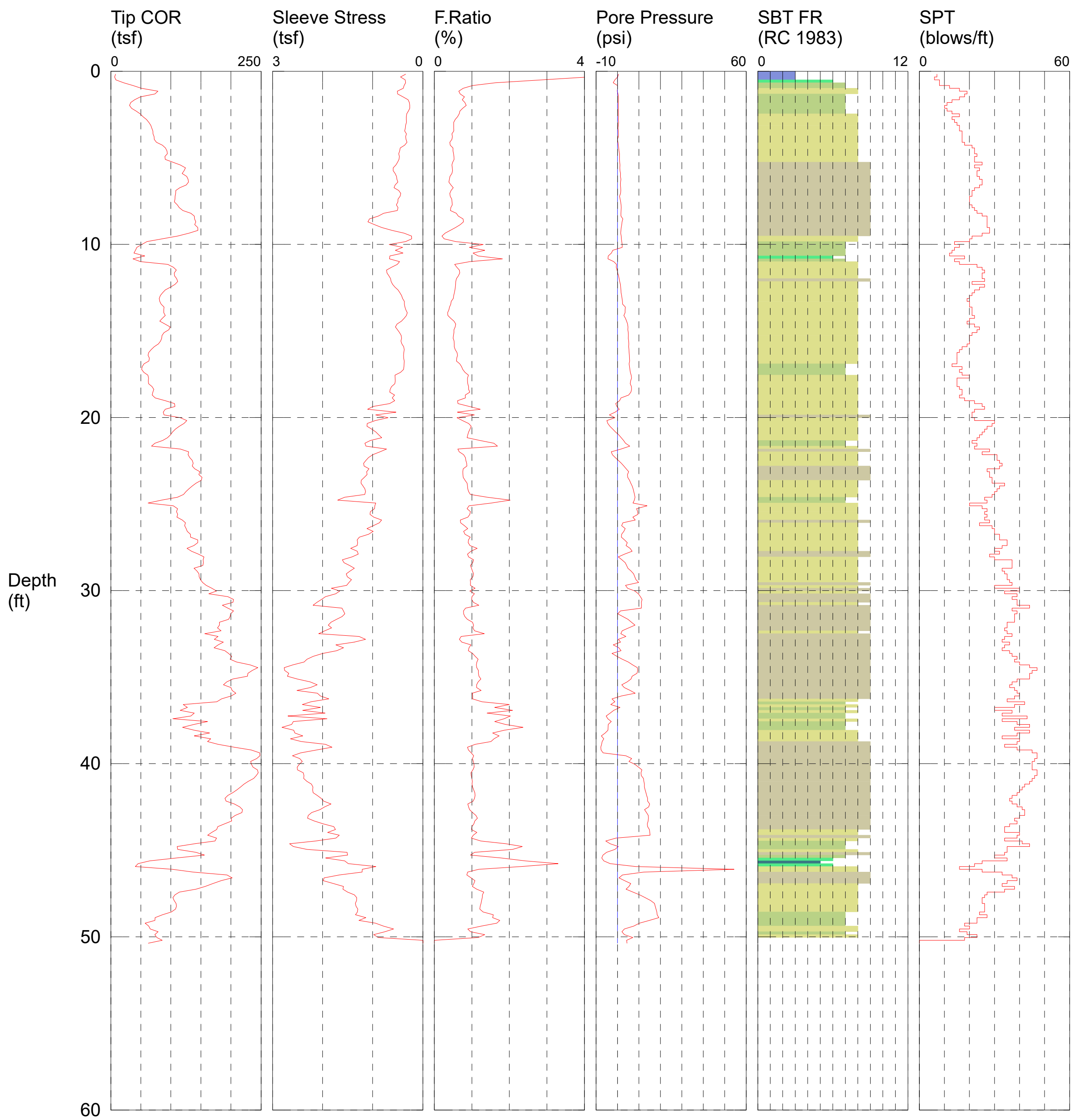
	Test Pit TP-6 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: Approximately 44 Feet	**Project:** Jennings Substation **Project No:** 2494.01 **Date Excavated:** September 21, 2021

Depth (ft)	Material Description	Sample	PID	%M	Testing
	Grass, over 2 inches of dark brown, silty sand, some organics, with fine roots (Topsoil), over medium dense, moist, brown, gravelly SAND, some silt. Coarse sand and fine gravel are crushed rock (Fill)				
1		S-1 @ 0.5 feet	<1	6	ACM
	Several pieces of plastic observed at approximately 1.5 feet				
2		S-2 @ 1.8 feet	<1	8	ACM
	Loose to medium dense, moist, orange-brown, SAND, some silt, trace gravel				
3					
		S-3 @ 3.3 feet	<1	20	GSA
4	Loose to medium dense, moist, gray, fine to medium SAND, trace gravel				
5					
6		S-4 @ 5.5 feet	<1	20	
	Moderate seepage observed at approximately 6 feet.				
7					
8	Grades to medium to coarse sand	S-5 @ 7.5 feet	<1		
	Test pit TP-6 completed at approximately 8.3 feet. Groundwater observed at approximately 6 feet.				

CPT-01

CPT CONTRUCTOR: In Situ Engineering
CUSTOMER: ZipperGeo
LOCATION: Marysville
JOB NUMBER: 000
COMMENT: Snohomish PUD
COMMENT:

OPERATOR: Okbay
CONE ID: DDG1369
TEST DATE: 9/22/2021 9:09:50 AM
PREDRILL: None
BACK FILL: 20% Grout + Bentonite Chips
SURFACE PATCH: None



TOTAL DEPTH: 50.361 ft

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

LABORATORY PROCEDURES AND RESULTS

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 ASTM D 2488. 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 explorations 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.

Atterberg Limits

Atterberg limits are used primarily for classification and indexing of cohesive soils. The liquid and plastic limits are two of the five Atterberg limits and are defined as the moisture content of a cohesive soil at arbitrarily established limits for liquid and plastic behavior, respectively. Liquid and plastic limits were established for selected samples in general accordance with ASTM D 423 and ASTM D 424, respectively. The results of the Atterberg limits are presented on a plasticity chart in this appendix where the plasticity index (liquid limit minus plastic limit) is related to the liquid limit. The plastic limits and liquid limits are also presented adjacent to appropriate samples on the exploration logs in Appendix A.

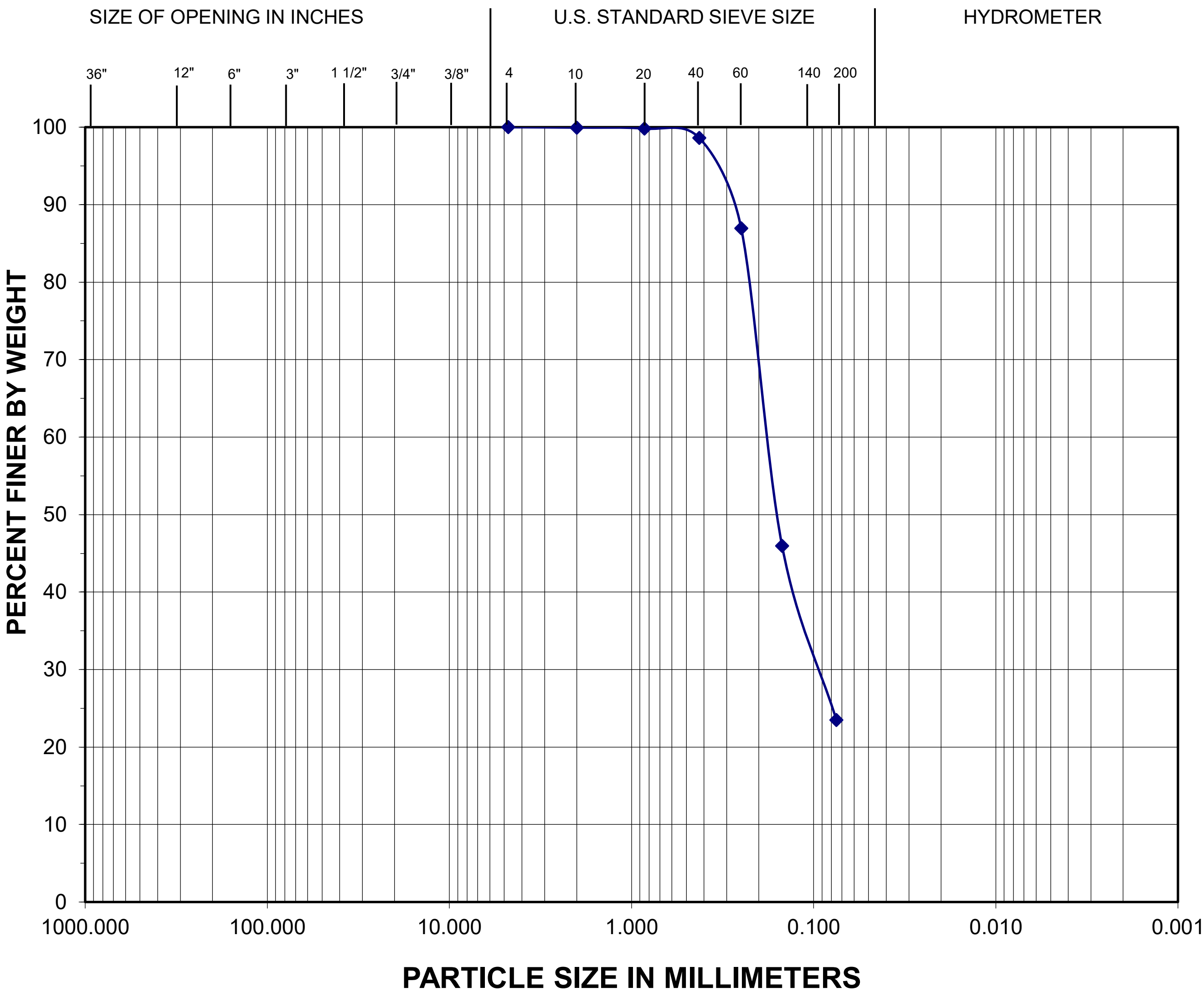
Asbestos Containing Material (ACM)

