associated earth sciences incorporated

Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report

CHASE RESIDENCE

Mercer Island, Washington

Prepared For:

BRAD AND JUDY CHASE

Project No. 20220141E001
August 16, 2022

Associated Earth Sciences, Inc.

www.aesgeo.com

Kirkland | Mount Vernon | Tacoma

associated
earth sciences
incorporated

August 16, 2022
Project No. 20220141E001

Brad and Judy Chase
4467 Forest Avenue SE
Mercer Island, Washington 98040

Subject: Subsurface Exploration, Geologic Hazard, and
Geotechnical Engineering Report
Chase Residence
4525 Forest Avenue SE
Mercer Island, Washington

Dear Mr. and Mrs. Chase

Associated Earth Sciences, Inc. (AESI) is pleased to present the enclosed copies of the above-referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and geotechnical engineering studies and offers geotechnical recommendations for the design and development of the proposed project.

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/jh – 20220141E001-003

Kirkland | Tacoma | Mount Vernon
425-827-7701 | www.aesgeo.com

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND GEOTECHNICAL ENGINEERING REPORT

CHASE RESIDENCE

Mercer Island, Washington

Prepared for:
Brad and Judy Chase
4467 Forest Avenue SE
Mercer Island, Washington 98040

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

August 16, 2022
Project No. 20220141E001

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and geotechnical engineering study for the proposed project. The location of the site is shown on the "Vicinity Map," Figure 1, and the approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2. In the event that any changes in the nature, design, or location of the proposed improvements are planned, 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 preliminary design and development of the subject project. Our study included reviewing available geologic literature, advancing three exploration soil borings, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and groundwater conditions. Geotechnical engineering studies were completed to assess geologic hazards on the subject site and to formulate geotechnical recommendations for landslide hazard mitigation, site preparation, grading, building foundations and floor slabs, allowable foundation soil bearing pressures, anticipated foundation settlement, and drainage considerations. This report summarizes our fieldwork and offers preliminary recommendations based on our present understanding of the project. We recommend that we be allowed to review the recommendations presented in this report and revise them, if needed, when the project design has been finalized.

1.2 Authorization

Our study was accomplished in general accordance with our scope of work and cost proposal, dated April 20, 2022. This report has been prepared for the exclusive use of Mr. and Mrs. Chase and their 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 4525 Forest Avenue SE in Mercer Island, Washington (King County Parcel No. 7700100205). The 1.37 acre parcel slopes down to the west from Forest Avenue SE to the shore of Lake Washington, with an upper, eastern level area and a lower, western area along Lake Washington. The existing single-family residence, originally constructed in 1934, is located at the lower, western portion of the parcel and consists of a two-story structure with a daylight basement. The property is bounded by Lake Washington to the west, by Forest Avenue SE to the east, and by developed residential parcels to the north and south. The total vertical relief across the site is approximately 96 feet with slope inclinations ranging from 20 to 40 percent. With the exception of the driveway leading down from Forest Avenue SE to the existing residence, the slope is undeveloped but shows signs of past grading in places. The slope is moderately vegetated with both coniferous and deciduous trees of various ages and sizes. The groundcover along the slope is well established with low-lying shrubs and underbrush.

Our understanding of the project is based on our communications with the Client and review of a preliminary design layout by the architect, Olson Kundig, shown on Figure 2. We understand that the current plan includes the demolition of the existing residence and construction of a new single-family residence on the lower, western portion of the subject site. The new residence will consist of three story home, which includes a basement level at an elevation of approximately 24-feet. The subject site lies within Erosion, Seismic and Landslide Hazard Areas, as delineated in the City of Mercer Island *Geological Hazard Maps*. Therefore, the City of Mercer Island will require a geotechnical study for the proposed project. We have been requested to explore the subsurface conditions in the vicinity of the proposed residence at the lower, western portion of the site to provide geotechnical recommendations for the planned project.

3.0 SUBSURFACE EXPLORATION

The subsurface exploration and geologic site reconnaissance was conducted on July 12, 2022, and consisted of advancing three exploration borings to gain subsurface information about the site. 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 Figure 2.

The conclusions and recommendations presented in this report are based on the exploration borings completed for this study. The locations and depths of the explorations were completed within site access 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 beyond 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

For this study, three hollow-stem auger exploration borings were performed by Geologic Drill Partners, an independent firm working under subcontract to AESI, at the approximate locations shown on Figure 2. Logs of our exploration borings, labeled EB-1, EB-2, and EB-3, are included with this report in Appendix A. The borings for this study were completed by advancing a 6-inch outer-diameter, hollow-stem auger with a track-mounted drill rig. During the drilling process, samples were obtained at generally 2.5- to 5-foot-depth intervals. After completion of drilling, each borehole was backfilled with bentonite chips and capped with sod cold mix asphalt.

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 is 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 exploration borings were continuously observed and logged by a geologist from our firm. 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 laboratory testing, as necessary. The exploration logs presented in Appendix A are based on the N-values, field observations, drilling action, and laboratory test results, if conducted.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field exploration accomplished for this study, visual reconnaissance of the site, and review of applicable geologic literature. The following section presents more detailed subsurface information organized from the shallowest (youngest) to the deepest (oldest) sediment types.

4.1 Stratigraphy

Fill

Existing fill was encountered at ground surface at the locations of EB-1 and EB-2 to a depth of approximately 4 feet below the existing grade. Fill should also be expected around the existing home, buried utility lines, and landscaped/graded areas. The existing fill was observed to be loose to medium stiff and consisted of silty sand ranging to sandy silt with varying organic contents. The existing fill requires removal or other remedial preparation below planned building areas and paving. Excavated existing fill may be suitable for reuse in structural fill applications if specifically allowed by project specifications, and if any organic or other deleterious materials are removed.

Holocene Lake Deposits

Sediments encountered below the fill in EB-1 generally consisted of loose to medium dense or soft to medium stiff, silty fine sand ranging to silt with trace gravel. The sediments were observed to be either massively bedded or stratified within the sampler. We interpret these sediments to be representative of natural, Holocene age lake deposits that were formed by sedimentation in a lacustrine environment. Lake deposits typically consist of unconsolidated silt, sand and gravel, and can contain cobbles, shells, and organic debris such as logs. The lake deposits extended to a depth of approximately 22 feet below existing grade. Due to their variable density and potential for wood and other organic debris, the Holocene lake deposits are not considered suitable for foundation support.

Holocene Mass-Wastage Deposits

The sediments encountered below the existing fill in EB-2 and near the ground surface in EB-3 generally consisted of medium stiff to stiff, sandy silt with organic inclusions. These sediments displayed a disturbed texture and indistinct morphology or bedding. We interpreted these sediments to be representative of Holocene age mass-wastage deposits. These sediments generally consist of landslide debris, colluvium, and soils which flank the slopes in the site vicinity. The mass-wastage deposits extended to a depth of approximately 9 feet in EB-2 and 12.5 feet in EB-3. Due to the high variability in the consistency and density of the material it is unsuitable for building support.

Pre-Olympia Non-Glacial Deposits

The sediments encountered below the lake deposits in EB-1 and the mass-wastage deposits in EB-2 consisted of very dense silty fine sand and very stiff silt. These sediments displayed either massive or stratified bedding within the sampler and contained abundant micas, indicative of a non-glacial environment. We interpret these sediments to be pre-Olympia in age. These sediments were deposited prior to the Olympia nonglacial interval that occurred from 15,000 to

60,000 years before present and have been consolidated by at least one glaciation. The high relative density characteristic of these sediments is due to their consolidation by the massive weight of the glacial ice that overrode them subsequent to their deposition. The pre-Olympia non-glacial sediments extended beyond the maximum depth explored of 31.5 feet at the location of EB-1 and extended to a depth of 12.5 feet at the location of EB-2. The pre-Olympia sediments are considered suitable for support of the proposed structure with suitable preparation.

Pre-Olympia Glacial Diamict

The sediments encountered below the pre-Olympia non-glacial sediments in EB-2 and mass wastage deposits in EB-3 generally consisted of very dense, dark brown to dark brownish gray, silty, fine sand and very stiff to hard, dark gray massive silt with trace to some gravel. These sediments displayed an unsorted, till-like "diamict" texture, and also contained dropstones commonly associated with a glaciolacustrine environment. The samples did not react when exposed to hydrochloric acid. We interpret these sediments to be glacially derived and pre-Olympia in age due to the observed unsorted texture, dark coloration and high relative densities. These sediments were deposited prior to the Olympia nonglacial interval that occurred from 15,000 to 60,000 years before present and have been consolidated by at least one glaciation. The high relative density characteristic of these sediments is due to their consolidation by the massive weight of the glacial ice that overrode them subsequent to their deposition. The pre-Olympia glacial diamict sediments extended beyond the maximum depth explored of 21.5 and 31.5 feet at the locations of EB-2 and EB-3, respectively. The pre-Olympia sediments are considered suitable for support of the proposed structure with suitable preparation.

