GEOTECHNICAL ENGINEERING REPORT
LABAN RESIDENCE IMPROVEMENTS

10 BROOK BAY DRIVE
MERCER ISLAND, WASHINGTON

ZGA Project No. 2560.01
February 27, 2023

Prepared for:
Mina & Balsa Laban

Prepared by:

ZipperGeo
Geoprofessional Consultants

ZipperGeo
Geoprofessional Consultants

ZGA Project No. 2560.01
February 27, 2023

Mina & Balsa Laban
10 Brook Bay Drive
Mercer Island, Washington 98040

Subject: Geotechnical Engineering Report
Laban Residence Improvements
10 Brook Bay Drive
Mercer Island, Washington 98040

Dear Mr. & Mrs. Laban,

In accordance with your request, Zipper Geo Associates, LLC (ZGA) has completed our geotechnical engineering report for the Laban Residence Improvements project. This report presents the findings of our subsurface exploration and our geotechnical recommendations for the project. Our services were completed in general accordance with our *Proposal for Geotechnical Engineering Services* (Proposal No. P21309) dated December 1, 2021. Authorization to proceed was provided by you on May 4, 2022. 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

2/27/23 2/27/23

James P. Georgis, L.E.G
Principal

Robert A. Ross, P.E.
Principal

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING REPORT
LABAB RESIDENCE IMPROVEMENTS
10 BROOK BAY DRIVE
MERCER ISLAND, WASHINGTON 98040

Project No. 2560.01
February 27, 2023

1.0 INTRODUCTION

This report documents the surface and subsurface conditions encountered at the project site and our geotechnical engineering recommendations for the project. The project description, site conditions, and our geotechnical conclusions and design recommendations are presented in the text of this report. Supporting data including detailed exploration logs, field exploration procedures, and results of laboratory testing are presented as appendices.

Our geotechnical engineering scope of services for the project included a site reconnaissance, subsurface evaluation, laboratory testing, geotechnical analysis, and preparation of this report. The geotechnical subsurface evaluation completed for this study included one exploratory boring (B-1) completed to a depth of 24 feet below grade.

1.1 Site Description

The subject site is a developed single-family residential property located at 10 Brook Bay Drive in Mercer Island, Washington. The site is a 0.41-acre, roughly triangular parcel located on the west side of Mercer Island. The site is bordered to the north by Brook Bay Drive and to the east, west, and south by developed single-family residential properties.

The site has a total of about 35 feet of topographic relief. The southeastern portion of the site generally consists of a moderate to steep, northwest-facing slope with a portion of the slope exceeding 40 percent. The northwestern portion of the site is relatively level to very gently sloping down to the northwest on the order of 2 to 4 percent.

The property supports a 3,410 square foot, wood-framed, single-family residence with a daylight basement reportedly build in 1973 and renovated in 2001 to 2002. The house is located near the toe of the moderate to steep northwest-facing slope. The daylight basement level includes a slab-on-grade concrete floor in the attached garage and crawl spaces below living areas. The house is accessed by a long, arcuate concrete driveway that extends southeast from Brook Bay Drive. A small west to northwest flowing stream crosses the northern portion of the property and is located about 5 to 15 feet north of the existing residence. The stream crosses below the driveway through a culvert pipe. Existing site conditions are shown on the enclosed *Site and Exploration Plan,* Figure 1.

1.2 Project Understanding

Based on a review of architectural drawings prepared by Floisand Studio, we understand that the project includes the following primary elements.

- Encloser of the entry stairs on the north-central side of the residence with new foundations to support the enclosure.
- An expansion of the basement level including an elevator. The expansion will extend about 14 feet south of the southern limit of the existing garage and central entry area. We understand that the expansion will include new cast-in-place concrete retaining walls.
- We understand that temporary excavations needed to construct the expanded basement retaining walls will compromise support for some of the existing shallow spread footings to the south and east of the new basement area. We understand that these foundations will be substantially unloaded at the time of construction by removal of portions of the timber-framed structure above. We further understand that the following three options to address this temporary loss of foundation support are under considerations.
    - Demolish the existing shallow foundations during construction of the new basement retaining walls and replace them with new foundations after the new retaining walls are backfilled with compacted structural fill.
    - Provide vertical support for the existing foundations using driven pipe piles.
    - Provide both vertical support and temporary shoring for existing foundations. We understand that temporary shoring would likely consisting of driven pipe piles or small W shape beams with timber lagging. We understand that the temporary shoring option is primarily being considered along a portion of the existing east perimeter shallow foundation.
- A new covered second-story deck on the west side of the residence. We understand that the west side of the deck will likely be supported by new posts and foundations. We anticipate that this will include the replacement of the existing on-grade concrete patio in this area.

1.3 Geologic Hazard Areas & Infiltration Considerations

The City of Mercer Island GIS Portal maps the southeastern portions of the site as Landslide, Steep Slope, and Erosion Hazard Areas, and the northwestern portion of the site as a Seismic Hazard Area. Chapter 19 of the Mercer Island City Code regulates development activities in Geologic Hazard Areas. We understand that the remodel will result in no net increase to impervious surfacing and that an evaluation of infiltration feasibility is not required.

2.0 SUBSURFACE CONDITIONS

2.1 Published Geologic Information

We assessed the geologic setting of the site and surrounding vicinity by reviewing the *Geologic Map of Mercer Island, Washington,* by Kathy G. Troost & Aaron P. Wisher, dated October 2006. The northwest-facing moderate to steep slope that comprises the southeastern portion of the site and what appears to include the southeastern portion of the residence are mapped as undifferentiated Pre-Olympia nonglacial deposits (Qpon). The Pre-Olympia deposits are glacially consolidated and typically provide good foundation support in their undisturbed condition. The level to gently sloping northwestern portion of the site and what appears to include the northwestern portion of the residence and western patio area are mapped as Quaternary alluvium (Qal). Alluvial deposits typically consist of silt, sand, and/or gravel deposited by flowing water. These deposits are normally consolidated and can be susceptible to settlement when in a very loose to medium dense condition and liquefaction if saturated. A deep-seated ancient landslide complex is mapped northeast of the subject property.

2.2 Soil Conditions

The geotechnical subsurface evaluation completed for this project included one exploratory boring (B-1) completed on March 21, 2022. The boring was completed on the west side of the residence adjacent to the existing slab-on-grade patio and second story wood deck and extended to depths of about 24 feet below existing grade. The approximate exploration location is presented on the enclosed *Site and Exploration Plan,* Figure 1.

Our geotechnical evaluation also included a review of two borings (ECI-1 and ECI-2) completed by others in 2002 near the south-central and southeast corner of the house and near the planned basement expansion area. These borings were located on the northwest-facing slope and extended about 16½ feet below grade. The approximate locations of ECI-1 and ECI-2 are presented on Figure 1. Descriptive log of ECI-1 and ECI-2 are presented in *Appendix C.*

Soil samples recovered from our boring were visually classified in general accordance with the Unified Soil Classification System. A detailed, descriptive log of the subsurface exploration and the procedures utilized in the subsurface exploration program are presented in *Appendix A*. Stratification boundaries on the boring log represent the approximate depth of changes in soil types, although the transition between materials may have been gradual.

ZGA Boring B-1 was located adjacent to the proposed deck on the west side of the residence and encountered soils interpreted as fill over alluvium over Pre-Olympia deposits. Fill was encountered from the surface to a depth of about 7 feet and generally consisted of very loose to loose, silty sand with trace gravel to sand with silt and gravel. Alluvial (stream) deposits were encountered below the fill and extended from about 7 to 12 feet below grade and generally consisted of medium dense sand with some silt to interbedded silt and silty sand. Pre-Olympia deposits consisting of hard silt with trace to some fine sand was encountered below the alluvium to a depth of about 22 feet. A predominantly granular Pre-Olympia deposit consisting of dense gravelly sand with some silt was encountered from 22 feet to the maximum depth explored at about 24 feet below grade. Boring B-1 was terminated at 24 feet with auger refusal.

Borings ECI-1 and ECI-2 (by others) were completed on the north-west facing slope near the planned basement expansion and encountered soils interpreted as fill and Pre-Olympia deposits. ECI-1 encountered about 7 feet of fill consisting of very loose to loose sand with silt over stiff silt that extended to the maximum depth explored at 16½ feet. ECI-2 encountered about one foot of dense sand over medium stiff to stiff silt, which extended to a depth of about 15 feet. Dense sand with silt was encountered from 15 to the maximum depth explored at 16½ feet below grade.

2.3 Groundwater Conditions

Perched groundwater was encountered at a depth of 7 feet in ZGA boring B-1 at the time of drilling. The groundwater in B-1 appeared to be perched within the alluvium encountered above the low permeability Pre-Olympia silt deposits. Groundwater was not indicated in borings ECI-1 or ECI-2 at the time of drilling.

Groundwater and soil moisture conditions should be expected to vary throughout the year due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the explorations were performed. Therefore, groundwater and soil moisture conditions during construction or at other times in the life of the development may be different than indicated on the logs.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

Based on the results of our subsurface explorations, laboratory testing, and geotechnical engineering analyses, the proposed residential improvements are feasible from a geotechnical perspective. Due to the presence of very loose to loose fill and medium dense to stiff alluvial soils below proposed foundations and the risk of liquefaction within the saturated alluvium under the design seismic event, we recommend that foundations be supported on driven pipe piles to provide adequate support and limit total and differential settlements to acceptable levels.

Geotechnical engineering recommendations for foundations, retaining walls, and concrete slabs 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 Geologic Hazard Area Considerations

Section 19.16.010 of the Mercer Island City Code defines Geologic Hazard Areas as areas susceptible to erosion, sliding, earthquake, or other geological events based on a combination of slope (gradient or aspect), soils, geologic material, hydrology, vegetation, or alterations, including Landslide Hazard Areas, Erosion Hazard Areas, and Seismic Hazard Areas.

The City of Mercer Island GIS Portal maps the southeastern portions of the site as Landslide, Steep Slope, and Erosion Hazard Areas, and the northwestern portion of the site as a Seismic Hazard Area. Our evaluation of the site relative to Geologic Hazard Areas is presented below.

