associated earth sciences incorporated

Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report

WANG RESIDENCE

Mercer Island, Washington

Prepared For:

MR. WENXUE WANG

Project No. 20190389E001
November 12, 2019

Associated Earth Sciences, Inc.
911 5th Avenue
Kirkland, WA 98033
P (425) 827 7701

associated
earth sciences
incorporated

November 12, 2019
Project No. 20190389E001

Mr. Wenxue Wang
6454 East Mercer Way
Mercer Island, Washington 98040

Subject: Subsurface Exploration, Geologic Hazard,
and Geotechnical Engineering Report
Wang Residence
6454 East Mercer Way
Mercer Island, Washington 98040

Dear Mr. Wang:

We are pleased to present the enclosed copy of the referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and geotechnical engineering studies, and offers recommendations for the design and development of the proposed project. Our recommendations are preliminary in that construction details have not been finalized at the time of this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions, or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Bruce L. Blyton, P.E.
Senior Principal Engineer

BLB/ms - 20190389E001-2

Kirkland Office | 911 Fifth Avenue | Kirkland, WA 98033 P | 425.827.7701
Mount Vernon Office | 508 S. Second Street, Suite 101 | Mount Vernon, WA 98273 P | 425.827.7701
Tacoma Office | 1552 Commerce Street, Suite 102 | Tacoma, WA 98402 P | 253.722.2992
www.aesgeo.com

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND GEOTECHNICAL ENGINEERING REPORT

WANG RESIDENCE

Mercer Island, Washington

Prepared for:
Mr. Wenxue Wang
6454 East Mercer Way
Mercer Island, Washington 98040

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue
Kirkland, Washington 98033
425-827-7701

November 12, 2019
Project No. 20190389E001

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of Associated Earth Sciences Inc.'s (AESI's) subsurface exploration, geologic hazards assessment, and geotechnical engineering study for the proposed residential development at the subject property. Our recommendations are preliminary in that construction details have not been finalized at the time of this report. If changes in the nature, design, or layout of the project are made, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the design of the project. Our study included reviewing selected geologic literature, drilling exploratory borings, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow groundwater. A geologic hazards assessment, preliminary infiltration feasibility assessment, and geotechnical engineering studies were completed to formulate our recommendations for suitable geologic hazard mitigation techniques, site preparation and grading, the types of suitable foundations and floors, allowable foundation soil bearing pressure, anticipated foundation and floor settlement, and drainage considerations. This report summarizes our current fieldwork and offers recommendations for development based on our present understanding of the project. We recommend that we be allowed to review any revisions to project plans and update the recommendations in this report as needed.

1.2 Authorization

Written authorization to proceed with this study was granted by Mr. Wenxue Wang. Our study was accomplished in general accordance with our proposal, dated September 23, 2019. This report has been prepared for the exclusive use of Mr. Wang and his agents, for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

The subject site is the existing single-family residential property located at 6454 East Mercer Way in Mercer Island, Washington (King County Parcel No. 3024059118). The subject parcel lies along the shore of Lake Washington. The parcel is roughly rectangular in shape and gently slopes down to the lake shoreline to the east. The existing residence is a single-story building with a paved parking loop in front and a concrete slab patio behind. A grass lawn backyard extends down to a sandy shoreline and a 100-foot-long dock that extends out into Lake Washington. We understand that the current plan includes the demolition of the existing residence and construction of a new single-family residence in the same location. The site lies within Seismic and Landslide Hazard Areas, as delineated in the City of Mercer Island *Geological Hazard Maps*.

3.0 SUBSURFACE EXPLORATION

The site exploration was conducted on October 14, 2019, and consisted of three exploration borings: EB-1 through EB-3. The various types of materials and sediments encountered in the explorations, as well as the depths where characteristics of these materials changed, are indicated on the exploration boring logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field. If changes occurred between sample intervals in our borings, they were interpreted. The locations of the exploration borings are shown on the “Site and Exploration Plan,” Figure 2.

The conclusions and recommendations presented in this report are based on the exploration borings completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, interpolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Borings

The borings were drilled using a hand-portable hollow-stem auger drill rig. During the drilling process, samples were generally obtained at 2.5 to 5-foot intervals. The borings were continuously observed and logged by a representative from our firm. The exploration logs presented in the Appendix are based on the field logs, drilling action, and study of the collected samples.

Disturbed but representative samples were obtained by using the Standard Penetration Test (SPT) procedure in accordance with *American Society for Testing and Materials* (ASTM) D-1586. This test and sampling method consists of driving a standard, 2-inch outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance (“N”) or blow count. If a total of 50 blows are recorded at or before the end of one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. These values are plotted on the attached boring logs.

The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in watertight containers. The samples were then transported to our laboratory for further visual classification and geotechnical laboratory testing, as necessary.

The various types of soil and groundwater elevations, as well as the depths where soil and groundwater characteristics changed, are indicated on the exploration boring logs presented in the Appendix of this report. Our explorations were approximately located by measuring from known site features.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of selected geologic literature. The general distribution of geologic units is shown on the exploration logs. The explorations generally encountered alluvial fan deposits or lake deposits over older glacially derived sediments.

4.1 Stratigraphy

The following sections present more detailed subsurface information organized from the youngest (shallowest) to the oldest (deepest) sediment types.

Sod/Topsoil/Landscaping Bark

A surficial soil layer of sod and topsoil approximately 3 to 4 inches thick was observed in EB-1 and EB-2 while approximately 4 inches of landscaping bark was encountered at EB-3. Due to their high organic content, sod, mulch, and topsoil materials are not considered suitable for foundation, roadway, or slab-on-grade floor support, or for use in a structural fill.

Existing Fill

Fill soils (those not naturally placed) were encountered in exploration boring EB-2 to a depth of approximately 5.5 feet below the ground surface. These soils generally consisted of very loose to loose, silty to very silty, fine to medium sand, ranging to sandy silt with occasional fine organics present. We interpret the existing fill as being placed during the construction of the existing residence. Due to their loose consistency, the existing fills are not considered suitable for foundation support or slab-on-grade floor support.

