associated earth sciences incorporated

Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report

HOU RESIDENCE
Mercer Island, Washington

Prepared For:
SANG HOU

Project No. 170403E001
July 13, 2017

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Project No. 170403E001

Sang Hou
7022 East Mercer Way
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Subject: Geotechnical Engineering Report
Hou Residence
48XX East Mercer Way
Mercer Island, Washington

Dear Mr. Hou:

Associated Earth Sciences, Inc. (AESI) is pleased to present a copy of our referenced report. This report summarizes the results of our 2007 subsurface exploration, geologic hazard, and geotechnical engineering studies performed for a previous owner and offers geotechnical recommendations for the design and development of the proposed project in regards to the updated building codes.

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

Matthew A. Miller, P.E.
Principal Engineer

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GEOTECHNICAL ENGINEERING REPORT

HOU RESIDENCE

Mercer Island, Washington

Prepared for:
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Mercer Island, Washington 98040

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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our 2007 subsurface exploration, and provides updated geotechnical engineering recommendations for the design and construction of the proposed single-family residence. The location of the project site is shown on the "Vicinity Map," Figure 1. The approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2. If there are any substantial changes in the nature, design, or location of the proposed improvements, 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 update our previous report. Our study included a site visit on July 10, 2017 to the site and updating our geotechnical design recommendations to the current standard and references to the latest codes. We recommend that we be allowed to review project plans prior to construction to verify that our geotechnical recommendations have been correctly interpreted and incorporated into the design.

1.2 Authorization

Written authorization to proceed with this study was provided by the client. This report has been prepared for the exclusive use of the client, 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 property was located at 48XX East Mercer Way in Mercer Island, Washington. The property is approximately 0.53 acre, according to the King County tax records. The property generally slopes from the southwest down to the northeast. There is a steep slope on the western portion of the property sloping down at approximately 1.5H:1V (Horizontal:Vertical). The total elevation change across the property is approximately 70 feet. The property is moderately forested with Douglas fir, hemlock, cedar trees, and deciduous trees, such as alder, vine maple, and cottonwood. There is a small stream that flowed through the northern portion of the property, and the ground surface slopes down to the stream level as well.

This report was completed with an understanding of the project based on a topographic survey and proposed site plan from W&H Pacific, Inc. Present plans call for construction of a single-family residence with grading for a driveway and landscaping.

We visited the site on July 10, 2017 to observe the conditions of the site and compare them to those for which our original study was conducted. Since our previous report was completed no alterations to the site have been made.

3.0 SUBSURFACE EXPLORATION

The site exploration was conducted on March 12, 2007, and consisted of two exploration borings and conducting a geologic and geologic hazard reconnaissance to gain 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 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, extrapolation 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 two borings were completed on the property using a hand-portable drill rig advancing a 3.75-inch-inside-diameter, hollow-stem auger. During the drilling process, samples were obtained at 5-foot intervals. The borings were continuously observed and logged by an engineering geologist from our firm. The exploration logs presented in the Appendix are based on the field logs, drilling action, and inspection of the samples secured.

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 ground water elevations, as well as the depths where soil and ground water characteristics changed, are indicated on the exploration boring logs presented in the Appendix of this report. Our explorations and reconnaissance 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 applicable geologic literature. Our findings are in general agreement with K.G. Troost and A.P. Wisher, 2006, *The Geologic Map of Mercer Island, Washington,* Pacific Northwest Geologic Mapping Project. As shown on the field logs, the borings generally encountered slide debris deposits overlying medium stiff to stiff Lawton clay sediments. The following section presents more detailed subsurface information organized from the upper (youngest) to the lower (oldest) sediment types.

4.1 Stratigraphy

Slide Debris

A loose, moist to wet, brown to gray, silty sand with some orange oxidation was encountered at and near the ground surface of both exploration borings completed for this study. Due to the composition, sample characteristics, and in-place density, this soil was interpreted to be slide debris. The slide debris sediments were deposited in recent to ancient slides that have occurred after the deposition of the Vashon glacial sediments approximately 15,000 years ago. Although deposited from past slides, we did not observe signs of recent ground movement during our site visits.

Lawton Clay - Qvlc

A stiff to hard, clayey silt deposit was encountered in both exploration borings completed for this study and was interpreted to be Lawton clay glaciolacustrine deposits. The Lawton clay sediments were underlying the slide debris deposits at approximately 10 feet and 12.5 feet below the ground surface in exploration borings EB-1 and EB-2, respectively. The glaciolacustrine clayey silt was deposited in freshwater lakes or slow-moving rivers far ahead of the advancing Vashon age glacial ice sheet and also overridden by several thousand feet of ice. These sediments are stiff to hard, have low-compressibility characteristics, and are relatively impermeable. The glaciolacustrine sediments are suitable for direct foundation support or steel pipe pile support.