Five samples of existing fill material were collected from the test pits and 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. The 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:

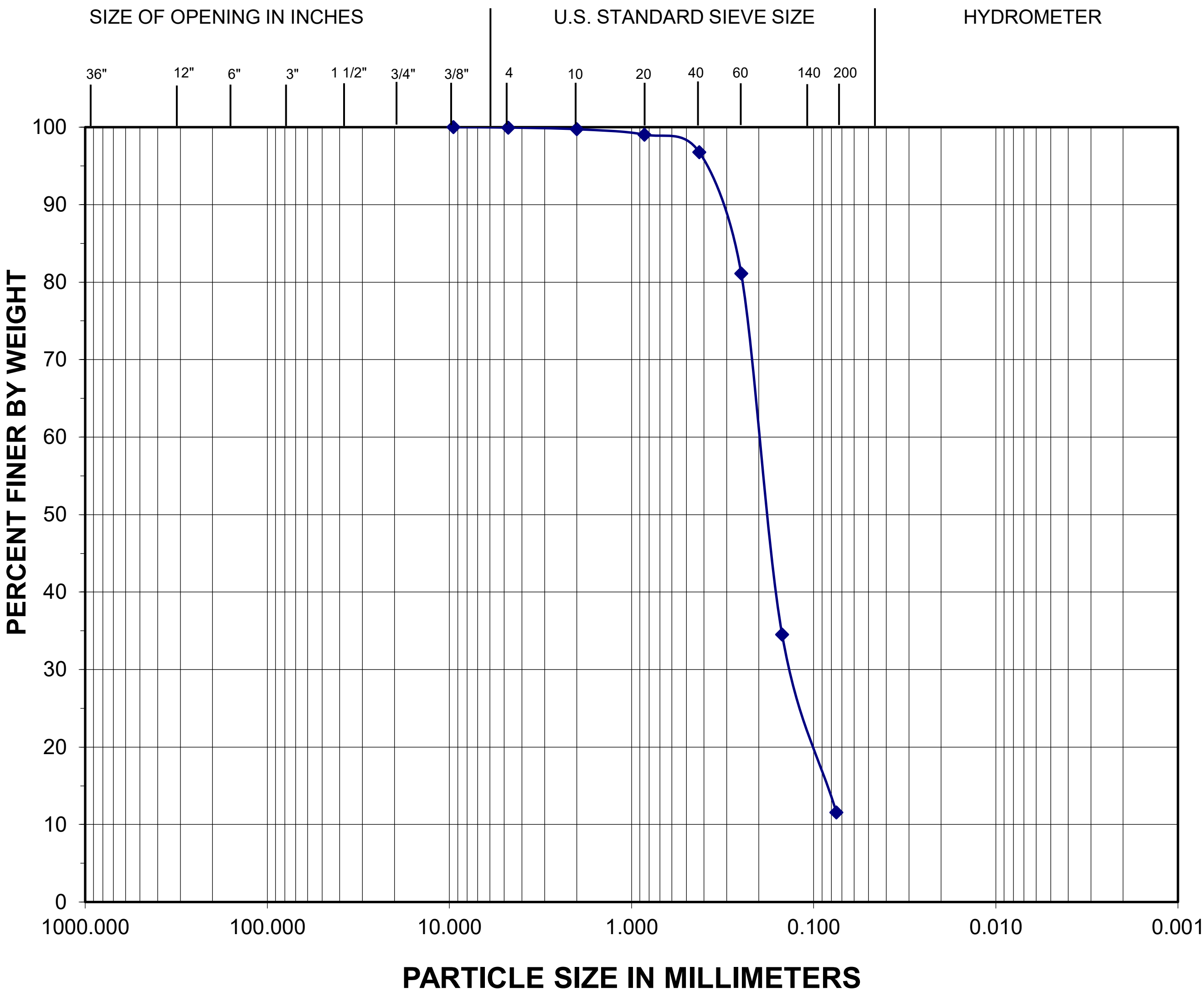
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-5	10-11.5	22.1	23.5	Silty SAND

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 11/4/2021	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

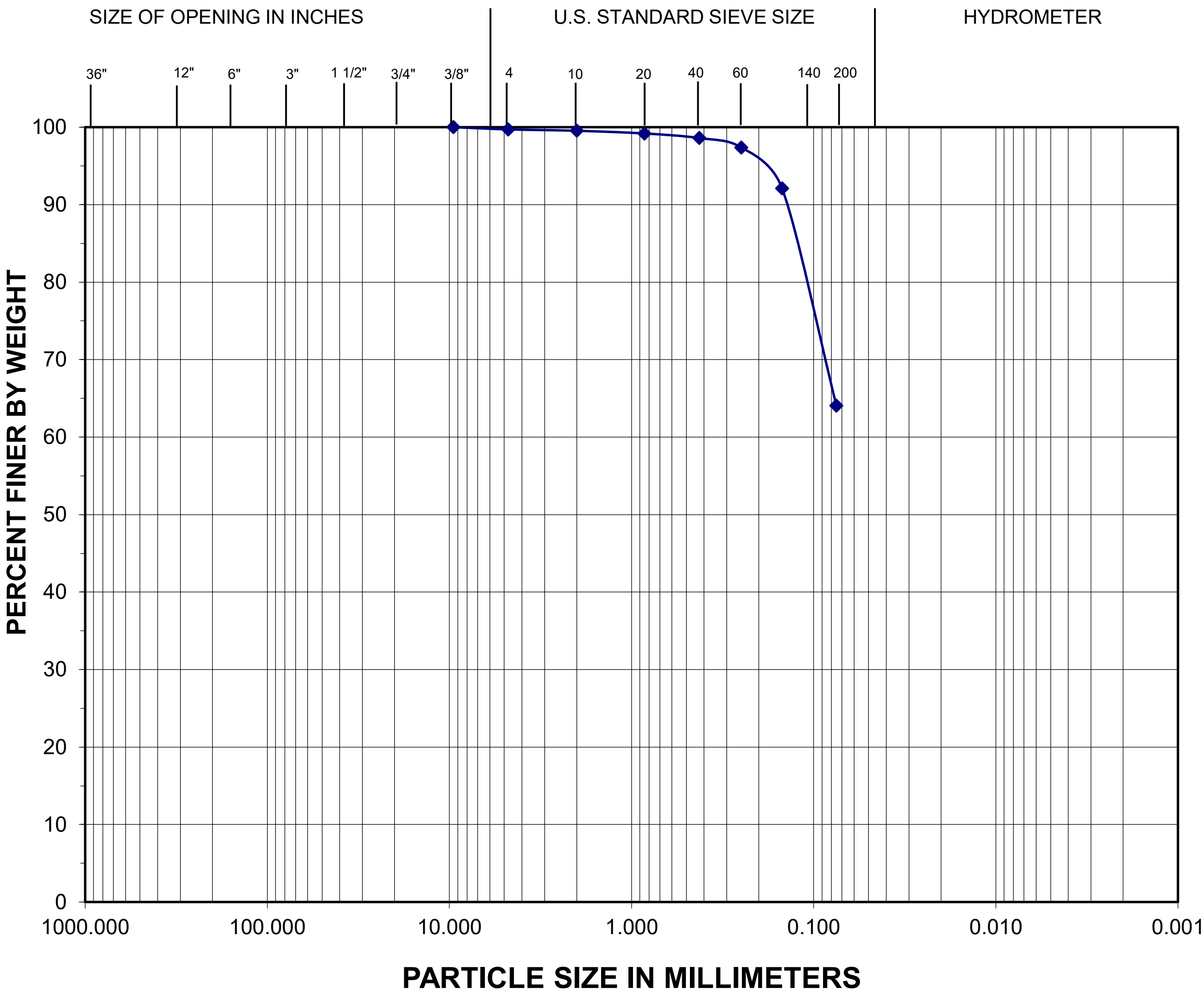
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-9	25-26.5	26.6	11.6	SAND, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 11/4/2021	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

