GEOTECHNICAL ENGINEERING REPORT
CROSSWIND SUBSTATION
17601 – 59th Avenue NE
Arlington, Washington

Project No. 2679.01
19 September 2023

Prepared for:
Snohomish County PUD No. 1

Prepared by:

ZipperGeo

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Project No. 2679.01
19 September 2023

Snohomish County PUD No. 1
Distribution & Engineering Services Division, PO Box 1107
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Attention: Mr. Jeff Colon, PE, Principal Engineer

Subject: Geotechnical Engineering Report
Crosswind Substation
17601 – 59th Avenue NE
Arlington, Washington

Dear Mr. Colon:

In accordance with your request, Zipper Geo Associates, LLC (ZGA) has completed the subsurface exploration and geotechnical engineering evaluation for the proposed Crosswind 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 Professional Services Contract No. CW2250618. Written authorization to proceed was provided by the District on 2 February 2023. We appreciate the opportunity to be of service to you on this project. If you have any questions or if we may be of further service, please do not hesitate to contact us.

Sincerely,
Zipper Geo Associates, LLC

David C. Williams, LG, LEG
Principal Engineering Geologist
Signed 9.19.23

Justin L. Brooks, L.G.
Project Geologist
Signed 9.19.23

Robert A. Ross, P.E.
Managing Principal
Signed 9.19.23

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INTRODUCTION

This report summarizes the geotechnical engineering exploration and analysis completed for the proposed Crosswind Substation project in Arlington, Washington. Two borings (B-1 and B-2), six test pits (TP-1 through TP-6), and one cone penetrometer (CPT-1) were completed by ZGA to depths ranging from approximately 8 to 60 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 a relatively flat 1.4-acre gravel-surfaced lot in the southeast corner of the District's Arlington Microgrid facility. The site is located 0.2 miles south of 180th Street NE and 0.4 miles east of 59th Avenue NE and near the BNSF Railroad right-of-way. The railroad right-of-way adjoins the site at the east, industrial/commercial buildings and lots are to the south, and District facilities lie north and east. An asphalt-paved access drive is on the west. A stormwater infiltration drywell is located a short distance west of the site's northwest corner. The site and immediate vicinity are illustrated on the *Site and Exploration Plan*, Figure 1.

Site History

The proposed substation location is one of several function-specific areas within the Microgrid facility. The District retained GeoEngineers to completed multiple phases of geotechnical exploration and analysis, and we have relied upon information provided in the reports to supplement ZGA's substation-specific exploration and analysis. The GeoEngineers reports that we reviewed are listed below, and selected exploration logs are included in Appendix A:

- GeoEngineers, *Hydrogeologic Assessment, Proposed Pole Yard, Arlington, Washington*, File No. 0482-051-03, dated 26 April 2016;

- ____, *Geotechnical Engineering Services, North County Project, Arlington, Washington*, File No. 0482-051-03, dated 29 December 2017;

- ____, *Updated Groundwater Monitoring Data (Addendum No. 2), North County Project, Arlington, Washington*, File 0482-051-04, dated 20 June 2018;

- ____, *Geotechnical Engineering Services, Update 1 – Revision 1, North County Community Office Project, Early Site Development Phase, Arlington, Washington*, Field Nol 0482-051-04, dated 5 February 2021.

Project Description

A new double bank substation is proposed for construction on the site. Site improvements are expected to include:

- Dead end towers (termination structures) in the southern 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 and substation crushed rock placement to achieve a yard finished grade of approximately elevation 138 feet.

SITE CONDITIONS

Surface Conditions

The substation site is a relatively level area with ground surface elevations between about 135 and 136 feet. The site is mantled with about 4 to 6 inches of ¾-inch crushed gravel over a non-woven geotextile. A pre-cast concrete and steel vault in the north-central portion of the lot contains a groundwater monitoring well monument (B-9) installed by GeoEngineers in 2017. A fire hydrant is located near the northeast corner near the road. The District has material stored to the north and south of the site. We observed standing water throughout the lot during a site visit on 14 February 2023 following previous heavy rain. The access road on the east side is asphalt, about 20-feet wide and in a serviceable condition.

Subsurface Conditions

Local Geologic Conditions

We assessed the geologic setting of site and the surrounding vicinity by reviewing the *Geologic Map of the Arlington West 7.5 Minute Quadrangle, Snohomish County, Washington* (US Geological Survey, Map MF-1740, 1985). The published geologic mapping indicates the site is underlain by Vashon Recessional Outwash, Marysville Sand Member (Qvrm). 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 minimum thickness of about 65 feet. Subsurface conditions disclosed by the explorations advanced by ZGA and others are consistent with the published mapping. ZGA's explorations disclosed recent 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.

Our understanding of subsurface conditions is based upon observation of six test pits, two borings, and one cone penetrometer test (CPT). In addition, we reviewed the logs of three borings and one test pit completed by GeoEngineers at the substation site. Approximate exploration locations, as well as pertinent surface features, are shown on Figure 1. Observed soil conditions are summarized below.

Fill

The explorations disclosed about 4 to 6 inches of ¾-inch crushed gravel over a non-woven geotextile. We observed ponded water atop this layer throughout the site during our 14 February site visit. We attribute this to a thin layer of silt masking the surface of the geotextile at its interface with the crushed surfacing.

We observed apparent disturbed native soils or undocumented fill material consisting of brown to red-brown sand with trace to some silt extending to depths of approximately 1 to 1.5 feet at the test pit locations. We observed scattered woody debris with sizes ranging from roots to wood bark to an approximately 18-inch long and 4-inch diameter log.

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 medium dense to dense sand with gravel and a low fines content (the soil fraction passing the US No. 200 sieve). The soils above the water table were generally in a moist condition. The test pits were terminated at relatively shallow depths of approximately 7 to 8 feet due to caving associated with the relatively low density and low fines content of the material in combination with groundwater seepage

The deeper recessional deposits as disclosed by CPT-1 consist of medium dense to dense sand. The CPT disclosed horizons of medium dense to dense silty sand as well as horizons of dense to very dense gravelly sand. At about 28 feet bgs (below ground surface), a thin horizon of medium stiff silty clay to clayey silt was identified. Borings B-1 and B-2 disclosed somewhat similar conditions, with medium dense sand with trace silt and a variable gravel content to about 20 feet with medium dense sand to the borings' approximate 26-1/2 foot termination depth.

Groundwater

We observed groundwater seepage at depths of approximately 7 to 8-1/2 feet while excavating the test pits and at approximately 9 feet and 8 feet while advancing borings B-1 and B-2, respectively. We observed groundwater at approximately 10 feet at the infiltration drywell near the northwest corner of the site. At the previously installed well, GeoEngineers, B-9, henceforth referred to as GEB-9, we measured the depth to groundwater at 7.2 feet and 7.7 feet.

Our groundwater observations 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: Groundwater Observations				
Exploration/Feature	**Approximate Groundwater Depth/Elevation (feet)**	**Observation Date**	**Groundwater Depth/Elevation (feet)**	**Observation Date**
B-1	9 / 127	2.28.23	NA**	NA
B-2	8 / 127	2.28.23	NA	NA
TP-1	8-1/2 / 122-1/2	2.28.23	NA	NA
TP-2	7-1/2 / 123-1/2	2.28.23	NA	NA
TP-3	7 / 124	2.28.23	NA	NA
TP-4	7 / 124	2.28.23	NA	NA
TP-5	7-1/2 / 123-1/2	2.28.23	NA	NA
TP-6	7-1/2 / 123-1/2	2.28.23	NA	NA
GEB-9*	7.7 / 129.2	2.28.23	7.2 / 129.7	3.29.23
Deluge drywell	10 / 126	2.14.23	NA	NA
*Groundwater depth measured relative to the rim of the flush-mount well monument at elevation 136.9 feet per District survey **NA Not Applicable				

The GeoEngineers 20 June 2018 *Updated Groundwater Monitoring Data (Addendum No. 2)* includes groundwater data for GEB-9 (advanced in the north-central portion of the substation site) and GEB-4 (advanced about 150 feet west of the substation site's southwest corner). The shallowest groundwater levels observed by GeoEngineers were approximately elevations 128 feet and 130-1/2 feet at the GEB-4 and GEB-9 locations, respectively. These were measurements made manually with an electronic well sounder, rather than data downloaded from transducers installed in the wells. These elevations correspond to approximate depths below existing grade of 7-1/2 feet and 6-1/2 feet at the GEB-4 and GEB-9 locations, respectively.

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 shallow disturbed native soils / undocumented fill material 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 recessional outwash likely to be encountered during grading has a relatively low fines content and may be considered moderately moisture-sensitive relative to grading.

- We anticipate that deeper excavations for vaults and conduits may encounter groundwater during the wetter time of year, most likely necessitating dewatering. 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 native recessional outwash soils is favorable from the stormwater infiltration perspective.

- Based on our analyses, we estimate total settlement resultant from seismically-induced liquefaction of approximately 1 to 3 inches. We estimate differential seismic settlement of approximately ½ to 1½ inches over a horizontal distance of 40 feet. If these levels of seismically-induced liquefaction settlement are not acceptable for conventional spread footings, we recommend considering the installation of stone columns as 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).

Regulated Geologic Hazard Areas

Part V of Chapter 20.93.600 of the Arlington Municipal Code (AMC) defines regulated geologic hazard areas as follows:

"Geologic hazard areas" means lands or areas susceptible to erosion, sliding, earthquakes, liquefaction, or other geological events.

Erosion Hazard Areas

"Erosion hazard areas" are as defined by the USDA Soil Conservation Service, United States Geologic Survey, or by the Department of Ecology Coastal Zone Atlas. The following classes are high erosion hazard areas.

(A) Class 3, class U (unstable) includes severe erosion hazards and rapid surface runoff areas;
(B) Class 4, class UOS (unstable old slides) includes areas having severe limitations due to slope; and,
(C) *Class 5, class URS (unstable recent slides).*

The project site is essentially level and lacks significant slopes. It is our opinion that the site presents a low erosion hazard per the AMC definition.

Landslide Hazard Areas

"Landslide hazard areas" include areas subject to severe risk of landslide based on a combination of geologic, topographic, and hydrologic factors. Landslide hazard include any of the following:

(A) Areas characterized by slopes greater than fifteen percent and impermeable soils (typically silt and clay) frequently interbedded with permeable granular soils (predominantly sand and gravel) or impermeable soils overlain with permeable soils or springs or groundwater seepage; Low Hazard. Areas with slopes of less than 15 percent.
(B) Any area that has exhibited movement during the Holocene epoch (from ten thousand years ago to present) or which is underlain by mass wastage debris of that epoch;
(C) Any area potentially unstable due to rapid stream incision, stream bank erosion or undercutting by wave action;
(D) Any area located on an alluvial fan presently subject to or potentially subject to inundation by debris flows or deposition of stream-transported sediments;
(E) Any area with a slope of thirty-three percent or greater and a vertical relief of ten or more feet except areas composed of consolidated rock;
(F) Any area with slope defined by the United States Department of Agriculture Soil Conservation Service as having a severe limitation for building site development; and,
(G) Any shoreline designated or mapped as class U, UOS, or URS by the Department of Ecology Coastal Zone Atlas.

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 AMC definition.

Seismic Hazard Areas

Seismic Design Considerations: The seismic performance of the proposed site improvements was evaluated in accordance with the 2018 International Building Code (IBC). The seismic basis of design for the 2018 IBC, which refers to the American Society of Civil Engineers (ASCE) 7-16, is a risk-targeted maximum considered earthquake (MCE_R), which represents an earthquake with a 2 percent probability of exceedance in 50 years (2,475-year return period).

Ground Fault Rupture: Based on review of the United States Geological Survey *Quaternary Fault and Fold Database of the United States* the nearest fault to the site is the South Whidbey Island Fault Zone mapped about 17 miles south-southwest of the site. Based on the mapped location of the fault relative to the site, it is our opinion that the risk associated with fault surface rupture at the site is low.

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 the 2018 IBC and ASCE 7-16. Specifically, our analysis used the following primary seismic ground motion parameters.

- A Modified Peak Ground Acceleration (PGA_M) of 0.52g based on Site Class D, per Section 11.8.3 of ASCE 7-16 (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 7.03 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 CLiq (Version 3.5.2.10) developed by GeoLogismiki. Our analysis was based on CPT-01 completed to a depth of about 60 feet below existing grade within the proposed development area and assumed a conservative groundwater depth of 2 feet during the design earthquake. The approximate exploration location is shown on the enclosed *Site and Exploration Plan, Figure 1*. Based on our analysis, a generally non-liquefiable crust of material exists in the upper 15 feet of the site. Below this crust, portions of the Marysville Sand Member have a moderate to high liquefaction potential during the design earthquake down to the full depth of the CPT exploration.

Liquefaction Settlement: The site is mantled by a generally dense and non-liquefiable crust on the order of 15 feet thick. As such, liquefaction-indicated settlements observed at the surface will initiate from potentially liquefiable layers present below the non-liquefiable crust. Research and case histories have shown that the expression of liquefaction-induced settlement at the ground surface is a function of the depth of the liquefiable layers, with deeper liquefiable layers contributing less to ground surface settlement than similar thickness shallow liquefiable layers (Cetin et al., 2009). Cetin proposed use of a "depth weighting factor" (DF_i) that reduces the impact of deep liquefiable layers on the estimated surface settlement. This factor is included in the CLiq program and was used in our settlement analysis.

Based on our analyses, we estimate a total seismic settlement of approximately 1 to 3 inches. We estimate a differential seismic settlement of approximately ½ to 1½ inches over a horizontal distance of 40 feet. If these levels of seismic induced liquefaction settlement are not acceptable for conventional spread footings, we recommend that mitigative measures such as a mat foundation or stone columns be considered.

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. Given the relatively level nature of the site, lack of a free-face condition, and 15-foot-thick non-liquefiable crust, it is our opinion that the potential for distress at the site from lateral spreading is low.

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. Based on the results of the field evaluation, 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 of ASCE 7-16 do not apply, a ground motion hazard analysis may be required. Site Class D describes soils that are considered stiff with a shear wave velocity between 600 and 1,200 feet per second, average Standard Penetration Test values between 15 and 50, and an undrained shear strength between 1,000 and 2,000 psf.

Table 2: IBC Seismic Design Criteria	
Parameter	**Value**
2018 International Building Code Site Classification (IBC) [1]	Site Class F [2,3]
Site Latitude/Longitude	48.1560 /-122.1422
Spectral Short-Period Acceleration, S_S	1.050g
Spectral 1-Second Acceleration, S_1	0.375g
Site Coefficient for a Short Period, F_A	1.080
Site Coefficient for a 1-Second Period, F_V	See ASCE Section 11.4.8
Spectral Acceleration for a 0.2-Second Period, S_{MS}	1.134g
Spectral Acceleration for a 1-Second Period, S_{M1}	See ASCE Section 11.4.8
Design Short-Period Spectral Acceleration, S_{DS}	0.756g
Design 1-Second Spectral Acceleration, S_{D1}	See ASCE Section 11.4.8

1. IBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile.
2. CPT-01 completed by ZGA for this study extended to a maximum depth of about 60 feet below grade. ZGA therefore reviewed logs for CPT-1 and CPT-2 completed by GeoEngineers in 2017 (including shear wave velocity test results) about 2,000 and 1,200 feet west of the site, respectively, to determine IBC Site class with and without regard to liquefaction.
3. Per the *2018 International Building Code* and *ASCE 7-16*, Chapter 20, any profile containing soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils.

Engineering Soil Units

For purposes of describing soil conditions observed at the exploration locations and for reference in other sections of this report, soils with similar engineering characteristics were grouped together into Engineering Stratigraphic Units or ESUs. The following paragraphs provide our interpretation of ESUs encountered at the exploration locations. ESUs are described in a top down stratigraphic sequence described in the logs. The reader is referred to the logs attached in Appendix A for information regarding subsurface conditions.

<u>ESU 1 – Disturbed native soils and undocumented fill:</u> We observed soils interpreted to be disturbed native soil or undocumented fill at the test pit and boring locations to depths of about 1 to 1-1/2 feet below existing site grade. ESU 1 soils generally consisted of loose crushed rock above a non-woven geotextile and underlying loose to medium dense sand with a variable silt and gravel content as well as some woody debris. Engineering properties of ESU 1 soils are characterized as low strength and compressible materials.

<u>ESU 2 – Medium dense recessional outwash and compacted structural fill:</u> Soils interpreted to be shallow medium dense recessional outwash soils were observed at all of the exploration locations. Engineering

properties of ESU 2 soils are characterized as moderate strength low compressibility materials, and compacted structural fill is included in this category.

ESU 3 – Dense recessional outwash: Soils interpreted to be dense recessional outwash soils were observed at a depth of about 18 to 26-1/2 feet at the boring B-1 location, from about 18 to 23 feet at the boring B-2 location, and generally below about 5 feet at the CPT-1 location. Engineering properties of ESU 3 soils are characterized as high strength low compressibility materials.

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: At the time of our site visits, all but a small portion of the north end of the proposed substation had been stripped of vegetation, graded, and covered in a non-woven geotextile and about 4 to 6 inches of crushed gravel. In preparation for grading, we recommend removal of any existing surficial vegetation, root mass, organic topsoil, and deleterious debris if present. These materials should be wasted from the substation footprint.

During our site visits that occurred during or shortly after heavy rain, we observed standing water across the site. We observed that silt and fine sand derived from the crushed gravel borrow that had been placed above the geotextile had washed down to the interface with the geotextile and was masking its surface, reducing its water transmissivity, and allowing standing water to develop. In order to increase the overall infiltration rate of the substation, we recommend stripping the crushed gravel (it may be stockpiled and used subsequently as structural fill) and removing the geotextile.

We also recommend selective removal of existing undocumented fill material or disturbed native soils containing substantial organics or deleterious debris and any relic organic topsoil from within the yard below structure and conduit run locations should it be encountered in excavations.

Variation in the undocumented fill and disturbed native soil depth and composition 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 currently below the geotextile and that will be 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 as this material has already been subject to heavy vehicle traffic and will not be subject to additional loading following construction of the new substation.

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 and gravel 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 concerns 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 moisture-sensitive soils that may be disclosed during grading 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

The District has indicated that the yard will have a finished elevation of 138 feet. This will necessitate placing about 1 to 2 feet of fill to allow placing a minimum of 8 inches of crushed surfacing base course

and 4 inches of substation rock. 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 below future improvements that will bear on these soils, they 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 project’s stormwater management design relies on infiltration occurring through the existing soils following construction of the yard embankment. Consequently, additional compaction of the subgrade soils exposed following removal of the existing geotextile fabric, except as described above, is not recommended. The need for scarification of over-dense soils should be made at the time of construction following ZGA’s subgrade observations.

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.

<u>Re-use of On-site Soils:</u> Soil expected to be encountered in excavations include predominantly native soil typically consisting of sand and gravel with a relatively low fines content. The fines content of shallow soil samples that we tested (as deep as about 12 feet and likely to be encountered in excavations) ranged from approximately 1 to 9 percent. Overall, the native recessional outwash will be well-suited for use as structural fill. We observed the highest fines content in the very shallow soils, and using these materials as structural fill during wet weather could be difficult due to the high fines content and moisture sensitivity.

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

Table 3: 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
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 4: 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 rock ballast 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.

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

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

Excavation Quantities: It has been our experience that grading calculations need to accommodate a "shrink or swell" factor when comparing in-place soil volumes to truck volumes. We recommend considering that the in-place volume of soil removed from excavations will increase by approximately 25 to 40 percent when measured on a loose cubic 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: Groundwater observations and measurements made as of the time that this report was prepared are described in Table 1. In summary, we have observed groundwater at depths of about 7 to 8-1/2 feet on site and at about 10 feet in the nearby UIC deluge drywell. ZGA is continuing to monitor groundwater at the GEB-9 well on site and others nearby, and quarterly summaries will be provided to the District.

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 to one of the open fields to the west of the site, into a nearby storm sewer, or Baker tank. Also, we suggest that the District consider using the existing UIC deluge drywell near the northwest corner of the site for a similar purpose until such time as it is decommissioned. 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 seasonal shallow groundwater condition, 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 medium dense native sand and gravel with a low fines content.

The vaults will exert a relatively low bearing pressure on the existing soils, and we estimate that up to approximately 1/2 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: Vaults installed below groundwater 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 previous GeoEngineers groundwater observations, we recommend considering a seasonal high groundwater elevation of about 130-1/2 feet.

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 medium dense native granular recessional outwash soils and properly compacted structural fill (ESU 2 soils) are adequate for support of shallow foundations. We have been provided with the following maximum compressive loads for the transfer, switchgear enclosure, and dead end structures as follows:

- Transformer: 100 kips
- Switchgear enclosure: 75 kips
- Dead end structures: 75 kips

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 ESU 2 soils. 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).

Recommended criteria for shallow foundations are summarized below.

