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GEOTECHNICAL REPORT

Little Lake Forest Park Trailhead Improvements

ENUMCLAW, WASHINGTON

May 9, 2022

Shannon & Wilson No: 106797-001

SHANNON & WILSON

Submitted To: SAGE Architectural Alliance
2006 East Miller Street, Suite A
Seattle, WA 98112
Attn: Ms. Valerie Thiel

Subject: GEOTECHNICAL REPORT, LITTLE LAKE FOREST PARK TRAILHEAD IMPROVEMENTS, ENUMCLAW, WASHINGTON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to SAGE Architectural Alliance. Our scope of services was specified in our agreement for consultant services with SAGE Architectural Alliance dated September 1, 2021. This report presents the results of our geotechnical explorations, and recommendations for construction of trailhead improvements at the site. and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON

Nikolas Polzin
Geotechnical Staff

MARTIN W. PAGE
STATE OF WASHINGTON
30308
REGISTERED
PROFESSIONAL ENGINEER

Martin Page
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Martin Page, PE, LEG
Vice President
Geotechnical Engineer

NLP:MWP/nlp

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1 INTRODUCTION

We understand this geotechnical report will be used for design of new trailhead facilities at Little Lake Forest Park. This report should not be used for other purposes without Shannon & Wilson's review. Our scope of services included:

- Visiting the site to perform five shallow hand-auger explorations, Dynamic Cone Penetrometer (DCP) testing, and marking monitor well locations.
- Subcontracting installation of two 20-foot-deep monitoring wells.
- Logging and collecting Standard Penetration Test samples at both monitoring well locations.
- Performing limited laboratory index testing for determination of soil infiltration characteristics.
- Completing geotechnical analyses for pavement, shallow foundation, and stormwater infiltration.
- Preparing this report.

The authorized scope of services was based on your objectives, schedule, budget, and the report purpose. Our scope of services did not include:

- Evaluating the presence of cultural resources or hazardous materials at or around the site.
- Removing observation wells that we installed at the site. It is the Owner's responsibility to properly decommission subsurface installations in accordance with Washington State Department of Ecology regulations when use of the observation wells is no longer needed.

If a service is not specifically indicated in this report, do not assume that it was performed.

2 SITE AND PROJECT DESCRIPTION

Or understanding of the project is based on publicly available information, observations made during our site visits, and information provided to us by SAGE Architectural Alliance. The project site is located at 29103 SE 434th Street, Enumclaw, WA 98022, King County Parcel Number 1920079101. The site is bounded to the north and west by private residential lots, to the south by the closed Enumclaw Landfill, and to the east by the remainder of the

Little Lake Forest Park land. Figure 1 shows the project site in relation to nearby geographic features.

The boundary and topographic survey completed by King County, dated December 17, 2021, shows the site topography is nearly flat, with a slight downward slope to the west. Pronounced slopes are present at the northwest and southwest corners of the parcel. These slopes generally decrease in elevation toward the west with an inclination of between 2 Horizontal to 1 Vertical (2H:1V) to 10H:1V. The site is currently occupied by an existing one-story residential structure, well house, outbuildings, and gravel-surfaced driveways.

Our understanding of the proposed site improvements is based on the development site plan prepared by King County Parks. We understand that the project consists of demolishing the existing site improvements and then constructing a new parking lot, restroom, picnic shelter, and play area along with supporting infrastructure. The project design will be completed using the current King County Building Code and Surface Water Design Manual.

3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

The explorations and well installations were performed to evaluate geotechnical soil and groundwater conditions at the site. Our observations are specific to the locations, depths, and times noted in the logs and report figures and may not be applicable to all areas of the site. No amount of exploration or testing can precisely predict the characteristics, quality, or distribution of subsurface and site conditions. Potential variation includes, but is not limited to:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (natural and man-made) may result in changes to site and subsurface conditions.
- Groundwater levels and flow directions may fluctuate due to seasonal variations.

If conditions different from those described herein are encountered during project design and construction, we should review our description of the subsurface conditions and reconsider our conclusions and recommendations.

3.1 Site Geology

Our understanding of the site geologic conditions is based on the published *Geologic Map of the Snoqualmie Pass 30x60 Minute Quadrangle* (Tabor and others, 2000). The project site is located at the margin of the Puget Lowland and Cascade Mountain physiographic regions. The site is mapped as being underlain by ice contact deposits formed at the end of the most recent glacial advance which was approximately 15,000 years ago. These deposits are interpreted as ice contact deposits representative of sediment deposited near the margins of glaciation consisting primarily of gravel and sand with lenses of clay and silt (Tabor and others, 2000). Ice contact deposits also commonly contain cobbles and boulders.

3.2 Subsurface Soils

We evaluated the site subsurface soil conditions through both shallow hand-auger explorations and deeper mechanical soil borings. The hand-auger borings were completed by Shannon & Wilson personnel. The mechanical borings and well installation were completed by our subcontractor, Holt Services, Inc. The approximate locations of our explorations are indicated in Figure 2. Further details of our exploration procedures are included in Appendix B.

In general, we observed two soil units at the project site: human-placed fill and ice contact deposit. The soil types observed generally agree with the mapped site geology and apparent recent human influence at the site. Descriptions of each soil type are provided below.

Human-Placed Fill (Hf):

- Consists of loose to medium dense, brown, Silty Sand with Gravel. Gravel is typically angular to subangular crushed rock.
- Observed at all exploration locations near the ground surface.
- Typically less than 2 feet thick.
- Locally thicker in the vicinity of MW-01 likely due to past site regrading activities on the downslope side of the site.

Ice Contact Deposits (Qvi):

- Consists of medium dense to very dense, red-brown to gray-brown, Silty Gravel with Sand.
- Scattered cobbles observed in drill cuttings and inferred by drill action.
- Includes a clean sand layer observed in MW-02, between 7 and 9 feet below ground surface (bgs).

3.3 Groundwater

The Washington State Department of Ecology well log database contains one well record that appears to match the well known to be on the subject property. The well log and well were completed in 1990 and recorded as Well Report ID 90266. The well log indicates that the static groundwater level is approximately 70 feet bgs. The logged lithology suggests that shallower soils may act as an aquitard, perching shallower groundwater above this depth.

We did not observe the presence of groundwater at the site during our explorations. It is unlikely that shallow excavations for the project will encounter the static groundwater surface below the site. Monitoring wells were installed in both borings, MW-01 and MW-02, to a depth of 20 feet. Further study of groundwater levels at the site may be completed using these monitoring wells.

4 SEISMIC DESIGN CONSIDERATIONS

4.1 Code-Based Parameters

We understand the seismic design for the new building will be performed according to the 2018 International Building Code (IBC) (International Code Council, Inc., 2017), which incorporates the American Society of Civil Engineers (ASCE) Standard 7-16 (ASCE, 2017) for seismic design. Computation of the seismic forces is based on seismological input and site soil response factors. The seismological inputs are horizontal peak ground acceleration, horizontal response spectrum acceleration at 0.2-second period (S_S), and horizontal response spectrum acceleration at 1.0-second period (S_1). The design ground motion is the risk-targeted maximum considered earthquake (MCE_R) that corresponds to a target risk of 1% probability of collapse in 50 years.

The 2018 IBC outlines procedures for developing a design response spectrum based on the design spectral acceleration values. The seismological input parameters for S_S and S_1 were evaluated using the mapped values provided in the 2018 IBC. The mapped values correspond to a Site Class B/C. The mapped values are multiplied by site soil response factors as provided in the 2018 IBC to calculate the corresponding MCE_R spectral accelerations S_{MS} and S_{M1}. The design spectral accelerations, S_{DS} and S_{D1}, are two-thirds of the SMS and SM1, respectively. The site soil response factors are dependent upon the seismic Site Class as determined by the procedure detailed in Chapter 20 of ASCE 7-16.

Our Site Class determinations are limited by the depths of the explorations made for the project. The borings for this project extended to approximately 20 feet bgs. Our evaluation of seismic site class is based on the presence of dense soil at shallow depths observed during our explorations and our understanding of the regional geology. Based on our evaluation of the site subsurface soil conditions, the site can be classified as Site Class D. Exhibit 4-1 presents the code-based seismic response spectrum values for the project site.

Exhibit 4-1: Response Spectrum Parameters for IBC 2018 Site Class D

Parameter	ASCE 7-16
Peak Ground Acceleration, PGA (g)	0.478
Short-Period Spectral Acceleration, S_s (g)	1.127
Spectral Acceleration at 1-Second Period, S_1 (g)	0.388
MCE_R Spectral Response Acceleration Coefficient, S_{MS} (g)	1.352
MCE_R Spectral Response Acceleration Coefficient, S_{M1} (g)	0.772
Design Spectral Response Acceleration Coefficient, S_{DS} (g)	0.901
Design Spectral Response Acceleration Coefficient, S_{D1} (g)	0.515
Site Modified PGA_M (g)	0.573

g = Acceleration due to gravity; T = Fundamental Period of Structure

The above response spectra parameters for Site Class D were evaluated based on mapped seismological inputs and the tabulated site coefficients provided in the 2018 IBC and ASCE 7-16. These parameters do not represent a site-specific site response analysis. In accordance with the 2018 IBC Section 1612.2.3 and ASCE 7-16 Section 11.4.8, a ground motion hazard analysis (site-specific site response analysis) is required for Site Class D sites where S_1 is greater than or equal to 0.2 g, as occurs at this site. However, the guidance of Section 11.4.8 allows the use of tabulated site coefficients if the value of the seismic response coefficient, C_S, is determined by ASCE 7-16 Equation 12.8-2 for values of $T \leq 1.5$ Ts and taken as equal to 1.5 times the value computed in accordance with either ASCE 7-16 Equation 12.8-3 for $TL \geq T > 1.5\ T_s$ or ASCE 7-16 Equation 12.8-4 for $T > T_L$ (see ASCE 7-16 Section 11.4.8 Exception 2).

4.2 Earthquake-Induced Geologic Hazards

Earthquake-induced geologic hazards that may affect a given site include landsliding, fault rupture, settlement, and liquefaction and associated effects (such as loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, and lateral spreading). Because of the relatively dense condition of project site soils, the risks of landslide, settlement, and other adverse effects of liquefaction are considered low.

4.2.1 Fault Surface Rupture

The nearest mapped active fault is the Tacoma Fault Zone (Czajkowski and Bowman, 2014). This fault strands are interpreted to extend from the Kitsap peninsula to near Lake Tapps, approximately 12.5 miles away from the project site. Other faults are mapped closer to the project site, but are not known to be currently active. Due to the distance between the site and the nearest known active fault, it is our opinion that the risk of ground surface rupture is low.

4.2.2 Liquefaction and Lateral Spreading

Liquefaction and lateral spreading occur as a result of soil strength loss during seismic shaking due to increased pore water pressures. Liquefaction triggering typically requires that the site soil be in a very loose to medium dense state and be saturated with groundwater. Based on our understanding of the site subsurface soils, the potential for liquefaction and lateral spreading is low. The soils underlying the site are in a generally dense to very dense condition. Where looser soil is present, it is not saturated with ground water.