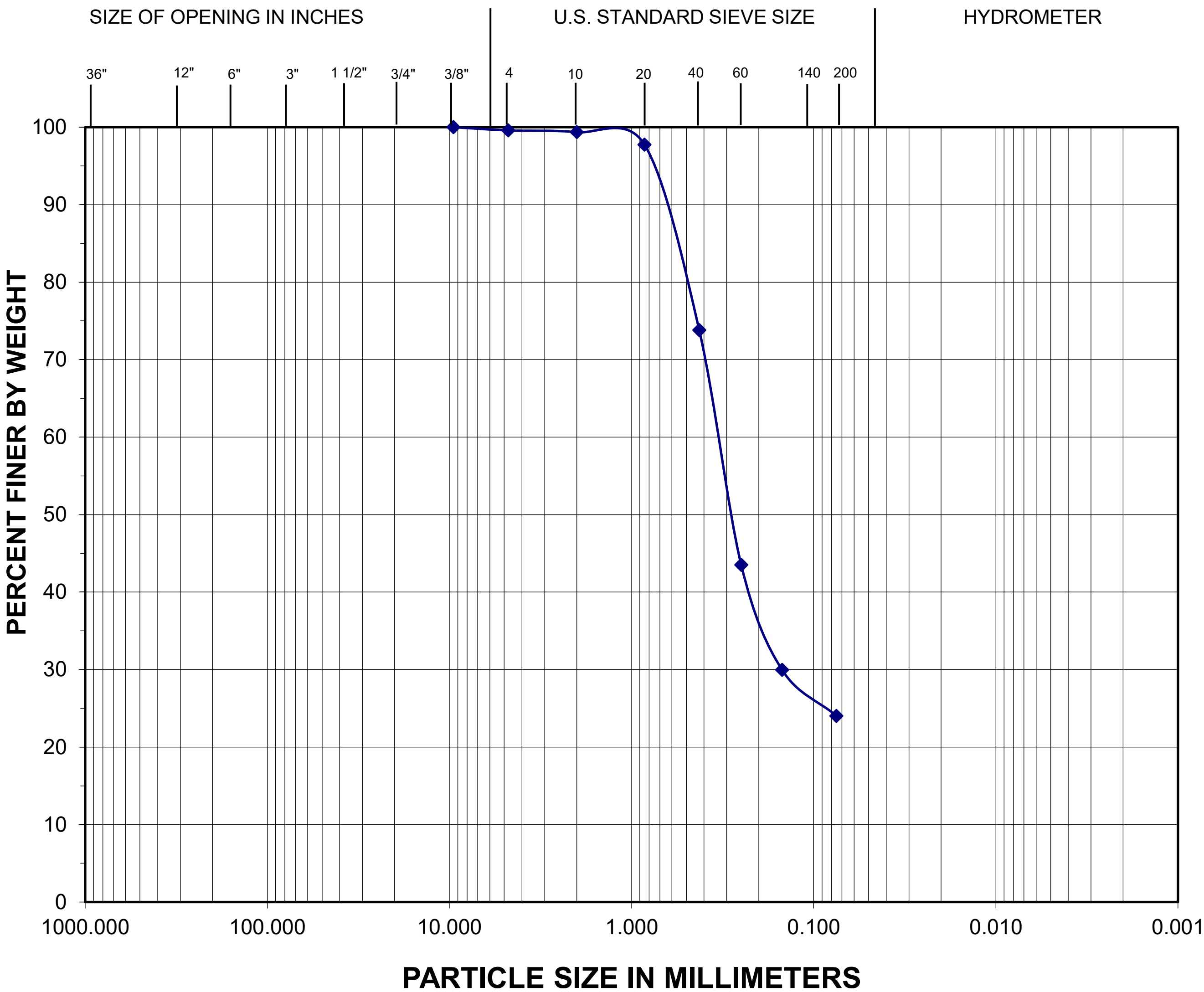
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-13	45-46.5	26.6	64.1	Sandy SILT, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 11/4/2021	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

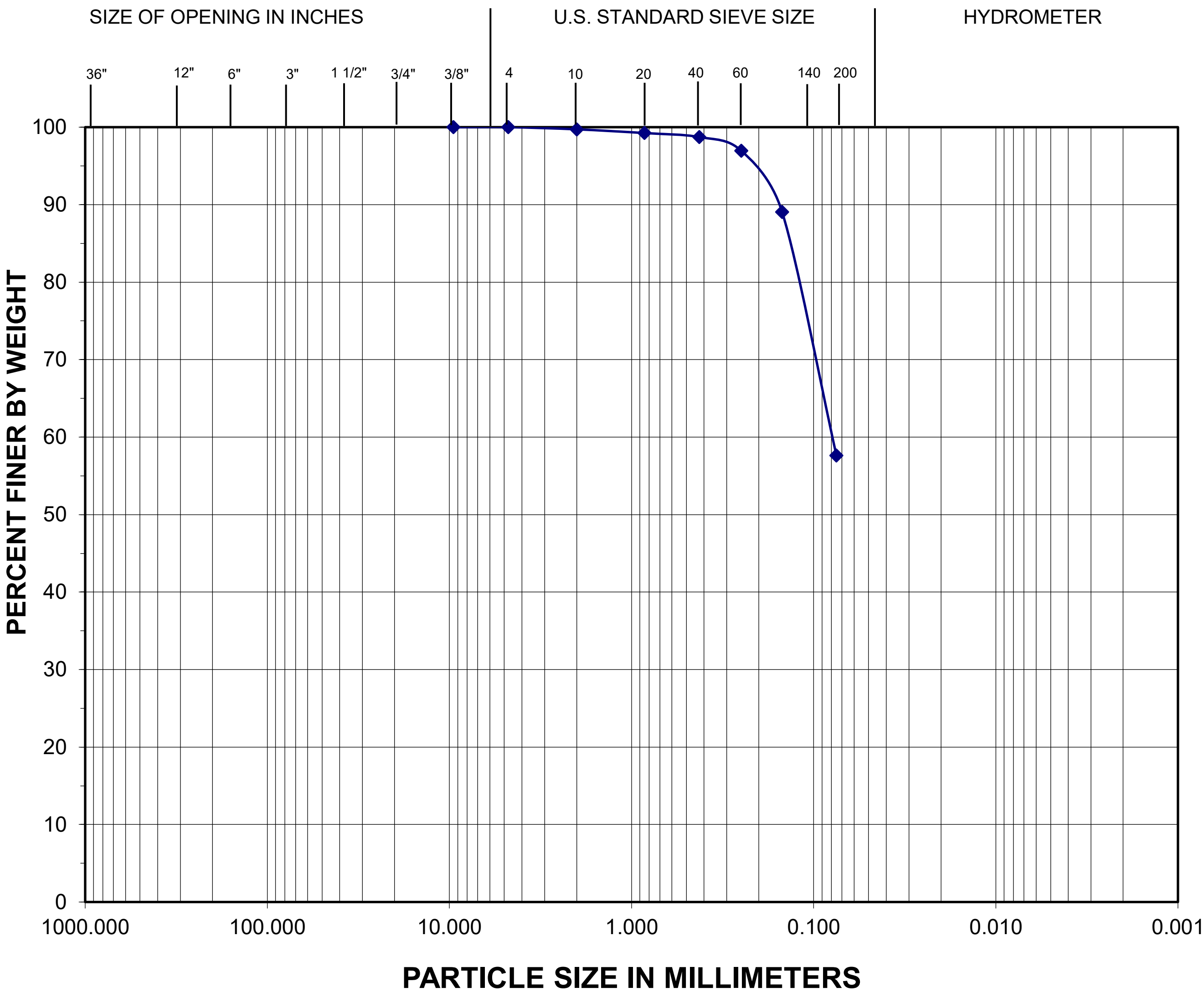
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-7	S-3	7.5-9	26.2	24.0	Silty SAND

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 11/4/2021	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