<u>Net allowable bearing pressure:</u> 3,500 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 for standard column foundation:</u> 4 feet

<u>Minimum base dimension for continuous foundation:</u> 14 inches

Minimum embedment for frost protection: 18 inches

Approximate total settlement: F-type footings: less than 1 inch; transformer slab, switchgear enclosure slab, and dead end structures: less than 1/2 inch

Estimate differential settlement: One half of total settlement over 40 feet

Ultimate passive resistance: 480 pcf. This value assumes that foundations are backfilled with native sand gravel 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 and does not include a factor of safety.

Modulus of subgrade reaction: 25 tons/ft^3.

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. 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 relatively shallow groundwater conditions will likely necessitate the use of casing during installation.

We understand that the District may 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 referenced boring B-2 location.** 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 design of the proposed drilled pier foundations and direct bury poles.

Table 5A: Recommended Soil Parameters Based on Boring B-2					
Depth interval in feet below existing grade	**Soil Condition**	**Average Standard Penetration Resistance (N)**	**Correlated Pressuremeter Elastic Modulus (kips/in²)¹**	**Soil Wet Density (pcf)**	**Internal Friction Angle (Ø, in degrees)**
0 – 2	Med. dense Sand with some silt and gravel (Fill)	17	1.96	115	32
2 – 18	Med. dense gravelly Sand, trace silt	24	2.45	130²	34
18 – 23	Dense Sand with some silt, trace gravel	36	3.19	135²	38
23 – 26.5	Med. dense Sand, some silt, trace gravel	29	2.77	130²	36

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 depth of 8 feet. Subtract 62.4 pcf for buoyant unit density.

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Table 5B: Recommended Soil Parameters Based on Boring B-2						
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 – 2	Med. dense Sand with some silt and gravel (Fill)	45	0.57	0.4	21	3.25/0.31
2 – 18	Med. dense gravelly Sand, trace silt	55	0.57	0.4	11[3]	3.54/0.28
18 – 23	Dense Sand with some silt, trace gravel	70	0.57	0.4	21[3]	4.2/0.24
23 – 26.5	Med. dense Sand, some silt, trace gravel	64	0.57	0.4	20[3]	3.85/0.26

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 at 8 feet.
4. Passive resistance in the upper 1.5 feet should be neglected entirely.

Recommended geotechnical input parameters for use in drilled foundation lateral analysis programs are provided in the table below.

Approx. Elevation (ft.) [Approx. Depth (ft.)]	Effective Unit Weight γ' (pcf)	Friction Angle, ϕ (degrees)		Cohesion, C (psf)	Modulus of Horizontal Subgrade Reaction, k (pci)		ε_{50} (%)	p-y Soil Model
		Static	Seismic #		Static	Seismic		
135 – 133 [0 – 2]	115	32	32	0	90	90	0	Sand above groundwater table
133 – 127 [2 – 8]	130	34	34	0	90	90	0	Sand above groundwater table
127 – 120 [8 – 15]	68	34	34	0	60	60	0	Sand below groundwater table
120 – 117 [15 – 18]	68	34	9	0	60	20	0	Sand below groundwater table
117 – 112 [18 – 23]	73	38	38	0	125	125	0	Sand below groundwater table
112 – 108.5 [23 – 26.5]	68	36	11	0	60	20	0	Sand below groundwater table

Table 6: Soil Parameters for L-PILE Analysis Based On ZGA Boring B-2 and CPT-1*

* B-2 and CPT-1 were advance at ground surface elevation of approximately 135 feet
# Values for the Seismic condition include liquefaction effects

The planned yard finished grade elevation of 138 feet was provided to us when this report was prepared. The grade difference between the existing grade and the finished grade will be established by placing granular fill material compacted to at least 95 percent density per ASTM D 1557. Soil parameters for L-PILE analysis for granular fill compacted as described are presented in the table below.

Table 7: Soil Parameters for L-PILE Analysis For Compacted Structural Fill								
Approx. Elevation (ft.) [Approx. Depth (ft.)]*	Effective Unit Weight γ' (pcf)	Friction Angle, ϕ (degrees)		Cohesion, C (psf)	Modulus of Horizontal Subgrade Reaction, k (pci)		ε_{50} (%)	p-y Soil Model
		Static	Seismic #		Static	Seismic		
138 – 135 [0 – 3]*	130	34	34	0	90	90	0	Sand above groundwater table
*Relative to proposed finished grade = elevation 138 feet								

Drilled Shaft End Bearing Capacity

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. We recommend determining nominal unit base resistance via the following equation:

q_{BN} (expressed in tons/ft^2) = 0.60 X N_{60} (less than or equal to 30 tons/ft^2)

where q_{BN} = nominal unit base resistance and N_{60} is the average blowcount value between the pier base and two diameters beneath the base. For example, for a 4-foot diameter pier installed to 10 feet below the planned finished yard grade of 138 feet, a nominal unit base resistance of 16.8 tons/ft^2 may be considered.

Drilled Shaft Uplift Capacity

We recommend incorporating an ultimate uplift capacity due to skin friction between concrete piers and the surrounding soil of 0.38 tons/ft^2. The weight of the piers may be added to the skin friction value.

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 test pits, drilling, and probing processes did not suggest the presence of 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, particularly below groundwater. The contractor should be prepared to install full-depth casing or a sleeve through caving soil zones (temporary casing may be removed following concrete placement). 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 poles 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.

As described previously, the site is characterized by a shallow groundwater condition; previous monitoring by GeoEngineers identified a seasonal high groundwater elevation of about 130-1/2 feet. In the event that there is a need to place concrete in a dry drilled shaft, it is our understanding that some District contractors on other projects have elected to construct the foundation using a sacrificial steel casing, or a corrugated metal pipe (CMP) sleeve, in combination with a concrete plug at the bottom. In order to reduce the risk of destabilizing the granular soil at the bottom of the shaft, the use of slurry is recommended. Minimum levels of slurry in the excavation should be in general accordance with Section 6-19.3(4)B of the 2022 WSDOT Standard Specifications and as pertinent to the project site conditions and the District specifications.

In this case, the shaft would need to be drilled deeper than the design foundation depth such that concrete can be tremied into the base of the casing. The concrete plug installed at the base of the casing or sleeve would need to be thick enough to counteract the buoyant force at the base, and this will be dependent upon the groundwater depth at the time of construction. Once the concrete has cured, it will likely be feasible to pump the casing of water so that concrete can be installed via the free fall method, rather than via tremie through accumulated water, or that a direct-bury pole can be installed in the dry. It should be noted that the concrete plug may shrink during curing, and that some leakage around the plug may occur. Also, it should be noted that a permanent casing or sleeve will need to be cleaned prior to concrete placement.

Concrete Placement: Concrete for drilled piers should normally be placed via the free fall method in dry boreholes. 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 8. 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.

Table 8: IBC Non-constrained Pole Lateral Bearing Pressure	
ZGA Boring	**Recommended Lateral Bearing Pressure ($lbs/ft^2/ft$) of Embedment Depth[1,2,3]**
B-2	1.5 to 2 feet: 150 2 to 18 feet: 185 18 to 23 feet: 225 23 to 26.5 feet: 200

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 estimated at about elevation 130-1/2 feet

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.

Augercast Piles

We understand that augercast piles may be used in lieu of drilled pier foundations in some cases. Our recommendations regarding design and construction of augercast piles follow.

<u>Pile Resistance</u>: This section presents ultimate axial resistances for 24-inch diameter augercast piles. The resistances presented below were determined in general accordance with the methods presented in Geotechnical Engineering Circular No. 8, Design and Construction of Continuous Flight Auger (CFA) Piles (FHWA, 2007).

The ultimate axial compressive resistances provided in the tables below include side friction and end bearing. The capacities provided below assume that the finished grade will be elevation 138 feet, or about 3 feet above the grade at which boring B-2 was advanced. The ultimate axial compressive and uplift resistances ignore the contribution of side resistance in liquefiable soil zones. The foundation loads provided to us are relatively low and the estimated settlements are less than 1/2 inch. The capacities presented below assume a center-to-center spacing of no less than six pile diameters. For a closer spacing, ZGA can provide revised capacities due to group effects. The allowable capacities have a safety factor of 2.5 applied. Please note that the axial compressive capacities presented below do take into account the structural fill that will be added to the site and the resultant pile lengths.

Table 9: Axial Pile Capacities (based on ZGA boring B-2)				
Pile Diameter, (in.)	**Allowable Axial Static Compressive Resistance (kips)**	**Allowable Axial Seismic Compressive Resistance[1] (kips)**	**Allowable Static Uplift Resistance[2] (kips)**	**Allowable Seismic Uplift Resistance[2] (kips)**
Tip Elevation = 115 feet (23 feet long)				
24	124	112	56	41
1. Recommended downdrag loads should be subtracted from these values. 2. Weight of pile should be added to the allowable uplift values.				

We recommend that appropriate load and resistance factors be used in accordance with the applicable industry standard used for this project. The resistance factors used should assume that no field verification (such as load testing) of the recommended resistances will be performed during construction.

Pile Downdrag Loads: Liquefaction settlement during a design seismic event will result in downdrag loads on the piles. Design downdrag loads should be applied to piles in combination with other loads.

Table 10: Downdrag Load	
Pile Diameter (inches)	**Downdrag Load (kips/pile)**
24	18

Lateral Resistance: Lateral loads can be resisted by a combination passive pressure soil resistance acting on embedded portions of the pile caps and lateral resistance of the piles. Recommendations for passive resistance are provided in the Shallow Foundations section of this report. Recommended geotechnical input parameters for use in lateral pile analysis programs are provided in Tables 7 and 8 .

Augercast Pile Construction Considerations: Augercast piles should be installed to the recommended pile tip elevations using a continuous-flight, hollow-stem auger. As is common practice, the pile grout would be pumped under pressure through the hollow stem as the auger is withdrawn.

We recommend that the augercast piles be installed by a contractor experienced in their placement and using suitable equipment. Grout pumps must be fitted with a volume-measuring device and a pressure gauge so that the volume of grout placed in each pile and the pressure head can be easily determined. While grouting, the rate of auger withdrawal must be controlled such that the volume of grout pumped is equivalent to at least 115 to 120 percent of the theoretical drilled hole volume. However, larger grout volumes may occur because the grout may tend to flow out into loose soil zones. A minimum grout line pressure of 100 psi must be maintained while grouting. Also, a minimum head of grout of 8 feet should be maintained above the auger tip at all times as the auger is being retracted from the hole. We recommend that there be a waiting period of at least 24 hours between installation of piles spaced closer than about 10 feet center-to-center in order to avoid disturbance of concrete undergoing curing in a previously cast pile.

Although no apparent obstructions were encountered within the recommended pile depths while advancing borings B-1 and B-2 and CPT-1, below-grade obstructions may be encountered during pile installation. The use of pre-excavation or other techniques may be required to remove obstructions and the contractor should be prepared to use these or other similar procedures where necessary. If pile refusal occurs above the recommended pile tip elevation, the pile should be relocated in accordance with the recommendations of the project structural engineer.

It should be noted that the recommended pile tip elevations and capacities presented above are based on assumed uniformity of soil conditions across the site. There may be unexpected variations in the depth to and characteristics of the supporting soils. In addition, no direct information regarding the capacity of augercast piles (e.g., driving resistance data) is obtained while this type of pile is being installed. Therefore, it is particularly important that the installation of augercast piles be completed under the direct observation of an experienced geotechnical engineer. Accordingly, we recommend that pile installation be monitored by a member of our staff who will observe installation procedures and evaluate the adequacy of individual pile installations. Additionally, we recommend construction specifications similar to those recommended in Geotechnical Engineering Circular No. 8, Design and Construction of Continuous Flight Auger Piles (FHWA 2007) be used for the project.

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 medium dense sand and gravel with a low 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 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 11: Saturated Hydraulic Conductivity Summary		
Exploration / Sample	**Approximate sample depth (feet)**	**Unfactored Saturated Hydraulic Conductivity (inches per hour)**
B-1 / S-5	10	41.3
B-2 / S-3	5	50.6
TP-1 / S-1	2.5	48.5
TP-1 / S-2	4	59.2
TP-1 / S-3	6-1/2	80.3
TP-6 / S-1	1	34.7
TP-6 / S-2	4	58.5
TP-6 / S-3	7-1/2	105.8

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 determined via the grain size method (CF_T). Table 3.3.1 also requires applying correction factors for site variability and number of locations tested (CF_V) and the degree of influent control to prevent siltation and bio-buildup (CF_M). Based upon the site conditions, testing, and our experience with projects of a similar nature, we applied values of 0.5, 0.4, and 0.9 for CF_V, CF_T, and CF_M, respectively. We recommend using a factored rate (K_{sat}) of 17 inches/hour for the *in situ* native outwash sand and gravel 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

Previous groundwater monitoring by GeoEngineers included recording a seasonal high elevation of approximately 130-1/2 feet, or about 5-1/2 feet below existing grade and likely about 6-1/2 feet below substation finished grade. The depth of groundwater is not likely to adversely affect the substation's ability to adequately infiltrate stormwater falling on the site, in our opinion.

Storage Considerations

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.

ZipperGeo
Geoprofessional Consultants

Terracon

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.

We recently received from Iron Mountain Quarry the results of recent permeability testing of their CSBC completed by Krazan & Associates, Inc. The test results, which are included in Appendix B, were conducted on full samples of the CSBC, i.e., the plus 3/4-inch fraction was not removed prior to testing. Consistent with our conclusion pointed out above, the tests indicated permeability rates for two samples of 168.5 to 170.5 inches/hour when compacted to 98 percent of the modified Proctor maximum dry density.

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.

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 provided that fill with a high void ratio and permeability are required.

Table 12: 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 provided for our review indicate that the substation footprint encompasses slightly less than one acre. It appears that groundwater mounding analysis is not necessary per the *Manual* given the documented groundwater depth relative to the anticipated site improvements.

Driveway Flexible Pavement Section Recommendations

In the event that the substation is provided with an asphalt-paved driveway, we recommend considering the criteria described below. 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.

Deluge UIC Well Decommissioning

We understand that planned site improvements will render the existing deluge system UIC well unnecessary and that the District may decommission it. Toward that end, we offer the following:

- Per Washington Administrative Code (WAC) 173-218-040 *UIC well classification including allowed and prohibited wells*, the well meets the criteria a *Class V injection well*, a type that commonly includes drywells installed for stormwater management purposes [WAC 173-218-040(5) and WAC 173-218-040(5)(a)(i)].

- Per WAC 173-218-120 *Decommissioning a UIC well* Section (3)(b), *Class V wells must be decommissioned by filling or plugging the well so that it will not result in an environmental, public health, or safety hazard, and will not serve as a channel for movement of water or pollution to an aquifer*.

- In addition, per Section 3(b0(i), *UIC wells that are in contact with an aquifer, even if they are in contact with only the seasonal high aquifer, must be decommissioned in accordance with the most applicable method found in Chapter 173-160 WAC Minimum standards for construction and maintenance of wells.*

- The drywell meets the criteria for a "dug well" per *WAC 173-160-381 What are the standards for decommissioning a well?* Section (2)(3), in our opinion.

- Per *WAC 173-160-381 What are the standards for decommissioning a well? any well which is unusable, abandoned, <u>or whose use has been permanently discontinued</u>* (emphasis added by ZGA), *or which is in such disrepair that its continued use is impractical or is an environmental, safety or public health hazard shall be decommissioned. The decommissioning procedure (as prescribed by these regulations) must be recorded and reported as required by the department.*

- *(3) Dug wells -*

  (a) The following criteria are required for the decommissioning of all dug wells:

  (i) Remove all debris, accumulated sediments, and obstructions that impede decommissioning or that may contaminate the aquifer from within the dug well.

  (ii) Dug wells may have a maximum of three feet of soil cover from top of sealing material to land surface.

  (iii) Dug wells shall be sealed with either unhydrated bentonite, neat cement, neat cement grout, or concrete. <u>The use of controlled density fill (CDF), bentonite slurry, or fly ash is prohibited</u> (emphasis added by ZGA).

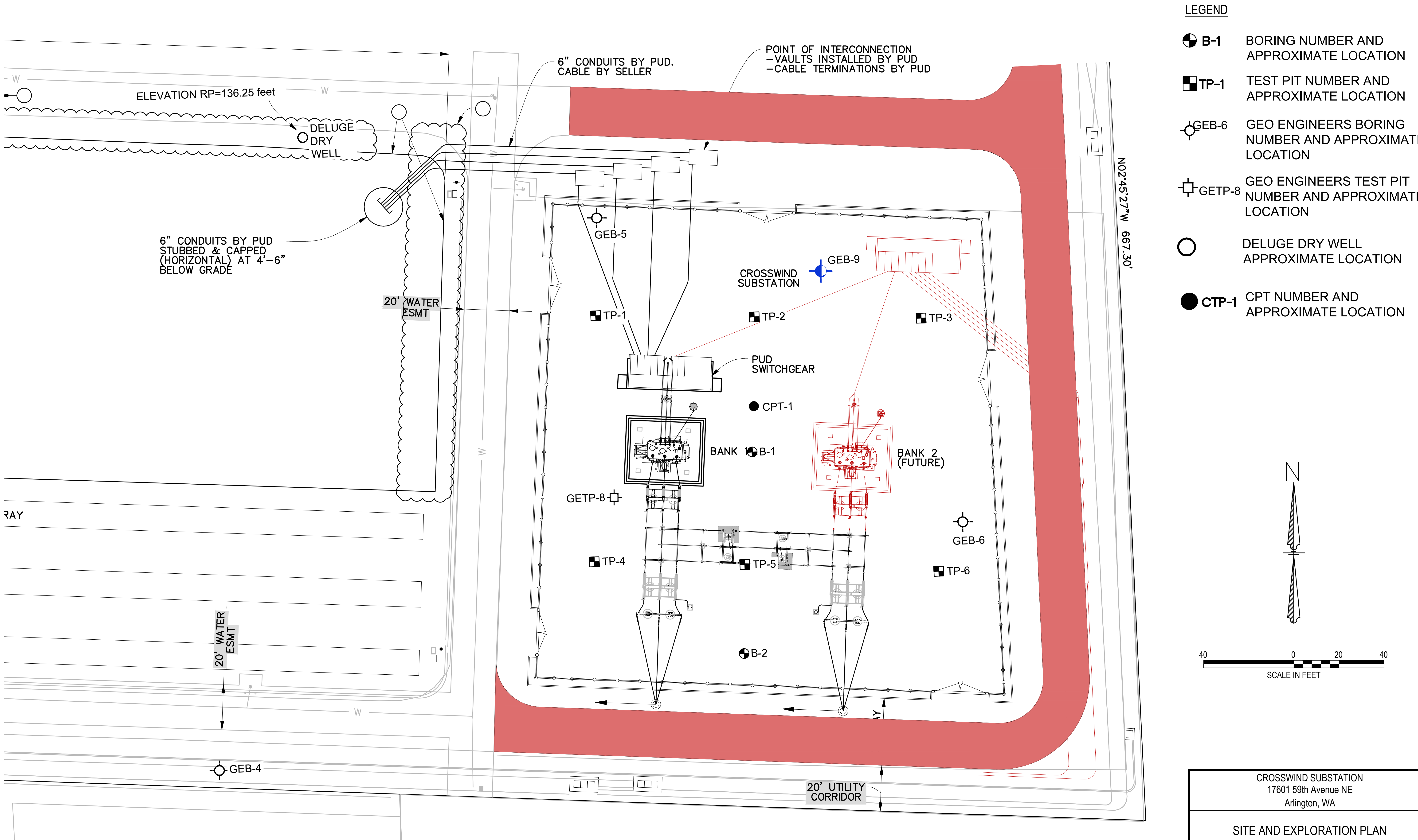
Please note that until such time as the drywell is permanently abandoned, the District is not required to decommission it.

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.



CROSSWIND SUBSTATION
17601 59th Avenue NE
Arlington, WA

SITE AND EXPLORATION PLAN

4/15/2023	Job No.	2679.01
Zipper Geo Associates, LLC 19019 36th Ave. W.,Suite E Lynnwood, WA, 98036	FIGURE SHT. 1 of 1	1

REFERENCE: SNOHOMISH COUNTY PUBLIC UTILITIES DISTRICT SITE PLAN DATED JANUARY 26, 2023

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 two borings (B-1 and B-2), advancing one cone penetrometer test (CPT-1), and excavating six test pits (TP-1 through TP-6). The approximate exploration locations are presented on Figure 1, the *Site and Exploration Plan*. Exploration locations were determined in the field using steel and fiberglass tapes by measuring distances from existing site features shown on the *Central Arlington Rebuild Concept A* plan, dated 26 August 2021, provided by the District. The approximate ground surface elevation at the exploration locations was interpolated from contours shown on Sheet SV1.08, *North County Community Office*, dated 22 March 2022. 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 trailer-mounted drill rig operated by an independent drilling company (Geologic Drilling Partners) 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.

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.

Exploration Logs by Others

The 29 December 2017 GeoEngineers report *Geotechnical Engineering Services, North County Project, Arlington, Washington* (File No. 0482-051-03) includes the logs of numerous explorations completed at the Microgrid site. This appendix includes the logs of one test pit and four borings that GeoEngineers completed in or very near the proposed substation location, the approximate locations of which are illustrated on Figure 1.