Lake Deposits

Sediments encountered below the surficial grass/topsoil horizon in boring EB-1 and below existing fill in EB-2, consisted of very loose/very soft to medium dense/stiff, gray, silty fine sand and sandy silt, which we interpreted as lake deposits. These lake-bottom sediments were deposited in the quiescent Lake Washington waters before the lake was lowered in 1916.

Alluvial Fan Deposits

Sediments interpreted as Holocene-age alluvial fan deposits were encountered below the topsoil in exploration boring EB-3 to the total depth explored. These deposits generally consisted of loose to medium dense, grayish brown, fine sand with variable silt content and trace gravel. We interpret that these Holocene-age alluvial fan deposits were likely deposited soon after deglaciation, from drainages on the eastern slope of Mercer Island, located west of the site.

Pre-Olympia (?) Coarse-Grained Glacial Deposits

Sediments encountered below the lake deposits in exploration borings EB-1 and EB-2 at a depth of approximately 24 feet consisted of medium dense to very dense, gray, silty, medium sand with trace gravel interpreted as pre-Olympia-age coarse-grained glacial deposits. These sediments were deposited by an active ice sheet prior to the Olympia nonglacial interval and were subsequently compacted by the weight of overriding glacial ice during subsequent glaciations.

4.2 Published Geologic Literature

We reviewed a published geologic map of the project area, *Geologic Map of Mercer Island, Washington* by Troost and Wisher (GeoMapNW 2009). The referenced map indicates that the site is expected to be underlain at shallow depths by lake deposits, with alluvial fan deposits and pre-Olympia-age deposits mapped nearby. Our on-site explorations and interpretations are generally consistent with the conditions depicted on the published map.

4.3 Hydrology

Groundwater seepage was observed at depths of 4 to 9.5 feet in the borings at the time of our exploration. The exploration logs in the Appendix depict specific instances and depths of groundwater seepage. We interpret the observed groundwater seepage as being hydraulically connected to Lake Washington. The seepage in EB-3 was higher in elevation, was contained within the fan deposits, and may include contributions from the upland slope. The duration and quantity of groundwater seepage can be expected to vary with changes in seasonal precipitation, on- and off-site land usage, and other factors.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow groundwater conditions as observed and discussed herein.

5.0 LANDSLIDE HAZARDS AND MITIGATIONS

Landslide Hazard Areas are identified by combinations of historic, topographic, geologic, and hydrologic characteristics and are defined by the City of Mercer Island *Unified Land Development Code* (ULDC) Chapter 19.16.010 as:

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 *Mercer Island Landslide Hazard Assessment* by Troost and Wisher (GeoMapNW 2009) maps the subject site as a suspect or known Landslide Hazard Area due to the area being potentially underlain by subaerial or subaqueous debris from past movements. While our site-specific exploration did not observe landslide or mass wasting deposits, they may be present nearby. The presence of these deposits would not change the recommendations contained in this report.

The site topography is gently sloping to flat. The site is approximately 350 feet from the nearest mapped known landslide area and approximately 150 feet from the nearest, upslope steep slope area. Based on the distance of significant slopes to the site, it is our opinion that the risk of damage to the proposed project by landsliding is low.

No additional landslide hazard mitigation is recommended for this project beyond application of the recommendations contained in this report. No detailed assessment of slope stability was prepared as part of this report and none is warranted, in our opinion.

6.0 SEISMIC HAZARDS AND MITIGATIONS

Earthquakes occur regularly in the Puget Lowland. Most of these events are small and are not felt by people. However, large earthquakes do occur, as evidenced by the 2001, 6.8-magnitude event; the 1965, 6.5-magnitude event; and the 1949, 7.2-magnitude event. The 1949 earthquake appears to have been the largest in this region during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 is likely within a given 20-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

6.1 Surficial Ground Rupture

The subject site is located within the mapped limits of the Seattle Fault Zone. Recent studies by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington,* Geology, v. 22, p.71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazard,* geological Society of America Bulletin, July 1999, v. 111, n. 7, p. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. The recognition of this fault is relatively new, and data pertaining to it are limited, with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island.

The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval, the potential for surficial ground rupture is considered to be low during the expected life of the structure, and no mitigation efforts beyond complying with the current 2015 *International Building Code* (IBC) are recommended.

6.2 Seismically Induced Landslides

Due to the relatively flat site topography and distance to significant slopes, it is our opinion that the risk of damage to the proposed project by seismically induced landsliding is low. Provided that the recommendations presented in this report are properly followed, no additional landslide hazard mitigation is recommended for this project.

6.3 Liquefaction

Liquefaction is a process through which unconsolidated soil loses strength as a result of vibrations, such as those which occur during a seismic event. During normal conditions, the weight of the soil is supported by both grain-to-grain contacts and by the fluid pressure within the pore spaces of the soil below the water table. Extreme vibratory shaking can disrupt the grain-to-grain contact, increase the pore pressure, and result in a temporary decrease in soil shear strength. The soil is said to be liquefied when nearly all the weight of the soil is supported by pore pressure alone. Liquefaction can result in deformation of the sediment and settlement of overlying structures. Areas most susceptible to liquefaction include those areas underlain by non-cohesive silt and sand with low relative densities, accompanied by a shallow water table.

Our explorations throughout the site encountered loose, granular sediments that were below the groundwater table (saturated) that, based on the characteristics, are considered susceptible to liquefaction.

In order to mitigate liquefaction hazard to the proposed new residence, we recommend foundations extend through potentially liquefiable soils and penetrate into the underlying glacially consolidated native soils. Buried utilities should be constructed with flexible joints where they enter the new residence to limit the risk of water, gas, and other potentially hazardous pipe ruptures. In order to mitigate the potential effects of liquefaction-induced settlement, structural elements, such as pavement or other hardscapes, that are not supported by deep foundations should be constructed of flexible materials such as asphalt materials or unbonded pavers. Preliminary design recommendations concerning use of deep foundations are presented in the "Foundations" section of this report.

It is our opinion that the risk of damage to the proposed project by liquefaction is low, provided the recommendations contained in this report are properly followed during design and construction. A quantitative liquefaction analysis was not completed as part of this study.