4.2.3 Slope Stability

The ground surface within the project area is nearly flat. The slopes present at the site are generally composed of sufficiently strong soil and have inclination shallow enough to be minimally susceptible to failure during seismic events. In our opinion, the risk of seismically induced slope failure at the site is low.

5 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 Infiltration Feasibility

We evaluated infiltration rates as they relate to the feasibility of on-site stormwater management facilities at the project site in general accordance with the requirements and procedures in the Stormwater Management Manual for Western Washington (SWMMWW) (Washington State Department of Ecology [Ecology], 2019). Our evaluations were completed to provide a preliminary assessment feasibility for using infiltration Best Management Practices (BMPs) at the project site. The soil infiltration rates presented in this report should not be used for final infiltration facility design. Design rates should be determined using Pilot Infiltration Tests performed at the proposed infiltration facility location and elevation, as described in the King County Surface Water Design Manual. Infiltration feasibility is not strictly dependent upon soil infiltration rate. The physical and chemical suitability of the soil should be evaluated during final design if the infiltration facility will also be used for stormwater treatment.

We estimated the infiltration rate of the site soil utilizing the Massmann grain-size correlation provided in the SWMMWW (Ecology, 2019). The calculated initial saturated hydraulic conductivity (K_{sat}), or infiltration rates, varied between 6.5 and 8 inches per hour. Based on the calculated infiltration rate, presence of granular soil, and absence of shallow groundwater at the site, it is our opinion that stormwater infiltration BMPs are feasible for this project.

5.2 Pavement Design

We understand a new parking lot and driveway will be constructed on the site. It is our understanding that these paved areas will primarily experience standard-duty: passenger vehicle, light truck, and horse trailer loading depending on location. We understand that parking and driveway areas will be paved with hot-mix asphalt (HMA) pavement and/or portland cement concrete (PCC). For both pavement types, we recommend using heavy duty pavement in the drive aisles, and standard duty in the parking areas. Heavy duty paving may be used in all areas if desired.

We provide pavement thickness recommendations in Sections 5.2.2 and 5.2.3, which are based on assumptions relating to pavement loading, design life, and construction materials. Specific design parameters were not provided by the project design team for use in our analyses. The pavement section designs provided below should be revised if anticipated traffic, loading, or design life are strictly defined during project design.

5.2.1 Subgrade Conditions

We evaluated subgrade conditions at the project site using DCP testing. The results of these tests were used to estimate an average subgrade resilient modulus (M_R) for the site. Based on the DCP testing, we recommend using a M_R of 10,000 pounds per square inch for design. In order to achieve uniform subgrade conditions matching our design assumption, pavement subgrade should be prepared in accordance with the recommendations of Section 5.4

5.2.2 Hot-Mix Asphalt Pavement

Typical HMA pavement sections consist of HMA, a crushed surfacing base course (CSBC), and native or fill subgrade. Exhibit 5-1 provides a summary of our recommended HMA pavement sections.

Exhibit 5-1: Hot-Mix Asphalt Pavement Summary (20-Year Design Life)

Pavement Section	Asphalt Thickness (inches)	CSBC Thickness (inches)
Standard-Duty	2	4
Heavy-Duty	4	6

5.2.3 Portland Cement Concrete Pavement

Typical PCC pavement sections consist of PCC, CSBC, and native subgrade. Exhibit 5-2 provides a summary of our recommended PCC pavement sections.

Exhibit 5-2: Portland Cement Concrete Pavement Summary (20-Year Design Life)

Pavement Section	PCC Thickness (inches)	CSBC Thickness (inches)
Standard-Duty	3	6
Heavy-Duty	6	6

5.3 Shallow Foundations

5.3.1 Bearing Capacity

Shallow foundations may bear in the existing site soil provided the soil is compacted to meet the requirements of structural fill. Foundation subgrade preparation should be completed in accordance with the recommendations of Section 5.4. Foundations bearing on compacted native soil or structural fill may be designed for an allowable bearing pressure of 3,000 pounds per square foot for static conditions. This allowable capacity includes a factor of safety of approximately 2.5. For load combinations, including wind and earthquake loading, this allowable bearing capacity may be increased by 33%.

This allowable bearing capacity assumes that foundations are vertically loaded, constructed with a horizontal base, and bear on horizontal soil surfaces. If foundations are to be constructed on sloping grades, further analysis of bearing capacity and slope stability should be completed. Minimum footing widths should be 18 inches for continuous spread footings and 24 inches for isolated column footings.

5.3.2 Settlement

For shallow foundations designed as described in Section 5.3.1, we estimate settlements of an isolated foundation will be less than ½ inch, differential settlement (between adjacent footings or over a 20-foot-long span of continuous footing) less than ¼ inch. These settlements are expected to occur as the structural loads are applied due to the relatively granular nature of the soil.

5.3.3 Lateral Resistance

Resistance to lateral loading may be developed against foundations through interface friction acting on the bottom of the footing, or by passive pressures acting on the vertical portions of the footing below grade. Interface friction should be estimated as 0.4 times the vertical load on the footing. Passive pressure should be calculated using a triangular pressure distribution varying with depth D below ground calculated as 300D pounds per cubic foot. These values are based on the assumption that the footings extend at least 1.5 feet below the lowest adjacent exterior grade and are properly drained, and that the backfill around the footings is compacted in accordance with the recommendations for structural fill described in Section 5.5. The equivalent fluid unit weight and coefficient of friction provided here include a factor of safety of 1.5 to limit lateral deflection.

5.4 Subgrade Preparation

Below new structural foundations, floor slabs, and hardscape surfaces, all existing unsuitable materials, i.e., soft clay and organic soils, should be removed. Subgrade surfaces should be clear of debris and loose soil. Existing site fill soil should be compacted to 95% of its maximum dry density, as determined by ASTM D1557 (ASTM, 2021). Subgrade preparation should generally conform to the requirements of the Washington State Department of Transportation (WSDOT) Standard Specifications Section 2.09.3(3)C (WSDOT, 2020).

After subgrade preparation is completed, all areas to receive new structural foundations, slab-on grade, or hardscape elements should be evaluated by a representative of the geotechnical engineer by probing with a standard soil probe and observing the condition of the subgrade material. Large subgrade areas, such as parking lots or floor slabs, may be evaluated by proof-rolling with a fully loaded dump truck or equivalent piece of rubber-tired equipment.

If soft or unsuitable soils are encountered, they should be removed and replaced with compacted structural fill, as specified in Section 5.5. We recommend that native and backfill soils below structural foundations hardscape surfaces be compacted to at least 95% of the maximum soil dry density as determined by the Modified Proctor compaction test per ASTM D1557 (ASTM, 2021).

5.5 Structural Fill

Fill placed beneath structures or against structures such as footings, retaining walls, or hardscape surfaces should be structural fill. Structural fill should be placed in horizontal lifts, compacted to at least 95% of its Modified Proctor maximum dry density, and be deemed to be in a dense and unyielding condition by a qualified geotechnical engineer. The moisture content for structural fill should be within 2% of the optimum moisture content at the time of installation. The thickness of loose lifts should not exceed 12 inches for heavy equipment compactors or 6 inches for hand-operated compactors. Effective lift thicknesses will vary depending on the fill material and compaction equipment used. Lift thickness may need to be smaller than recommended above if inadequate compaction is observed. Fill placed in areas where structural fill is not required and settlement is acceptable should be compacted to 90% of its Modified Proctor maximum dry density. All compacted surfaces should be sloped to promote drainage and mitigate ponding.

Compaction of backfill adjacent to retaining walls or existing footings can result in a higher lateral earth pressure against the wall or settlement of foundations. Heavy equipment should stay behind a line extending upward from the base of the walls at 0.5H:1V, or 3 feet from the wall, whichever is greater. The backfill within this zone should be compacted with hand-operated equipment or smaller machine-operated equipment. In such areas, the maximum lift thickness of fill should be reduced to 4 inches. We recommend that the backfill around the structure be brought up in uniform horizontal layers on all sides of the structure being backfilled.

On-site soil could be used as structural backfill where free-draining soil is not required, provided particles larger than 3 inches are removed from the soil prior to placement. On-site soil contains enough fines (particles smaller than 0.075 millimeter) to be moisture-sensitive and difficult to compact when above optimum moisture content. If on-site soil is not able to be compacted as required, imported backfill soil should be used. On-site soil not suitable for structural backfill could be used as backfill within landscaped areas where settlement is acceptable.

Imported structural backfill should meet the gradation requirements of Section 9-03.14(1), Gravel Borrow, or Section 9-0.3.9(3) Crushed Surfacing Base Course of the WSDOT Standard Specifications (WSDOT, 2020), or similar free-draining material as approved by the project engineer.

If fill is to be placed during periods of wet weather or under wet conditions, it should have the added requirement that the percentage of fines (material passing the No. 200 sieve based on wet-sieving the minus ¾-inch fraction) be limited to 5%. All fines should be nonplastic.

6 CONSTRUCTION CONSIDERATIONS

We have identified considerations for earthwork and dewatering for the project to assist you in developing geotechnical related plans and specifications, but not to dictate methods or sequences used by contractors. Prospective contractors should undertake their own independent review and evaluation of all information to arrive at decisions concerning the planning of the work; the selection of equipment, means and methods, techniques, and sequences of construction; establishment of safety precautions; and evaluation of the influence of construction on adjacent sites.

6.1 General Earthwork

Excavations could be accomplished with conventional excavating equipment, such as dozers, front-end loaders, and excavators. We observed cobbles in the subsurface soils during our explorations. In our experience, boulders are commonly found in glacial soils. The Contractor should be prepared to remove cobbles and boulders from excavations if encountered on site.

Removal of the surficial vegetation layer may be necessary to expose suitable subgrade surfaces. For estimation purposes, we recommend a striping depth of 6 inches in primarily grassy areas, and 24 inches near removed trees.

6.2 Temporary and Permanent Slopes

For safe working conditions and prevention of ground loss, excavation slopes and/or shoring should be the responsibility of the Contractor because the Contractor will be at the site to observe and control the work. All current and applicable safety regulations regarding excavation slopes and shoring should be followed.

For cost estimating and planning purposes only, temporary excavation slopes will likely require slopes of 1.5H:1V, consistent with Occupational Safety and Health Administration (OSHA) Type C soils (OSHA, 2015). Flatter slopes may be required based on the actual conditions encountered, particularly where groundwater is encountered. Materials and equipment should be kept back from the top of site slopes a distance of at least half the slope height. Steeper slopes could be achieved by using temporary and/or permanent retaining walls.

Consistent with conventional practice, the Contractor should be responsible for the actual temporary excavation slopes, including methods, sequence, and schedule of construction. The Contractor is able to observe the nature and conditions of the subsurface materials encountered and should evaluate the factors discussed above. If instability is observed, slopes should be flattened or shored. All excavations should be accomplished in accordance with local, state, and federal safety regulations.

We recommend permanent slopes constructed of structural fill be no steeper than 2H:1V. Permanent slopes constructed of fill compacted to a lower density than required of structural fill should be no steeper than 3H:1V.

6.3 Construction Drainage

We recommend that site drainage measures be incorporated into the project construction regardless of the time of year when construction occurs. Surface runoff can be controlled by careful grading practices. Typically, these include shallow perimeter ditches or low earth berms to direct water away from the work area. Temporary sumps and pumps could be used to collect and convey surface water from the work area. All collected water should be conveyed in a controlled manner to an approved discharge point. Treatment of site water prior to discharge may be required by the permitting agency.

