

# GEOTECHNICAL ENGINEERING REPORT

SCRIBER LAKE LOW-INCOME HOUSING  
5707 & 5723 198<sup>TH</sup> STREET SW  
LYNNWOOD, WASHINGTON

ZGA Project No. 2955.01  
November 12, 2025

Prepared for:  
**BLACKFISH CAPITAL**



Prepared by:

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ZGA Project No. 2955.01

November 14, 2025

Mr. Johnny Vong  
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2442 Market St NW Suite 562  
Seattle, Washington 98107

**Subject: *Revised Draft Geotechnical Engineering Report***

Scriber Lake Low-Income Housing

5707 & 5723 198<sup>th</sup> Street SW

Lynnwood, Washington 98036

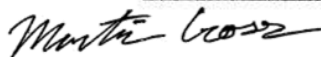
Dear Mr. Vong,

In accordance with your request and written authorization, Zipper Geo Associates, LLC (ZGA) has completed our geotechnical engineering report for the above-referenced project. This report presents the findings of the subsurface evaluation and geotechnical recommendations for project design and construction. Our services were completed in general accordance with our proposal (P25091) dated July 14, 2025. Our services were authorized by you on August 8, 2025, via signature of a Professional Services Agreement.

Sincerely,

**Zipper Geo Associates, LLC**

**DRAFT**



Martin R. Cross, LG

Project Geologist

Signed: x/xx/25

Robert A. Ross, PE

Managing Principal

Signed: x/xx/25

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Figure 1 – Site and Exploration Plan

**APPENDICES**

Appendix A – Subsurface Exploration Procedures and Logs

Appendix B – Laboratory Testing Procedures and Results

Appendix C – Nelson Geotechnical Associates. Inc. Test Pit Logs, dated July 10, 2024

**GEOTECHNICAL ENGINEERING REPORT**  
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**5707 & 5723 198<sup>TH</sup> STREET SW**  
**LYNNWOOD, WASHINGTON 98036**

Project No. 2955.01  
November 14, 2025

**1.0 INTRODUCTION**

This report presents the surface and subsurface conditions encountered at the site and our geotechnical engineering recommendations for the proposed Scriber Lake low-income housing project. Our scope of services included reviewing readily available geologic data, reviewing the previously completed geotechnical report prepared by Nelson Geotechnical Associates, Inc. (NGA), dated July 10, 2024, a site reconnaissance, subsurface evaluation, laboratory testing, geotechnical engineering analysis, and preparation of this report. The project description, site conditions, and our geotechnical conclusions and recommendations are presented in the text of this report. Supporting data including detailed exploration logs and field exploration procedures, the results of laboratory testing, and other analyses results are presented as appendices.

**1.1 Site Description**

The project site consists of two neighboring parcels along the northern side of 198<sup>th</sup> ST SW, address number 5723 on the west side (Snohomish County Parcel No. 00565300000401) and 5707 on the eastern side (Snohomish County Parcel No. 00565300000404). The project site is bordered by commercial property to the north and west, by Scriber Lake Park to the west, and 198<sup>th</sup> ST SW to the south. Overall site topography slopes from the southwest to the northeast to east towards Scriber Lake Park. The maximum topographic relief of the project site is approximately 21 feet. Each property is discussed individually below.

Property 5723 consists of a developed 0.41-acre lot, currently occupied by a single-story, approximately 1,746 square foot building with a daylight basement, located in the center of the property. The majority of the remaining property is surfaced with asphalt parking and driveways, a small playground on the southwestern side of the building, and a shed on the northeastern corner of the property. Asphalt Driveways connect to the site from 198<sup>th</sup> St SW on the southeastern and southwestern corners of the property. A row of conifer trees is located along the eastern and western sides of the site, with the remaining portion of the site surfaced with typical landscaping. The property slopes gently downward towards the northeast.

Property 5707 consists of a partially developed 0.8-acre lot, currently occupied by a 1-story single-family home with a daylight basement, on the southern side of the lot. An asphalt driveway connects the property at the southwestern corner to 198<sup>th</sup> St SW, extending along the western side of the home. There is a concrete pad located on the northern side of the home. The area between the southern side of the building and 198<sup>th</sup> St SW is surfaced with typical residential landscaping, except for a large steel cargo container, located in the center of this area. The remaining portion of the property to the north of the residence is forested with a mixture of large conifer trees, alders, small shrubs, and blackberries. While

on site, we observed pieces of old concrete foundations amongst the brush on the northern side of the property, although it is unknown if this is remnants of an older structure or debris. The southern half of the property slopes moderately downward to the east, towards the neighboring Scriber Lake Park, with relatively level areas to the north and south of the house. The northern half of the property slopes gently downward to the east to southeast, towards Scriber Lake Park.

## **1.2 Project Understanding**

Based on the plans provided to us at the time of this report, we understand that the proposed project will consist of constructing a new six-story building, with a partial daylight basement, occupied by low-income housing, on the project site. The daylight basement footprint is limited to approximately 4,440 square feet located on the southeastern corner of the building and will include the leasing office, bike storage, and other amenities. The first floor will primarily consist of a parking garage with limited apartments on the southern side, with the remaining stories completely occupied with apartments. Finished floor elevation for the daylight basement and at-grade Level 2 of the building are currently planned at 360 ft and 372 ft, respectively.

The complex will include a one-way driveway entering the site on the eastern side of the building and exiting on the western side, both connecting to 198<sup>th</sup> St SW. Several retaining walls are planned to retain cuts/fills and landscape planters. A majority of the site retaining walls are expected to consist of cast-in-place concrete cantilever walls. However, a proposed fill retaining wall along the eastern property line is expected to consist of a mechanically stabilized earth wall. The maximum exposed height of site retaining walls is currently planned at about 12 ft.

We anticipate the project will include related improvements such as new underground utilities and landscaping. We understand a new approximately 3,200 square-foot stormwater detention vault will be installed on the eastern side of the site, located below the entrance driveway. At the time of this report, project plans indicated the bottom of vault elevation will be located at approximately 347 feet at the center of the vault, grading to 347.5 at the sidewalls of the vault.

## **2.0 SUBSURFACE CONDITIONS**

### **2.1 General**

In order to characterize subsurface soil and groundwater conditions at the project site, we reviewed readily available published surficial geologic mapping and completed five (5) geotechnical soil borings, two borings on the western property and three on the eastern property. The following sections describe our understanding of subsurface soil and groundwater conditions based on the mapping, our borings, and the test pits from the NGA 2024 report.

### **2.2 Published Surficial Geologic Mapping**

We assessed the geologic conditions of the site and surrounding vicinity by reviewing the publication Geologic Map of the Edmonds East and Part of the Edmonds West Quadrangles, Washington, U.S. Geological Survey, Miscellaneous Field Studies, Map MF-1541, dated 1983. The geologic map indicates

that the site is mantled by glacial till (Qvt), with younger alluvium (Qyal) mapped directly to the north and west, surrounding Scriber Lake. Glacial till is described as unsorted, unstratified, compact, sand, silt, and clay with gravel to very large boulders. Glacial till is typically dense to very dense in nature due to being overridden during historical glacial advance. Glacial till generally has a high fines content (the soil fraction passing the US No. 200 sieve) which contributes to the soil's relatively low permeability in combination with the soil density. Younger alluvium is described as poorly drained fluvial sediments consisting of mostly sand and gravel, with some organic-rich mud. These deposits are normally consolidated and generally loose to medium dense in nature. Subsurface soil conditions encountered in our borings appear to correspond to deposits of glacial till noted in the above-referenced geologic mapping.

### **2.3 Soil Conditions**

In addition to reviewing geologic mapping, we completed five (5) geotechnical soil borings at the site on August 21 and 22, 2025, as described above. Approximate locations of each boring are shown on the attached Figure 1, Site and Exploration Plan. Soil borings B-1 through B-4 were advanced to about 28-¼ to 31 feet below existing grade and B-5 was advanced to 10-¼ feet below grade. In addition, we reviewed the test pit logs detailed in the previously prepared geotechnical engineering report from property 5707, prepared by Nelson Geotechnical Associates. Inc. (dated July 10, 2024).

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

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

**ESU 1 – Probable Fill:** Soils interpreted to be probable undocumented fill material were encountered in each boring below the pavement in borings B-1 and B-2 and below the topsoil in borings B-3 through B-5, extending to between 2 to 3 feet below existing grade. Test pits TP-1 through TP-4 and INF-1 from the NGA report indicated loose to medium dense soils, similar to those ESU 1 soils encountered in our

explorations, were encountered below the topsoil in each test pit, extending to three to 5 feet below existing grade. ESU 1 soils observed in our explorations generally consisted of loose to medium dense, moist, light brown to brown, sand with varying gravel and silt content. Trace wood and concrete debris were observed within the fill in B-2 and B-4.

**ESU 2 – Glacial Till:** Soils interpreted to be glacial till were encountered in each boring below the probable fill ESU 1 soils, extending to the termination of each boring between 28 to 31-½ feet below existing grade. Test pits TP-1 through TP-4 and INF-1 from the NGA report indicated medium dense to very dense soils, similar to those ESU 2 soils encountered in our explorations, were encountered between 3 to 5 feet, extending to the termination of each test pit between 5-½ to 8 feet below existing grade. ESU 2 soils observed in our explorations generally consisted of dense to very dense, moist, gray-brown to gray, sand with gravel and varying silt content to sandy gravel, with varying silt content.

A summary of soil conditions encountered in our borings, including approximate ESU 2 contact elevations are presented in the table below.

Table 1: Summary of Soil Conditions				
Exploration Location	Thickness of ESU 1 – Fill (ft)	Thickness of ESU 2 – Glacial Till (ft)	ESU 2 Glacial Till Contact Elevation (ft) <sup>1</sup>	Total Exploration Depth (ft)
B-1	2	29-½	366	31-½
B-2	2	26-¼	367-½	28-¼
B-3	2	29-½	357	31-½
B-4	3	28-½	364-½	31-½
B-5	4	6-¼	361-½	10-¼

<sup>1</sup>Contact elevations were based on ground surface elevations at the exploration locations, obtained from the *Boundary and Topographic Survey* prepared by AHBL, dated September 3, 2025.

## 2.4 Groundwater Conditions

Groundwater was not encountered in our borings at the time of drilling. Groundwater measurements were taken from the dedicated monitoring wells installed in borings B-1, B-3, and B-4 on August 28, 2025 and November 7, 2025. A summary of groundwater elevations observed in each boring is presented in Table 2 below. According to the NGA report, groundwater was not encountered in test pits TP-1 through TP-4 at the time of excavation. Seepage was observed in INF-1 at approximately 5.4 feet; however, this was observed after completion of an infiltration test in that test pit, therefore it is our opinion that it does not reflect the groundwater table on site. Given the relatively high density of the underlying ESU 2 soils, it is possible that perched groundwater may develop on the contact during the wet season or periods of heavy rainfall. Groundwater levels should be expected to seasonably vary with higher groundwater levels in the winter and spring months and lower groundwater levels during drier summer months.

<b>Table 2: Summary of Groundwater Observations</b>				
Boring Number	Approx. Surface Elevation (ft) <sup>2</sup>	At Time of Drilling	Groundwater Elevation (ft) 8/28/2025	Groundwater Elevation (ft) 11/7/2025
B-1 <sup>1</sup>	368	Dry	Dry	343.1
B-2	369.5	Dry	NA	NA
B-3 <sup>1</sup>	359	Dry	336.7	336.4
B-4 <sup>1</sup>	367.5	Dry	343.7	342.7
B-5	365.5	Dry	NA	NA

<sup>1</sup>Groundwater monitoring well installed.

<sup>2</sup>Ground surface elevations at the exploration locations were obtained from the *Boundary and Topographic Survey* prepared by AHBL, dated September 3, 2025.

### 2.5 Summary of Laboratory Testing

Laboratory testing was completed on select soil samples between 2½ to 7½ feet below grade. Tests included moisture content and grain size analysis. Results of moisture content testing are shown on the boring logs presented in *Appendix A* and grain size analysis results are presented in *Appendix – B*. Table 3 below provides a summary of laboratory testing results completed on non-organic soil samples.

<b>Table 3: Summary of Laboratory Testing</b>				
Engineering Stratigraphic Unit	In Situ Moisture Content Range (% of dry weight)	Average In Situ Moisture Content (% of dry weight)	Fines Content Range (% of dry weight)	Average Fines Content (% of dry weight)
ESU-2 – Glacial Till	2 – 17	7	5 – 53	22

## 3.0 CONCLUSIONS AND RECOMMENDATIONS

### 3.1 General

Based on soil and groundwater conditions observed in our explorations, the proposed project appears feasible from the geotechnical perspective. Selected aspects of the site conditions that should be considered during design and construction are summarized below.

- Soils observed in our borings are generally favorable with respect to stie grading and foundation support.
- Reuse of onsite, non-organic, soils as structural fill during grading will be feasible provided that the soil moisture content can be adequately controlled prior to compaction. Reuse of native soils during periods of wet weather may be very difficult due to their relatively high fines content.