3.2.1 Landslide Hazard Areas

Section 19.16.010 of the Mercer Island City Code defines Landslide Hazard Areas as those areas subject to landslides based on a combination of geologic, topographic, and hydrologic factors, including:

1. Areas of historic failures;
2. Areas with all three of the following characteristics:
    a) Slopes steeper than 15 percent; and
    b) Hillsides intersecting geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock; and
    c) Springs or ground water seepage;
3. Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements;
4. Areas potentially unstable because of rapid stream incision and stream bank erosion; or
5. Steep Slope. Any slope of 40 percent or greater calculated by measuring the vertical rise over any 30-foot horizontal run.

The southeast portion of the site is mapped by the City of Mercer Island as a Landslide and Steep Slope Hazard Area. Based on our site reconnaissance and review of the site topographic survey, it is our opinion that the southeast portion of the site appears consistent with criteria 5 for Steep Slopes. The approximate extent of the steep slope area is shown on *Figure 1*. As such, we are in general concurrence with the mapped designation.

3.2.2 Erosion Hazard Areas

Section 19.16.010 of the Mercer Island City Code defines Erosion Hazard Areas as those areas greater than 15 percent slope and subject to a severe risk of erosion due to wind, rain, water, slope and other natural agents including those soil types and/or areas identified by the U.S. Department of Agriculture's Natural Resources Conservation Service as having a "severe" or "very severe" rill and inter-rill erosion hazard.

The southeastern portion of the site is mapped by the City of Mercer Island as an Erosion Hazard Area. The U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS) Web Soil Survey maps the entire site and nearby vicinity as Kitsap silt loam (KpB) 2 to 8 percent slopes. This soil type is considered by the NRCS as having a moderate erosion hazard and as such would not meet the City of Mercer Island criteria for an Erosion Hazard Area. However, the site topographic map presented in *Figure 1* indicates that most of the northwest-facing slope in the southeastern portion of the site exceeds 15 percent and the NRCS considers the Kitsap silt loam (KpD) 15 to 30 percent slopes as having a severe erosion hazard. As such, we are in general concurrence with the mapped designation.

3.2.3 Seismic Hazard Areas

Section 19.16.010 of the Mercer Island City Code defines Seismic Hazard Areas as areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, soil liquefaction or surface faulting.

The northwestern portion of the site is mapped by the City of Mercer Island as a Seismic Hazard Area. Based on our review, the site is located within the Seattle Fault Zone and soils considered susceptible to liquefaction during the design seismic event were encountered in boring B-1. Based on these conditions, we are in general concurrence with the mapped designation, and it is our opinion that the relatively level portion of the northwestern portion of the parcel meets the criteria for a Seismic Hazard Area.

3.2.4 Statement of Risk

Per Section 19.07.060.D.2 of the Mercer Island City Code, development within geologic hazard areas require that a Geotechnical Engineer licensed within the State of Washington provide a statement of risk with supporting documentation indicating that one of the following conditions can be met:

a) The geologic hazard area will be modified, or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe; or
b) An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area; or
c) Development practices are proposed for the alteration that would render the development as safe as if it were not located in a geologic hazard area; or
d) The alteration is so minor as not to pose a threat to the public health, safety, and welfare.

Based on a review of architectural drawings prepared by Floisand Studio, we understand that the planned remodel includes the primary elements described in Section 1.2 of this report. The planned front entry and eastern patio site improvements are located within a Seismic Hazard Area due to the potential for liquefaction during the design seismic event. This report includes recommendation for pipe pile support of new foundations to mitigate potential liquefaction induced settlement. In accordance with Criteria a) above, it is our opinion that the risk to the lot and adjacent property will be eliminated or mitigated such that the site is determined to be safe provided that the geotechnical design recommendations presented in this report are incorporated into the project plans and implemented during construction.

3.3 Seismic Design Considerations

The seismic performance of the development was evaluated relative to seismic hazards resulting from ground shaking associated with a design seismic event with a 2 percent probability of exceedance in 50 years corresponding to a 2,475-year return period as determined in accordance with the 2018 International Building Code (IBC).

3.3.1 Fault Surface Rupture

The USGS Quaternary Fault Web Mapping Application indicates that the site is within the Seattle Fault Zone. The Seattle Fault zone is a collective term for a series of four or more east-west trending reverse fault splays located near the southern margin of the Seattle basin. The site is about ¼ mile from the nearest mapped splay within the fault zone. The fault zone is about 2.5 to 4 miles wide (north-south) and extends from the Kitsap Peninsula near Hood Canal to the Sammamish Plateau east of Lake Sammamish. Most of the fault zone is concealed by Holocene glacial and post-glacial deposits and is primarily mapped based on the location of magnetic anomalies. Geologic evidence indicates that ground surface rupture from movement on the Seattle Fault zone occurred about 1,050 years ago. The geologic record suggests that potential future movement of the fault zone may not occur for several thousand years (Johnson, et al., 1999, 2002). Given the relatively long return period of the Seattle Fault zone and the location of the nearest mapped splay within the fault zone relative to the project site, it is our opinion that the risk of ground surface rupture at the site is low.

3.3.2 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 0.626g, based on Figure 22-9 of ASCE 7-16.
- A Modified Peak Ground Acceleration (PGA_M) of 0.688g 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 USGS National Seismic Hazard Mapping Project deaggregation data for a seismic event with a 2% probability of exceedance in 50 years (2,475 year return period).

Our liquefaction analysis was completed using the computer program LiquefyPro Version 5.8. Our analysis was based on boring B-1 completed to a depth of about 24 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 within a saturated layer of alluvium encountered from about 7 to 12 feet below grade. The zone of saturation is due to a perched groundwater condition above hard, low permeability silt encountered at 12 feet.

3.3.3 Liquefaction Settlement

Based on our analyses, we estimate a total seismic settlement of about 3½ inches. Due to the shallow groundwater table, we estimate a differential seismic settlement of about 2½ inches over a horizontal distance of 40 feet.

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

3.3.5 IBC Seismic Design Parameters

Per the 2018 IBC seismic design procedures and ASCE 7-16, the presence of liquefiable soils requires a Site Class definition of F. However, through reference to Sections 11.4.8 and 20.3.1 of ASCE 7-16, the 2018 IBC allows site coefficients F_a and F_v to be determined assuming that liquefaction does not occur for structures with fundamental periods of vibration less than 0.5 seconds. Based on the results of the field evaluation, Site Class D may be used to determine the values of F_a and F_v in accordance with Sections 11.4.8 and 20.3.1 of ASCE 7-16. If exceptions for Site Class D presented in Section 11.4.8 of ASCE 7-16 do not apply, a ground motion hazard analysis may be required. Site Class D describes soils that are considered stiff with a shear wave velocity between 600 and 1,200 feet per second, average Standard Penetration Test values between 15 and 50, and an undrained shear strength between 1,000 and 2,000 psf.

IBC Seismic Design Criteria

Parameter	Value
2018 International Building Code Site Classification (IBC) [1]	Site Class F [2,3]
Site Latitude/Longitude	47.5525 /-122.2320
Spectral Short-Period Acceleration, S_S	1.461g
Spectral 1-Second Acceleration, S_1	0.507g
Site Coefficient for a Short Period, F_A	1.000
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}	1.461g
Spectral Acceleration for a 1-Second Period, S_{M1}	See ASCE Section 11.4.8
Design Short-Period Spectral Acceleration, S_{DS}	0.974g
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 24 feet below grade. ZGA therefore determined the Site Class assuming that dense to very dense glacially consolidated soils with an average n value greater than 30 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.

3.4 Site Preparation

3.4.1 Existing Structure Removal

We anticipate that site preparation will begin with demolition and removal of the existing second-story wood deck on the west side of the house and the north-central entry. We recommend that all existing foundation elements and utilities be completely demolished and removed from those portions of the building that will be reconstructed. We recommend that utilities outside the proposed building envelope be abandoned in accordance with City of Mercer Island guidelines.

3.4.2 Erosion Control Measures

Stripped surfaces and soil stockpiles are typically a source of runoff sediments. We recommend that silt fences, berms, straw wattles, and/or swales be installed around the downslope side of stripped areas and stockpiles and along the nearby stream in order to capture runoff water and sediment. If earthwork occurs during wet weather, we recommend that soil stockpiles and temporary cut slopes be protected with anchored plastic sheeting.

3.4.3 Temporary Drainage

Stripping, excavation, grading, and subgrade preparation should be performed in a manner and sequence that will provide drainage, control erosion, and protect prepared subgrades. The near surface soils primarily consist of silty sand with variable gravel content and have fines contents in excess of 5 percent by weight and are therefore susceptible to disturbance and erosion when wet. The site should be graded to prevent water from ponding in construction areas and flowing into excavations. Successful drainage of saturated zones due to accumulations of surface water could be relatively slow. Runoff from the surrounding areas should not be allowed to flow into excavations. We recommend that asphalt berms, sandbag, or ditches be used to divert runoff around the excavation area to a suitable discharge location. Drainage measures, such as ditches, sumps, and pumps, could also be employed at the base of the excavation for the below-grade portion of the building to minimize disturbance of the building subgrade from surface water runoff. A layer of crushed rock could be used to limit subgrade disturbance at the base of excavations.

3.4.4 Subgrade Preparation

Once site preparation and excavation are complete, we recommend that all areas that do not require over-excavation and are at design subgrade elevation or areas that will receive new structural fill be compacted to a firm and unyielding condition. Once exposed, subgrades should be evaluated by a representative of ZGA to assess the subgrade adequacy and to detect soft and/or yielding soils. 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. We anticipate that once subgrade soils become disturbed, they will be difficult to recompact due to the moisture conditions of the soil. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the subgrade soils.

If earthwork or construction activities take place during extended periods of wet weather, or if the *in-situ* moisture conditions are elevated above the optimum moisture content, the soils at the proposed excavation depths could become unstable or not be compactable. A layer of crushed rock or crushed concrete could be placed over the exposed subgrade to limit disturbance. We anticipate a 6- to 8-inch layer of clean crushed aggregate or crushed concrete would be sufficient to maintain suitable subgrade conditions, although the specific thickness and material type should be determined by the contractor at the time of construction based on weather, anticipated construction traffic, exposure duration, and actual soil and groundwater conditions encountered at the base of the excavation.

3.4.5 Freezing Conditions

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be compacted prior to placing subsequent lifts of structural fill or pouring foundations. Alternatively, the frozen material could be stripped from the subgrade to expose unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

3.5 Structural Fill Materials and Placement

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.

3.5.1 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 in order to complete the necessary Proctor tests.