4.2 Geologic Mapping

Review of the regional geologic map titled *The Geologic Map of Mercer Island, 2006* (K.G. Troost, A.P. Wisher, GeoMapNW, scale 1:12,000) indicates that the area of the subject site is underlain by pre-Olympia non-glacial and pre-Olympia glacial diamict deposits with Holocene lake deposits mapped near the existing shoreline of Lake Washington. The geologic map also delineates an overprint of mass-wastage deposits, encompassing the subject site and adjacent parcels, and extending along the steeply sloping terrain to the north, south and east of the subject site. Our interpretation of the sediments encountered at the subject site is in general agreement with the regional geologic map in that we encountered mass-wastage deposits overlying glacial and non-glacial sediments interpreted to be pre-Olympia age.

Review of regional soils mapping (*Soil Survey of King County Area, Washington*, U.S. Department of Agriculture [USDA], Soils Conservation Service [SCS] now referred to as Natural Resources Conservation Service [NRCS]) on the NRCS *Web Soil Survey* indicates that the subject site is underlain by Kitsap silt loam, 2 to 8 percent slopes (KpB), and Kitsap silt loam, 15 to 30 percent slopes (KpD). The Kitsap soils are formed from the weathering glaciolacustrine (glacial lake) deposits along terraces and strongly dissected terrace fronts. The NRCS indicates that the erosion hazard rating of the Kitsap soils is severe where it is present on slopes with inclinations of 15 percent or greater. Our interpretation of the materials encountered in our explorations is generally consistent with the regional soils map in that we encountered fine grained glacial sediments which displayed characteristics of a glaciolacustrine environment.

4.3 Hydrology

Groundwater was encountered in EB-1 at a depth of 5 feet below the surface and may represent the local ground water table. The elevation of the groundwater observed in EB-1 is approximately the same elevation as the water level observed in the lake and is likely tied to the water levels in Lake Washington. Because it takes time for groundwater in an open boring to equilibrate to a static level, groundwater level observed during drilling may be somewhat lower than actual static conditions. The groundwater was generally observed within the Holocene lake deposits overlying the glacially consolidated sediments at depth. The presence of groundwater should be anticipated during excavation of deeper foundations and cuts within these sediments and during the wet season within the fractured mass-wastage sediments at the subject site. Groundwater management and drainage should be incorporated into both the temporary and permanent drainage plan for the site if deeper cuts are anticipated into saturated deposits.

It should be noted that the occurrence and level of groundwater seepage at the site may vary in response to such factors as changes in season, precipitation, and site use. The explorations for this study were conducted in early July of 2022.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic and ground water conditions as observed and discussed herein. Review of the City of Mercer Island *Geological Hazard Maps* indicates that the subject site lies within Erosion, Seismic and Landslide Hazard Areas, as delineated in the City of Mercer Island. Based on our findings, it is our opinion that the project can be undertaken safely as long as the recommendations in this report are incorporated into the project plans and adhered to during construction.

5.0 LANDSLIDE HAZARDS AND RECOMMENDED MITIGATION

We understand that the slope east of the existing home qualifies as a Landslide Hazard Area as defined by the *Unified Land Development Code* (ULDC) based on slope inclinations and subsurface conditions observed at the subject site. Landslide Hazard Areas are defined in Chapter 19.16.010 of the ULDC as stated below.

Landslide hazard areas: 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.

As stated, we encountered mass-wastage deposits in our explorations. In our opinion, the source of the mass wasting deposits is likely the slope to the east, which extends far beyond the subject site property line. The sloping area on, and to the east of the subject site is considered a known slide area by the City of Mercer Island as shown on the *Mercer Island Landslide Hazard Assessment* map, with several identified landslide locations mapped across the slope. Based on our review of the Light Detection and Ranging (LiDAR) image encompassing the subject site shown on Figure 3, the slopes leading upward from the area of the subject site to the upland include several bowl-shaped slide features, including to the east of the subject site, across

Forest Ave. SE. Given the broad nature of the delineated landslide hazard area upslope of the subject site and neighboring parcels, the ability to mitigate risks associated with landslides occurring along these slopes, based on the relative size of the slope complex as compared to the subject site, is limited. In addition, based on our field observations and document review, it is our opinion that the area surrounding the property is likely underlain by landslide deposits. Local slope stability mitigation for the planned structure is feasible using the recommendations presented in this report.

Within the subject parcel, the slope east of the existing home extends upwards towards the eastern property line and is crossed by a paved driveway which gives access to the existing residence and the two adjacent parcels. The slope is generally inclined at a 3H:1V (Horizontal:Vertical) with one localized flat area on the upper, eastern portion of the property. Sections of the slope near the driveway appear to have been steepened during past grading. The toe of the slope near the house is supported by an approximate 5- to 6-foot-high rockery retaining wall. The vertical relief of the slope is approximately 80 feet from the top of the rockery wall to the eastern property line. The slope is moderately to densely vegetated with 4- to 36-inch-diameter deciduous and coniferous trees and other understory plants and shrubs. During our site reconnaissance, we found no visual evidence of tension cracks, emergent seepage, hummocky topography, or other indications of recent slope instability observed on any of the site slopes. We also observed that the trees located on the steep slope area were generally oriented vertically, suggesting that ongoing, deep-seated slope movement is not occurring at the subject site.

Based on the preliminary site plans provided by the Client and architect, the proposed home will expand the building pad of the existing residence and will require cuts into the slope. The expansion of the building footprint will require the removal of the existing rockery at the toe of the slope. Due to the limited area on east side of the proposed structure, creating a temporary or permanent cut slope does not appear feasible. We recommend that the cuts along the east side of the proposed home be supported by a shoring wall. We anticipate that a cantilever or anchored soldier pile wall, consisting of wide-flange steel piles, suitably embedded in the underlying dense natural soils, will provide mitigation for the risk of localized, shallow earth movement of the existing slope leading down to the proposed residence. Design details for this wall are discussed within the “Design Recommendations” section of this report. This opinion is dependent upon site grading and construction practices being completed in accordance with the geotechnical recommendations presented in this report. There is a moderate risk of shallow landslides and slope erosion occurring on the steep slope areas of the property outside the currently planned construction area, this risk can be mitigated by following the drainage and erosion mitigation recommendations contained in this report.

6.0 SEISMIC HAZARDS AND MITIGATION

The following discussion is a general assessment of seismic hazards that is intended to be useful to the project design team in terms of understanding seismic issues, and to the structural engineer for design.

All of Western Washington is at risk of strong seismic events resulting from movement of the tectonic plates associated with the Cascadia Subduction Zone (CSZ), where the offshore Juan de Fuca plate subducts beneath the continental North American plate. The site lies within a zone of strong potential shaking from subduction zone earthquakes associated with the CSZ. The CSZ can produce earthquakes up to magnitude 9.0, and the recurrence interval is estimated to be on the order of 500 years. Geologists infer the most recent subduction zone earthquake occurred in 1700 (Goldfinger et al., 2012[1]). Three main types of earthquakes are typically associated with subduction zone environments: crustal, intraplate, and interplate earthquakes. Seismic records in the Puget Sound region document a distinct zone of shallow crustal seismicity (e.g., the Seattle Fault Zone [SFZ]). These shallow fault zones may include surficial expressions of previous seismic events, such as fault scarps, displaced shorelines, and shallow bedrock exposures. The shallow fault zones typically extend from the surface to depths ranging from 16 to 19 miles. A deeper zone of seismicity is associated with the subducting Juan de Fuca plate. Subduction zone seismic events produce intraplate earthquakes at depths ranging from 25 to 45 miles beneath the Puget Lowland including the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event) and interplate earthquakes at shallow depths near the Washington coast including the 1700 earthquake, which had a magnitude of approximately 9.0. 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 or lateral spreading, 3) liquefaction, 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

6.1 Surficial Ground Rupture

Seattle Fault

The site is located within the mapped limits of the SFZ. The SFZ is a broad east – west oriented zone that extends from approximately Issaquah to Alki beach, and is approximately 2.5 to 4 miles in width from north to south. The SFZ is speculated to contain multiple distinct fault “strands”,

---

[1] Goldfinger, C., Nelson, C.H., Morey, A.E., Johnson, J.E., Patton, J.R., Karabanov, E., Gutierrez-Pastor, J., Eriksson, A.T., Gracia, E., Dunhill, G., Enkin, R.J, Dallimore, A., and Vallier, T.,2012, *Turbidite Event History—Methods and Implications for Holocene Paleoseismicity of the Cascadia Subduction Zone*: U.S. Geological Survey Professional Paper 1661–F, 170.

some of which are well understood and some of which may be poorly understood or unknown. Mapping of individual fault strands is imprecise, as a result of pervasive modification of the land surface by development, which has obscured possible surficial expression of past seismic events. Studies by the U.S. Geological Survey and others have provided evidence of surficial ground rupture along strands of the Seattle Fault (USGS, 2010[2]; Pratt et. al, 2015[3]; Haugerud, 2005[4]; Liberty et. al, 2008[5]). According to 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. Based on our review of the Washington State Department of Natural Resources (WADNR) website, inferred fault traces associated with the SFZ are located about 1.7 miles north and 1.9 miles south of the site. Due to the suspected long recurrence interval, and the distance of the site to the fault traces, the potential for surficial ground rupture along the SFZ is considered to be low during the expected life of the proposed structure.