6.4 Ground Motion/Seismic Site Class (2015 *International Building Code*)

Structural design of the new residence should follow 2015 IBC standards. The site is underlain in areas by soils that are considered vulnerable to potential failure under seismic loading (liquefiable) and would ordinarily be classified as Site Class "F," as defined in Table 20.3-1 of *American Society of Civil Engineers* (ASCE) *7 - Minimum Design Loads for Buildings and Other Structures*. However, the proposed project consists of a conventional, single-family residence that we anticipate will have a fundamental period of vibration less than 0.5 seconds. Therefore, we recommend that the project be designed in accordance with Site Class "D" and as permitted by the exception defined by ASCE 7 Chapter 20.3.1 for structures having fundamental periods of vibration equal to or less than 0.5 seconds.

7.0 EROSION HAZARD AND MITIGATION

Based on the relatively flat site topography and presence of fine-grained soils, the erosion hazard at the site is considered low to moderate and a properly developed, constructed, and maintained erosion control plan consistent with local standards and best management practices is recommended for this project. Maintaining cover measures atop disturbed ground provides significant reduction to the potential generation of turbid runoff and sediment transport. During the local wet season (October 1st through March 31st), exposed soil should not remain uncovered for more than 2 days, unless it is actively being worked. Ground-cover measures can include erosion control matting, plastic sheeting, straw mulch, crushed rock, recycled concrete, or mature hydroseed.

To mitigate the erosion hazards and potential for off-site sediment transport, we recommend the following:

1. Earthwork and foundation construction should be timed to be completed during seasonally drier weather (typically April through October).

2. Site access and construction staging areas should be surfaced with crushed quarry rock to reduce sediment track-out as recommended in Section 10.3 of this report.

3. All erosion and sediment control measures for the work area should be installed prior to any ground disturbing activity.

4. During the wetter months of the year, or when large storm events are predicted during the summer months, the work area should be covered/stabilized so that if showers occur, the work area can receive the rainfall without excessive erosion or sediment transport.

5. All disturbed areas should be revegetated as soon as possible. If it is outside of the growing season, the disturbed areas should be covered with mulch.

6. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not limited to, covering with plastic sheeting, the use of low stockpiles in flat areas, or the use of straw bales/silt fences around pile perimeters.

7. Runoff from work areas should be collected and treated as necessary to reduce sediment load and turbidity to the required levels prior to discharging to receiving waters or drain systems.

Additional environmental protections that are beyond the scope of this study may be required based on the proximity of the work area to Lake Washington.

8.0 STATEMENT OF RISK

For Section 17.07.060(D) of the Mercer Island ULDC, the City of Mercer Island requires a statement of risk by the geotechnical engineer. It is the opinion of AESI that the development practices proposed for the alteration would render the proposed addition as safe as if it were not located in a geologic hazard area provided the recommendations in this report are followed.

III. PRELIMINARY DESIGN RECOMMENDATIONS

9.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the site is suitable for the proposed project provided the risks discussed are accepted and the recommendations contained herein are properly followed. The foundation-bearing stratum for the building is deep and was observed at our exploration locations approximately 25 feet below the existing ground surface. Due to access constraints and the depth of the bearing soils, a driven pipe-pile foundation is recommended for the planned development.

Groundwater was measured at approximately 4 to 9.5 feet below the ground surface at the locations of our exploration borings, and we anticipate groundwater levels to be associated with the elevation of Lake Washington. Groundwater on the western portion of the site was somewhat elevated relative to lake level. Therefore, for any excavations extending near or below lake level, such as may be needed for deep utility trenches, the contractor should be prepared to dewater the excavations. Also, walls or structures below the groundwater table must be designed for combined soil and hydrostatic pressures and for buoyant forces.

No suitable receptor for stormwater infiltration was observed and therefore infiltration is not recommended for this project.

The following report sections provide recommendations regarding site preparation, grading, foundations, retaining walls, floor support, and drainage, including temporary dewatering.

10.0 SITE PREPARATION

Site preparation of building and paving areas should include removal of all existing buildings, grass, trees, brush, debris, and any other deleterious materials. Buried utilities should be removed from foundation areas and should be abandoned in place or removed from below planned new paving. Any depressions below planned final grades should be backfilled with structural fill, as discussed under the “Structural Fill” section of this report.

Existing topsoil should be stripped from structural areas. The observed in-place depth of sod and topsoil at the exploration locations is presented on the exploration logs in the Appendix. After stripping, remaining roots and stumps should be removed from structural areas. All native soils or existing fill soils to remain that are disturbed by stripping and grubbing operations should be recompacted as described below for structural fill.

Where existing fill is present at the proposed subgrade of lightly loaded pavement or slabs-on-grade, the topmost 12 inches of subgrade should be recompacted to a firm and unyielding condition. If recompaction is not possible, the unsuitable soils should be removed and replaced with crushed rock or structural fill.

10.1 Temporary Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, temporary, unsupported cut slopes can be planned at 1.5H:1V (Horizontal: Vertical) in unsaturated existing fill, alluvial fan deposits, and lake deposits.

These slope angles are for areas where groundwater seepage is not present at the faces of the slopes. If ground or surface water is present when the temporary excavation slopes are exposed, flatter slope angles or temporary dewatering may be required. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times.

10.2 Site Disturbance

Site soils contain a significant portion of fine-grained material, which makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed, and the area brought to grade with structural fill. Due to the expected wet subgrade conditions, we recommend that the building footprint area be graded smooth and sloped toward the lake. To provide access for pole installation equipment, the building footprint should then be blanketed with a minimum of 6 inches of clean, crushed, 2-inch rock (railroad ballast) prior to start of other site work. AESI can provide field design recommendations for these areas, if needed.

10.3 Wet Season Construction

If wet season construction is expected, crushed rock fill could be used to provide construction staging areas. The stripped subgrade should be observed by the geotechnical engineer, and should then be covered with a geotextile fabric, such as Mirafi 500X or equivalent. Once the fabric is placed, we recommend using a crushed rock fill layer at least 10 inches thick in areas where construction traffic is expected.