3.5.2 Re-Use of Site Soils as Structural Fill

The near surface fill and alluvial soils encountered in boring B-1 appear suitable for reuse as structural fill from a compositional standpoint provided it is placed and compacted in accordance with the recommendations presented in this report. Soils exposed in the basement expansion area are expected to primarily consist of moisture-sensitive silt. Laboratory test results included in the boring logs for ECI-1 and ECI-2 indicate the silt has a fines content over 95 percent, moisture contents typically over 30 percent, and a plasticity index of about 33. Based on this data, we do not recommend the reuse of the onsite silt as structural fill.

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 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.

3.5.3 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. Imported fill should consist of well-graded sand or sand and gravel. Under wet conditions, the fines content should be limited to less than 5 percent (based by weight on the minus No. 4 sieve fraction using the wet sieve analysis). Typically, soils containing less than 5 percent fines can be compacted under a wider variety of weather conditions.

During extended periods of dry weather, we recommend imported fill meet the requirements of Common Borrow as specified in Section 9-03.14(3), Option 1, of the WSDOT Standard Specifications. During wet weather, higher-quality structural fill might be required, as Common Borrow may contain enough fines to be moisture sensitive. In this case, we recommend that imported structural fill meet the requirements of Gravel Borrow as specified in Section 9-03.14(1) of the WSDOT Standard Specifications.

3.5.4 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) 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. If the in-situ moisture conditions at the time of earthwork prevent adequate compaction of the soils, the soil will need to be aerated and dried to achieve the minimum recommended compaction levels for structural fill.

3.5.5 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. 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.

3.5.6 Compaction Criteria

Our recommendations for soil compaction are summarized in the following table. We recommend that a representative from ZGA 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*	

3.6 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. Therefore, 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 if worker access is necessary. 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 and groundwater conditions at the time of construction. Adjustments to the slope angles should be determined by the contractor at that time.

For preliminary planning purposes, we recommend a maximum temporary slope inclination of 1½H:1V (Horizontal:Vertical) in loose to medium dense alluvial and undocumented fill soils (WAC Type C soil) encountered in ZGA boring B-1 and within the upper 7 feet in boring ECI-1. We recommend a maximum temporary slope inclination of 1H:1V in undisturbed stiff silt soils (WAC Type B soil) encountered below 7 feet in boring EC-1 and within the entire depth explored at ECI-2. These preliminary temporary cut slope recommendations assume that the soils are in a dewatered, unsaturated condition. Flatter temporary slopes may be necessary, depending the site and subsurface conditions at the time of construction. In all cases, cut slopes and shoring should conform to applicable Federal, State and/or local safety guidelines.

We recommend that all permanent cut or fill slopes constructed in native soils be designed at a 2½H:1V (Horizontal:Vertical) inclination or flatter. All permanent cut and fill slopes should be adequately protected from erosion both temporarily and permanently.

3.7 Temporary Driven Soldier Pile Shoring

We understand that a temporary cantilever soldier pile shoring wall utilizing driven piles is being considered along a portion of the existing east perimeter shallow foundation to allow for construction of the basement expansion. Walls of this type are often constructed using 3-to-6-inch diameter driven pipe piles or small W shape beams. Typical beam sizes include W6x15, W6x20, W6x25, W8x25, W8x28, and W8x31. Construction of this types of wall typically require a specialty contractor and the type of pile used is often controlled by the contractor's equipment and capabilities. As such, we recommend that the shoring designer coordinate their efforts with a shoring contractor. McDowell Pile King is a local contractor experienced in this type of shoring wall construction.

Driven soldier pile walls consist of vertical steel piles or beams driven to a sufficient depth below the bottom of the planned cut in order to satisfy design force and moment equilibrium analysis requirements. Once the piles are installed, the excavation proceeds and lagging (typically dimensional lumber for temporary applications) is installed between the flanges of vertical beams and in front or behind vertical pipe piles to support the cut as the excavation extends down.

The following sections of this report provide geotechnical recommendations for a temporary cantilever driven soldier pile shoring system. The shoring design criteria presented in this report should be used by the shoring designer to design an appropriate shoring system. The shoring design should be reviewed by Zipper Geo Associates for conformance with design criteria presented herein. It is generally not the purpose of this report to provide specific criteria for construction methods, materials or procedures for shoring. It should be the responsibility of the shoring designer and contractor select appropriate materials and methods for design and construction.

3.7.1 Driven Soldier Pile Shoring and Lagging Design Parameters

The design of shoring is generally accomplished using empirical relationships and apparent earth pressure distributions. These earth pressure distributions or envelopes do not represent the precise distribution of earth pressures but rather constitute hypothetical pressures from which the shoring system can be designed. Additionally, pressures must be selected to limit deflections, both vertical and horizontal, of nearby settlement sensitive structures, roadways and utilities, if present. The design of soldier pile and lagging shoring should include lateral pressures exerted by the adjacent soil, surcharge loads from adjacent buildings, and other surcharges such as traffic, construction materials, or temporary soil stockpiles adjacent to the excavation. Lateral load resistance can be mobilized by passive pressures on members that extend below the bottom of the excavation.

Design of soldier pile shoring should be based on either "active" or "at-rest" lateral earth pressures, depending on the degree of deformation that the shoring wall can tolerate. Lateral wall movement for soldier pile shoring designed using active earth pressures typically range from about 0.2 percent to 0.5 percent of the wall height. The lateral movement is typically accompanied by vertical settlement of about 0.15 percent to 0.5 percent of the wall height with the maximum occurring immediately behind the wall face and trending to zero at a distance of roughly two times the wall height. If existing utilities or buildings within the zone of influence are considered to be insensitive to this degree of settlement, then it would be appropriate to design utilizing active earth pressures. An assumed "at-rest" earth pressure condition theoretically assumes no movement of the soil behind the shoring, however, some lateral deflection and settlement should realistically be anticipated due to construction practices and/or the fact that it is not possible to construct a perfectly stiff shoring system.

The attached *Lateral Earth Pressure for Temporary Driven Soldier Pile Wall, Figure 2,* provides our recommendations for cantilever soldier pile shoring design. Surcharge loads, if present, must be applied. Figure 3 provides pressure diagrams for lateral earth pressures resulting from vertical surcharges behind shoring walls. For traffic surcharges located within a 1H:1V envelope extending up from the bottom of excavation elevation at the face of the shoring wall, we recommend an equivalent soil surcharge of 2 feet (250 psf) be added.

We recommend that lagging be designed in general accordance with Section 6-16.3(6) of the WSDOT Standard Specifications. For purposes of lagging design, the site soils may be classified as Type 1 soils. Prompt and careful installation of lagging will reduce potential loss of ground. The requirements for lagging should be made the responsibility of the shoring subcontractor to prevent soil failure, sloughing and loss of ground and to provide safe working conditions. We recommend all void space between the lagging and soil be backfilled. We recommend the backfill consist of free-draining sand and gravel in order to prevent the build-up of post-construction hydrostatic pressure behind the wall.

We understand that the soldier piles may be used to provide temporary or permanent support of existing foundations. Piles used for permanent foundation support should include adequate corrosion protection or be designed for an appropriate cross sectional area loss over the design life of the structure. Based on our review of project plans and discussions with the project structural engineer Malsam Tsang, we understand that the soldier pile shoring wall will consist of driven W8x48 beams with an embedment of about 12 feet and a maximum design axial load of 6.34 kips per pile. Based on the site subsurface conditions and our calculations, these beams would have an ultimate vertical capacity (end bearing plus skin friction) of 100 kips per pile. This indicates a safety factor of about 15. Given the light loads and high safety factor, it is our opinion that axial load testing of driven W8x48 soldier pile beams is not needed. Section 3.8 of this report provides allowable axial capacities for driven pipe piles.

3.7.2 Shoring Monitoring Plan

Any time an excavation is made below the level of existing buildings, utilities or other structures, there is risk of damage even if a well-designed shoring system has been planned. In order to establish the condition of existing facilities prior to construction, we recommend that the owner and/or owners representatives make a complete inspection and evaluation of pavements, structures, and utilities around the proposed excavation. This inspection should be directed towards detecting any existing signs of damage, particularly those caused by settlement or lateral movement. The observations should be documented by pictures, notes, survey drawings, or other means of verification. The contractors also should establish for their own records the existing conditions prior to construction.

Prior to and during construction, the monitoring program should include measurements of the horizontal and vertical movements of the retained soils, adjacent structures, and the shoring system itself. At least two reference lines should be established adjacent to the excavation at horizontal distances back from the excavation face of about 1/4H and H, where H is the final excavation height. Monitoring of the shoring system should include measurements of vertical and horizontal movements at the top of each soldier pile. If local wet areas are noted within the excavation, additional monitoring points should be established at the direction of ZGA.

The measuring system used for shoring monitoring should have an accuracy of at least 0.01-foot. All reference points on the existing structures should be installed and readings taken prior to commencing the excavation. All reference points should be read prior to and during critical stages of construction. The frequency of readings will depend on the results of previous readings and the rate of construction. As a minimum, readings should be taken about once a week throughout construction until the permanent basement walls are completed and braced up to the ground level of the building. All readings should be reviewed by the geotechnical and structural engineers.

3.8 Pipe Pile Foundation Support

Boring B-1 disclosed about 12 feet of fill and alluvial soils that generally consisted of very loose to medium dense silty sand with some gravel to sand with silt. In our opinion, these soils are not suitable for direct foundation support. In addition, the saturated alluvium is considered susceptible to liquefaction induced settlement during the 2018 IBC design seismic event. To limit potential total and differential static and seismic settlements to acceptable levels, we recommend supporting new foundations on driven pipe piles.

Pipe piles consist of relatively small diameter steel pipe that is driven into the ground with a pneumatic or hydraulic jackhammer, or percussion driver, to a designated “refusal” criteria. Individual pipe sections of 5 to 20 feet in length are commonly used. Successive pipe lengths are either compression coupled, threaded, or welded together. Once the piles are installed, the top of the piles are cut off to a pre-determined elevation, and lengths of reinforcing steel, or top plates are connected to the pile top. The tops of the piles are then incorporated into the new foundations or floor slabs as determined by the structural engineer. Geotechnical design recommendations and construction considerations for pipe pile foundations are presented in the following sections.