4.2 Hydrology

Ground water was encountered at approximately 9 feet below the surface in exploration boring EB-1 and at approximately 5 feet below the surface in exploration boring EB-2. The ground water is interpreted to be the local ground water table. Mottling of the upper portions of the deposit indicates that at times, the ground water has been higher than at the time of the exploration. It should be noted that fluctuations in the level of the ground water may occur due to the time of the year, variations in rainfall, and lake levels. The quantity and duration of flow from excavations made into the perched zone will vary, depending on season, topography, and soil grain size.

II. GEOLOGIC HAZARDS AND MITIGATIONS

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

5.0 SLOPE STABILITY ASSESSMENT

The City of Mercer Island geologic hazard maps indicate that the site is located in a steep slope hazard area. Therefore, the hazard must be addressed in the design of the foundation. The site's existing slopes are moderately inclined within the proposed building pad, with a steep slope up to the southwest and along the east edge of the lot. The near-surface soil underlying the site consists primarily of a loose slide debris deposit overlying a stiff to hard, clayey silt glaciolacustrine deposit (Lawton clay). We observed the site for indications of slope instability, such as bowed or tilted trees, naturally occurring terraced topography, tension cracks, reversed drainage gradients, and unvegetated soil exposures. We did not observe surface features that would indicate ongoing slope movement on the site or in the immediate vicinity. However, there was indication of ancient slide debris deposits on the site. Due to the loose nature of the shallow soils on the site, it is our opinion that the mitigations on the site should include the use of a deep foundation or spread footing placed at an elevation below the encountered slide debris.

6.0 SEISMIC HAZARDS AND MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small and are usually not felt by people. However, large earthquakes do occur, as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history. Evaluation of return rates indicates that an earthquake of a magnitude between 6.0 and 7.0 is likely within a given 25- to 40-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 nearest known fault trace to the project is the Seattle Fault located approximately 2.5 miles to the north. Recent studies by the United States Geological Survey (USGS) (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 Hazards,* Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) suggest that a northern trace of the east-west trending Seattle Fault (a thrust fault zone) may show evidence of surficial ground rupture. The recognition of the Seattle 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 intervals for movement along this fault system are still unknown, although they are hypothesized to be in excess of several thousand years. Due to the suspected long recurrence intervals and the distance to these fault zones, 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 stiff to medium stiff subsurface conditions found during the investigation, the field and subsurface observations noted in Section 5.0, and the medium dense characteristics of the native slope to the west of the site, it is our opinion that the risk of seismically induced landslides is low to moderate and in the upper soil sequence. Therefore, as noted previously, we recommend the use of a deep foundation using pipe piles to mitigate the potential risk.

6.3 Liquefaction

The encountered stratigraphy has a low potential for liquefaction due to the grain-size distribution of the native sediments and the density of the underlying glacially consolidated Lawton clay sediments. Therefore, no liquefaction mitigation efforts are needed.

6.4 Ground Motion

Based on the site stratigraphy and visual reconnaissance of the site, it is our opinion that any earthquake damage to the proposed structure, when founded on a suitable bearing stratum, would be caused by the intensity and acceleration associated with the event and not any of the above-discussed impacts. Structural design of the improvements should follow 2015 IBC standards using Site Class “E” as defined in 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 City of Mercer Island erosion hazard maps indicate that the site is located in an erosion hazard area. Therefore, the hazard must be addressed in the development of the site. The primary area of concern for erosion hazards on this property is the steep slope on the southwestern portion of the property. Due to the steepness (approximately 1.5H:1V) and the slope length (over 60 feet), the erosion-related hazard potential is considered to be moderate. It is our opinion that the native vegetation and ground cover on this slope should not be removed or altered.

The other area of erosion potential is the area where the house and driveway are planned to be constructed. Due to the gentler slope conditions in this area, the erosion-related hazard potential is considered to be low, and special mitigation will not be required beyond the implementation of a Temporary Erosion and Sedimentation Control (TESC) Plan. This plan and a Storm Water Pollution Prevention Plan (SWPPP) will more than likely be conditions of the National Pollutant Discharge Elimination System (NPDES) construction permit. TESC requirements vary between the wet season and the dry season. Between November 1st and April 1st, soil that is to be undisturbed for more than 24 hours is typically required to have temporary cover applied. Drainage control also needs to be established onsite to route turbid runoff to sediment traps or a treatment facility, and to prevent turbid runoff from flowing onto adjacent properties or to sensitive receiving waters. To provide temporary cover, straw mulch, plastic sheeting, or erosion control blankets are typically used. When soil needs to be covered for a longer period of time, temporary seeding can be implemented. Construction entrances and heavy construction traffic areas should be stabilized with crushed rock or asphalt treated base (ATB) to mitigate subgrade degradation, sediment tracking, and turbid runoff. Also, earthwork operations may need to be limited or stopped during periods of heavy rainfall and inclement weather. Upon request, Associated Earth Sciences, Inc. (AESI) can recommend which best management practices (BMPs) should be used in the TESC Plan, help field-fit the BMPs selected for maximum effectiveness, and perform field inspections to assess BMP performance and to provide maintenance recommendations. These field inspections may be required by the Washington State Department of Ecology (Ecology) or the City of Mercer Island for TESC compliance. AESI is also available to prepare a turbidity monitoring plan, if required.