- Based on the definitions and requirements outlined in Section 17.10.100 of the City of Lynnwood Municipal Code, it is our opinion that the project site does not include landslide, erosion, or seismic geologically hazardous areas.
- Given the varying site topography, and the proposed bottom of footing elevations of 360 and 372 feet, we anticipate that the building foundations will bear on a combination of compacted structural fill and dense ESU 2 native soils. We have provided two foundation bearing capacities in this report reflecting both scenarios.
- The proposed stormwater detention vault on the southeastern side of the site is located in close proximity to the perimeter foundation of the daylight basement portion of the eastern side of the building. Based on finished floor elevations for this area and the bottom of vault elevation provided to us at the time of this report, it appears that the vault will be subject to a surcharge from the building foundation. We have provided two options for transferring the bearing stress of the footings deeper below grade to prevent imposing a surcharge on the vault walls, if the vault is not designed for these loads.

The following sections of this report present specific geotechnical recommendations for the project. Our recommendations are based on the observed soil conditions at specific exploration locations. Differing soil conditions than those observed at the boring locations may become evident during construction. Our recommendations are further based on the assumption that earthwork for site grading, utilities, foundations, floor slabs, and pavements will be monitored by a representative from ZGA.

### **3.2 Geologically Hazardous Areas**

According to the City of Lynnwood Geologically Hazardous Areas Map, the project site is not mapped within any geologically hazardous areas (GHAs), however, the area immediately around Scriber Lake, to the northeast to east of the project site, is mapped as Seismic Hazard Area GHA. Section 17.10.100 of the City of Lynnwood Municipal Code (LMC) indicates Geologically Hazardous Areas are divided into Landslide Hazard Areas, Erosion Hazard Areas, and Seismic Hazard areas. As part of our report, we have evaluated the presence or absence of geologically hazardous areas as defined by the LMC.

#### **3.2.1 Landslide Hazard Area**

Section 17.10.100 of the LMC defines landslide hazard areas as *Areas with slopes steeper than 40 percent. Areas between 15 to 40 percent that are underlain by soils largely consisting of silt and clay. Areas with slopes steeper than 15 percent with zones of emergent water such as ground water seepage or springs. And Areas of landslide deposits regardless of slope.* Based on the topographic survey of the site provided to us at the time of the report, the slopes on the eastern property range from approximately 10 to 34 percent, while the western property slopes gently at a maximum of about 5 percent. In general, our explorations encountered dense to very dense glacial till between 2 to 3 feet below grade, consisting of sand and gravel with varying silt content. Groundwater seepage and springs were not observed on the

slopes at the time of our site reconnaissance. Based on the slope inclinations, soil conditions encountered, and lack of emergent water on the slope, it is our opinion that the site does not meet the criteria for a Landslide Hazard Area GHA as defined by Section 17.10.100 of the LMC.

### **3.2.2 Erosion Hazard Area**

Section 17.10.100 of the LMC defines erosion hazard areas as *lands underlain by soils identified by the U.S. Department of Agriculture Natural Resources Conservation Service (NRCS) as having “severe” or “very severe” erosion hazards*. According to the USDA NRCS online Web Soil Survey, the majority of the project site is underlain by Alderwood-Urban land complex, 2 to 8 percent slopes, with the eastern side of the eastern property mapped as Mukilteo Muck. Soils encountered on site in our explorations and those previously detailed in the NGA report do not appear to correspond to soils described as Mukilteo Muck, therefore it is our opinion that these soils are not present on site. Alderwood-urban land complex 2 to 8 percent slopes are not considered as having severe or very severe erosion hazards. Soils encountered on site generally consisted of shallow dense to very dense glacial till, although shallow surficial soils were loose to medium dense in nature. Based on the lack of severe or very severe erosion hazard soils on site, it is our opinion that the site does not meet the criteria for an Erosion Hazard Area GHA as defined by Section 17.10.100 of the LMC.

Soils with relatively high fines contents were encountered through our explorations. When exposed, these site soils will be susceptible to erosion. Detailed recommendations regarding erosion control measures are presented in the *Erosion Control Measures* section of this report.

### **3.2.3 Seismic Hazard Area**

The project site is not mapped as a seismic hazard area according to City of Lynnwood Geologically Hazardous Areas Map, however, the area immediately around Scriber lake, to the northeast to east of the project site, is mapped as Seismic Hazard Area GHA. Section 17.10.100 of the LMC defines a seismic hazard area as *lands that are underlain by soft or loose saturated soils that are subject to liquefaction settlement or spreading during earthquake induced ground shaking*.

Liquefaction is a phenomenon wherein cohesionless soils below the groundwater table build up excess pore water pressure during earthquake loading. Liquefaction typically occurs in loose, cohesionless soils located below the groundwater table. The potential hazardous impacts of liquefaction include liquefaction-induced settlement and lateral spreading. Groundwater was not encountered in our borings at the time of drilling, however, groundwater measurements taken from the monitoring wells installed in B-3 and B-4 on August 28, 2025 indicated groundwater at approximately 22 and 23.8 feet, respectively. According to the NGA 2024 report, groundwater was not encountered in their test pits. As described previously, soil conditions encountered on site generally consisted of about 2 to 3 feet of loose to medium dense sand and gravel over dense to very dense glacial till soils. According to the Washington State Department of Natural Resources *Washington Geologic Information Portal – Natural Hazards Maps*, the project site is identified as having a “Very Low” liquefaction susceptibility. Based on the soil and groundwater conditions observed, it is our opinion that the risk of liquefaction at the project site is low.

### **3.3 Site Preparation**

#### **3.3.1 Erosion Control Measures**

We expect site preparation will begin with installation of erosion control measures. Stripped surfaces and soil stockpiles are typically a source of runoff sediments. We recommend that silt fences, berms, and/or swales be installed around the downslope side of stripped areas and stockpiles, in order to capture runoff water and sediment. If earthwork occurs during wet weather, we recommend that all stripped surfaces be covered with straw to reduce runoff erosion, whereas soil stockpiles should be protected with anchored plastic sheeting.

#### **3.3.2 Temporary Drainage**

Stripping, excavation, grading, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and provide proper control of erosion. The site should be graded to prevent water from ponding in construction areas and/or flowing into and/or over excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage if inclement weather is forecasted. Accumulated water must be removed from subgrades and work areas immediately and prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely manner.

#### **3.3.3 Clearing, Grubbing and Stripping**

In preparation for grading, we recommend removal of all existing surficial vegetation and deleterious debris such as trash. Following clearing of surficial vegetation and deleterious debris, organic-rich topsoil (soils containing more than 4 percent organic material by weight) should be stripped from any locations that are to receive structural fill, footing and slab subgrades, and the utility trenches. These materials should be wasted away from the proposed development area or removed from the project site. A significant portion of the eastern property is currently occupied by trees, shrubs, and other vegetation. Our borings on the eastern property encountered between 4 to 7 inches of organic rich topsoil. However, based on the nature of the project site and the site history, the thickness of surficial organic material should be expected to vary and may be deeper than observed at the exploration locations.

#### **3.3.4 Subgrade Preparation and Protection**

Once site preparation is complete, all areas that are at design subgrade elevation or areas that will receive new structural fill should be compacted to a firm and unyielding condition if possible. In the event the exposed subgrade becomes unstable, yielding, or unable to be compacted due to high moisture conditions, we recommend that the materials be over-excavated and replaced with structural fill compacted in accordance with the recommendations detailed in the *Structural Fill Materials, Placement, and Compaction* section of this report.

#### **3.3.5 Freezing Conditions**

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be compacted prior to placing subsequent lifts of structural fill or pouring foundations. Alternatively,

the frozen material could be stripped from the subgrade to expose unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

### **3.4 Structural Fill Materials, Placement, and Compaction**

Structural fill includes any material placed below foundations, floor slabs, and pavement sections, within utility trenches, and behind retaining walls. Prior to the placement of structural fill, all surfaces to receive fill should be prepared as previously recommended in the Site Preparation section of this report.

#### **3.4.1 Re-Use of Site Soils as Structural Fill**

The suitability for re-use of site soils as structural fill depends on the composition and moisture content of the soil. Soils encountered in our explorations generally consisted of gravel and sand with varying fines content (that portion passing the U.S. No. 200 sieve). Based on soil conditions encountered in our borings, we anticipate site soils will be suitable for reuse as structural fill. Grain size analysis results performed on select soil samples between 2½ to 7½ feet below grade, indicated fines contents ranging from about 5 to 53 percent, with an average of 22 percent. As the fines content increases, the soil becomes increasingly sensitive to small changes in moisture content. Soils containing more than about 5 percent fines cannot be consistently compacted to the appropriate levels when the moisture content is more than approximately 2 percent above or below the optimum moisture content (per ASTM D1557). Optimum moisture content is the moisture content which results in the greatest compacted dry density with a specified compactive effort. In situ moisture contents collected from the upper 10 feet ranged from about 2 to 12 percent. As a result, we expect that moisture conditioning (drying or wetting) of site soils during construction may be necessary to achieve moisture contents for compaction, likely dependent on the time of year the work is taking place in.

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

We recommend that site soils used as structural fill have less than 3 percent organics on a dry weight basis, have no woody debris greater than ½-inch in diameter, and contain no other deleterious materials. We recommend that all pieces of organic material greater than ½-inch in diameter be picked out of the fill before it is placed and compacted. Deleterious debris includes waste building materials, organics, trash, and asphalt and, if encountered, it should be removed from the soil prior to its re-use as structural fill.

### **3.4.2 Imported Structural Fill**

Imported structural fill may be required for the project for several uses including backfilling building and utility subgrade excavations, and replacement of unsuitable foundation and slab subgrade soils (if encountered). For these general purpose imported materials, we recommend imported fill meet the following requirements:

- **During Extended Periods of Dry Weather:** Common Borrow Options 1 or 2 per Section 9-03.14(3) of the 2022 WSDOT Standard Specifications (WSDOT Standard Specs.). The on-site soils would be classified as Common Borrow.
- **During Wet Weather:** Gravel Borrow per Section 9-03.14(1) of the WSDOT Standard Specs.

It should be noted that Common Borrow typically contains a significant fraction of fines (silt and clay). During wet weather, Common Borrow will quickly become too wet for reuse as structural fill.

### **3.4.3 Moisture Content**

The suitability of soil for use as structural fill will depend on the prevailing weather at the time of construction, the moisture content of the soil, and the fines content (that portion passing the U.S. No. 200 sieve) of the soil. As the amount of fines increases, the soil becomes increasingly sensitive to small changes in moisture content. Soils containing more than about 5 percent fines cannot be consistently compacted to the appropriate levels when the moisture content is more than approximately 2 percent above or below the optimum moisture content (per ASTM D1557). Optimum moisture content is that moisture content which results in the greatest compacted dry density with a specified compactive effort. The moisture content of fill at the time of placement should be within plus or minus 2 percent of optimum moisture content for compaction as determined by the ASTM D1557 test method.

Imported fills should be delivered to the site at a moisture content within  $\pm 2$  percent of optimum. This is important as there will be minimal room on this site to moisture condition imported fill materials should they arrive at the site wet or dry of optimum. In order to reduce the risk of wet or dry imported soils, we recommend proposed borrow sources be investigated and tested by ZGA prior to importing them to the site.

### **3.4.4 Fill Placement and Compaction**

We recommend that structural fill be placed in horizontal lifts of a thickness that can be compacted to the recommended levels with the equipment used (typically 8 to 15 inches).

Our recommendations for soil compaction are summarized in the following table. Structural fill for roadways and utility trenches in municipal rights-of-way should be placed and compacted in accordance with Lynnwood codes and standards. We recommend that a geotechnical engineer be present during grading so that an adequate number of density tests may be conducted as structural fill placement occurs.

<b>Table 4: Recommended Compaction Levels</b>	
<b>Location</b>	<b>Minimum Percent Compaction*</b>
All fill below building floor slabs and foundations	95
Upper 2 feet of fill below exterior slabs and pavements	95
Pavement and exterior slab fill below two feet	92
Retaining wall backfill less than 2 feet from wall	90 – 92
Retaining wall backfill greater than 2 feet from wall	95
Upper two feet of utility trench backfill	95
Utility trenches below two feet	92
Landscape areas	90
* <i>ASTM D1557 Modified Proctor Maximum Dry Density</i>	

### 3.5 Temporary and Permanent Slopes

Temporary excavations are expected to construct the building and for installation of underground utilities. Temporary excavation slope stability is a function of many factors, including:

- 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; and
- The length of time the excavation remains open.

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

Based upon our review of WAC Chapter 296-155-66401 (Appendix A – Soil Classification), we have interpreted the soils disclosed by the explorations and likely to be present in most excavations as consistent with the Type B 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.

We recommend permanent cut and fill slopes be inclined no steeper than 2H:1V.

### **3.6 Construction Dewatering**

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

### **3.7 IBC Seismic Design Considerations.**

For development of IBC seismic ground motion parameters, we recommend Site Class D. Based on our review of geologic mapping, the nearest known or suspected active fault is a fault trace from the southern Whidbey Island fault zone, located approximately ¼-mile northeast of the project site.

### **3.8 Shallow Foundation Recommendations**

Based on the soil conditions encountered on site and our analyses, conventional, shallow, spread footings appear feasible for support of building foundation loads provided that the foundation subgrades are prepared in accordance with this report. Based on the plans provided to us at the time of this report, we understand finished floor elevations are anticipated to be at elevation 360 feet for the daylight basement level and 372 feet for the first-floor elevation. Given the varying site topography, finished floor elevations, and soil conditions encountered in our explorations, we anticipate that bottom of footing elevations will be located either above or below the contact with high density ESU 2 soils depending on their location on site. We recommend that footings bear directly on dense to very dense ESU-2 soils or on compacted structural fill extending down to dense to very dense ESU-2 soils. ESU 2 soils were encountered at depths of 2 to 3 feet below existing grade, at elevations ranging from 357 to 367-½ feet. Refer to Table 1 and the attached boring logs for specific contact elevations at each boring location. These soils should provide adequate support for the anticipated foundation loads provided foundation subgrades are prepared in accordance with this report. Our recommendations for shallow spread footings are provided below.

#### **3.8.1 Foundation Subgrade Preparation**

We recommend footings bear directly on dense to very dense ESU 2 soils or compacted structural fill extending down to dense to very dense ESU 2 soils. For foundations bearing on structural fill, we recommend that all fill is placed and compacted in accordance with the *Structural Fill* section of this report. New fill placed below shallow foundations should extend laterally beyond the edges of the foundations a distance of 1.5 feet times the new fill thickness. Excavations required to remove unsuitable soils will need to consider the recommended lateral extents of new fill.

The base of all foundation excavations should be free of water, loose soil, or debris prior to placing concrete, and should be compacted as recommended in this report. Concrete should be placed soon after excavating and compaction 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.

### 3.8.2 Shallow Foundation Geotechnical Design Parameters Recommendations

As described above, we anticipate foundation subgrades will consist of a combination of dense to very dense native ESU 2 soils or compacted structural fill extending down to dense to very dense ESU 2 soils. We have provided two allowable bearing pressures, one for each anticipated foundation subgrade condition. Geotechnical design parameter recommendations for shallow foundations are provided in Table 5 below.

<b>Table 5: Shallow Foundation Geotechnical Design Parameters</b>	
<b>Parameter</b>	<b>Recommended Value</b>
Net Allowable Bearing Pressure (Structural fill extending to dense ESU 2 soils)	2,500 psf
Net Allowable Bearing Pressure (Dense to very dense ESU 2 soils)	5,000 psf
Minimum Footing Dimensions	Per IBC
Minimum Embedment Depth for Frost Protection	18 in
Estimated Total Settlement	1 in
Estimated Differential Settlement	½ total settlement over 40 ft
Ultimate Passive Resistance <sup>1</sup>	490 pcf
Ultimate Coefficient of Base Friction <sup>1</sup>	0.47

<sup>1</sup>Does not include a safety factor.

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

## 3.9 Backfilled Retaining Walls

### 3.9.1 General

We understand that the project will include backfilled permanent retaining walls. Recommendations for backfilled retaining walls are provided below.

### 3.9.2 Lateral Earth Pressures

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

free to translate or rotate) should be designed for at-rest earth pressures. Non-yielding walls include walls that are braced to another wall or structure, and wall corners.

Assuming that walls are backfilled and drained in accordance with this report, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 33 pcf (active earth pressure). Non-yielding walls should be designed using an equivalent fluid density of 52 pcf (at-rest earth pressure).

The above-recommended lateral earth pressures assume no buildup of hydrostatic pressure behind the wall. If groundwater is allowed to saturate the backfill soils, hydrostatic pressures will act against a retaining wall; however, if the recommended drainage system is included with each retaining wall, we do not expect that hydrostatic pressures will develop.

Surcharge pressures due to sloping backfill, adjacent footings, vehicles, construction equipment, etc. must be added to these lateral earth pressure values. For traffic loads, we recommend adding a uniform lateral earth pressure of 70 psf for yielding walls and 110 psf for non-yielding walls to the earth pressures recommended above. For other surcharge loads such as sloping backfill or adjacent footings, we should be consulted to provide additional recommendations.

Building foundation walls with an exposed height greater than 6 ft should also be designed to resist incremental additional lateral earth pressures from seismic shaking during the design earthquake event. For level backfill conditions, we recommend a uniformly distributed seismic earth pressure of  $9H$ , where  $H$  is the height of the wall, be applied to the walls.

### **3.9.3 Bearing, Lateral Resistance, & Footing Embedment Depths**

For recommended bearing capacities and lateral resistance parameters, refer to the shallow foundations section above for design purposes. For retaining walls with a level toe slope, the bottom of footings should be located at least 18 inches below finished grade at the bottom of wall. For retaining walls with a sloping ground surface at the toe of the wall, the bottom of footings should be embedded adequately such that the front face of the footing is offset a minimum distance of 4 ft from the face of the toe slope.

### **3.9.4 Retaining Wall Drainage**

Adequate drainage measures must be installed to collect and direct subsurface water away from retaining walls. All backfilled walls should include a drainage aggregate zone extending two feet from the back of the wall for the full height of the wall. The drainage aggregate should consist of material meeting the requirements of WSDOT 9-03.12(2) Gravel Backfill for Walls. A minimum 4-inch diameter, perforated PVC drainpipe should be provided at the base of backfilled walls to collect and direct subsurface water to an appropriate discharge point. We recommend placing a non-woven geotextile, such as Mirafi 140N, or equivalent, around the free draining backfill material. Wall drainage systems should be independent of other drainage systems such as roof drains.

### **3.10 Stormwater Detention Vault**

We understand that the proposed project will include installation of a new stormwater detention vault located on the southeastern portion of the site, below the paved access driveway. Plans provided to us at the time of this report indicate that the bottom of vault elevation will range from 347.5 feet on the sides of the vault, sloping down to 347 feet along the centerline of the vault, with a top elevation of 356 feet. Bottom of vault foundation type and elevation were not available at the time of this report. We understand the inside dimensions of the vault will be a width of 20 feet (East to west) and a length of 160 feet (north to south). Based on our analysis, conventional spread footings and a base slab will provide adequate support for the vault. We anticipate that foundation subgrade soils will generally consist of native dense to very dense ESU 2 soils. For geotechnical design parameters for foundations refer to the *Shallow Foundation Recommendations* section of this report. For lateral earth pressure recommendations refer to the *Backfilled Retaining Walls* section of this report.

Adequate drainage measures must be installed to collect and direct subsurface water away from the vault walls. We recommend that vault walls include a drainage aggregate zone extending at least 12 inches from the back of wall for the full height of the wall. The drainage aggregate should consist of material meeting the requirements of WSDOT 9-03.12(2) Gravel Backfill for Walls. A minimum 4-inch diameter perforated PVC drainpipe should be provided at the base of the backfilled walls to collect and direct subsurface water to an appropriate discharge point. The drainage aggregate zone should be separated from the onsite soils and adjacent structural fill using a non-woven geotextile such as Mirafi 140N, or equivalent. The foundation/wall drain should discharge independently to an appropriate discharge location.

#### **3.10.1 Surcharge Loading from East Building Line on Stormwater vault**

As currently planned, the east building line is located in close proximity (~7 to 8 ft) to the proposed underground stormwater detention vault. As currently configured, building foundations and planned fills/retaining walls proposed along the east building line will impose a surcharge on the west wall of the detention vault therefore increasing lateral earth pressures. There may be alternatives to reduce or eliminate surcharging on the vault wall. Once structural design commences, we recommend the project structural engineer consult with ZGA to evaluate surcharge impacts and potential alternatives to reduce or eliminate surcharge loading on the stormwater vault west wall.

### **3.11 East Site Retaining Wall**

The project is expected to include a retaining wall along the east property line to support fill used to raise site grades. The retaining wall is expected to consist of a contractor-designed mechanically stabilized earth (MSE) wall with a maximum exposed height of about 12 ft. Recommended design parameters for the east MSE wall are provided below.

With the exception of the southern 70 ft of the wall, we recommend the wall have a minimum embedment depth of 18 inches. The existing ground surface east of the southern 70 ft of the wall slopes steeply

downward to the east (the toe slope). For this section of the wall, we recommend the wall be embedded such that the face of wall at the bottom of the wall is offset a minimum distance of 4 ft from the face of the toe slope.

<b>East MSE Wall Recommended Design Parameters</b>	
<b>Parameter</b>	<b>Recommended Value</b>
Compacted Granular Fill within the Reinforcement Zone <sup>1</sup>	
Total Unit Weight, $\gamma$ (pcf)	130
Friction Angle, $\Phi$	38
Cohesion, C (psf)	0
Retained Soil <sup>2</sup>	
Total Unit Weight, $\gamma$ (pcf)	125
Friction Angle, $\Phi$	34
Cohesion, C (psf)	0
Foundation Soils	
Total Unit Weight, $\gamma$ (pcf)	125
Friction Angle, $\Phi$	34
Cohesion, C (psf)	0
Design Peak Horizontal Ground Acceleration, $A_s$ (g)	0.44
Horizontal Seismic Coefficient, $k_h$ (g)	0.22
Minimum Reinforcement Length (ft)	0.7H or 6 feet, whichever is greater where H is the total wall height (exposed + embedded)
Minimum Reinforcement Long Term Design Strength (lbs/ft)	2,230

<sup>1</sup>Assumes fill meeting the requirements of Gravel Backfill for Structural Earth Wall per Section 9-03.14(4) of the WSDOT Standard Specifications.

<sup>2</sup>Assumes site soils placed and compacted in accordance with the recommendations in this report.

We recommend submittals for contractor-designed MSE walls meet the requirements of Section 6-13 of the 2025 WSDOT Standard Specifications.

### **3.12 On-Grade Concrete Slabs**

The following sections provide recommendations for on-grade concrete slabs.

#### **3.12.1 Subgrade Preparation**

After removal of organic material and other items noted in the Site Preparation section of this report, we recommend that at least the upper 12 inches of material below the slab base be moisture conditioned and compacted to a firm and unyielding condition and to a minimum 95 percent of the modified Proctor maximum dry density per ASTM D1557.

### **3.12.2 Capillary Break**

To provide a capillary break and uniform slab bearing surface, we recommend on-grade slabs be underlain by a minimum 4-inch-thick layer of compacted, crushed rock meeting the requirements of WSDOT Standard Specification Section 9-03.9(3), Crushed Surfacing Top Course, with the modification of a maximum of 7 percent passing the U.S. No. 200 sieve. Alternatively, clean angular gravel such as No. 7 aggregate per WSDOT 9-03.1(4)C could be used for this purpose. Alternative capillary break materials should be submitted to the geotechnical engineer for review and approval before use.

### **3.12.3 Vapor Barrier**

From a geotechnical perspective, a vapor barrier is not necessary beneath a slab-on-grade floor unless moisture sensitive floor coverings and/or adhesives are used. If a vapor barrier is used, we recommend using a minimum 15-mil thick, puncture-resistant, proprietary product such as Stego Wrap, or an approved equivalent that is classified as a Class A vapor retarder in accordance with ASTM E 1745. Overlap lengths and the appropriate tape used to seal the laps should be in accordance with the vapor retarder manufacturer's recommendations. When conditions warrant the use of a vapor retarder, the slab designer and slab contractor should refer to ACI 302 and ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder/barrier.

## **3.13 Drainage Considerations**

### **3.13.1 Surface Drainage**

Final site grades should be sloped to carry surface water away from buildings and other drainage-sensitive areas. Additionally, site grades should be designed such that concentrated runoff on softscape surfaces is avoided. Any surface runoff directed towards softscaped slopes should be collected at the top of the slope and routed to the bottom of the slope and discharged in a manner that prevents erosion.

### **3.13.2 Building Perimeter Foundation Drains**

We recommend a footing drain system be installed around the perimeter of the building. Footing drains should consist of a minimum 4-inch diameter, Schedule 40, rigid, perforated PVC pipe placed at the base of the heel of the footing with the perforations facing down. The pipe should be surrounded by a minimum of 6 inches of clean free-draining granular material conforming to 2024 WSDOT Standard Specification 9-03.12(4), Gravel Backfill for Drains. A non-woven filter fabric such as Mirafi 140N, or equivalent, should envelope the free-draining granular material. At appropriate intervals such that water backup does not occur, the drainpipe should be connected to a tightline system leading to a suitable discharge. Cleanouts should be provided for future maintenance. The tightline system must be separate from the roof drain system. Roof drains should be connected to a separate solid wall PVC tightline system and routed to a suitable discharge location.

## **3.14 Stormwater Infiltration Feasibility**

We understand that new stormwater facilities will be installed as part of the project. As part of our services, we performed an initial evaluation of onsite stormwater infiltration feasibility. Section 13.40.055

of the LMC indicates that the City of Lynnwood adopts the Stormwater Management Manual for Western Washington, prepared by the Washington State Department of Ecology, therefore our evaluation was performed in accordance with the 2024 Stormwater Management Manual for Western Washington (*Manual*). The results of our feasibility evaluation are presented below.

Our explorations performed on site encountered glacial till soils (ESU 2) at shallow depths across the site. As described previously, glacial till soils are overridden during glacial advance, and therefore are considered glacially consolidated.

Per Volume V-5.4 of the *Manual*, infiltration rate determination within glacially consolidated soils must use in-situ field measurements via small or large pilot infiltration testing (PIT). The *Manual* only allows infiltration rate determination via the mechanical grain size testing for soils which are unconsolidated by glacial advance. Given the requirements outlined in Volume V-5.4 of the *Manual*, determination of the design infiltration rate of onsite soils will require small- or large-scale PITs. General considerations regarding on site stormwater infiltration are presented below.

Glacial till soils (ESU 2) typically have low to very low permeability due to their relatively high fines content and high density. Glacial till typically consists of a medium dense to dense weathered horizon underlain by a higher density, unweathered horizon. Stormwater infiltration into unweathered glacial till is generally not feasible in many cases because of the soil's low permeability, although in some cases even a very low infiltration rate may be adequate given specific site circumstances and design requirements. Infiltration into the less dense weathered glacial till, albeit at low rates, can be feasible in some situations. Glacial till soils encountered onsite included relatively high sand and gravel content, therefore infiltration into these soils may be feasible depending on the location and the invert depth of the infiltration facility. We recommend infiltration feasibility be further evaluated as the project design progresses.

### **3.15 Asphalt and Concrete Pavement Recommendations**

We understand the proposed project will include asphalt-paved driveways and parking stalls. Pavement subgrade and section recommendations are summarized below.

It should be noted that asphalt pavements are not maintenance-free. The following pavement sections represent our minimum recommendations for an average level of performance during a 20-year design life; therefore, an average level of maintenance will likely be required. Thicker asphalt, base, and subbase courses would offer better long-term performance, but would cost more initially. Conversely, thinner courses would be more susceptible to "alligator" cracking and other failure modes. As such, pavement design can be considered a compromise between a high initial cost and low maintenance costs versus a low initial cost and higher maintenance costs.

Pavement subgrade soils are anticipated to consist of native sand with varying silt content to sandy gravel with a variable silt content. These soils were observed to be medium dense to very dense in nature. Our

analysis assumes the pavement section subgrade will have a minimum California Bearing Ratio (CBR) value of 10.

For light duty pavements including parking stalls, we recommend a flexible pavement section consisting of a minimum of 2 inches of asphalt over 6 inches (compacted thickness) of crushed surfacing base course. For heavy duty pavement sections along major drive aisles and site entrances, we recommend a pavement section consisting of 3 inches of asphalt over 6 inches (compacted thickness) of crushed surfacing base course. These pavement sections assumes that the upper 12 inches of subgrade soil will be prepared as recommended below. Washington State Department of Transportation (WSDOT) specification codes cited herein refer to the 2024 edition of the WSDOT *Standard Specifications*.

- Subgrade preparation and compaction: We recommend that subgrade preparation comply with WSDOT Specification 2-03.3(14)C Compacting Earth Embankments – Method C. If portions of the pavement subgrade soil cannot be compacted to the required level, the material should be removed to a depth of 12 inches and replaced with Crushed surfacing Base Course compacted in accordance with WSDOT Specification 2-03.3(14)C Compacting Earth Embankments – Method C.
- HMA: We recommend that HMA conform to Section 9-02.1(4) for PG 58S-22 or PG 58H-22 Performance Graded Asphalt Binder as presented in the WSDOT Standard Specifications. We also recommend that the gradation of the asphalt aggregate conform to the aggregate gradation control points for a ½-inch mix as presented in WSDOT Specification 9-03.8(6), HMA Proportions of Materials.
- Crushed Surfacing Top Course: We recommend that Crushed Surfacing Base Course conform to Section 9-03.9(3) of the WSDOT Standard Specifications.
- Compaction and Paving: All base material should be compacted in accordance with WSDOT Specification 2-03.3(14)C Compacting Earth Embankments – Method C. We recommend that HMA be compacted to a minimum of 92 percent and a maximum of 96 percent of the theoretical maximum density. Placement and compaction of HMA should conform to the requirements of Section 5-04 of the WSDOT Standard Specifications. This includes weather limitations as specified in Section 5-04.3(1) and maximum nominal asphalt lift thickness as specified in Section 5-04.3(7), Table 6.

#### **4.0 CLOSURE**

The analysis and recommendations presented in this report are based, in part, on the explorations completed for this study. The number, location, and depth of the explorations were completed within the constraints of budget and site access so as to yield the information to formulate our recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend that ZGA be provided an opportunity to review the final plans and specifications

when they become available in order to assess that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design.

The performance of earthwork, structural fill, foundations, and temporary shoring depend greatly on proper site preparation and construction procedures. We recommend that Zipper Geo Associates, LLC be retained to provide geotechnical engineering services during the earthwork-related construction phases of the project. If variations in subsurface conditions are observed at that time, a qualified geotechnical engineer could provide additional geotechnical recommendations to the contractor and design team in a timely manner as the project construction progresses.

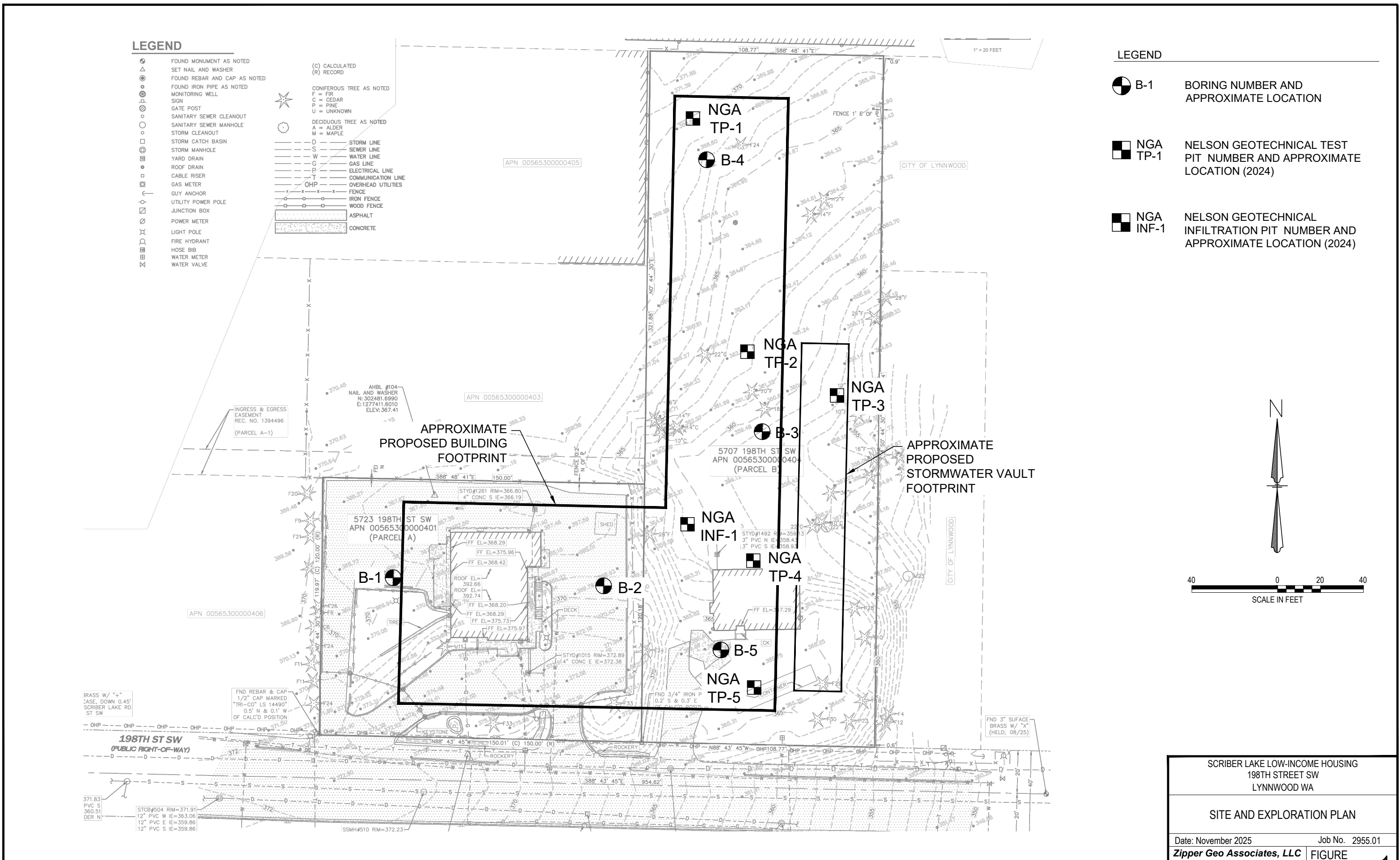
This report has been prepared for the exclusive use of Blackfish Capital, and their agents, for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made.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

**LEGEND**

- ⊕ FOUND MONUMENT AS NOTED
  - △ SET NAIL AND WASHER
  - ⊙ FOUND REBAR AND CAP AS NOTED
  - ⊙ FOUND IRON PIPE AS NOTED
  - ⊙ MONITORING WELL
  - ⊙ SIGN
  - ⊙ GATE POST
  - ⊙ SANITARY SEWER CLEANOUT
  - ⊙ SANITARY SEWER MANHOLE
  - ⊙ STORM CLEANOUT
  - ⊙ STORM CATCH BASIN
  - ⊙ STORM MANHOLE
  - ⊙ YARD DRAIN
  - ⊙ ROOF DRAIN
  - ⊙ CABLE RISER
  - ⊙ GAS METER
  - ⊙ GUY ANCHOR
  - ⊙ UTILITY POWER POLE
  - ⊙ JUNCTION BOX
  - ⊙ POWER METER
  - ⊙ LIGHT POLE
  - ⊙ FIRE HYDRANT
  - ⊙ HOSE BIB
  - ⊙ WATER METER
  - ⊙ WATER VALVE
- (C) CALCULATED RECORD
  - (R) RECORD
  - CONIFEROUS TREE AS NOTED
    - F = FIR
    - C = CEDAR
    - P = PINE
    - U = UNKNOWN
  - DECIDUOUS TREE AS NOTED
    - A = ALDER
    - M = MAPLE
  - D --- STORM LINE
  - S --- SEWER LINE
  - W --- WATER LINE
  - G --- GAS LINE
  - E --- ELECTRICAL LINE
  - T --- COMMUNICATION LINE
  - OHP --- OVERHEAD UTILITIES
  - FENCE
  - IRON FENCE
  - WOOD FENCE
  - ASPHALT
  - CONCRETE

**LEGEND**

- ⊙ B-1 BORING NUMBER AND APPROXIMATE LOCATION
- ⊙ NGA TP-1 NELSON GEOTECHNICAL TEST PIT NUMBER AND APPROXIMATE LOCATION (2024)
- ⊙ NGA INF-1 NELSON GEOTECHNICAL INFILTRATION PIT NUMBER AND APPROXIMATE LOCATION (2024)



REFERENCE: BOUNDARY AND TOPOGRAPHIC SURVEY, AHBL, SEPTEMBER 3, 2025.

SCRIBER LAKE LOW-INCOME HOUSING 198TH STREET SW LYNNWOOD WA	
SITE AND EXPLORATION PLAN	
Date: November 2025	Job No. 2955.01
Zipper Geo Associates, LLC 19019 36th Ave. W., Suite E Lynnwood, WA	FIGURE SHT.1 of 1

**APPENDIX A**  
**SUBSURFACE EXPLORATION PROCEDURES AND LOGS**

## APPENDIX A SUBSURFACE EXPLORATION PROCEDURES AND LOGS

### **Subsurface Exploration Description**

Our field exploration for this project included advancing four five soil borings on August 22 and 23, 2025. The approximate exploration locations are shown on the enclosed Site and Exploration Plan, Figure 1. The exploration locations were determined by measuring distances from existing site features and structures with a fiberglass tape. Ground surface elevations for the exploratory boring locations were interpolated from the *Boundary and Topographic Survey*, prepared by AHBL, dated September 3, 2025. As such, the exploration locations should be considered accurate only to the degree implied by the means and methods used to define them. The following sections describe our procedures associated with the explorations. Descriptive logs of the explorations are enclosed in this appendix.

### Boring Procedures

The borings were advanced using a track-mounted drill rig operated by an independent drilling company (Boretac 1) working under subcontract to ZGA. The borings were advanced using hollow stem auger drilling methods. A ZGA geologist continuously observed the borings, logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further evaluation and testing. Samples were generally obtained by means of the Standard Penetration Test at 2.5-foot intervals throughout the drilling operation.

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

**APPENDIX B**  
**LABORATORY TESTING PROCEDURES AND RESULTS**

# BORING LOG: B-1

# B-1

Logged By: MRC  
 Location: See figure 1.  
 Elevation: 368 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA  
 Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing
			No.	Type		▲ NValue 10 20 30 40 50 ● % Fines ( ) 10 20 30 40 50 ○ % Moisture 10 20 30 40 50						
Ground Surface: asphalt <span style="float: right;">EL 368 ft</span>												
0	2- to 4-inches of asphalt over loose to medium dense, moist, brown, SAND with gravel, some silt (Probable fill). (ESU 1)											
2	Very dense, moist, gray, gravelly SAND with silt (Glacial Till) (ESU 2)											
3	Trace cobbles.	S-1	▽					○4				50/4
5		S-2	▽					○5	●30			82
8	Grades to very dense, moist, gray, gravelly SAND, some silt.	S-3	▽					○6				50/6
10	Very dense, moist, gray, SAND with gravel, with silt.	S-4	▽					○8				50/4
14	Very dense, moist, gray, SAND, with gravel, some silt. (Glacial Till) (ESU 2)	S-5	▽					○7				50/4
15												
20								○9				

RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT 3.27.25 / zipper-geo-associates / admin / September 08, 2025 02:59 PM

# BORING LOG: B-1

# B-1

Logged By: MRC  
 Location: See figure 1.  
 Elevation: 368 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA

Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing					
			No.	Type		NValue											
						▲	10	20	30	40	50						
						●	10	20	30	40	50						
						○	10	20	30	40	50						
20	Very dense, moist, gravel, SAND, with gravel, trace silt. (Glacial Till) (ESU 2)	S-6	▽			○ <sub>9</sub>						50/5					
21																	
22																	
23																	
24																	
25	Very dense, most, gray, gravelly SAND.	S-7	▽			○ <sub>4</sub>						50/2					
26																	
27																	
28																	
29																	
30	Boring completed at approximately 31.5 feet bgs. No groundwater observed at time of drilling. Well tag ID No. BQ - 138	S-8	▽			○ <sub>5</sub>						50/3					
31																	
32																	
33																	
34																	

RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT 3.27.25 / zipper-geo-associates / admin / September 08, 2025 02:59 PM

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

PROJECT TITLE:  
 Scriber Lake Low Income Housing

CLIENT:  
 Blackfish Capital

PROJECT NO.:  
 2955.01

SHEET:  
 2 of 2

# BORING LOG: B-2

# B-2

Logged By: MRC  
 Location: See figure 1.  
 Elevation: 369.5 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA  
 Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing	
			No.	Type		▲ NValue	● % Fines (	○ % Moisture	10	20			30
0	Ground Surface: asphalt												
	EL 369.5 ft												
0	2- to 4.5-inches of asphalt over loose to medium dense, moist, light brown, SAND, with gravel, trace silt (Probable Fill). (ESU 1)												
2	2 ft EL 367.5 ft												
2	Dense, damp, gray brown, sandy GRAVEL, some silt. (Glacial Till) (ESU 2)					○2	●9	▲				45	
3			S-1	▽									
4													
5	Very dense, moist, gray brown SAND with gravel to gravelly SAND, some silt.					○3		▲				76	
6			S-2	▽									
7													
8	Grades to SAND with gravel and silt					○5	●27	▲				75	
9			S-3	▽									
10	Gravelly spoils, some cobbles.					○5		▲				50/6	
11			S-4	▽									
12													
13													
14													
15			S-5	▽		○3		▲				50/2	
16													
17													
18													
19													
20						○7							

RSLog / 1ZGA Geotechnical Soil Boring Log CURRENT 3.27.25 / zipper-geo-associates / admin / September 08, 2025 02:59 PM

**BORING LOG: B-2**

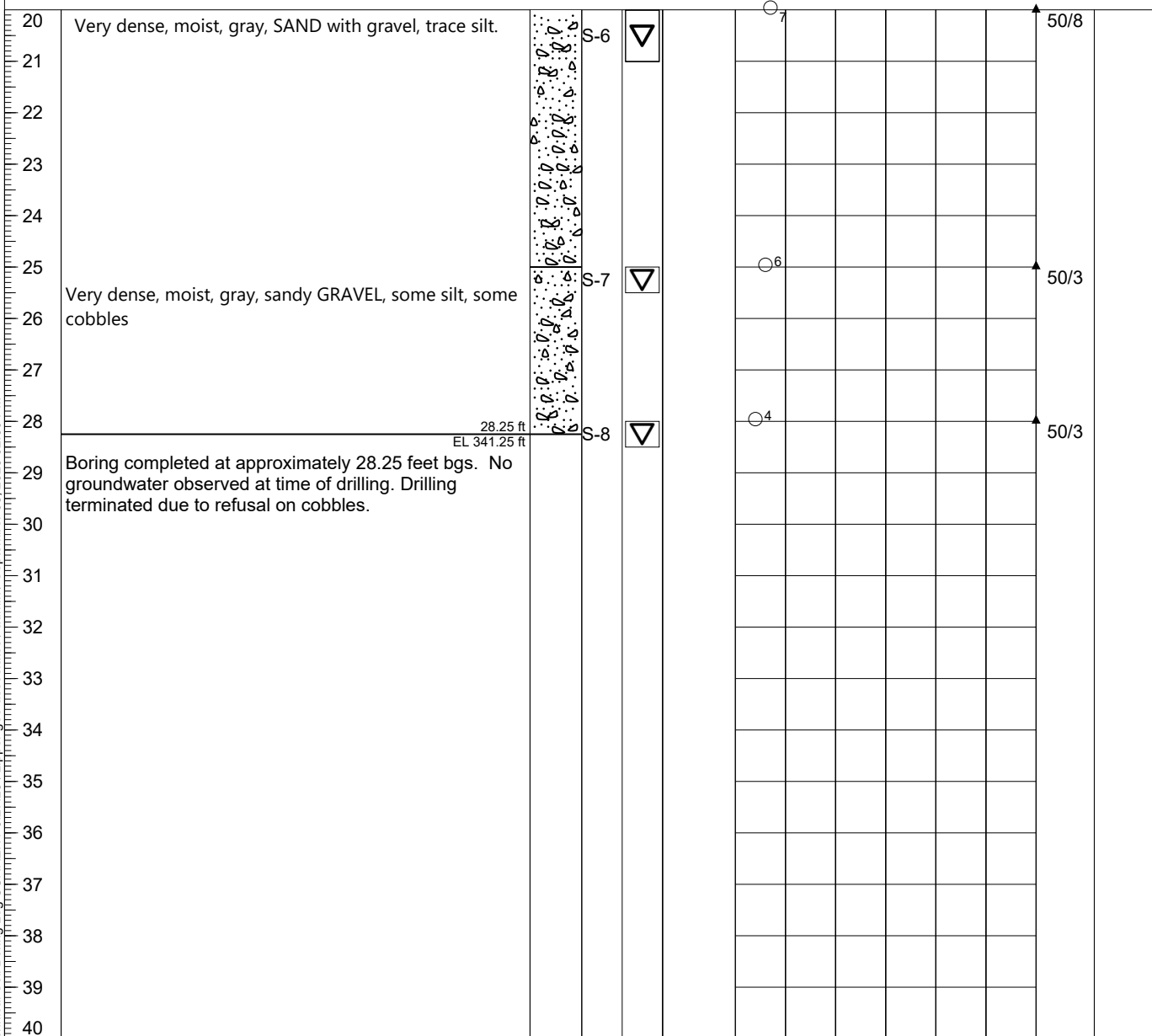
**B-2**

Logged By: MRC  
 Location: See figure 1.  
 Elevation: 369.5 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA  
 Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing
			No.	Type		▲ NValue						
						10	20	30	40	50		
						● % Fines (						
						10	20	30	40	50		
						○ % Moisture						
						10	20	30	40	50		



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

PROJECT TITLE: Scriber Lake Low Income Housing  
 CLIENT: Blackfish Capital

PROJECT NO.: 2955.01  
 SHEET: 2 of 2

# BORING LOG: B-3

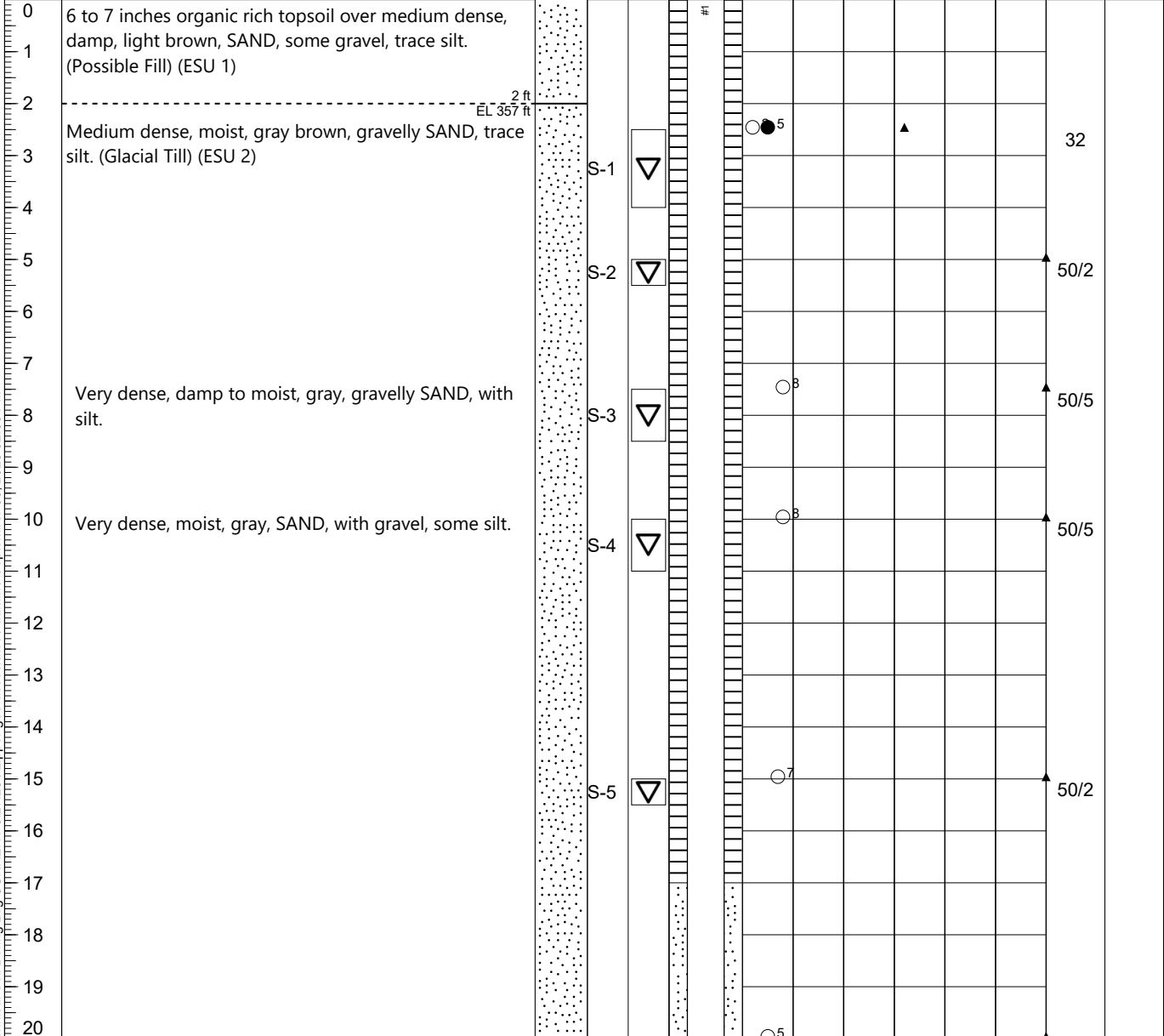
# B-3

Logged By: MRC  
 Location: See figure 1.  
 Elevation: 359 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA  
 Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)  
 Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing
			No.	Type		N Value						
						10	20	30	40	50		
						% Fines (						
					10	20	30	40	50			
					% Moisture							
					10	20	30	40	50			

Ground Surface: grass and blackberries. EL 359 ft



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# BORING LOG: B-3

# B-3

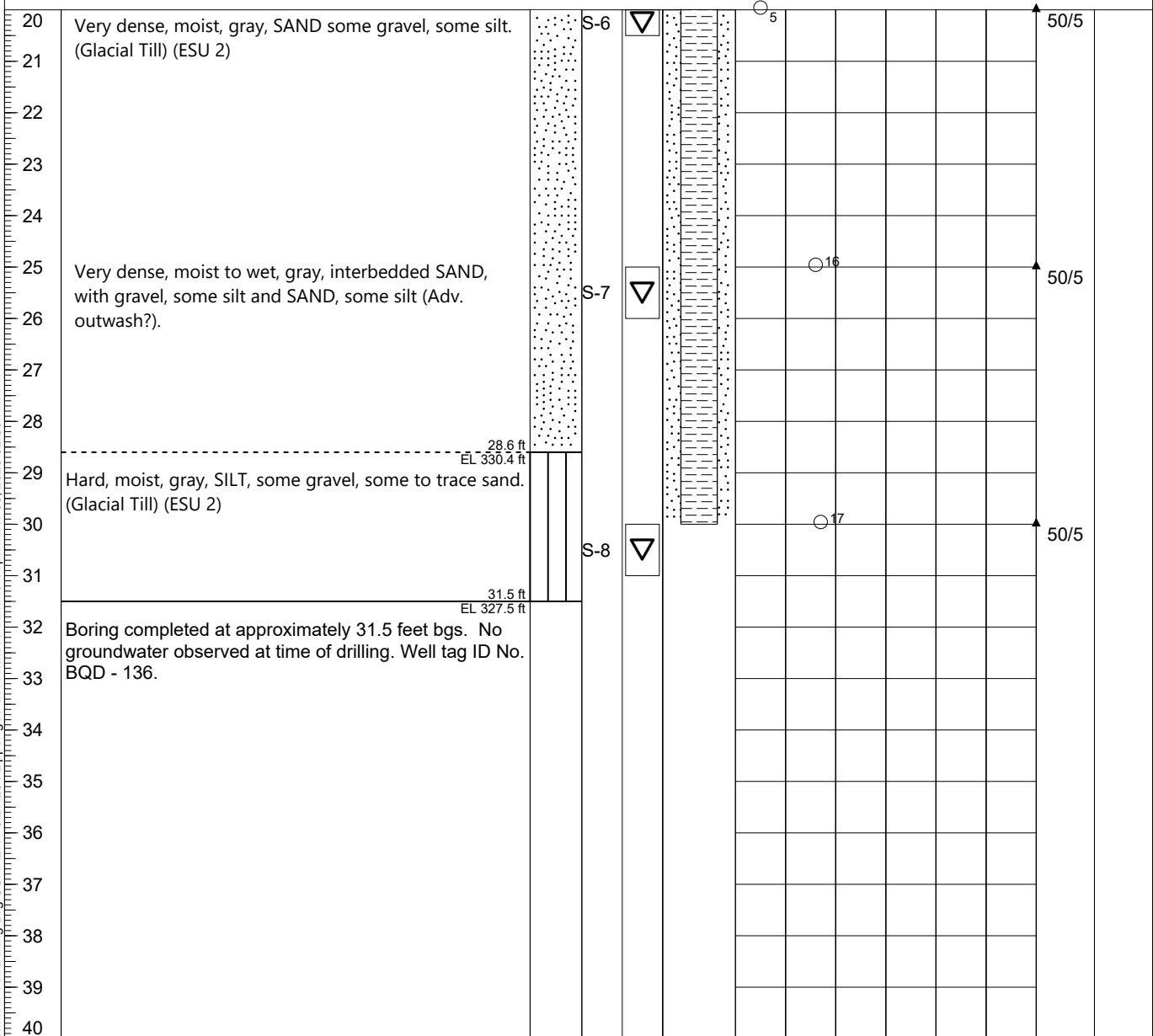
Logged By: MRC  
 Location: See figure 1.  
 Elevation: 359 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA

Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing
			No.	Type		▲ NValue	● % Fines (	○ % Moisture	10	20		



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# BORING LOG: B-4

# B-4

Logged By: MRC  
 Location: See figure 1.  
 Elevation: 367.5 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA  
 Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing
			No.	Type		▲ N Value 10 20 30 40 50 ● % Fines ( ) 10 20 30 40 50 ○ % Moisture 10 20 30 40 50						
Ground Surface: Grass and blackberries <span style="float: right;">EL 367.5 ft</span>												
0	4 to 5 inches of organic rich topsoil over loose, light brown, sandy GRAVEL, some silt (Probable fill) (ESU 1)	▽										
3	Very dense, damp, light brown to gray brown, gravelly SAND, trace silt (Glacial Till) (ESU 2)	▽	S-1								50/3	
5	Medium dense, moist, orange brown, sandy SILT, trace gravel.(Glacial Till) (ESU 2)	▽	S-2								39	
8	Grades to very dense, moist, gray, SAND, with silt, trace gravel.	▽	S-3								50/5	
10	Very dense, moist, interbedded SAND, some gravel, and silty SAND, with fine laminations (ESU 2).	▽	S-4								50/4	
15	Very dense, moist, gray, gravelly SAND, some silt.	▽	S-5								50/5	
20												

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ZipperGeo

Geoprofessional Consultants

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

PROJECT TITLE: Scriber Lake Low Income Housing  
 CLIENT: Blackfish Capital

PROJECT NO.: 2955.01  
 SHEET: 1 of 2

# BORING LOG: B-4

# B-4

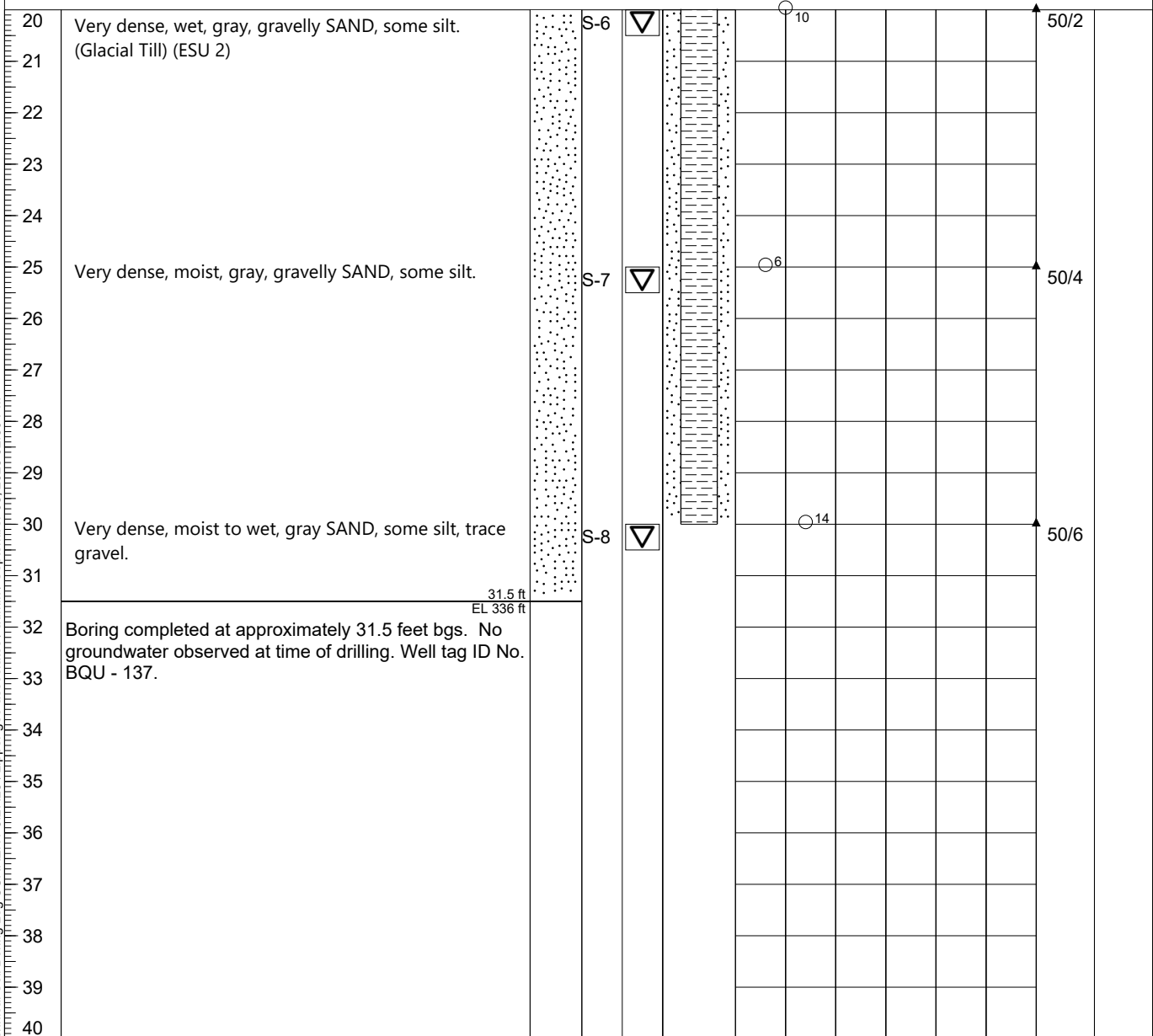
Logged By: MRC  
 Location: See figure 1.  
 Elevation: 367.5 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA

Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing	
			No.	Type		N Value							
						▲	10	20	30	40	50		
						●	10	20	30	40	50		
						○	10	20	30	40	50		



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# BORING LOG: B-5

# B-5

Logged By: MRC  
 Location: See figure 1.  
 Elevation: 365.5 ft  
 Drill Date: 08-22-2025  
 Reviewer: MRC

Project Address: 5707 & 5723 198th Street  
 SW, Lynnwood WA  
 Drilling Company: Boretect1, Inc  
 Drill Rig: HSA EC95 Track Rig  
 Drilling Method: Auger (Hollow-Stem)

Borehole Diameter: 8 inch  
 Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples		Piezometer / Well	Standard Penetration Test					SPT Number (N)	Testing	
			No.	Type		▲ NValue	● % Fines (	○ % Moisture	10	20			30
Ground Surface: organics <span style="float: right;">EL 365.5 ft</span>													
0	Medium dense, moist, light brown, SAND with gravel, trace silt (Probable Fill) (ESU 1)	[Symbol]											
4	Very dense, moist, gray, SAND with gravel, some silt. (Glacial Till) (ESU 2)	[Symbol]											
5					S-1	▽	○ 5					50/3	
10					S-2	▽	○ 7					50/4	
10.3	Boring completed at approximately 10.3 feet bgs. No groundwater observed at time of drilling.												

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## LABORATORY TESTING PROCEDURES

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

### **Visual Classification**

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

### **Moisture Content Determinations**

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

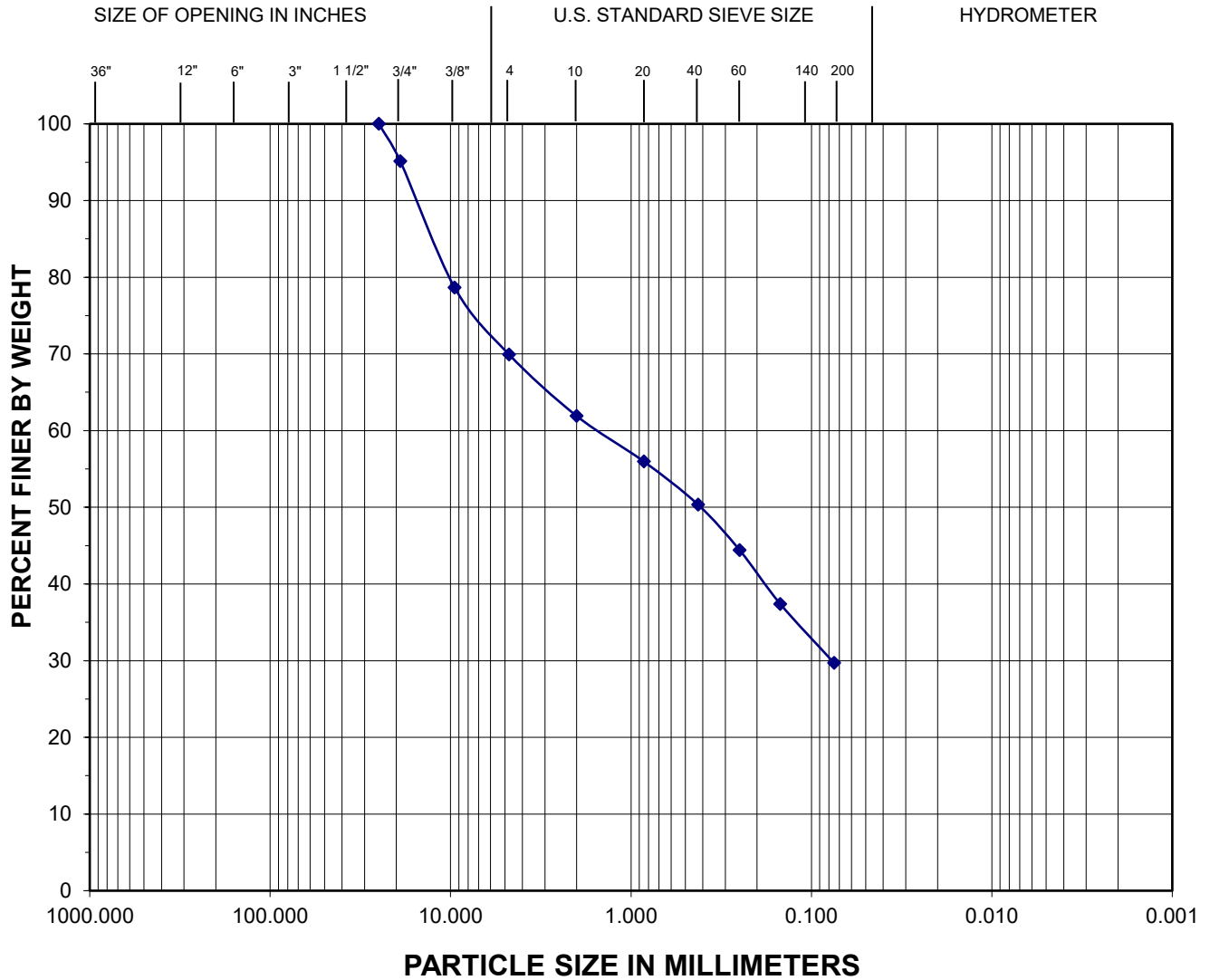
### **Grain Size Analysis**

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

# GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

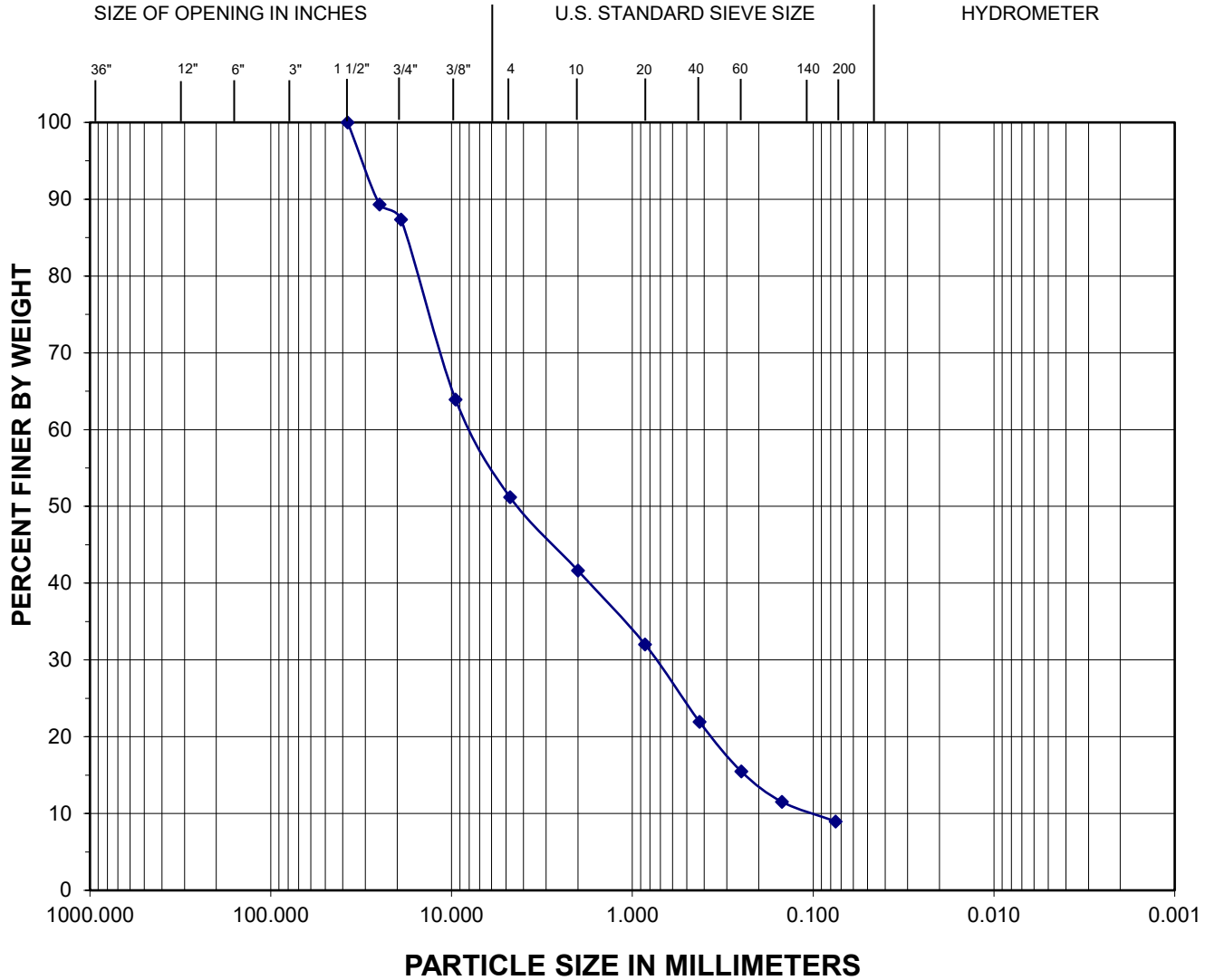
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-2	5	5.2	29.7	gravelly SAND, with silt

<b>Zipper Geo Associates, LLC</b> Geotechnical and Environmental Consultants	PROJECT NO: 2955.01	PROJECT NAME:
	DATE OF TESTING: 9/3/2025	Scriber Lake Low-Income Housing

# GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

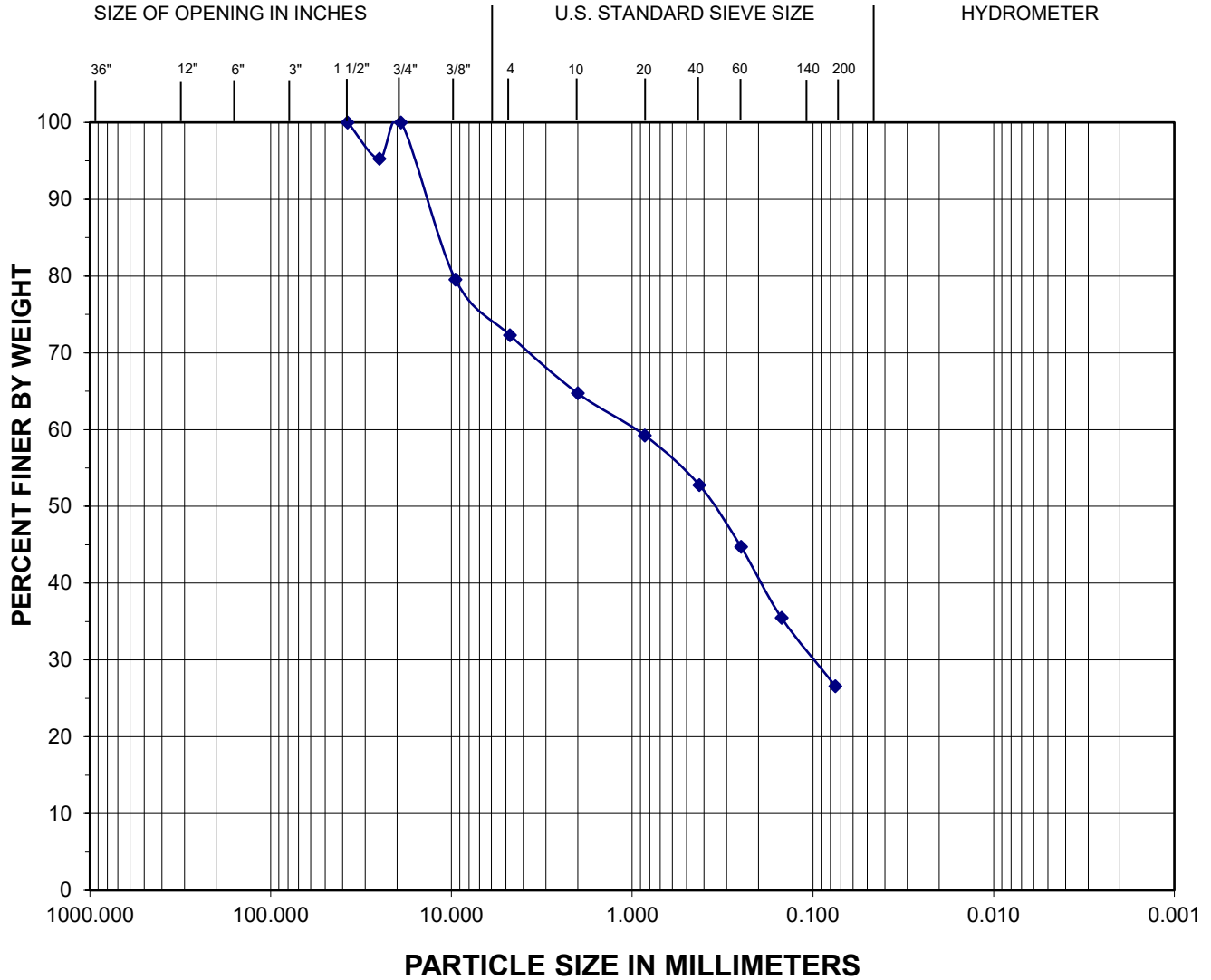
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-1	2.5	1.6	9.0	sandy GRAVEL, some silt

<b>Zipper Geo Associates, LLC</b> Geotechnical and Environmental Consultants	PROJECT NO: 2955.01	PROJECT NAME:
	DATE OF TESTING: 9/3/2025	Scriber Lake Low-Income Housing

# GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

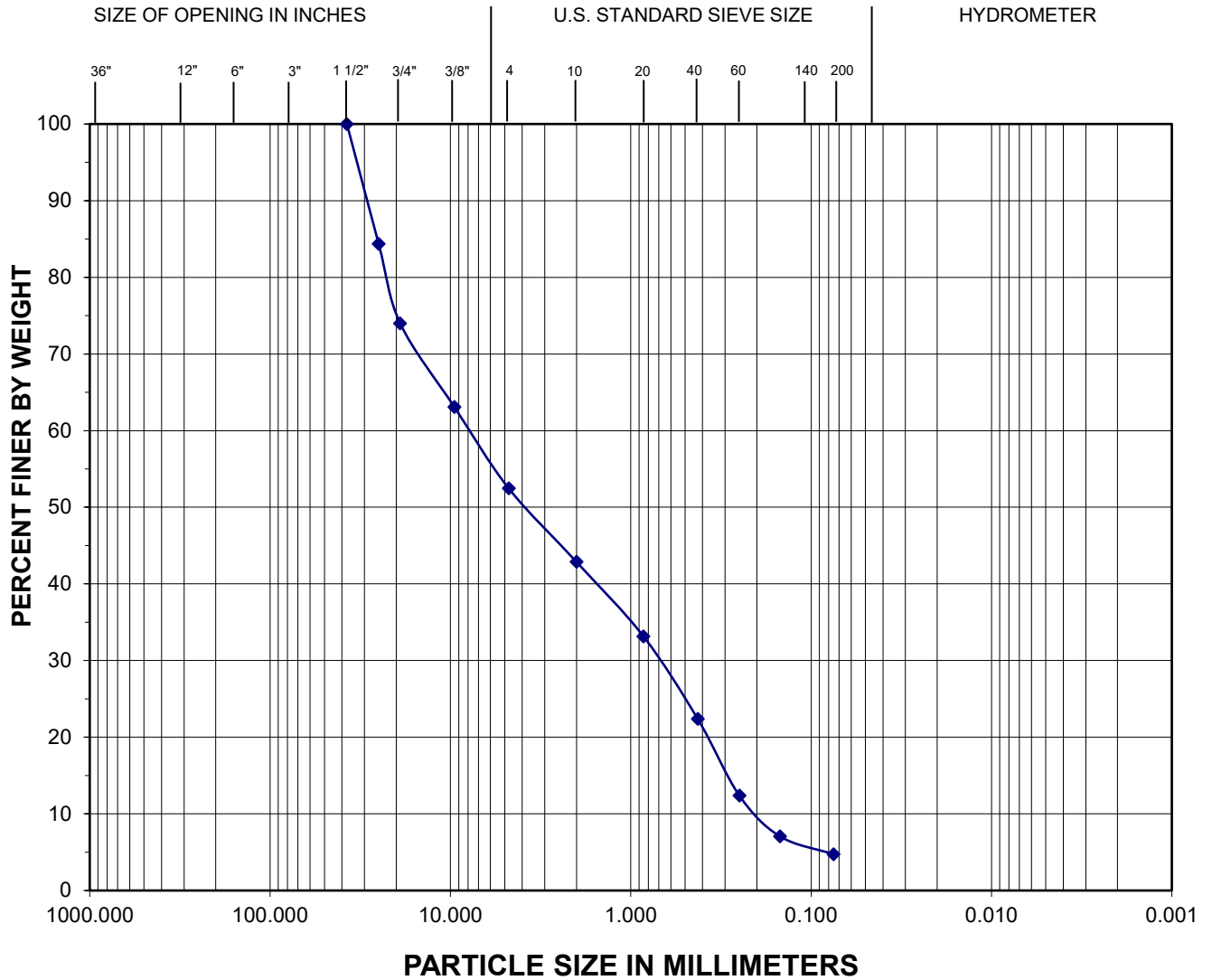
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-3	7.5	4.9	26.6	SAND with gravel and silt

<b>Zipper Geo Associates, LLC</b> Geotechnical and Environmental Consultants	PROJECT NO: 2955.01	PROJECT NAME:
	DATE OF TESTING: 9/3/2025	Scriber Lake Low-Income Housing

# GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

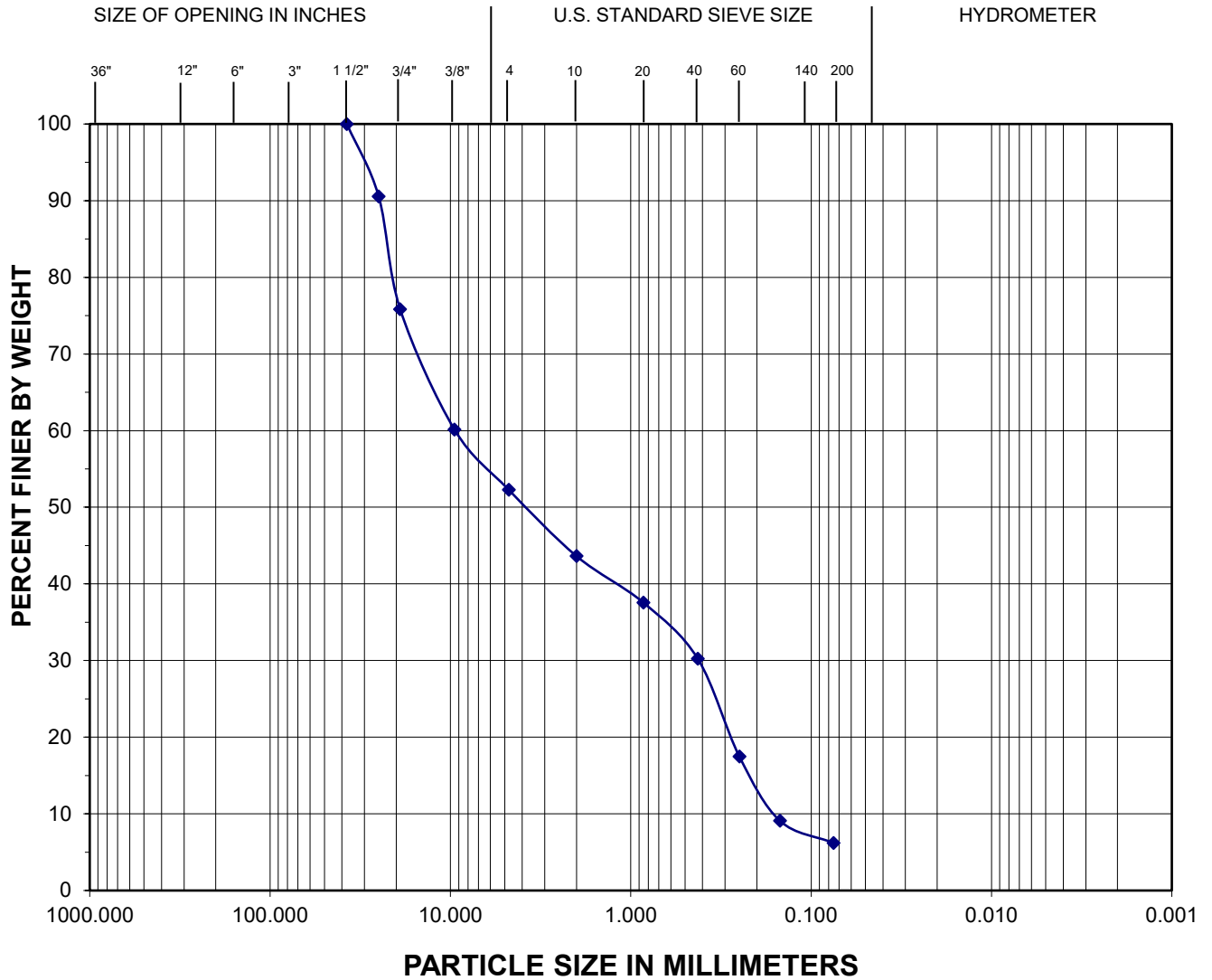
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-3	S-1	2.5	2.3	4.7	gravelly SAND, trace silt

<b>Zipper Geo Associates, LLC</b> Geotechnical and Environmental Consultants	PROJECT NO: 2955.01	PROJECT NAME:
	DATE OF TESTING: 9/3/2025	Scriber Lake Low-Income Housing

# GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

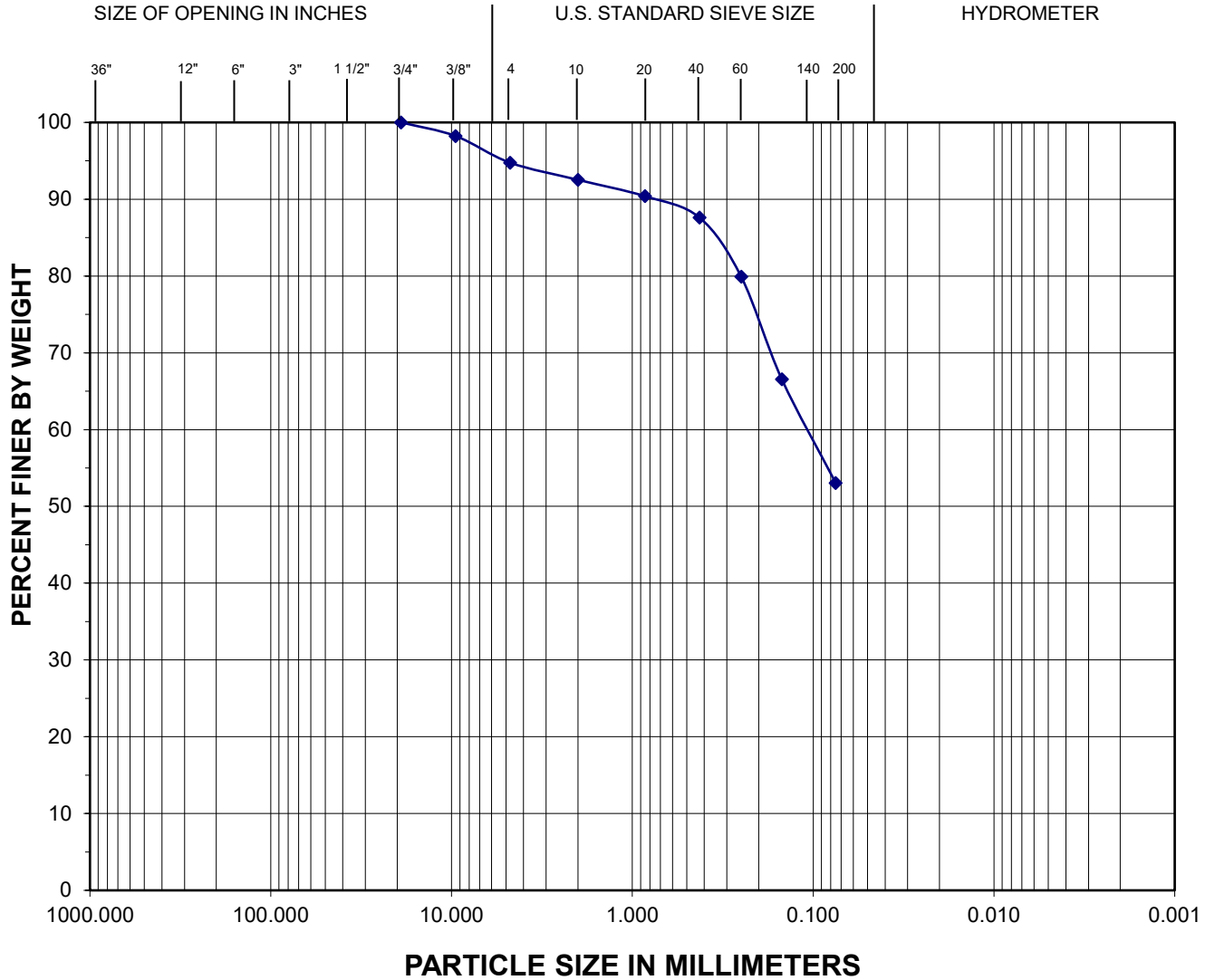
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-4	S-1	2.5	1.9	6.2	sandy GRAVEL, some silt

<b>Zipper Geo Associates, LLC</b> Geotechnical and Environmental Consultants	PROJECT NO: 2955.01	PROJECT NAME:
	DATE OF TESTING: 9/3/2025	Scriber Lake Low-Income Housing

# 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-4	S-2	5	11.9	53.1	sandy SILT, trace gravel

<b>Zipper Geo Associates, LLC</b> Geotechnical and Environmental Consultants	PROJECT NO: 2955.01	PROJECT NAME:
	DATE OF TESTING: 9/3/2025	Scriber Lake Low-Income Housing

**APPENDIX C**  
**NELSON GEOTECHNICAL ASSOCIATES. INC. TEST PIT LOGS, DATED JULY 10, 2024**

# LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
--------------	-----	------------------

---

**INFILTRATION PIT ONE**

0.0 – 0.2		<b>TOPSOIL</b>
0.2 – 1.0		GRAY-BROWN, SILTY FINE TO COARSE SAND (MEDIUM DENSE, DRY) ( <b>FILL</b> )
1.0 – 1.7	SM	RED-BROWN, SILTY FINE TO MEDIUM SAND WITH TRACE GRAVEL AND 1/8-INCH ROOTS (LOOSE-MEDIUM DENSE, DRY)
1.7 – 2.3	SP	GRAY-TAN, FINE TO COARSE SAND WITH GRAVEL AND SILT AND TRACE IRON OXIDATION STAINING (MEDIUM DENSE, DRY-MOIST)
2.3 – 3.4	SP	GRAY, MEDIUM TO COARSE SAND WITH FINE GRAVEL (LOOSE-MEDIUM DENSE, MOIST)
3.4 – 3.8	SP	GRAY-BROWN, FINE SAND WITH TRACE FINE GRAVEL (VERY DENSE, MOIST)
3.8 – 4.8	SP	GRAY MEDIUM TO COARSE SAND WITH FINE GRAVEL (LOOSE-MEDIUM DENSE, MOIST)
4.8 – 5.5	SM	GRAY-BROWN, SILTY MEDIUM TO COARSE SAND WITH FINE GRAVEL (LOOSE-MEDIUM DENSE, WET)

SAMPLES WERE COLLECTED AT 3.0 AND 5.5 FEET  
 GROUNDWATER SEEPAGE WAS ENCOUNTERED AFTER TESTING AT 5.4 FEET  
 MINOR CAVING WAS ENCOUNTERED AT 2.3 FEET  
 INFILTRATION PIT MET REFUSAL ON GRAVEL AT 5.5 FEET ON 06/20/2017

**TEST PIT ONE**

0.0 – 0.8		<b>TOPSOIL</b>
0.8 – 3.0	SM	RED-BROWN, SILTY FINE TO COARSE SAND WITH GRAVEL AND COBBLES AND TRACE 1/2-INCH ROOTS (LOOSE, MOIST)
3.0 – 5.0	SP-SM	GRAY-TAN, FINE TO COARSE SAND WITH GRAVEL AND SILT (LOOSE-MEDIUM DENSE, MOIST)
5.0 – 8.0	SM	GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL (DENSE, DRY)

SAMPLES WERE COLLECTED AT 2.0, 2.5, 4.0, 5.5, AND 8.0 FEET  
 GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED  
 MINOR CAVING WAS ENCOUNTERED BELOW 1.5 FEET  
 TEST PIT COMPLETED AT 8.0 FEET ON 06/20/2017

**TEST PIT TWO**

0.0 – 0.4		<b>TOPSOIL</b>
0.4 – 3.0	SM	RED-BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL AND ORGANIC DEBRIS, INCLUDING 6-INCH DIAMETER STUMPS AND ROOTS (LOOSE, DRY-MOIST)
3.0 – 5.0	SP-SM	GRAY-BROWN, FINE TO COARSE SAND WITH SILT AND GRAVEL WITH TRACE COBBLES (LOOSE-MEDIUM DENSE, MOIST)
5.0 – 8.0	SM	GRAY SILT AND FINE TO MEDIUM SAND WITH GRAVEL (VERY DENSE, WET-DRY)

SAMPLES WERE COLLECTED AT 1.5, 4.9, 5.0, AND 7.0 FEET  
 GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED  
 NO CAVING WAS ENCOUNTERED  
 TEST PIT COMPLETED AT 8.0 FEET ON 06/20/2017

CTC:

**NELSON GEOTECHNICAL ASSOCIATES, INC.**  
 FILE NO 997117  
 FIGURE 5

## LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
<b>TEST PIT THREE</b>		
0.0 – 0.5		<b>TOPSOIL</b>
0.5 – 2.2	SM	RED-BROWN, SILTY MEDIUM TO COARSE SAND WITH TRACE ROOTS (MEDIUM DENSE, MOIST)
2.2 – 5.0	SP	GRAY, MEDIUM SAND WITH TRACE IRON OXIDATION STAINING (LOOSE-MEDIUM DENSE, MOIST)
5.0 – 7.0	SM	GRAY SILT AND FINE TO MEDIUM SAND WITH GRAVEL (VERY DENSE, DRY)
		SAMPLES WERE COLLECTED AT 1.0, 4.5, AND 5.2 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED MINOR CAVING WAS ENCOUNTERED BELOW 2.2 FEET HAND AUGER TEST HOLE MET REFUSAL AT 7.0 FEET ON 06/20/2017
<b>TEST PIT FOUR</b>		
0.0 – 0.3		<b>TOPSOIL</b>
0.3 – 2.0	SM	RED-BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		NO SAMPLES WERE COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED NO CAVING WAS ENCOUNTERED HAND AUGER TEST HOLE TERMINATED AT 2.0 FEET DUE TO UTILITIES ON 06/20/2017
<b>TEST PIT FIVE</b>		
0.0 – 0.7		<b>TOPSOIL</b>
0.7 – 3.0	SM	TAN, SILTY FINE TO COARSE SAND WITH FINE GRAVEL AND TRACE COBBLES (LOOSE-MEDIUM DENSE, MOIST)
3.0 – 6.0	SP	GRAY-BROWN, FINE TO COARSE SAND WITH GRAVEL AND TRACE COBBLES (MEDIUM DENSE, MOIST)
6.0 – 7.9	SM	GRAY, SILT AND FINE TO COARSE SAND WITH GRAVEL (DENSE-VERY DENSE, DRY)
		SAMPLES WERE COLLECTED AT 2.0, 2.5, 6.0, AND 7.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED MINOR CAVING WAS ENCOUNTERED BELOW 0.7 FEET HAND AUGER TEST HOLE COMPLETED AT 8.2 FEET ON 06/06/2017