3.8.1 Axial Pile Capacity

We recommend that galvanized, steel pipe be utilized for the project. Two-inch diameter piles, consisting of schedule 80 pipe, are typically installed with a 90-pound jack hammer. Three- to 6-inch diameter pin piles, consisting of schedule 40 pipe, are typically installed using a hydraulic hammer attached to a small excavator. We recommend that the piles be driven to “refusal”. Refusal is defined as one inch or less of penetration into the ground over a specified time interval (in seconds) of sustained driving. The time interval used to define refusal is based on the type of hydraulic hammer used. Refusal criteria time intervals for several hammer types typically used to install 2-, 3-, 4-, and 6-inch diameter pin piles are presented in the following table.

Hammer Refusal Criteria						
Hammer Model No.	Actual Hammer Weight (lbs.)	Hammer Foot-Pound Class	Refusal Criteria (seconds/inch)			
			2” dia.	3” dia.	4” dia.	6” dia.
Jackhammer	90	-	60	-		
TB 225	650	550	-	12		
TB 325	850	850	-	10	16	
TB 425	1100	1100	-	6	10	20
TB 725	2000	2000	-	-	4	10
TB 830	3000	3000				6

We recommend using allowable axial compressive capacities of 6 kips, 12 kips, 20kips, and 30 kips, respectively, for 2-, 3-, 4-, and 6-inch diameter, steel pipe pile driven to refusal as described herein. The recommended allowable capacities include a factor of safety of 2. Based on the subsurface conditions encountered in our explorations, we recommend a minimum pile tip elevation of 33 feet to provide embedment into the Pre-Olympia deposits. Deeper embedment may be necessary to achieve refusal conditions and the design bearing capacity.

Due to the relatively small diameter of pin piles, we recommend that lateral resistance of vertical pin piles be neglected for permanent loading conditions. Instead, lateral loads should be accommodated by battered piles and/or passive soil pressure on the face of grade beams, tie-beams and other buried foundation elements. An allowable passive resistance of 250 pcf may be utilized for those foundations embedded at least 18 inches below grade. The upper 1 foot of embedment should be neglected when evaluating passive resistance. Sliding friction on the base of pile supported footings should be ignored as the weight of the structure is primarily carried by the supporting piles and not the soil. We also recommend that uplift resistance of small-diameter pin piles be neglected. A structural engineer should prepare the pile support design.

If battered pipes will be designed to support both vertical and lateral loads, we recommend a maximum batter of 1H:6V (Horizontal:Vertical). We recommend that piles used in this manner be battered in opposite directions in an “A” frame configuration. If a combination of vertical piles to support vertical loads and battered piles to resist lateral loads are used, then the battered piles may be inclined up to 1V:1H. Batter pile inclinations should be limited as needed to prevent piles from extending beyond property lines. Vertical and lateral allowable pile capacities for battered piles are equal to the vector component of the axial pile load (not axial capacity).

3.8.2 Settlement

We estimate that settlement of foundations supported on pipe piles designed and installed as recommended will be on the order of ½ inch or less. Most of this settlement is expected to occur rapidly as loads are applied. Post-construction differential settlements should be minor.

3.8.3 Pile Load Tests

In our opinion, 2-inch diameter pipe piles driven to refusal do not need to be load tested. We recommend that a minimum of 3 percent of 3- to 6-inch diameter piles (up to 5 piles maximum and 1 pile minimum for each diameter) be load tested in compression in accordance with the requirements presented in ASTM D 1143, *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load, Quick Load Test Method for Individual Piles*. In order to document consistent installation between the tested and untested pipe piles, we recommend that ZGA monitor all pipe pile installations and load testing.

3.8.4 Construction Considerations

Obstructions including cobbles, boulders, or undisclosed objects from past site development may be encountered and impede the penetration of individual pin piles. The contractor and structural engineer should be prepared to adjust the location of the pin piles if obstructions are encountered.

3.9 On-Grade Concrete Slabs

We anticipate that the new basement and entry may include a slab-on-grade concrete floor. We also understand that a new on-grade concrete patio may replace the existing concrete patio on the west side if the building.

Based on the subsurface exploration completed for this project, we anticipate that loose to medium dense existing fill and alluvium may be present below some of the anticipated slab subgrade locations and elevations. The existing concrete patio on the west side of the house appears to be in serviceable condition with only minor cosmetic cracking. As such, we anticipate that new concrete slabs supported on similar soils will perform in a similar fashion. However, supporting on-grade slabs on these soils has a low risk of settlement and associated cracking. The following recommendations for on-grade concrete slabs assume that this risk is acceptable to the owner. If the potential for minor settlements associated with the existing fill and alluvial soils are not acceptable, we recommend that the project utilize a structural slab supported on driven pipe piles. Alternatively, the potential for cracking can be controlled to occur at specific locations through the use of steel reinforcement within the slab and/or properly designed control joints.

3.9.1 Subgrade Preparation

We recommend that the exposed subgrade be compacted to a firm and non-yielding condition and to at least 95 percent of the modified Proctor maximum dry density per ASTMD 1557. Where unsuitable soil is present or where soils cannot be compacted to the recommend level, we recommend that the material be over-excavated and replaced with common borrow or select borrow, depending on the prevailing weather conditions. Subgrades should be prepared in accordance with the recommendations presented in the Subgrade Preparation section of this report.

3.9.2 Slab Base

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

3.9.3 Vapor Retarder

From a geotechnical perspective, a vapor retarder is not needed for outdoor slabs such as a patio. Where potential slab moisture is a concern or where moisture sensitive floor coverings are planned, we recommend that a 10- to 15-mil moisture barrier be installed beneath all interior slabs. We recommend using a puncture-resistant product such as Stego Wrap or an approved equivalent that is classified as a Class A vapor retarder in accordance with ASTM E1745. Puncturing the vapor barrier should be avoided; construction traffic should not be allowed to drive over any vapor barrier material. The slab designer and contractor should refer to ACI 302 for procedures and cautions regarding the use and placement of a vapor retarder. We recommend that installation of the vapor barrier be completed in accordance with the manufacturer's recommendations.

3.9.4 Subgrade Modulus

For design of on-grade concrete slabs supported on medium dense to very dense native soils or compacted structural fill, we recommend a vertical modulus of subgrade reaction of 150 pounds per cubic inch (pci) be used.

3.10 Backfilled Walls

We anticipate that the basement expansion will include new backfilled cast-in-place concrete retaining walls. Geotechnical design recommendations for backfilled walls are presented below. For recommended foundation support and lateral resistance parameters, refer to Section 3.8 *Pipe Pile Foundation Support*.

3.10.1 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 as described in the following paragraphs, 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 retaining walls with level backfill conditions, we recommend that an unfactored uniformly distributed seismic pressure of 14H 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. Section 3.11 of this report provides drainage recommendations for walls. If groundwater is allowed to saturate the backfill soils, hydrostatic pressures will act against a retaining wall.

3.11 Drainage Considerations

3.11.1 Surface Drainage

Final site grades should be sloped to carry surface water away from the building and other drainage-sensitive areas. Additionally, site grades should be designed such that concentrated runoff on softscape surfaces is avoided.

3.11.2 Footing Drains

We recommend that foundations for the new entry and other continuous shallow spread footings 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 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. We recommend that the new footing drain be connected to the buildings existing footing drain system. Roof drains should be connected to the footing drain system.

3.11.3 Free Standing Cast-In-Place Concrete Retaining Walls

Adequate drainage measures must be installed to collect and direct subsurface water away from subgrade walls and elevator pits. All backfilled walls should include a drainage aggregate zone extending two feet minimum from the back of wall for the full height of the wall. The drainage aggregate should consist of material meeting the requirements of WSDOT 9-03.12(2) Gravel Backfill for Walls. A minimum 4-inch diameter, 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 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.

3.11.4 Walls Cast Against Soldier Pile Walls

Permanent drainage of walls cast directly against soldier pile walls with timber lagging should be provided with prefabricated drainage matting (such as Miradrain or J-Drain 400). We recommend that it be placed on the entire outside face of shoring for the full width and height of the walls where feasible. In areas where the timber lagging is recessed inside the web area between pipes, the drainage matting shout extend horizontally from pile web to pile web. Prior to concreting, we recommend that the open edges of all drainage composites be covered with sheet plastic to reduce the risk of the shotcrete plugging the drainage composite. Near the bottom of the wall, a prefabricated drainage strip (such as Drain Grate) should be connected to the drainage matting. The drainage composite should be fitted with 3-inch minimum diameter weep hole pipes that will extend through the face of the permanent foundation wall. The weep hole pipes should be connected to a tightline system leading to a suitable discharge.

The prefabricated drainage mat and protective sheet plastic should not be considered a waterproofing membrane. The primary purpose of the drainage mat is to provide a means of draining groundwater seepage and preventing the build-up of hydrostatic pressure behind a wall. If the potential for groundwater seepage and/or the development of efflorescence on the interior wall against the cut side of the site are unacceptable, a waterproofing membrane should be installed between the drainage mat covered shoring wall and the concrete basement wall. Waterproofing measures should be designed and detailed by a waterproofing specialist.

3.12 Stormwater Infiltration

We understand that the remodel will result in no net increase to impervious surfacing and that an evaluation of infiltration feasibility is not required.