6.2 Seismically Induced Landslides

As discussed in Section 5.0, "Landslide Hazards and Mitigation," given the broad nature of the delineated landslide hazard area upslope of the subject site and neighboring parcels, the ability to mitigate risks associated with landslides occurring under both static and seismic conditions along these slopes, based on the relative size of the slope complex as compared to the subject site, is limited. In addition, based on our field observations and document review, it is our opinion that the area surrounding the property is likely underlain by mass-wastage/landslide deposits. It is our opinion that the impact from the project to the stability of the existing slopes can be suitably mitigated provided the recommendations in this report are incorporated into the project plans and adhered to during construction.

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

---

[2] U.S. Geological Survey, 2010, Quaternary fault and fold database for the United States, accessed November 10, 2010, from USGS web site: http://earthquake.usgs.gov/hazards/qfaults/

[3] Pratt, et al., 2015, Kinematics of shallow backthrusts in the Seattle fault zone, Washington State: Geosphere, v. 11, no. 6, p. 1-27).

[4] Haugerud, R.A., 2005, Preliminary geologic map of Bainbridge Island, Washington: U.S. Geological Survey Open-File Report 2005-1387, version 1.0, 1 sheet, scale 1:24,000.

[5] Liberty, Lee M.; Pratt, Thomas L., 2008, Structure of the eastern Seattle fault zone, Washington State -New insights from seismic reflection data: Bulletin of the Seismological Society of America, v. 98, no. 4, p. 1681-1695.

strength. The soil is said to be liquefied when nearly all of 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 very soft to stiff, non-cohesive silt and very loose to medium dense, non-silty to silty sands with low relative densities, accompanied by a shallow water table.

Loose, saturated sediments were encountered at the location of EB-1, which was situated closest to Lake Washington between the shoreline and the existing residence. The soils observed at the location of EB-2, within the footprint of the proposed residence, consisted of loose to medium dense mass-wastage deposits overlying dense to very dense glacially consolidated sediments. Adverse groundwater conditions were not observed within EB-2 or at the location of EB-3, upslope from the proposed house. Based on these findings, it appears that the liquefiable sediments are located predominately near the shoreline in areas underlain by loose lake deposits. Due to the loose to medium dense mass-wastage deposits observed at the location of EB-2, and the potential for liquefiable lake deposits to be present below the proposed building, we recommend that new foundations be supported by small diameter pipe piles that fully penetrate the loose mass-wastage deposits and liquefaction prone sediments. This will mitigate the potential for building settlement caused by loose or liquefiable sediments under both static and seismic conditions. It is our opinion that the risk of damage to the proposed home and site improvements by liquefaction is low, provided the recommendations in this report are incorporated into the project plans and adhered to during construction.

6.4 Ground Motion

Structural design of the buildings should follow 2018 IBC standards. We recommend that the project be designed in accordance with Site Class “D” as defined in IBC Table 20.3-1 of *American Society of Civil Engineers* (ASCE) *7 - Minimum Design Loads for Buildings and Other Structures*.

7.0 EROSION HAZARDS AND MITIGATION

The sediments which underly the proposed development area contains significant quantities of silt and fine sand and is considered to be highly sensitive to disturbance when wet and erosion where it is present below sloping areas. The NRCS has mapped the soils on the site as Kitsap silt loam. The NRCS erosion hazard rating of this soil type is “severe” where present on slopes steeper than 15 percent. Those portions of the site with slope inclinations exceeding 15 percent classify as Erosion Hazard Areas under the *Mercer Island Unified Land Development Code*.

In order to mitigate erosion hazards and the potential for off-site sediment transport, we recommend the following best management practices (BMPs):

1. To the extent practical, earthwork should be avoided during the wet season, October 1st through April 30th. In addition to the increased risk of erosion hazards during this timeframe, the City of Mercer Island requires a Seasonal Development Limitation Waiver for land clearing, grading, filling and foundation work taking place within an Erosion, Potential Slide, or Steep Slope Hazard area between October 1st and April 1st.

2. The winter performance of a site is dependent on a well-conceived plan for control of site erosion and stormwater runoff. The site plan should include ground-cover measures and staging areas. The contractor should be prepared to implement and maintain the required measures to reduce the amount of exposed ground.

3. Temporary erosion and sedimentation control (TESC) elements and perimeter flow control should be established prior to the start of grading.

4. During the wetter months of the year, or when significant storm events are predicted during the summer months, the work area should be stabilized so that if showers do occur, it can receive the rainfall without excessive erosion or sediment transport. The stabilization process should include establishing temporary stormwater conveyance channels through work areas to route runoff to the approved treatment/discharge facilities.

5. All areas of disturbed soil should be revegetated as soon as possible. If it is outside of the growing season, the disturbed areas should be covered with mulch. Straw mulch provides a cost-effective cover measure and can be made wind-resistant with the application of a tackifier after it is placed.

6. Surface runoff and discharge should be controlled during and following development. Uncontrolled discharge may promote erosion and sediment transport.

7. 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 stockpiles with plastic sheeting, or the use of silt fences around pile perimeters.

8. If the area of development will cover an area greater than 1 acre in size, it will be required to obtain a Construction Stormwater General Permit per the Washington State Department of Ecology (Ecology). Under this permit, a Certified Erosion and Sediment Control Lead (CESCL) will be required to make weekly site visits to monitor erosion control, BMPs, and levels for turbidity and pH. AESI is available to help prepare permit application documents and can provide CESCL monitoring as requested.

It is our opinion that with the proper implementation of the TESC plans and by field-adjusting appropriate erosion mitigation (BMPs) throughout construction, the potential adverse impacts from erosion hazards on the project may be mitigated.

8.0 STATEMENT OF RISK

For Section 19.07.160.B.3 of the ULDC, the City of Mercer Island requires a statement of risk by the geotechnical engineer. It is Associated Earth Sciences, Inc.'s (AESI's) opinion that provided that the recommendations contained in this report are followed, the development practices proposed for the alteration would render the development as safe as if it were not located in a geologic hazard area and do not adversely impact adjacent properties.

III. DESIGN RECOMMENDATIONS

9.0 INTRODUCTION

It is our opinion that, from a geotechnical standpoint, the site is suitable for the proposed improvements provided that the recommendations contained herein are properly followed. The near-surface soils consisted of existing fill and loose to medium dense lake deposits or mass-wastage deposits. Foundation loads from the proposed residence foundation should be extended through these sediments, into the dense to very dense pre-Olympia age deposits. We recommend that this is accomplished through the use of a pipe pile and grade beam foundation system for the proposed new residence. The majority of the proposed home appears to require pipe piles, but conventional spread footings may be feasible in some locations, specifically the east side of the home, if foundation grades fully penetrate the loose surficial sediments into the dense to very dense pre-Olympia sediments. We recommend that a single type of foundation support be used in the design of the home, but we may revise our recommendations based on final plan review. We have included conventional spread footing recommendations to the “Foundations” section of this report in the case that conventional spread footings are deemed feasible, based on the final plans. We also anticipate that the proposed residence will require cuts into the adjacent slope to achieve desired site and foundation grades. As such we recommend that where these cuts are required, the slope be supported by a cantilever or anchored soldier pile wall, consisting of wide-flange steel piles, suitably embedded in the underlying dense natural soils. This will provide mitigation for the risk of localized, shallow earth movement of the existing slope leading down to the proposed residence.