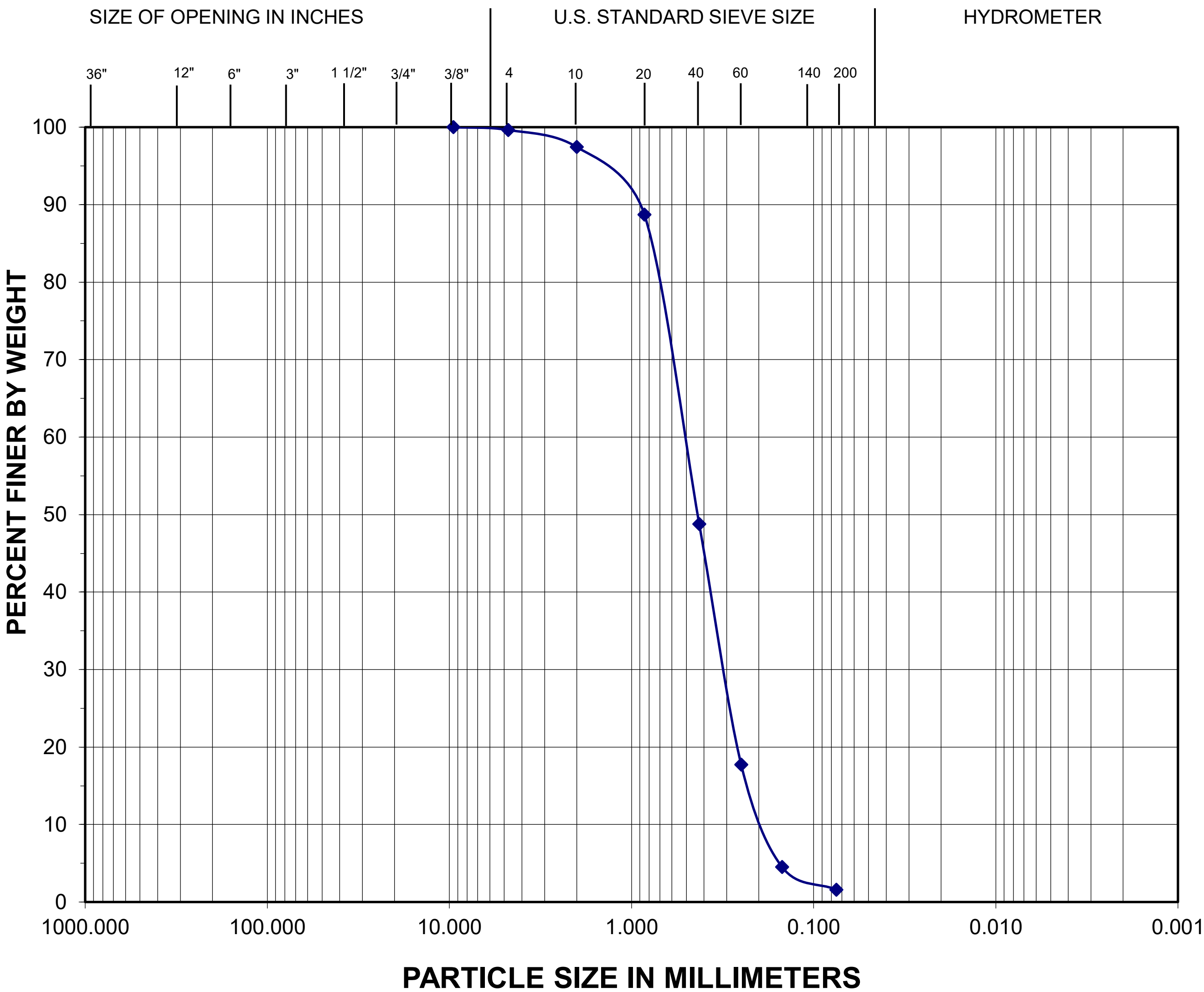
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-7	S-9	30-31.5	28.4	57.6	Sandy SILT

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 11/4/2021	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

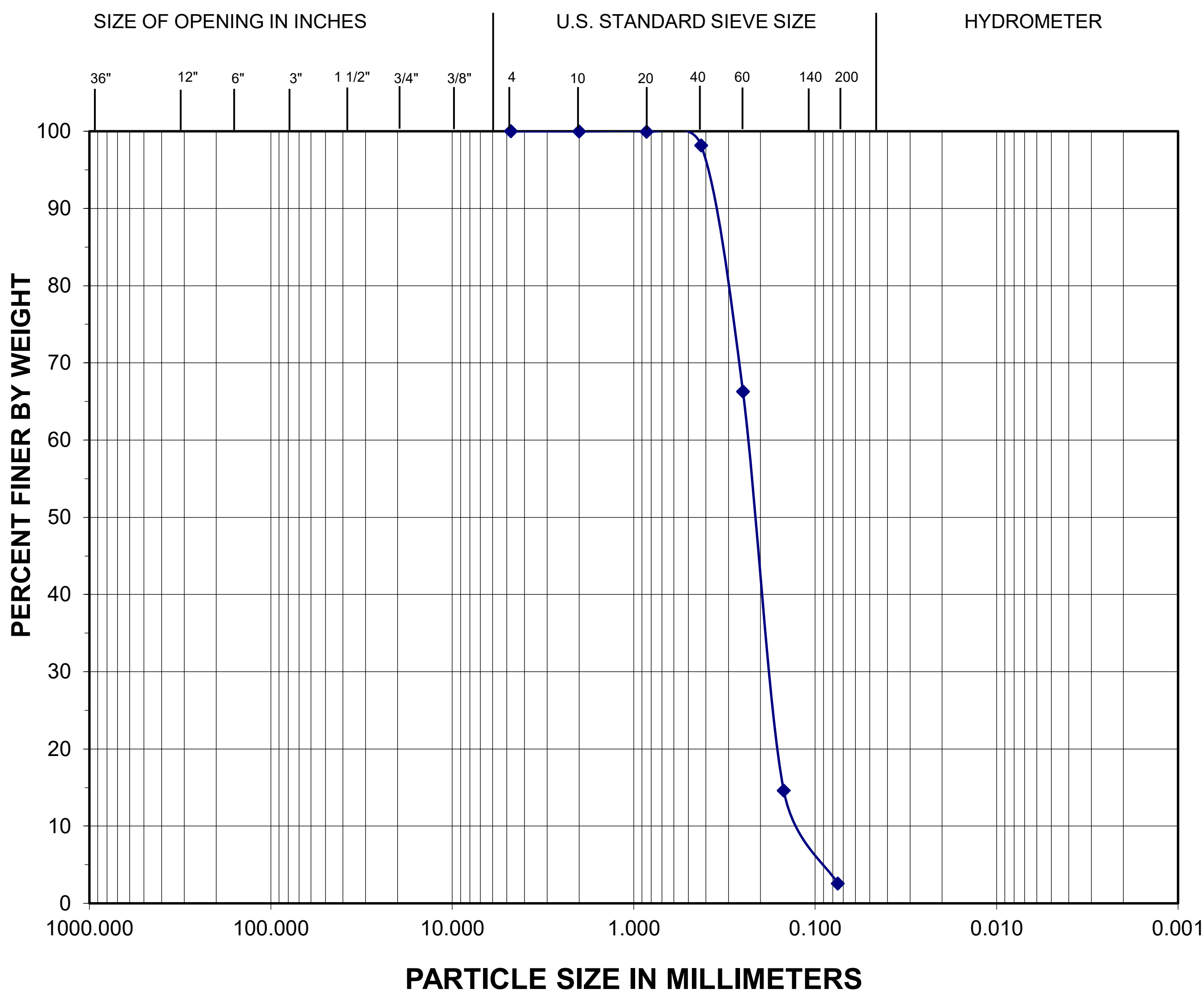
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-1	S-3	5.5	17.5	1.6	SAND, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 9/28-9/30	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