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Geologic Drill	Bore Hole Dia.:	6 inch
Top Elevation:	136 Feet	Drilling Method:	HSA	Hammer Type:	Cathead
Date Drilled:	2.28.2023	Drill Rig:	Trailer rig	Logged by:	JLB

B-1

SOIL DESCRIPTION

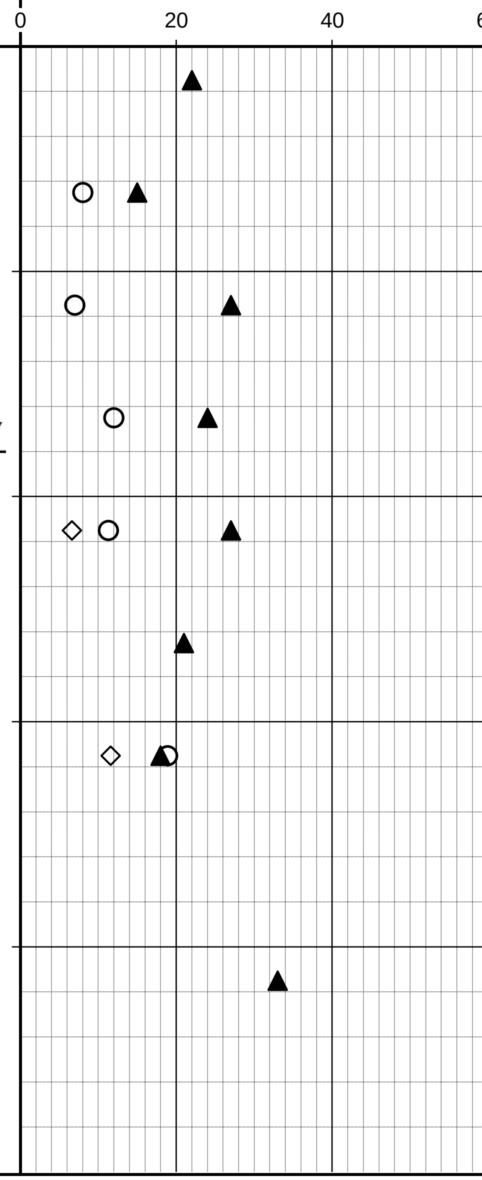
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	Soil Description	Sample Number	Recovery	Groundwater	Blow counts	Testing
0	6 inches of 3/4-inch crushed gravel over geotextile fabric. (FILL) (ESU 1)	S-1	4		22	ACM
	Medium dense, moist, brown, SAND trace silt; fine sand (Qvrm) (ESU 2)	S-2	18		15	
5	Medium dense, moist, brown, SAND with gravel, some silt; coarse to fine subrounded gravel. (Qvrm) (ESU 2)	S-3	18		27	
	Medium dense, wet to saturated, grey-brown to grey, SAND and gravelly SAND with some silt; coarse to fine subrounded gravel; saturated at 9 feet. (Qvrm) (ESU 2)	S-4	18	2.28.2023	24	
10		S-5	18		27	GSA
		S-6	18		21	
15		S-7	18		18	GSA
	Dense, saturated, gray, SAND trace silt; medium to fine sand. (Qvrm) (ESU 3)					
20		S-8	18		33	
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

Crosswind Substation

17601 59th Avenue NE

Arlington, WA

Date: 3.15.2023 Project No.: 2679.01

Zipper Geo Associates

19019 36th Ave. W, Suite E

Lynnwood, WA

BORING LOG: B-1

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Geologic Drill	Bore Hole Dia.:	6 inch	**B-1**
Top Elevation:	136 Feet	Drilling Method:	HSA	Hammer Type:	Cathead	
Date Drilled:	2.28.2023	Drill Rig:	Trailer rig	Logged by:	JLB	

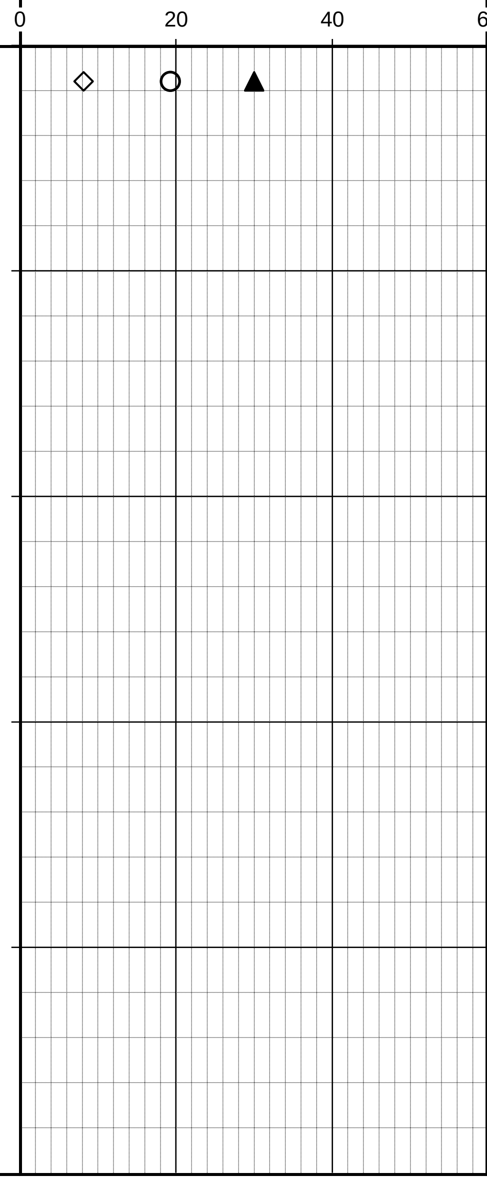
SOIL DESCRIPTION

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	Groundwater	Blow counts	Testing
25	Medium dense, saturated, grey, SAND trace gravel, some silt. (Qvrm) (ESU 2)	S-9	18		30	GSA
	Borehole completed at 26.5 feet below ground surface (bgs). Groundwater observed at 9 feet bgs at time of drilling. Drilling mud added at 10 feet due to heaving conditions.					
30						
35						
40						
45						
50						

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

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Crosswind Substation
17601 59th Avenue NE
Arlington, WA

Date: 3.15.2023 Project No.: 2679.01

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

BORING LOG: B-1

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Geologic Drill	Bore Hole Dia.:	6 inch
Top Elevation:	135 Feet	Drilling Method:	HSA	Hammer Type:	Cathead
Date Drilled:	2.28.2023	Drill Rig:	Trailer rig	Logged by:	JLB

B-2

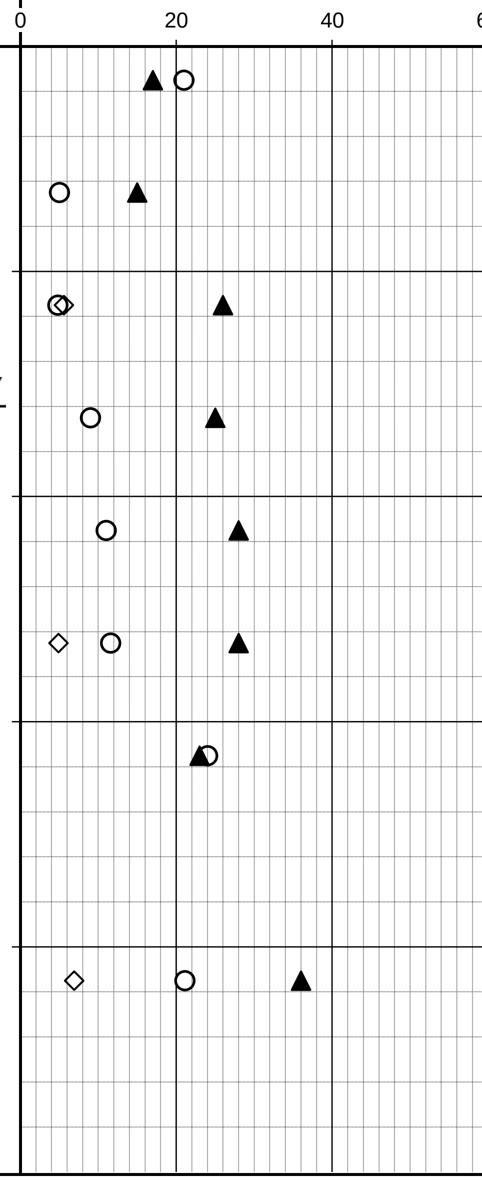
SOIL DESCRIPTION

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

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



Depth (ft)	Soil Description	Sample Number	Recovery	Groundwater	Blow counts	Testing
0	6 inches of 3/4 inch crushed gravel over geotextile fabric. (FILL) (ESU 1)	S-1	15		17	ACM
	Medium dense, moist to saturated, brown, gravelly SAND, trace silt; fine subrounded gravel, saturated at 8 feet. (Qvrm) (ESU 2)	S-2	18		15	
5		S-3	18		26	GSA
		S-4	18	▼ 2.28.2023	25	
10		S-5	18		28	
		S-6	18		28	GSA
15	Medium dense to dense, saturated, grey-brown, SAND trace gravel, some silt; medium to fine sand. (Qvrm) (ESU 2)	S-7	18		23	
	Dense, saturated, grey-brown SAND with trace gravel and some silt (Qvrm) (ESU 3)					
20		S-8	18		36	GSA
	Medium dense to dense, saturated, grey-brown, SAND trace gravel, some silt; medium to fine sand. (Qvrm) (ESU 2)					
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
- ▼ 11/2/12 Groundwater level at time of drilling (ATD) or on date of measurement.

◇ % 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

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Crosswind Substation
17601 59th Avenue NE
Arlington, WA

Date: 3.15.2023 Project No.: 2679.01

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

BORING LOG: **B-2**

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Geologic Drill	Bore Hole Dia.:	6 inch
Top Elevation:	135 Feet	Drilling Method:	HSA	Hammer Type:	Cathead
Date Drilled:	2.28.2023	Drill Rig:	Trailer rig	Logged by:	JLB

B-2

Depth (ft)	SOIL DESCRIPTION	SAMPLES: Sample Number	Recovery	Groundwater	PENETRATION RESISTANCE (blows/foot)	Blow counts	Testing
	The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.				▲ Standard Penetration Test; △ Hammer Weight and Drop: ___ (0, 20, 40, 60)		
25	Medium dense to dense, saturated, grey-brown, SAND trace gravel, some silt; medium to fine sand. (Qvrm) (ESU 2)	S-9	18			29	
	Borehole completed at 26.5 feet below ground surface (bgs). Groundwater observed at 8 feet bgs at time of drilling. Drilling mud added at 15 feet bgs due to heaving conditions.						
30							
35							
40							
45							
50							

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

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Consol. = Consolidation Test

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Crosswind Substation
17601 59th Avenue NE
Arlington, WA

Date: 3.15.2023 Project No.: 2679.01

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

BORING LOG: B-2

ZipperGeo
Geoprofessional Consultants

	Test Pit TP-1 **Location:** See Attached Site and Exploration Plan in Figure 1 **Approximate Ground Surface Elevation:** 136 feet	**Project:** Crosswind Substation **Project Number:** 2679.01 **Date Excavated:** 2.22.2023			
Depth (ft)	**Material Description**	**Sample**	**PID**	**% M**	**Testing**
1	About 4 inches of 3/4 inch crushed GRAVEL over non-woven geotextile over loose, moist, brown, SAND with woody debris/logs max 16 inches long and 4 inches diameter. (FILL)	S-ACM			ACM
2					
3	Loose to medium dense, moist, yellow-brown, SAND trace gravel, trace silt; medium to fine sand. (Qvrm)	S-1 @ 2-1/2 feet	0.1	9	GSA
4					
5	Loose to medium dense, moist, yellow-brown, gravelly SAND trace silt; subrounded gravel. (Qvrm)	S-2 @ 4 feet	0.1	6	GSA
6					
7	Loose to medium dense, moist, grey, sandy GRAVEL trace silt; subrounded gravel. (Qvrm)	S-3 @ 6-1/2 feet	0.2	4	GSA
8					
9	Loose to medium dense, grey, wet, GRAVEL with cobbles, with sand; subrounded cobbles, subrounded gravel. (Qvrm)	S-4 @ 8 feet	0.1		
10					
11	Exploration completed at 8-1/2 feet on 2.22.2023 Moderate groundwater seepage observed at approximately 8 feet at time of excavation				
12	Slight to moderate caving observed from 7-8 feet				
13					
14					

Note: PID is the displayed hydrocarbon concentration in parts per million (ppm).

ZipperGeo
Geoprofessional Consultants

	Test Pit TP-2 **Location:** See Attached Site and Exploration Plan in Figure 1 **Approximate Ground Surface Elevation:** 136 feet	**Project:** Crosswind Substation **Project Number:** 2679.01 **Date Excavated:** 2.22.2023			
Depth (ft)	**Material Description**	**Sample**	**PID**	**% M**	**Testing**
1	About 4 inches of 3/4 -inch crushed GRAVEL over non-woven geotextile over loose, moist, brown, SAND some organics/woody debris. (FILL)	S-ACM			ACM
2	Loose to medium dense, moist, yellow-brown, SAND some silt; medium to fine sand. (Qvrm)	S-1 @ 1-1/2 feet	0.1	5	
3					
4					
5					
6	Loose to medium dense, moist, yellow-brown, SAND with gravel; subrounded gravel. (Qvrm)	S-2 @ 5 feet	0.1	5	
7					
8	Loose to medium dense, wet, grey-brown, SAND with cobbles, with gravel; subrounded cobbles, subrounded gravel. (Qvrm)	S-3 @ 7 feet	0.1	8	
9					
10	Exploration completed at 7-1/2 feet on 2.22.2023 Moderate groundwater seepage observed at approximately 7-1/2 feet at time of excavation				
11	Slight caving observed from approximately 7 feet				
12					
13					
14					

Note: PID is the displayed hydrocarbon concentration in parts per million (ppm).

Test Pit TP-3

Location: See Attached Site and Exploration Plan in Figure 1

Approximate Ground Surface Elevation: 136 feet

Project: Crosswind Substation

Project Number: 2679.01

Date Excavated: 2.22.2023

Depth (ft)	Material Description	Sample	PID	% M	Testing
1	About 3 inches of 3/4 inch crushed GRAVEL over non-woven geotextile over loose, moist, brown, SAND with silt, some organics/woody debris. (FILL)	S-ACM			ACM
2	Loose to medium dense, moist, red-brown, SAND trace silt; medium to fine sand. (Qvrm)	S-1 @ 1-1/2 feet	0.2	5	
3					
4	Loose to medium dense, moist, yellow-brown, SAND with cobbles, with gravel; subrounded cobbles, subrounded gravel. (Qvrm)	S-2 @ 3 feet	0.1	4	
5					
6					
7	Loose to medium dense, wet, grey, GRAVEL with sand; subrounded gravel. (Qvrm)	S-3 @ 6-1/2 feet	0.1	6	
8	Exploration completed at 7-1/2 feet on 2.22.2023				
9	Moderate groundwater seepage observed at approximately 7 feet at time of excavation				
10	No caving observed.				
11					
12					
13					
14					

Note: PID is the displayed hydrocarbon concentration in parts per million (ppm).

ZipperGeo
Geoprofessional Consultants

Test Pit TP-4

Location: See Attached Site and Exploration Plan in Figure 1

Approximate Ground Surface Elevation: 135 feet

Project: Crosswind Substation

Project Number: 2679.01

Date Excavated: 2.22.2023

Depth (ft)	Material Description	Sample	PID	% M	Testing
1	About 3 inches of 3/4-inch crushed GRAVEL over non-woven geotextile over loose, moist, red-brown, SAND with silt, some organics/woody debris, with FE. (FILL)	S-1 @ 1 feet	0.2		ACM
2					
3	Loose to medium dense, moist, brown, SAND trace to with gravel, trace silt; with subrounded cobbles at 6 feet; subrounded gravel. (Qvrm)	S-2 @ 2-1/2 feet	0.1		
4					
5		S-3 @ 4-1/2 feet	0.1		
6					
7	Loose to medium dense, wet, grey, SAND with gravel; subrounded gravel, coarse to fine sand. (Qvrm)	S-4 @ 6 feet	0.1		
8		S-5 @ 7 feet	0.1		
9	Exploration completed at 8 feet on 2.22.2023 Moderate groundwater seepage observed at approximately 7 feet at time of excavation				
10	Moderate caving observed at approximately 7 feet.				
11					
12					
13					
14					

Note: PID is the displayed hydrocarbon concentration in parts per million (ppm).

ZipperGeo
Geoprofessional Consultants

	Test Pit TP-5				**Project:** Crosswind Substation
	Location: See Attached Site and Exploration Plan in Figure 1				**Project Number:** 2679.01
	Approximate Ground Surface Elevation: 136 feet				**Date Excavated:** 2.22.2023
Depth (ft)	**Material Description**	**Sample**	**PID**	**% M**	**Testing**
1	4 inches of 3/4-inch crushed GRAVEL over non-woven geotextile over loose to medium dense, moist, red-brown, SAND trace to some wood debris, with Fe. (FILL)	S-ACM			ACM
2		S-1 @ 1-1/2 feet	0.2	7	
3	Loose to medium dense, moist, yellow-brown, SAND some gravel, trace silt; subrounded gravel. (Qvrm)				
4		S-2 @ 3 feet	0.1	4	
5					
6	Loose to medium dense, moist, grey-brown, GRAVEL with sand, with cobbles at 7 feet; subrounded cobbles; subrounded gravel. (Qvrm)	S-3 @ 5 feet	0.1	2	
7					
8		S-4 @ 7 feet	0.1	9	
9	Exploration completed at 8 feet on 2.22.2023 Moderate seepage observed at approximately 7-1/2 feet at time of excavation Slight to moderate caving observed from approximately 7-8 feet.				
10					
11					
12					
13					
14					

Note: PID is the displayed hydrocarbon concentration in parts per million (ppm).

ZipperGeo
Geoprofessional Consultants

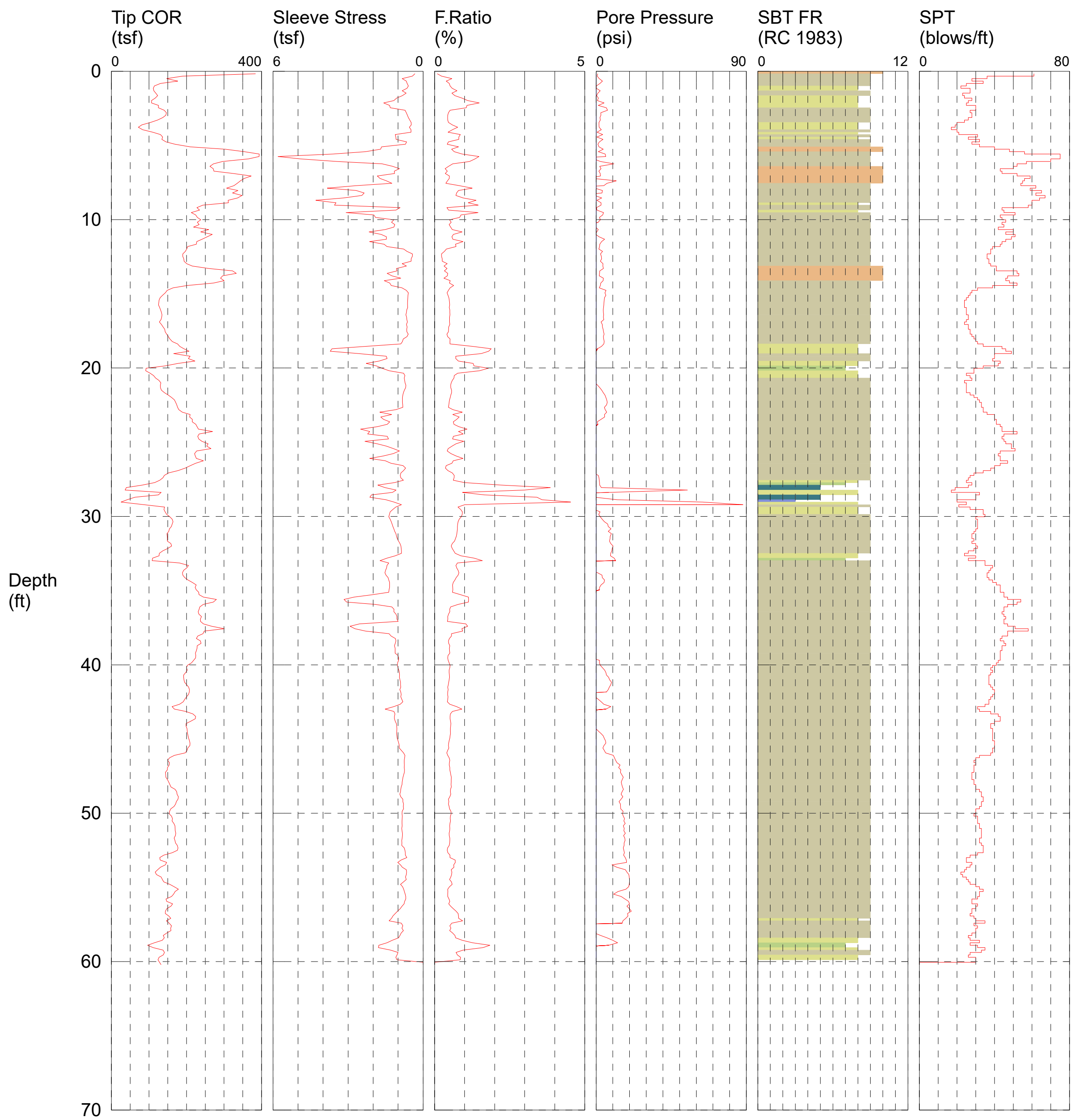
	Test Pit TP-6				**Project:** Crosswind Substation
	Location: See Attached Site and Exploration Plan in Figure 1				**Project Number:** 2679.01
	Approximate Ground Surface Elevation: 136 feet				**Date Excavated:** 2.22.2023
Depth (ft)	**Material Description**	**Sample**	**PID**	**% M**	**Testing**
1	3-4 inches of 3/4-inch crushed GRAVEL over non-woven geotextile over loose to medium dense, moist, red-brown, SAND trace gravel, some silt, trace to some wood debris, with Fe. (FILL)	S-1 @ 1 feet	0.1	12	GSA ACM
2					
3					
4					
5	Loose to medium dense, moist, yellow-brown, gravelly SAND, trace silt; subrounded gravel. (Qvrm)	S-2 @ 4 feet	0.1	6	GSA
6					
7					
8	Loose to medium dense, grey-brown, moist, gravelly SAND with cobbles, trace silt; subrounded cobbles, subrounded gravel. (Qvrm)	S-3 @ 7-1/2 feet	0.1	10	GSA
9					
10	Exploration completed at approximately 8 feet.				
11	Moderate groundwater seepage observed at approximately 7-1/2 feet at time of excavation				
12	Slight caving observed from approximately 7-8 feet.				
13					
14					

Note: PID is the displayed hydrocarbon concentration in parts per million (ppm).