10.0 SITE PREPARATION

Once the existing home has been demolished, any remaining foundation elements should be removed. Site preparation within the proposed building area should include removal of all vegetation, topsoil, and any other deleterious materials. If conventional spread footings and slab on grade floor support is utilized, existing fill beneath planned footing and floor slab areas should be removed. Any depressions below planned final grades resulting from demolition activities should be backfilled with structural fill, as discussed under the “Structural Fill” section of this report. After stripping of the surficial sod/topsoil horizon has been completed, any remaining roots and stumps should be removed from structural areas. All soils disturbed by stripping and grubbing operations should be recompacted as described below for structural fill.

10.1 Site Drainage and Surface Water Control

The site should be graded to prevent water from ponding in construction areas and/or flowing into excavations. Exposed grades should be crowned, sloped, and smooth drum-rolled at the end of each day to facilitate drainage. Accumulated water must be removed from subgrades and work areas immediately prior to performing further work in the area. Equipment access will be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased, if drainage efforts are not accomplished in a timely sequence.

Final exterior grades should promote free and positive drainage away from the planned new addition at all times. Water must not be allowed to pond or to collect adjacent to foundations or within the immediate building area.

10.2 Subgrade Protection

If building construction will proceed during the winter, we recommend the use of a working surface of sand and gravel, crushed rock, or quarry spalls to protect exposed soils, particularly in areas supporting concentrated equipment traffic. During wet season construction, staging areas and areas that will be subjected to repeated heavy loads, a minimum thickness of 8 inches of quarry spalls or 12 inches of pit run sand and gravel is recommended. In addition to work during wet weather, an armored surface as described above will be required for the heavy machinery associated with the installation of shoring and deep foundation elements.

10.3 Subgrade Compaction

Following the recommended clearing, site stripping, and planned excavation, the stripped subgrade should be observed by the geotechnical engineer prior to structural fill placement to identify any soft/loose yielding soils or existing fills. If any loose natural sediments are encountered the contractor should attempt to recompact the subgrade to a firm and unyielding state. If loose/soft, yielding natural sediments or fill soils are encountered, they should be removed to a stable subgrade. The subgrade should then be recompacted to a firm and unyielding condition. Low areas and excavations may then be raised to the planned finished grade with compacted structural fill. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering-controlled conditions in accordance with the project specifications.

10.4 Wet Weather Conditions

Since site soils are moisture-sensitive and the site contains regulated steep slope and landslide hazards, we recommend that construction occurs during the dry season. If construction does proceed during an extended wet weather construction period, the moderately moisture-sensitive site soils may become easily disturbed and too wet to use for structural fill. In addition to the City of Mercer Island requires a Seasonal Development Limitation Waiver for land clearing, grading, filling and foundation work taking place within an Erosion, Potential Slide, or Steep Slope Hazard area between October 1st and April 1st.

10.5 Temporary and Permanent Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction based on the local conditions encountered at that time. For planning purposes, we anticipate that temporary, unsupported cut slopes in areas of existing fill or medium dense, mass-wastage sediments can be made at a maximum slope of 1.5H:1V (Horizontal:Vertical). Temporary, unsupported cut slopes within the dense or very stiff glacially consolidated sediments can be planned at a maximum slope of 1H:1V. Temporary vertical cuts up to 4 feet in height may be planned in all of these materials. Flatter inclinations may be recommended in areas of seepage. 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.

Permanent cut and structural fill slopes should not exceed an inclination of 2H:1V.

10.6 Frozen Subgrades

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be recompacted prior to placing subsequent lifts of structural fill. Alternatively, the frozen material could be stripped from the subgrade to reveal unfrozen soil prior to placing subsequent lifts of fill. 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.

11.0 STRUCTURAL FILL

Placement of structural fill may be necessary to establish desired grades in some areas or to backfill utility trenches. All references to structural fill in this report refer to subgrade preparation, fill type, and 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.

11.1 Subgrade Compaction

After overexcavation/stripping has been performed to the satisfaction of the geotechnical engineer/engineering geologist, the exposed ground should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, suitable recompaction may be difficult or impossible to attain 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 the exposed ground is approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades.

10.2 Structural Fill Compaction

Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to at least 95 percent of the modified Proctor maximum dry density using *ASTM International* (ASTM) D-1557 as the standard. Utility trench backfill should be placed and compacted in accordance with applicable municipal codes and standards. The top of the compacted fill should extend horizontally a minimum distance of 3 feet beyond footings or pavement edges before sloping down at an angle no steeper than 2H:1V. Fill slopes should either be overbuilt and trimmed back to final grade or surface-compacted to the specified density.

11.3 Moisture-Sensitive Fill

Soils in which the amount of fine-grained material (smaller than No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. The use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. Excavated portions of the granular, onsite sediments may be suitable for use as structural fill provided that they are free of roots, oversized rocks, and other deleterious materials and exhibit a moisture content at the time of construction compatible with achieving the recommended compaction specification. Because some of the onsite soils contain a high percentage of silt, compaction of these sediments to the recommended minimum density will only be achievable over a narrow range of moisture contents and use of these materials for structural fill is not recommended. Maximum rock size for structural fill applications should be limited to diameters of approximately 6 inches or less.

Construction equipment traversing the site when the silty on-site sediments are very moist or wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction of the natural materials cannot be attained, 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.

11.4 Structural Fill Testing

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material at least 3 business days in advance to perform a Proctor test and determine its field compaction standard.

A representative from our firm should observe 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

We recommend a pipe pile foundation to support the proposed structure. These piles will transfer the foundation load through the existing fill, lake deposits, and mass-wastage deposits to the underlying suitable bearing soils. Based on our explorations performed for this study, we anticipate that native soils suitable for foundation support may be closest to the foundation grade at the northeast portion of the proposed structure and on the order of 20 feet in depth at the southwest portion of the structure. This is based on the proposed basement elevation of 24 feet, provided by Olson Kundig. We have provided recommendations for conventional spread footings in addition to pipe pile recommendations in the event that spread footings are deemed feasible upon review of the final plans, but we recommend that a pipe pile supported foundation be utilized for design purposes.

12.1 Pipe Pile Foundation

Pipe piles should extend through the loose surficial sediments and penetrate the underlying dense pre-Olympia sediments until refusal.

The pipe piles should consist of galvanized steel pipe, driven to refusal, which is defined as less than a given amount of penetration during a time interval of continuous driving using a pneumatic or hydraulic hammer. The time interval of continuous driving and the allowable pile load is dependent upon the pipe pile diameter and the weight of the driving hammer. We have provided design recommendations for small diameter piles in Table 1.

Table 1
Pipe Pile Design Parameters

Nominal Pipe Diameter	Minimum Wall Thickness	Minimum Hammer Size	Allowable Axial Capacity	Driving Time (seconds/inch)
3-inch	Schedule 40	850 pounds	12 kips	10
4-inch	Schedule 40	1,100 pounds	17 kips	10
6-inch	Schedule 40	3,000 pounds	30 kips	6

In order for the stated pile capacities to apply, the pipe piles should be driven to refusal, which is defined as less than 1 inch of penetration during the specified period of continuous driving. They should also completely penetrate the existing fill and mass-wastage debris. Due to varying subsurface conditions within the underlying sediments, it cannot be known at what depths refusal will be encountered during pile driving. For preliminary estimating purposes, pile lengths on the order of 10 to 30 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.

Refusal depths may greatly vary between piles installed only a few feet apart. Geotechnical observation of pile installation is recommended.

Anticipated settlement of pipe 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. AESI, acting as the owner’s field representative, would determine the required pile lengths, and keep records of pertinent installation data. Geotechnical monitoring of pipe pile installation will likely be required by the City of Mercer Island.

No lateral capacity would be provided by vertically installed pipe piles. Lateral resistance can be derived from passive soil resistance against the buried portion of the foundation (i.e., the pile cap) or from the installation of batter piles. Lateral resistance for batter piles should be taken as the horizontal component of the applied axial pile load. Batter piles are typically installed at 1H:4V inclination. Base friction between the base of the pile caps/grade beam and the underlying soil should be ignored for pipe pile foundations.

Pile load tests should be performed on a minimum of 3 percent (1 pile minimum, 5 piles maximum) of the piles to verify suitable vertical capacity. Load tests should be performed with a calibrated jack and dial gauge in general accordance with ASTM D-1143-81 “quick load test” method. A successful load test will demonstrate a near-linear load-deflection relationship up to at least 200 percent of design load with total permanent deflection of less than ¼ inch.