8.0 STATEMENT OF RISK

For Section 19.07.020(E) of the *Mercer Island Unified Land Development Code* (ULDC), the City of Mercer Island requires a statement of risk by the geotechnical engineer. It is AESI's opinion that the development practices proposed for the alteration would render the development as safe as if it were not located in a geologic hazard area, provided the recommendations in this report are followed.

III. DESIGN RECOMMENDATIONS

9.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the property is suitable for the proposed development, provided the risks discussed are accepted and the recommendations contained herein are properly followed. The bearing strata of Lawton clay sediments were encountered at a depth of approximately 10 to 12 feet in our explorations and will provide suitable support for steel pipe piles. Conventional spread footing foundations constructed to bear on medium dense to dense native sediment or on approved structural fill soil or rock trenches could be utilized to provide foundation support. However, in consideration of the depth to medium dense sediments and the shallow ground water seepage observed at the time of our exploration and site visits, it is our opinion that overexcavation and site preparation for conventional footings would not be feasible with these field conditions.

10.0 SITE PREPARATION

Site preparation of areas where structural fill is required for future structures or to achieve the desired grades for driveways should include removal of all trees, brush, debris, and any other deleterious material. Where present, the upper organic topsoil should be removed and the remaining roots grubbed. Areas where loose surficial soils exist due to grubbing operations should be recompacted in place, or if this is not feasible due to either soil composition or moisture content, the loose soil should be removed and replaced as subsequently recommended for structural fill placement. We recommend that road and parking areas be proof-rolled with a loaded dump truck or other heavy equipment to identify any soft or yielding areas. Soft areas should be overexcavated and backfilled with structural fill.

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in the loose slide debris sediments may be planned at a maximum slope of 1.5H:1V. Temporary cut slopes in any fill soils encountered should be limited to 1.5H:1V as well. Temporary slopes in Lawton clay sediments should be limited to 1H:1V. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. The slide debris sediments, although prone to caving, may stand near vertical during utility trench construction long enough to install a trench box for all trenching operations over 4 feet deep. In addition, WISHA/OSHA regulations should be followed at all times.

As a standard, permanent slopes in structural fill or cut slopes should not exceed a 2H:1V inclination. Permanent slopes in landscaping fill should be limited to 3H:1V.

The slide debris and Lawton clay sediments encountered in the exploration borings contained a high percentage 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. Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock or ATB. AESI can provide field design recommendations for these areas, if needed.

11.0 STRUCTURAL FILL

Due to the topography of the site and utility installation to be performed, structural fill may be required to establish the desired grades, primarily along the east side of the house where the driveway will be located. 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.

After overexcavation/stripping has been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of 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 stripping and subgrade preparation 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, placed in maximum 8-inch loose lifts with each lift being compacted to 95 percent of the modified Proctor maximum density using: ASTM D-1557 as the standard. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of the perimeter footings or pavement edges before sloping down at an angle of 2H:1V.

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 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. The on-site soils generally contained significant amounts of silt and are considered moisture-sensitive. In addition, construction equipment traversing the site when the soils are wet can cause considerable disturbance. 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.

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.

11.1 Keying and Benching

All structural fill planned to be placed on existing slopes steeper than 20 percent (5H:1V) are required to have a keyway constructed at the toe of the fill body and the slope to be benched prior to placing fill. The keyway should be excavated a minimum of 2 feet down into firm, medium dense to dense, native sediments and be a minimum of 8 feet in width. The width of the benches should be established in the field to fit the contour and gradient of the slope being filled.

12.0 FOUNDATIONS

As previously stated, we recommend the use of steel pipe piles. However, if the grades are adjusted, it may be possible to use conventional spread footings bearing on medium dense sediments or structural fill. However, ground water will be an issue during the construction of conventional spread footings. Recommendations for both types of foundations are included in this section, although it is our opinion that pipe piles are best suited for the conditions encountered.

12.1 Pipe Pile Foundations

Pipe piles consisting of 4- or 6-inch-diameter, driven steel pipe sections will provide suitable support for the proposed residence. The pipe piles should be driven to refusal with equipment appropriate to the pipe diameter. Multiple pipe sections should be joined with compression fittings that fit inside the pipe or welding of the pipe sections. Table 1 summarizes typical wall thicknesses, driving equipment, refusal criteria, and allowable axial compressive loads for each pipe diameter. If higher allowable loads are desired, on-site load testing of at least two piles should be performed to at least 200 percent of the design load to verify that the pile capacities are achievable in the site soils. The load test procedures should be observed by an AESI representative and the test results reviewed by an AESI geotechnical engineer.