CPT-01

CPT Contractor: In Situ Engineering
CUSTOMER: ZipperGeo
LOCATION: Arlington
JOB NUMBER: 2679.01

OPERATOR: Forinash
CONE ID: DDG1351
TEST DATE: 2/24/2023 9:46:02 AM
Coring: 0ft
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: None



TOTAL DEPTH: 60.203 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

GeoEngineers Exploration Logs

Drilled	Start: 6/14/2012	End: 6/14/2012	Total Depth (ft)	26.5	Logged By: SMJ Checked By: BPD	Driller	Geologic Drill Explorations, Inc.	Drilling Method	Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Track Rig
Surface Elevation (ft) Vertical Datum	134.5 NAVD88	Top of Casing Elevation (ft)	
Easting (X) Northing (Y)		Horizontal Datum	

DOE Well I.D.: BHJ196
A 2 (in) well was installed on to a depth of 20 (ft).

Groundwater

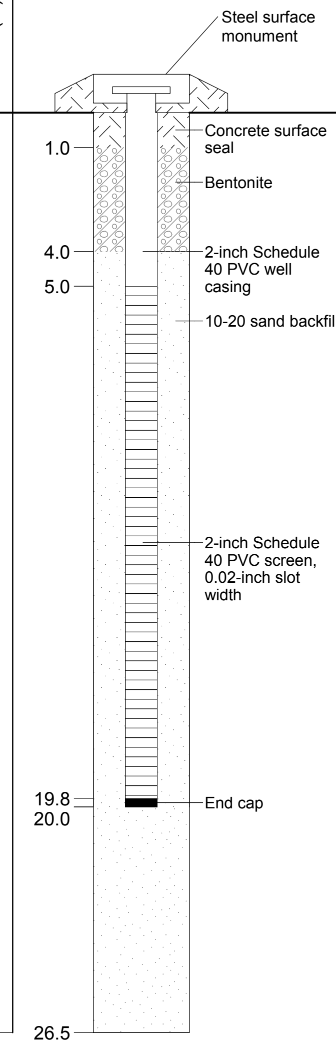
Date Measured	Depth to Water (ft)	Elevation (ft)
6/25/2012	5.7	128.8

Notes: Elevation estimated from base survey map

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0								TS	9 inches topsoil and roots		
									SM	Light brown silty fine to medium sand with gravel, trace organic matter (loose, moist)		
		▮	18	8		1 MC					11	
130									SP-SM	Light brown fine to medium sand with silt and gravel (medium dense, moist) (recessional outwash)		
	5	▮	18	22		2 MC	▼				6	
		▮	12	22		3						
										Becomes wet		
125									SP	Grayish brown fine to medium sand with gravel (medium dense, wet)		
	10	▮	18	23		4						
									SP-SM	Grayish brown fine to medium sand with silt and occasional gravel (medium dense, wet) Driller added drilling mud at 13 feet		
120												
	15	▮	18	21		5						
									SM	Grayish brown silty fine to medium sand (medium dense to dense, wet)		
115												
	20	▮	18	30		6 MC				1 foot heave at 20 feet	12	
110												
	25	▮	18	12		7						

WELL LOG



Notes: See Figure A-1 for explanation of symbols.
Elevation estimated from base survey map

Log of Boring B-4

GEOENGINEERS

Project: Arlington Site Development
Project Location: Arlington, Washington
Project Number: 0482-051-03

Drilled	Start 6/13/2012	End 6/13/2012	Total Depth (ft) 26.5	Logged By SMJ / Checked By BPD	Driller Geologic Drill Explorations, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 136.5 Vertical Datum NAVD88	Hammer Data Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment Diedrich D-50 Track Rig
Easting (X) Northing (Y)	System Datum	Groundwater Date Measured / Depth to Water (ft) / Elevation (ft) See Remarks
Notes: Elevation estimated from base survey map		

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0								TS	6 inches topsoil and roots			Recently mowed weeds and blackberries
135									SP-SM	Grayish brown fine to medium sand with silt, trace roots (loose, moist) (recessional outwash)			
			14	9		1 / MC					6		
	5		18	15		2 / SA				Becomes medium dense with gravel	7	8	
130			12	16		3				Becomes wet			Groundwater seepage observed at 8 feet
	10		18	23		4							Driller added drilling mud at 10 feet
125									SW-SM	Grayish brown fine to coarse sand with silt and gravel (medium dense, wet)			
	15		10	27		5							
120									SM	Grayish brown silty fine to medium sand (medium dense, wet)			
	20		12	12		6							
115													
	25		12	22		7							
110													

Notes: See Figure A-1 for explanation of symbols.
Elevation estimated from base survey map

Log of Boring B-5

GeoEngineers

Project: Arlington Site Development
Project Location: Arlington, Washington
Project Number: 0482-051-03

Figure A-6
Sheet 1 of 1

Drilled	Start 6/13/2012	End 6/13/2012	Total Depth (ft)	26.5	Logged By SMJ / Checked By BPD	Driller	Geologic Drill Explorations, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	136.5 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Track Rig
Easting (X) / Northing (Y)		System / Datum		Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	See Remarks
Notes: Elevation estimated from base survey map					

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0								TS	6 inches topsoil and roots			Recently mowed weeds and blackberries
135									SM	Light brown silty fine to medium sand with occasional gravel, organic matter (medium dense, moist) (recessional outwash)			
			12	14		1					20		
									SP-SM	Light brown fine to medium sand with silt and occasional gravel (medium dense, moist) (recessional outwash)			
	5		12	24		2					7		
130													
			18	30		3				Becomes medium dense to dense and wet			Groundwater seepage observed at 7½ feet
										2 inch silt lense			
									SW-SM	Grayish brown fine to coarse sand with gravel (medium dense to dense, wet)			
	10		17	30		4							
125													
									SM	Grayish brown silty fine to medium sand with occasional gravel (medium dense, wet)			
	15		18	28		5							
120													
	20		18	25		6							
115													
	25		18	24		7			SM	Brown silty fine to medium sand (medium dense, wet)			3 feet heave at 25 feet
110													

Notes: See Figure A-1 for explanation of symbols.
Elevation estimated from base survey map

Log of Boring B-6

GeoEngineers

Project: Arlington Site Development
Project Location: Arlington, Washington
Project Number: 0482-051-03

Drilled	Start 3/8/2017	End 3/8/2017	Total Depth (ft)	26.5	Logged By NS, Checked By KMS	Driller	Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track-mounted
Surface Elevation (ft) / Vertical Datum	137 / NAVD88	Top of Casing Elevation (ft)	137
Easting (X) / Northing (Y)	1320413 / 424926	Horizontal Datum	WA State Plane North NAD83 (feet)

DOE Well I.D.: BJY257
A 2 (in) well was installed on 3/8/2017 to a depth of 20 (ft).

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
4/13/2017	7.4	129.6

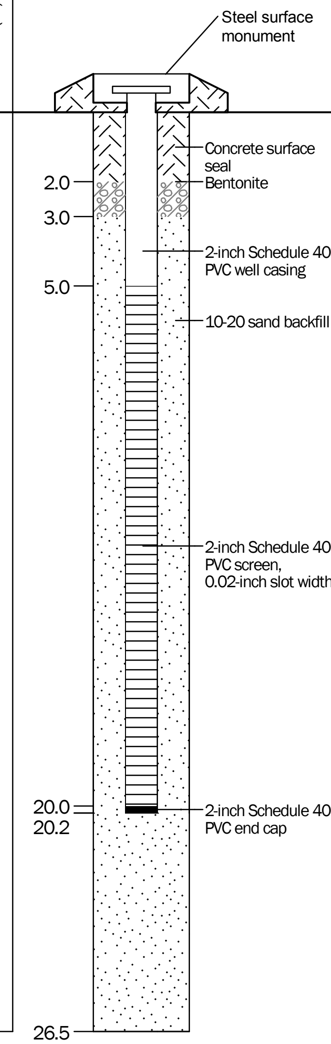
Notes: Environmental field screening was completed on each soil sample. Sheen and head space vapor were not observed unless otherwise noted.

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
137	0					TS	Topsoil with roots (upper 6 inches)		
		16	13	1 / MC		SM	Reddish brown silty fine to medium sand with occasional organic matter (medium dense, moist)	7	
	5	4	62*	2		SP-SM	Light brown fine to medium sand with silt and occasional gravel and cobbles (medium dense to dense, moist) (recessional outwash)		
		8	36*	3 / SA	▼	SP-SM	Light brown fine to coarse sand with silt, gravel and occaional cobbles (medium dense, wet)	7	9
	10	10	23	4					
		9	23	5 / %F		SP	Light gray fine to coarse sand with gravel (medium dense, wet)	15	4
	15	18	21	6			Becomes gray		
						SP-SM	Gray fine to medium sand with silt and occasional gravel (medium dense, wet)		
	20	18	19	7 / %F		SP	Gray fine to medium sand (medium dense, wet)	20	4
	25	18	18	8					

Elevation scale: 135, 130, 125, 120, 115

* Sampler bouncing on rock, blow count not representative.

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Hand-held GPS (±18 ft), Vertical approximated based on Survey Basemap (±1 ft)

Log of Boring B-8

GEOENGINEERS

Project: Arlington Local Office Replacement
Project Location: Arlington, Washington
Project Number: 0482-051-03

Drilled	Start 3/8/2017	End 3/8/2017	Total Depth (ft)	26.5	Logged By NS / Checked By KMS	Driller	Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track-mounted	DOE Well I.D.: BJY258 A 2 (in) well was installed on 3/8/2017 to a depth of 20 (ft).
Surface Elevation (ft) / Vertical Datum	136 / NAVD88	Top of Casing Elevation (ft)	137	
Easting (X) / Northing (Y)	1321085 / 424465	Horizontal Datum	WA State Plane North NAD83 (feet)	

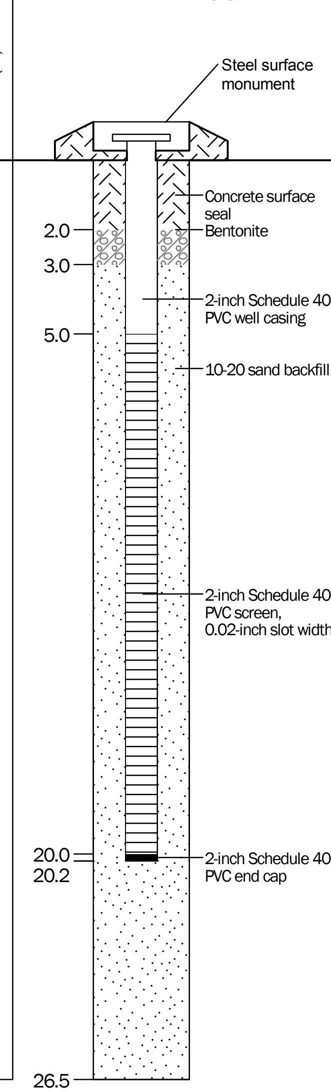
Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
4/13/2017	2.7	133.3

Notes: Environmental field screening was completed on each soil sample. Sheen and head space vapor were not observed unless otherwise noted.

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
135	0				TS	Topsoil with roots (upper 6 inches)		
					SP-SM	Brown fine to coarse sand with silt and occasional gravel (loose, wet) (recessional outwash)		
		10	8	1 / %F			14	5
130	5	12	12	2 / SA	SP	Brown fine to coarse sand with gravel (medium dense, wet)	20	2
		14	20	3				
125	10	12	26	4 / %F	SP-SM	Brown fine to medium sand with silt and occasional gravel (medium dense, wet)	15	6
		8	29	5				
120	15	18	28	6				
115	20	18	40	7 / %F		Becomes dense	14	7
110	25	18	24	8	SP-SM	Brown fine sand with silt (medium dense, wet)		

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Hand-held GPS (±18 ft), Vertical approximated based on Survey Basemap (±1 ft)

Bellingham: Date:4/24/17 Path:P:\0\0482051\GINT\048205103.GPJ DBTemplate/LibTemplate:GEOENGINEERS_DF_STD_US_APRIL_2017.GDT/GEI8_GEOTECH_WELL_%F

Log of Boring B-9

GEOENGINEERS

Project: Arlington Local Office Replacement
Project Location: Arlington, Washington
Project Number: 0482-051-03

Figure A-4
Sheet 1 of 1

Date Excavated: 6/15/2012 Logged By: SMJ

Equipment: Rubber Tire Backhoe Total Depth (ft) 12.0

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0	X	1		TS		8-inches topsoil and roots		Recently mowed weeds and blackberries Probe(P) = 12 inches
135	1				SM		Reddish brown silty fine to medium sand with organic matter (loose to medium dense, moist)		P = 8 inches
134	2								P = 8 inches
133	3	X	2 SA					17	P = 4 inches SA (%F = 27)
132	4								
131	5								
130	6	X	3 SA		SP		Light brown fine to medium sand with gravel (medium dense, moist) (recessional outwash)	5	P = 2 inches SA (%F = 3)
129	7								
128	8								
127	9	X	4						
126	10						Grades to grayish brown and becomes wet		
125	11				SP-SM		Grayish brown fine to medium sand with gravel and occasional cobbles (medium dense, wet)		
124	12	X	5						

Moderate groundwater seepage observed at 10 feet.
Moderate caving observed at 10 feet.

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.
Elevation estimated from base survey map

Log of Test Pit TP-8

GEOENGINEERS

Project: Arlington Site Development
Project Location: Arlington, Washington
Project Number: 0482-051-03

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.

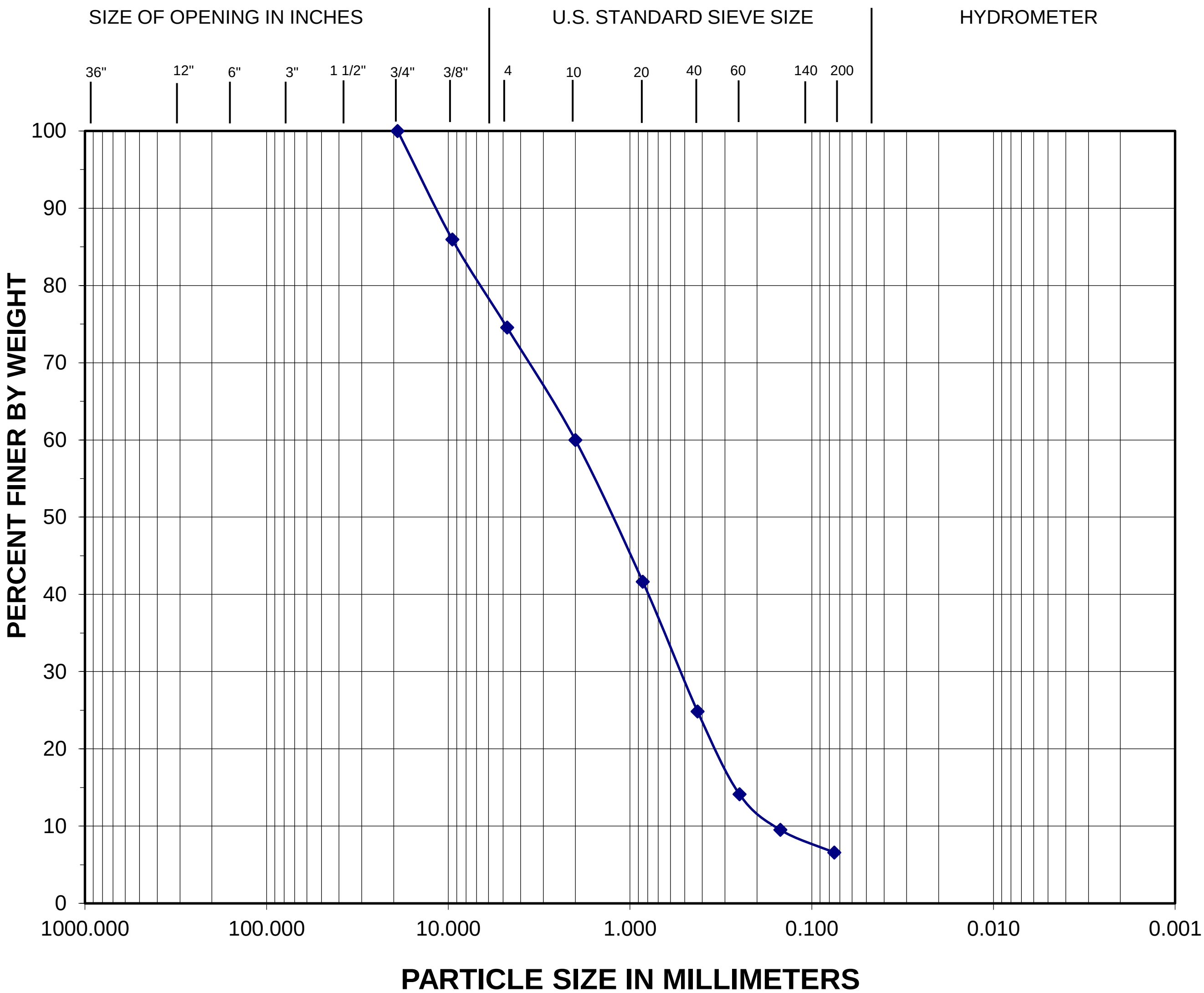
Asbestos Containing Material (ACM)

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:

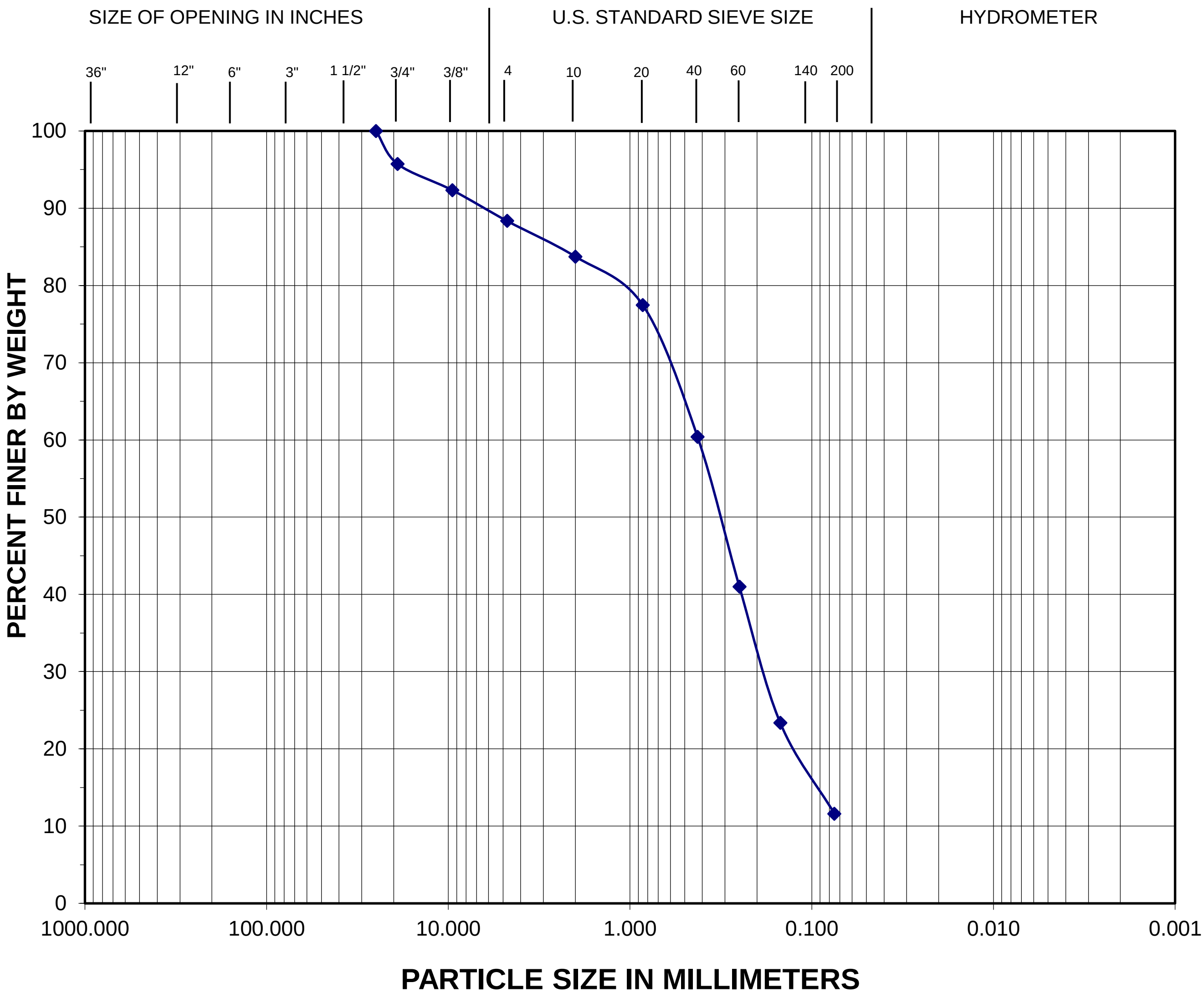
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-5	10	11.3	6.6	Gravelly SAND, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