6.4 Wet Weather Earthwork

In the Puget Sound region, wet weather conditions typically persist between October and May. However, wet weather conditions can occur at any time of year. Earthwork performed during wet weather would likely cost more and take longer to complete. The near-surface soils at the site generally contain enough silt to produce an unstable mixture and are susceptible to softening when wet. Standing water on the soil surface, along with construction activity, could result in disturbance and an unstable surface that could require overexcavation and replacement with clean crushed rock.

The following recommendations are applicable for footings, general excavation, and floor slabs:

- If there is to be traffic over the exposed subgrade, the subgrade should be protected from disturbance. A stabilized access road constructed of a coarse aggregate, such as quarry spalls placed atop a woven geotextile, could be placed immediately following excavation on the undisturbed soils. This could be done, as needed, to protect the exposed soils and act as a working surface.
- Construction should be observed on a part-time basis by Shannon & Wilson personnel to determine that all unsuitable materials are removed, suitable drainage is achieved, and that an appropriate bearing surface results.

- Covering work areas with plastic and/or sloping, ditching, pumping from sumps, and other dewatering measures should be employed as necessary to permit proper completion of the work.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soil and placement and compaction of clean structural fill can be accomplished on the same day or sooner.
- The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with an excavator located so that equipment does not track over the excavated area. In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil.
- Common measures, such as silt fences and/or straw bales or wattles placed at the perimeter of the site, can be used to control erosion. A wheel wash or quarry spalls at the site entrance will also limit soils spread to nearby streets from site equipment if trucks are expected to frequently enter and exit the project site. These measures should be maintained by the Contractor to ensure that they are functioning as intended.

The above recommendations apply for all weather conditions but are most important for wet weather earthwork. They should be incorporated into the contract specifications for excavations, foundation, and pavement construction.

7 CLOSURE

This geotechnical report was prepared for the exclusive use of SAGE Architectural Alliance and King County Department of Natural Resources and Parks for design of new trailhead facilities at Little Lake Forest Park. This geotechnical report should not be used without our approval if any of the following occurs:

- Conditions change due to natural forces or human activity under, at, or adjacent to the site.
- Assumptions stated in this geotechnical report have changed.
- Project details change or new information becomes available such that our evaluations, analyses, conclusions, and recommendations may be affected.
- If the site ownership or land use has changed.
- More than ten years has passed since the date of this report.

If any of these occur, we should be retained to review the applicability of our evaluations, analyses, conclusions, and recommendations. Shannon & Wilson has prepared the enclosed

"Important Information About Your Geotechnical/Environmental Report" to assist you and others in understanding the use and limitations of our reports.

8 REFERENCES

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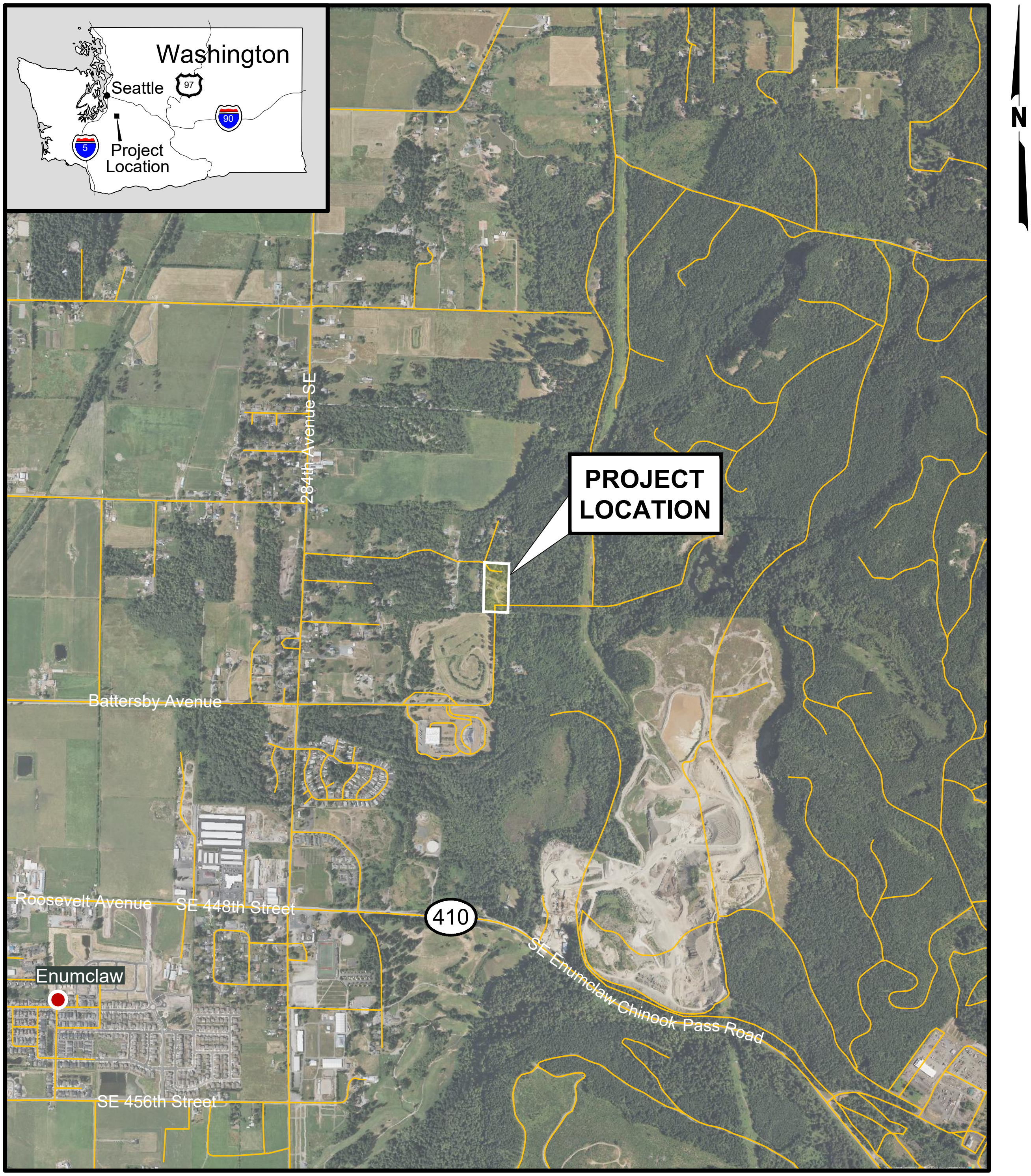


0 2000 4000

Approximate Scale in Feet

Little Lake Forest Park Trailhead
Seattle, Washington

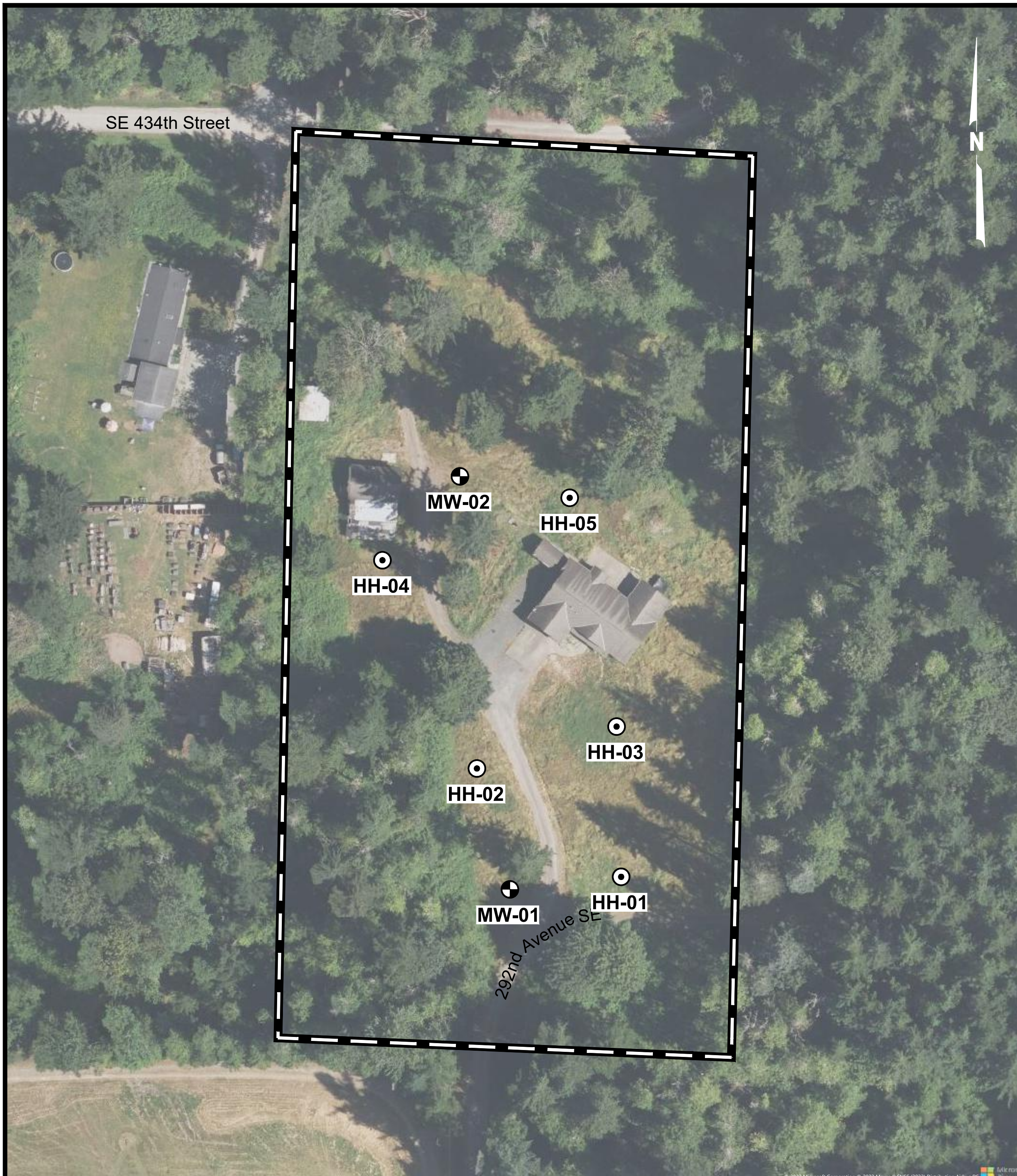
SITE VICINITY MAP

February 2022 106797-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

Filename: C:\Users\jrs\CAD Group Dropbox\JDrive_SEA\106797\001\106797-001 Vicinity Map.dwg Date: 02-14-2022 Login: JRS



LEGEND

HH-01 Dynamic Cone Penetrometer Designation and Approximate Location

MW-01 Monitoring Well Designation and Approximate Location

0 100 200

Scale in Feet

Little Lake Forest Park Trailhead
Seattle, Washington

SITE AND EXPLORATION PLAN

February 2022 106797-001

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FIG. 2

Filename: C:\Users\jrs\CAD Group Dropbox\JDrive_SEA\106797\001\106797-001 Explorations.dwg Layout: Figure 2 Date: 02-14-2022 Login: JRS

APPENDIX A: GEOTECHNICAL EXPLORATIONS

Appendix A

Geotechnical Explorations

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Figures

APPENDIX A: GEOTECHNICAL EXPLORATIONS

A.1 GENERAL

The subsurface exploration program consisted of two mechanical soil borings, and five manual explorations. The purpose of the program was to develop an understanding of the subsurface conditions at the site. Groundwater monitoring wells were also installed at each boring location to a depth of approximately 20 feet.