Due to the potential ground vibrations associated with a driven pipe pile foundation, we recommend a detailed photo/video survey of the existing adjacent facilities (buildings, sidewalks, utilities, etc.) prior to construction. The purpose of the survey is to document any cracks, settlement, or other conditions that may exist prior to pile driving activities. Ongoing surveillance should be continued during construction. If any features are discovered that might be related to vibrations or settlement, construction should be stopped until the cause is determined and corrected.

12.2 Conventional Shallow Foundation

Spread footings may be utilized for building support when founded on the very dense pre-Olympia deposits, or on structural fill placed over these dense natural sediments. For footings bearing directly on the dense pre-Olympia natural sediments or on AESI-approved structural fill, an allowable soil bearing pressure of 2,500 pounds per square foot (psf) may be used for design purposes, including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings for the proposed addition should be buried a minimum of 18 inches into the surrounding soil for frost protection. No minimum burial depth is required for interior footings; however, all footings must penetrate to the prescribed stratum, and no footings should be founded in or above loose, organic, or existing fill soils.

It should be noted that the area bound by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

Anticipated settlement of footings founded as described above should be on the order of ¾ inch or less. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the design bearing capacity of the soils has been attained and that construction conforms to the recommendations contained in this report. Such inspections may be required by the governing municipality. Perimeter footing drains should be provided, as discussed under the "Drainage Considerations" section of this report.

13.0 LATERAL WALL PRESSURES

All backfill behind retaining walls or around foundation units should be placed as per our recommendations for structural fill and as described in this section of the report. Horizontally backfilled retaining walls that are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid equal to 35 pounds per cubic foot (pcf). Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid of 55 pcf. Retaining walls that retain sloping backfill at a maximum angle of 2H:1V should be designed using an equivalent fluid pressure of 55 pcf for yielding conditions or 65 pcf for fully restrained conditions.

In accordance with the 2018 IBC, permanent retaining wall design should include seismic design parameters. Based on the site soils and assumed wall backfill materials, we recommend a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. A rectangular pressure distribution of 5H and 10H psf (where H is the height of the wall in feet) should be included in design for "active" and "at-rest" loading conditions, respectively. The resultant of the rectangular seismic surcharge should be applied at the midpoint of the walls.

The lateral pressures presented above are based on the conditions of a uniform horizontal backfill consisting of the on-site, natural, glacial sediments or imported sand and gravel 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 wall.

Footing drains must 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 the ground surface using imported, washed gravel against the walls placed to be continuous with the footing drain.

13.1 Passive Resistance and Friction Factors

For foundation grade beams/keyways 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 300 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.

In the case that shallow foundations are cast directly on dense native sediments, lateral loads can be resisted by friction between the foundation and the dense sediments or supporting structural fill soils, and by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with structural fill and compacted to at least 95 percent of the maximum dry density to achieve the passive resistance provided below. We recommend the following allowable design parameters:

- Passive equivalent fluid = 300 pcf
- Coefficient of friction = 0.30 *Spread Footings Only

14.0 FLOOR SUPPORT

Due to the loose nature of the subgrade soils, we recommend that structural pile/grade beam support be provided for settlement-sensitive, slab-on-grade floors. The floors should be cast atop a minimum of 4 inches of washed pea gravel or washed crushed rock to act as a capillary break where moisture migration through the slabs is to be controlled. The capillary break material should be overlain by a 10-mil-thick vapor barrier material prior to concrete placement. American Concrete Institute (ACI) recommendations should be followed for all concrete placement.

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 communicated with the capillary break material.

15.0 SOLDIER PILE WALL

We anticipate that a cantilever or anchored soldier pile wall, consisting of wide-flange steel piles, suitably embedded in the underlying natural soils, will provide mitigation for the risk of localized, shallow earth movement of the existing slope leading down to the proposed residence. This wall may consist of buried soldier piles at or above the base of the slope and faced with lagging to provide mitigation for shallow erosion along the slope face. Treated timber lagging should be used to support the soil between the piles. A structural engineer should design the wall system based on the soil parameters provided in this report.

The construction sequence for soldier pile wall systems typically involves installing each pile to the minimum specified embedment depth below the ground surface (if buried) or base of wall excavation (lagged portion) under the observation of the geotechnical engineer or designated field representative. Drilled and grouted piles should be allowed to set for at least 72 hours prior to beginning excavation. Once the piles have been installed to the satisfaction of the geotechnical engineer, excavation may proceed in vertical sections of 4 feet or less. The actual height of the excavated sections that provide a stable excavation face should be adjusted in the field, depending on actual soil and groundwater conditions at the time of excavation, but should not exceed 4 feet. Treated timber lagging, as specified by the structural engineer, should be installed and backfilled with permeable soils to prevent the buildup of water behind the lagging boards. No excavation sections should be left open overnight.

We recommend that a cantilever or single-row tieback soldier pile wall system be designed to resist an active lateral earth pressure of 45(H) pounds psf for the height of the wall retaining mass-wastage deposits and an active earth pressure of 35(H) for the height of the wall retaining pre-Olympia age deposits, presented as a triangular distribution for a level backslope. For an angled backslope up to 2H:1V (Horizontal:Vertical), an active lateral earth pressure of 85(H) psf may be used for the height of the wall retaining mass-wastage deposits and an active earth pressure of 55(H) may be used for the height of the wall retaining pre-Olympia age deposits. The active earth pressure acts over the pile spacing above the excavation base. Based on the sediments observed at the location of EB-3 and the layout shown on the preliminary site plan (Figure 2), we estimate that approximately 12.5 feet of the retained soil height will consist of mass-wastage deposits. This may vary based on the location of the wall on the subject site. An allowable passive resistance of 300(D) psf can also be assumed to act over twice the pile width (or grouted diameter) below the excavation base for the portion of the pile embedded in dense pre-Olympia sediments. The upper 2 feet on the passive side of the piles should be neglected and truncated from a triangular distribution. We recommend a minimum depth of embedment of 10 feet below the base of this excavation for all piles. These recommendations for lateral earth pressures are illustrated on Figure 4.

If adjacent structures, slopes, heavy construction traffic, materials stockpiling, or other substantial surcharges are to be applied during construction, these surcharges should also be included in the design.

Soil conditions may differ from those described in this report. While no indication of larger clasts was observed within the mass-wastage deposits or pre-Olympia sediments during our study, large cobbles or boulders may still be encountered during pile installation.

The drilling contractor should be prepared to encounter groundwater and to use casing, drilling slurry, or other methods of stabilizing the hole in the case of caving or heaving soil conditions. If more than 6 inches of standing water or slough is present at the bottom of the boring prior to grout placement, the contractor should be prepared to use a tremie pipe to place grout continuously from the bottom up. Grout may consist of lean-mix concrete or controlled density fill (CDF), as specified by the structural engineer, to ease chipping for lagging installation.

Timber lagging can be designed to resist reduced lateral earth pressures as a result of soil arching between piles. For the site soils, the lagging can be designed to resist 50 percent of the calculated lateral load at any given point. Caving could be experienced when excavating and installing lagging between piles. Overexcavation of soils behind the lagging should be avoided. Excavation should extend just far enough to allow lagging installation. Any void spaces behind lagging should be filled with pea gravel or other suitable free-draining material to prevent caving and loss of support for adjacent ground.

We recommend that corrosion protection be used for the structural elements of the soldier pile wall designed to act as a permanent structure. We also recommend that the seismic surcharges incorporated into the design of the permanent foundation elements also be incorporated into the design of the soldier pile wall.

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 for the full wall height using imported, washed gravel against the walls. Stormwater collected from the wall or other site drainage should discharge via tightline drain to an approved receptor or past the toe of the slope.

15.1 Tiebacks

Tiebacks may be needed to provide additional lateral resistance. Grouted tieback anchors are frequently used for this application. A potential alternative to a grouted anchor would be helical or driven anchors (Chance®, MantaRay®, etc.). AESI can provide design values for these systems if needed.

For tiebacks used in the soldier pile wall system, the anchors must be located far enough behind or below the soldier pile wall to develop anchorage within a stable soil mass to prevent system failure or excessive deformation. We recommend that this anchorage be obtained behind an assumed failure plane defined by a horizontal line extending a distance equal to 15 feet behind the base of the retained excavation, which then rotates 60 degrees from the horizontal and extends upward to the ground surface. The area between this assumed failure plane and the retained excavation is referred to as the “no-load zone.” These recommendations are presented on Figure 4. The anchor loads are transmitted to the surrounding soil by side friction or adhesion with the soil. Grouted tieback anchors completed using hollow-stem auger techniques within the natural pre-Olympia sediments may be designed for a presumptive allowable shaft friction of 1,000 psf. Alternatively, for 6-inch-diameter, pressure-grouted anchors installed within the natural pre-Olympia sediments, a presumptive allowable shaft friction of 2,000 psf can be used. Presumptive anchor design loads should be confirmed by proof-testing, as outlined subsequently. All anchors should be a minimum of 10 feet in length past the no-load zone. Tieback anchors should be installed at an angle of at least 15 degrees below the horizontal.