11.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades or to backfill around foundations and utilities. All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials, as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used. For backfill of buried utilities in the right-of-way, the backfill should be placed and compacted in accordance with the City of Mercer Island codes and standards.

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the surface of the exposed ground should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain, and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer/engineering geologist, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of ASTM D-1557. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of the perimeter footings or roadway edges before sloping down at a maximum angle of 2H:1V.

The contractor should note that any proposed fill soils should be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material at least 72 hours in advance to perform a Proctor test and determine its field compaction standard.

Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather and dry subgrade conditions. Construction equipment traversing the site when the soils are wet can cause considerable disturbance. Alternatives to drying site soils include using imported granular soils suitable for use in structural fill, or possibly treating wet soils with Portland cement.

If fill is placed during wet weather or if proper compaction cannot be obtained, a select, import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil, with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction, and at least 25 percent retained on the No. 4 sieve.

A representative from our firm should inspect the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid the owner in developing a suitable monitoring and testing frequency.

12.0 FOUNDATIONS

Due to the presence of liquefiable soils under the site, we recommend the use of steel pipe piles for the planned residence and any appurtenant structures. Recommendations for pipe pile foundations are included in this section. Pipe piles should extend through the liquefiable sediments and penetrate the underlying glacially consolidated pre-Olympia glacially derived sediments. For preliminary estimating purposes, pile lengths on the order of 30 to 40 feet should be assumed. Actual pile lengths may differ significantly from the estimated range depending on local variations in soil conditions, pile size, and driving equipment used. Pile lengths can best be determined by driving a series of test piles.

12.1 Pipe Pile Foundations

Pipe piles for new buildings should consist of 3-, 4-, or 6-inch-diameter pipe, depending on the required structural loads and accessibility for piledriving equipment. The piles should be galvanized steel pipe, driven with a suitable hammer to the refusal criteria shown in Table 1. The following table provides required minimum hammer weights, refusal criteria, and allowable loads for pipe piles.

Table 1
Pipe Pile Design Parameters

Pipe Diameter (inches)	Wall Thickness	Minimum Hammer Size (pounds)	Refusal Criterion* (seconds)	Allowable Axial Compressive Load** (kips)
3	Schedule 40	400	25	10
4	Schedule 40	650	20	20
6	Schedule 40	1,500	15	30

* Refusal is defined as less than 1 inch of penetration in “X” seconds under constant driving.

** Allowable load to be verified by load tests in accordance with *American Society for Testing and Materials* (ASTM) D-1143 “quick load test.”

Anticipated settlement of pile-supported foundations should be less than ½ inch. Pile installation must be observed by AESI to verify that the design bearing capacity of the piles has been attained and that construction conforms to the recommendations contained herein. The City of Mercer Island may also require such inspections.

Lateral resistance can be derived from passive soil resistance against the buried portion of the foundation (i.e., the grade beam) or from the installation of batter piles. A passive equivalent fluid of 200 pounds per cubic foot (pcf) can be used to account for lateral resistance. Lateral resistance for batter piles should be taken as the horizontal component of the axial pile load. Batter piles are typically installed at 1H:4V inclination.

Pile Inspections

The actual total length of each pile may be adjusted in the field based on required capacity and conditions encountered during driving. Since completion of the pile takes place below ground, the judgment and experience of the geotechnical engineer or their field representative must be used as a basis for determining the required penetration and acceptability of each pile. Consequently, use of the presented pile capacities in the design requires that the installation of all piles be observed by a qualified geotechnical engineer or engineering geologist from our firm, who can interpret and collect the installation data and examine the contractor’s operations. AESI, acting as the owner’s field representative, would determine the required lengths of the piles and keep records of pertinent installation data. A final summary report would then be distributed following completion of pile installation.

Load testing should be performed to verify that the design-bearing capacity of the piles has been attained. Because of the variation in the soil types and their densities, we recommend that AESI monitor the load-testing program. A common pile load-testing program would consist of one or more 200-percent verification tests of the design bearing capacity of the pile in the soil. Verification test piles are usually loaded in 25-percent increments that are held for 2 minutes up to the final load of 200-percent design load. The 200-percent load is commonly held for 20 minutes and creep-measured. The load is then reduced by 25-percent increments to evaluate the effect of elasticity in the pile to overall displacement.

13.0 LATERAL WALL PRESSURES

All backfill behind foundation or basement walls or around foundation units should be placed per our recommendations for structural fill and as described in this section of the report. Horizontally backfilled walls that are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid pressure equal to 35 pcf. Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid pressure of 50 pcf. Walls with sloping backfill up to a maximum gradient of 2H:1V should be designed using an equivalent fluid pressure of 55 pcf for yielding conditions or 75 pcf for fully restrained conditions. If parking areas are adjacent to walls, a surcharge equivalent to 2 feet of soil should be added to the wall height in determining lateral design forces.

As required by the 2015 IBC, retaining wall design should include a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. Considering the site soils and the recommended wall backfill materials, we recommend a seismic surcharge pressure of 5H and 10H psf, where H is the wall height in feet for the “active” and “at-rest” loading conditions, respectively. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the walls.

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of excavated on-site soils or imported structural fill, compacted to 90 percent of ASTM D-1557. A higher degree of compaction is not recommended, as this will increase the pressure acting on the walls. A lower compaction may result in settlement of the slab-on-grade or other structures supported above the walls. Thus, the compaction level is critical and must be tested by our firm during placement. Surcharges from adjacent footings or heavy construction equipment must be added to the above values. Perimeter footing drains should be provided for all retaining walls, as discussed under the “Drainage Considerations” section of this report.

It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum 1-foot-wide blanket drain to within 1 foot of finish grade for the full wall height using imported, washed gravel against the walls. If situations exist where a footing drain is not feasible for a foundation wall or retaining wall, the wall should be designed for saturated lateral earth pressures and a hydrostatic surcharge. We should be allowed to offer situation-specific recommendations if this situation arises. The use of drainage improvements as recommended herein does not alleviate the need for waterproofing where finished spaces are planned on the interior side of basement walls. Backfilled walls with finished interior space should be waterproofed in accordance with recommendations of the building designer.