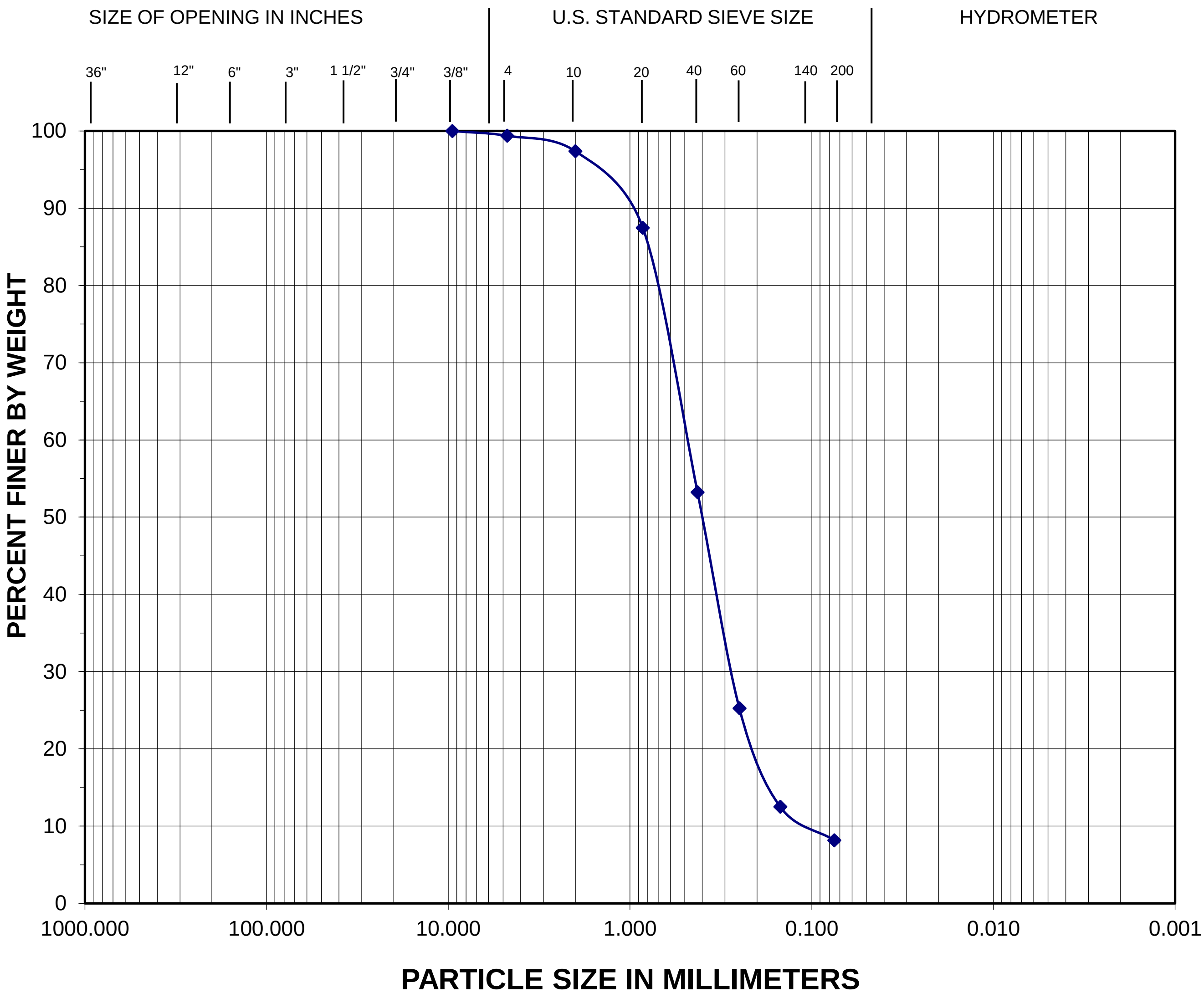
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-7	15	18.9	11.6	SAND, some gravel and silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

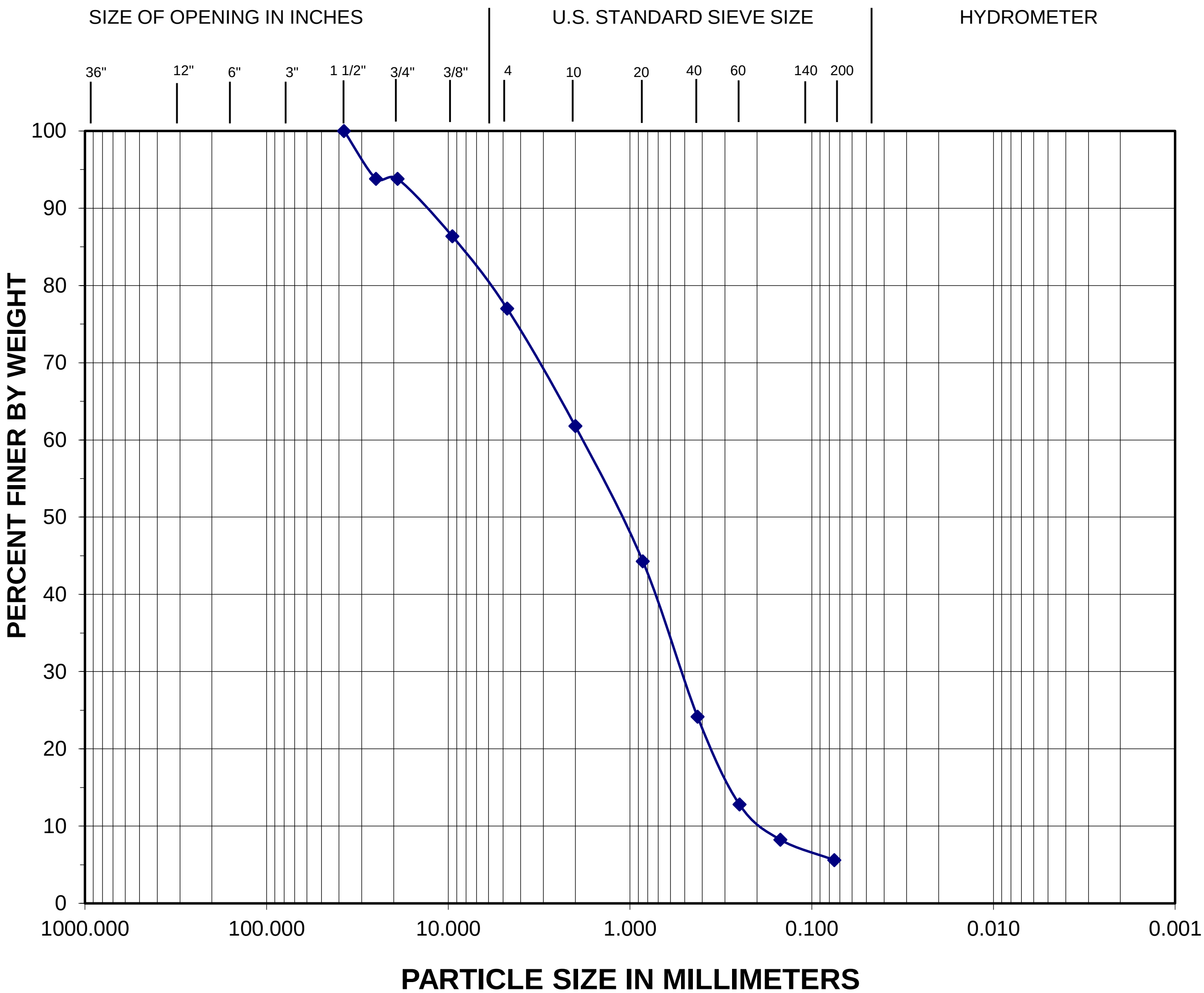
Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-9	25	19.3	8.2	SAND, some silt, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS
Test Results Summary

ASTM D6913



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

Comments:

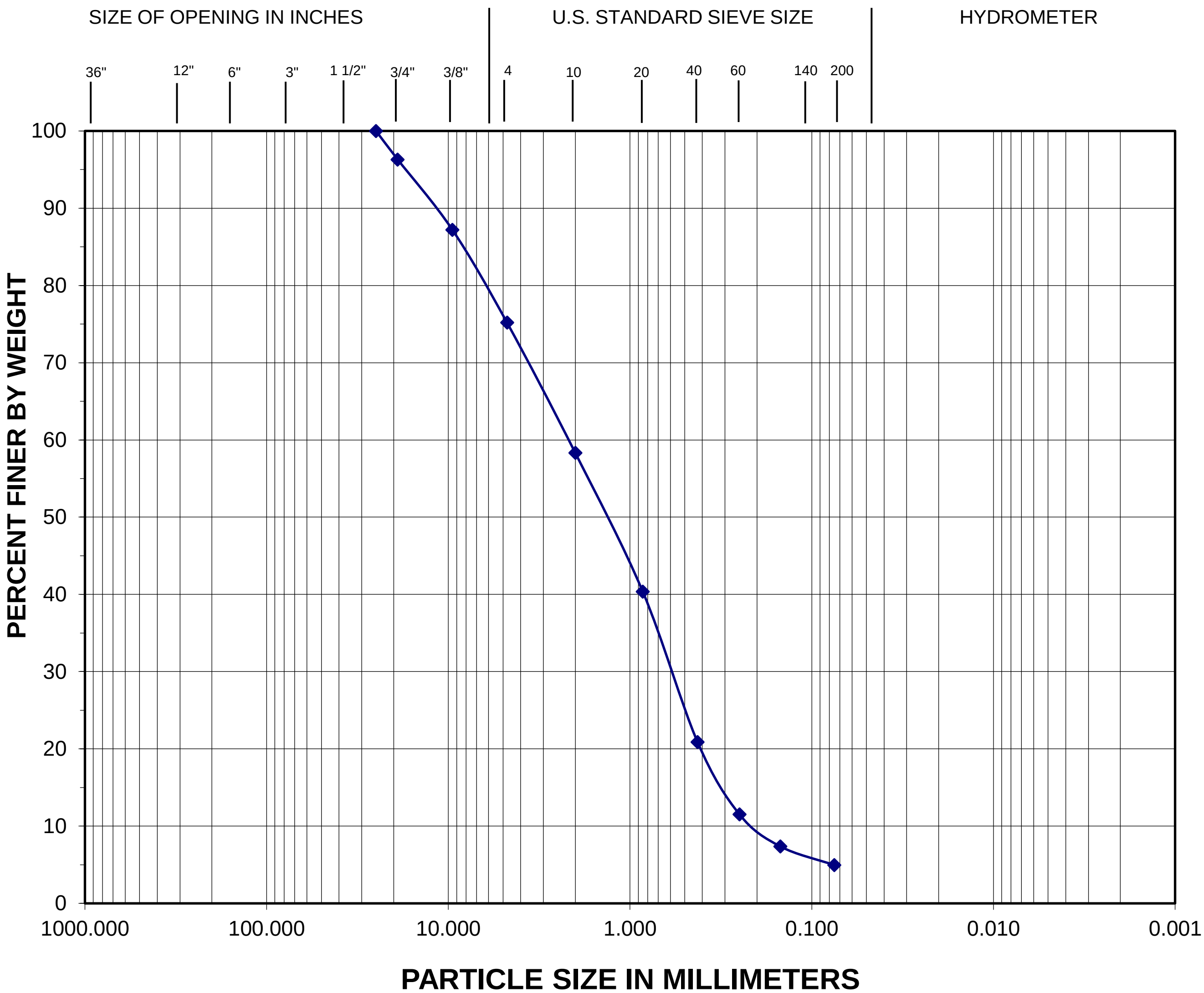
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-3	5	4.8	5.6	Gravelly SAND, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

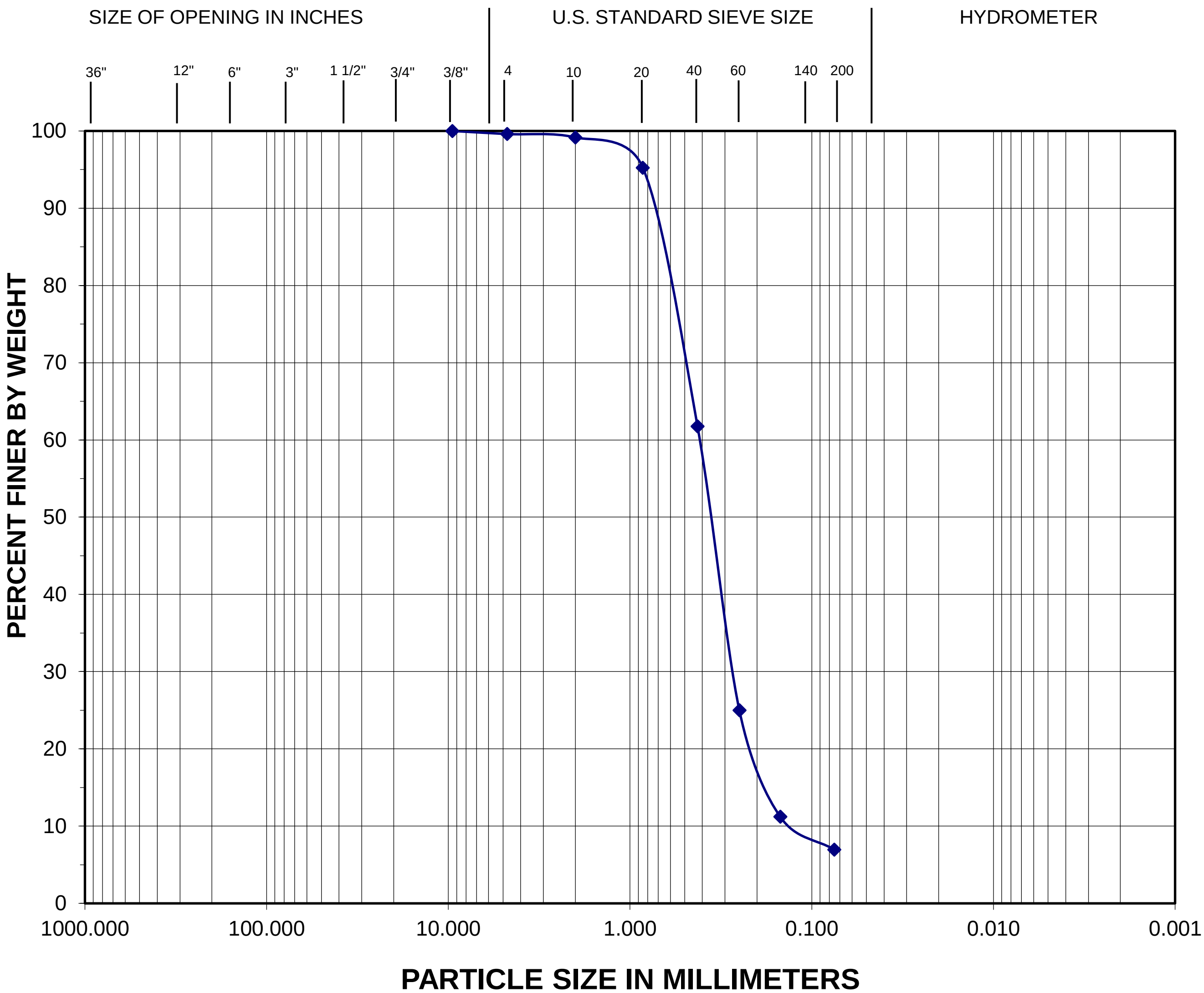
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-6	12.5	11.6	4.9	SAND, with gravel, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

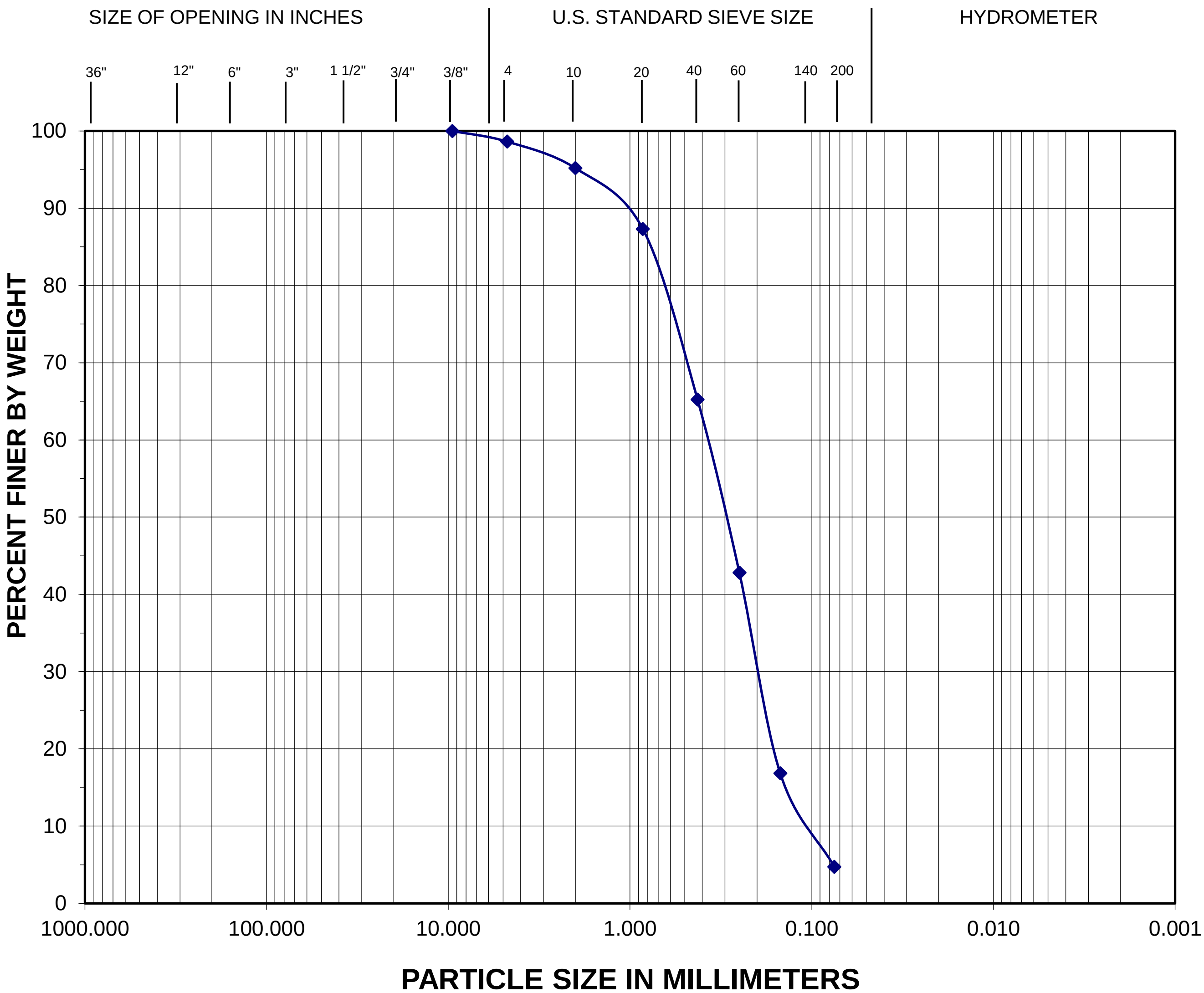
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-8	20	21.1	6.9	SAND, some silt, Trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