Care must be exercised when installing tiebacks to avoid existing utilities and foundations. We recommend for this site that each anchor be sized for a design or allowable load of not more than 50 percent of the ultimate load available through the anchor (as indicated by 200-percent verification tests). Anchors should be tested and evaluated according to Post-Tensioning Institute (PTI) guidelines. The test anchors should be capable of holding the ultimate load without excessive yield or creep so that a factor of safety of at least 2.0 is available for production anchors should further stressing occur. The rods or cables should transmit the anchor load to the soldier pile in such a manner to avoid eccentric loading.

15.2 Anchor Tests

A series of anchor tests should be performed to verify the design and ultimate skin friction or adhesion of the tieback anchors. Because of the variation in the soil types and their densities, we recommend that AESI monitor the anchor test program. A common anchor testing program would consist of at least two 200-percent verification tests of the design or allowable load in the soil plus proof-loading every production anchor to 130 percent of the design load. Verification test anchors are usually loaded in 25-percent increments that are held for 5 minutes up to the final load of 200-percent design load. The 200-percent load is commonly held for an hour and creep measured. The other component of the anchor test program for the project would be proof-loading each of the production anchors to 130 percent of the design load. Each anchor should withstand this load for at least 5 minutes. The anchor should then be locked off at the design load.

Subsequent to locking off the tiebacks at the design load, all of the tieback holes should be backfilled to prevent possible collapse of the holes and any related consequences. Typically, sand is used as backfill material; however, most non-cohesive mixtures are suitable (subject to approval by the geotechnical engineer) provided there is no bonding to the tierods.

16.0 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, polyvinyl chloride (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, and the drains should be constructed with sufficient gradient to allow gravity discharge away from the building. In addition, all cast-in-place 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 structure to achieve surface drainage. At no time should water be allowed to discharge onto the steep onsite slopes. All collected runoff must be tightlined to a City-approved location.

17.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

Our recommendations are preliminary in that definite building locations and construction details have not been finalized at the time of this report. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. If significant changes in grading are made, we recommend that AESI perform a geotechnical review of the plans prior to final design completion. 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 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 this current scope of work. If these services are desired, please let us know, and we will prepare a proposal.

We have enjoyed working with you on this study and are confident that 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

____
Peter E. Linton, L.G.
Senior Staff Geologist

____
Jeffery P. Laub, P.E., L.G., L.E.G.
Associate Engineer/Geologist

BRUCE L. BLYTON
26690

Bruce
Blyton

Digitally signed
by Bruce Blyton
Date: 2022.08.16
13:51:29 -07'00'

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

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Site and Exploration Plan
	Figure 3:	LiDAR-Based Topography
	Figure 4:	Temporary Soldier Pile Retaining Wall Design Criteria
	Appendix:	Exploration Logs

DATA SOURCES / REFERENCES:
USGS: 7.5' SERIES TOPOGRAPHIC MAPS, ESRI/I-CUBED/NGS 2013
KING CO: STREETS, CITY LIMITS, PARCELS, PARKS 9/21
LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

N
0 2000
FEET

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

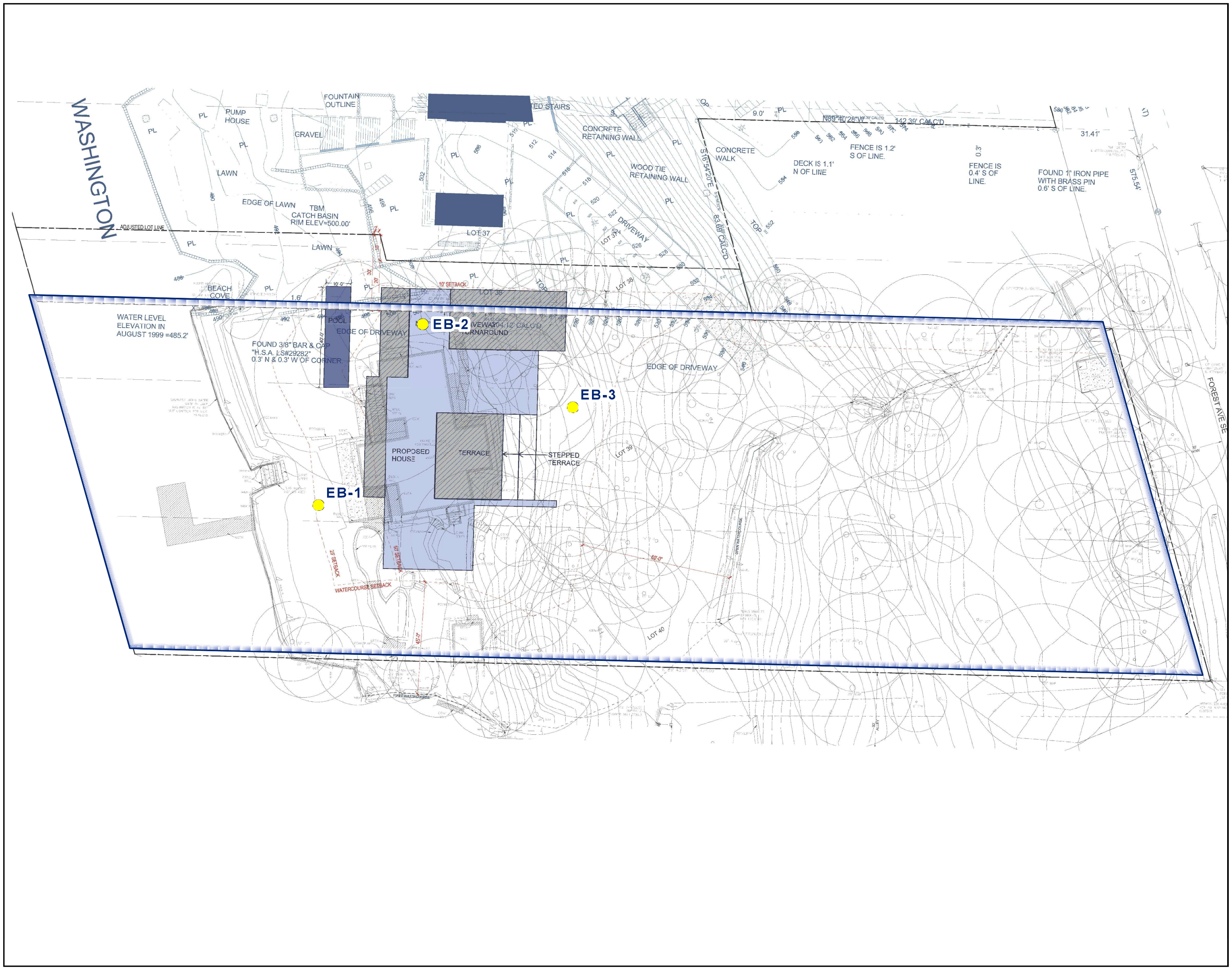
associated earth sciences incorporated

VICINITY MAP

CHASE RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
20220141E001	7/22	1

G:\GIS_Projects\aaY2022\220141 Chase Residence\APRX\20220141E001 F1 VM_Chase.aprx | 20220141E001 F1 VM_Chase | 7/14/2022 9:30 AM



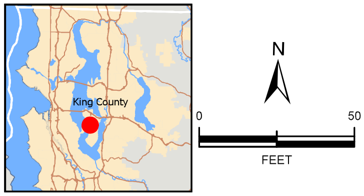
LEGEND

▭ SITE

● EXPLORATION BORING

DATA SOURCES / REFERENCES:
OLSON KUNDIG, CHASE RESIDENCE, SITE PLAN, A1.00, UNDATED
SITE BOUNDARY FROM KING CO. PARCEL DATA, 4/22
GEOREFERENCED USING KING CO AERIAL PICTOMETRY INT. 2021

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE



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

CHASE RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
20220141E001	7/22	2

G:\GIS_Projects\aaY2022\220141 Chase Residence\APRX\20220141E001 F2 SP_Chase.aprx | 20220141E001 F2 SP_Chase | 7/14/2022 9:08 AM