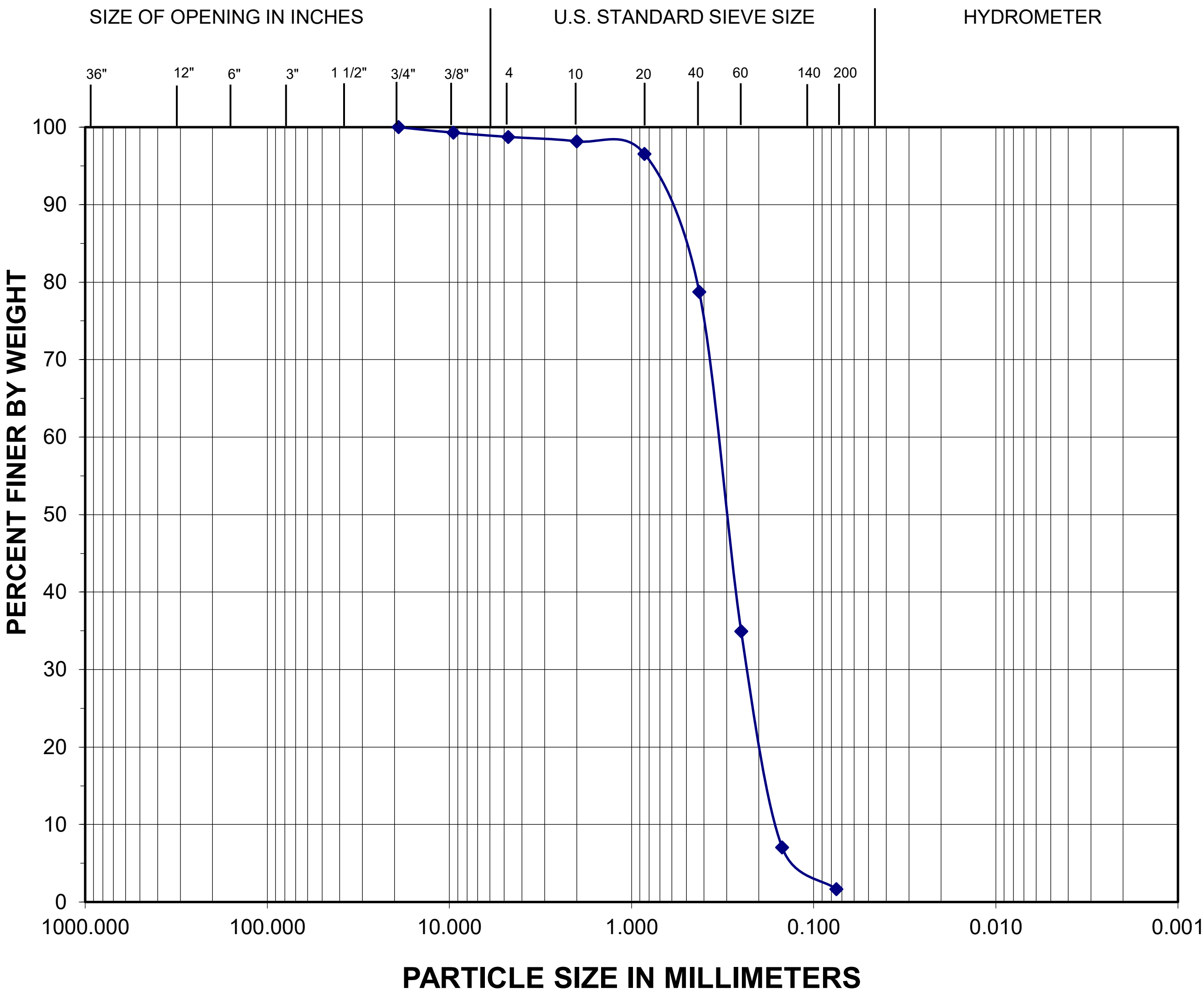
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-1	S-4	7	29.4	2.5	SAND, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 9/28-9/30	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

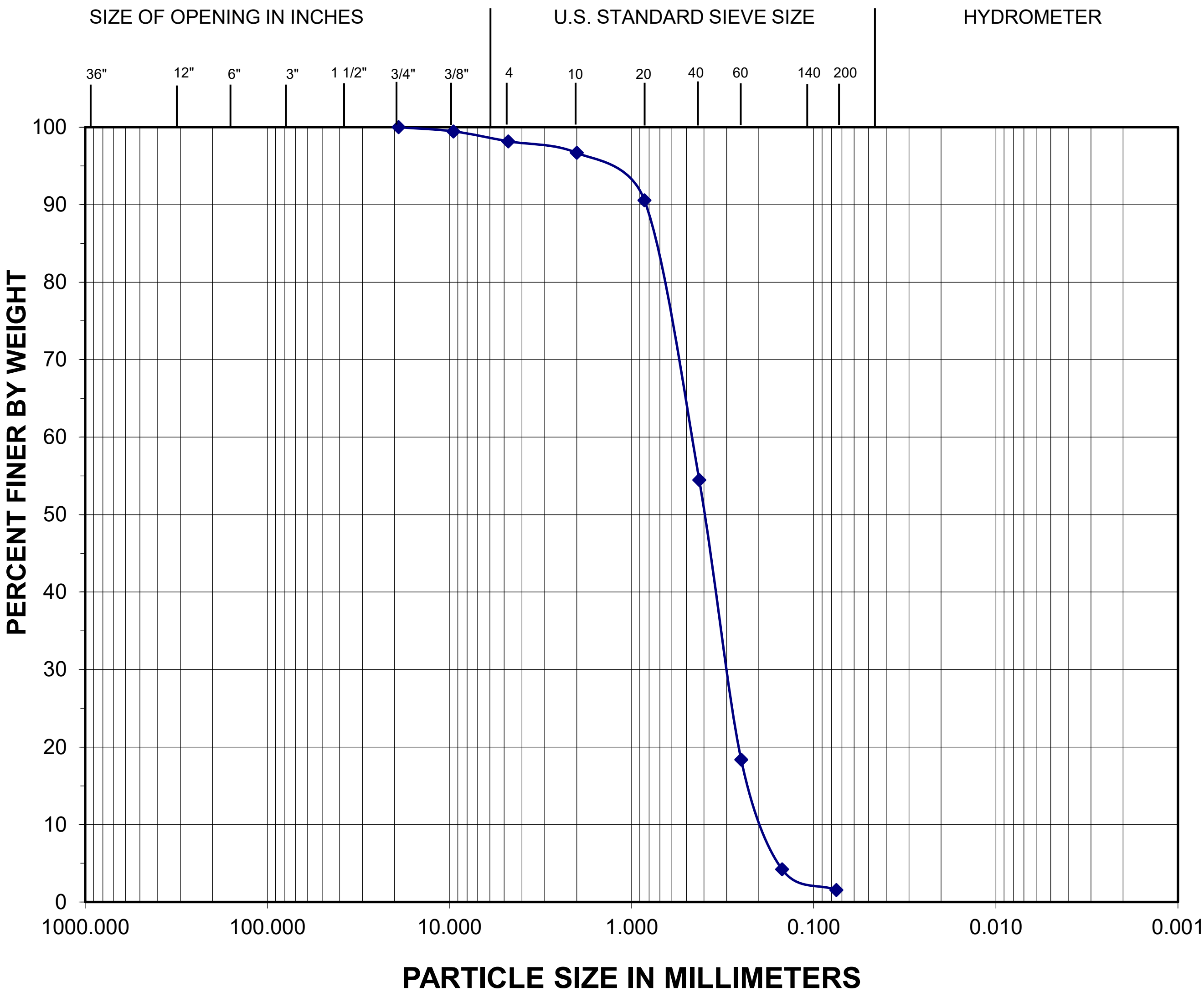
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-2	S-2	3.5	25.1	1.7	SAND, trace silt and gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 9/28-9/30	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