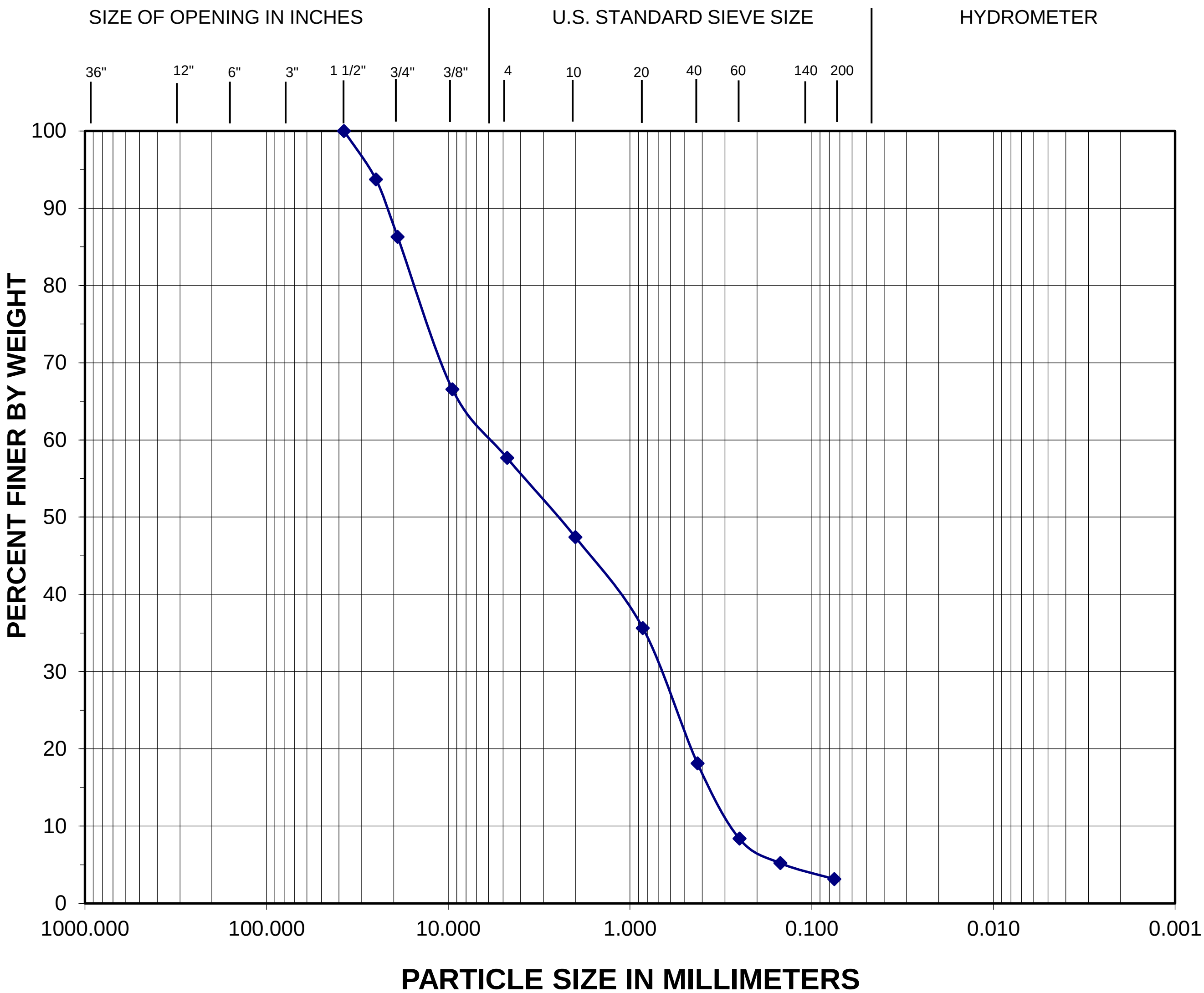
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-1	S-1	2.5	9.4	4.7	SAND, trace silt and gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind 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:

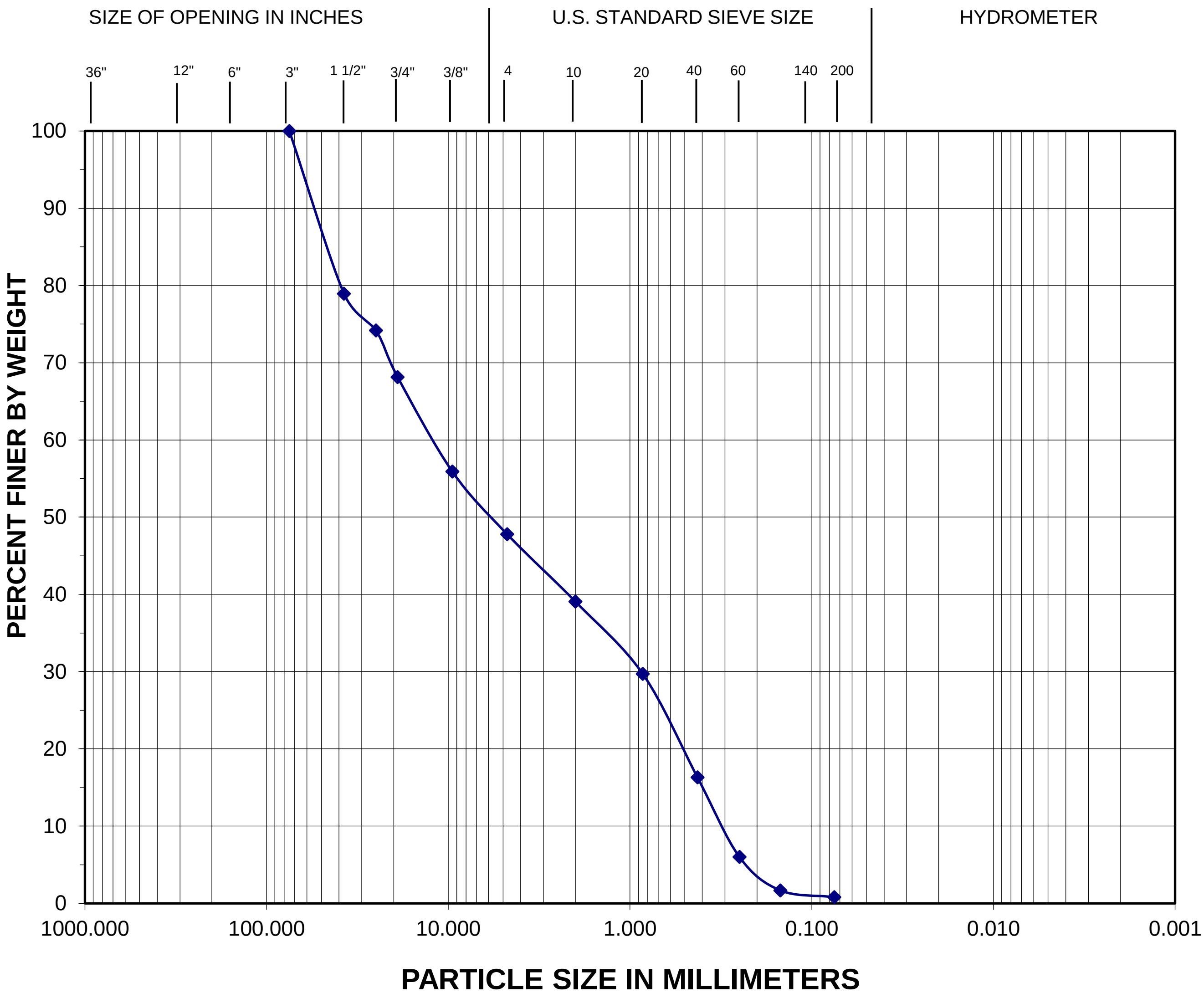
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-1	S-2	4	5.8	3.1	Gravelly SAND, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

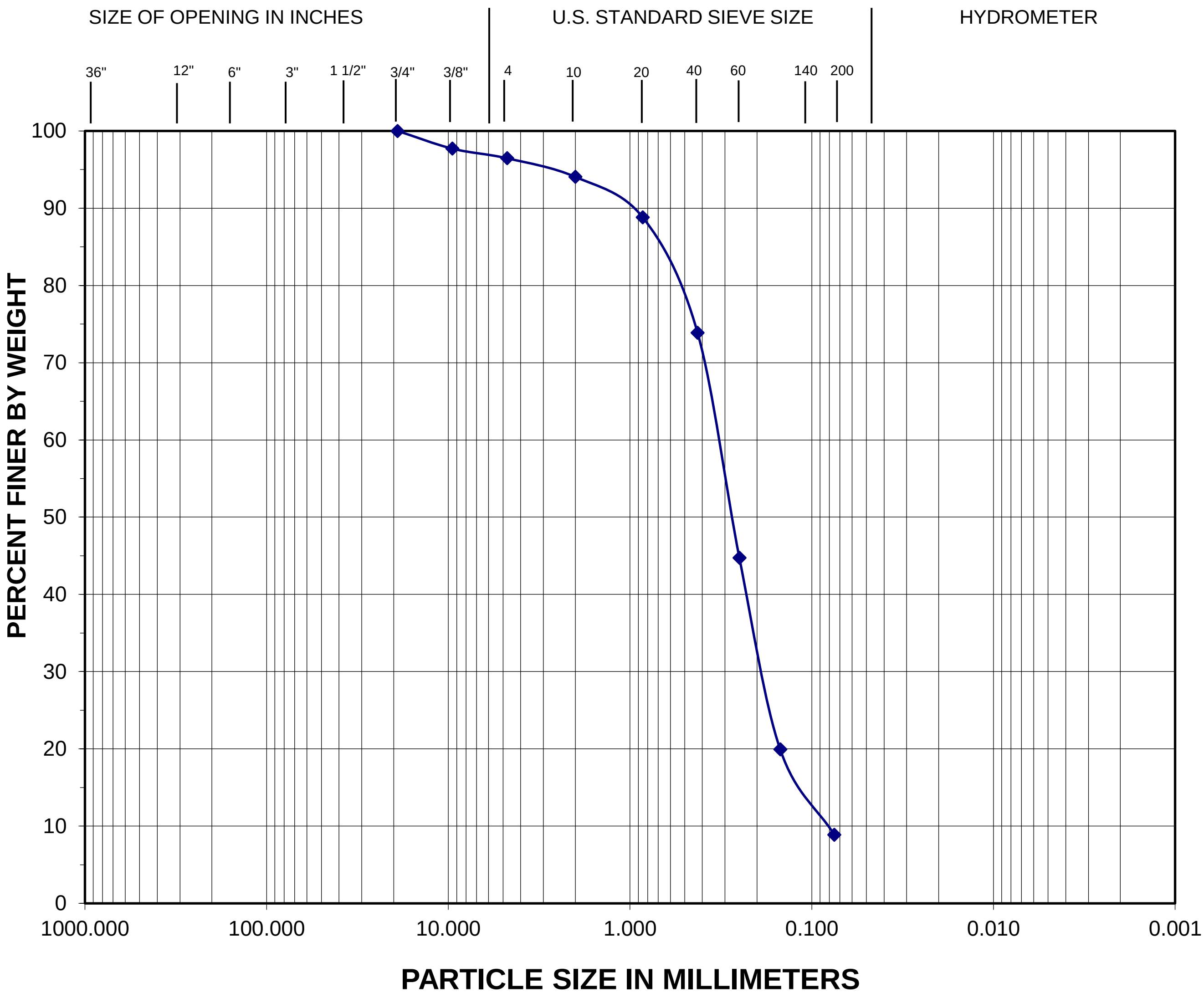
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-1	S-3	6.5	4.1	0.8	Sandy Gravel, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

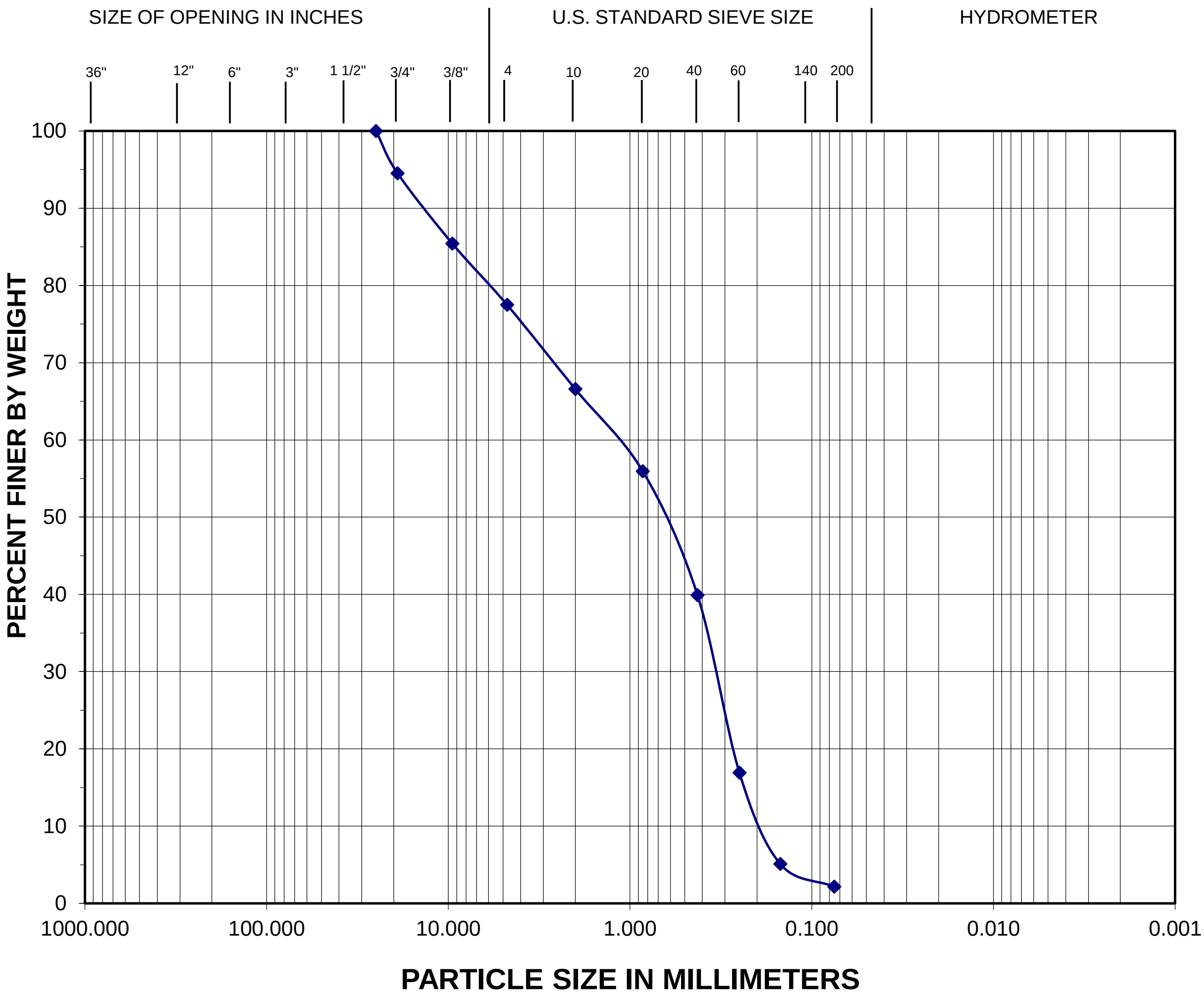
Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-6	S-1	1	12.2	8.9	SAND, some silt, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS
Test Results Summary

ASTM D6913



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

Comments:

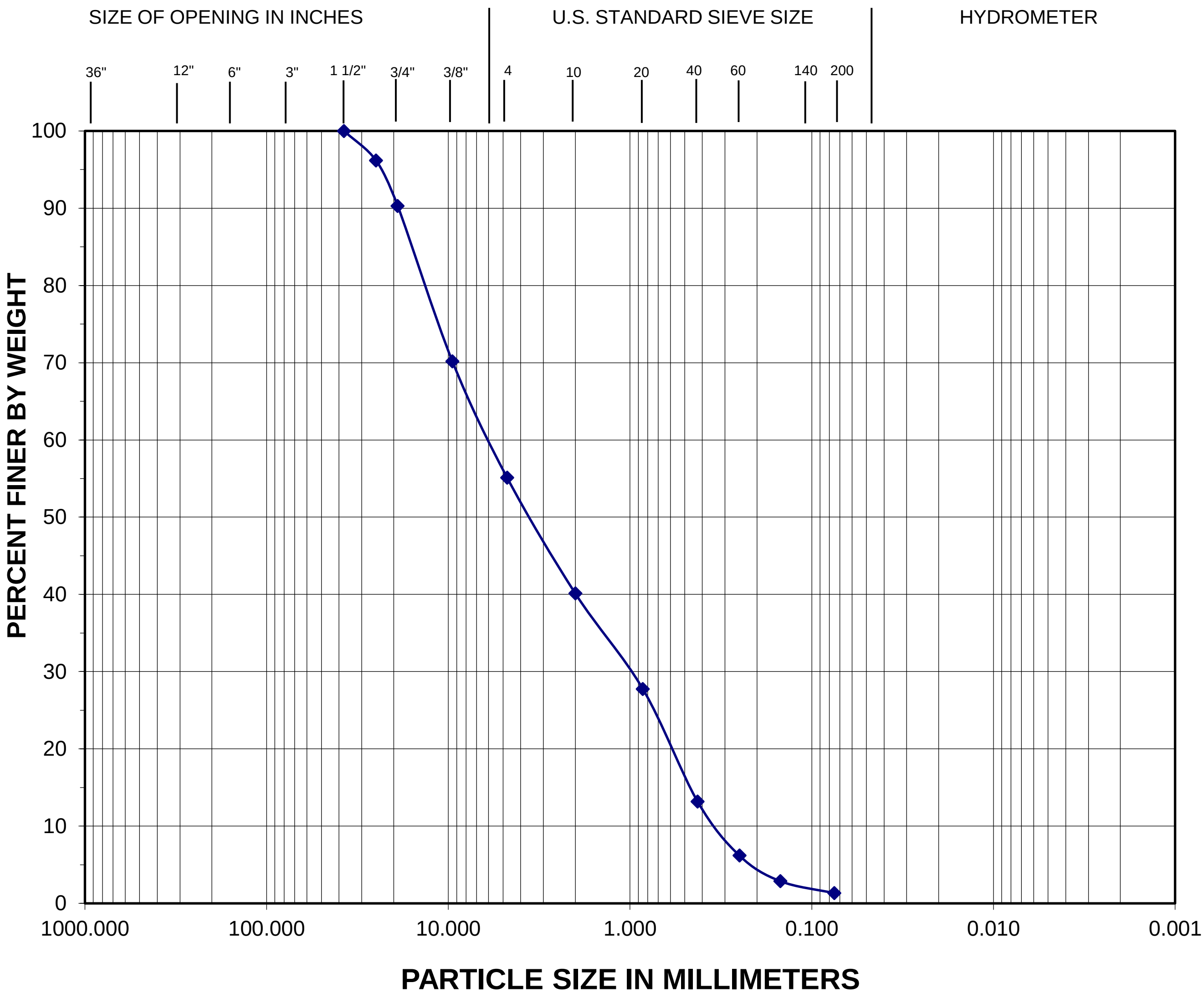
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-6	S-2	4	6.1	2.1	SAND, with gravel, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

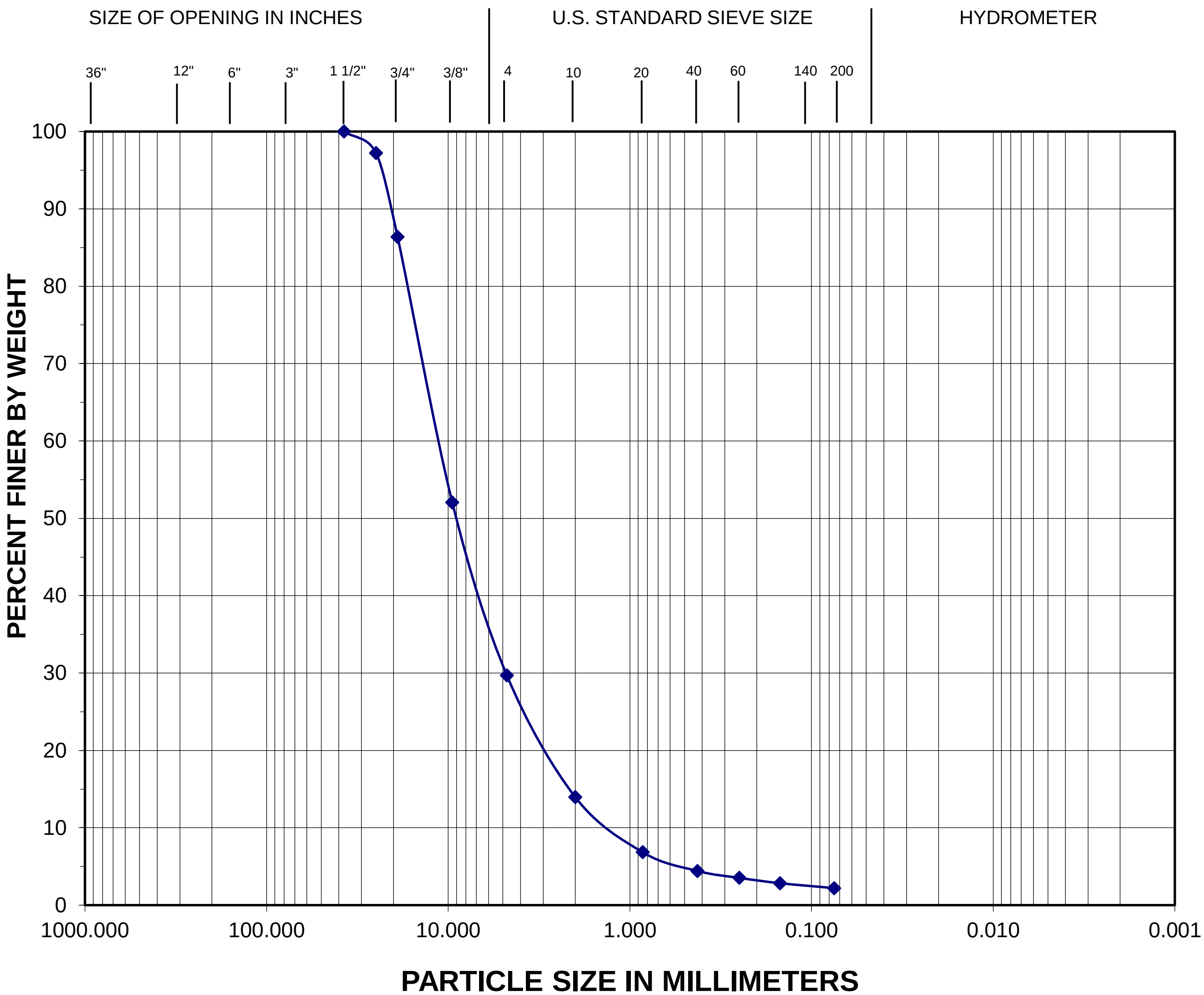
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-6	S-3	7.5	9.5	1.3	Gravelly SAND, trace silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind 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
Grab	032923	Stockpile	0.3	2.2	Iron Mtn. CSBC

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2679.01 DATE OF TESTING: 3/2/2023	PROJECT NAME: Crosswind Substation

March 3, 2023

NVL
INDUSTRIAL HYGIENE SERVICES
LABORATORY + MANAGEMENT + TRAINING

Justin Brooks
Zipper Geo Associates, LLC
19019 36th Avenue West, Suite E
Lynnwood, WA 98036

RE: Bulk Asbestos Fiber Analysis; NVL Batch # 2303348.00

Client Project: 2679.01
Location: Arlington WA

Dear Mr. Brooks,

Enclosed please find test results for the 8 sample(s) submitted to our laboratory for analysis on 3/1/2023.