G:\GIS_Projects\aaY2022\220141 Chase Residence\APRX\20220141E001 F3 Lidar_Chase.aprx | 20220141E001 F3 Lidar_Chase | 7/13/2022 4:59 PM

LEGEND

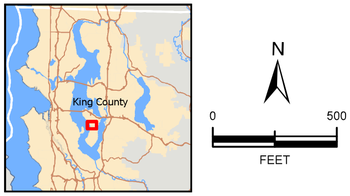
🟨 SITE

～ CONTOUR 20 FT

～ CONTOUR 5 FT

DATA SOURCES / REFERENCES:
PSLC: KING COUNTY 2016, GRID CELL SIZE IS 3'.
DELIVERY 1 FLOWN 2/24/16 - 3/28/16
CONTOURS FROM LIDAR
KING CO: STREETS, PARCELS, 4/22

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE



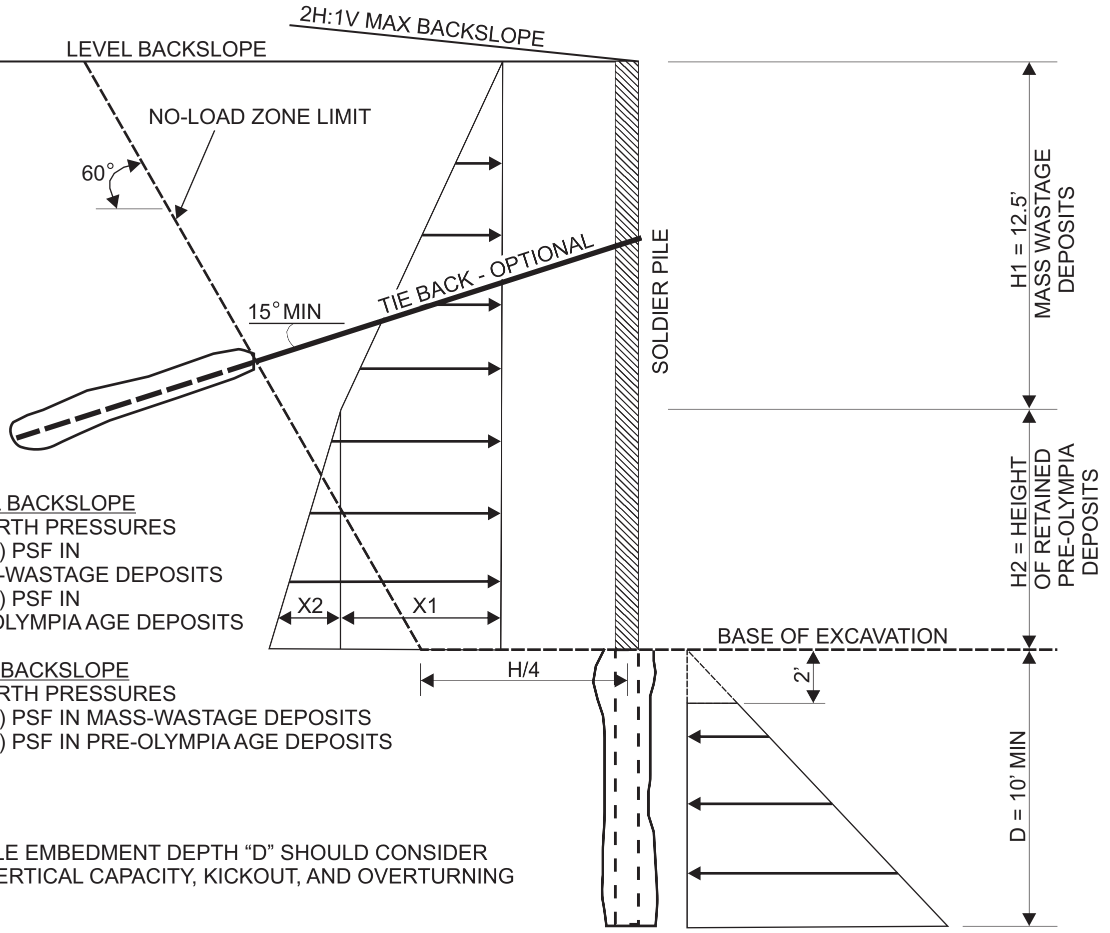
BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

associated earth sciences incorporated

LIDAR BASED TOPOGRAPHY

CHASE RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
20220141E001	7/22	3



FOR LEVEL BACKSLOPE
ACTIVE EARTH PRESSURES
X1 = 45 (H1) PSF IN
MASS-WASTAGE DEPOSITS
X2 = 35 (H2) PSF IN
PRE-OLYMPIA AGE DEPOSITS

FOR 2H:1V BACKSLOPE
ACTIVE EARTH PRESSURES
X1 = 85 (H1) PSF IN MASS-WASTAGE DEPOSITS
X2 = 55 (H2) PSF IN PRE-OLYMPIA AGE DEPOSITS

300 (D) PSF

PASSIVE PRESSURE ACTS
OVER TWICE PILE DIAMETER

PASSIVE PRESSURE TRUNCATED
2 FEET BELOW BASE OF EXCAVATION

NOTES:

1. SOLDIER PILE EMBEDMENT DEPTH “D” SHOULD CONSIDER NECESSARY VERTICAL CAPACITY, KICKOUT, AND OVERTURNING RESISTANCE.

2. ALL TIEBACK ANCHORS - IF USED - SHALL BE PRESTRESSED TO 130 PERCENT OF DESIGN LOAD AND LOCKED OFF AT 100 PERCENT OF DESIGN LOAD. AT LEAST TWO ANCHORS ON EACH SIDE OF THE EXCAVATION SHALL BE PRESTRESSED TO 200 PERCENT AND MONITORED FOR CREEP. TIE-BACK ANCHOR ZONE IS TO BE LOCATED BEHIND THE NO-LOAD ZONE AND FULLY EMBEDDED IN PRE-OLYMPIA DEPOSITS.

3. ALLOWABLE TIEBACK - SOIL ADHESION = SEE REPORT TEXT.

4. PASSIVE PRESSURES INCLUDE A FACTOR OF SAFETY OF 1.5.

5. ALLOWABLE SKIN FRICTION OF SOLDIER PILE - 750 PSF OVER DEPTH “D-2”. ALLOWABLE END BEARING = 18 KSF.

6. DIAGRAM DOES NOT INCLUDE HYDROSTATIC PRESSURES OR SURCHARGES AND ASSUMES WALLS ARE SUITABLY DRAINED TO PREVENT BUILDUP OF HYDROSTATIC PRESSURE.

7. DIAGRAM IS ILLUSTRATIVE AND NOT REFERENCED TO A PARTICULAR LOCATION.

8. DIAGRAM DOES NOT INCLUDE PRESSURES DUE TO SURFACE SURCHARGES FROM ANY ADJACENT STRUCTURES, SLOPES, STOCKPILED MATERIALS, OR CONSTRUCTION EQUIPMENT. THESE PRESSURES MUST BE PROVIDED BY THE STRUCTURAL ENGINEER.

9. BASE OF EXCAVATION SHALL BE DEFINED AS THE FOUNDATION SUBGRADE ELEVATION.

associated
earth sciences
incorporated

TEMPORARY SOLDIER PILE
RETAINING WALL DESIGN CRITERIA
CHASE RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
20220141E001	8/22	4

220141 Chase \ 20220141E001 F4 Soldier.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)

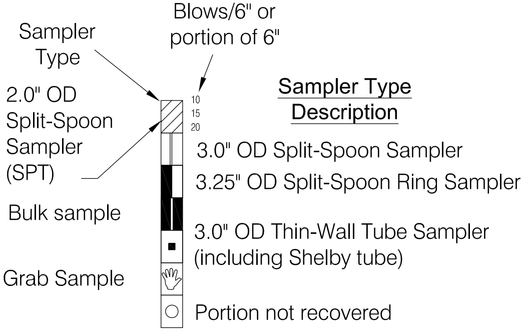
[3] Estimated Percentage

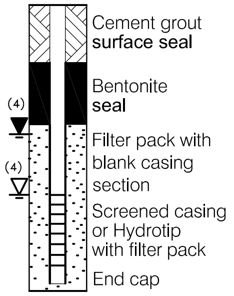
Component	Percentage by Weight
Trace	<5
Some	5 to <12
Modifier (silty, sandy, gravelly)	12 to <30
Very *modifier* (silty, sandy, gravelly)	30 to <50