3.14 Ground Vibration Considerations

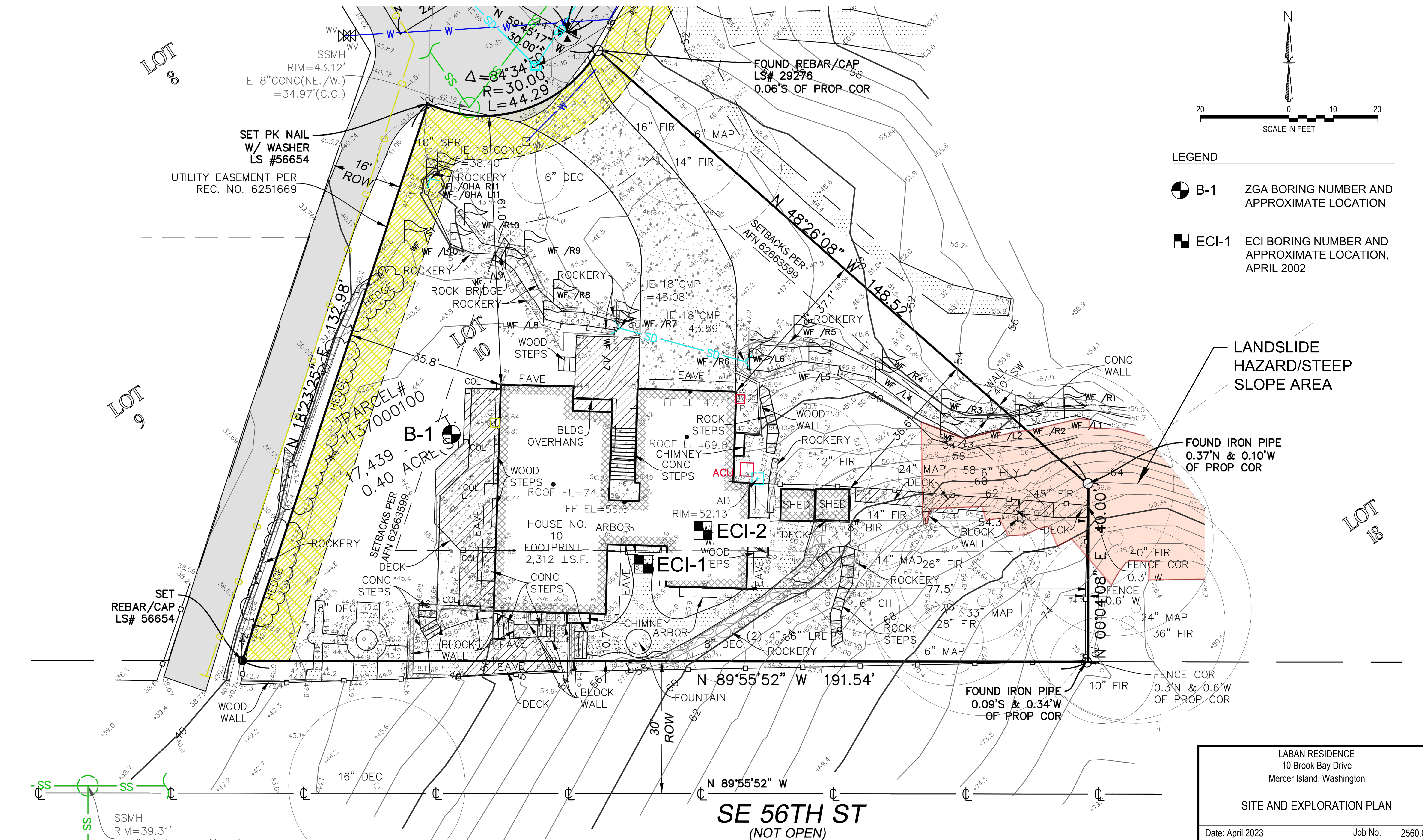
The installation of driven pipe piles for foundation support and driven W-shape beams for shoring, if used, typically generates low to moderate ground vibrations, depending on the pile diameter/beam size and drive hammer size. Vibration damage to nearby portions of the existing structure may be possible depending on the condition and proximity of the building element at the time of construction. We recommend that the contractor monitor the existing building for signs of distress during pile driving and take appropriate measures to mitigate damage, if observed.

4.0 CLOSURE

The analysis and recommendations presented in this report are based, in part, on explorations completed by ZGA. The number, location, and depth of the explorations were completed within the constraints of budget and site access so as to yield the information to formulate our geotechnical recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend we be provided an opportunity to review the final plans and specifications when they become available in order to assess that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design.

The performance of earthwork, structural fill, floor slabs, foundations, and retaining walls 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 Mina & Balsa Laban, 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, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Zipper Geo Associates, LLC reviews the changes and either verifies or modifies the conclusions of this report in writing.

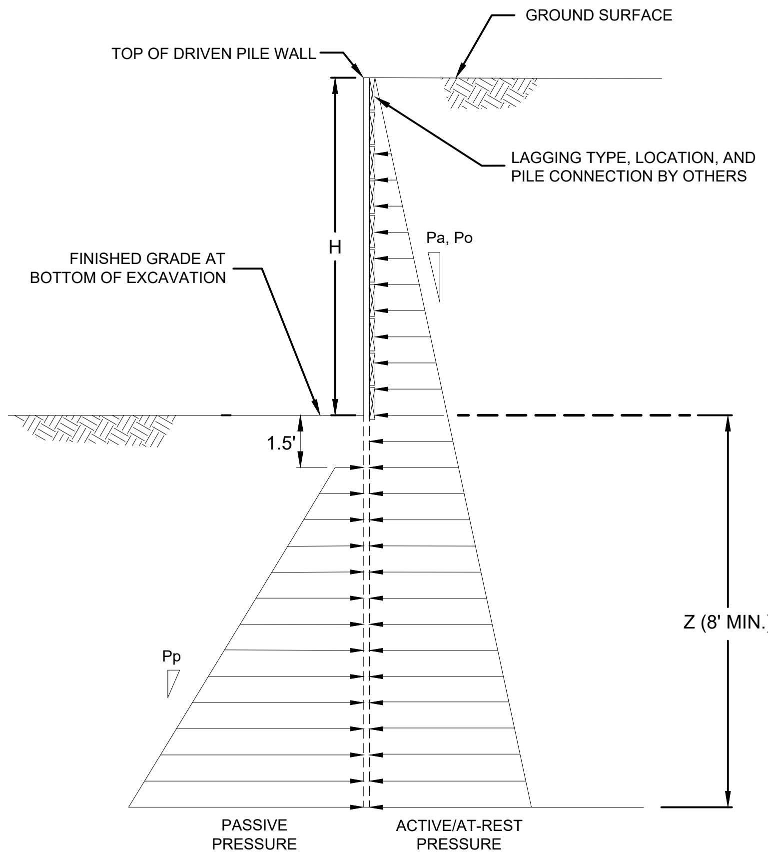


LABAN RESIDENCE
10 Brook Bay Drive
Mercer Island, Washington

SITE AND EXPLORATION PLAN

Date: April 2023	Job No.	2560.01
Zipper Geo Associates, LLC 19019 36th Ave. W.,Suite E Lynnwood, WA	FIGURE SHT.1of 1	1

FIGURE TAKEN FROM "TOPOGRAPHIC & BOUNDARY SURVEY" PARCEL NO. 1137000100 LABAN RESIDENCE 10 BROOK BAY RD, MERCER ISLAND, WA 98040, PRODUCED BY TERRANE 4/20/23



CANTILEVER PRESSURE DIAGRAM

DRIVEN SOLDIER PILE SHORING DESIGN PARAMETERS			
BACKSLOPE	ACTIVE EARTH PRESSURE, Pa (pcf)	AT-REST EARTH PRESSURE, Po (pcf)	ULTIMATE PASSIVE PRESSURE RESISTANCE, Pp (pcf)
LEVEL	35	55	425
REFER TO SECTION 3.7 OF GEOTECHNICAL REPORT FOR END BEARING CAPACITY OF DRIVEN PIPE PILES.			

NOTES:

1. ALL DIMENSIONS IN FEET.

2. PASSIVE PRESSURE APPLIES OVER THREE DRIVEN PIPE PILE DIAMETERS (OR THREE TIMES THE FLANGE WIDTH IF DRIVEN W OR H SHAPE BEAMS ARE USED) OR PILE SPACING, WHICHEVER IS LESS. PASSIVE LATERAL EARTH PRESSURE PRESENTED HEREIN IS ULTIMATE. AN APPROPRIATE SAFETY FACTOR SHOULD BE APPLIED

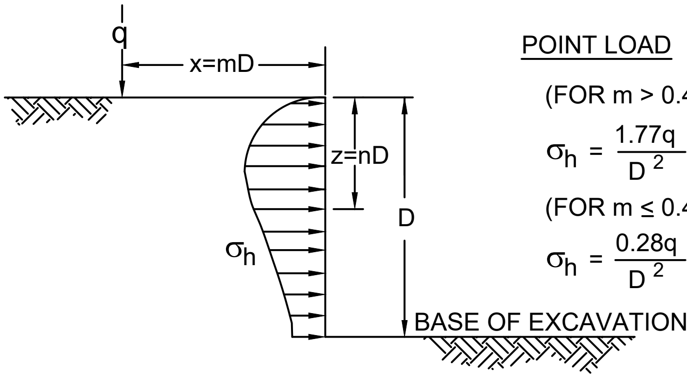
3. ACTIVE/AT-REST PRESSURE APPLIES OVER PILE SPACING ABOVE EXCAVATION BASE AND OVER ONE PILE DIAMETER/FLANGE WIDTH BELOW EXCAVATION BASE.

4 SHORING DESIGN MUST SATISFY FORCE AND MOMENT EQUILIBRIUM ANALYSES.

5. DESIGN LAGGING IN ACCORDANCE WITH SECTION 6-16.3(6) OF THE WSDOT STANDARD SPECIFICATIONS FOR SOIL TYPE 1.

6. SEE REPORT TEXT AND FIGURE 3 FOR CALCULATION OF SURCHARGE LOADS ACTING ON THE WALL, AND FOR ADDITIONAL RECOMMENDATIONS.

LABAN RESIDENCE 10 Brook Bay Drive Mercer Island, Washington	
LATERAL EARTH PRESSURES TEMPORARY DRIVEN SOLDIER PILE WALL	
Date: February 2023	Job No. 2560.01
Zipper Geo Associates, LLC 19019 36th Ave. W.,Suite E Lynnwood, WA, 98036	FIGURE 2 SHT. 1 of 1



POINT LOAD

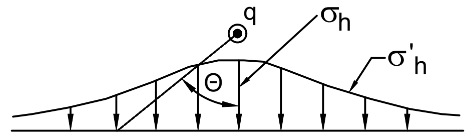
(FOR m > 0.4)

$$\sigma_h = \frac{1.77q}{D^2} \bullet \frac{m^2 n^2}{(m^2 + n^2)^3}$$

(FOR m ≤ 0.4)