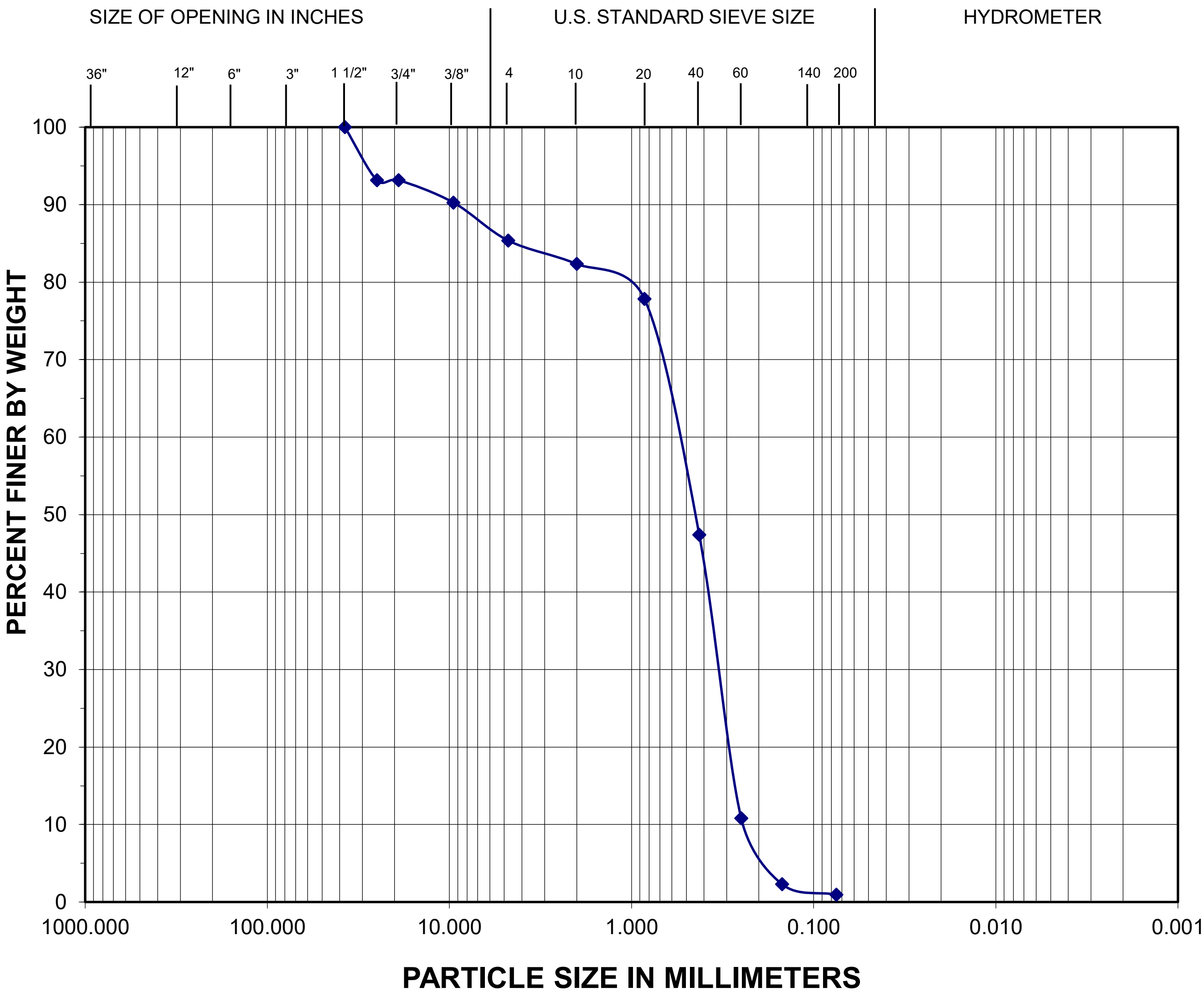
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-3	S-2	3	19.0	1.6	SAND, trace gravel and silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 9/28-9/30	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

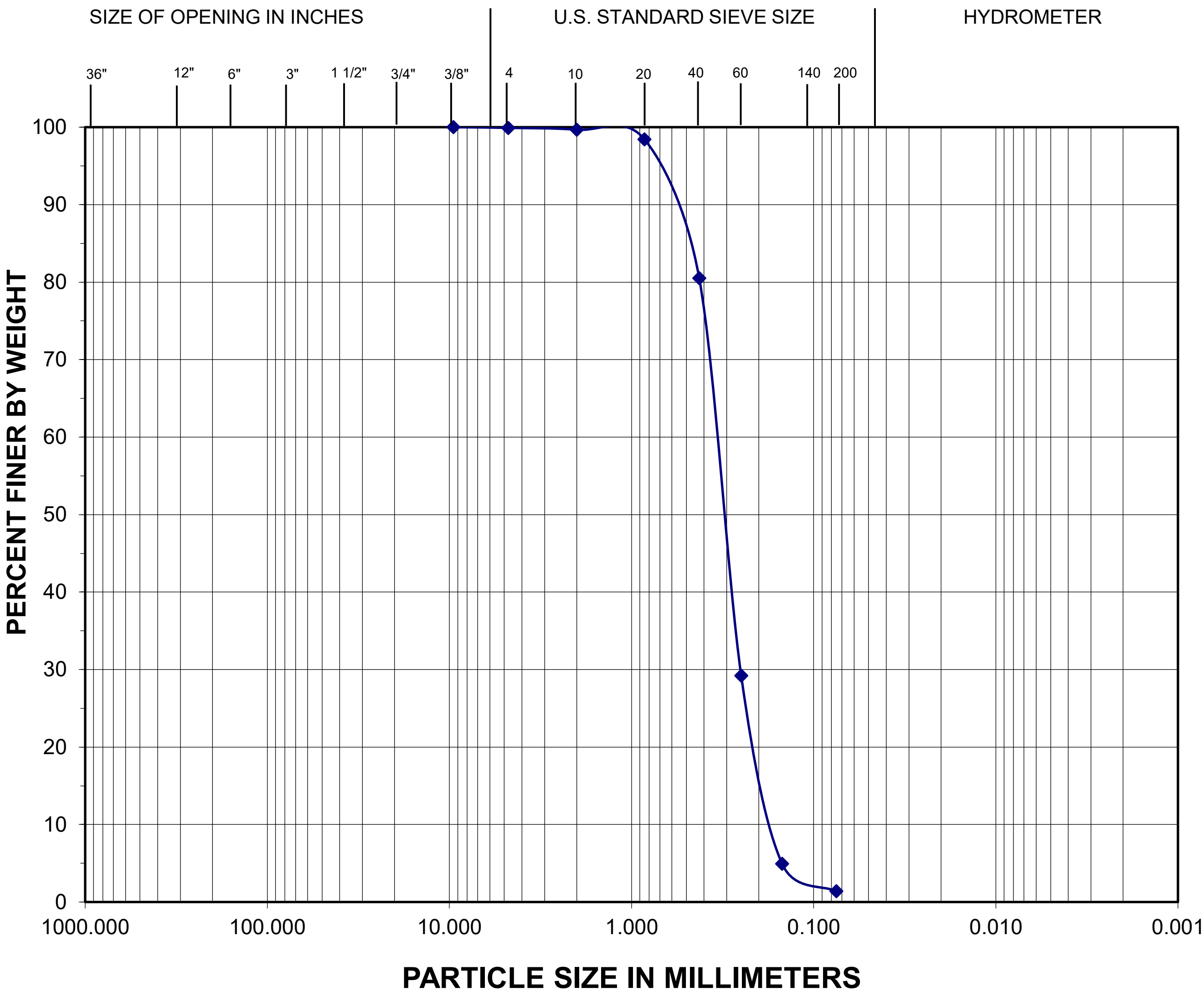
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-4	S-3	3	24.3	1.0	Gravelly SAND

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 9/28-9/30	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

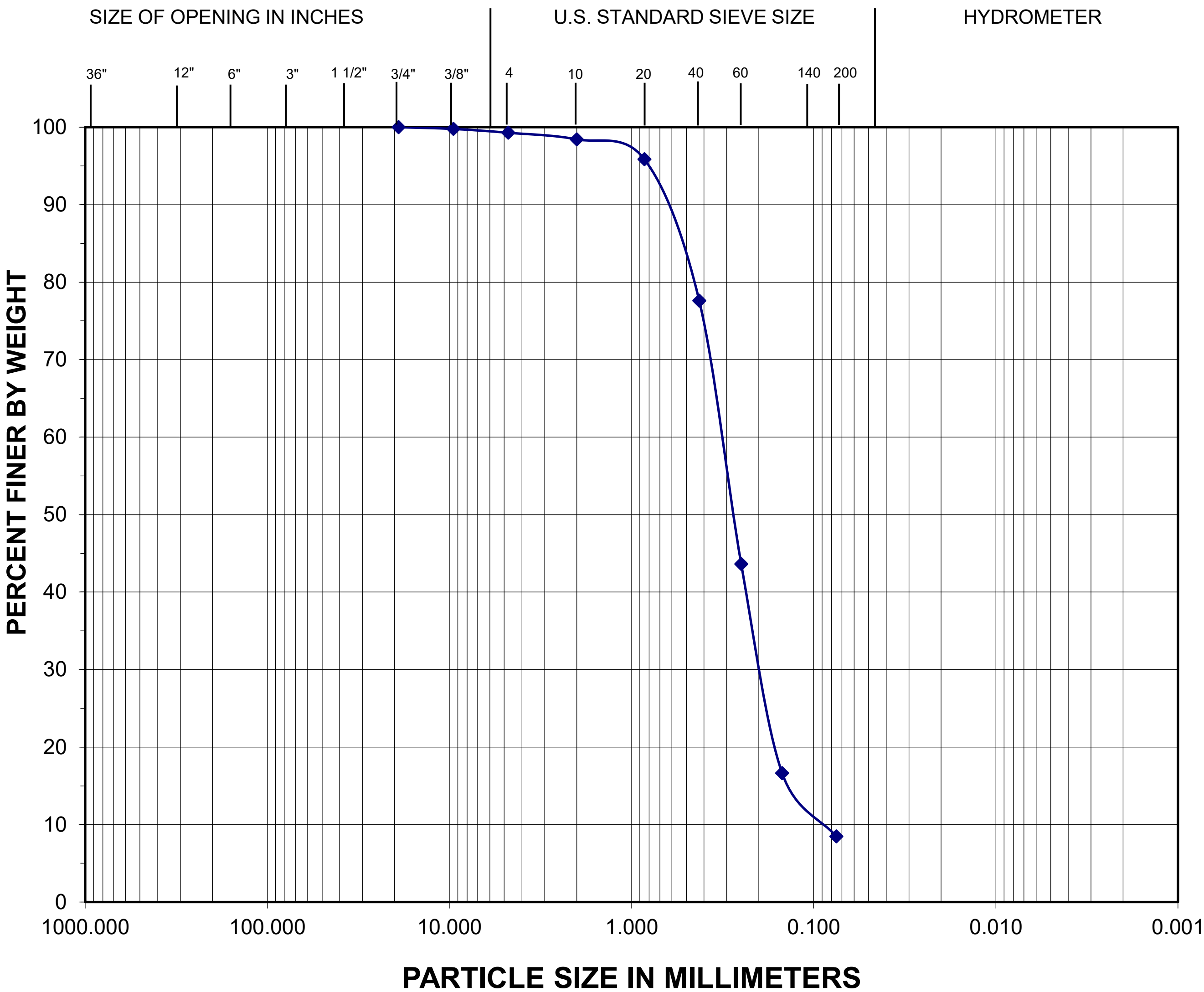
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-5	S-3	4.5	22.7	1.4	SAND, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 9/28-9/30	PROJECT NAME: Jennings Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-6	S-3	3.3	19.8	8.5	SAND, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2494.01 DATE OF TESTING: 9/28-9/30	PROJECT NAME: Jennings Substation