A.2 GEOTECHNICAL BORINGS

Geotechnical borings were drilled to depths between approximately 20 feet below ground surface at two locations near the north and south sides of the project sites. The boring locations are shown in the Site and Exploration Plan, Figure 2, in the main report. Logs of our borings are included in this appendix as Figures A-2 and A-3.

Holt Services, Inc. drilled the borings on February 2, 2022, under subcontract to Shannon & Wilson. Holt used a truck-mounted Mobile B-58 drill rig and advanced the borings using hollow-stem auger (HSA) drilling techniques. HSA drilling consists of advancing continuous-flight augers to remove soil from the borehole. Holt dispersed the drill cuttings on site. Holt installed 2-inch-diameter monitoring wells with 8-inch flush surface monuments. A profile of the monitor well installation is included in our boring logs.

A.3 SAMPLING AND CLASSIFICATION

A representative from Shannon & Wilson was present during the explorations to observe drilling and excavation, retrieve representative soil samples for subsequent laboratory testing, and prepare descriptive field logs of the explorations. We based soil sample classification on the ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM, 2017a), and ASTM D2488, Standard Practice for Description and Identification of Soils (Visual/Manual Procedure) (ASTM, 2017b). We used the Unified Soil Classification System, as described in Figure A-1 of this appendix, to classify the material encountered.

We obtained disturbed soil samples in conjunction with the Standard Penetration Test (SPT). We performed SPTs in general accordance with ASTM D1586, Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils (ASTM, 2018). We collected SPTs in the borings at 2.5-foot intervals. The SPT consists of driving a 2-inch-outside-diameter split-spoon sampler a total distance of 18 inches below the bottom of the drill hole with a 140-pound hammer falling 30 inches. The number of blows required to advance the split spoon from 6 to 18 inches of penetration is termed the Standard Penetration Resistance (N-value). This value is an empirical parameter that provides a means for evaluating the relative density, or compactness, of granular soils and the consistency, or stiffness, of cohesive soils. Figure A-1 presents the terminology used to describe the relative density or consistency of the soil.

A.4 GEOTECHNICAL HAND EXPLORATION

Geotechnical hand excavations were completed at depths of approximately 2 feet below ground surface at five locations near proposed trailhead improvements. These explorations were excavated using a shovel and hand-auger. We collected a bulk sample of the near surface soil at each hand exploration location. Prior to excavating, we performed Dynamic Cone Penetrometer (DCP) testing at all hand exploration locations. DCP test results and calculated soil behavior parameters are presented in Figures A-4 through A-8.

A.5 REFERENCES

ASTM International, 2017a, Standard practice for classification of soils for engineering purposes (unified soil classification system), D2487-17: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 10 p., available: www.astm.org.

ASTM International, 2017b, Standard practice for description and identification of soils (visual/manual procedure), D2488-17e1: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 13 p., available: www.astm.org.

ASTM International, 2018, Standard test method for standard penetration test (SPT) and split-barrel sampling of soils , D1586-18: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 26 p., available: www.astm.org.

SOIL DESCRIPTION AND LOG KEY

Little Lake Forest Park Trailhead Improvements
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Fig. A-1
Sheet 1 of 2

Shannon & Wilson uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

Structure[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch-thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch-thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

Angularity and Shape[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

Standard Penetration Test (SPT)[3]

Hammer	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diameter cathead 2-1/4 rope turns, > 100 rpm. If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less or 10 blows for 0 inch.

Moisture Content

Dry	Absence of moisture, dusty, dry to the touch.
Moist	Damp but no visible water.
Wet	Visible free water, from below water table.

Gradation

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

Cementation[1]

Weak	Crumbles/breaks with handling or slight finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

Plasticity[2]

Nonplastic	Cannot roll a 1/8-in. thread at any water content.	PI < 4
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 < PI < 10
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 < PI < 20
High	It takes considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	PI > 21

Additional Terms

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

Notes:

[1]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

[2]Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

[3]Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

Unified Soil Classification System (USCS)
Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488

Major Divisions			Symbol	Typical Identifications
Coarse-Grained Soils *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	Well-graded Gravel; Well-graded Gravel with Sand
			GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	Silty Gravel; Silty Gravel with Sand
			GC	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	Well-graded Sand; Well-graded Sand with Gravel
			SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	Silty Sand; Silty Sand with Gravel
			SC	Clayey Sand; Clayey Sand with Gravel
Fine-Grained Soils *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor		PT	Peat or other highly organic soils (see ASTM D4427)

Acronyms and Abbreviations

ATD	At Time of Drilling	MgO	Magnesium Oxide	psi	Pounds per Square Inch
Diam.	Diameter	mm	Millimeter	PVC	Polyvinyl Chloride
Elev.	Elevation	MnO	Manganese Oxide	rpm	Rotations per Minute
ft	Feet	NA	Not Applicable or Not Available	SPT	Standard Penetration Test
FeO	Iron Oxide	NP	Nonplastic	USCS	Unified Soil Classification System
gal	Gallons	O.D.	Outside Diameter	q_u	Unconfined Compressive Strength
Horiz.	Horizontal	OW	Observation Well	VWP	Vibrating Wire Piezometer
HSA	Hollow-Stem Auger	pcf	Pounds per Cubic Foot	Vert.	Vertical
I.D.	Inside Diameter	PID	Photoionization Detector	WOH	Weight of Hammer
in	Inches	PMT	Pressuremeter Test	WOR	Weight of Rods
lbs	Pounds	ppm	Parts per Million	Wt	Weight

Well and Backfill Symbols

- Bentonite Cement Grout
- Bentonite Grout
- Bentonite Chips
- Silica Sand
- Perforated or Screened Casing
- Surface Cement Seal
- Asphalt or Cap
- Slough
- Inclinometer or Non-perforated Casing
- Instrumentation Riser or Electrical Lead
- Vibrating Wire Piezometer with Designation

Relative Density
Cohesionless Soils

N, SPT, Blows/ft	Relative Density
< 4	Very loose
4 - 10	Loose
10 - 30	Medium dense
30 - 50	Dense
> 50	Very dense

Relative Consistency
Cohesive Soils

N, SPT, Blows/ft	Relative Consistency
< 2	Very soft
2 - 4	Soft
4 - 8	Medium stiff
8 - 15	Stiff
15 - 30	Very stiff
> 30	Hard

Percentages[1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

Notes:

Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).

Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.

No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

Total Depth:	21.5 ft.	Northing:		Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~ 856 ft.	Easting:		Drilling Company:	Holt Services	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Mobile B-58	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

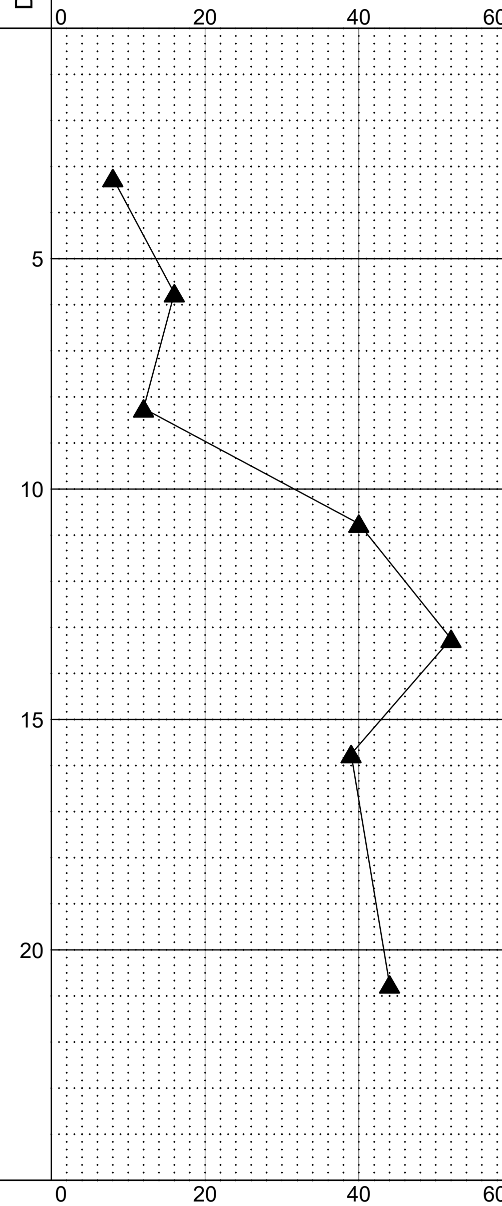
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
Loose, brown *Silt with Sand and Gravel*; moist. (Fill) - Organic layer at approximately 4 feet.		1
Medium dense to dense, red-brown to gray-brown, *Silty Gravel with Sand*; moist. - Cobbles in soil cuttings.	4.0	2, 3, 4, 5, 6, 7
BOTTOM OF BORING COMPLETED 2/2/2022	21.5	

Ground Water: Water Level Not Determined

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Well Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

Little Lake Forest Park Trailhead Improvements
Enumclaw, Washington

LOG OF BORING MW-01

February 2022 106797-001

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FIG. A-2

VERSION 1
FINAL

Total Depth:	*21.5 ft.*	Northing:		Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*8 in.*
Top Elevation:	*~ 852 ft.*	Easting:		Drilling Company:	*Holt Services*	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	*Mobile B-58*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

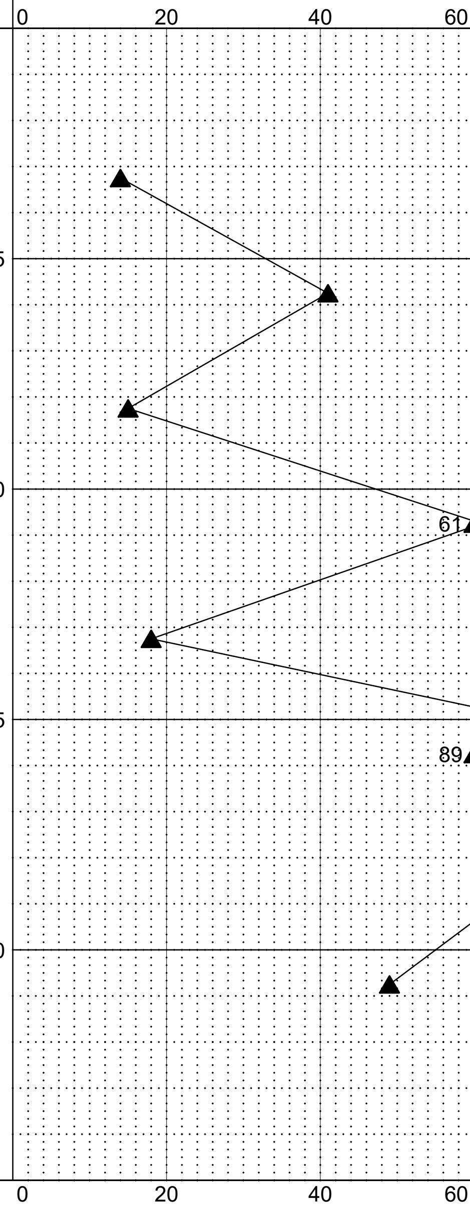
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
Medium dense, red-brown, *Silty Sand with Gravel*; moist.		1
Dense, red-brown, *Silty Gravel with Sand*; moist.	4.0	2
Medium dense, yellow-brown, *Poorly Graded Sand*; moist.	7.0	3
Medium dense to dense, *Silty Gravel with Sand*; moist.	9.0	4, 5, 6, 7
BOTTOM OF BORING COMPLETED 2/2/2022	21.5	