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 EB-1

Chase Residence — Sheet: 1 of 2

Mercer Island, WA | Start Date: 7/12/22 | Logged By: PL

20220141E001 | Ending Date: | Approved By: CMM

Driller/Equipment: Geologic Drill Partners/Mini-Track Drill
Hammer Weight/Drop: 140Lbs/30"
Hole Diameter (in): 7 Inches
▼ Groundwater Depth ATD (ft): 5
Total Depth (ft): 31.5
Ground Surface Elevation (ft): ≈23
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot (10 20 30 40 50+)	Other Tests
0		1		**Grass/Topsoil - 4 inches** **Fill** Moist, brown, silty, fine SAND, trace gravel; scattered organics (SM).		5 6 4	10	
2.5		2		Moist, brown, silty, fine SAND, trace gravel; poor recovery (SM).		3 2 2	4	
				Holocene Lake Deposits				
5		3		Wet, brownish gray, very silty, fine SAND ranging to sandy silt, trace gravel; massive (SM/ML).	▼	4 3 4	7	
7.5		4		Wet, gray, fine SAND, some silt; interbeds with gray silt; stratified (SP-SM/ML).		3 1 2	3	
10		5		Wet, gray, silty, fine SAND, trace gravel; faintly stratified (SM).		5 6 6	12	
12.5								
15		6		No recovery.		3 6 6	12	
17.5								

associated earth sciences incorporated

Exploration Boring EB-1

Chase Residence
Sheet: 2 of 2
Mercer Island, WA
Start Date: 7/12/22
Logged By: PL
20220141E001
Ending Date:
Approved By: CMM

Driller/Equipment: Geologic Drill Partners/Mini-Track Drill
Hammer Weight/Drop: 140Lbs/30"
Hole Diameter (in): 7 Inches
▼ Groundwater Depth ATD (ft): 5
Total Depth (ft): 31.5
Ground Surface Elevation (ft): ≈23
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot (10, 20, 30, 40, 50+)	Other Tests
20		7		Very moist, brown, fine SAND, some to trace silt, trace gravel; massive; gray, silty, gravel lodged in tip of sampler; blow counts may be overstated (SP-SM).		16 35 2	41	
22.5				**Pre Olympia Non-Glacial** Grinding drill action at 22 feet.				
25		8		Moist, orangish brown, silty, fine SAND with beds of sandy silt, trace gravel; stratified; abundant micas (SM).		39 50'5"	50/5"	
27.5				Firm but smooth drill action.				
30		9		Moist, grayish green, SILT with beds of gray fine sand, some silt; stratified; abundant mica (ML/SP-SM).		18 4 50/5"	50/5"	
32.5				Groundwater encountered at 5 feet ATD, corresponds to water level in Lake Washington. Samples did not react when exposed to diluted hydrochloric acid.				
35								
37.5								

8/2/2022

20220141E001

associated earth sciences incorporated

Exploration Boring EB-2

Chase Residence | Sheet: 1 of 2

Mercer Island, WA | Start Date: 7/12/22 | Logged By: PL

20220141E001 | Ending Date: 7/12/22 | Approved By: CMM

Driller/Equipment: Geologic Drill Partners/Mini-Track Drill
Hammer Weight/Drop: 140Lbs/30"
Hole Diameter (in): 7 Inches
▼ Groundwater Depth ATD (ft): N/A

Total Depth (ft): 21.5
Ground Surface Elevation (ft): ≈34
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample	Description	Water Level	Blows/6"	Blows/Foot	Other Tests
0	1	**Asphalt - 3 inches** **Crushed Rock - 2 inches** **Fill** Moist, brown, silty, fine SAND ranging to sandy silt, trace gravel; chaotic texture; organics inclusions (SM/ML)		20 5 5	10	
2.5	2	Moist, dark brown to brown, sandy, SILT, trace gravel; disturbed texture; organic inclusions (ML).		6 2 3	5	
		Holocene Mass-Wastage Deposits				
5	3	Moist, brown and dark brown, sandy, SILT, trace gravel; becomes sandier in tip; disturbed texture; organic inclusions (ML).		2 3 3	6	
7.5	4	Moist, brown, SILT, trace gravel; disturbed texture; organic inclusions (ML).		6 4 10	14	
		Pre-Olympia Non-Glacial				
10	5	Moist, brown and orangish brown with banded oxidation, SILT; massive; bed of dark brown sand near tip of sampler (ML).		5 11 14	25	
12.5		**Pre-Olympia Glacial Diamict**				
15	6	Moist, brown to dark grayish brown, silty, fine SAND ranging to sandy silt, some gravel; unsorted (SM/ML). Grinding drill action at 16.5 feet.		14 21 44	65	
17.5						

Blows/Foot: 10 20 30 40 50+

8/2/2022

20220141E001

associated earth sciences incorporated

Exploration Boring EB-2

Chase Residence		Sheet: 2 of 2
Mercer Island, WA	Start Date: 7/12/22	Logged By: PL
20220141E001	Ending Date: 7/12/22	Approved By: CMM

Driller/Equipment: Geologic Drill Partners/Mini-Track Drill
Hammer Weight/Drop: 140Lbs/30"
Hole Diameter (in): 7 Inches
▼ Groundwater Depth ATD (ft): N/A

Total Depth (ft): 21.5
Ground Surface Elevation (ft): ≈34
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot (10 20 30 40 50+)	Other Tests
20		7		Very moist, gray and orangish brown, silty, fine SAND ranging to fine sand, some silt, some gravel; unsorted; graduational transition; poor recovery (SM/SP-SM).		20 26 37	63	
				No groundwater encountered. Refusal due to gravel. Samples did not react when exposed to diluted hydrochloric acid.				
22.5								
25								
27.5								
30								
32.5								
35								
37.5								

associated
earth sciences
incorporated

Exploration Boring

EB-3

Chase Residence		Sheet: 1 of 2
Mercer Island, WA	Start Date: 7/12/2022	Logged By: PL
20220141E001	Ending Date: 7/12/2022	Approved By: CMM

Driller/Equipment: Geologic Drill Partners/Mini-Track Drill
Hammer Weight/Drop: 140Lbs/30"
Hole Diameter (in): 7 Inches
▼ Groundwater Depth ATD (ft): N/A

Total Depth (ft): 31.5
Ground Surface Elevation (ft): ≈48
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot 10 20 30 40 50+	Other Tests
0		1		**Asphalt - 3 inches** **Crushed Rock - 2 inches** **Holocene Mass-Wastage Deposits** Moist, brown to dark brown, silty, fine SAND, trace gravel; chaotic; disturbed texture; organic inclusions and staining (SM).		5 5 3	8	
2.5		2		Moist, brown to dark brown, sandy, SILT; disturbed texture; organic inclusions and small roots; zones of silty fine sand (ML).		3 3 3	6	
5		3		Moist, dark brownish gray, SILT, trace gravel; disturbed texture; organic inclusions (ML).		3 4 5	9	
7.5		4		Moist, dark brownish gray, SILT; disturbed texture; dark brown organic staining in places (ML).		4 4 4	8	
10		5		Moist, dark gray and brownish gray, SILT; disturbed texture; fractured; some oxidized fractures (ML).		3 3 3	6	
12.5				**Pre-Olympia Glacial Diamict**				
15		6		Moist, dark gray, SILT; massive (ML).		7 6 16	25	
17.5								

8/2/2022

20220141E001

associated earth sciences incorporated

Exploration Boring

EB-3

Chase Residence
Sheet: 2 of 2
Mercer Island, WA
Start Date: 7/12/2022
Logged By: PL
20220141E001
Ending Date: 7/12/2022
Approved By: CMM

Driller/Equipment: Geologic Drill Partners/Mini-Track Drill
Hammer Weight/Drop: 140Lbs/30"
Hole Diameter (in): 7 Inches
▼ Groundwater Depth ATD (ft): N/A
Total Depth (ft): 31.5
Ground Surface Elevation (ft): ≈48
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot (10 20 30 40 50+)	Other Tests
20		7		Top 2 inches: Moist, dark gray, SILT; massive (ML).		21 50/6"	50/6"	
				Lower 10 inches: Moist, brown to dark brownish gray, silty, fine SAND, some gravel; sharp contact (SM).				
22.5								
25		8		Moist, dark gray, SILT; massive with a band of greenish gray silt; occasional dropstones (ML).		8 22 22	44	
27.5								
30		9		Moist, dark gray and olive, SILT, trace gravel; unsorted (ML).		39 29 37	66	
				No groundwater encountered. Samples did not react when exposed to diluted hydrochloric acid.				
32.5								
35								
37.5								