13.1 Passive Resistance and Friction Factors

Retaining wall grade beams cast directly against undisturbed dense soils in a trench may be designed for passive resistance against lateral translation using an allowable equivalent fluid equal to 200 pcf. The passive equivalent fluid pressure diagram begins at the top of the grade beam; however, total lateral resistance should be summed only over the depth of the actual key. Since the structure will be pile-supported, we do not recommend using base friction for resistance to lateral loads.

13.2 Buoyant/Uplift Forces

Although the current groundwater levels were approximately 4 to 9.5 feet below the ground surface at the time our explorations were performed, the groundwater levels in this area can be higher during other times in the year, particularly during the summer season when the lake is typically kept 1½ to 2 feet higher than during the winter. Therefore, if structures are planned to extend below the summertime lake level, they should be designed to resist buoyant forces. Figure 3 provides a diagram and worksheet for the calculation of buoyant forces and resistance. For preliminary planning purposes, we recommend that the approach assumes a groundwater table at the elevation of the permanent exterior wall drainage system for the incorporation of buoyant forces in design. Typical design features to account for buoyant forces are thickened slabs, base slab extensions, increased footing widths, increased overlying soil weight, or an engineered anchoring system, such as helical anchors.

As shown on Figure 3, the dry unit weight of the soils present in the upper portion of the existing site stratigraphy may be assumed to be 100 pcf. The buoyant unit weight of the soil in the upper portion of the existing site stratigraphy may be assumed to be 35 pcf. If a base slab extension is chosen to resist uplift, we recommend replacing the existing soil above this floor extension with a gravelly sand. The buoyant weight of a gravelly sand may be assumed to be 65 pcf for design purposes.

14.0 FLOOR SUPPORT

Due to the loose nature of the subgrade soils, we recommend that structural support be provided for settlement-sensitive, slab-on-grade floors. Slab-on-grade floors should be cast atop a minimum of 4 inches of washed pea gravel or washed crushed “chip” rock with less than 3 percent passing the U.S. No. 200 sieve to act as a capillary break. The floors should also be protected from dampness by covering the capillary break layer with an impervious moisture barrier at least 10 mils in thickness.

An underslab drainage system is recommended to provide positive drainage beneath the floor slabs. For preliminary planning, an underslab system should consist of a series of 4-inch diameter polyvinyl chloride (PVC), perforated drain lines placed approximately 20 feet on-center. The drain lines should have an invert located a minimum of 12 inches below the slab base and be connected to discharge into perimeter footing drains. The drain trenches should be filled with pea gravel, which communicates with the capillary break material. Underslab drainage systems that cannot drain via gravity flow should discharge to a sump pump system which includes a battery-powered backup system and maintenance alarms.

15.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter foundation walls should be provided with a drain at the base of the footing elevation. Drains should consist of rigid, perforated, PVC pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set at or slightly below the bottom of the footing grade beam, and the drains should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel blanket that extends to within 1 foot of the surface and is continuous with the foundation drain. Roof and surface runoff should not discharge into the foundation drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage. All collected runoff must be tightlined to a City-approved location.

Where gravity drainage is not possible, the portion of the structure below the groundwater (lake level) must be designed for combined soil and hydrostatic and buoyant forces, as described in the "Buoyant/Uplift Forces" section of this report. Also, we anticipate that moisture will likely wick through the submerged portion of the foundation elements, either through joints in the footings/walls or due to the inherent porosity of concrete. Therefore, we recommend that suitable moisture protection be placed between the concrete foundation shell of any buried portion of the structure and proposed interior floor slabs and finishes.

15.1 Preliminary Temporary Dewatering Recommendations

Groundwater was measured at roughly 4 to 9.5 feet below the ground surface at our exploration locations. Prior to site work and construction, the contractor should be prepared to provide excavation drainage/dewatering and subgrade protection, as necessary, for deep utility trenches and excavations. Water levels inside the excavation should be drawn down a minimum of 2 to 3 feet below the base of the excavation in order to avoid heaving conditions during construction.

15.2 Preliminary Infiltration Feasibility

The feasibility of stormwater infiltration depends upon the presence of a suitable receptor native soil of sufficient thickness, extent, permeability, and vertical separation from groundwater. Site soils were observed to consist of alluvial fan deposits and lake sediments that are saturated at relatively shallow depths below the ground surface. Based on these characteristics, infiltration into site soils is not considered feasible.

16.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to construction. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations for residences and of retaining walls depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of the current scope of work. If these services are desired, please let us know, and we will prepare a cost proposal.

We have enjoyed working with you on this study and are confident these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

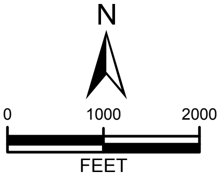
Joshua S. P. Greer, G.I.T.
Staff Geologist

Jeffrey P. Laub, L.G., L.E.G.
Senior Engineering Geologist

BRUCE L. BLYTON
26830
11/12/19

Bruce L. Blyton, P.E.
Senior Principal Engineer

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Site and Exploration Plan
	Figure 3:	Uplift Resistance Diagram Worksheet
	Appendix:	Exploration Logs



DATA SOURCES / REFERENCES:
USGS: 7.5' SERIES TOPOGRAPHIC MAPS, ESRI/I-CUBED/NGS 2013
KING CO: STREETS, CITY LIMITS 1/19, PARCELS 4/19
LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