Table 1
Pipe Pile Summary

Pile Inside Diameter	Wall Thickness	Typical Installation Equipment	Refusal Criteria* (seconds/inch)	Allowable Axial Compressive Load** (kips)
3-inch	Schedule 40	650 lb jackhammer	20	10
4-inch	Schedule 40	850 lb hammer	15	16
6-inch	Schedule 40	1,250 lb hammer	15	20

* Based on listed installation equipment. Other equipment may alter refusal criteria.
** Allowable loads may be increased with acceptable load testing to twice the design load.

If uplift loads are expected to be placed on the piles at any time, the connections should also be securely welded. It should be noted that the uplift capacity of pipe piles is typically not significant, and is not used for design. Piles may be battered up to 15 degrees to develop lateral capacity. Battered piles inclined up to 15 degrees should be designed with an allowable axial compressive capacity equal to that used for vertical piles. Although vertical pipe piles can provide small uplift and lateral capacities, we recommend that these contributions be neglected in designing the new foundation system. Lateral resistance at the foundation level may be provided by passive resistance, as described in the following section. The structural engineer should provide pile spacing, locations, splicing details, foundation connection details, and any other structural design recommendations that are needed. No minimum pile spacing requirements are necessary for pipe piles from a geotechnical standpoint.

Since pipe piles are driven until specific refusal criteria are achieved, rather than to a specific depth, accurate estimation of pile lengths is not possible. We recommend that AESI be retained to observe pile installation to confirm that our recommendations have been implemented, to verify that appropriate installation procedures are used, and that the appropriate refusal criteria are achieved. The City of Mercer Island may require this inspection as a condition of permitting.

Passive Resistance

Grade beams and pile caps that are backfilled with structural fill may be designed for passive resistance against lateral translation using an equivalent fluid equal to 250 pounds per cubic foot (pcf). The passive resistance value includes a factor of safety equal to 3 in order to reduce the amount of movement necessary to generate passive resistance.

12.2 Spread Footings

Spread footings may be used for building support when founded on stiff to hard Lawton clay native soils found at a depth of about 12 feet, or structural fill placed as previously discussed. To limit differential settlements between footings that bear on both structural fill and medium stiff to stiff native soils, we recommend that an allowable bearing pressure of 2,000 pounds per square foot (psf) be utilized 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 should be buried at least 18 inches into the surrounding soil for frost protection; interior footings require only 12 inches burial. However, all footings must penetrate to the prescribed bearing stratum, and no footing should be founded in or above loose, organic, or existing fill soils.

It should be noted that the area bounded 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 edges of steps or cuts in the bearing soils.

Anticipated settlement of footings founded on medium dense native soils or approved structural fill should be less than 1 inch. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements. Footing subgrades excavated into the recessional sediments should be compacted prior to placing concrete. 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 with 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 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 walls that are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid equal to 40 pcf. Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid of 55 pcf. Walls with sloping backfill at a maximum angle of 2H:1V should be designed for 55 pcf for yielding conditions and 75 pcf for 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.

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of on-site sandy silts 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. Surcharges from adjacent footings, heavy construction equipment, or sloping ground must be added to the above values. It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. Perimeter footing drains should be provided for all retaining walls, as discussed under the "Drainage Considerations" section of this report.

13.1 Passive Resistance and Friction Factors

Retaining wall footings/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 250 pcf. The passive equivalent fluid pressure diagram begins at the top of the footing; however, total lateral resistance should be summed only over the depth of the actual key.

The allowable friction coefficient for footings cast directly on undisturbed dense soils may be taken as 0.30. Since it will be difficult to excavate these soils without disturbance, the soil under the footings must be recompacted to 95 percent of the above-mentioned standard for this value to apply.

14.0 FLOOR SUPPORT

Slab-on-grade floors should be constructed to bear on structural fill or pre-rolled, medium dense, native soil. 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.

15.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. Drains should consist of 6-inch rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel or drain rock. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing and should be constructed with sufficient gradient to allow gravity discharge away from the building. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel blanket provided over the full height of the wall that ties into the footing drain. Roof and surface runoff should not discharge into the footing 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.

16.0 ROCKERIES

The original project plans show rockeries that form grade separations between the subject lot and the lot to the south at the edge of the driveway about 6 feet high. Rockeries may be used to prevent erosion and face cut slopes. Rockeries that face fill slopes that are greater than 4 feet in height should not be used in place of retaining walls unless the backfill soil is reinforced, especially where structures and roadways are adjacent to them. In any case, structures should be set back from rockeries so that a 1H:1V line extending up from the rear base of the rockery does not intersect the foundation.