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

NVLAP
Testing
Lab Code: 102063-0

Enc.: Sample Results

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

Bulk Asbestos Fibers Analysis
By Polarized Light Microscopy

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

Attention: Mr. Justin Brooks
Project Location: Arlington WA

Batch #: 2303348.00
Client Project #: 2679.01
Date Received: 3/1/2023
Samples Received: 8
Samples Analyzed: 8
Method: EPA/600/R-93/116

Lab ID: 23021416 **Client Sample #: TP 1**
Location: Arlington WA
Layer 1 of 1 **Description:** Gray sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		Asbestos Type: %
Sand, Fine grains, Fine particles	None Detected	ND	**None Detected ND**

Lab ID: 23021417 **Client Sample #: TP 2**
Location: Arlington WA
Layer 1 of 1 **Description:** Brown sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		Asbestos Type: %
Sand, Fine grains, Fine particles	None Detected	ND	**None Detected ND**

Lab ID: 23021418 **Client Sample #: TP 3**
Location: Arlington WA
Layer 1 of 1 **Description:** Brown sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		Asbestos Type: %
Sand, Fine grains, Fine particles	None Detected	ND	**None Detected ND**

Lab ID: 23021419 **Client Sample #: TP 4**
Location: Arlington WA
Layer 1 of 1 **Description:** Brown sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		Asbestos Type: %
Sand, Fine grains, Fine particles	None Detected	ND	**None Detected ND**

Lab ID: 23021420 **Client Sample #: TP 5**
Location: Arlington WA
Layer 1 of 1 **Description:** Brown sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		Asbestos Type: %
Sand, Fine grains, Fine particles	None Detected	ND	**None Detected ND**

Sampled by: Client
Analyzed by: Akane Yoshikawa **Date:** 03/03/2023
Reviewed by: Nick Ly **Date:** 03/03/2023

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

ASB-02

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. Justin Brooks
Project Location: Arlington WA

Batch #: 2303348.00
Client Project #: 2679.01
Date Received: 3/1/2023
Samples Received: 8
Samples Analyzed: 8
Method: EPA/600/R-93/116

Lab ID: 23021421 **Client Sample #: TP 6**
Location: Arlington WA

Layer 1 of 1 **Description:** Beige sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		**Asbestos Type: %**
Sand, Fine grains, Fine particles	None Detected	ND	**None Detected ND**

Lab ID: 23021422 **Client Sample #: B 1**
Location: Arlington WA

Layer 1 of 1 **Description:** Gray sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		**Asbestos Type: %**
Sand, Fine grains, Cementitious particles	None Detected	ND	**None Detected ND**

Lab ID: 23021423 **Client Sample #: B 2**
Location: Arlington WA

Layer 1 of 2 **Description:** Brown sandy material

Non-Fibrous Materials:	Other Fibrous Materials:%		**Asbestos Type: %**
Sand, Fine grains, Fine particles	None Detected	ND	**None Detected ND**

Layer 2 of 2 **Description:** Black fibrous material

Non-Fibrous Materials:	Other Fibrous Materials:%		**Asbestos Type: %**
Binder/Filler	Synthetic fibers	97%	**None Detected ND**

Sampled by: Client
Analyzed by: Akane Yoshikawa **Date:** 03/03/2023
Reviewed by: Nick Ly **Date:** 03/03/2023

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

ASB-02

ASBESTOS LABORATORY SERVICES

NVL

Company	Zipper Geo Associates, LLC	**NVL Batch Number**	**2303348.00**
Address	19019 36th Avenue West, Suite E Lynnwood, WA 98036	**TAT**	2 Days **AH** No
		Rush TAT	
Project Manager	Mr. Justin Brooks	**Due Date**	3/3/2023 **Time** 1:35 PM
Phone	(425) 582-9928	**Email**	jbrooks@zippergeo.com
Cell	(813) 205-3481	**Fax**	(425) 582-9930

Project Name/Number: 2679.01 **Project Location:** Arlington WA

Subcategory PLM Bulk

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

Total Number of Samples 8 Rush Samples

	Lab ID	Sample ID	Description	A/R
1	23021416	TP 1		A
2	23021417	TP 2		A
3	23021418	TP 3		A
4	23021419	TP 4		A
5	23021420	TP 5		A
6	23021421	TP 6		A
7	23021422	B 1		A
8	23021423	B 2		A

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

Office Use Only	Print Name	Signature	Company	Date	Time
Received by	Hieu Ta		NVL	3/1/23	1335
Analyzed by	Akane Yoshikawa		NVL	3/3/23	
Results Called by					
☐ **Faxed** ☐ **Emailed**					

Special Instructions:

Date: 3/1/2023
Time: 2:32 PM
Entered By: Hilary Crumley

CHAIN of CUSTODY SAMPLE LOG

2303348

LABORATORY • MANAGEMENT • TRAINING

Client	Zipper Geo Associates, LLC
Street	19019 36th Avenue West, Suite E Lynnwood, WA 98036
Project Manager	Mr. Justin Brooks
Project Location	ARLINGTON, WA
Phone:	(425) 582-9928
Fax:	(425) 582-9930
NVL Batch Number	
Client Job Number	2679.01
Total Samples	8
Turn Around Time	☐ 1 Hr ☐ 6 Hrs ☐ 3 Days ☐ 10 Days ☐ 2 Hrs ☐ 1 Day ☐ 4 Days ☐ 4 Hrs ☑ 2 Days ☐ 5 Days
	Please call for TAT less than 24 Hrs
Email address	jbrooks@zippergeo.com
Cell	(813) 205-3481

☐ **Asbestos Air** ☐ PCM (NIOSH 7400) ☐ TEM (NIOSH 7402) ☐ TEM (AHERA) ☐ TEM (EPA Level II) ☐ Other ____

☑ **Asbestos Bulk** ☑ PLM (EPA/600/R-93/116) ☐ PLM (EPA Point Count) ☐ PLM (EPA Gravimetry) ☐ TEM BULK

☐ **Mold/Fungus** ☐ Mold Air ☐ Mold Bulk ☐ **Rotometer Calibration**

METALS	Det. Limit	Matrix		RCRA Metals	☐ All 8	Other Metals
☐ Total Metals	☐ FAA (ppm)	☐ Air Filter	☐ Paint Chips in %	☐ Arsenic (As)	☐ Lead (Pb)	☐ All 3
☐ TCLP	☐ ICP (ppm)	☐ Drinking water	☐ Paint Chips in cm2	☐ Barium (Ba)	☐ Mercury (Hg)	☐ Copper (Cu)
☐ Cr 6	☐ GFAA (ppb)	☐ Dust/wipe (Area)	☐ Waste Water	☐ Cadmium (Cd)	☐ Selenium (Se)	☐ Nickel (Ni)
	☐ CVAA (ppb)	☐ Soil	☐ Other	☐ Chromium (Cr)	☐ Silver (Ag)	☐ Zinc (Zn)

☐ **Other Types of Analysis** ☐ Fiberglass ☐ Nuisance Dust ☐ Other (Specify) ____ ☐ Silica ☐ Respirable Dust

Condition of Package: ☐ Good ☐ Damaged (no spillage) ☐ Severe damage (spillage)

Seq. #	Lab ID	Client Sample Number	Comments (e.g Sample are, Sample Volume, etc)	A/R
1		TP 1	SAND to SILTY SAND	
2		TP 2		
3		TP 3		
4		TP 4		
5		TP 5		
6		TP 6		
7		B-1		
8		B-2		
9				
10				
11				
12				
13				
14				
15				

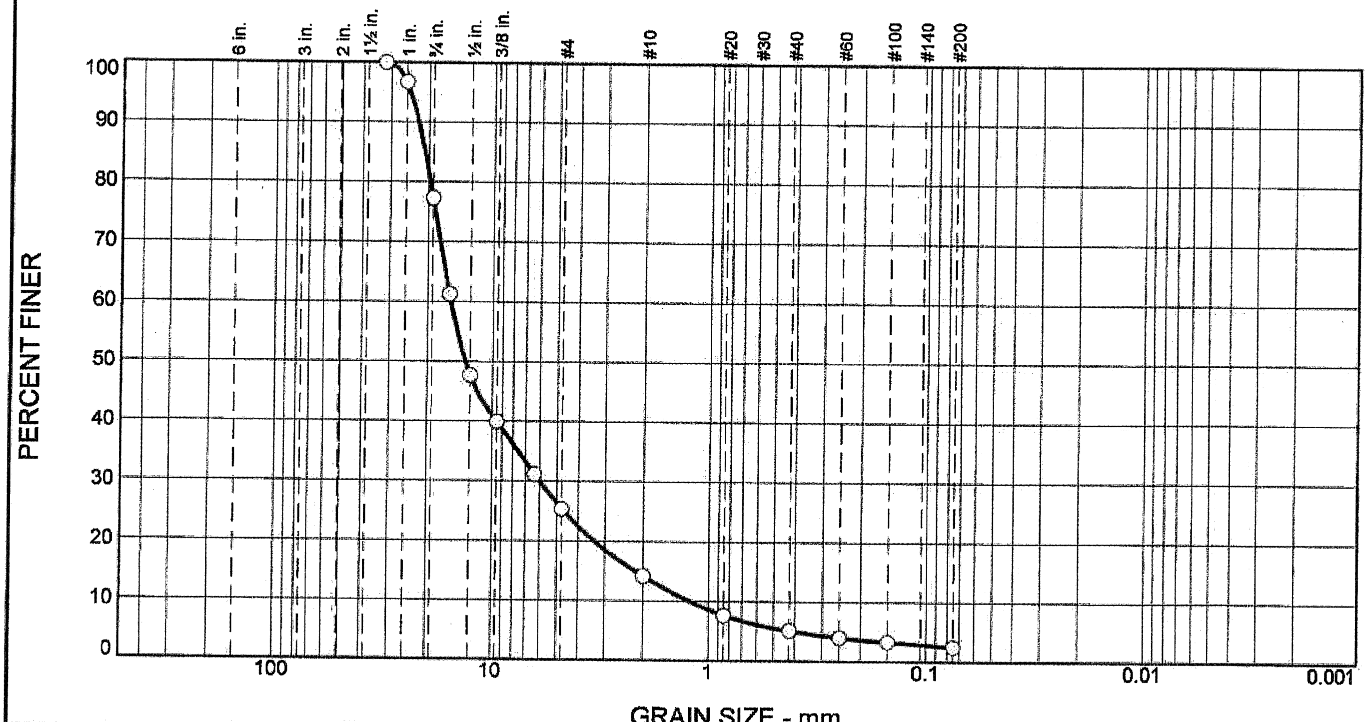
	Print Below	Sign Below	Company	Date	Time
Sampled by	Justin L Brooks	[signature]	Zipper Geo Assoc	~~3-1-23~~ 2.28	13:29
Relinquished by	Justin Brooks	[signature]	Zipper Geo	3-1-23	1329
Received by	Hieu Ta	[signature]	NVL Labs	3/1/23	1335
Analyzed by					
Results Called by					
Results Faxed by					

Special Instructions: Unless requested in writing, all samples will be disposed of two (2) weeks after analysis.

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

Grain size distribution plots and permeability testing results for Iron Mountain Quarry crushed surfacing base course follow

Krazan & Associates Sieve Analysis



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	22.7	51.9	11.1	9.2	2.6	2.5	

Test Results (ASTM C-136 & ASTM C-117)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.25	100.0		
1	96.8		
.75	77.3		
.625	61.2		
.5	47.7		
.375	39.9		
.25	31.1		
#4	25.4		
#10	14.3		
#20	7.7		
#40	5.1		
#60	4.0		
#100	3.3		
#200	2.5		

* (no specification provided)

Material Description

Permeable Base Course Aggregate

Atterberg Limits (ASTM D 4318)

PL= NP LL= NP PI= NP

Classification

USCS (D 2487)= GW AASHTO (M 145)= A-1-a

Coefficients

D_{90}= 22.3387 D_{85}= 20.8670 D_{60}= 15.6250
D_{50}= 13.3510 D_{30}= 6.0182 D_{15}= 2.1603
D_{10}= 1.2184 C_u= 12.82 C_c= 1.90

Remarks

Fracture Face (WSDOT T-335): 100% (Spec = 75% MIN)
Permeability Rate (ASTM D-2434): 170.5 in/hr. (Spec = 40 inches per hour at 98% compaction).

Date Received: 4/11/2022 **Date Tested:** 4/11/2022

Tested By: Thomas Khilfeh

Checked By: Cole Demas

Title: Lab Manager

Location: Client Supplied
Sample Number: 77953

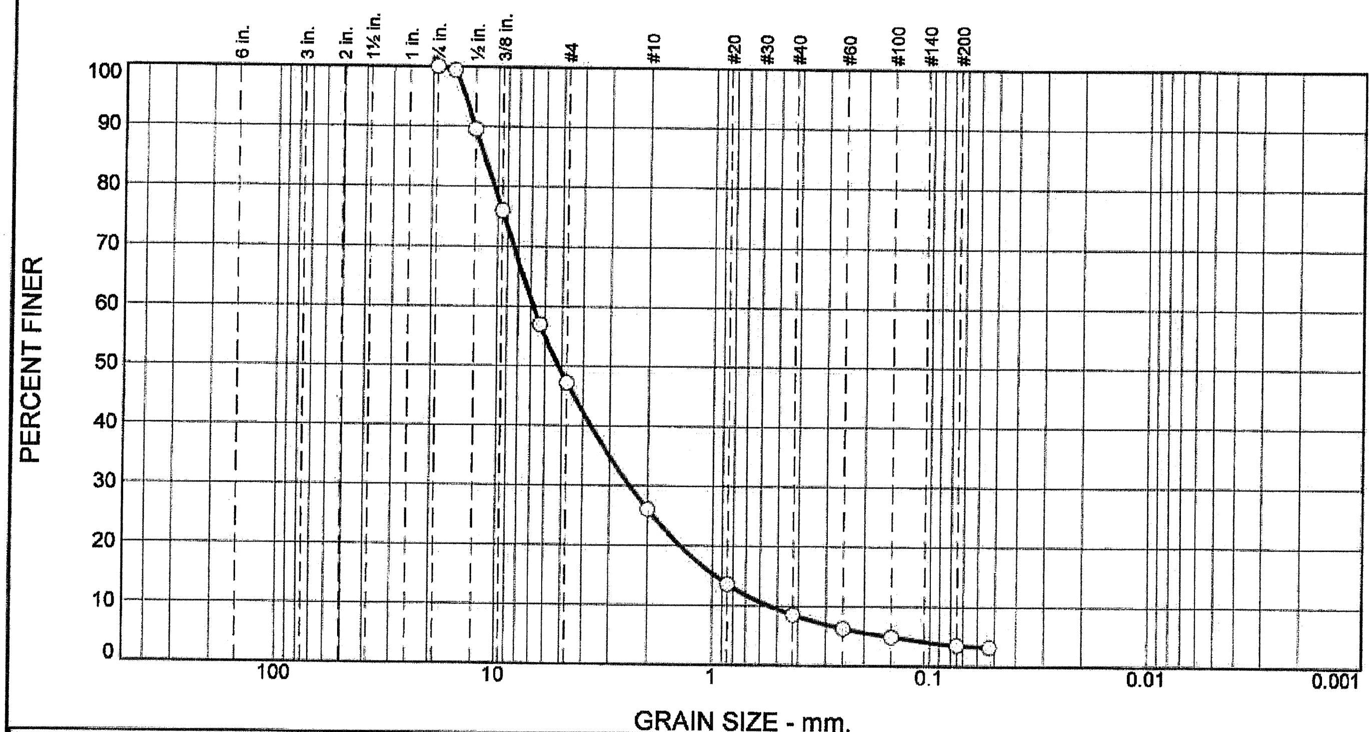
Date Sampled: 4/11/2022

Krazan

Client: Iron Mountain Quarry
Project: 2022 Control Sample

Project No: 09622094 **Figure**

Krazan & Associates Sieve Analysis



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	52.8	21.1	17.5	5.1	3.5	

Test Results (ASTM C-136 & ASTM C-117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.625	99.4		
.5	89.5		
.375	75.9		
.25	57.0		
#4	47.2		
#10	26.1		
#20	13.6		
#40	8.6		
#60	6.3		
#100	4.9		
#200	3.5		
#270	3.2		

* (no specification provided)

Material Description

Permeable Top Course Aggregate

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NP **PI=** NP

Classification

USCS (D 2487)= GW **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 12.8232 D_{85}= 11.5908 D_{60}= 6.8198
D_{50}= 5.2088 D_{30}= 2.4245 D_{15}= 0.9681
D_{10}= 0.5445 C_u= 12.53 C_c= 1.58

Remarks

Fracture Face (WSDOT T-335): 100% (Spec = 75%Min)
Permebility Rate (ASTM D-2434): 168.5 in/hr (Spec = 20 inches per hour at 98% compaction).