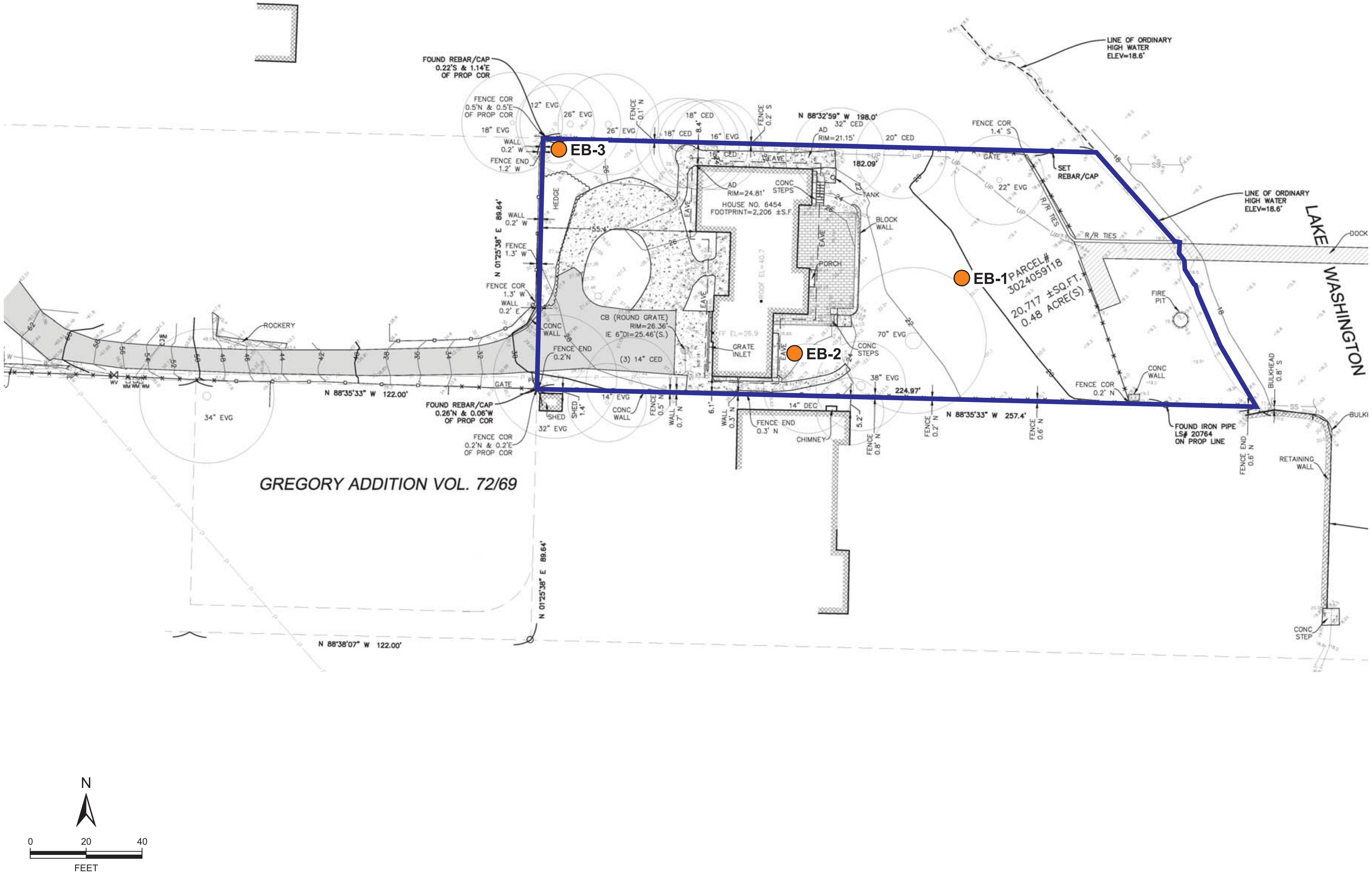
associated
earth sciences
incorporated

VICINITY MAP

WANG RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
190389E001	10/19	1

Document Path: G:\GIS_Projects\aaY2019\190389 Wang Property\aprx_mxd\190389E001 F1 VM_WangRes.aprx



LEGEND:

● **EB** EXPLORATION BORING

▬ SITE BOUNDARY

CONTOUR INTERVAL = 2'

NOTE: LOCATION AND DISTANCES SHOWN ARE APPROXIMATE.

NOTES:

1. BASE MAP REFERENCE: TERRANE, TOPOGRAPHIC AND BOUNDARY SURVEY, WANG RESIDENCE, 5/3/18.

BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION.

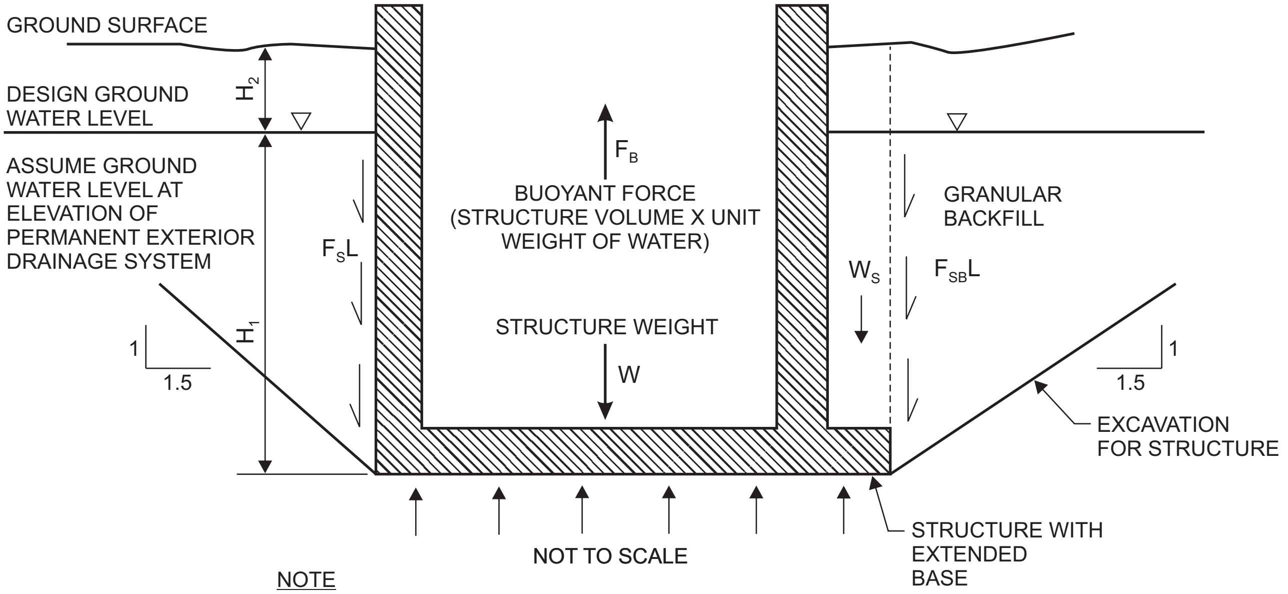
associated
earth sciences
incorporated

SITE AND EXPLORATION PLAN

WANG RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
190389E001	10/19	2

190389 Wang \ 180389E001 F2 S-E Plan.cdr



NOTE

UPLIFT ON STRUCTURE BOTTOM COULD RESULT IN HIGH MOMENTS IN BOTTOM SLAB.