The following notes present rockery considerations and should be used in conjunction with Figure 3:

A) The base of the rockery should be started by excavating a trench to a minimum depth of 12 inches below subgrade into firm, undisturbed ground. If loose, soft, or disturbed materials exist at the base rock location, they should be removed and replaced with free-draining sand and gravel, or crushed rock. This backfill material should be placed as noted in the "structural fill" section of this report.

B) The base rock should have a minimum width (perpendicular to the line of the rockery) of 40 percent of the height of the rockery. All rocks should also meet the following weight requirements:

Height of Rockery	Minimum Weight of Rock
Above 5 feet	500/2,200 pounds, graded, top/bottom rocks
5 feet or less	500/1,000 pounds, graded, top/bottom rocks

C) The rock material should all be as nearly rectangular as possible. No stone should be used that does not extend through the wall. The rock material should be hard, sound, durable, and free from weathered portions, seams, cracks, or other defects. The rock density should be a minimum of 160 pcf.

D) Rock selection and placement should be such that there will be minimum voids and, in the exposed face of the wall, no open voids over 8 inches across in any direction. The rocks should be placed in a manner such that the longitudinal axis of the rock will be at right angles or perpendicular to the rockery face. Each rock should be placed so as to lock into two rocks in the lower tier. After setting each rock course, all voids between the rocks should be chinked on the back with quarry rock to eliminate any void sufficient to pass a 2-inch square probe. Rockeries should be limited to 10 feet in height facing the lodgement till and advance outwash soils.

E) A drain consisting of rigid, perforated PVC pipe enclosed in a 12-inch-wide, pea-gravel trench should be placed behind the lower course of rock to remove water and prevent the buildup of hydrostatic pressure behind the wall. The remainder of the wall backfill should consist of quarry spalls with a maximum size of 4 inches and a minimum size of 2 inches. This material should be placed to a 12-inch minimum thickness between the entire wall and the cut material. The backfill material should be placed in lifts to an elevation approximately 6 inches below the top of each course of rocks as they are placed until the uppermost course is placed. Any backfill material falling onto the bearing surface of a rock course should be removed before the setting of the next course.

F) Any asphalt paving should be sloped to drain away from the rockery. In addition, the areas above rockeries should be planted with grass as soon as possible after rockery construction to reduce erosion.

G) Fill faced with rockeries greater than 4 feet in height should include the use of geogrid reinforcement. The reinforcement should extend from the back of the rock into the fill at least a distance equal to the height of the rockery.

17.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

At the time of this report, site grading, structural plans, and construction methods have not been finalized. 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 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 foundation depends on proper site preparation and construction procedures. These inspections may be required by the City of Mercer Island as a part of the building permit conditions. 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 cost 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.

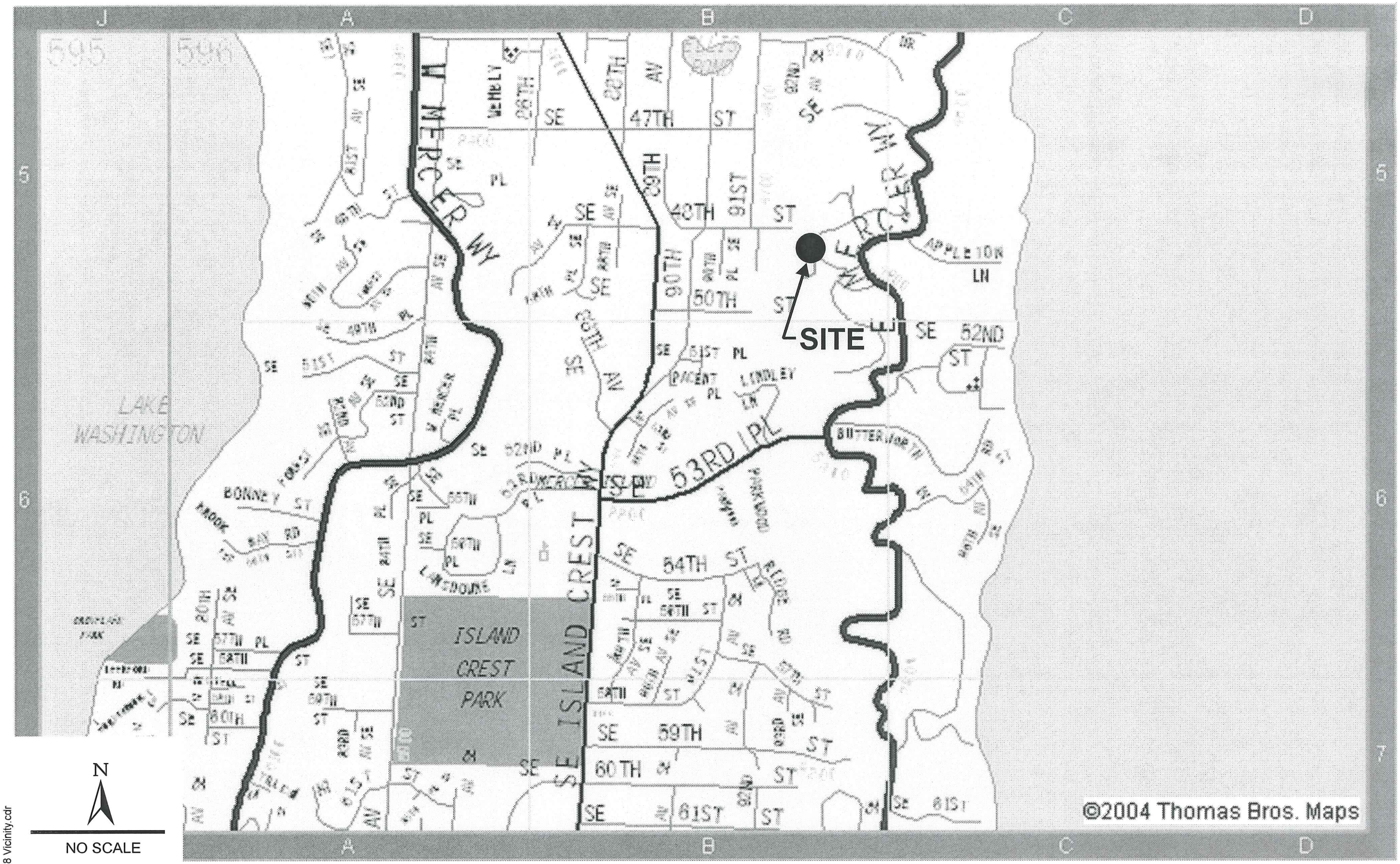
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

MATTHEW A. MILLER
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

7-13-07

Matthew A. Miller, P.E.
Principal Engineer

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Site and Exploration Plan
	Figure 3:	Typical Rockery
	Appendix:	Exploration Logs



070068 Skall Residence \ 070068 Vicinity.cdr

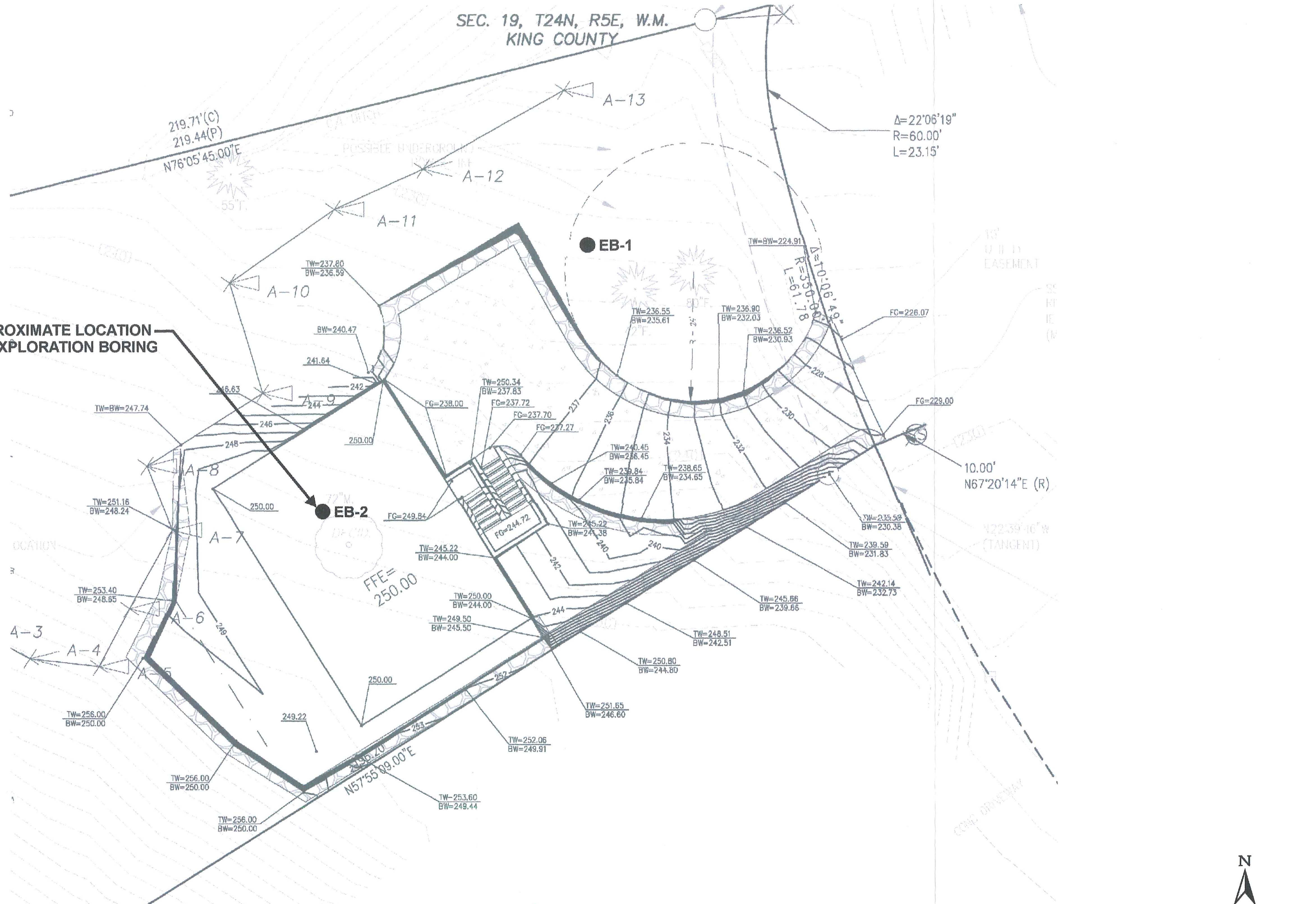
Associated Earth Sciences, Inc.