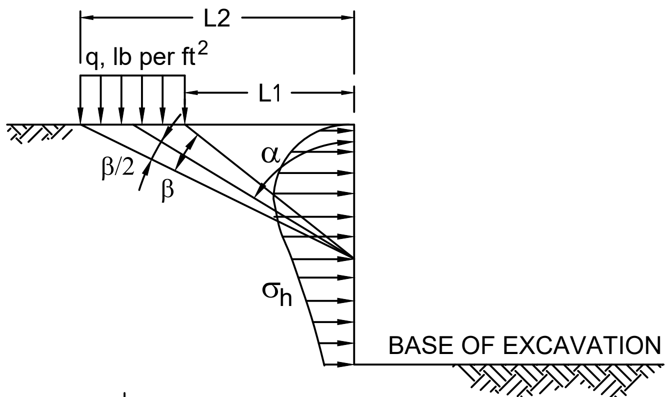
$$\sigma_h = \frac{0.28q}{D^2} \bullet \frac{n^2}{(0.16 + n^2)^3}$$

$$\sigma'_h = \sigma_h \cos^2 (1.1{*}\Theta)$$



PLAN VIEW OF WALL

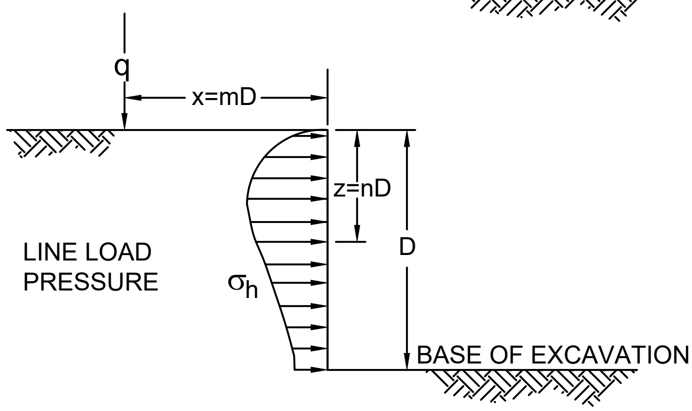
Θ in degrees



STRIP LOADING PARALLEL TO EXCAVATION

$$\sigma_h = \frac{2q}{\pi} (\beta - \sin \beta \cos 2\alpha)$$

α and β in radians



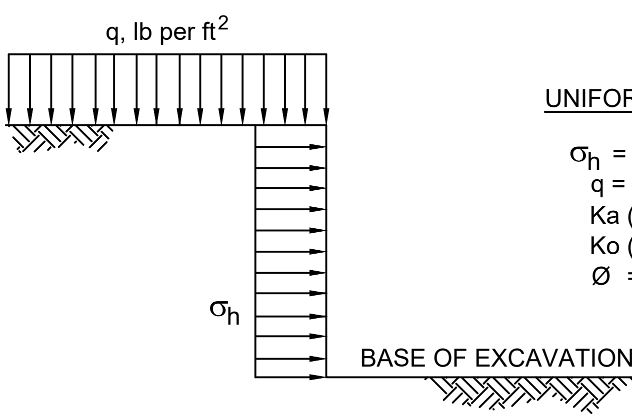
LINE LOAD

(FOR m > 0.4)

$$\sigma_h = \frac{1.28q}{D} \bullet \frac{m^2 n}{(m^2 + n^2)^2}$$

(FOR m ≤ 0.4)

$$\sigma_h = \frac{q}{D} \bullet \frac{0.2n}{(0.16 + n^2)^2}$$



UNIFORM LOAD DISTRIBUTION

σ_h = (Ka or Ko) q
q = Vertical pressure in psf
Ka (Active) = $\tan^2(45 - Ø/2)$
Ko (At Rest) = 1 - Sin Ø
Ø = 34°

LABAN RESIDENCE 10 Brook Bay Drive Mercer Island, Washington	
LATERAL PRESSURES FROM SURCHARGE LOADS	
Date: February 2023	Job No. 2560.01
Zipper Geo Associates, LLC 19019 36th Ave. W.,Suite E Lynnwood, WA, 98036	FIGURE 3 SHT. 1 of 1

APPENDIX A

SUBSURFACE EXPLORATION PROCEDURES AND LOGS

APPENDIX A
SUBSURFACE EXPLORATION PROCEDURES AND LOGS

Subsurface Exploration Description

Our field exploration included one boring completed on March 21, 2022. The approximate exploration location is presented on the enclosed *Site and Exploration Plan, Figure 1*. The exploration location was determined by measuring distances from existing site features shown on Figure 1. Ground surface elevations for the exploratory boring location were interpolated from topographic lines and spot elevations presented on Figure 1. As such, the exploration location and elevation should be considered accurate only to the degree implied by the measurement methods. The following sections describe our procedures associated with the exploration. A descriptive log of the exploration is enclosed in this appendix.

<u>Soil Borings</u>

The boring was advanced by Geologic Drill, Inc. working under subcontract to our firm. The boring was advanced using a limited access Acker drill rig 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 visual classification and testing.

Throughout the drilling operation, soil samples were obtained at 2.5- to 5-foot intervals by means of the Standard Penetration Test (ASTM: D-1586). This testing and sampling procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or “blow count” (N value). If a total of 50 blows are struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring log describes the vertical sequence of soils and materials encountered in the boring, based primarily upon our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our log indicates the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our log also graphically indicates the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring, as well as any laboratory tests performed on these soil samples. If groundwater was encountered in a borehole, the approximate groundwater depth and date of observation is depicted on the log. The boring log presented in this appendix is based upon the drilling action, observation of the samples secured, laboratory test results, and field logs. The various types of soils are indicated as well as the depth where the soils or characteristics of the soils changed. It should be noted that these changes may have been gradual, and if the changes occurred between sample intervals, they were inferred.

Boring Location:	See Figure 1, Site and Exploration Plan	Drilling Company:	Geologice Drill	Bore Hole Dia.:	5-inch
Top Elevation:	45 Feet	Drilling Method:	Hollow Stem Auger	Hammer Type:	Cathead
Date Drilled:	3/21/2022	Drill Rig:	Acker	Logged by:	BGF

B-1

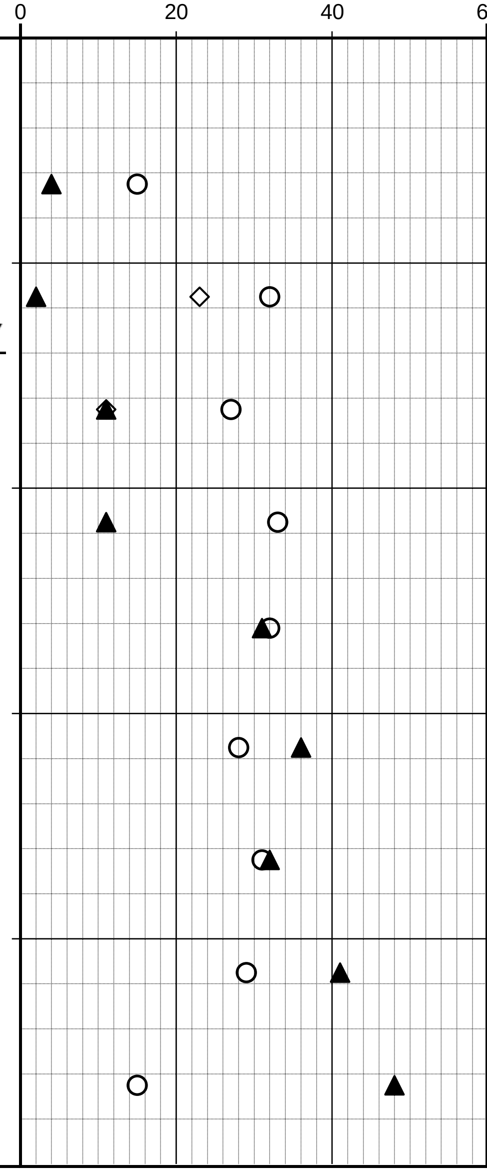
SOIL DESCRIPTION

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

PENETRATION RESISTANCE (blows/foot)

▲ Standard Penetration Test

△ Hammer Weight and Drop:



Depth (ft)	Soil Description	Sample Number	Recovery (in.)	Ground Water	Blow Counts	Testing
0	1-2 inches of grass sod over					
	Loose, wet, brown, silty SAND, trace gravel (Possible Fill)	S-1	8		4	
5	Very loose, wet, grayish-brown, SAND, with silt, trace gravel	S-2	18		2	GSA
	Medium dense, saturated, grayish-brown, SAND, some silt (Alluvium)	S-3	18	ATD	11	GSA
10	Stiff, saturated, brown to tan, SILT, interbedded silty SAND	S-4	18		11	
	Hard, wet, tan, SILT, trace to some interbedded sand, trace gravel (Pre-Olympia Deposits)	S-5	18		31	
15	Hard, wet, light brown to tan, SILT, trace to some fine sand	S-6	18		36	
	Hard, wet, light brown to tan, SILT, trace to some fine sand	S-7	18		32	
20	Hard, wet, brown - tan, sandy SILT, trace to some gravel	S-8	18		41	
	Dense, wet to saturated, brown-gray, gravely SAND, some silt (Pre-Olympia Deposits)	S-9	18		48	
25	Boring completed at about 24 feet. Perched groundwater observed at approximately 7 feet at time of drilling.					

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

Laban Residence
10 Brook Bay Drive
Mercer Island, WA

Date: April 2022 Project No.: 2560.01

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

BORING LOG: B-1

APPENDIX B

GEOTECHNICAL LABORATORY TESTING
PROCEDURES AND RESULTS

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.

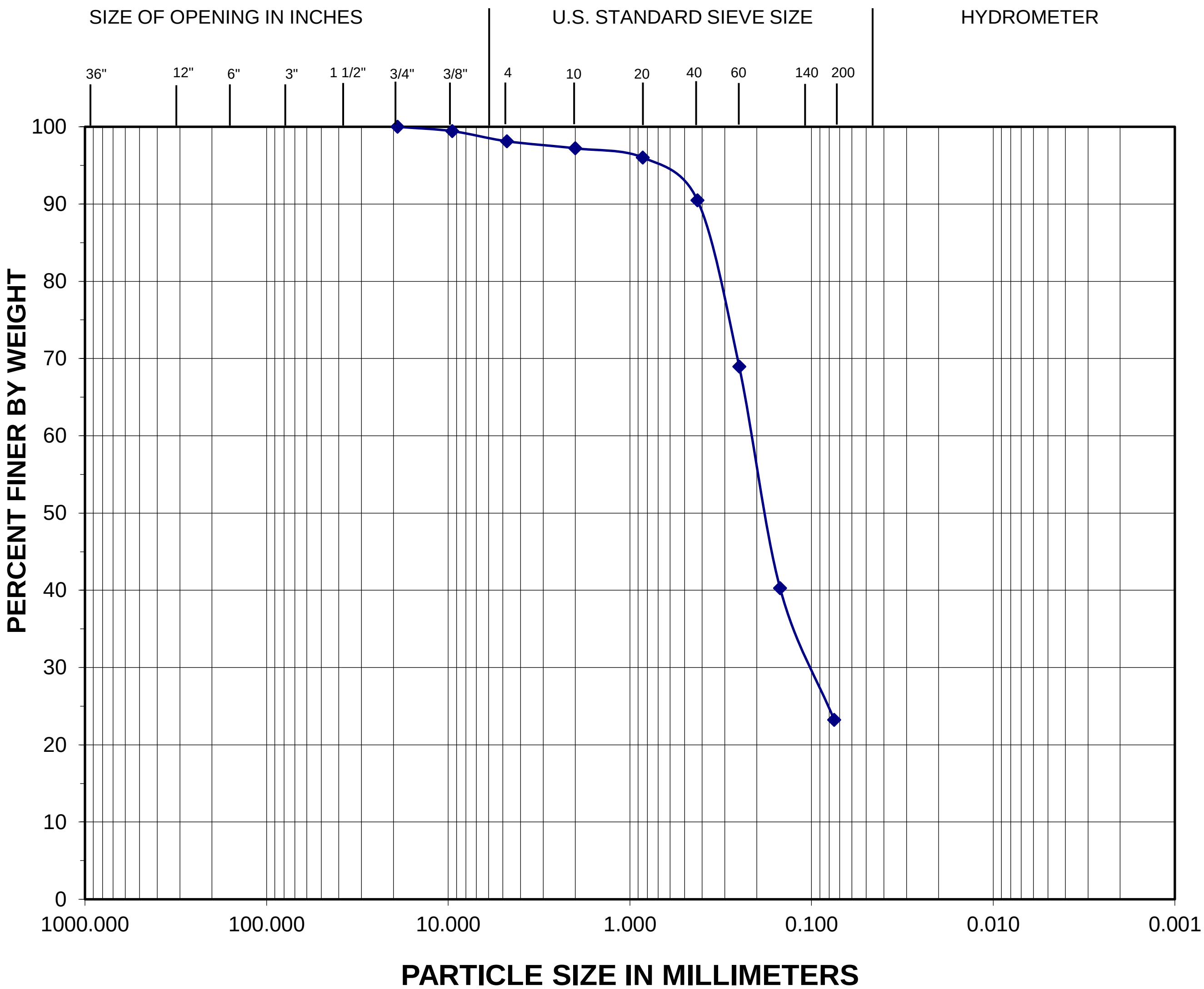
Grain Size Analysis

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.

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

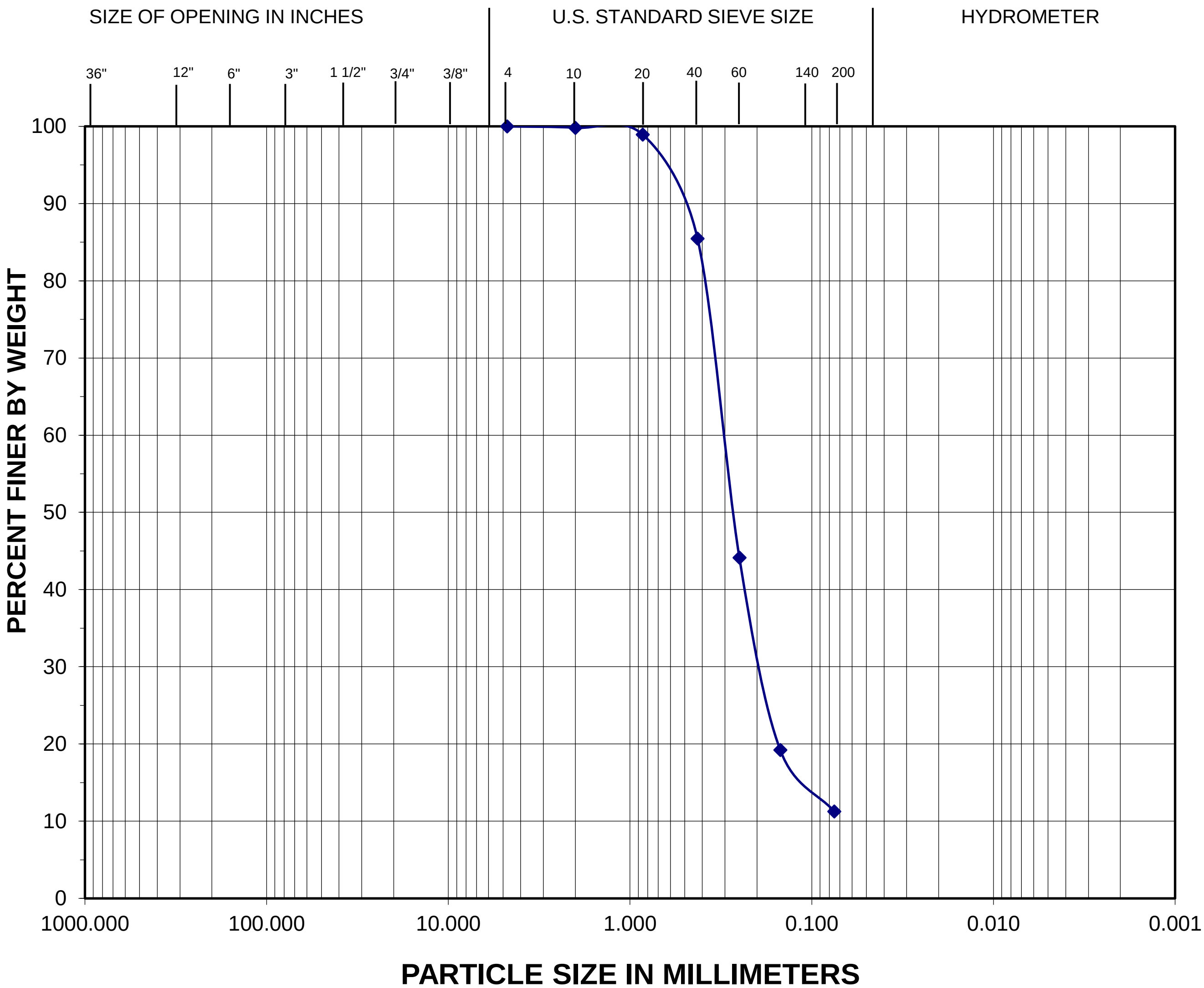
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-2	5 ft.	32.4	23.2	SAND, with silt, trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2560.01 DATE OF TESTING: 3/24/2022	PROJECT NAME: Laban Residence

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-3	7.5 ft.	26.6	11.2	SAND, some silt

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2560.01 DATE OF TESTING: 3/24/2022	PROJECT NAME: Laban Residence

APPENDIX C

SUBSURFACE EXPLORATION LOGS AND LABORATORY TEST RESULTS BY OTHERS

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTION
Coarse Grained Soils	Gravel And Gravelly Soils	Clean Gravels (little or no fines)		GW gw	Well-Graded Gravels, Gravel-Sand Mixtures, Little Or No Fines
				GP gp	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little Or No Fines
	More Than 50% Coarse Fraction Retained On No. 4 Sieve	Gravels With Fines (appreciable amount of fines)		GM gm	Silty Gravels, Gravel-Sand-Silt Mixtures
				GC gc	Clayey Gravels, Gravel-Sand-Clay Mixtures
More Than 50% Material Larger Than No. 200 Sieve Size	Sand And Sandy Soils	Clean Sand (little or no fines)		SW sw	Well-Graded Sands, Gravelly Sands, Little Or No Fines
				SP sp	Poorly-Graded Sands, Gravelly Sands, Little Or No Fines
	More Than 50% Coarse Fraction Passing No. 4 Sieve	Sands With Fines (appreciable amount of fines)		SM sm	Silty Sands, Sand - Silt Mixtures
				SC sc	Clayey Sands, Sand - Clay Mixtures
Fine Grained Soils	Silts And Clays	Liquid Limit Less Than 50		ML ml	Inorganic Silts & Very Fine Sands, Rock Flour, Silty-Clayey Fine Sands; Clayey Silts w/ Slight Plasticity
				CL cl	Inorganic Clays Of Low To Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean
				OL ol	Organic Silts And Organic Silty Clays Of Low Plasticity
More Than 50% Material Smaller Than No 200 Sieve Size	Silts And Clays	Liquid Limit Greater Than 50		MH mh	Inorganic Silts, Micaceous Or Diatomaceous Fine Sand Or Silty Soils
				CH ch	Inorganic Clays Of High Plasticity, Fat Clays
				OH oh	Organic Clays Of Medium To High Plasticity, Organic Silts
Highly Organic Soils				PT pt	Peat, Humus, Swamp Soils With High Organic Contents

Topsoil		Humus And Duff Layer
Fill		Highly Variable Constituents

The discussion in the text of this report is necessary for a proper understanding of the nature of the material presented in the attached logs.

DUAL SYMBOLS are used to indicate borderline soil classification.

C	TORVANE READING, tsf
qu	PENETROMETER READING, tsf
W	MOISTURE, % dry weight
P	SAMPLER PUSHED
*	SAMPLE NOT RECOVERED
pcf	DRY DENSITY, lbs. per cubic ft.
LL	LIQUID LIMIT, %
PI	PLASTIC INDEX

- 2" O.D. SPLIT SPOON SAMPLER
- 24" I.D. RING OR SHELBY TUBE SAMPLER
- WATER OBSERVATION WELL
- ▽ DEPTH OF ENCOUNTERED GROUNDWATER DURING EXCAVATION
- ▼ SUBSEQUENT GROUNDWATER LEVEL W/ DATE

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

LEGEND

Proj. No. 10057 | Date Apr. 2002 | Plate A1

Boring Log

Project Name: Post Residence: 10 Brook Bay Road				Sheet 1 of 1
Job No. 10057	Logged by: SSR	Start Date: 3/29/02	Completion Date: 3/29/02	Boring No.: B-1
Drilling Contactor: Geologic Drill		Drilling Method: HSA		Sampling Method: SPT
Ground Surface Elevation: 100'		Hole Completion: ☐ Monitoring Well ☐ Piezometer ☒ Abandoned, sealed with bentonite		

General Notes	W (%)	No. Blows Ft.	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Grass
						SP/SM	Brown poorly graded SAND with silt, loose, moist (Fill)
				1			-trace organics
				2			
	22.8	4		3			
				4			
	8.1	10		5			-9% fines -contains gravel
				6			
				7			-silt content increases
						ML	Brown SILT, medium dense, moist
	31.1	17		8			
				9			-iron oxide staining -becomes gray -96% fines
				10			
				11			
				12			
				13			-occasional gravel
				14			
	34.1	27		15			
				16			
							Boring terminated at 16.5 feet below existing grade. No groundwater encountered during drilling. Boring backfilled with bentonite and cuttings.