$$\text{FACTOR OF SAFETY} = \frac{W + F_S L}{F_B}$$

$$\text{FACTOR OF SAFETY WITH EXTENDED BASE} = \frac{W + W_S + F_{SB} L}{F_B}$$

W = STRUCTURE WEIGHT IN KIPS

W_S = WEIGHT OF SOIL ABOVE STRUCTURE BASE IN KIPS

F_S = SHEARING RESISTANCE OF SOIL TO STRUCTURE WALLS

$= 0.011\ H_2^2 + 0.005\ H_1^2 + 0.021\ H_1\ H_2$ (IN KIPS PER FOOT OF STRUCTURE WALL)

$$\left(\frac{\gamma (H)^2}{2} \text{TAN}\,\delta\right) K_O + \left(\frac{\gamma - \gamma_{WATER}\ (H)^2}{2}\ \text{TAN}\,\delta\right) K_O + \left(\gamma\ H_2 H_1\ \text{TAN}\,\delta\right)$$

F_{SB} = SHEARING RESISTANCE OF SOIL $= 0.016\ H_2^2 + 0.007 H_1^2 + 0.032\ H_1\ H_2$

$$K_O \left(\frac{\gamma (H_2)^2 \text{TAN}\,\phi}{2}\right) + K_O\left(\frac{\gamma_{BUOYANT}\ (H_1)^2 \text{TAN}\,\phi}{2}\right) + K_O\left(\gamma\ H_2 H_1\ \text{TAN}\,\phi\right)$$

ASSUME:

γ = 0.100 KCF

ϕ = 30°

δ = 21°

K_O = 0.5

L = PERIMETER LENGTH AROUND BASE OF STRUCTURE IN FEET

associated earth sciences incorporated

UPLIFT RESISTANCE DIAGRAM WORKSHEET

WANG RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
190389E001	11/19	3

190389 Wang \ 190389E001 F3 UP-RES.cdr

APPENDIX

Exploration Logs

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	▨	**GW**	Well-graded gravel and gravel with sand, little to no fines
			▨	**GP**	Poorly-graded gravel and gravel with sand, little to no fines
		≥12% Fines[5]	▨	**GM**	Silty gravel and silty gravel with sand
			▨	**GC**	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	▨	**SW**	Well-graded sand and sand with gravel, little to no fines
			▨	**SP**	Poorly-graded sand and sand with gravel, little to no fines
		≥12% Fines[5]	▨	**SM**	Silty sand and silty sand with gravel
			▨	**SC**	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		▨	**ML**	Silt, sandy silt, gravelly silt, silt with sand or gravel
			▨	**CL**	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			▨	**OL**	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		▨	**MH**	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			▨	**CH**	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			▨	**OH**	Organic clay or silt of medium to high plasticity
Highly Organic Soils			▨	**PT**	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50
	Consistency	**SPT[2] blows/foot**
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

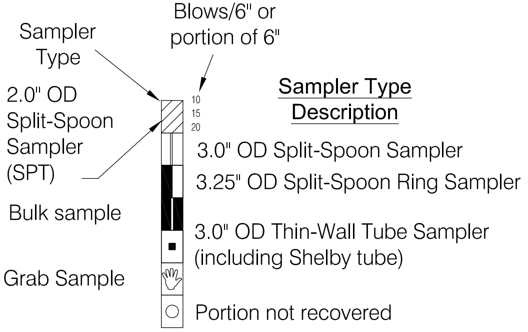
[3] Estimated Percentage

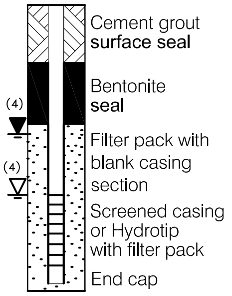
Component	Percentage by Weight
Trace	<5
Some	5 to <12
Modifier (silty, sandy, gravelly)	12 to <30
Very *modifier* (silty, sandy, gravelly)	30 to <50

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

Symbols





[1] Percentage by dry weight
[2] (SPT) Standard Penetration Test (ASTM D-1586)
[3] In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
[4] Depth of ground water
▼ ATD = At time of drilling
▽ Static water level (date)
[5] Combined USCS symbols used for fines between 5% and 12%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

blocks \ dwg \ log_key.dwg LAYOUT: Layout 4 -2014 Qty Chng

associated
earth sciences
incorporated

EXPLORATION LOG KEY

FIGURE A1

associated earth sciences incorporated

Exploration Boring

Project Number	Exploration Number	Sheet
190389E001	EB-1	1 of 1

Project Name Wang Residence
Location Mercer Island, WA
Driller/Equipment CN Drilling / Acker
Hammer Weight/Drop 140# / 30

Ground Surface Elevation (ft) ~20
Datum NAVD 88
Date Start/Finish 10/14/19,10/14/19
Hole Diameter (in) 6.25