VICINITY MAP
SKALL RESIDENCE
MERCER ISLAND, WASHINGTON

FIGURE 1

DATE 4/07

PROJ. NO. KE070068A



Reference: W AND H PACIFIC

070068 Skall Residence \ 070068 Site and Explr.cdr

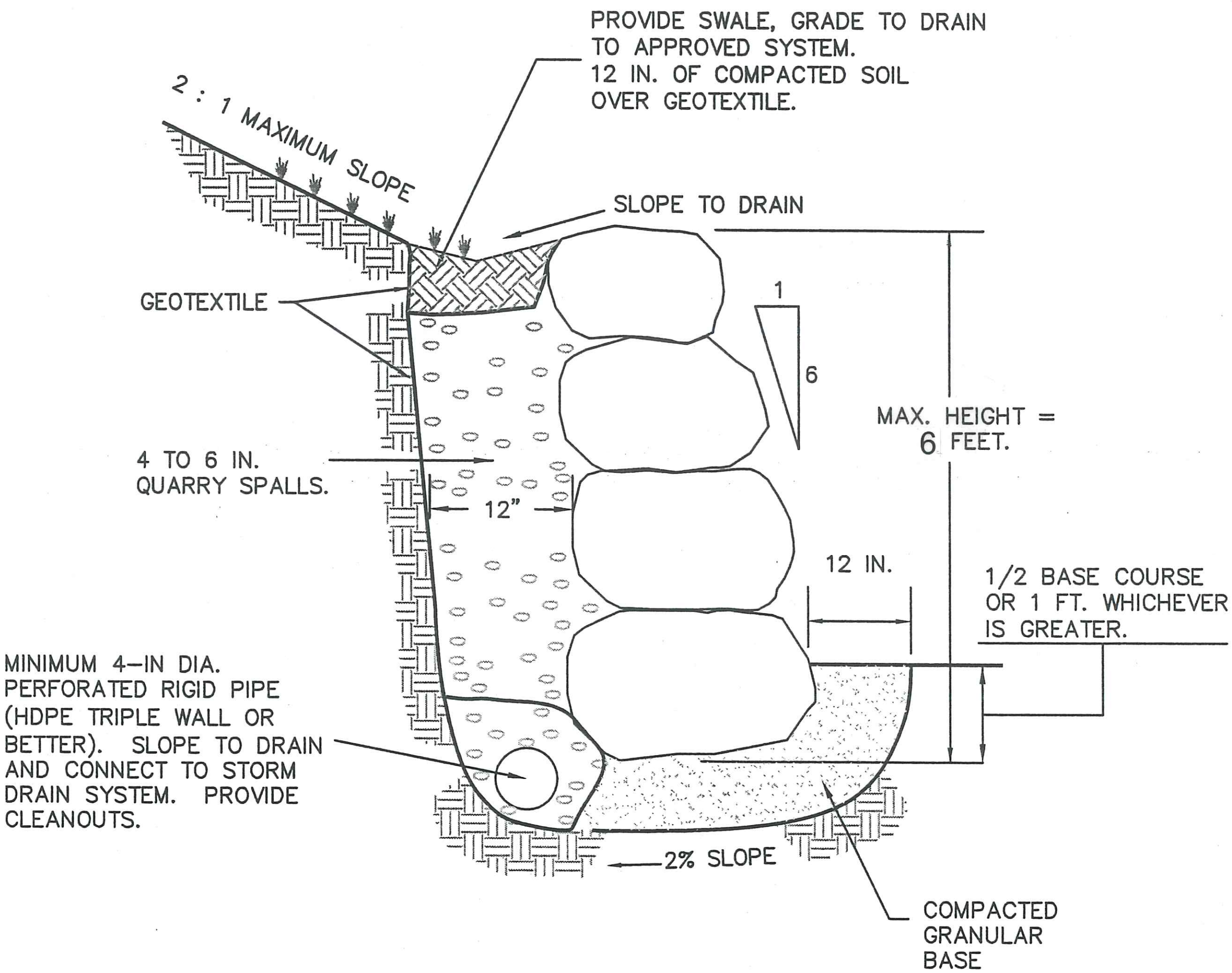
Associated Earth Sciences, Inc.