November 8, 2021

Dave Williams
Zipper Geo Associates, LLC
19019 36th Avenue West, Suite E
Lynnwood, WA 98036

RE: Bulk Asbestos Fiber Analysis; NVL Batch # 2119161.00

Client Project: Jennings Substation 2494.01
Location: Marysville, WA

Dear Mr. Williams,

Enclosed please find test results for the 5 sample(s) submitted to our laboratory for analysis on 11/2/2021.

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,

Nick Ly, Technical Director

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. Dave Williams
Project Location: Marysville, WA

Batch #: 2119161.00
Client Project #: Jennings Substation 2494.01
Date Received: 11/2/2021
Samples Received: 5
Samples Analyzed: 5
Method: EPA/600/R-93/116

Lab ID: 21126580 **Client Sample #: TP-5, S-1**

Location: Marysville, WA

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

Layer 1 of 1 **Description:** Brown loose crumbly material with debris

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

Lab ID: 21126581 **Client Sample #: TP-6, S-1**

Location: Marysville, WA

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

Layer 1 of 1 **Description:** Light brown loose crumbly material with debris

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

Lab ID: 21126582 **Client Sample #: TP-6, S-2**

Location: Marysville, WA

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

Layer 1 of 1 **Description:** Gray/brown loose crumbly material with debris

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

Lab ID: 21126583 **Client Sample #: B-2, S-1**

Location: Marysville, WA

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

Sampled by: Client
Analyzed by: Hilary Crumley **Date:** 11/05/2021
Reviewed by: Nick Ly **Date:** 11/08/2021

Nick Ly, Technical Director

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 600/M4-82-020 Methods 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. Dave Williams
Project Location: Marysville, WA

Batch #: 2119161.00
Client Project #: Jennings Substation 2494.01
Date Received: 11/2/2021
Samples Received: 5
Samples Analyzed: 5
Method: EPA/600/R-93/116

Layer 1 of 1 **Description:** Brown loose crumbly material with debris

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

Lab ID: 21126584 **Client Sample #: B-3, S-1**

Location: Marysville, WA

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

Layer 1 of 1 **Description:** Tan loose crumbly material with debris

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

Sampled by: Client
Analyzed by: Hilary Crumley **Date:** 11/05/2021
Reviewed by: Nick Ly **Date:** 11/08/2021

Nick Ly, Technical Director

ASBESTOS LABORATORY SERVICES

Company	Zipper Geo Associates, LLC	**NVL Batch Number**	**2119161.00**
Address	19019 36th Avenue West, Suite E Lynnwood, WA 98036	**TAT**	5 Days **AH** No
		Rush TAT	
Project Manager	Mr. Dave Williams	**Due Date**	11/9/2021 **Time** 10:35 AM
Phone	(425) 582-9928	**Email**	dwilliams@zippergeo.com
Cell	(425) 218-4619	**Fax**	(425) 582-9930

Project Name/Number: Jennings Substation 2494.01 **Project Location:** Marysville, 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	21126580	TP-5, S-1		A
2	21126581	TP-6, S-1		A
3	21126582	TP-6, S-2		A
4	21126583	B-2, S-1		A
5	21126584	B-3, S-1		A

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

Office Use Only	Print Name	Signature	Company	Date	Time
Received by	Hieu Ta		NVL	11/2/21	1035
Analyzed by	Hilary Crumley		NVL	11/5/21	
Results Called by					
☐ **Faxed** ☐ **Emailed**					

Special Instructions: Samples were dried prior to analysis.

Date: 11/2/2021
Time: 3:24 PM
Entered By: Fatima Khan

NVL
INDUSTRIAL HYGIENE SERVICES
LABORATORY • MANAGEMENT • TRAINING

WALK-IN SAMPLE SUBMITTAL FORM

❑ Asbestos

2119161

First DAVE Last WILLIAMS Company ZIPPER GEO ASSOC., LLC
Address 19019-36TH AVE. WEST, S.E Cell (425) 218-4619
LYNNWOOD, WA 98036 Email dwilliams@zippergeo.com
Phone 425-218-4619

Project Name/Number JENNINGS SUBSTATION 2494.01 Project Location MARYSVILLE, WA

Pricing	1-Hr	2-Hr	4-Hr	1-Day
Asbestos	75.00	70.00	65.00	50.00
Lead	N/A	75.00	70.00	50.00
Mold	N/A	N/A	105.00	82.50

Turn Around Time
- ❑ 1 Hour (Asbestos only)
- ❑ 2 Hours (Lead only)
- ❑ 4 Hours (Asbestos, Lead, & Mold)
- ❑ 24 Hours (Asbestos, Lead, & Mold)

5-DAY TURNAROUND

Total Number of Samples 5

	Sample ID	Description	A/R
1	TP-5, S-1	EPA 600/R-93-116 ASBESTOS BY PLM	
2	TP-6, S-1		
3	TP-6, S-2		
4	B-2, S-1		
5	B-3, S-1		
6			
7			
8			
9			
10			

	Print Name	Signature	Company	Date	Time
Sampled by	DAVE WILLIAMS	David C Williams	ZGA	11.2.21	
Relinquish by					

Office Use Only

	Print Name	Signature	Company	Date	Time
Received by	Hieu Ta	Hieu Ta	NVL Labs	11/2/21	1035 DB
Analyzed by					
Called by					
Faxed/Email by					

2119161

Kelly Au Vu

From: Dave Williams <dwilliams@zippergeo.com>
Sent: Tuesday, November 2, 2021 12:47 PM
To: Client Services
Subject: RE: Jennings Substation - On Hold

Sorry about that. "I relinquish my samples to NVL Labs."

Regards,

David C. Williams, LG, LEG, Principal
ZipperGeo
Geoprofessional Consultants
19019 - 36th Avenue West, Suite E
Lynnwood, Washington 98036
Office: 425-582-9928
Mobile: 425-218-4619
www.zippergeo.com

From: Client Services <ClientServices@nvllabs.com>
Sent: Tuesday, November 2, 2021 12:46 PM
To: Dave Williams <dwilliams@zippergeo.com>
Cc: Client Services <ClientServices@nvllabs.com>; Hilary Crumley <Hilary.C@nvllabs.com>; Hieu Ta <hieu.t@nvllabs.com>
Subject: Jennings Substation - On Hold
Importance: High

Hi Dave,

Please see the attached COC.

We are missing the relinquished by signature at the bottom of the page, please sign and return at your earliest convenience. If you are unable to digitally sign, please respond to this email stating, "I relinquish my samples to NVL Labs."

We will be placing this batch on hold.
Thanks & Regards,

Client Services

NVL
INDUSTRIAL HYGIENE SERVICES
LABORATORY • MANAGEMENT • TRAINING

www.nvllabs.com
Your feedback is very important to us!

ph: 206.547.0100 | fax: 206.634.1936

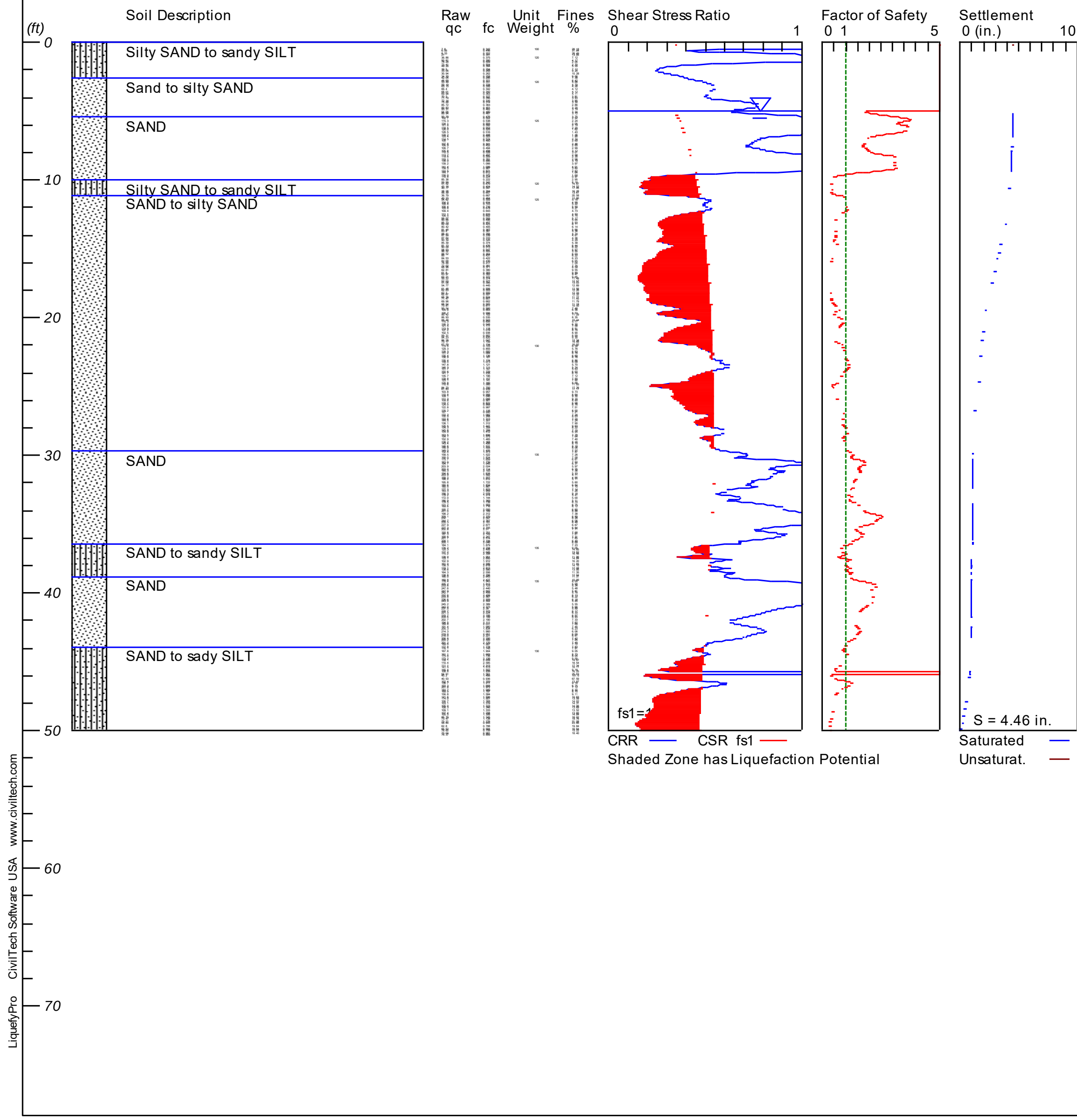
APPENDIX C

LIQUEFACTION ANALYSIS OUTPUT PLOT

LIQUEFACTION ANALYSIS

Jennings Substation, Proj. No. 2494.01

Hole No.=CPT-01 Water Depth=5 ft Surface Elev.=Approx. 42 ft. Magnitude=7.08 Acceleration=0.532g



CivilTech Corporation **Liquefaction Analysis** **Plate A-1**

APPENDIX D
GROUNDWATER MOUNDING ANALYSIS DATA SHEETS

Saturated and Unsaturated Data Input

Project Name: 2494.01, Entry, MS and CSBC Trial, 2.8.23

Unsaturated Analysis:
- ○ Yes
- ◉ No

Runoff Data: MANUAL

Overflow: NONE

Design High Water Elevation: 45.50

Area at Starting Water Level (ft²):	6400.00
Volume Between Starting Water Level &Estimated High Water Level (ft³):	6280.00
Pond Length to Width Ratio (L/W):	2.80
Elevation of Effective Aquifer Base (ft):	17.82
Elevation of Seasonal High Groundwater Table (ft):	41.50
Elevation of Starting Water Level (ft):	41.50
Elevation of Pond Bottom (ft):	43.25
Average Effective Storage Coefficient of Soil for Unsaturated Analysis:	0.30
Unsaturated Vertical Hydraulic Conductivity (ft/d):	24.00
Factor of Safety for Kvu (typically 2.0):	1.00
Saturated Horizontal Hydraulic Conductivity (ft/d):	520.00
Average Effective Storage Coefficient of Soil for Saturated Analysis:	0.30
Average Effective Storage Coefficient of Pond (typically 1.0):	1.00

Specify Hydraulic Control Features

Groundwater Control: ☐ Top ☐ Bottom ☐ Left ☐ Right

Distance to Edge of Pond:

Elevation of Water Level:

Impervious Barrier: ☐ Top ☐ Bottom ☐ Left ☐ Right

Elevation of Barrier Bottom:

Runoff Data: Manual

Stress Period Number	Increment of Time (hrs)	Volume of Runoff (ft³)	Stress Period Number	Increment of Time (hrs)	Volume of Runoff (ft³)
Unsat	0.00	0.00	15	15.00	101.93
1	1.00	5.23	16	16.00	60.11
2	2.00	1,664.88	17	17.00	39.20
3	3.00	577.62	18	18.00	28.75
4	4.00	229.98	19	19.00	20.91
5	5.00	117.60	20	20.00	15.68
6	6.00	67.98	21		
7	7.00	182.95	22		
8	8.00	1,889.64	23		
9	9.00	1,735.44	24		
10	10.00	10,015.32	25		
11	11.00	14,764.20	26		
12	12.00	1,591.68	27		
13	13.00	4,390.85	28		
14	14.00	188.18	29		

✔ OK ✘ Cancel

Saturated and Unsaturated Data Input

Project Name: 2494.01, Yard MS and CSBC, 2.8.23

Unsaturated Analysis: ○ Yes ◉ No

Runoff Data: MANUAL   Overflow: NONE

Design High Water Elevation: 45.67

Area at Starting Water Level (ft^2):	41270.00
Volume Between Starting Water Level &Estimated High Water Level (ft^3):	185715.00
Pond Length to Width Ratio (L/W):	1.00
Elevation of Effective Aquifer Base (ft):	17.82
Elevation of Seasonal High Groundwater Table (ft):	41.50
Elevation of Starting Water Level (ft):	41.50
Elevation of Pond Bottom (ft):	42.00
Average Effective Storage Coefficient of Soil for Unsaturated Analysis:	0.30
Unsaturated Vertical Hydraulic Conductivity (ft/d):	24.00
Factor of Safety for Kvu (typically 2.0):	1.00
Saturated Horizontal Hydraulic Conductivity (ft/d):	520.00
Average Effective Storage Coefficient of Soil for Saturated Analysis:	0.30
Average Effective Storage Coefficient of Pond (typically 1.0):	0.40

Specify Hydraulic Control Features

Groundwater Control: ☐ Top ☐ Bottom ☐ Left ☐ Right

Distance to Edge of Pond:

Elevation of Water Level:

Impervious Barrier: ☐ Top ☐ Bottom ☐ Left ☐ Right

Elevation of Barrier Bottom:

Runoff Data: Manual

Stress Period Number	Increment of Time (hrs)	Volume of Runoff (ft^3)	Stress Period Number	Increment of Time (hrs)	Volume of Runoff (ft^3)
Unsat	0.00	0.00	15	15.00	366.43
1	1.00	14.38	16	16.00	219.54
2	2.00	5,896.80	17	17.00	143.23
3	3.00	2,111.27	18	18.00	99.32
4	4.00	834.52	19	19.00	72.14
5	5.00	421.57	20	20.00	54.36
6	6.00	245.94	21		
7	7.00	614.46	22		
8	8.00	6,701.80	23		
9	9.00	6,278.39	24		
10	10.00	35,425.50	25		
11	11.00	53,090.60	26		
12	12.00	5,885.04	27		
13	13.00	1,637.16	28		
14	14.00	696.26	29		

✓ OK   ✗ Cancel