Depth (ft)	ST	Samples	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot	Other Tests
			Sod / Topsoil - 6 inches					
	◫	S-1	**Lake Deposits** Moist to very moist, dark gray with slight orange staining, SILT, some to trace fine sand; frequent small organics (ML).			1 0	▲1	
	◫	S-2	Very moist to moist, gray to dark brown, SILT, some fine sand; abundant fine organics (ML).		▼	1/18"	▲1/18"	
5	◫	S-3	Very moist to wet, dark gray, SILT; abundant fine organics; organic rich (ML).		▼	1/18"	▲1/18"	
	◫	S-4	Wet, gray, very silty, fine SAND ranging to sandy, SILT, some small organics (SM-ML).			1 2 4	▲6	
10	◫	S-5	Very moist to wet, dark gray, silty, fine to medium SAND; one brown silty interbed (4 inches thick); frequent fine organics (SM).			1 4 10	▲14	
15	◫	S-6	Very moist to wet, dark gray, very silty, fine SAND ranging to sandy, SILT, trace gravel, trace organics (SM-ML).			1 1 4	▲5	
20	◫	S-7	Wet, dark gray, very silty, fine to medium SAND (SM).			2 6 5	▲11	
25	☐	S-8	**Pre-Olympia (?) Coarse Grained Glacial Deposits** No recovery; driller notes hitting gravel; pushing rock; blowcounts possibly overstated.			5 20 33	▲53	
	◫	S-9	Wet, dark gray, silty, SAND, trace gravel; poor recovery; material not interpreted to be representative (SM).			50/5.5"	▲50/5.5"	
30			Bottom of exploration boring at 29 feet Groundwater encountered at 4 and 7 feet at time of drilling.					

Sampler Type (ST):

- ◫ 2" OD Split Spoon Sampler (SPT)
- ◫ 3" OD Split Spoon Sampler (D & M)
- ✋ Grab Sample
- ☐ No Recovery
- ◧ Ring Sample
- ▨ Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: JG
Approved by: JHS

associated earth sciences incorporated

Exploration Boring

Project Number	Exploration Number	Sheet
190389E001	EB-2	1 of 1

Project Name Wang Residence
Location Mercer Island, WA
Driller/Equipment CN Drilling / Acker
Hammer Weight/Drop 140# / 30
Ground Surface Elevation (ft) ~26
Datum NAVD 88
Date Start/Finish 10/14/19,10/14/19
Hole Diameter (in) 6.25

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
	S-1	**Topsoil** Moist, grayish brown, silty, fine SAND, trace gravel; frequent rootlets and organics (SM).		1 2 2	4
	S-2	**Fill** Slightly moist to moist, brownish gray, silty, fine SAND, trace small organics (SM).		3 3 4	7
5	S-3	Upper ~9 inches: moist, brownish gray, very silty, fine SAND; frequent small organics; thinly bedded; cuttings are loose (SM). **Lake Deposits** Lower ~9 inches: very moist, dark gray to dark brown, silty, fine SAND ranging to sandy, SILT (SM-ML).		1 0 1	1
	S-4	Moist to very moist, gray to dark brownish gray, very silty, fine SAND; zones of frequent small organics; thinly bedded (SM).	▼	1 2 4	6
10	S-5	Very moist to wet, dark gray, very silty, fine SAND, trace organics; thinly bedded; cuttings are wet and flowing (SM).		2 3 7	10
15	S-6	Very moist to wet, dark gray, very silty, fine SAND ranging to sandy, SILT, trace gravel, trace fine organics (SM-ML).		4 5 5	10
20	S-7	Very moist to wet, dark gray to dark brown with zones of orange oxidation, very silty, fine SAND, trace gravel, trace fine organics (SM).		3 4 7	11
25	S-8	**Pre-Olympia (?) Coarse Grained Glacial Deposits** Driller notes heaving sands at 24 feet. Very moist to wet, dark gray, fine to medium SAND, some silt to silty, trace gravel; faintly bedded (SM).		7 13 20	33
		Bottom of exploration boring at 26.5 feet Groundwater encountered at 9.5 feet.			
30					

Sampler Type (ST):

2" OD Split Spoon Sampler (SPT)	☐ No Recovery	M - Moisture
3" OD Split Spoon Sampler (D & M)	Ring Sample	▽ Water Level ()
Grab Sample	Shelby Tube Sample	▼ Water Level at time of drilling (ATD)

Logged by: JG
Approved by: JHS

associated earth sciences incorporated

Exploration Boring

Project Number	Exploration Number	Sheet
190389E001	EB-3	1 of 1

Project Name: Wang Residence
Location: Mercer Island, WA
Driller/Equipment: CN Drilling / Acker
Hammer Weight/Drop: 140# / 30

Ground Surface Elevation (ft): ~27
Datum: NAVD 88
Date Start/Finish: 10/14/19,10/14/19
Hole Diameter (in): 6.25

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot	Other Tests
		S-1		**Landscaping Bark / Topsoil - 4 inches** **Fill** Slighlty moist, brown, silty, fine SAND; frequent rootlets and organic debris (SM).			1 2 3	6	
		S-2		**Alluvial Fan Deposits** Moist, brownish gray with orange staining, silty, fine SAND, trace gravel (SM). Difficult drilling, driller notes large rock; adding water. Drill refusal at 4.5 feet due to obstruction. Moved 5 feet west towards property corner.			7 16 7	23	
5		S-3		Moist, light brown with slight orange staining, SILT, some fine sand, trace fine organics (ML).			2 3 4	7	
		S-4		Driller notes light groundwater. Very moist to wet, gray with orange staining, silty, fine SAND ranging to sandy, SILT; occasional fine organics (SM-ML).		▼	2 2 4	6	
10		S-5		Wet, grayish brown, silty, fine SAND, trace gravel; unsorted (SM). Cuttings are a grayish brown slurry.			9 9 6	15	
15		S-6		Very moist to wet, grayish brown, fine SAND, some silt to silty, trace gravel (SM).			4 6 7	13	
20		S-7		Wet, grayish brown, silty, fine SAND, trace gravel; faint stratification (SM).			8 10 16	26	
				Bottom of exploration boring at 21.5 feet Groundwater encountered at 7.5 feet.					
25									
30									

Blows/Foot scale: 10 20 30 40

Sampler Type (ST):

◫ 2" OD Split Spoon Sampler (SPT)
◫ 3" OD Split Spoon Sampler (D & M)
☐ Grab Sample
☐ No Recovery
◧ Ring Sample
▱ Shelby Tube Sample
M - Moisture
▽ Water Level ()
▼ Water Level at time of drilling (ATD)

Logged by: JG
Approved by: JHS