Date Received: 4/11/2022 **Date Tested:** 4/12/2022

Tested By: Thomas Khilfeh

Checked By: Cole Demas

Title: Lab Manager

Location: Client Supplied
Sample Number: 77954

Date Sampled: 4/11/2022

Krazan

Client: Iron Mountain Quarry
Project: 2022 Control Sample

Project No: 09622094 **Figure**

APPENDIX C
LIQUEFACTION ANALYSIS OUTPUT PLOT

Geoprofessional Consultants

Zipper Geo Associates, LLC
19019 36th Avenue West, Suite E
Lynnwood, Washington
(425) 582-9928

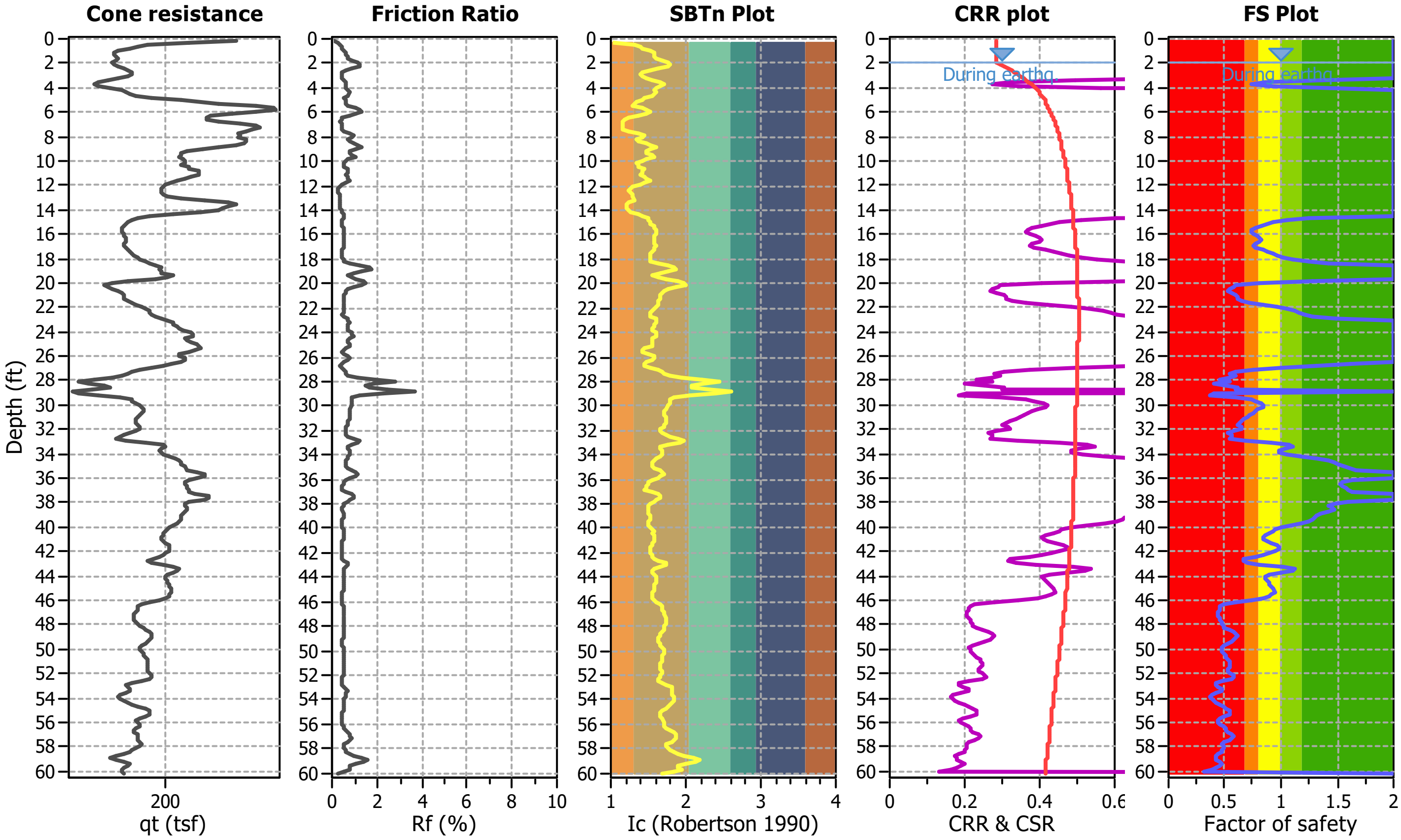
LIQUEFACTION ANALYSIS REPORT

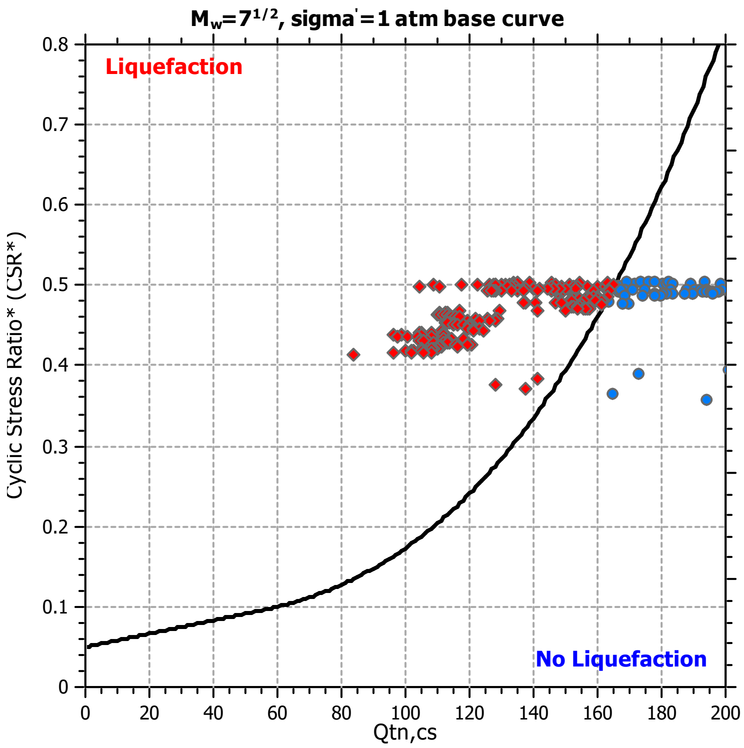
Project title : Crosswind Substation **Location : Arlington, Washington**

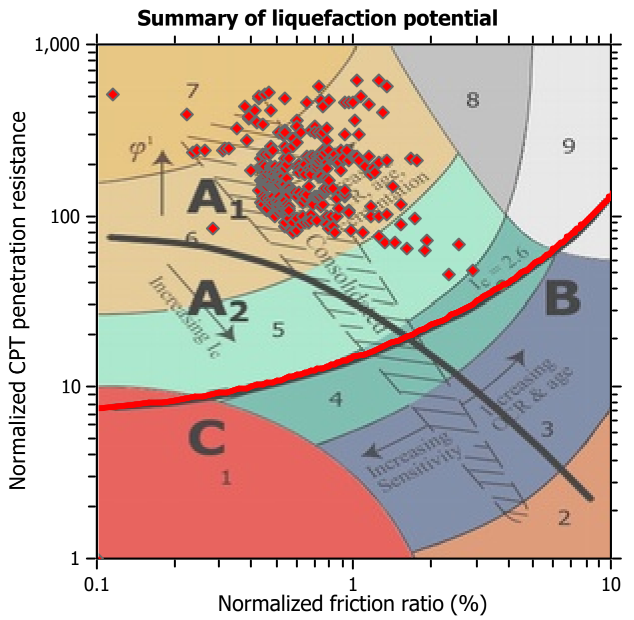
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.03	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.52	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

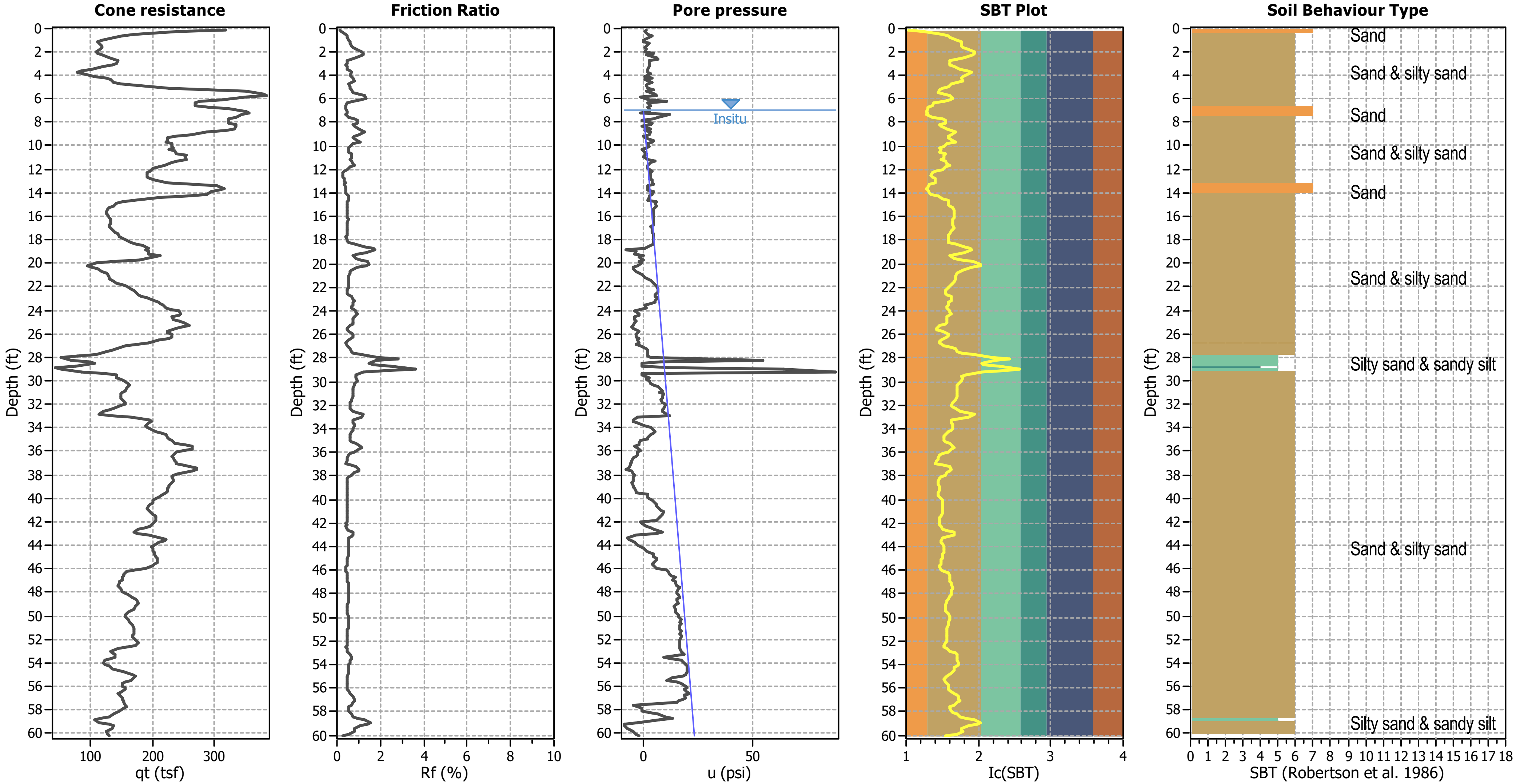






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



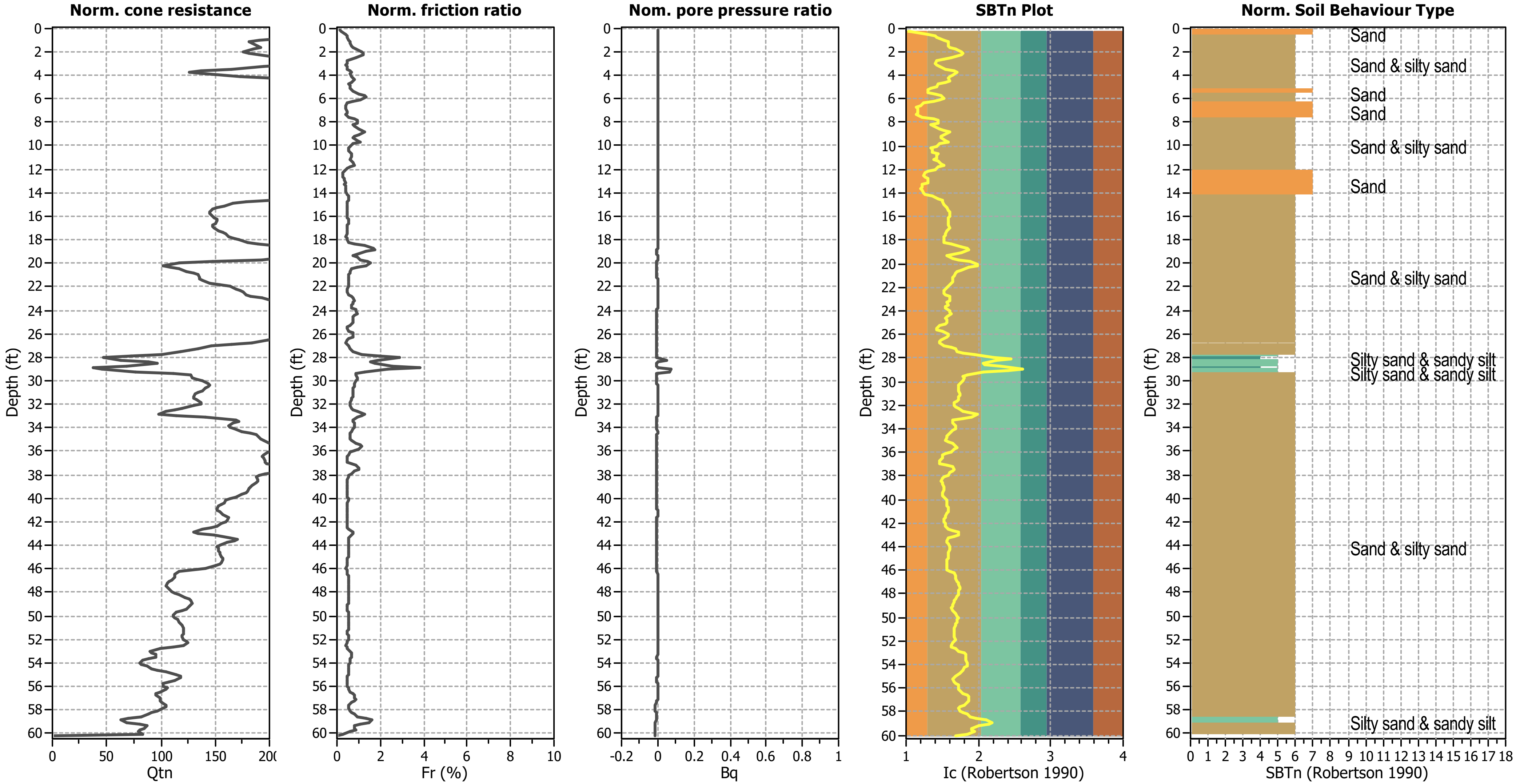
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.52	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



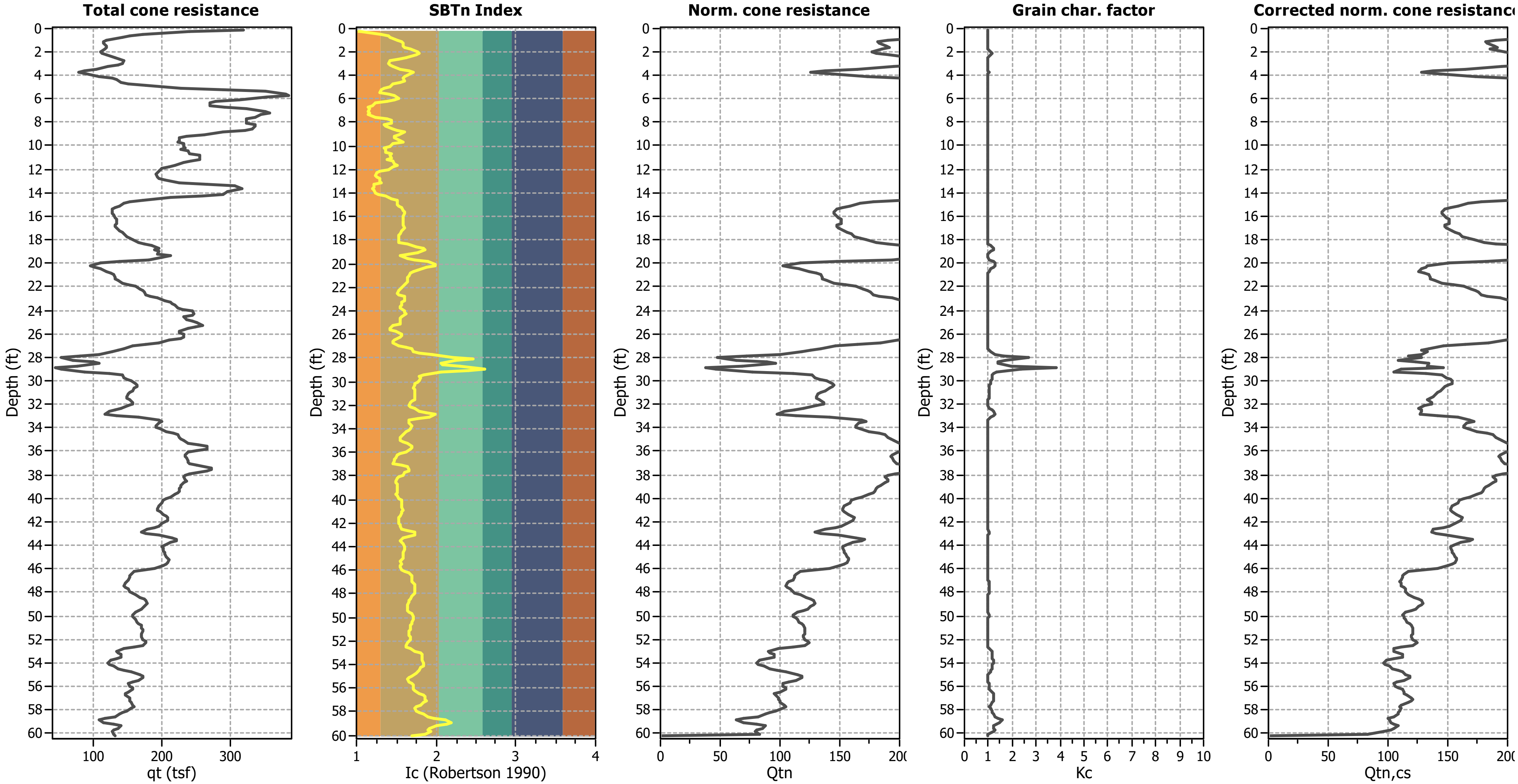
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.52	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

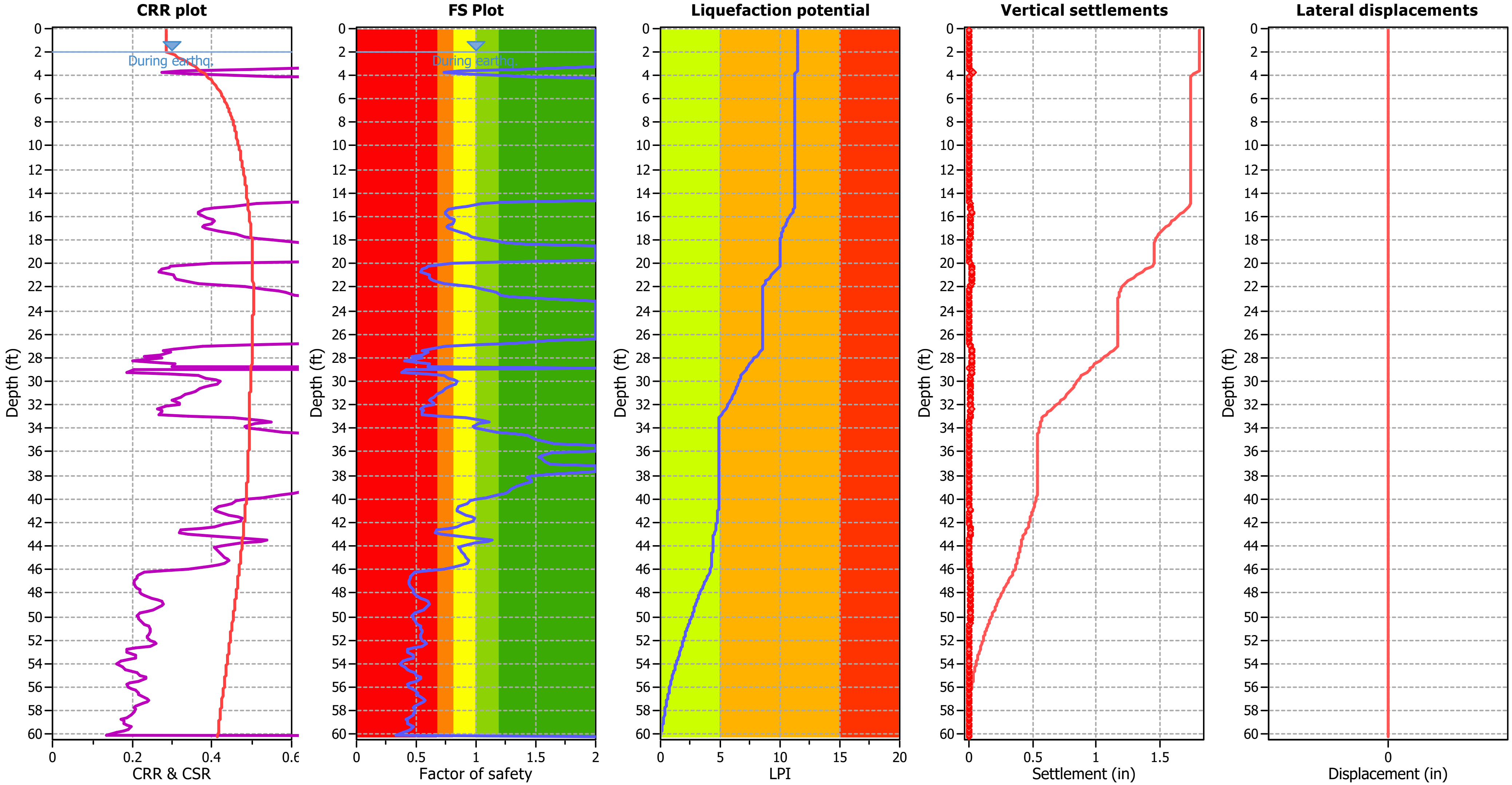
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.52	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.52	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk