GEOTECHNICAL ENGINEERING REPORT

ARLINGTON COMMERCIAL DEVELOPMENT
7530 204TH STREET NE
ARLINGTON, WASHINGTON

ZGA Project No. 2760.01
October 30, 2023

DRAFT

Prepared for:

WET RABBIT, LLC

Prepared by:

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ZGA Project No. 2760.01
October 30, 2023

Wet Rabbit, LLC
3213 West Wheeler Street, #750
Seattle, Washington 98199

Attention: Mr. Chris McClure

Subject: **Geotechnical Engineering Report - DRAFT**
Arlington Commercial Development
7530 204th Street NE
Arlington, Washington 98223
Parcel Numbers 310514-001-006-00 and 310514-001-010-00

Dear Mr. McClure,

In accordance with your request and written authorization, Zipper Geo Associates, LLC (ZGA) has completed the subsurface evaluation and geotechnical engineering report for the above-referenced project. This report presents the findings of the subsurface evaluation and geotechnical recommendations for the project design and construction. Our services have been completed in general accordance with our *Proposal for Geotechnical Services* (Proposal No. P23120) dated September 22, 2023. Written authorization to proceed was provided by you on September 22, 2023. We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Zipper Geo Associates, LLC

DRAFT

DRAFT

James P. Georgis, L.E.G.
Principal

Robert A. Ross, P.E.
Principal

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1.0 INTRODUCTION

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

1.1 Site Description

The site is comprised of two adjoining Snohomish County tax parcels. Tax Parcel 310514-001-006-00 is a developed site with a property address of 7530 204th Street NE in Arlington, Washington. According to Snohomish County Assessor's records, the developed parcel encompasses 0.86 acres of relatively level land with an unoccupied commercial building formerly occupied by Arlington Family Medicine located in the southeastern portion of the parcel. The building is a 3,520 square foot, one-story, wood-framed structure reportedly build in 1984. Asphalt surfaced drive lanes and parking are found to the north, west, and south of the commercial building. The development is services by underground utilities and includes a stormwater system. Vegetation includes ornamental plantings in parking lot islands and around the building, a row of large trees along the west and south property lines which appear to serve as a wind block, and a lawn south of the building. Asphalt pavements near the west and south rows of trees exhibit moderate root damage.

Tax Parcel 310514-001-010-00 is a relatively level, undeveloped site located east of and adjoining the southern portion of the above discussed developed parcel's east property line. The parcel is irregular in shape and according to Snohomish County Assessor's records encompasses 0.81 acres. It appears that some underground utilities may be located near the east property line. Vegetation within the undeveloped parcel primarily consists of well-developed tall grasses with dense blackberry brush along its west and north property lines.

A roughly "L" shaped drainage ditch is located along the northern portion of the developed parcels east property line and along the entire north property line of the undeveloped parcel. Running water of about 3 to 4 inches deep was observed in the drainage ditch during our site reconnaissance. The enclosed *Site and Exploration Plan*, Figure 1, includes an aerial photograph of the subject parcels.

1.2 Project Understanding

We understand that Wet Rabbit, LLC intends to redevelop the currently vacant developed parcel (310514-001-006-00) with a new Wet Rabbit-branded car wash facility. We understand that a layout of the redevelopment has not been finalized, but we expect redevelopment to generally include demolition of the existing structure, minor regrading, construction of a new drive-thru car wash tunnel (potentially along the west side of the site), and reconfiguration and repaving of the parking lot and drive lanes. We understand that development of the currently undeveloped parcel (310514-001-010-00) may include a one-story Quick Service Restaurant (QSR) or similar commercial development. Lightly loaded shallow foundations with slab-on-grade floors are anticipated.

We understand that stormwater management design will be completed in accordance with the 2014 Washington State Department of Ecology (Ecology) Stormwater Management Manual for Western Washington (Stormwater Manual). The type of infiltration system and system location(s) have not been determined. However, given groundwater conditions in this part of Arlington, we anticipate that infiltration systems could include shallow infiltration galleries below parking lot areas.

2.0 SUBSURFACE CONDITIONS

2.1 Published Geologic Information

We assessed the geologic setting of site and the surrounding vicinity by reviewing the *Geologic Map of the Arlington West Quadrangle, Snohomish County, Washington* (US Geological Survey, Map MF-1740, 1985). The published geologic mapping shows the site is underlain by Vashon Recessional Outwash, Marysville Sand Member. The Marysville Sand Member is described as mostly well-drained, stratified to massive outwash sand, with some fine gravel, and some areas of silt and clay. The sediments were deposited by melt water flowing south from the stagnating and receding Vashon glacier. The outwash is reported to have a maximum thickness of about 140 feet. Subsurface conditions disclosed by the explorations advanced for this evaluation are consistent with the published mapping.

2.2 Soil Conditions

The subsurface evaluation completed by ZGA for this project included four borings (B-1 through B-4) and one cone penetrometer test (CPT-01). CPT-01 was completed near the center of the proposed car wash development (currently developed western parcel) and extended to a depth of about 60 feet below grade. The CPT exploration included seismic shear wave velocity measurements. Borings B-1 and B-2 were completed in the north and south portions of the western parcel's existing parking lot, respectively, and extended about 36½ feet below grade. Borings B-3 and B-4 were completed in the undeveloped eastern parcel and extended about 31½ to 36½ feet below grade. Groundwater monitoring wells were installed in borings B-1 through B-3 to evaluate changes in groundwater levels through the wet season. The enclosed *Site and Exploration Plan,* Figure 1, presents the approximate locations of our subsurface explorations completed for this project.

Soil samples recovered from the borings were visually classified in general accordance with the *Explanation of Exploration Logs* enclosed in *Appendix A*. Detailed, descriptive logs of the subsurface explorations and the procedures utilized in the subsurface exploration program are also presented in

Appendix A. In general, our explorations disclosed asphalt pavement and topsoil over recessional glacial outwash deposits interpreted as the Marysville Sand Member. Generalized descriptions of subsurface soil conditions observed at the exploration locations are presented below.

2.2.1 Pavement Section

Borings B-1 and B-2 were completed in the parking lot of the western parcel and encountered about 2 inches of asphalt. The thickness of the pavement at the CPT location could not be accurately determined due to the small diameter of the conical probe used to complete the exploration.

2.2.2 Topsoil

Borings B-3 and B-4 were completed in the undeveloped eastern parcel and encountered about 3 to 4 inches of topsoil. In general, the topsoil consisted of loose, brown sand with silt and abundant fine roots.

2.2.3 Glacial Recessional Outwash

Soil interpreted as glacial recessional outwash deposits of the Marysville Sand Member were encountered below the pavement in the western parcel and below the topsoil in the eastern undeveloped parcel. The glacial outwash deposits extended to the maximum depth explored of 36½ feet in the borings and 60 feet in the CPT.

In general, the upper 5 to 7½ feet of the outwash consists of loose to medium dense sand with variable silt and silt with variable sand content. Soil mottling was observed in some of the samples collected in this depth range. From about 5 to 7½ feet down to about 15 to 17½ feet, the outwash generally consists of medium dense to very dense gravelly sand to sandy gravel with trace to some silt. From about 15 to 17½ feet to the maximum depth explored at 60 feet below grade, the outwash generally consists of loose to medium dense sand with trace to some silt.

2.3 Groundwater Conditions

Groundwater was observed in borings B-1 through B-3 about 25 to 30 feet below grade at the time of drilling. Groundwater monitoring wells were installed in borings B-1 through B-3. Groundwater monitoring well readings are typically more accurate than groundwater depth estimates completed at the time of drilling and are presented in the table below.

Groundwater Monitoring Well Data

Boring Number	Approx. Surface Elevation (ft)	Date of Groundwater Measurement	Approx. Depth to Groundwater (ft)	Approx. Groundwater Elevation (ft)
B-1	127	10-17-2023	25.41	101.59
B-2	129	10-17-2023	27.37	101.63
B-3	130	10-17-2023	28.62	101.38
Ground surface elevations at the exploration locations were obtained from Google Earth Pro imagery and should be considered approximate				

Groundwater levels, flow rates and soil moisture conditions should be expected to vary throughout the year. Fluctuations of the groundwater levels will likely occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the explorations were performed. Therefore, groundwater levels during construction or at other times in the life of the development should be expected to vary.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

Based on the results of our subsurface explorations, laboratory testing, and geotechnical engineering analyses, we conclude that the proposed development is feasible from a geotechnical standpoint, contingent on proper design and construction practices. The explorations completed for our evaluation encountered soils susceptible to seismic induced liquefaction settlements. This condition requires special design considerations, as discussed below.

The liquefaction potential of the development was evaluated in accordance with the 2018 International Building Code (IBC). Based on our analysis, we estimate total seismic settlements of approximately 2½ to 3 inches could occur during the design earthquake. We estimate differential seismic settlements of approximately 1¼ to 1½ inches over a horizontal distance of 40 feet. In our experience, these levels of total and differential seismic settlement are typically considered expectable for most structures from a collapse prevention perspective, but should be evaluated by the owner relative to risk management and the project structural engineer relative to 2018 IBC design requirements and the specific building design being considered. If these levels of potential seismic settlement are not considered acceptable for conventional spread footings, we recommend that a mat foundation be considered. Due to the presence of about 10 to 15 feet of dense to very dense gravelly soils above the potential zone of liquefaction, ground improvement methods, such as stone columns, are not recommended for liquefaction mitigation at this site. This report includes geotechnical design recommendations for both conventional spread and mat foundation options.

Geotechnical engineering recommendations for foundation systems and other earthwork related phases of the project are outlined below. The recommendations contained in this report are based upon the results of field and laboratory testing (which are presented in *Appendices A and B*), engineering analyses, and our current understanding of the proposed project. ASTM and Washington State Department of Transportation (WSDOT) specification codes cited herein respectively refer to the current manual published by the American Society for Testing & Materials and the current edition of the *Standard Specifications for Road, Bridge, and Municipal Construction, (M41-10)*.

3.2 Seismic Design Considerations

The seismic performance of the proposed development was evaluated relative to seismic hazards resulting from ground shaking associated with a design seismic event with a 2,475-year return period determined in accordance with the 2018 International Building Code (IBC). Conformance to the above criteria for seismic excitation does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of the IBC seismic design procedure is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a building may be damaged beyond repair, yet not collapse.

3.2.1 Ground Fault Rupture

Based on review of the United States Geological Survey *Quaternary Fault and Fold Database of the United States* there are no mapped Quaternary faults within 10 miles of the site. It is our opinion that the risk of ground surface rupture at the site is low.

3.2.2 Landsliding

Based on the relatively level topography of the site and surrounding vicinity, it is our opinion that the risk of earthquake-induced landsliding is low.

3.2.3 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 Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration of 0.441g, based on Figure 22-9 of ASCE 7-16.
- A Modified Peak Ground Acceleration (PGA_M) of 0.511g 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 6.8 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-1 completed to a depth of about 60 feet below existing grade. The approximate exploration location is shown on the enclosed *Site and Exploration Plans,* Figure 1. Our analysis indicates the potential for liquefaction below the groundwater table, which was modeled at depths ranging from 25 to 29 feet below grade in our analysis.

3.2.3.1 Liquefaction Settlement

Based on our analyses, we estimate a total seismic settlement of approximately 2½ to 3 inches. We estimate a differential seismic settlement of approximately 1¼ 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 a mat foundation be considered as discussed in subsequent sections of this report.

3.2.3.2 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 and lack of a free-face condition, it is our opinion that the potential for distress at the site from lateral spreading is low.

3.2.4 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.

IBC Seismic Design Criteria

Parameter	Value
2018 International Building Code Site Classification (IBC) [1]	Site Class F [2,3]
Site Latitude/Longitude	48.1803 /-122.1283
Spectral Short-Period Acceleration, S_S	1.038g
Spectral 1-Second Acceleration, S_1	0.371g
Site Coefficient for a Short Period, F_A	1.085
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}	1.126g
Spectral Acceleration for a 1-Second Period, S_{M1}	See ASCE Section 11.4.8
Design Short-Period Spectral Acceleration, S_{DS}	0.751g
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. The explorations completed for this study extended to a maximum depth of about 60 feet below grade. ZGA therefore determined the Site Class assuming that medium dense to dense glacial outwash soils with an average n value greater than 15 extend to 100 feet as suggested by published geologic maps for the project area.
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.

3.3 Site Preparation

3.3.1 Existing Structure Removal

The western parcel is currently developed with an unoccupied commercial building with associated asphalt pavement and concrete flatwork. After the building is removed, we recommend that any remaining foundation elements or other below grade structures, if encountered, be demolished and removed from the proposed development areas. We recommend that the resulting excavation be backfilled with compacted structural fill.

3.3.2 Existing Utility Removal

We recommend that all underground utilities within the prosed building pad be completely removed. Utility pipes outside the building envelope could be abandoned in place, provided they are fully grouted with controlled density fill (CDF) and the trench backfill is density tested to verify that it meets the compaction levels presented in the project specifications. Localized excavations made for removal of utilities or existing unsuitable trench backfill should be backfilled with structural fill as outlined in the following section of this report.

3.3.3 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.

3.3.4 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 upper 5 to 7½ feet of the site soils have a moderate fines (silt and clay) content and are therefore susceptible to disturbance and erosion when wet. 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.

3.3.5 Clearing and Stripping

Vegetation in the western developed parcel includes ornamental plantings in parking lot islands and around the building, a row of large trees along the west and south property lines which appear to serve as a wind block, and a lawn south of the building. Asphalt pavements near the west and south rows of trees exhibit moderate root damage. We anticipate that clearing and stripping depths on the order of 6 to 12 inches may be required in landscaped areas. Deeper excavations may be needed to remove tree roots along the south and west sides of the site. Vegetation within the undeveloped eastern parcel primarily consists of well-developed tall grasses with dense blackberry brush along its west and north property lines. Based on borings B-3 and B-4, we anticipate topsoil stripping depths on the order of 4 to 6 inches in the eastern parcel. Any excavations that extend below finish grades should be backfilled with structural fill as outlined subsequently in this report.

3.3.6 Subgrade Preparation and Protection

Once site preparation is complete, all areas that do not require over-excavation and are at design subgrade elevation or areas that will receive new structural fill should be compacted to a firm and unyielding condition, and to achieve the recommended compaction level within the upper 12 inches of exposed subgrade soil presented in *Section 3.5.6* of this report. Some moisture conditioning of site soils may be required to achieve a moisture content appropriate for compaction. This is generally within ±2 percent of the soil's optimum moisture content determined in accordance with ASTM D 1557 test procedure. Our laboratory testing indicates that, at the time our explorations were completed, in-situ moisture contents collected from the upper 5 feet of the site range from about 5 to 29 percent. As a result, we expect that moisture conditioning of site soils during construction will be required to achieve suitable moisture contents (plus or minus two percent of optimum) for compaction.

Earthwork should be completed during drier periods of the year when soil moisture content can be controlled by aeration and drying if possible. If earthwork or construction activities take place during extended periods of wet weather, it will be difficult to achieve a firm, non-yielding surface and recommended compaction levels. 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 removed to a sufficient depth in order to develop stable subgrade soils that can be compacted to the minimum recommended levels. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the subgrade soils.

Once compacted, subgrades should be evaluated through density testing and proof rolling with a loaded dump truck or heavy rubber-tired construction equipment weighing at least 20 tons to assess the subgrade adequacy and to detect soft and/or yielding soils. In the event that compaction fails to meet the specified criteria, the upper 12 inches of subgrade should be scarified, and moisture conditioned as necessary to obtain the specified compaction level. Those soils which are soft, yielding, or unable to be compacted to the specified criteria should be over-excavated and replaced with suitable material as recommended in the *Structural Fill* section of this report.

To protect stable subgrades in the wet season, we recommend using crushed rock. The thickness of the protective layer should be determined at the time of construction and be based on the moisture condition of the soil and the amount of anticipated traffic.

3.3.7 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

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 Laboratory Testing

Representative samples of on-site and imported soils to be used as structural fill should be submitted for laboratory testing at least four days in advance of its intended use and required density testing in order to complete the necessary Proctor tests.

3.4.2 Re-Use of Site Soils as Structural Fill

It is our opinion that the soils encountered on the site are adequate for reuse as structural fill from a compositional standpoint provided it is placed and compacted in accordance with the recommendations presented in this report. Our laboratory testing indicates that, at the time our explorations were completed, in-situ moisture contents collected from the upper 5 feet of the site range from about 5 to 29 percent. As a result, we expect that some moisture conditioning during construction of site soils will be required to achieve suitable moisture contents (plus or minus two percent of optimum) for compaction. Drying of over-optimum moisture soils may be achieved by scarifying or windrowing surficial materials during extended periods of dry weather. If encountered, soils which are dry of optimum may be moistened through the application of water and thorough blending to facilitate a uniform moisture distribution in the soil prior to compaction.

We recommend that site soils used as structural fill have less than 4 percent organics by weight, 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 reuse as structural fill.

3.4.3 Imported Structural Fill

If additional material is required for grading and fills, the appropriate type of imported structural fill will depend on the weather conditions. During extended periods of dry weather, we recommend imported fill meet the requirements of Common Borrow, Options 1 or 2 as specified in Section 9-03.14(3) of the 2023 Washington State Department of Transportation, *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT Standard Specifications). The on-site soils would generally be classified as Common Borrow. During wet weather, higher-quality (lower fines content) structural fill might be required, as Common Borrow may contain sufficient fines to be moisture sensitive. During wet weather we recommend that imported structural fill meet the requirements of Gravel Borrow as specified in Section 9-03.14(1) of the 2023 WSDOT Standard Specifications.

3.4.4 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 (such as most of the on-site soils encountered in the upper 2 feet of the site) 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.

3.4.5 Fill Placement

We recommend that structural fill be placed in horizontal lifts not exceeding 8 inches in loose thickness and each lift of fill be compacted using compaction equipment suitable for the soil type and lift thickness to the minimum levels recommended below based on the maximum laboratory dry density as determined by the ASTM D1557 Modified Proctor Compaction Test.

3.4.6 Compaction Criteria

Our recommendations for soil compaction are summarized in the following table. 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.

RECOMMENDED SOIL COMPACTION LEVELS	
Location	**Minimum Percent Compaction***
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
Upper two feet of utility trench backfill	95
Utility trenches below two feet	92
Landscape areas	90
* *ASTM D1557 Modified Proctor Maximum Dry Density*	

3.5 Utility Trenching and Backfilling

We recommend that utility trenching conforms to all applicable federal, state, and local regulations, such as OSHA and WISHA, for open excavations. Trench excavation safety guidelines are presented in WAC Chapter 296-155 and WISHA RCW Chapter 49.17.

3.5.1 Trench Dewatering

Groundwater was encountered in borings B-1 through B-3 at depths of about 25 to 30 feet below existing grade at the time of our evaluation. As such, significant construction dewatering associated with utility installations is not anticipated. However, some excavations for utilities and underground structures may encounter zones of perched groundwater that may develop above lower permeability layers in the site outwash soils. The amount of perched groundwater seepage that may be encountered in site excavations will likely be a function of the time of year, the size of the excavation, the excavation depth, and how long the excavation remains open. The type and extent of dewatering measures needed, if any, will be a function of the groundwater conditions at the time of construction. Temporary systems could include pumped sumps, wellpoints, or pumped wells. If dewatering becomes necessary, the appropriate type of dewatering system and means of water disposal should be determined by the contractor based on the conditions encountered.

3.5.2 Utility Subgrade Preparation

We recommend that all utility subgrades be firm and unyielding and free of soils that are loose, disturbed, or pumping. Soils that pump or yield should be removed and replaced. All structural fill used to replace over-excavated soils should be compacted as recommended in the *Structural Fill* section of this report.

3.5.3 Bedding and Initial Backfill

We recommend that a minimum of 4 inches of bedding material be placed above and below all utilities or in general accordance with the utility manufacturer's recommendations and local ordinances. We recommend that pipe bedding consist of Gravel Backfill for Pipe Zone Bedding as specified in Section 9-03.12(3) of the 2023 WSDOT Standard Specifications. All trenches should be wide enough to allow for compaction around the haunches of the pipe, or material such as pea gravel should be used below the spring line of the pipes to eliminate the need for mechanical compaction in this portion of the trenches. If water is encountered in the excavations, it should be removed prior to fill placement.

3.5.4 Trench Backfill

Materials, placement and compaction of utility trench backfill 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.

3.6 Temporary Shoring

We recommend that temporary shoring systems be used where excavations will be located adjacent to existing foundations, property lines, roadways or utilities, or where ground loss could damage existing facilities. A trench box is one type of support system which might be used. The zone between the trench box and the excavation face should be backfilled as necessary to limit ground movements. As an alternative, braced or unbraced shoring of various types could be considered.

3.7 Temporary and Permanent Slopes

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 exceedingly difficult under the variable circumstances to pre-establish a safe and “maintenance-free” temporary cut slope angle. For planning purposes, temporary cuts in loose soils on the order of 1.5H:1V and temporary cuts in medium dense to very dense soils on the order of 1H:1V could be considered. These planning level temporary cut slope inclinations assume a drained condition. Flatter slopes may be needed if groundwater seepage is present. However, it should be the responsibility of the contractor to maintain safe temporary 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. 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. The excavation should conform to applicable Federal, State, and Local regulations.

According to Chapter 296-155 of the Washington Administrative Code (WAC), the contractor should make a determination of excavation side slopes based on classification of soils encountered at the time of excavation. Temporary cuts may need to be constructed at flatter angles based upon the soil moisture, soil density, and groundwater conditions at the time of construction. Adjustments to the slope angles should be determined by the contractor at that time.

We recommend that all permanent cut or fill slopes constructed in native or properly compacted fill soils be designed at a 2H:1V (Horizontal:Vertical) inclination or flatter. All permanent cut and fill slopes should be adequately protected from erosion both temporarily and permanently.

3.8 Corrosion Considerations

Soils in the vicinity of the project site are considered to be very slightly corrosive to concrete and are not typically associated with high sulfate contents. As such, the relative degree of sulfate attack would be considered negligible and ASTM Type I/II Portland cement is suitable for all concrete below and at grade.

3.9 Shallow Foundations

Based on our analyses, it is geotechnically feasible to support the proposed buildings on conventional shallow footings provided that foundation subgrades are properly prepared. We anticipate that foundation subgrade soils will generally consist of loose to medium dense sand with variable silt content.

Our analyses indicate the potential for liquefaction induced settlement due to ground vibrations associated with the 2018 IBC design seismic event with a return period of 2,475 years. The foundation recommendations presented below assume that the levels of potential seismic settlement presented in

Seismic Design Considerations section of this report (total seismic settlement of approximately 2½ to 3 inches and differential seismic settlement of approximately 1¼ to 1½ inches over a horizontal distance of 40 feet) are considered acceptable and meet the Life Safety and Collapse Performance objectives of the 2018 IBC.

If these conditions and levels of seismic settlement are not acceptable for the planned building, we recommend that the mat foundation option presented in this report be utilized to mitigate potential seismic settlements. However, if a mat foundation is utilized for the building, the spread foundation recommendations presented below may be used for ancillary structures determined by the structural engineer and owner to be less sensitive to potential seismic settlements.

3.9.1 Foundation Subgrade Preparation

We recommend that the subgrade exposed at the bottom of foundation excavations be compacted to a firm and non-yielding condition and to at least 95 percent of the modified Proctor maximum dry density determined in accordance with ASTM D 1557. If the exposed subgrade cannot be compacted to the required density, we recommend that it be removed to a depth of 12 inches and be replaced with WSDOT Standard Specification Section 9-03.9(3), Crushed Surfacing Base Course compacted to at least 95 percent of the modified Proctor maximum dry density. We recommend that the over-excavation extend outside the limits of the footings a distance equal to the depth of over-excavation.

3.9.2 Allowable Bearing Pressure

Continuous and isolated column footings bearing on compacted glacial outwash or structural fill placed and compacted in accordance with this report may be designed for a maximum allowable, net, bearing capacity of 2,000 psf. A one-third increase of the bearing pressure may be used for short-term transient loads such as wind and seismic forces. The above-recommended allowable bearing pressure includes a 3.0 factor of safety.

3.9.3 Shallow Foundation Depth and Width

For frost protection, the bottom of all exterior footings should bear at least 18 inches below the lowest adjacent outside grade, whereas the bottoms of interior footings should bear at least 12 inches below the surrounding slab surface level. We recommend that all continuous wall and isolated column footings be at least 12 and 24 inches wide, respectively.

3.9.4 Lateral Resistance

Resistance to lateral loads can be calculated assuming an allowable passive resistance of 300 pcf equivalent fluid pressure (triangular distribution) and an allowable base friction coefficient of 0.35. The allowable passive resistance and friction coefficient include a 1.5 factor of safety. We recommend that passive resistance be neglected in the upper 18 inches of embedment.

3.9.5 Estimated Settlement

Assuming the foundation subgrade soils, and structural fill compaction are completed in accordance with recommendations presented herein, we estimate that total and differential static settlements will be less than 1 inch and ½ inch over 40 feet, respectively.

3.10 On-Grade Concrete Floor Slabs

Our analyses indicate the potential for liquefaction induced settlement due to ground vibrations associated with the 2018 IBC design seismic event with a return period of 2,475 years. The on-grade concrete floor slab recommendations presented below assume that the levels of potential seismic settlement presented in *Seismic Design Considerations* section of this report (total seismic settlement of approximately 2½ to 3 inches and differential seismic settlement of approximately 1¼ to 1½ inches over a horizontal distance of 40 feet) are considered acceptable. If these levels of seismic settlement are not acceptable, we recommend that the mat foundation option presented in this report be utilized to mitigate potential seismic settlements.

3.10.1 Subgrade Preparation

After excavation to subgrade elevation, the base of the excavation is frequently disturbed or altered due to utility excavations, construction traffic, desiccation, or rainfall. As a result, the slab-on-grade subgrade may become unsuitable for floor slab support. We recommend that the slab subgrade be compacted to a firm and non-yielding condition and to at least 95 percent of the modified Proctor maximum dry density determined in accordance with ASTM D 1557. At the time of slab base placement, the subgrade should be evaluated by ZGA to verify a firm and non-yielding condition and adequate compaction.

3.10.2 Slab Base

To provide a capillary break and uniform slab bearing surface, we recommend the on-grade slabs be underlain by a 4-inch thick layer of compacted clean crushed rock. In our opinion, the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction does not include a material well suited for this application below slabs. As such, we recommend that the capillary break conform to the graduation requirements for Type 21 or 22 Crushed Gravel as presented in the 2023 City of Seattle Standard Specifications for Road, Bridge, and Municipal Construction, Section 9-03.14, Mineral Aggregate Chart. Alternative capillary break materials should be submitted to the geotechnical engineer for review and approval before use.

3.10.3 Vapor Barrier

Where potential slab moisture is a concern or where moisture sensitive floor coverings are planned, we recommend using a 15-mil, 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.

3.11 Mat Foundation Option

Our analyses indicate the potential for liquefaction induced settlement due to ground vibrations associated with the IBC design seismic event with a return period of 2,475 years. As discussed in the *Seismic Design Considerations* section of this report, we estimate a total seismic settlement of approximately 2½ to 3 inches and a differential seismic settlement of approximately 1¼ to 1½ inches over a horizontal distance of 40 feet) for the 2018 IBC design earthquake. A mat foundation could be considered to mitigate this seismic settlement risk. Geotechnical recommendations for a mat foundation are presented below.

3.11.1 Subgrade Preparation

After excavation to subgrade elevation, the base of the excavation is frequently disturbed or altered due to utility excavations, construction traffic, desiccation, or rainfall. As a result, the foundation subgrade may become unsuitable. We recommend that the upper foot of mat foundation subgrade be compacted to a firm and non-yielding condition and to at least 95 percent of the modified Proctor maximum dry density determined in accordance with ASTM D 1557. At the time of slab base placement, the subgrade should be evaluated by ZGA to verify a firm and non-yielding condition and adequate compaction.

3.11.2 Mat Foundation Base

To provide a capillary break and uniform slab bearing surface, we recommend the on-grade slabs be underlain by a 6-inch thick layer of compacted clean crushed rock. In our opinion, the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction does not include a material well suited for this application below slabs. As such, we recommend that the capillary break conform to the graduation requirements for Type 21 or 22 Crushed Gravel as presented in the 2023 City of Seattle Standard Specifications for Road, Bridge, and Municipal Construction, Section 9-03.14, Mineral Aggregate Chart. Alternative capillary break materials should be submitted to the geotechnical engineer for review and approval before use.

3.11.3 Modulus of Subgrade Reaction

For mat foundation design supported on 6 inches of compacted structural fill as described above, we recommend a vertical modulus of subgrade reaction of 200 pounds per cubic inch (pci) be used. This vertical modulus is for a 1-foot by 1-foot loaded area.

3.11.4 Allowable Bearing Pressure and Static Settlement

The soil bearing capacities for mat foundations supported on granular soils are typically very large due to large foundation widths and are therefore typically controlled by allowable settlement. Based on a mat foundation width of about 40 feet, length of about 50 to 70 feet, and embedment depth of about 1.5 feet, we recommend an allowable, net soil bearing capacity of 3,000 psf to limit total static settlement to less than 1 inch and differential static settlement to less than ½ inch. A one-third increase of the bearing pressure may be used for short-term transient loads such as wind and seismic forces. Based on previous projects utilizing a mat foundation to support a structure of this size, we anticipate that actual loads would be substantially less than the allowable load of 3,000 psf. As such, total and differential static settlements are anticipated to be small. Settlements would occur elastically as loads are applied.

3.11.5 Mat Foundation Depth

For frost protection, we recommend that the bottom of the mat foundation around the perimeter of the building bear at least 18 inches below the adjacent outside grade.

3.11.6 Lateral Resistance

Resistance to lateral loads can be calculated assuming an allowable passive resistance of 300 pcf equivalent fluid pressure (triangular distribution) and an allowable base friction coefficient of 0.35. The allowable passive resistance and friction coefficient include a 1.5 factor of safety. We recommend that passive resistance be neglected in the upper 18 inches of embedment.

3.11.7 Vapor Barrier

Where potential slab moisture is a concern or where moisture sensitive floor coverings are planned, we recommend using a 15-mil, 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.

3.11.8 Estimated Seismic Settlements

As discussed in the *Seismic Considerations* section of this report, our analyses indicate the potential for liquefaction induced settlement during the 2018 IBC design earthquake. Based on our analyses, we estimate a total seismic settlement of the soils beneath the slab of approximately 2½ to 3 inches and a differential seismic settlement of approximately 1¼ to 1½ inches over a horizontal distance of 40 feet.

3.12 Backfilled Walls

We anticipate that the development may include some backfilled, cast-in-place (CIP), concrete, building foundation and/or landscape retaining walls. For recommended bearing capacities and lateral resistance parameters, refer to *Shallow Foundations* (Section 3.9) of this report. Please refer to Section 3.13.3 of this report for wall drainage recommendations. Additional recommendations for CIP retaining walls are provided below.

3.12.1 Lateral Earth Pressures

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

For backfilled walls, assuming they are backfilled and drained in accordance with Section 3.12.3 of this report, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (active earth pressure). Non-yielding walls should be designed using an equivalent fluid density of 55 pcf (at-rest earth pressure). Surcharge pressures due to sloping backfill, adjacent footings, vehicles, construction equipment, etc. must be added to these lateral earth pressure values. For traffic loads, we recommend using an equivalent two-foot soil surcharge of 250 psf. For retaining walls over 6 feet tall with level backfill conditions, we recommend that a uniformly distributed seismic pressure of 12H psf, where H is the height of the wall, be applied to the walls if required by code. The above equivalent fluid pressures are based on the assumption of 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.

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 and Wall Drains

We recommend that the building be provided with a footing drain system to reduce the risk of future moisture problems. The 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 2023 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.

3.13.2 Backfilled Retaining Wall Drains

Adequate drainage measures must be installed to collect and direct subsurface water away from subgrade walls. All backfilled walls should include a drainage aggregate zone extending 1½ feet from the back of wall for the full height of the wall. The drainage aggregate should consist of material meeting the requirements of 2023 WSDOT 9-03.12(2) Gravel Backfill for Walls. A minimum 4-inch diameter, Schedule 40, rigid, perforated PVC pipe should be provided at the base of backfilled walls with the perforations facing down to collect and direct subsurface water to an appropriate discharge point. The pipe should be surrounded by a minimum of 6 inches of clean free-draining granular material conforming to 2023 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.

3.14 Infiltration Considerations

We understand that stormwater management design will be completed in accordance with the 2014 Washington State Department of Ecology Stormwater Management Manual for Western Washington (2014 DOE Manual). The type of infiltration system and system location(s) have not been determined. However, given the relatively shallow nature of groundwater in this part of Marysville, we anticipate that infiltration systems may include shallow infiltration galleries below the parking lot.

Based on the results of our subsurface exploration and analysis, shallow stormwater infiltration systems appear geotechnically feasible. The following sections provide geotechnical design recommendations regarding stormwater infiltration into the recessional glacial recessional outwash deposits encountered on site.

3.14.1 Design Infiltration Rates

Soil conditions observed in the site explorations are consistent with normally consolidated glacial recessional outwash deposits of the Marysville Sand Member and generally consist of the following.

- Loose to medium dense sand with variable silt and silt with variable sand content in the upper 5 to 7½ feet. Soil mottling was observed in some of the samples collected in this depth range.
- From about 5 to 7½ feet down to about 15 to 17½ feet, the outwash generally consists of medium dense to very dense gravelly sand to sandy gravel with trace to some silt.
- From about 15 to 17½ feet to the maximum depth explored at 60 feet below grade, the outwash generally consists of loose to medium dense sand with trace to some silt.

Volume 3, Section 3.3.6 of the 2014 DOE Manual includes provisions for the determination of initial and design saturated hydraulic conductivities for receptor soils based on 1) Large-Scale Pilot Infiltration Tests, 2) Small-Scale Pilot Infiltration Tests, and 3) by means of the Soil Grain Size Analyses Method. We completed our evaluation of the design infiltration rate using the Soil Grain Size Analysis Method based on test results of soil samples collected in ZGA's explorations. Grain size analysis test results are presented in *Appendix B* of this report. The Design Infiltration Rates presented in the table below include the following correction factors in accordance with Volume 3, Section 3.3.6 of the 2014 DOE Manual.

- Site Variability and number of locations tested (CFv) = 0.5
- Test Method (CFt) = 0.4
- Degree of influent control to prevent siltation and bio-buildup (CFm) = 0.9

Summary of Grain Size Analysis Tests Relative to Stormwater Infiltration

Exploration	Sample Number	Sample Depth (ft)	Soil Formation	Initial (unfactored) Saturated Hydraulic Conductivity (in/hr)	Design Infiltration Rate[2] (in/hr)
B-1	S-3	7½	Marysville Sand Member	20.2	3.6
B-1	S-4	10	Marysville Sand Member	37.1	6.7
B-1	S-5	10	Marysville Sand Member	19.3	3.5
B-2	S-2	5	Marysville Sand Member	3.8	0.7
B-2	S-3	7½	Marysville Sand Member	12.5	2.3
B-2	S-4	10	Marysville Sand Member	15.6	2.8
B-2	S-5	15	Marysville Sand Member	14.7	2.6
B-3	S-2	5	Marysville Sand Member	39.9	7.2
B-3	S-3	7½	Marysville Sand Member	2.0	0.4
B-3	S-4	10	Marysville Sand Member	4.4	0.8
B-3	S-5	15	Marysville Sand Member	4.5	0.8
B-4	S-2	5	Marysville Sand Member	11.9	2.2
B-4	S-3	7½	Marysville Sand Member	8.3	1.5
B-4	S-5	15	Marysville Sand Member	28.4	5.1

1. Includes Correction Factors: CFv = 0.5, CFt = 0.4, and CFm = 0.9

The data above indicates that design infiltration rates are generally greater below a depth of about 5 to 7½ feet below grade and that design infiltration rates at boring B-3 located in the eastern portion of the undeveloped parcel are significantly lower than the rest of the study area. Therefore, the selection of an appropriate design infiltration rate will depend on the location and depth of the facility. ZGA is available to assist in the selection of an appropriate design infiltration rate once a system location and depth has been established.

3.14.2 Seasonal High Groundwater

Groundwater was measured at depths of 25.41, 27.37, and 28.62 feet below the ground surface in borings B-1, B-2, and B-3, respectively, on October 17, 2023. Based on the time of year the measurements were taken and our experience with groundwater conditions in this part of Arlington, we recommend a preliminary seasonal high groundwater depth of 20 feet below grade. ZGA will monitor groundwater levels in the on-site wells through the 2023-2024 wet season.

3.14.3 Infiltration System Overflow Considerations

The design of infiltration system is based on a number of design parameters with variable uncertainties. Therefore, we recommend that infiltration systems include an overflow when permitted by code to reduce the potential for water in the system from rising above the top of the system storage aggregate or the bottom of the base course for the parking lot pavement section during periods of unusually high or intense precipitation.

3.14.4 Water Quality Treatment Characteristics of Receptor Soils

Soil requirements for water quality treatment considerations are presented under Site Suitability Criteria (SSC) 4 and 6 in Volume 3, Section 3.3.7 of the 2014 DOE Manual. Laboratory test results relative to water quality treatment characteristics are summarized below. Analytical laboratory test results are presented in *Appendix B* of this report.

3.14.4.1 Organic Content

SSC-6 requires a minimum organic content of 1 percent for treatment facilities. Organic content tests completed on three samples collected from borings B-1, B-2, and B-4 (at depths of about 5, 7½, and 10 feet below grade) had organic contents ranging from 1.1 to 1.6 percent by weight with an average of about 1.4 percent by weight.

3.14.4.2 Cation Exchange Capacity

SSC-6 requires a minimum Cation Exchange Capacity (CEC) of 5 milliequivalents/100grams dry soil for treatment facilities. CEC tests completed on three samples collected from borings B-1, B-2, and B-4 (at depths of about 5, 7½, and 10 feet below grade) had CEC values ranging from 5.8 to 6.5 milliequivalents/100grams dry soil with an average of about 6.2 milliequivalents/100grams dry soil. It should be noted that SSC-6 states that lower CEC (less than 5 milliequivalents/100grams dry soil) may be considered if it is based on a soil loading capacity determination for the target pollutants that is accepted by the local jurisdiction.

3.15 Pavements

3.15.1 Asphalt Pavements

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. A 20-year pavement life typically assumes that an overlay will be placed after about 12 years. 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.

The native subgrade soils are anticipated to consist of a mixture of sand with varying portions of silt. Based on our experience with similar soils, we have estimated a California Bearing Ration (CBR) value of 15 percent for this project. No traffic loading was provided for this project. We have assumed relatively low traffic volumes.

We recommend that the upper 12 inches of pavement subgrades be prepared in accordance with the recommendations presented in the Subgrade Preparation section of this report.

We recommend that the crushed aggregate base course conform to Section 9-03.9(3) of the WSDOT Standard Specifications. All base material should be compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM: D 1557.

3.15.1.1 Asphalt Pavement Recommendations

For light duty pavements (parking stall areas), we recommend 2½ inches of asphalt concrete over 4 inches of crushed rock base course. For heavy duty pavements (main access roads, truck delivery routes, etc.), we recommend 3½ inches of asphalt concrete over 6 inches of crushed rock base course. We recommend that the asphalt concrete conform to Section 9-02.1(4) for PG 58-22 or PG 64-22, Performance Graded Asphalt Binder as presented in the 2022 WSDOT Standard Specifications. We also recommend that the gradation of the asphalt aggregate conform to the aggregate gradation control points for ½-inch mixes as presented in Section 9-03.8(6), HMA Proportions of Materials. We recommend that asphalt be compacted to a minimum of 92 percent and a maximum of 96 percent of the theoretical maximum density.

3.15.2 Concrete Pavements

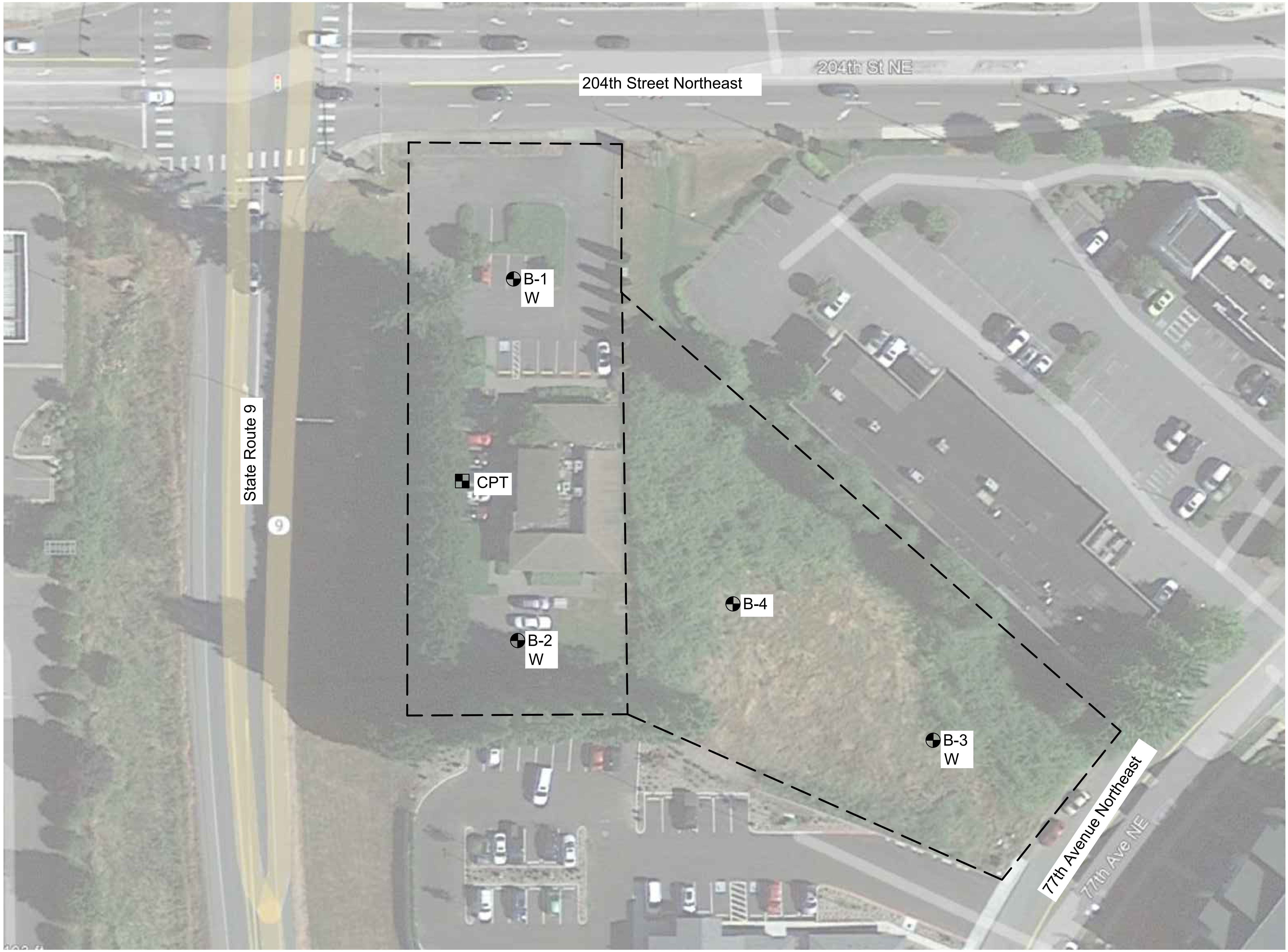
Concrete pavement design recommendations are based on an assumed modulus of rupture of 600 psi and a minimum compressive strength of 4,000 psi for the concrete. For light duty pavements, we recommend 5 inches of concrete over 4 inches of crushed aggregate base. For heavy duty pavements, we recommend 6 inches of concrete over 4 inches of crushed aggregate base. We recommend that concrete pavements be lightly reinforced with 6x6-W2.0xW2.0 welded wire fabric or equivalent to control cracking and have relatively closely spaced control joints on the order of 10 to 12 feet. We recommend that contraction joints be cut before the development of tensile stresses in the concrete. We recommend that the contraction joints be cut to a minimum depth of one inch. We further recommend that loading dock and trash enclosure pavements be reinforced with #4 bars at 15 inches each direction.

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 for our environmental scope and to formulate our geotechnical recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend that ZGA be provided an opportunity to review the final plans and specifications when they become available in order to assess that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design.

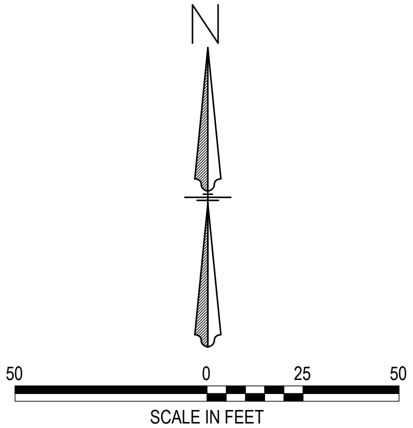
The performance of earthwork, structural fill, foundations, and pavements depend greatly on proper site preparation and construction procedures. We recommend that Zipper Geo Associates, LLC be retained to provide geotechnical engineering services during the earthwork-related construction phases of the 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 Wet Rabbit, LLC, and their agents, for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety and excavation support are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Zipper Geo Associates, LLC reviews the changes and either verifies or modifies the conclusions of this report in writing.



LEGEND

◐ B-1	BORING NUMBER AND APPROXIMATE LOCATION. "W" INDICATES WELL INSTALLATION.
▚ CPT-1	CPT NUMBER AND APPROXIMATE LOCATION.
— — —	APPROXIMATE PROPERTY BOUNDARY.



Proposed Arlington Commercial Development
7530 204th Street NE
Arlington, Washington

SITE AND EXPLORATION PLAN

Date: October 2023
Job No. 2760.01

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

FIGURE 1
SHT. 1 of 1

REFERENCE: (1) GOOGLE EARTH AERIAL EARTH BASEMAP DATED 2023. (2) ZGA FIELD MEASUREMENTS.

APPENDIX A

SUBSURFACE EXPLORATION PROCEDURES AND LOGS

APPENDIX A
SUBSURFACE EXPLORATION PROCEDURES AND LOGS

Subsurface Exploration Description

Our subsurface evaluation for this project included advancing four borings (B-1 through B-4) and one cone penetrometer test (CPT-1), completed in October 2023. The approximate locations of the explorations are presented on the *Site and Exploration Plan*, Figure 1. The exploration locations were determined by measuring with a fiberglass tape relative to existing site features. Ground surface elevations at the exploration locations were inferred from elevation data available through Google Earth Pro aerial imagery. As such, the exploration locations and elevations should be considered accurate only to the degree implied by the means and methods used to define them.

Soil Borings

The borings were advanced using a track-mounted drill rig operated by a local drilling company (Holocene Drill, Inc.) working under subcontract to ZGA. The borings were advanced using hollow stem auger drilling methods. A 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 obtained by means of the Standard Penetration Test at 2.5- 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 was 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 describes the vertical sequence of soils and materials encountered in the borings, 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 borings. If groundwater was encountered, the approximate groundwater depth, and date of observation, are depicted on the logs. Groundwater monitoring well installations are also graphically indicated on the logs.

Cone Penetrometer Test (CPT)

One electric cone penetrometer probe was completed using a truck-mounted probe rig operated by an independent firm (In-Situ Engineering, Inc.) working under subcontract to ZGA. An engineer from our firm continuously observed the probes while electronic monitoring equipment in the probe rig automatically logged the subsurface conditions. After the probe was completed, the probe hole was backfilled with a bentonite slurry.

Throughout the probing operation, soil and groundwater properties were measured at 5-centimeter depth intervals by means of the Cone Penetration Test (CPT) in general accordance with ASTM D3441. This testing procedure involves pushing a standard 1.5-inch diameter steel cone penetrometer into the soil with hydraulic rams. A cone penetrometer consists of a conical tip, a cylindrical sleeve, and a pressure transducer. As the penetrometer is pushed downward, the tip resistance, sleeve friction, and porewater pressure are measured electronically and plotted as a function of depth. Through interpretation, the resulting graphs can reveal soil types and groundwater levels, as well as the relative density of granular soils and the relative consistency of cohesive soils. The CPT exploration completed for this project also included measurements of soil shear wave velocity.

The enclosed CPT graph presents the vertical plots of several soil properties and groundwater pressures measured by the cone penetrometer at each probe location. These graphs also depict the Standard Penetration Resistance (N_{60}) corresponding to each test interval, based on published conversion charts.

EXPLANATION OF EXPLORATION LOGS

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

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

Coarse-Grained Soils

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

Fine-Grained Soils

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

MOISTURE

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

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

GRAIN SIZE

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

GRAIN SIZE MODIFIERS

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

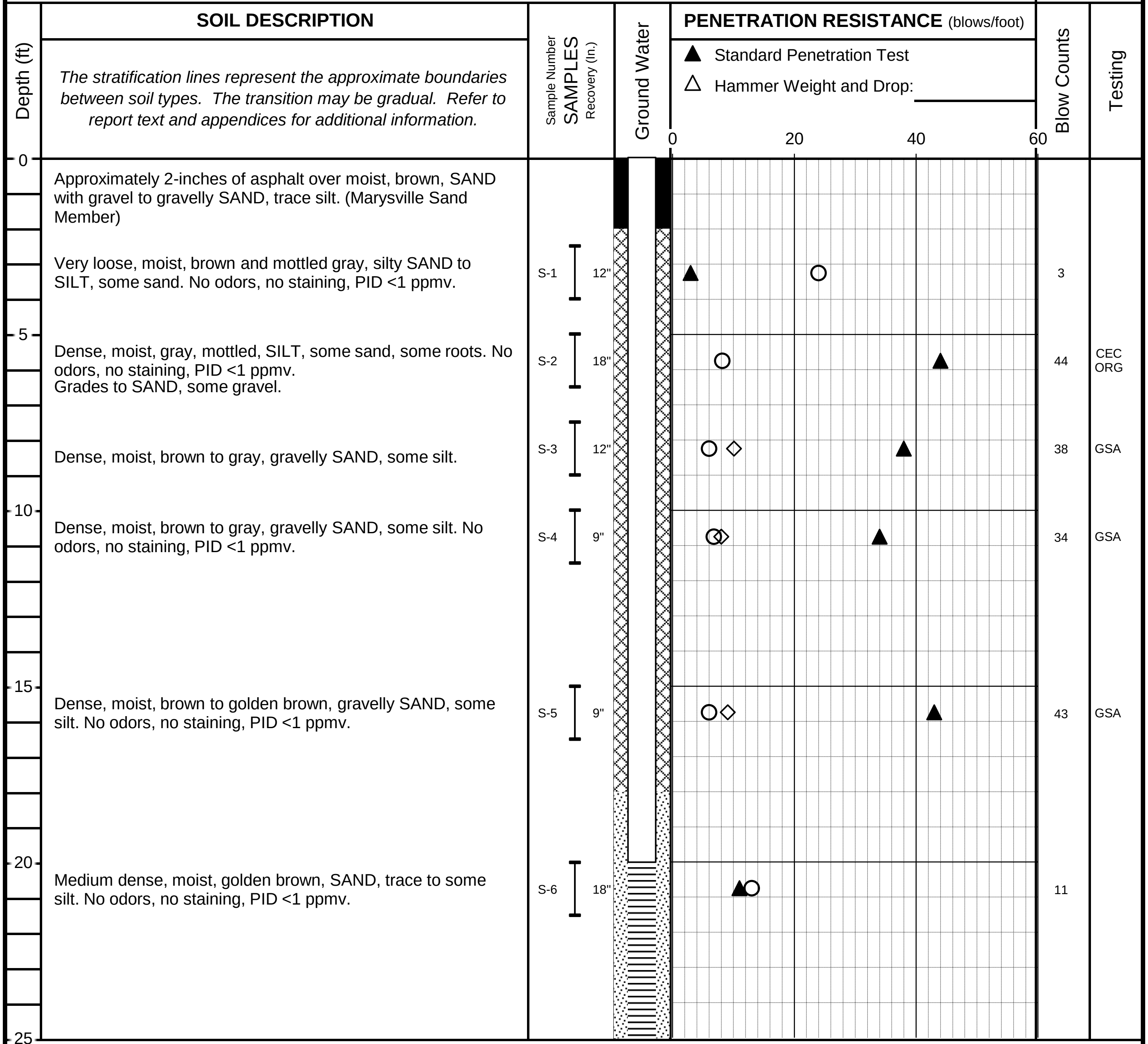
Boring Location: See Site and Exploration Plan
Top Elevation: 127 feet above mean sea
Date Drilled: 10/12/2023
Drilling Company: Holocene
Drilling Method: Hollow Stem Auger
Drill Rig: Diedrich D-50
Bore Hole Dia.: 8-inch
Hammer Type: Automatic
Logged by: KRN

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.

SOIL DESCRIPTION	Sample Number	Recovery (In.)	Blow Counts	Testing
Approximately 2-inches of asphalt over moist, brown, SAND with gravel to gravelly SAND, trace silt. (Marysville Sand Member)				
Very loose, moist, brown and mottled gray, silty SAND to SILT, some sand. No odors, no staining, PID <1 ppmv.	S-1	12"	3	
Dense, moist, gray, mottled, SILT, some sand, some roots. No odors, no staining, PID <1 ppmv. Grades to SAND, some gravel.	S-2	18"	44	CEC ORG
Dense, moist, brown to gray, gravelly SAND, some silt.	S-3	12"	38	GSA
Dense, moist, brown to gray, gravelly SAND, some silt. No odors, no staining, PID <1 ppmv.	S-4	9"	34	GSA
Dense, moist, brown to golden brown, gravelly SAND, some silt. No odors, no staining, PID <1 ppmv.	S-5	9"	43	GSA
Medium dense, moist, golden brown, SAND, trace to some silt. No odors, no staining, PID <1 ppmv.	S-6	18"	11	



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
Att. = Atterberg Limits

Arlington Commercial Development
7530 204th Street NE
Arlington, WA

Date: October 2023
Project No.: 2760.01

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

BORING LOG: B-1

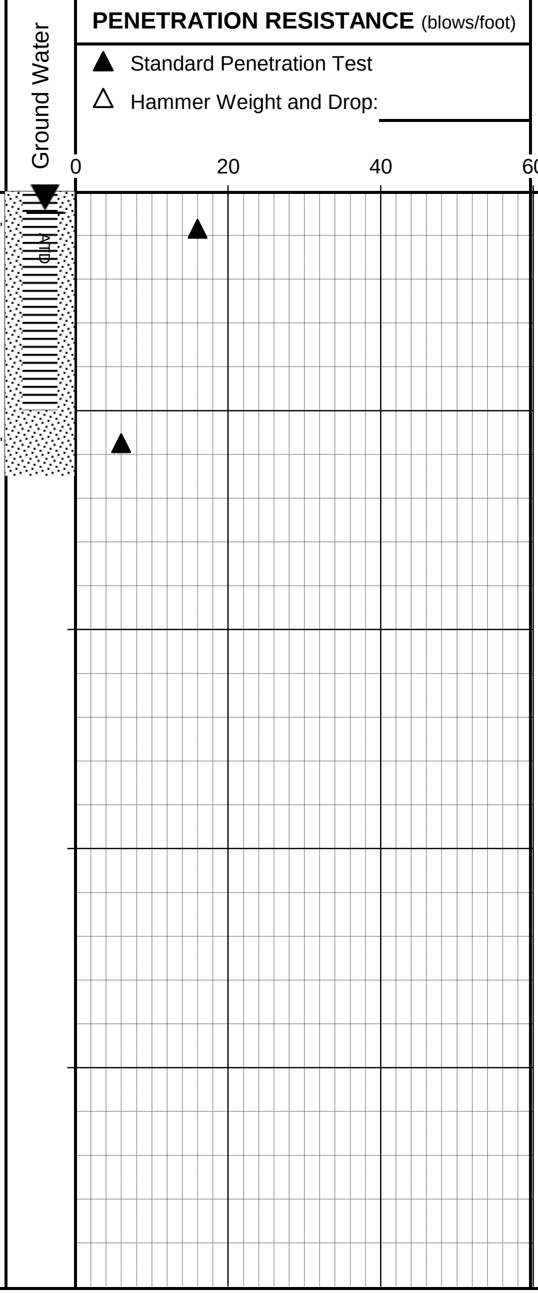
Boring Location: See Site and Exploration Plan
Top Elevation: 127 feet above mean sea
Date Drilled: 10/12/2023
Drilling Company: Holocene
Drilling Method: Hollow Stem Auger
Drill Rig: Diedrich D-50
Bore Hole Dia.: 8-inch
Hammer Type: Automatic
Logged by: KRN

B-1

SOIL DESCRIPTION

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

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Blow Counts	Testing
25	Medium dense, saturated, golden brown, SAND, trace to some silt. No odors, no staining, PID <1 ppmv.	S-7	18"	16	
30	Loose, saturated, golden brown, SAND, trace to some silt.	S-8	18"	6	
	Boring completed at approximately 31.5 feet. Groundwater was encountered at approximately 25 feet below ground surface at time of drilling. Ecology Well Tag ID BPX 514.				
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
200W = 200 Wash Analysis
Consol. = Consolidation Test
Att. = Atterberg Limits

Arlington Commercial Development
7530 204th Street NE
Arlington, WA

Date: October 2023
Project No.: 2760.01

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

BORING LOG: B-1

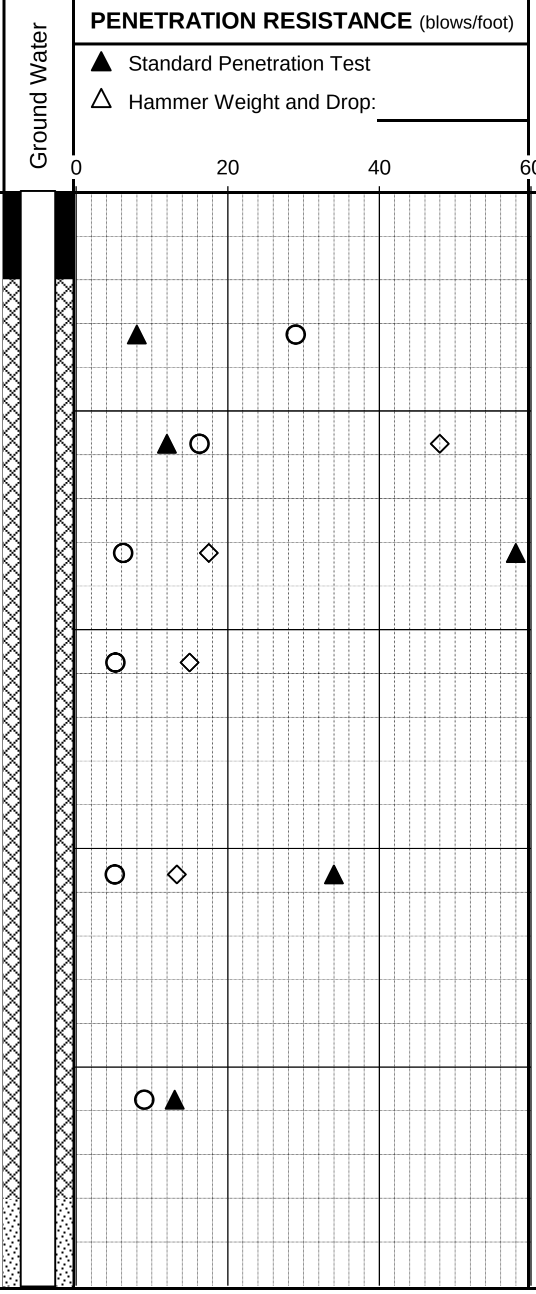
Boring Location:	See Site and Exploration Plan	Drilling Company:	Holocene	Bore Hole Dia.:	8-inch
Top Elevation:	129 feet above mean sea	Drilling Method:	Hollow Stem Auger	Hammer Type:	Automatic
Date Drilled:	10/12/2023	Drill Rig:	Diedrich D-50	Logged by:	KRN

B-2

SOIL DESCRIPTION

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

Depth (ft)	SOIL DESCRIPTION	Sample Number	Recovery (In.)	Blow Counts	Testing
0	Approximately 2-inches of asphalt over moist, brown SAND, with gravel to gravelly SAND, trace silt. (Marysville Sand Member)				
	Loose, moist, dark brown, silty SAND to sandy SILT, some fine roots. No odors, no staining, PID <1 ppmv. Grades to brown, decreasing roots, trace fine gravel.	S-1	18"	8	
5	Medium dense, moist, brown, silty SAND, trace gravel, trace roots. No odors, no staining, PID <1 ppmv.	S-2	18"	12	GSA
	Very dense, moist, light brown to brown, gravelly SAND, with silt. No odors, no staining, PID <1 ppmv.	S-3	18"	58	GSA CEC ORG
10	Very dense, moist, brown to dark brown, GRAVEL, with sand and silt. No odors, no staining, PID <1 ppmv.	S-4	18"	63	GSA
15	Dense, moist, brown, GRAVEL, with sand and silt. No odors, no staining, PID <1 ppmv.	S-5	18"	34	GSA
20	Medium dense, moist, golden brown, SAND with trace to some silt. No odors, no staining, PID <1 ppmv.	S-6	18"	13	
25					



SAMPLE LEGEND

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

GROUNDWATER LEGEND

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

◇ % Fines (<0.075 mm)

○ % Water (Moisture) Content

Plastic Limit ├──○──┤ Liquid Limit

Natural Water Content

TESTING KEY

GSA = Grain Size Analysis

200W = 200 Wash Analysis

Consol. = Consolidation Test

Att. = Atterberg Limits

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7530 204th Street NE
Arlington, WA

Date: October 2023

Project No.: 2760.01

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

BORING LOG: B-2

Boring Location: See Site and Exploration Plan
Top Elevation: 129 feet above mean sea
Date Drilled: 10/12/2023

Drilling Company: Holocene
Drilling Method: Hollow Stem Auger
Drill Rig: Diedrich D-50

Bore Hole Dia.: 8-inch
Hammer Type: Automatic
Logged by: KRN

B-2

SOIL DESCRIPTION

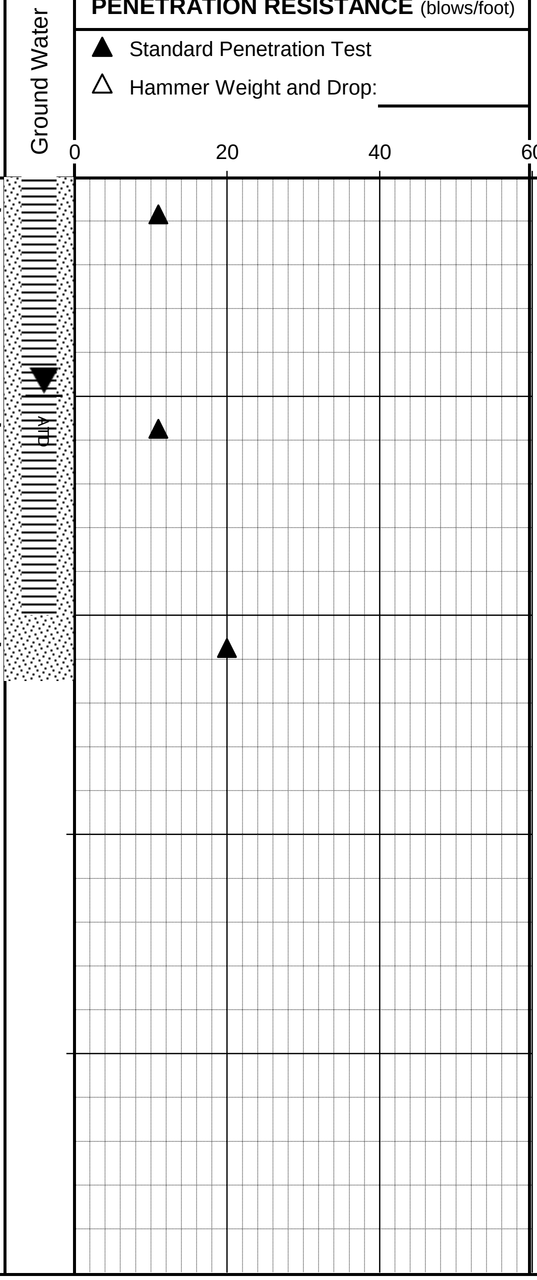
The stratification lines represent the approximate boundaries between soil types. The transition may be gradual. Refer to report text and appendices for additional information.

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Blow Counts	Testing
25	Medium dense, wet to saturated, golden brown, SAND with trace to some silt. No odors, no staining, PID <1 ppmv.	S-7	18"	11	
30	Medium dense, saturated, golden brown, SAND, trace to some silt. No odors, no staining, PID <1 ppmv.	S-8	18"	11	
35	Medium dense, saturated, golden brown, SAND, trace to some silt. No odors, no staining, PID <1 ppmv.	S-9	18"	20	
	Boring completed at approximately 36.5 feet. Groundwater was encountered at approximately 30 feet below ground surface at time of drilling. Ecology Well Tag ID BPX 515.				
40					
45					
50					

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



SAMPLE LEGEND

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

GROUNDWATER LEGEND

▨	Clean Sand
⊠	Bentonite
■	Grout/Concrete
☰	Screened Casing
☐	Blank Casing
▼ 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

Att. = Atterberg Limits

Arlington Commercial Development
7530 204th Street NE
Arlington, WA

Date: October 2023

Project No.: 2760.01

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

BORING LOG: B-2

Boring Location: See Site and Exploration Plan
Top Elevation: 130 feet above mean sea
Date Drilled: 10/12/2023
Drilling Company: Holocene
Drilling Method: Hollow Stem Auger
Drill Rig: Diedrich D-50
Bore Hole Dia.: 8-inch
Hammer Type: Automatic
Logged by: KRN

B-3

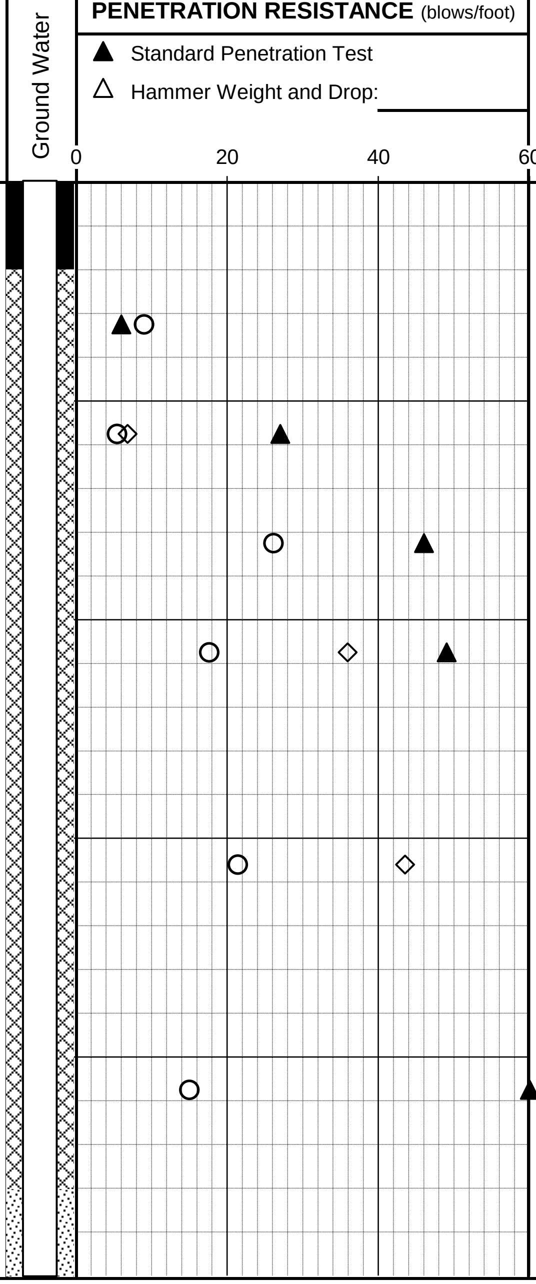
SOIL DESCRIPTION

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

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test
△ Hammer Weight and Drop: ______

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Blow Counts	Testing
0	Grass over approximately 3 inches of moist, brown, SAND with silt, abundant grass roots. (Marysville Sand member)				
	Loose, moist, brown, SAND with silt to silty SAND, some fine roots. No odors, no staining, PID <1 ppmv.	S-1	18"	6	
5	Medium dense, moist, dark brown, SAND with gravel, some silt. No odors, no staining, PID <1 ppmv.	S-2	18"	27	GSA
	Dense, moist to wet, brown, sandy SILT, trace gravel. No odors, no staining, PID <1 ppmv.	S-3	6"	46	GSA
10	Dense, moist to wet, brown, silty SAND, with gravel. No odors, no staining, PID <1 ppmv. Grades to gravelly SAND, some silt.	S-4	12"	49	GSA
15	Very dense, wet, brown, silty SAND some gravel. No odors, no staining, PID <1 ppmv.	S-5	9"	82	GSA
20	Very dense, wet, brown, SAND with silt, decreasing fines to some silt. No odors, no staining, PID <1 ppmv.	S-6	18"	91/11"	
25					



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

GROUNDWATER LEGEND
▨ Clean Sand
☒ Bentonite
■ Grout/Concrete
☰ Screened Casing
☐ Blank Casing
▼ Groundwater level at time of drilling (ATD) or on date of measurement.
11/2/12

◇ % Fines (<0.075 mm)
○ % Water (Moisture) Content
Plastic Limit ├──○──┤ Liquid Limit
Natural Water Content

TESTING KEY
GSA = Grain Size Analysis
200W = 200 Wash Analysis
Consol. = Consolidation Test
Att. = Atterberg Limits

Arlington Commercial Development
7530 204th Street NE
Arlington, WA

Date: October 2023
Project No.: 2760.01

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

BORING LOG: B-3

Boring Location: See Site and Exploration Plan
Top Elevation: 130 feet above mean sea
Date Drilled: 10/12/2023

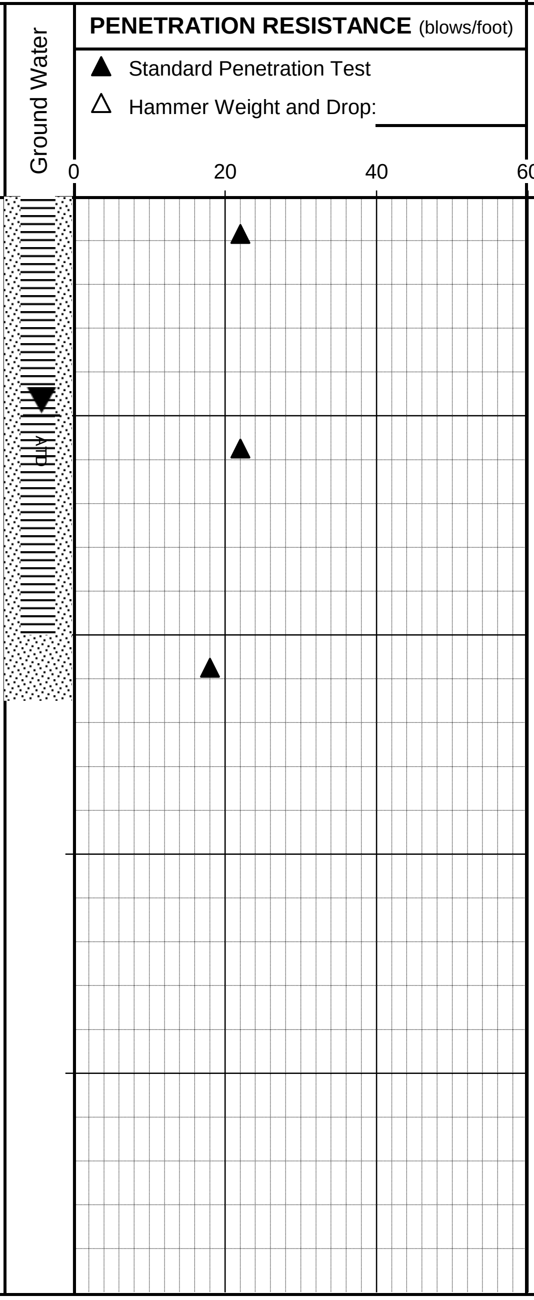
Drilling Company: Holocene
Drilling Method: Hollow Stem Auger
Drill Rig: Diedrich D-50

Bore Hole Dia.: 8-inch
Hammer Type: Automatic
Logged by: KRN

B-3

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

Depth (ft)	SOIL DESCRIPTION	Sample Number	Recovery (In.)	Blow Counts	Testing
25	Medium dense, wet, brown, SAND, trace to some silt. No odors, no staining, PID <1 ppmv.	S-7	18"	22	
30	Medium dense, saturated, brown, SAND, trace to some silt. No odors, no staining, PID <1 ppmv.	S-8	18"	22	
35	Medium dense, saturated, brown, SAND, trace to some silt. No odors, no staining, PID <1 ppmv.	S-9	18"	18	
	Boring completed at approximately 36.5 feet. Groundwater was encountered at approximately 30 feet below ground surface at time of drilling. Ecology Well Tag ID BPX 516.				
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
200W = 200 Wash Analysis
Consol. = Consolidation Test
Att. = Atterberg Limits

Arlington Commercial Development
7530 204th Street NE
Arlington, WA

Date: October 2023
Project No.: 2760.01

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

BORING LOG: B-3

Boring Location: See Site and Exploration Plan
Top Elevation: 130 feet above mean sea
Date Drilled: 10/12/2023

Drilling Company: Holocene
Drilling Method: Hollow Stem Auger
Drill Rig: Diedrich D-50

Bore Hole Dia.: 8-inch
Hammer Type: Automatic
Logged by: KRN

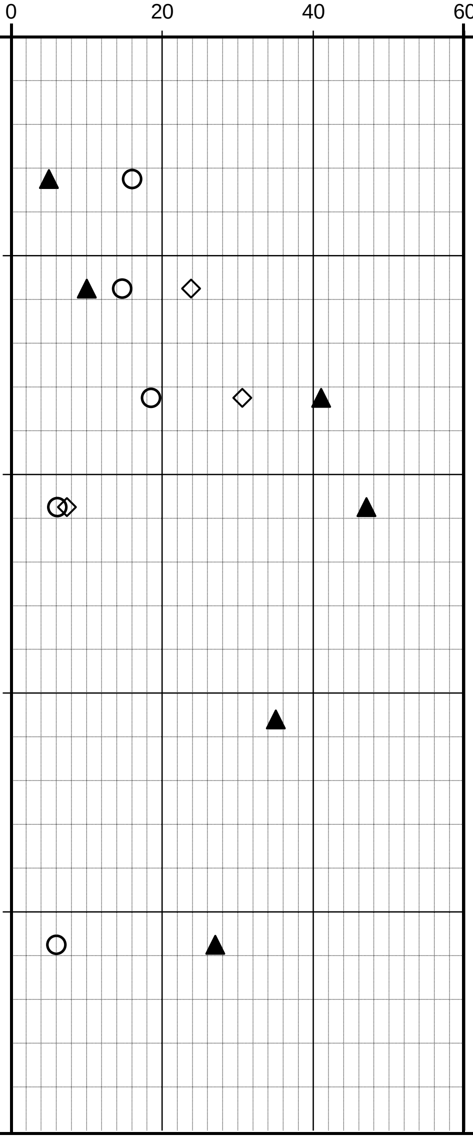
B-4

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

Depth (ft)	SOIL DESCRIPTION	Sample Number	Recovery (In.)	Ground Water	Blow Counts	Testing
0	Grass over approximately 4 inches of moist, brown, SAND with silt, abundant roots. (Marysville Sand Member)					
	Loose, moist, brown, SAND, with to some gravel, some silt, trace roots. No odors, no staining, PID <1 ppmv.	S-1	18"		5	
5	Loose, moist, brown, SAND, with silt, trace gravel. No odors, no staining, PID <1 ppmv.	S-2	18"		10	GSA
	Dense, moist, dark brown, silty SAND, trace gravel. No odors, no staining, PID <1 ppmv.	S-3	18"		41	GSA
10	Dense, moist, light brown, gravelly SAND, some silt. No odors, no staining, PID <1 ppmv.	S-4	9"		47	GSA CEC ORG
15	Dense, moist, light brown, gravelly SAND to sandy GRAVEL. No odors, no staining, PID <1 ppmv.	S-5	12"		35	
20	Medium dense, moist, light brown, SAND with gravel, trace silt. No odors, no staining, PID <1 ppmv.	S-6	12"		27	
25						

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test
△ Hammer Weight and Drop:



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

GROUNDWATER LEGEND
▨ Clean Sand
☒ Bentonite
■ Grout/Concrete
☰ Screened Casing
☐ Blank Casing
▼ Groundwater level at time of drilling (ATD) or on date of measurement.
11/2/12

◇ % Fines (<0.075 mm)
○ % Water (Moisture) Content
Plastic Limit ├──○──┤ Liquid Limit
Natural Water Content

TESTING KEY
GSA = Grain Size Analysis
200W = 200 Wash Analysis
Consol. = Consolidation Test
Att. = Atterberg Limits

Arlington Commercial Development
7530 204th Street NE
Arlington, WA

Date: October 2023
Project No.: 2760.01

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

BORING LOG: B-4

Boring Location: See Site and Exploration Plan
Top Elevation: 130 feet above mean sea
Date Drilled: 10/12/2023

Drilling Company: Holocene
Drilling Method: Hollow Stem Auger
Drill Rig: Diedrich D-50

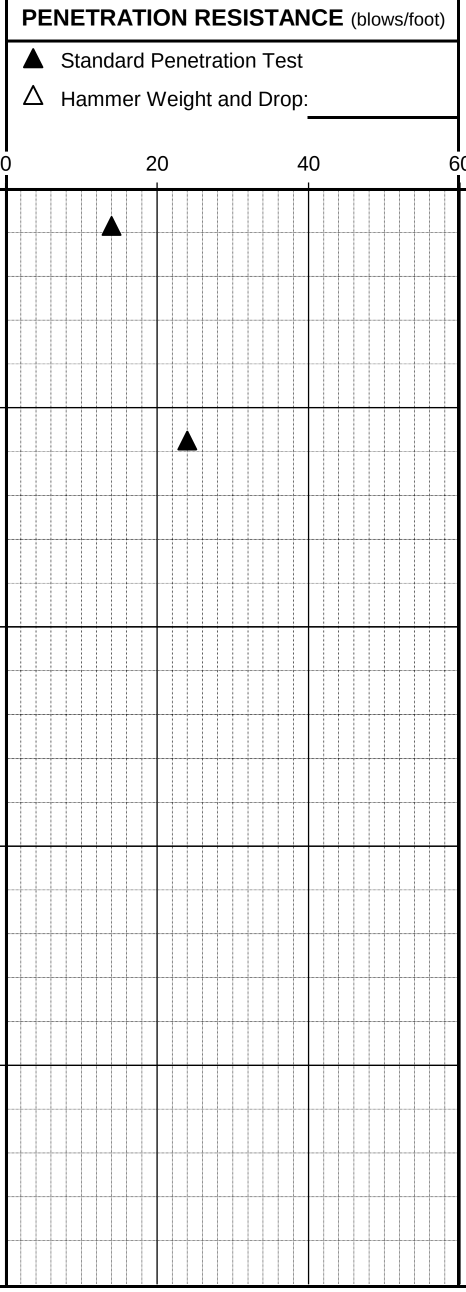
Bore Hole Dia.: 8-inch
Hammer Type: Automatic
Logged by: KRN

B-4

SOIL DESCRIPTION

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

Depth (ft)	Soil Description	Sample Number	Recovery (In.)	Ground Water	Blow Counts	Testing
25	Medium dense, moist, light brown, SAND with gravel, trace silt. No odors, no staining, PID <1 ppmv.	S-7	12"		14	
30	Medium dense, moist to wet, light brown, SAND with gravel, trace silt. No odors, no staining, PID <1 ppmv.	S-8	18"		24	
	Boring completed at approximately 31.5 feet. Groundwater was not encountered at time of drilling. Borehole backfilled with bentonite.					
35						
40						
45						
50						



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

GROUNDWATER LEGEND

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

TESTING KEY
GSA = Grain Size Analysis
200W = 200 Wash Analysis
Consol. = Consolidation Test
Att. = Atterberg Limits

◇ % Fines (<0.075 mm)
○ % Water (Moisture) Content
Plastic Limit ├──○──┤ Liquid Limit
Natural Water Content

Arlington Commercial Development
7530 204th Street NE
Arlington, WA

Date: October 2023
Project No.: 2760.01

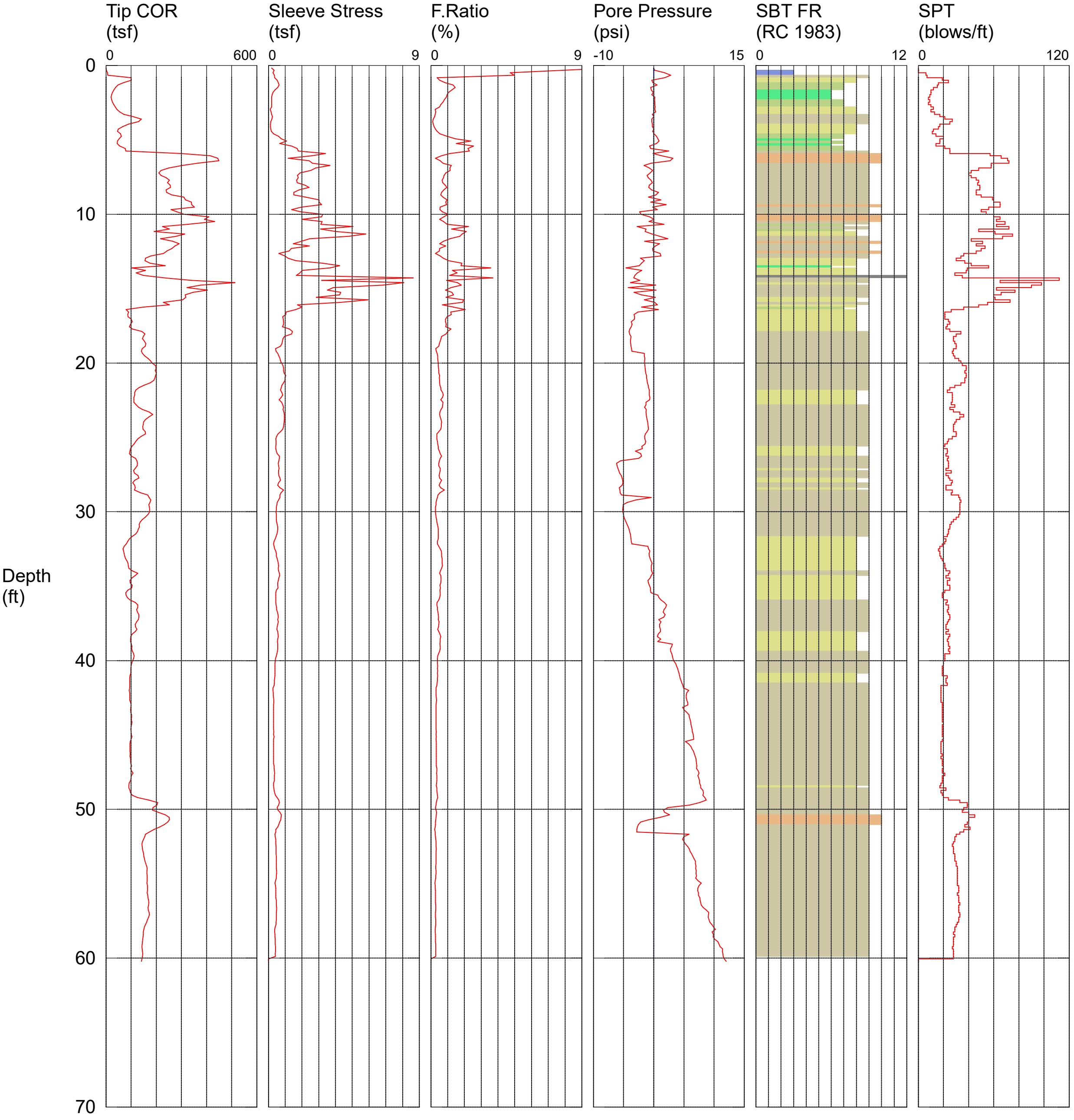
ZipperGeo
Geoprofessional Consultants
19019 36th Ave. W, Suite E
Lynnwood, WA

BORING LOG: B-4

CPT-01

CPT CONTRACTOR: In Situ Engineering
CUSTOMER: Zippergeo
LOCATION: Arlington
JOB NUMBER: 2760.01

OPERATOR: Forinash
CONE ID: DDG1369
TEST DATE: 10/17/2023 9:30:24 AM
PREDRILL: 0ft
BACKFILL: 20% Slurry & Bentonite Chips
SURFACE PATCH: Cold Patch

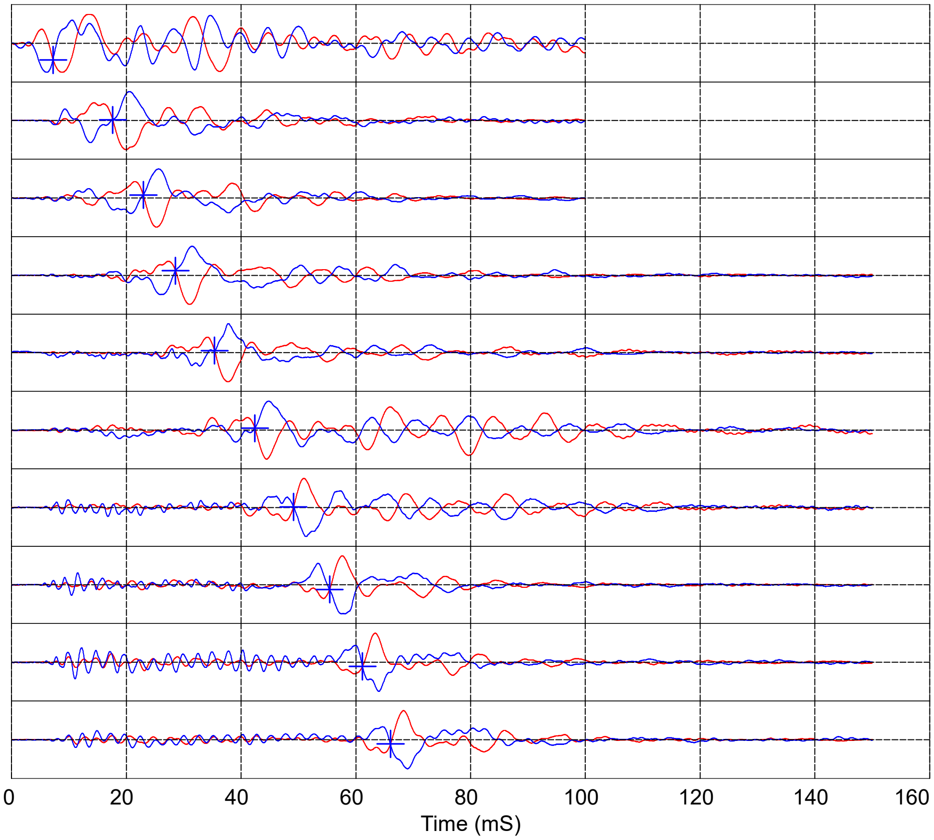


TOTAL DEPTH: 60.203 ft

🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟦 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

HOLE NUMBER: CPT-01



Depth 3.12ft	Ref*	Arrival 7.27mS	Velocity*
Depth 9.68ft	Ref 3.12ft	Arrival 17.66mS	Velocity 577.77ft/S
Depth 16.08ft	Ref 9.68ft	Arrival 22.97mS	Velocity 1181.03ft/S
Depth 22.47ft	Ref 16.08ft	Arrival 28.59mS	Velocity 1127.77ft/S
Depth 28.87ft	Ref 22.47ft	Arrival 35.39mS	Velocity 936.85ft/S
Depth 35.43ft	Ref 28.87ft	Arrival 42.42mS	Velocity 930.45ft/S
Depth 41.83ft	Ref 35.43ft	Arrival 49.14mS	Velocity 950.28ft/S
Depth 48.39ft	Ref 41.83ft	Arrival 55.43mS	Velocity 1041.82ft/S
Depth 54.79ft	Ref 48.39ft	Arrival 61.17mS	Velocity 1112.91ft/S
Depth 60.20ft	Ref 54.79ft	Arrival 66.05mS	Velocity 1107.69ft/S

Hammer to Rod String Distance (ft): 2.49
* = Not Determined

COMMENT:

APPENDIX B
LABORATORY TESTING PROCEDURES AND RESULTS

LABORATORY TESTING PROCEDURES

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

Visual Classification

Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with ASTM D2488. 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 to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM D2216. Moisture contents are presented on the exploration logs in Appendix A.

Grain Size Analysis

A grain size analysis presents the range in diameter of soil particles that comprise a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM: D6913. The results of the grain size determinations for the samples were used in classification of the soils and are presented in this appendix.

Cation Exchange Capacity

Cation exchange capacity tests were completed on representative samples collected from prospective infiltration receptor soils by an independent analytical laboratory in accordance with the Washington State Department of Ecology SW-846 9081 test method. Test results are presented in this appendix.

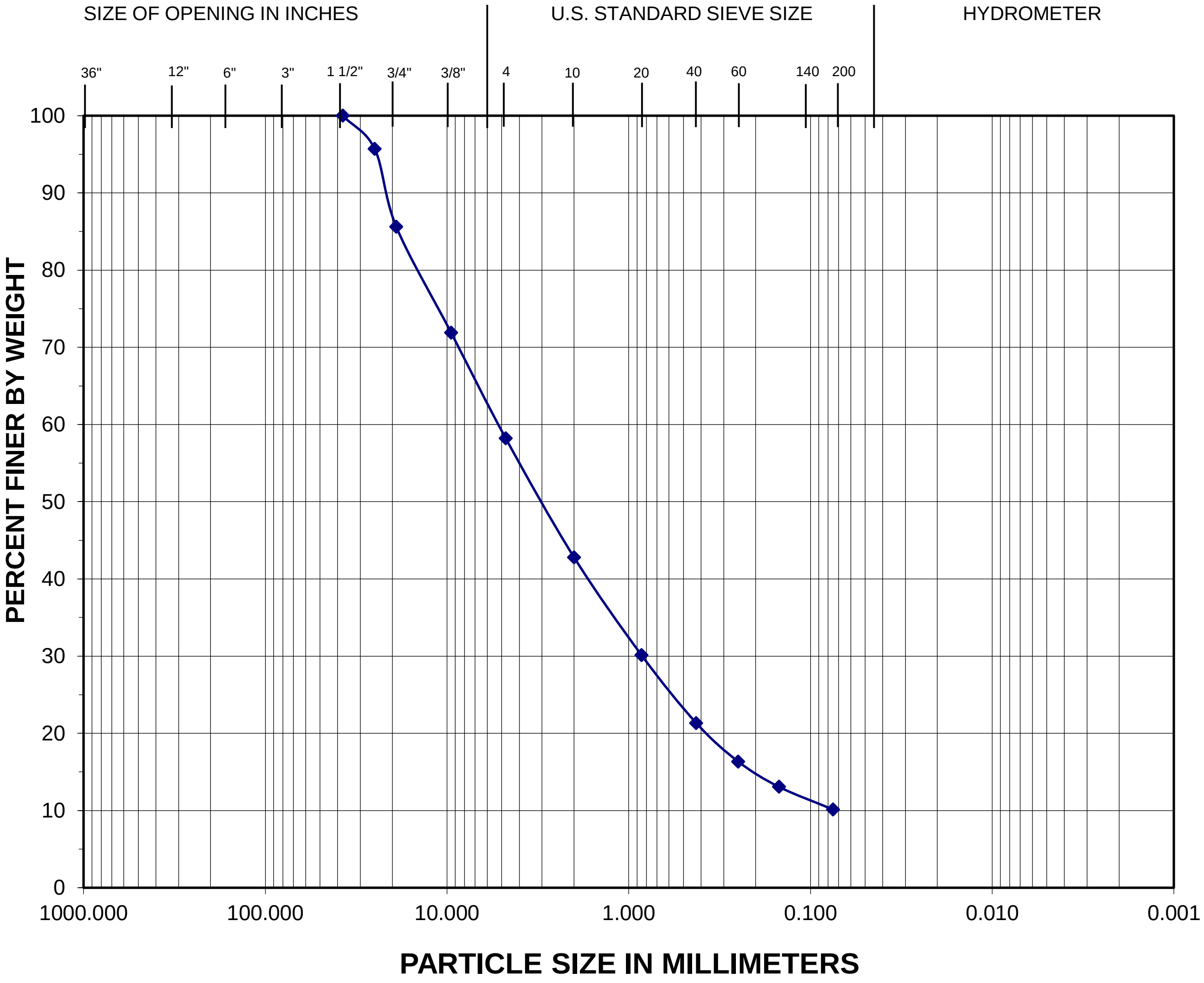
Organic Content Tests

Organic content tests were completed on representative samples collected from prospective infiltration receptor soils by an independent analytical laboratory in accordance with ASTM D2974 test method. Test results 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-3	7.5	5.6	10.1	Gravelly SAND, some silt

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

PROJECT NO: 2760.01

DATE OF TESTING: 10/16/2023

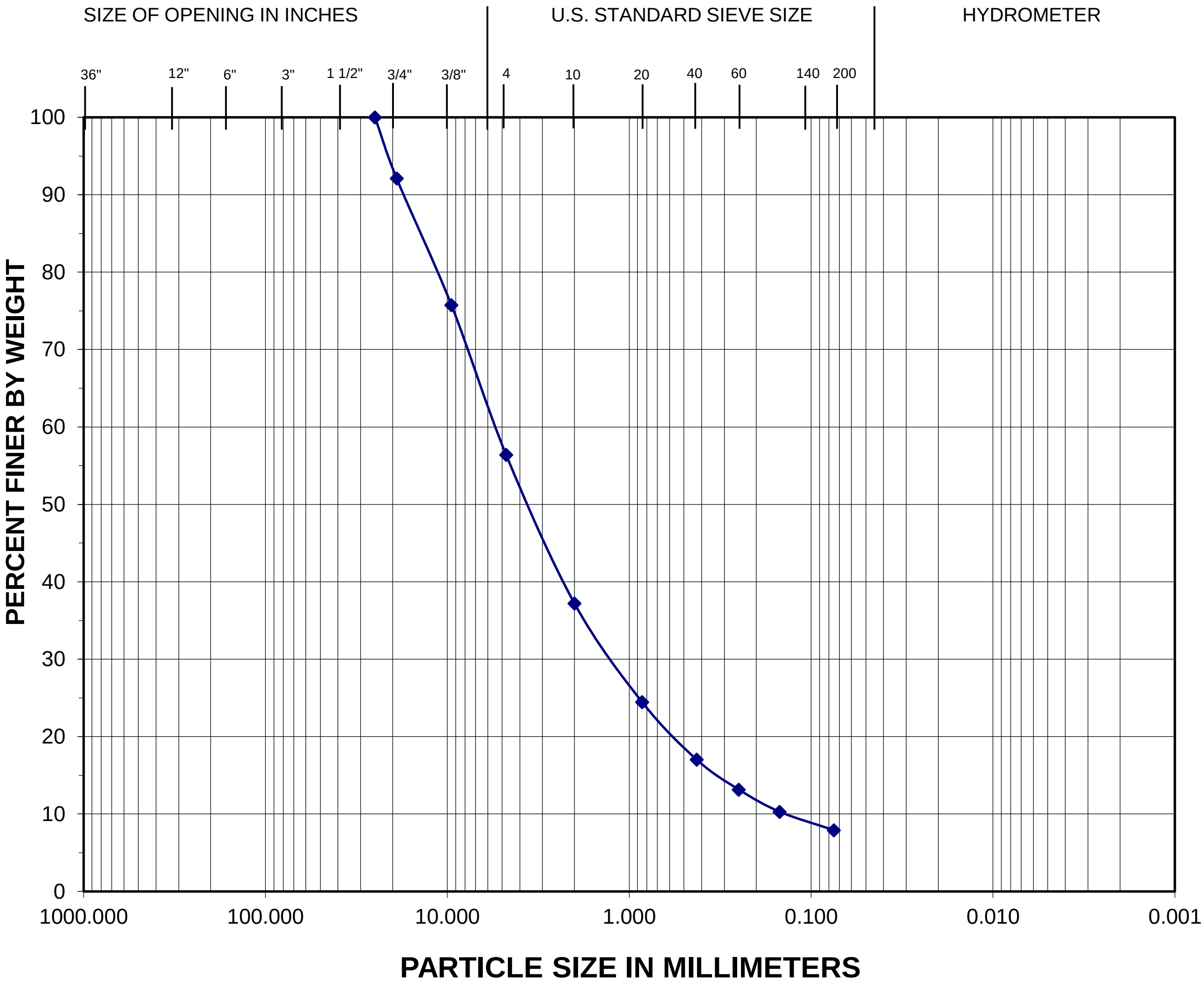
PROJECT NAME:

Arlington Commercial Development

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

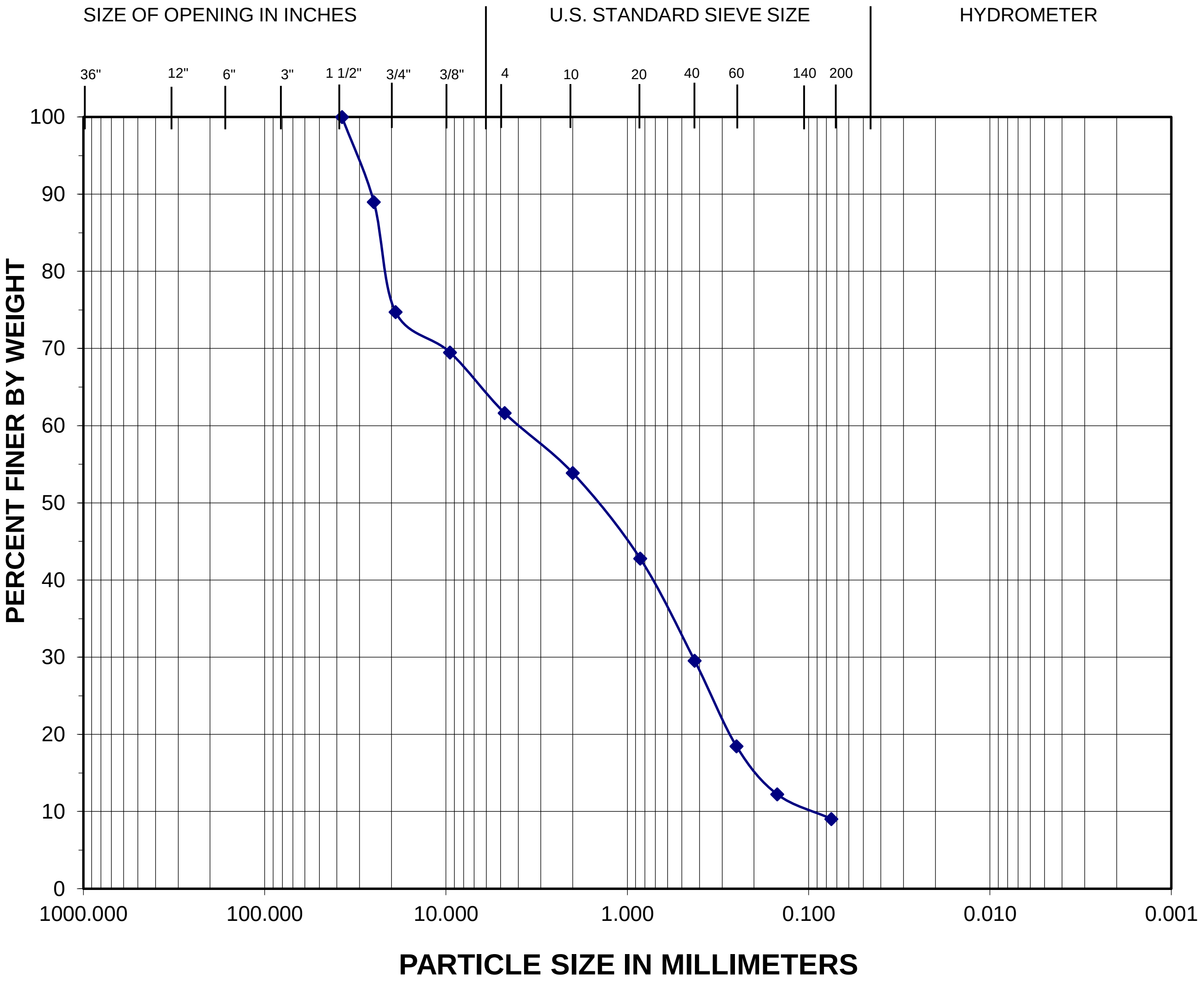
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-4	10	6.8	7.9	Gravelly SAND, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2760.01 DATE OF TESTING: 10/16/2023	PROJECT NAME: Arlington Commercial Development

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-5	15	6.1	9.1	Gravelly SAND, some silt

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

PROJECT NO: 2760.01

DATE OF TESTING: 10/16/2023

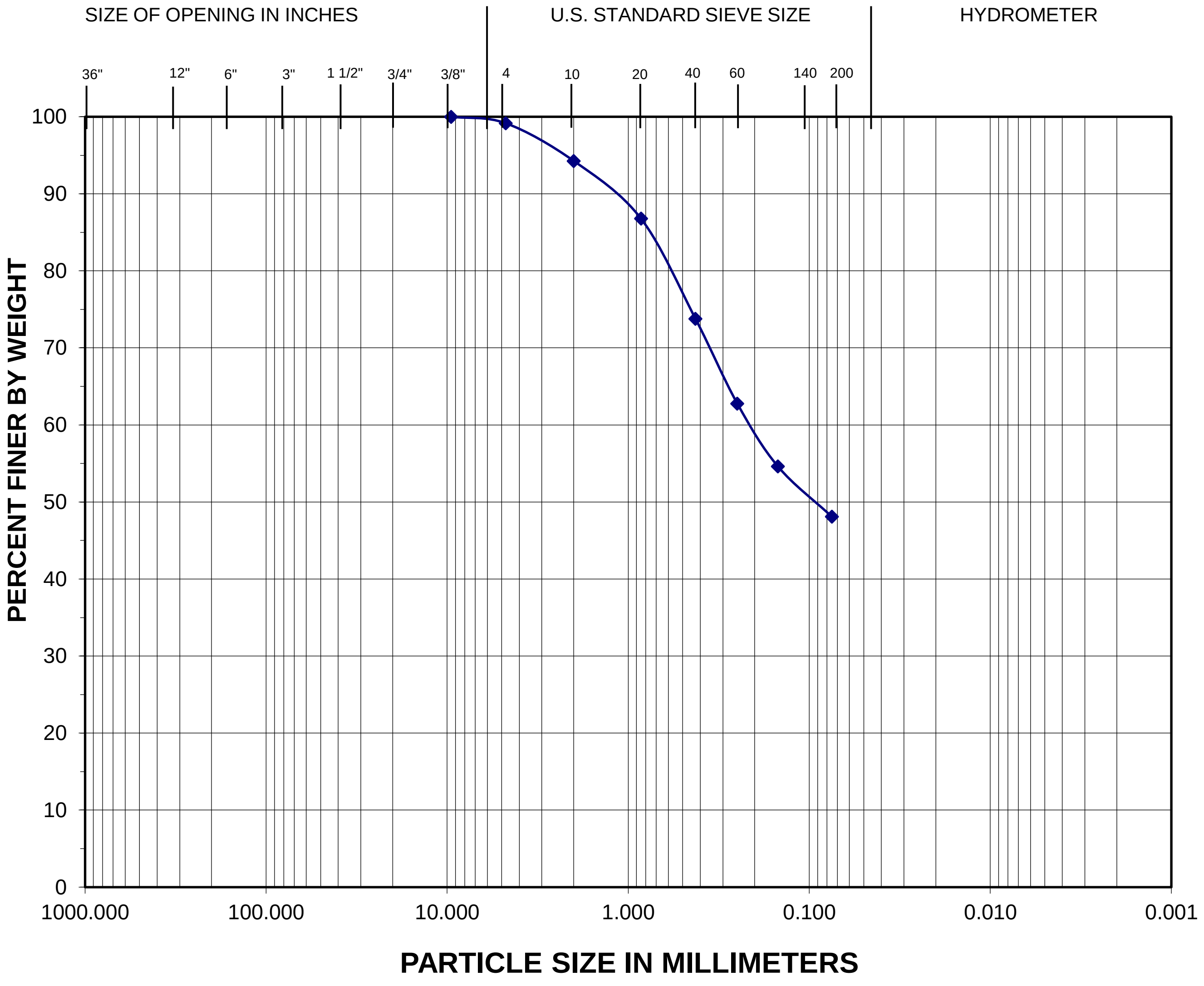
PROJECT NAME:

Arlington Commercial Development

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

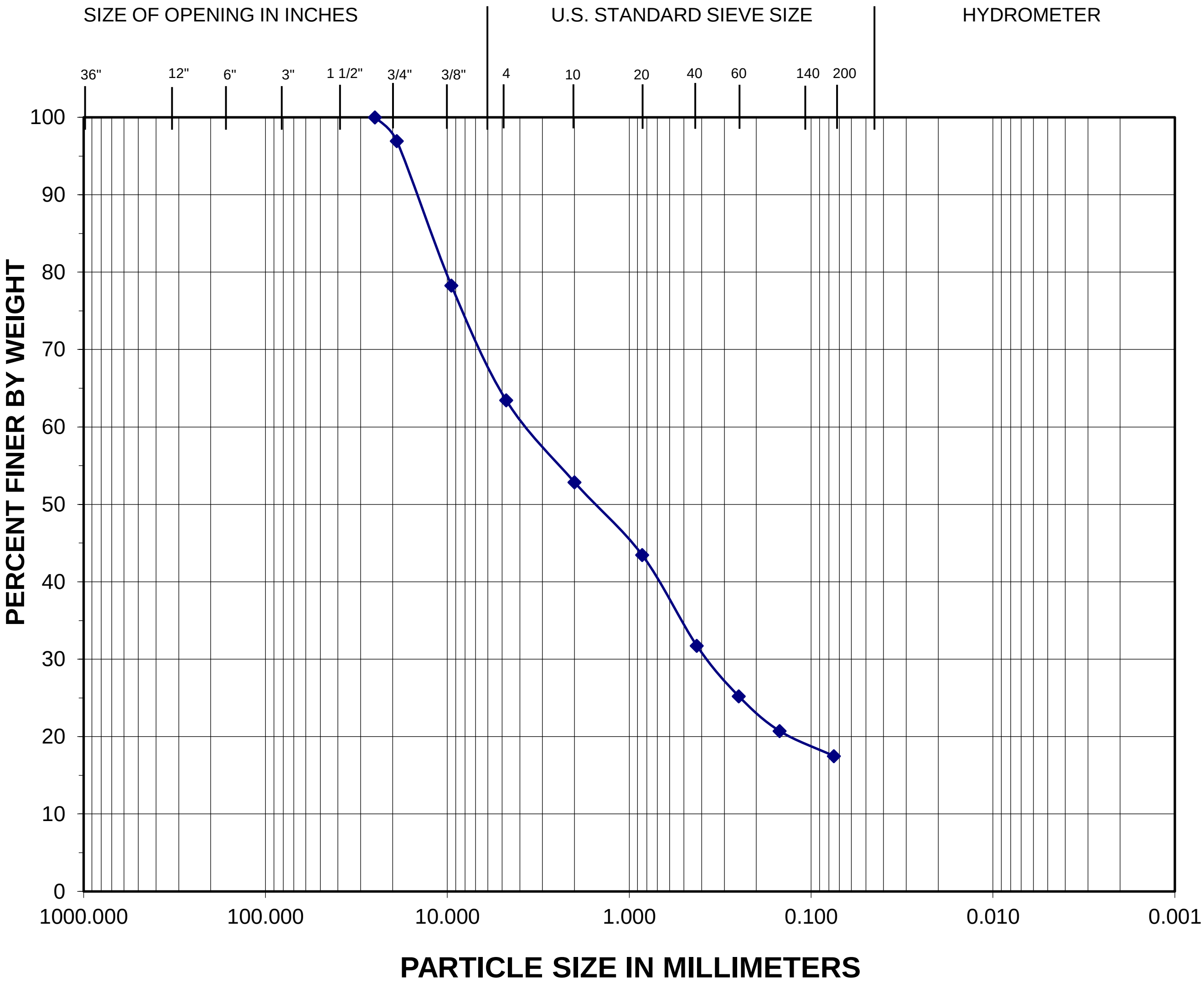
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-2	5	16.3	48.1	Silty SAND, trace grave

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2760.01 DATE OF TESTING: 10/16/2023	PROJECT NAME: Arlington Commerical Development

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	6.2	17.5	Gravelly SAND, with silt

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

PROJECT NO: 2760.01

DATE OF TESTING: 10/16/2023

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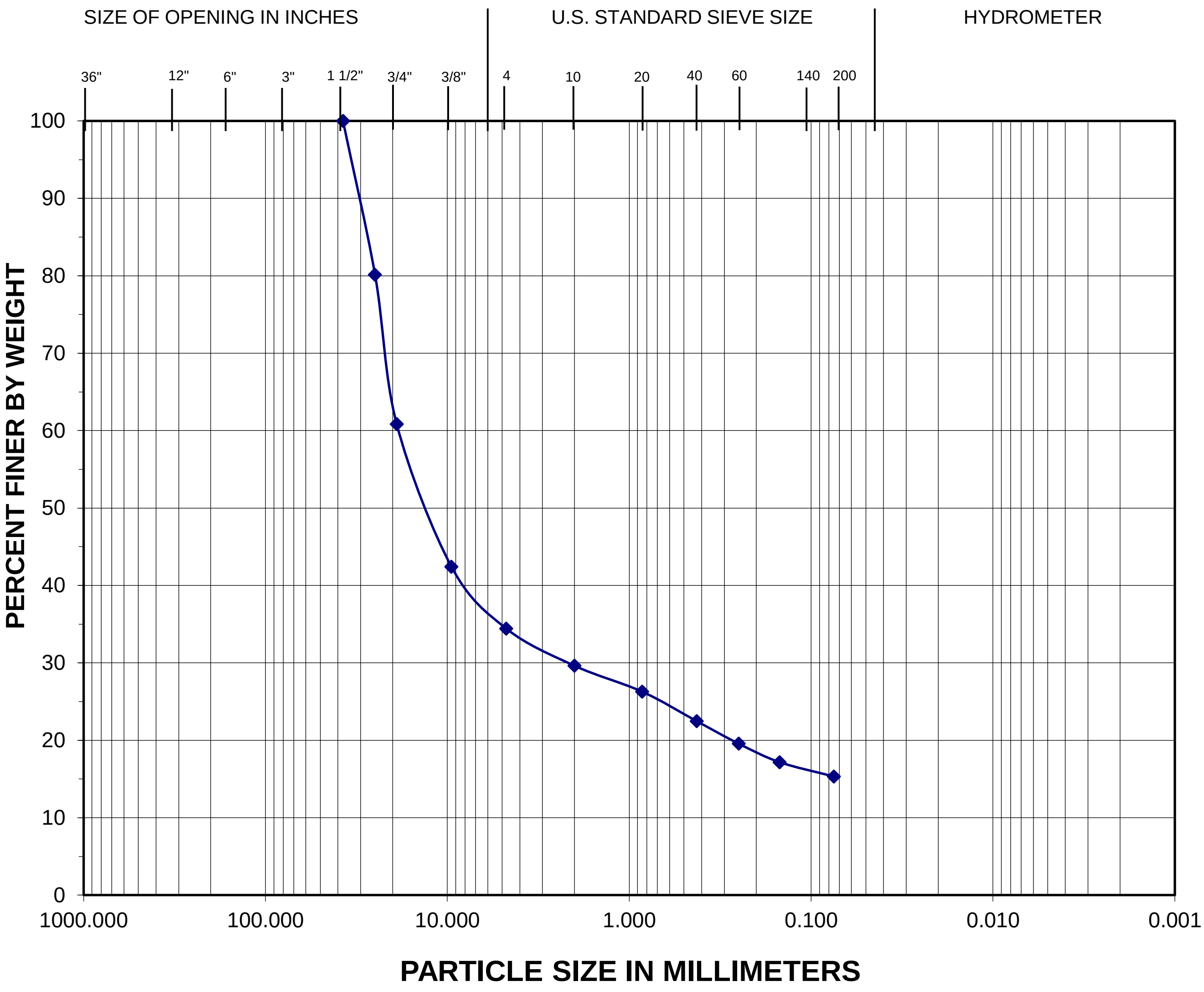
Arlington Commercial Development