SITE AND EXPLORATION PLAN
SKALL RESIDENCE
MERCER ISLAND, WASHINGTON

FIGURE 2

DATE 4/07

PROJ. NO. KE07006A



NOTES:

1. CALL FOR CLEAR AND GRADE INSPECTION PRIOR TO BASE COURSE BEING PLACED. VERIFICATION OF ROCKERY HEIGHT, FOUNDATION MATERIAL, AND ROCK SIZE BY CITY CLEARING AND GRADING INSPECTOR IS REQUIRED.
2. QUARRY SPALLS AND CRUSHED ROCK SHALL BE PLACED DIRECTLY FROM TRUCK OR OTHER SUITABLE CONTAINER IN ORDER TO MAINTAIN CLEAN BACKFILL.
3. ALL ROCKERIES OVER 30-IN IN HEIGHT ARE CONSIDERED A STRUCTURE PER THE LAND USE CODE AND MUST BE LOCATED OUTSIDE OF ALL STRUCTURE SETBACK LINES.
4. OPENINGS SHALL BE CHINKED WITH QUARRY SPALLS.
5. ROCKERIES GREATER THAN 4-FEET IN HEIGHT SHALL BE DESIGNED BY A CIVIL OR GEOTECHNICAL ENGINEER.
6. ROCKERIES CONSTRUCTED IN THE RIGHT-OF-WAY ARE SUBJECT TO TRANSPORTATION DEPARTMENT DEVELOPMENT STANDARDS.

Reference: City of Bellevue

blocks-city of belle gs\ec-17-aesi.dwg

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TYPICAL ROCKERY SECTION

48XX East Mercer Way
Mercer Island, WA

FIGURE 3

DATE 4-6-07

PROJECT NO. KE 070068A

APPENDIX

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Exploration Log

Project Number	Exploration Number	Sheet
KE070068A	EB-1	1 of 1

Project Name	Skall Residence	Ground Surface Elevation (ft)	
Location	Mercer Island, WA	Datum	N/A
Driller/Equipment	CN Drilling/Acker HSA	Date Start/Finish	3/12/07,3/12/07
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-1		Moist, brown, sandy SILT, with wood and organic debris. **Slide Debris**			1 2 5	▲7	
5		S-2		Drilling harder at 4 feet. Wet, light brown, sandy SILT, with some orange mottling. Cuttings very wet at 6 feet. Drove another sampler immediately after.			3 5 5	▲10	
10		S-3		**Lawton Clay** Moist to wet, gray, SILT, with fine sand. Cuttings saturated.			3 4 6	▲10	
15		S-4		Moist to wet, gray, clayey SILT.			3 6 10	▲16	
20		S-5		Sampler was low in hole by about one foot. Moist to wet, gray, clayey SILT.			11 13 13	▲26	
25		S-6		3 "counts" added - dropped the rod down auger. Moist, gray, clayey SILT, with some brown mottling, coarser sand near top of sampler.			6 11 15	▲26	
		S-7		Moist, gray, clayey SILT.			8 13 27	▲40	
30				Bottom of exploration boring at 29 feet Hole left open - filled with water to approximately 27 inches below ground surface.					
35									

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: ALG
Approved by: JNS

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE070068A	EB-2	1 of 1

Project Name	Skall Residence	Ground Surface Elevation (ft)	
Location	Mercer Island, WA	Datum	N/A
Driller/Equipment	CN Drilling/Acker HSA	Date Start/Finish	3/12/07,3/12/07
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-1		Moist, tan-gray, silty SAND and some black-brown topsoil. **Slide Debris**			1 3 3	6	
5		S-2		Moist to wet, brown to gray, SAND, with silt and gravel with some orange oxidation.			5 8 5	13	
10		S-3		Moist to wet, gray, clayey SILT.			4 4 6	10	
				Lawton Clay					
15		S-4		Moist, gray, clayey SILT.			8 12 19	31	
		S-5		Moist to wet, gray, clayey SILT (drove sampler 2 feet).			15 16 16 20	36	
				Bottom of exploration boring at 18 feet Exploration terminated due to refusal.					
20									
25									
30									
35									

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: ALG
Approved by: JNS