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

Boring Log
Post Residence: 10 Brook Bay Road
Mercer Island, Washington

Proj. No. 10057	Dwn. GLS	Date April 2002	Checked SSR	Date 4/30/02	Plate A2

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

BORING LOG 10057.GPJ ECI.GDT 4/29/02

Boring Log

Project Name: Post Residence: 10 Brook Bay Road				Sheet 1 of 1
Job No. 10057	Logged by: SSR	Start Date: 3/29/02	Completion Date: 3/29/02	Boring No.: B-2
Drilling Contactor: Geologic Drill		Drilling Method: HSA		Sampling Method: SPT
Ground Surface Elevation: 100'		Hole Completion: ☐ Monitoring Well ☐ Piezometer ☒ Abandoned, sealed with bentonite		

Surface Conditions: Wood Chip Play Area

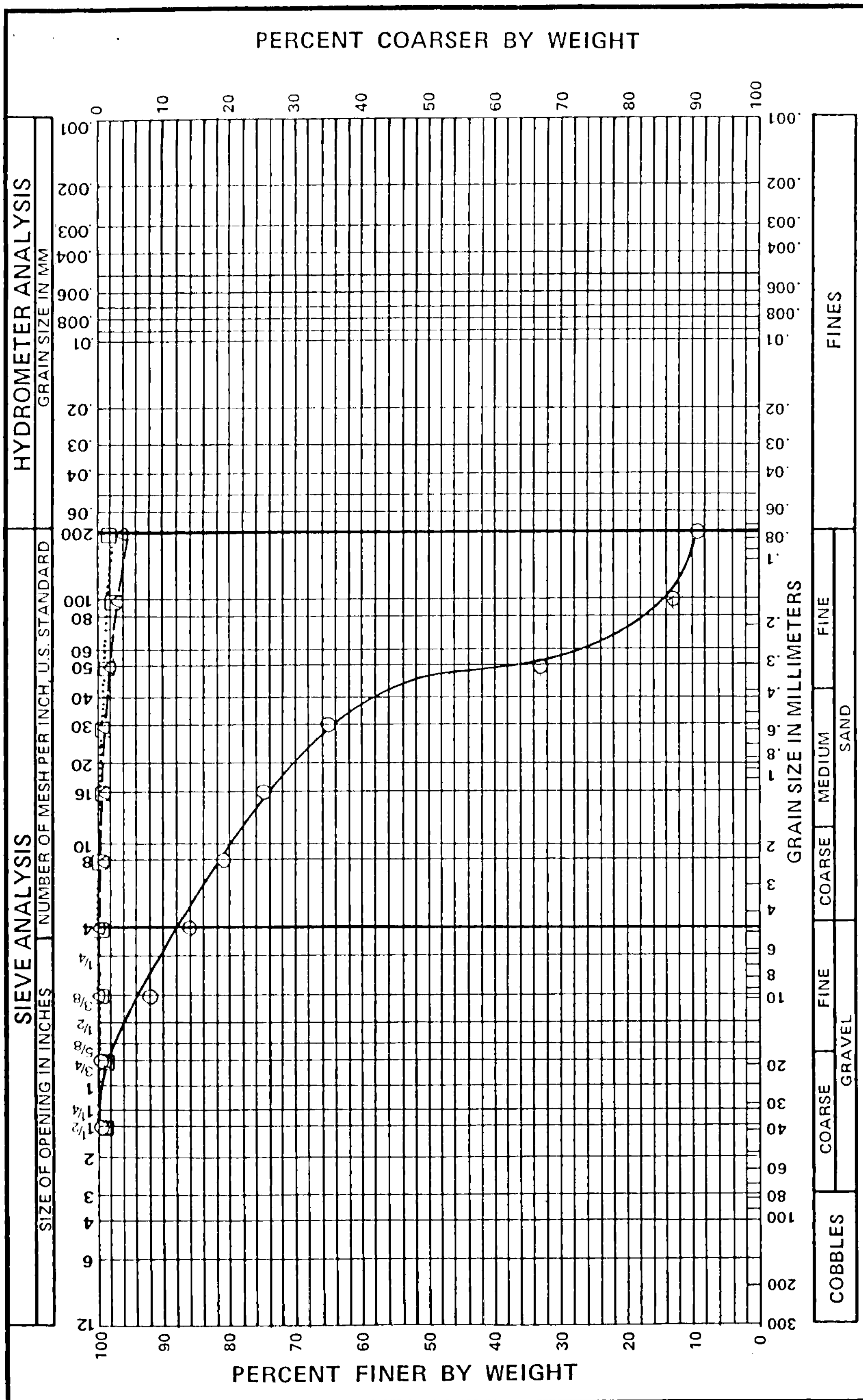
General Notes	W (%)	No. Blows Ft.	Depth Ft.	USCS Symbol	Description
			0	SM	Brown silty SAND, dense, moist
			1	ML	Brown SILT, medium dense, moist
			2		
	32.9	12	3		-iron oxide staining
			4		
	32.7	15	5		-becomes gray -98% fines
			6		
			7		
	30.4	22	8		-LL=41.2 PL=32.7 PI=8.5
			9		
	33.2	24	10		
			11		
			12		
			13		
			14		
	10.5	30	15	SP-SM	Brown poorly graded SAND with silt, dense, moist
			16		
					Boring terminated at 16.5 feet below existing grade. No groundwater encountered during excavation. Boring backfilled with bentonite and cuttings.

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

Boring Log
Post Residence: 10 Brook Bay Road
Mercer Island, Washington

Proj. No. 10057	Dwn. GLS	Date April 2002	Checked SSR	Date 4/30/02	Plate A3

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

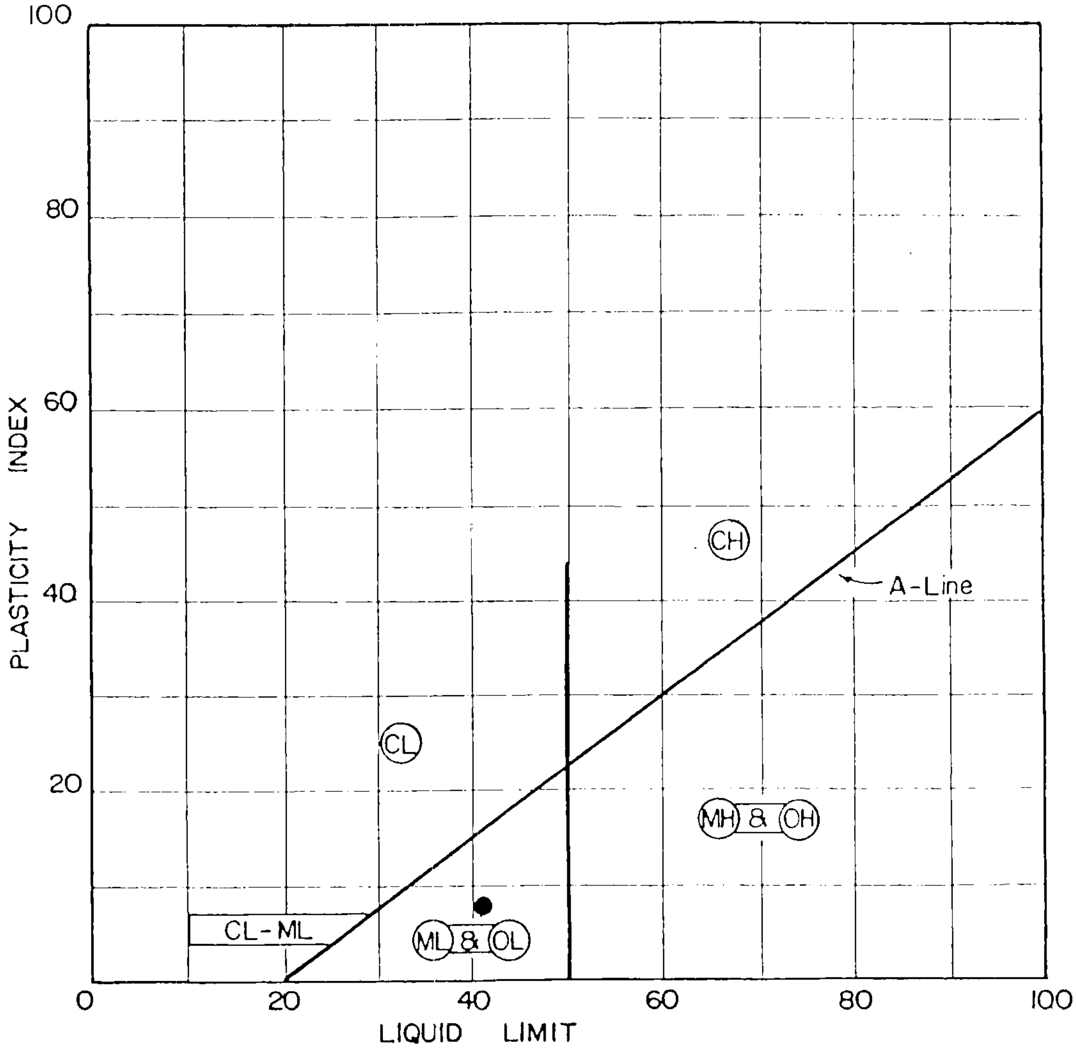


KEY	Boring or Test Pit No.	DEPTH (ft.)	USCS	DESCRIPTION	Moisture Content (%)	LL	PL
○———	B-1	5	SP-SM	Gray poorly graded Sand with SILT	8.1	--	--
△— — —	B-1	10	ML	Gray SILT	28.7	--	--
□········	B-2	5	ML	Gray SILT	32.7	--	--

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

GRAIN SIZE ANALYSES
Post Residence: 10 Brook Bay Road
Mercer Island, Washington

Proj. No. 10057	**Drwn.** GLS	**Date** Apr. 2002	**Checked** SSR	**Date** 4/30/02	**Plate** B1



Key	Boring/ Test Pit	Depth (ft.)	Soil Classification	USCS	L.L.	P.L.	P.I.	Natural Water Content
●	B-2	7.5	Brown SILT	ML	41.24	32.7	8.54	30.4

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

Atterberg Limits Test Data

Post residence: 10 Brook Bay road
Mercer Island, Washington

Proj. No. 10057	Date Apr. 2002	Plate B2