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-2	S-4	10	5.2	15.3	GRAVEL, with sand and silt

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

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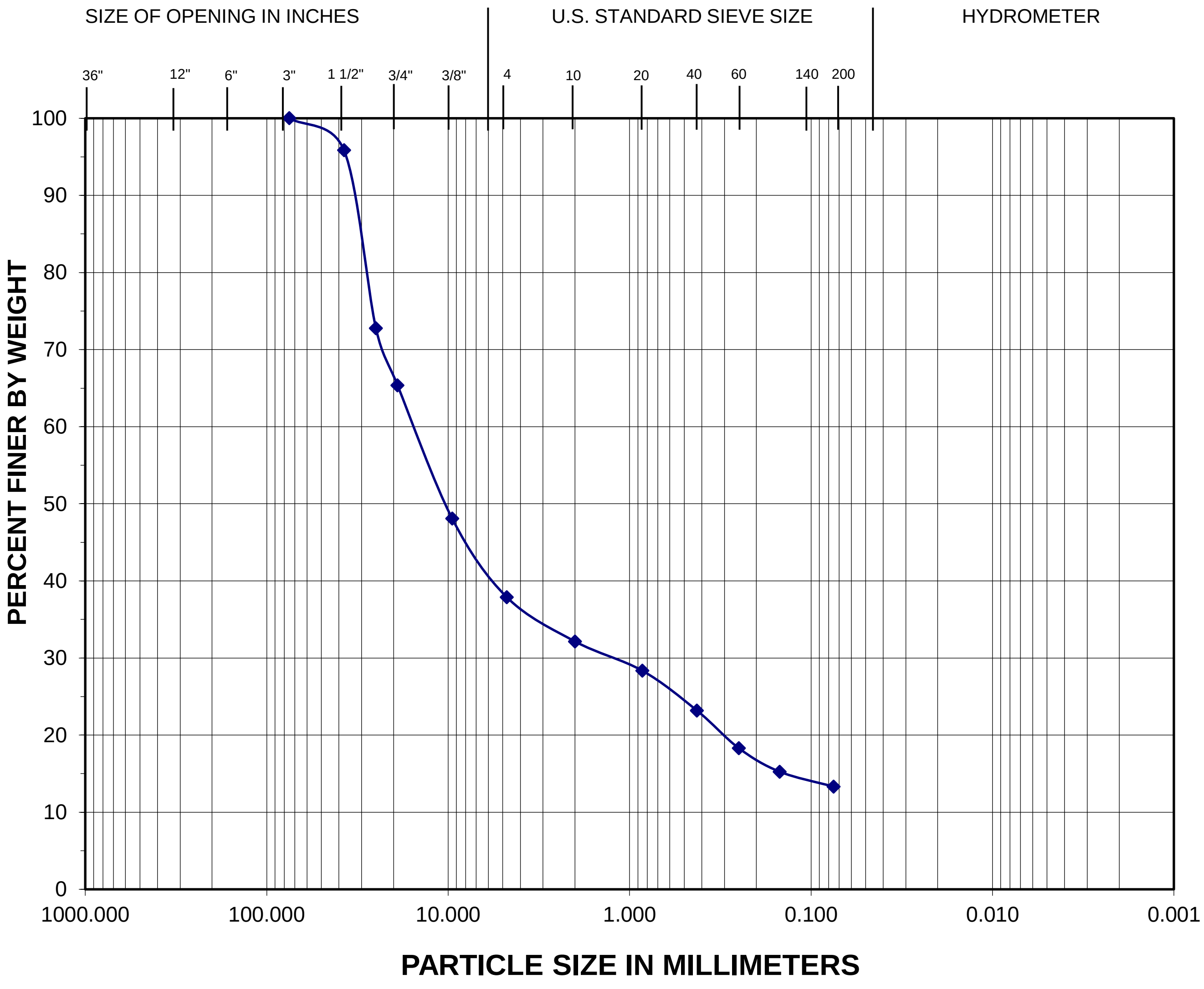
Arlington Commercial Development

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-2	S-5	15	5.1	13.3	GRAVEL, with sand and silt

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

PROJECT NO: 2760.01

DATE OF TESTING: 10/16/2023

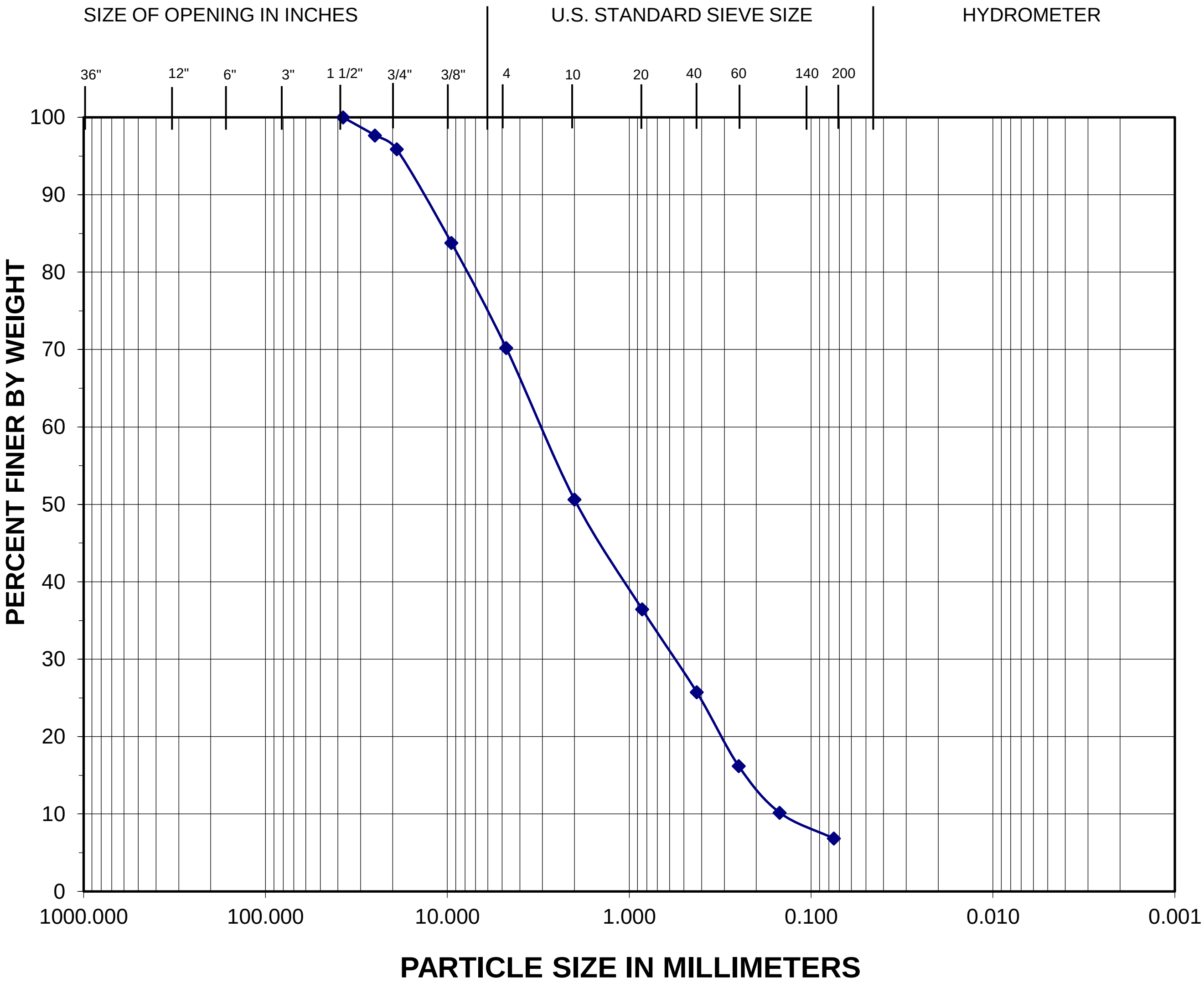
PROJECT NAME:

Arlington Commercial Development

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

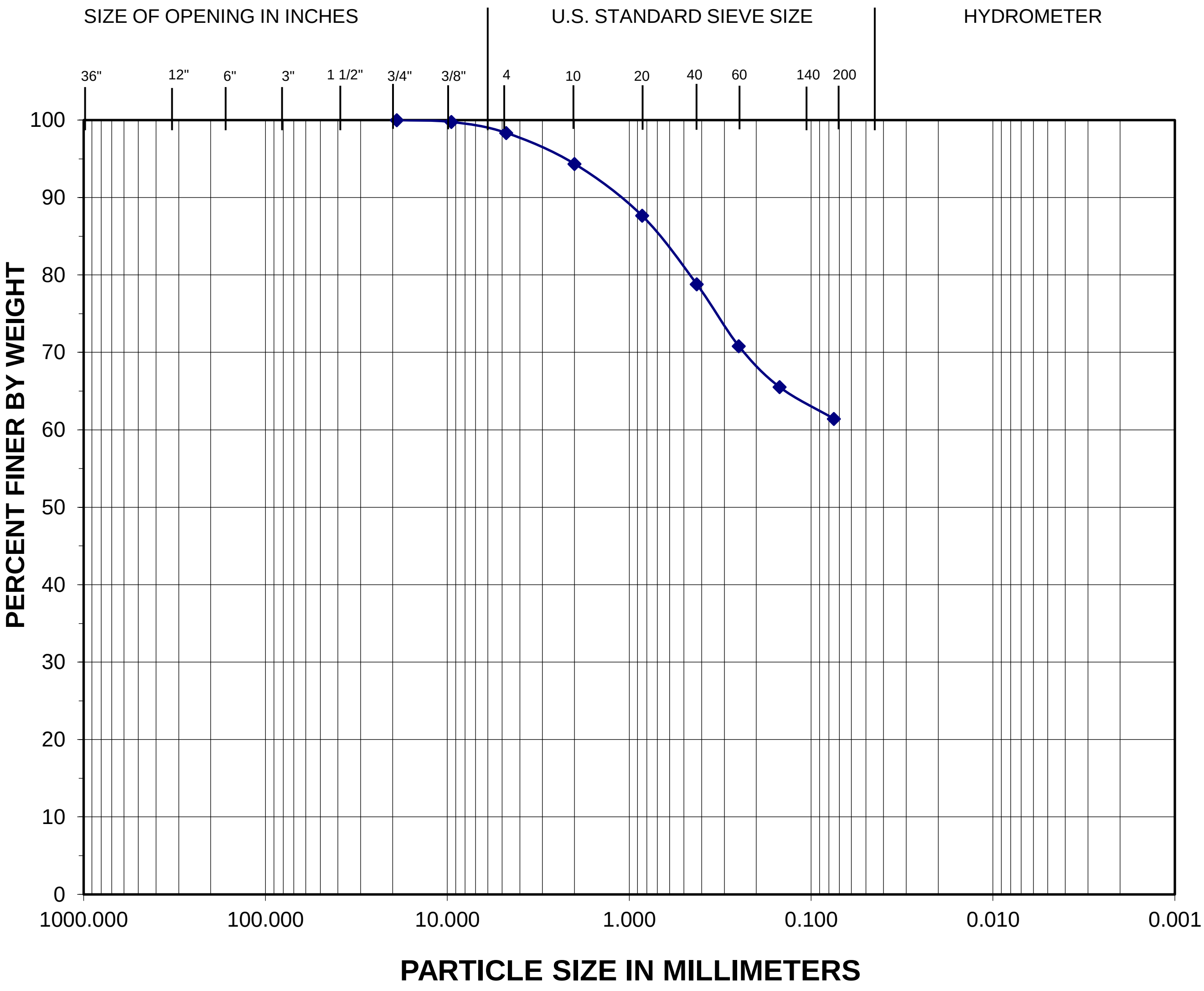
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-3	S-2	5	5.4	6.8	SAND, with gravel, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2760.01 DATE OF TESTING: 10/16/2023	PROJECT NAME: Arlington Commercial Development

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

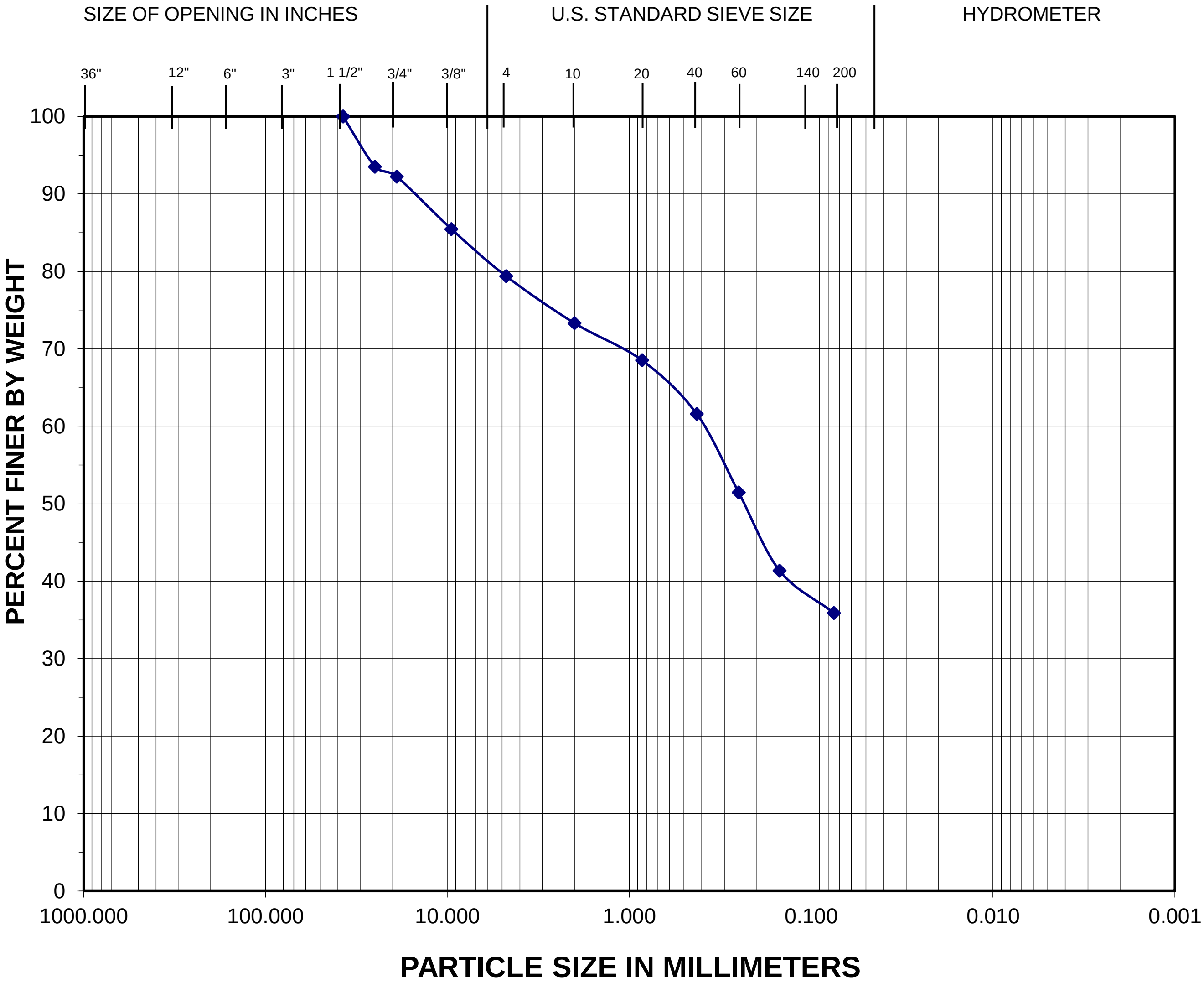
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-3	S-3	7.5	26.1	61.4	Sandy SILT, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2760.01 DATE OF TESTING: 10/16/2023	PROJECT NAME: Arlington Commercial Development

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-3	S-4	10	17.6	35.9	Silty SAND, with gravel

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

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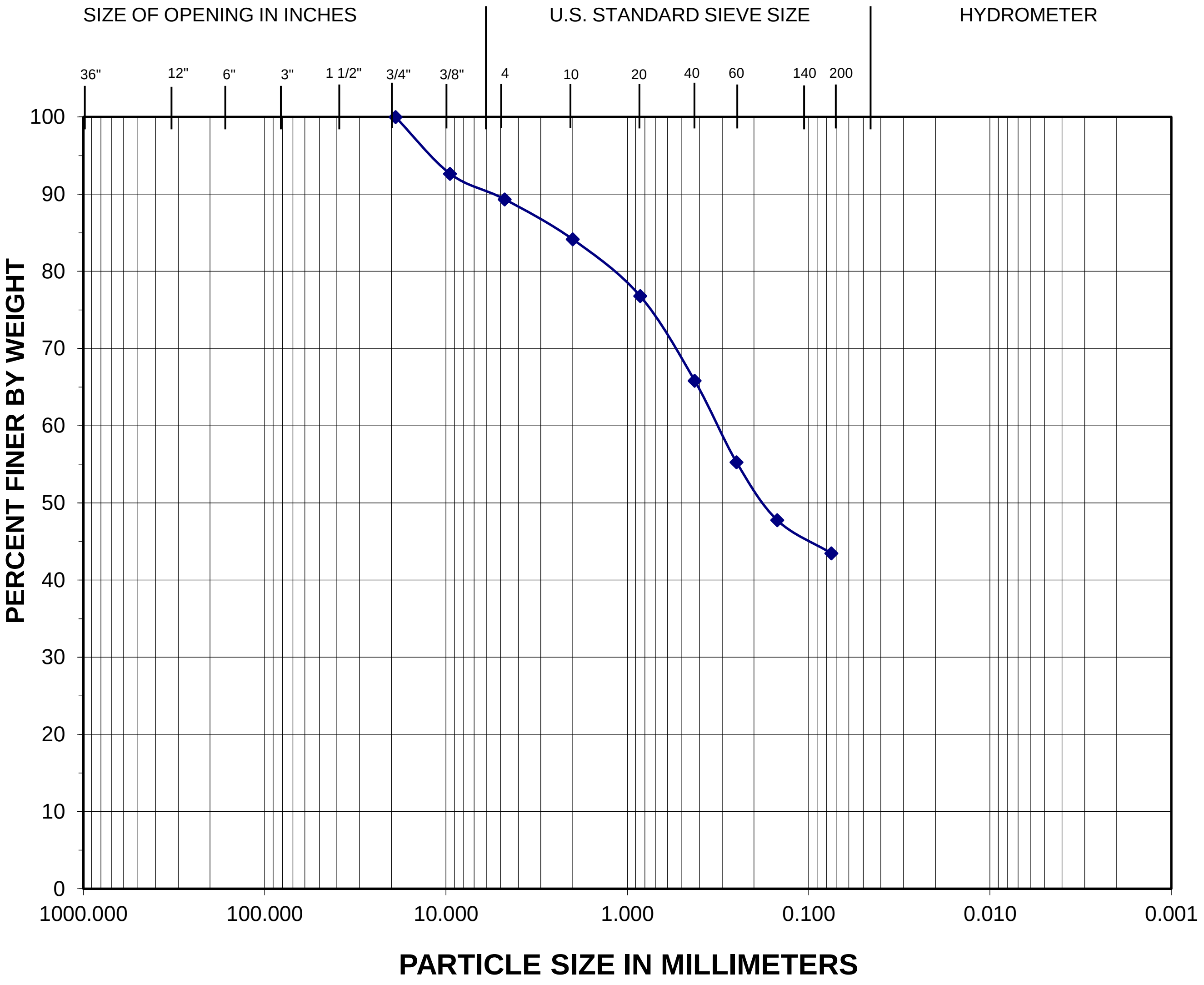
PROJECT NAME:

Arlington Commercial Development

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

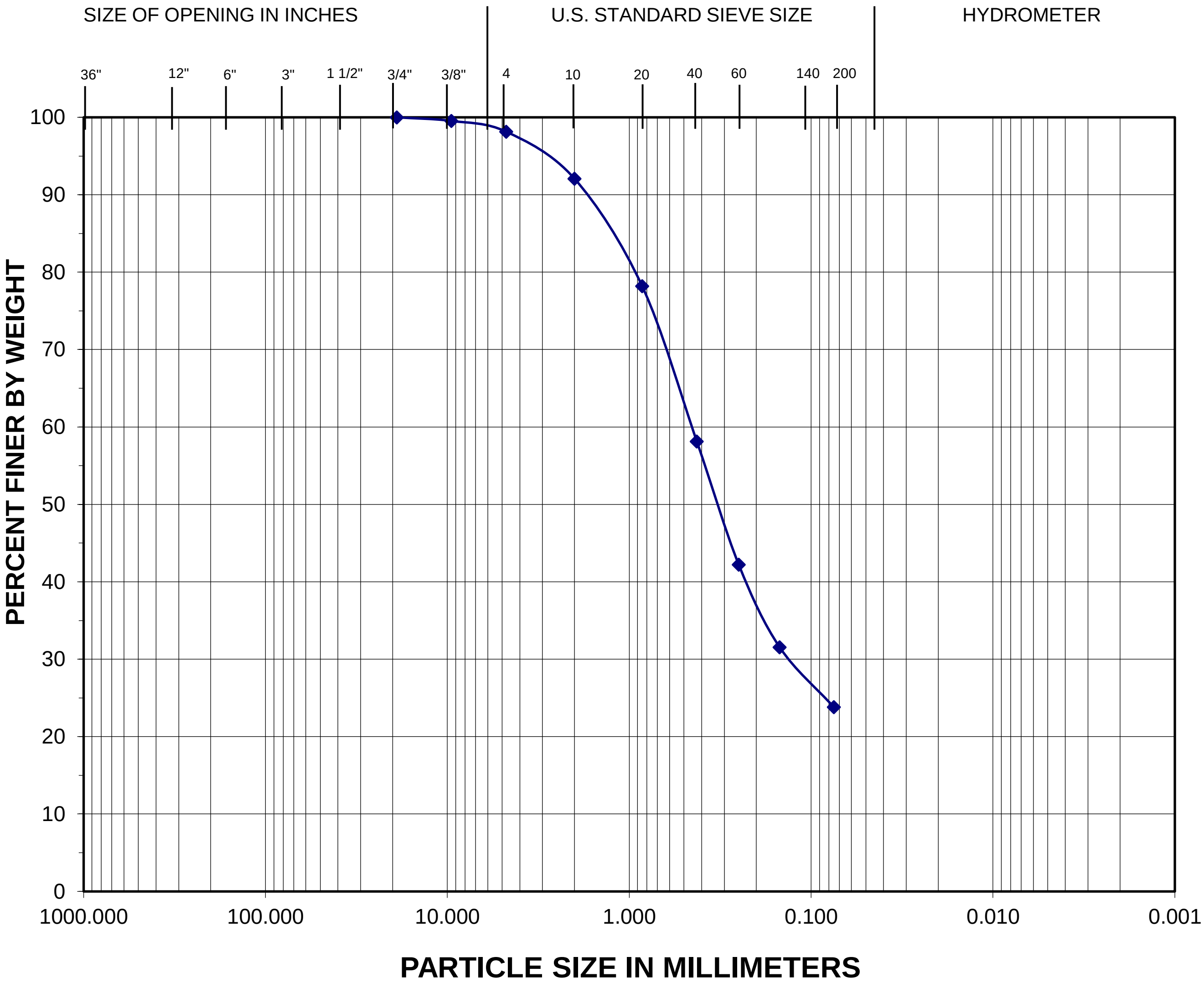
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-3	S-5	15	21.4	43.5	Silty SAND, some gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2760.01 DATE OF TESTING: 10/16/2023	PROJECT NAME: Arlington Commerical Development

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	14.7	23.8	SAND, with silt, trace gravel

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

PROJECT NO: 2760.01

DATE OF TESTING: 10/16/2023

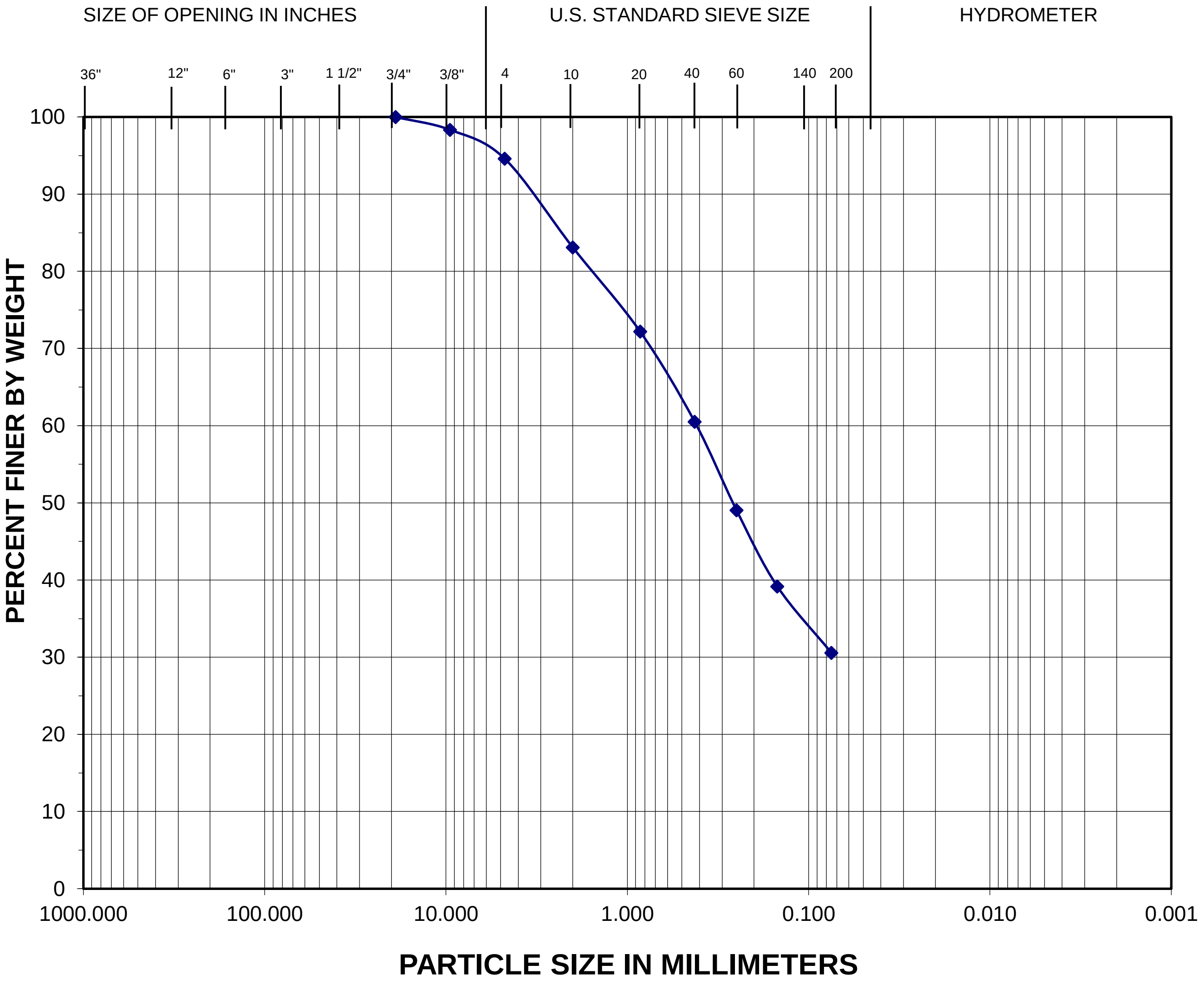
PROJECT NAME:

Arlington Commercial Development

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-3	7.5	18.5	30.6	Silty SAND, trace gravel

Zipper Geo Associates, LLC
Geotechnical and Environmental Consultants

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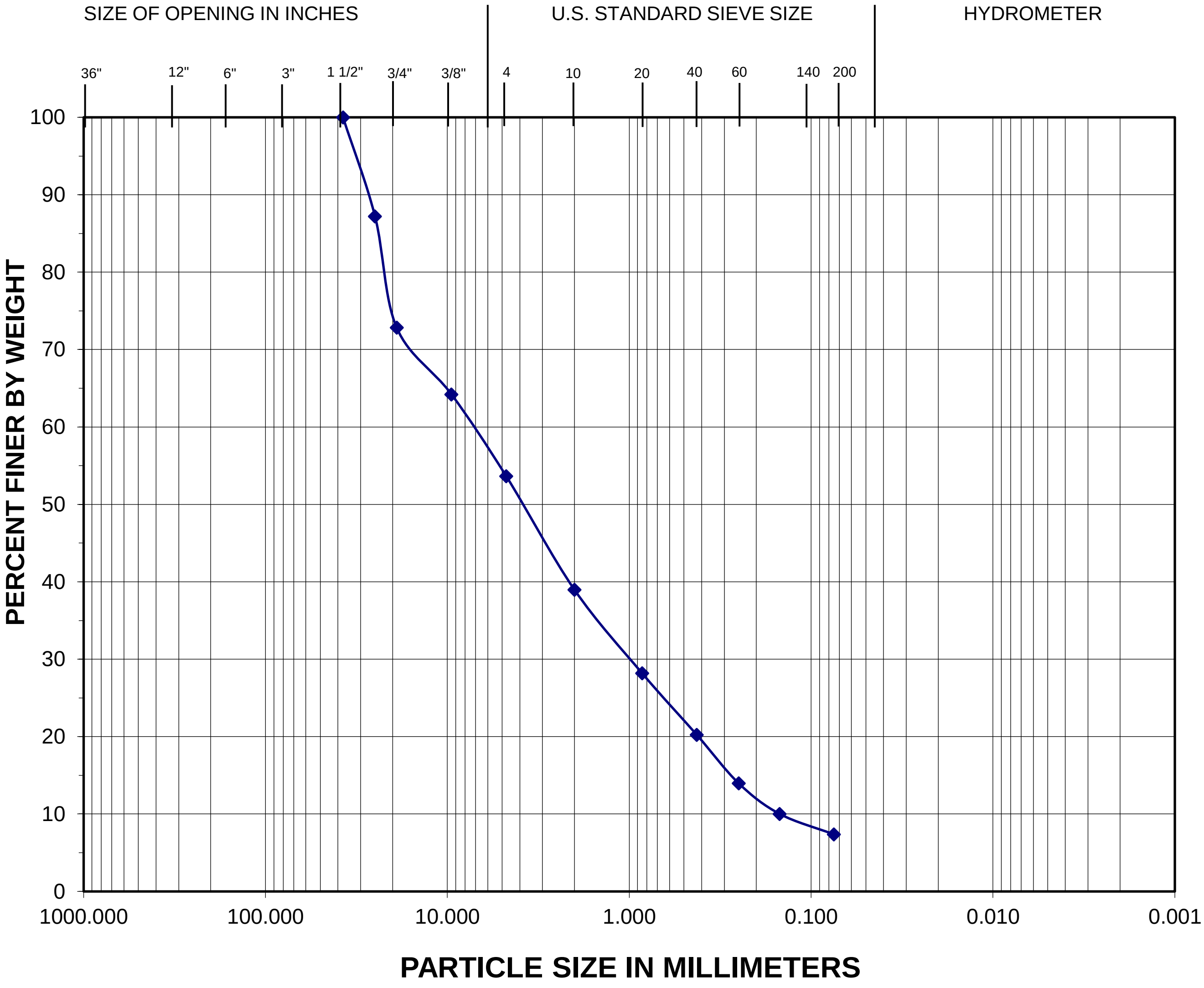
PROJECT NAME:

Arlington Commercial Development

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-5	15	6.1	7.4	Gravelly SAND, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2760.01 DATE OF TESTING: 10/16/2023	PROJECT NAME: Arlington Commercial Development

AMTEST
LABORATORIES

Am Test Inc.
13600 NE 126TH PL
Suite C
Kirkland, WA 98034
(425) 885-1664

Professional
Analytical
Services

Oct 25 2023
ZIPPER GEO ASSOCIATES, LLC
19019 36TH AVENUE W
SUITE E
LYNNWOOD, WA 98036
Attention: JAMES GEORGIS

Dear JAMES GEORGIS:

Enclosed please find the analytical data for your ARLINGTON COMMERCIAL DEVELOPMENT project.

The following is a cross correlation of client and laboratory identifications for your convenience.

CLIENT ID	MATRIX	AMTEST ID	TEST
B-1 S-2 5'	Soil	23-A018155	CONV
B-2 S-3 7.5'	Soil	23-A018156	CONV
B-4 S-4 10'	Soil	23-A018157	CONV

Your samples were received on Monday, October 16, 2023. At the time of receipt, the samples were logged in and properly maintained prior to the subsequent analysis.

The analytical procedures used at AmTest are well documented and are typically derived from the protocols of the EPA, USDA, FDA or the Army Corps of Engineers.

Following the analytical data you will find the Quality Control (QC) results.

Please note that the detection limits that are listed in the body of the report refer to the Practical Quantitation Limits (PQL's), as opposed to the Method Detection Limits (MDL's).

If you should have any questions pertaining to the data package, please feel free to contact me.

Sincerely,

Aaron W Young

Aaron Young
Vice President

Project #: 2760.01

BACT = Bacteriological
CONV = Conventionals

MET = Metals
ORG = Organics

NUT=Nutrients
DEM=Demand

MIN=Minerals

Am Test Inc.
13600 NE 126TH PL
Suite C
Kirkland, WA 98034
(425) 885-1664
www.amtestlab.com

Professional Analytical Services

ANALYSIS REPORT

ZIPPER GEO ASSOCIATES, LLC
19019 36TH AVENUE W
LYNNWOOD, WA 98036
Attention: JAMES GEORGIS
Project Name: ARLINGTON COMMERCIAL DEVELOPMENT
Project #: 2760.01
All results reported on an as received basis.

Date Received: 10/16/23
Date Reported: 10/25/23

AMTEST Identification Number **23-A018155**
Client Identification **B-1 S-2 5'**
Sampling Date **10/16/23**

Conventionals

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANALYST	DATE
Cation Exchange Capacity	5.8	meq/100g		0.5	SW-846 9081	CM	10/19/23

AMTEST Identification Number **23-A018156**
Client Identification **B-2 S-3 7.5'**
Sampling Date **10/16/23**

Conventionals

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANALYST	DATE
Cation Exchange Capacity	6.3	meq/100g		0.5	SW-846 9081	CM	10/19/23

AMTEST Identification Number 23-A018157
Client Identification B-4 S-4 10'
Sampling Date 10/16/23

Conventionals

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANALYST	DATE
Cation Exchange Capacity	6.5	meq/100g		0.5	SW-846 9081	CM	10/19/23

Aaron W Young

Aaron Young
Vice President

QC Summary for sample numbers: 23-A018155 to 23-A018157

DUPLICATES

SAMPLE #	ANALYTE	UNITS	SAMPLE VALUE	DUP VALUE	RPD
23-A017935	Cation Exchange Capacity	meq/100g	2.7	2.9	7.1
23-A018157	Cation Exchange Capacity	meq/100g	6.5	5.1	24.

STANDARD REFERENCE MATERIALS

ANALYTE	UNITS	TRUE VALUE	MEASURED VALUE	RECOVERY
Cation Exchange Capacity	meq/100g	2.0	2.0	100. %
Cation Exchange Capacity	meq/100g	2.0	2.0	100. %

BLANKS

ANALYTE	UNITS	RESULT
Cation Exchange Capacity	meq/100g	< 0.1
Cation Exchange Capacity	meq/100g	< 0.1

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
B-1,S-2	5.0	5.0	8.2	1.1								SM	Olive-brown, silty SAND with gravel
B-2,S-3	7.5	7.5	7.3	1.4								SM	Olive-brown, silty SAND with gravel
B-4,S-4	10.0	10.0	11.5	1.6								SM	Dark olive-brown, silty SAND with gravel

Notes:
1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GEOSCIENCES INC.

Laboratory Testing for Zipper Geo Associates, LLC
Arlington Commercial Development
Client Project No.: 2760.01

SUMMARY OF MATERIAL PROPERTIES

PAGE: 1 of 1

PROJECT NO.: 2012-102 T2600 FIGURE: 1

INDEX MATSUM 2 2012-102 T2600.GPJ 10/24/23