Ground Water: Water Level Not Determined

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Well Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

Little Lake Forest Park Trailhead Improvements
Enumclaw, Washington

LOG OF BORING MW-02

February 2022 106797-001

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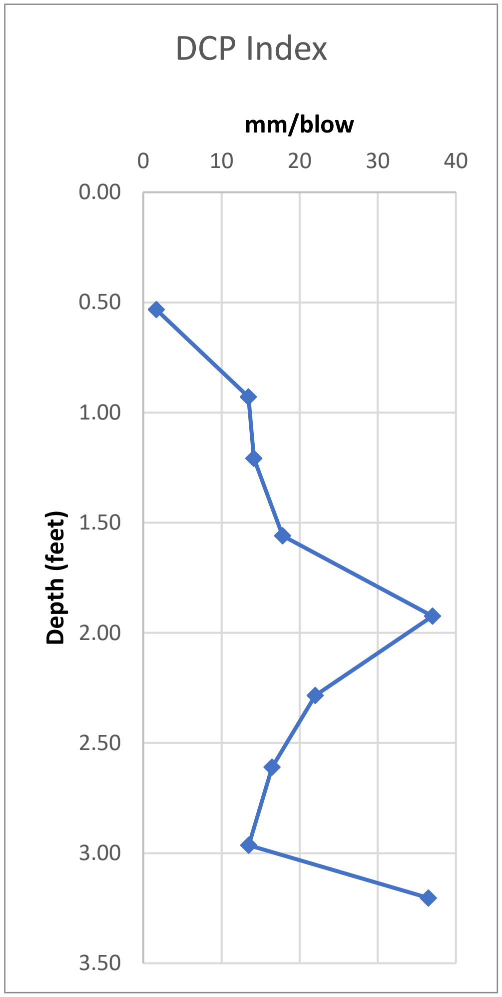
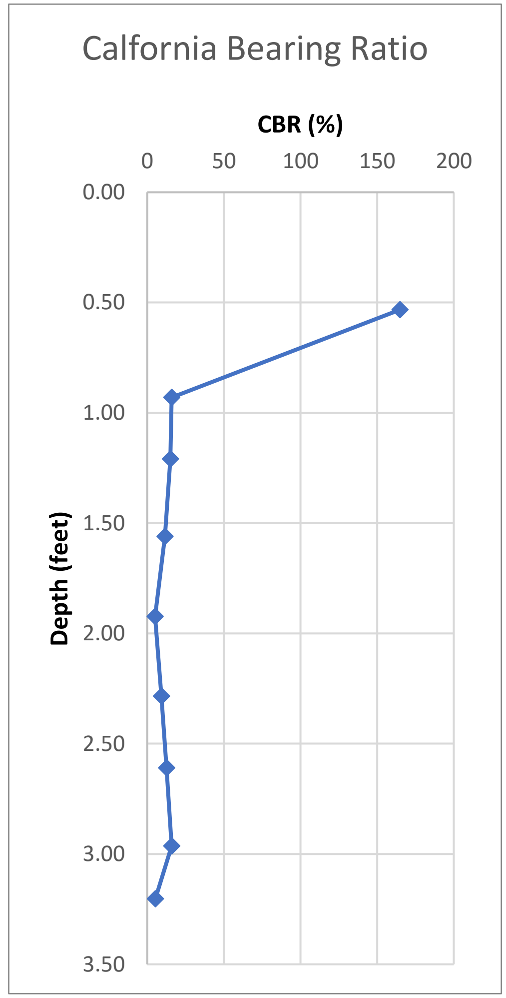
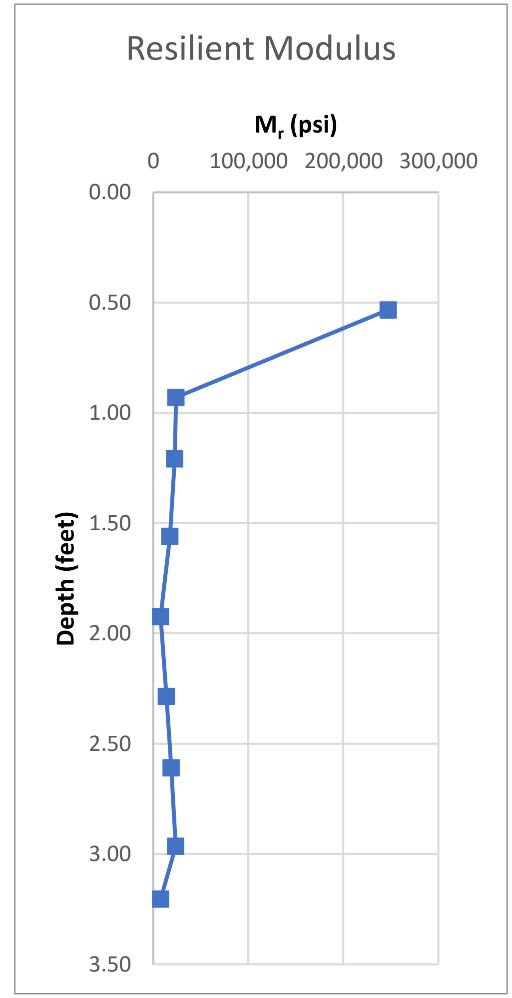
FIG. A-3

VERSION 1
FINAL

MASTER_LOG_E 106797-001.GPJ SHAN_WIL.GDT 02/16/22 Log: NLP Rev: NLP Typ: TVV







NOTES:

1. Test Started at apprxoimately 0.5 feet below ground surface, Depth plotted is relative to ground surface.
2. The low penetration index at 0.5 feet was due to driving on gravel and is not generally representative of the soil conditions.
3. California Bearing Ratio was determined by correlation with Pentration Index provided in *Description and application of Dual Mass Dynamic Cone Penetrometers* by Webster, Grau and Williams, 1992.
4. Unfactored Resilient Modulus M_r estimated as 1500*CBR following Heukelom and Klomp, 1962.

FIG. A-4

Little Lake Forest Park
Trailhead Improvements

DYNAMIC CONE PENETROMETER TEST RESULTS HH-01

February 2022 106797-001

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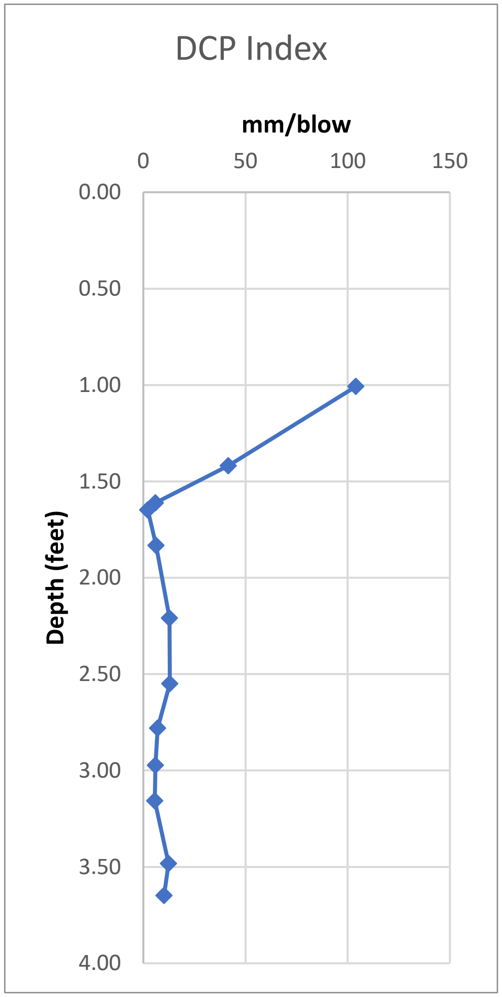
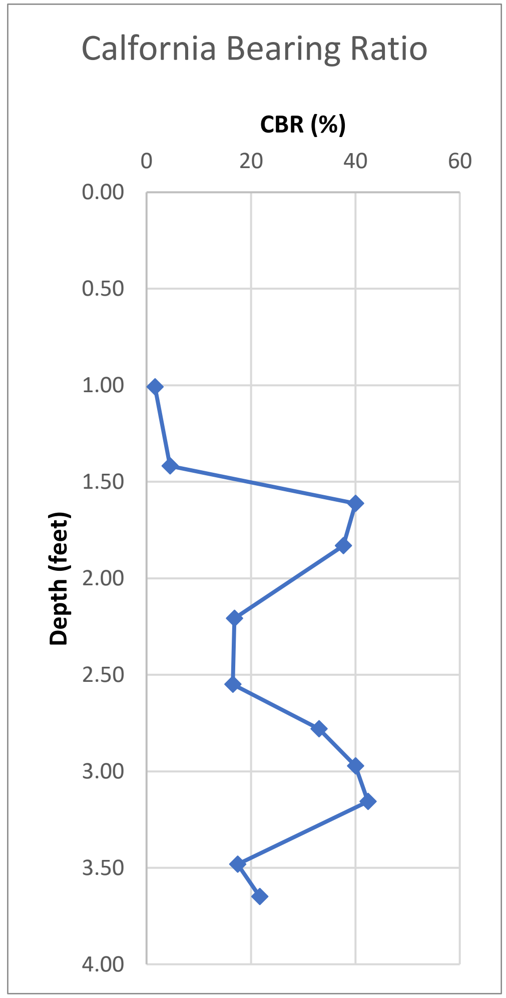
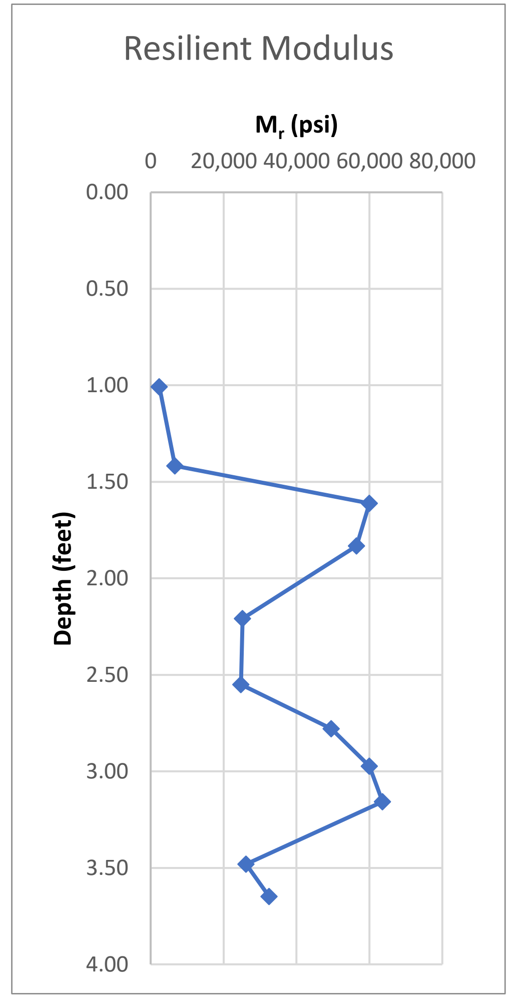
FIG. A-4







NOTES:

1. Test Started at apprxoimately 0.75 feet below ground surface, Depth plotted is relative to ground surface.
2. The low penetration index at 0.75 feet was due to driving on gravel and is not generally representative of the soil conditions.
3. California Bearing Ratio was determined by correlation with Pentration Index provided in *Description and application of Dual Mass Dynamic Cone Penetrometers* by Webster, Grau and Williams, 1992.
4. Unfactored Resilient Modulus M_r estimated as 1500*CBR following Heukelom and Klomp, 1962.

FIG. A-5

Little Lake Forest Park
Trailhead Improvements

DYNAMIC CONE PENETROMETER TEST RESULTS HH-02

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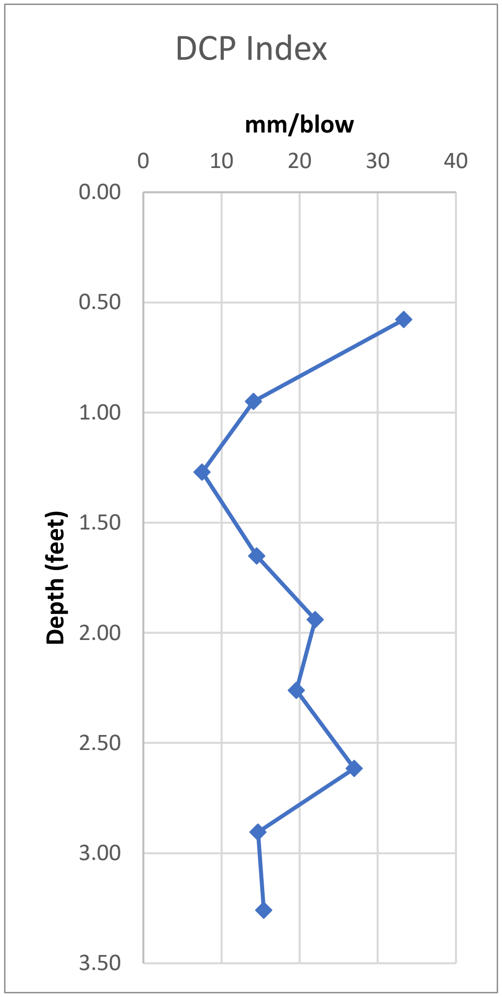
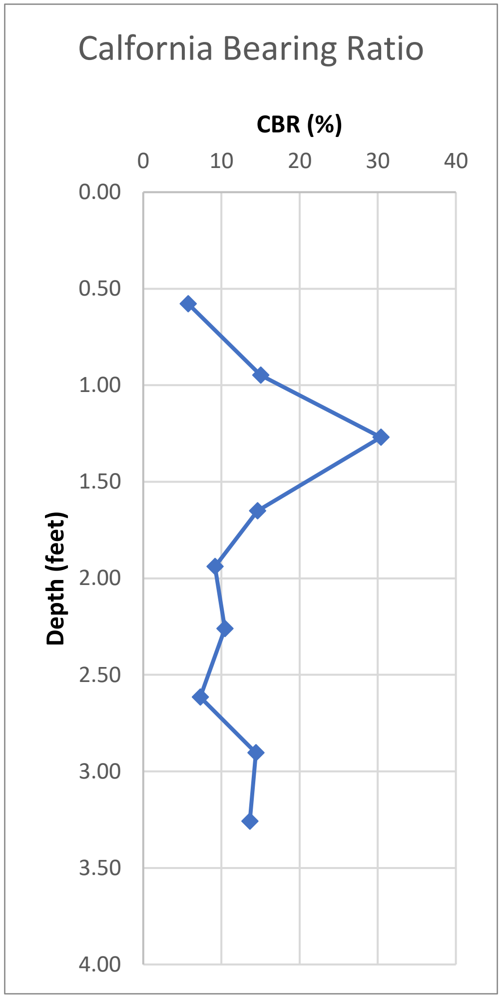
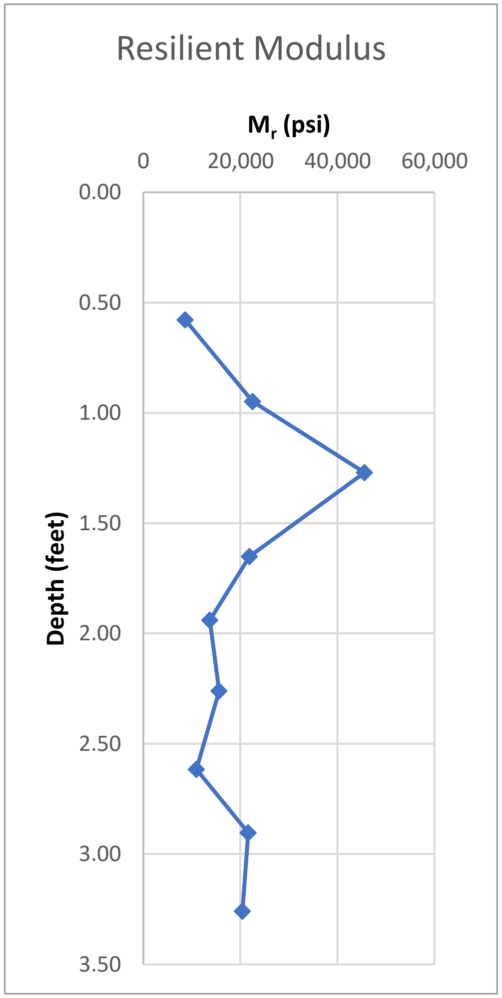
FIG. A-5







NOTES:

1. Test Started at apprxoimately 0.25 feet below ground surface, Depth plotted is relative to ground surface.
2. California Bearing Ratio was determined by correlation with Pentration Index provided in Description and application of Dual Mass Dynamic Cone Penetrometers by Webster, Grau and Williams, 1992.
3. Unfactored Resilient Modulus Mr estimated as 1500*CBR following Heukelom and Klomp, 1962.

FIG. A-6

Little Lake Forest Park
Trailhead Improvements

DYNAMIC CONE PENETROMETER TEST RESULTS
HH-03

February 2022 106797-001

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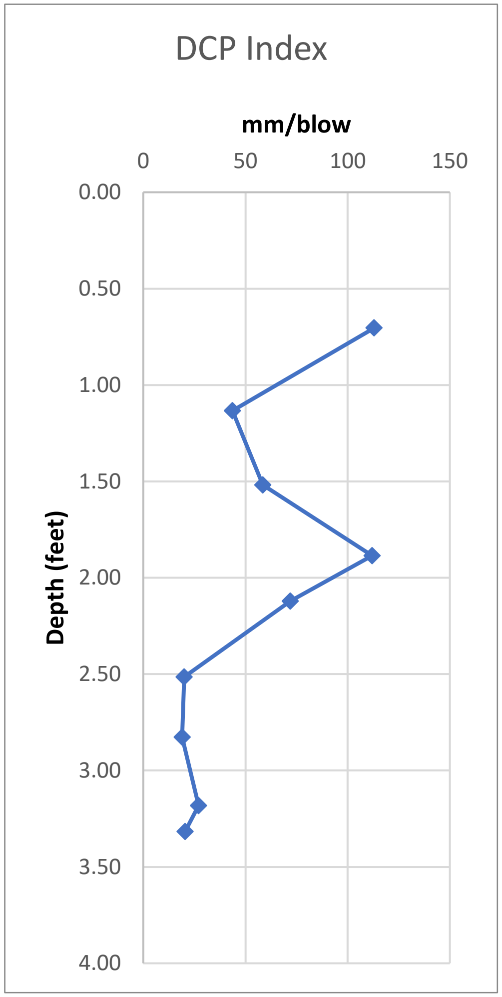
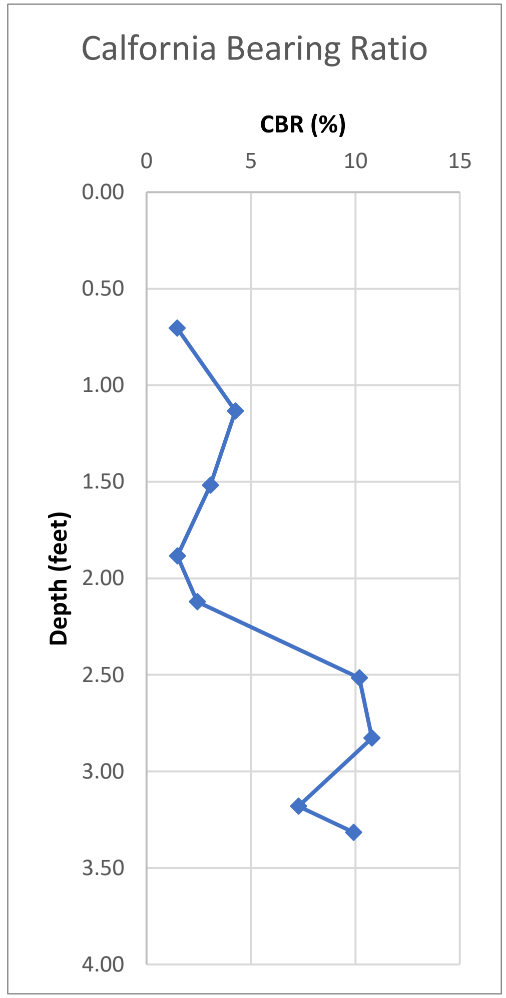
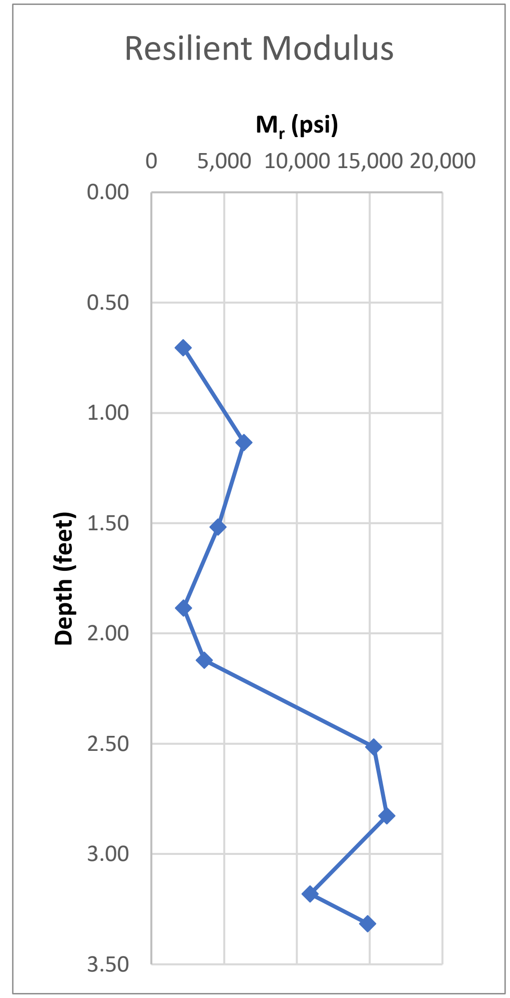
FIG. A-6







NOTES:

1. Test Started at apprxoimately 0.25 feet below ground surface, Depth plotted is relative to ground surface.
2. California Bearing Ratio was determined by correlation with Pentration Index provided in Description and application of Dual Mass Dynamic Cone Penetrometers by Webster, Grau and Williams, 1992.
3. Unfactored Resilient Modulus Mr estimated as 1500*CBR following Heukelom and Klomp, 1962.

FIG. A-7

Little Lake Forest Park
Trailhead Improvements

DYNAMIC CONE PENETROMETER TEST RESULTS
HH-04

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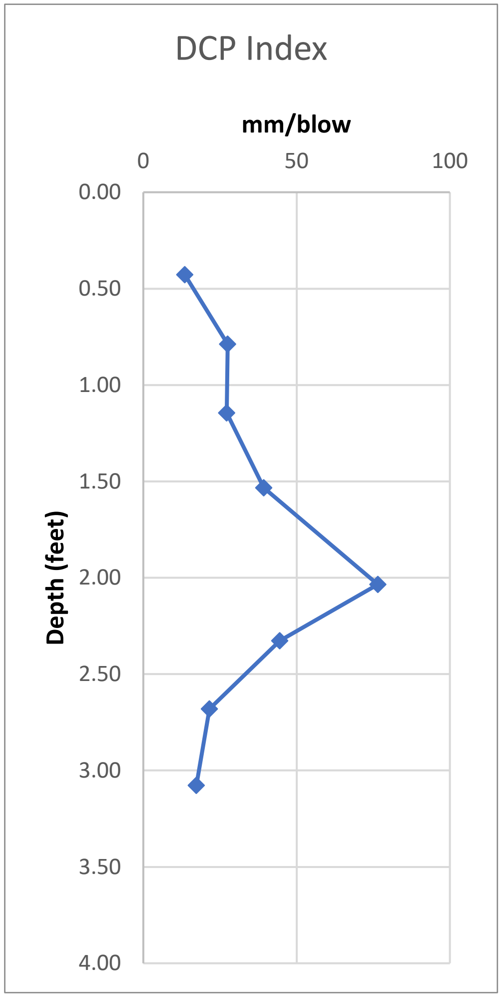
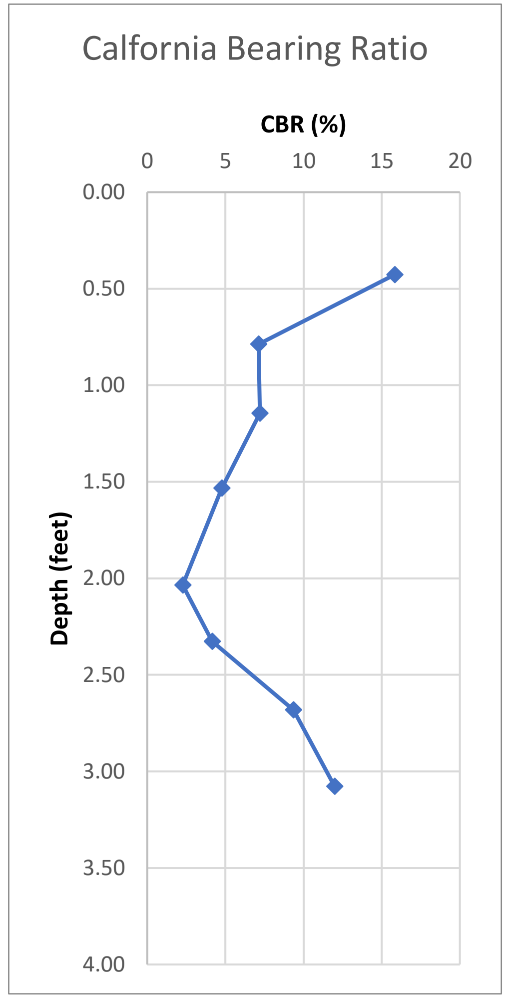
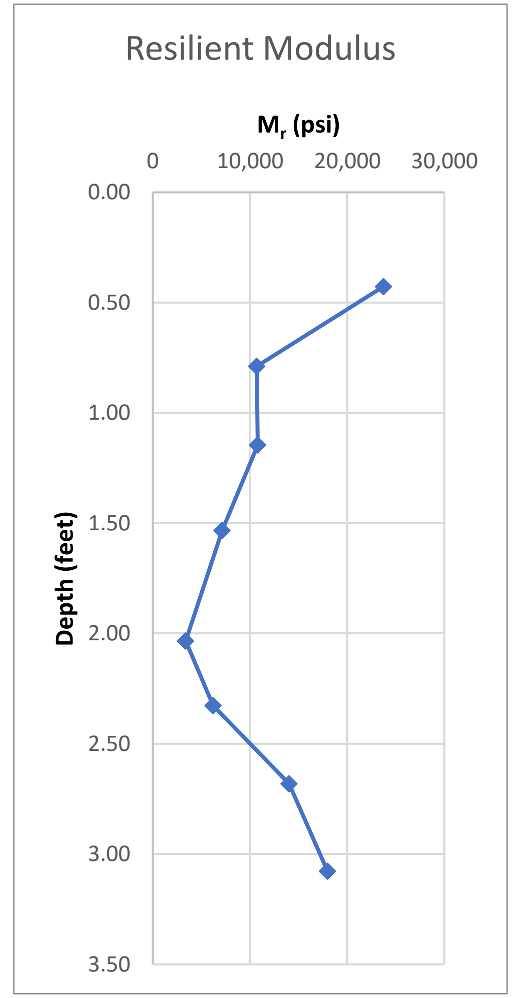
FIG. A-7

HH-05_DCP_data_sheet.xlsx 2/14/2022







NOTES:

1. Test Started at apprxoimately 0.25 feet below ground surface, Depth plotted is relative to ground surface.
2. California Bearing Ratio was determined by correlation with Pentration Index provided in Description and application of Dual Mass Dynamic Cone Penetrometers by Webster, Grau and Williams, 1992.
3. Unfactored Resilient Modulus Mr estimated as 1500*CBR following Heukelom and Klomp, 1962.

FIG. A-8

Little Lake Forest Park
Trailhead Improvements

DYNAMIC CONE PENETROMETER TEST RESULTS HH-05

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FIG. A-8

APPENDIX B: GEOTECHNICAL LABORATORY TESTING

Appendix B
Geotechnical Laboratory Testing

CONTENTS

Figures

APPENDIX B: GEOTECHNICAL LABORATORY TESTING

B.1 GENERAL

We performed geotechnical laboratory testing on selected soil samples retrieved from the borings in order to classify the soil and provide data for engineering studies. Our laboratory testing program included visual classification, water content determinations, and grain-size distribution analyses. A summary of our lab test results is provided as Figure B-1.

B.2 VISUAL CLASSIFICATION

We visually classified soil samples retrieved from the borings using a system based on ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM, 2020), and ASTM D2488, Standard Practice for Description and Identification of Soils (Visual/Manual Procedure) (ASTM, 2017a). We assigned a Unified Soil Classification System (USCS) group name and symbol based on our visual classification of particles finer than 76.2 millimeters (3 inches). We revised visual classifications, shown in our Appendix A boring logs, using results of the index tests discussed below.

B.3 WATER CONTENT DETERMINATION

We tested the water content of selected samples in accordance with ASTM D2216-19, Standard Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (ASTM, 2019). Comparison of the water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. We present water content test results graphically in the Appendix A boring logs.

B.4 FINES CONTENT DETERMINATION

We tested the fines content of selected samples in accordance with ASTM D1140-17, Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing (ASTM, 2017b). Fines content indicates whether a soil is considered to be coarse-grained (sand and gravel) or fine-grained (silt and clay), and is a primary input into liquefaction analyses. We present fines content test results graphically in the Appendix A boring logs.

B.5 GRAIN-SIZE DISTRIBUTION ANALYSIS

Grain-size distribution analyses separate soil particles through mechanical or sedimentation processes. Grain-size distributions are used to classify the granular component of soils and can correlate with soil properties, including frost susceptibility, permeability, shear strength, liquefaction potential, capillary action, and sensitivity to moisture. We plot grain-size distribution analysis results in this appendix as Figures B-2 and B-3. Grain-size distribution plots provide tabular information about each specimen, including USCS group symbol and group name, water content, constituent (i.e., cobble, gravel, sand, and fines) percentages, coefficients of uniformity and curvature, personnel initials, ASTM standard designation, and testing remarks.

We performed mechanical sieve analyses on selected soil specimens to determine the grain-size distribution of coarse-grained soil particles in accordance with ASTM D6913-017, Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates (ASTM, 2017c).

B.6 REFERENCES

ASTM International, 2017a, Standard practice for description and identification of soils (visual/manual procedure), D2488-17e1: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 13 p., available: www.astm.org.

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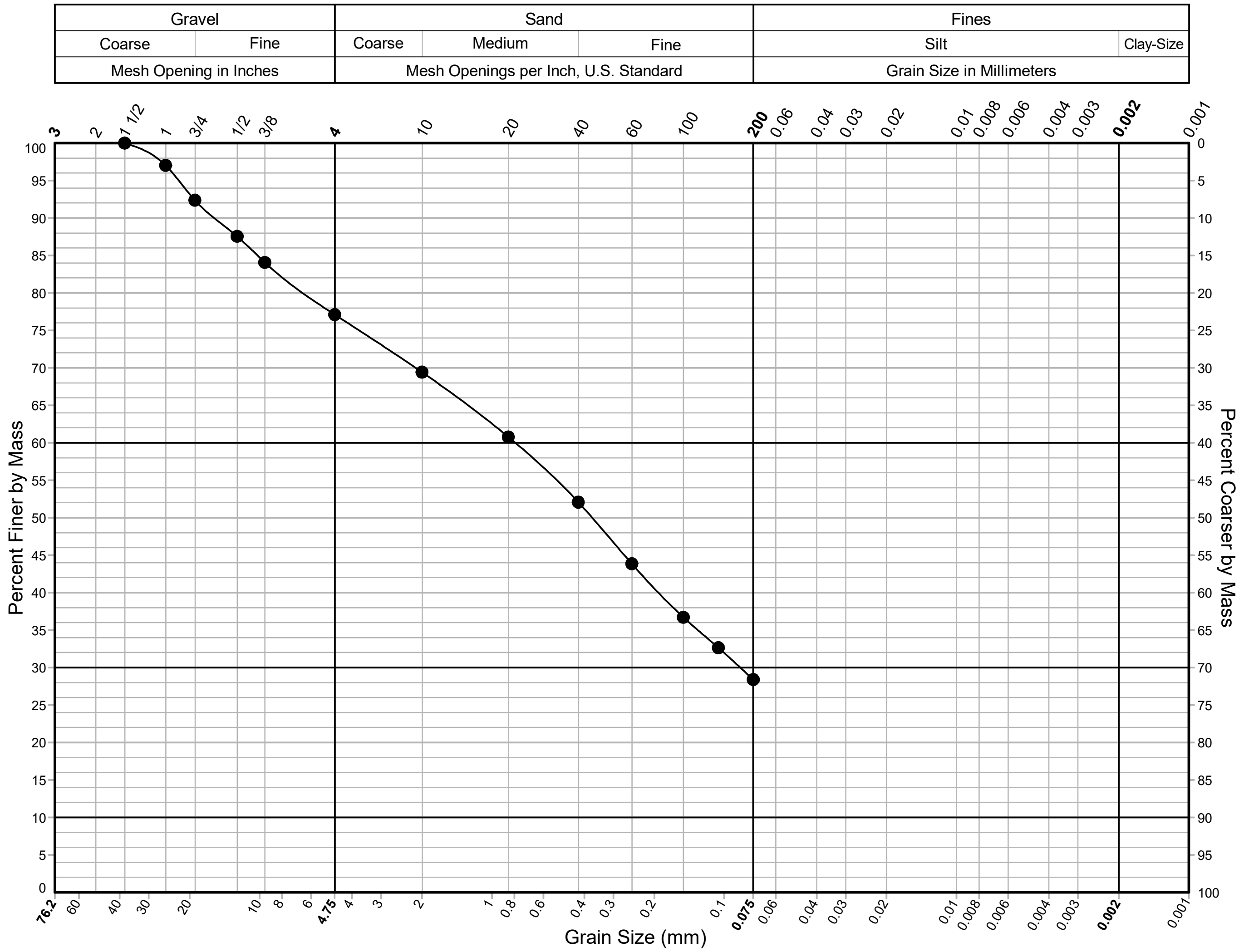
SHANNON & WILSON

GRAIN SIZE DISTRIBUTION PLOT

Little Lake Forest Park Trailhead
Enumclaw, Washington

BORING HH-1



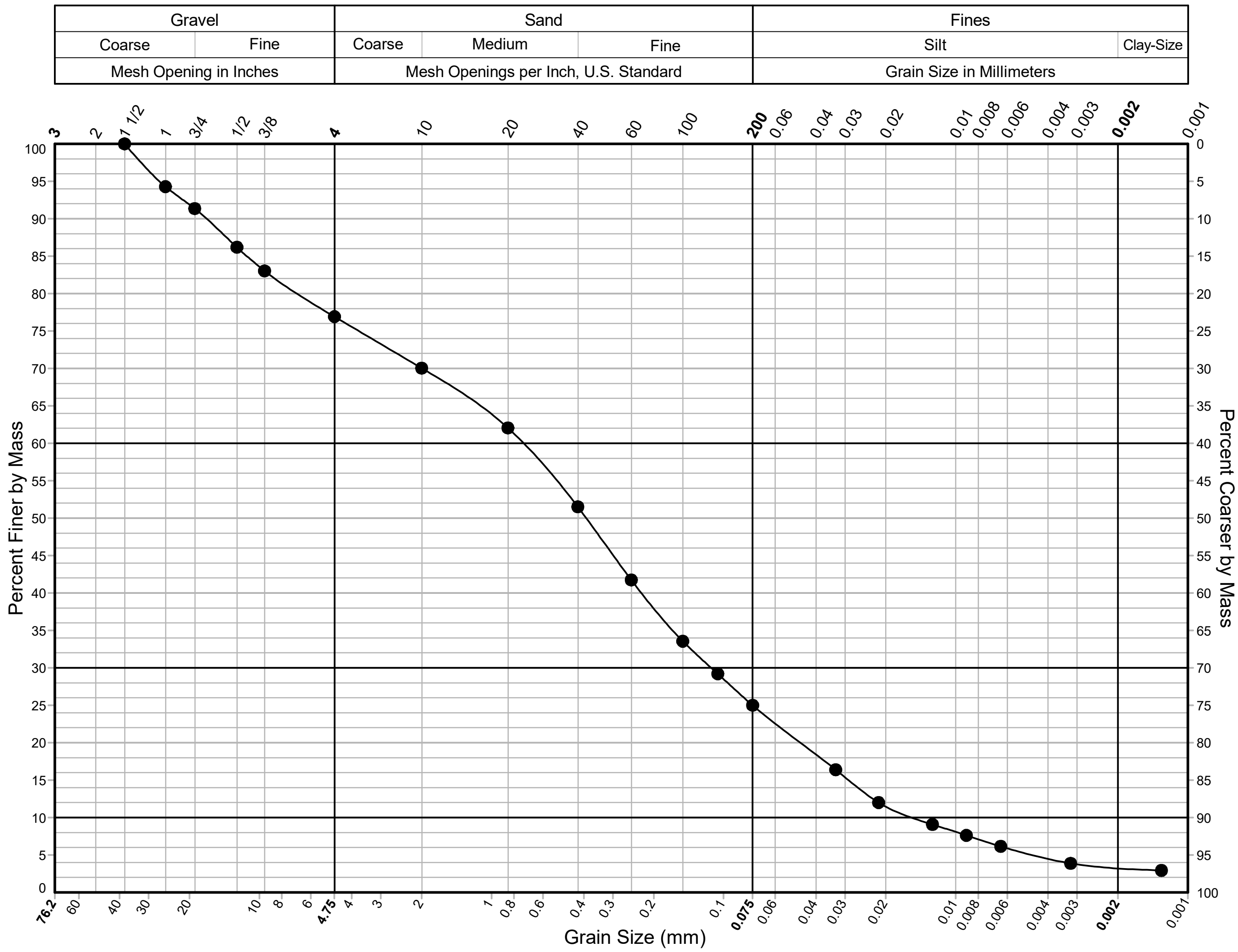
Sample Identification	Depth (ft)	USCS Group Symbol	USCS Group Name	Gravel %	Sand %	Fines %	< 20μm %	< 2μm %	WC %	Tested By	Review By	ASTM Std.
● HH-1, S-1*	0.5	SM	Silty Sand with Gravel	23	49	28			19.5	KCV		D6913

* *Test specimen did not meet minimum mass recommendations.*

GRAIN SIZE DISTRIBUTION PLOT

Little Lake Forest Park Trailhead
Enumclaw, Washington

BORING HH-2

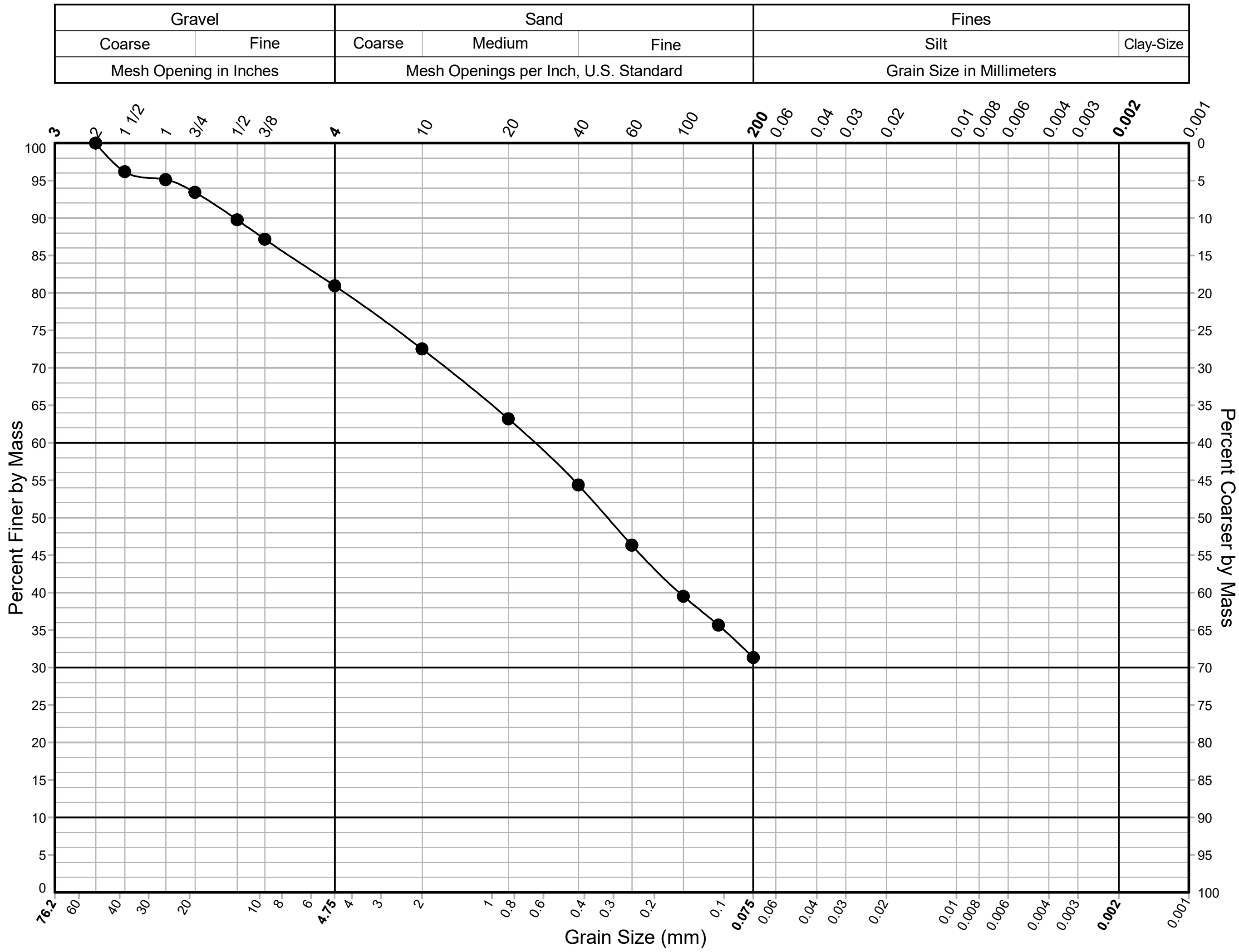


Sample Identification	Depth (ft)	USCS Group Symbol	USCS Group Name	Gravel %	Sand %	Fines %	< 20µm %	< 2µm %	WC %	Tested By	Review By	ASTM Std.
● HH-2, S-1	0.5	SM	Silty Sand with Gravel	23	52	25	12	3	22.5	KCV		D422

GRAIN SIZE DISTRIBUTION PLOT

Little Lake Forest Park Trailhead
Enumclaw, Washington

BORING HH-5



Sample Identification	Depth (ft)	USCS Group Symbol	USCS Group Name	Gravel %	Sand %	Fines %	< 20µm %	< 2µm %	WC %	Tested By	Review By	ASTM Std.
● HH-5, S-1*	0.5	SM	Silty Sand with Gravel	19	50	31			25.5	KCV		D6913

* *Test specimen did not meet minimum mass recommendations.*

IMPORTANT INFORMATION

Important Information

About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent

IMPORTANT INFORMATION

such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

IMPORTANT INFORMATION

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the GBA, Silver Spring, Maryland