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March 7, 2023

JN 23030

Ryan and Ashley Asdourian
5300 Butterworth Road
Mercer Island, Washington 98040
via email: rasdo@microsoft.com and ashleyk@microsoft.com

Subject: **Transmittal Letter – Geotechnical Engineering Study and Critical Area Study**
Proposed New Residence
5300 Butterworth Road
Mercer Island, Washington

Dear Mr. and Mrs. Asdourian:

Attached to this transmittal letter is our geotechnical engineering report and Critical Area Study related to geologic hazards for your proposed new development. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork, stormwater infiltration considerations, and design considerations for foundations, retaining walls, subsurface drainage, and temporary excavations. This work was authorized by your acceptance of our proposal, P-11314.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Marc R. McGinnis, P.E.
Principal Engineer

cc: **Sturman Architects** – Brad Sturman
via email: brad@sturmanarchitects.com

MRM:kg

GEOTECHNICAL ENGINEERING STUDY AND CRITICAL AREA STUDY
Proposed New Residence
5300 Butterworth Road
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study and Critical Area Study for the proposed new residence to be constructed on the subject property in Mercer Island. The scope of the Critical Area Study is intended to satisfy the requirements of section 19.07.110 of the Mercer Island City Code (MICC), which applies to Critical Area Studies.

From our discussions with Sturman Architects, we understand that the existing residence will be replaced with a new, larger home. The new house will have two floors, and will not include a basement. The residence will be located in approximately the same portion of the property as the current house, but may extend further to the east, toward Lake Washington. We expect that the lower floor will be near the existing site grade. Access will continue to be from the existing driveway that extends into the site from Butterworth Road.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site. The irregularly-shaped subject site extends east from Butterworth Road, with a narrow “leg” containing the driveway. To the east of the site is an unaddressed tract that extends at least 75 feet to the shore of Lake Washington. This tract appears to be for lakefront access for the neighborhood, with a narrow footpath extending between Butterworth Road and the tract along the north edge of the site. This footpath appears to be located within a walkway easement.

The existing residence is centrally located on the site. There is a detached garage located in the southwestern portion of the lot.

The subject site slopes gently downwards from west to east from Butterworth Road through the adjacent eastern tract to the shore of Lake Washington. There are no steep slopes on, or near, the site. West of Butterworth Road, the ground slopes moderately upward to East Mercer Way.

Residential properties containing single-family residences border the site to the north, west, and south. The adjacent northern house is approximately 10 feet from the common property line, but the south house is approximately 20 feet from the property line.

The City of Mercer Island’s GIS tool maps the subject site within two geologic hazard areas. The entirety of the site is mapped within both a seismic hazard area and a potential landslide hazard area. This mapping is the same for the adjacent north and south properties.

As discussed above, the subject site is very gently sloped. The planned development area is over 150 feet from the bottom of the moderately- to steeply-sloped properties located west of Butterworth Road. We did not observe any indications of recent slope instability on or around the site during our recent visit to the property. The site lies on a delta of sediments deposited over the last approximately 10,000 years by water flowing down the various ravines that extend from the upland areas of Mercer Island. This delta of loose sediments was subsequently exposed when the level of Lake Washington was lowered in 1916 by the opening of the Montlake Cut. On the Mercer Island *Landslide Hazard Map* (Troost and Wisher, 2009) there are no documented landslides close to the subject property. However, as would be expected, there have been documented episodes of slope movement on steeper areas closer to East Mercer Way. That said, we do not know of any documented episodes of deep-seated landslides in the site vicinity.

SUBSURFACE

The subsurface conditions were explored by drilling two test borings at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The test borings were drilled on March 2, 2023, using a track-mounted hollow-stem auger drill. Samples were taken at approximate 2.5- or 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 and 4.

Soil Conditions

The subsurface explorations conducted on the site generally encountered loose alluvial soils (soils deposited by moving water such as streams) consisting predominantly of peat, sand and silty sand beneath the ground surface. These loose to medium-dense, unconsolidated soils extended to depths of approximately 13 feet and 18 feet on the east (Boring 1) and west (Boring 2)

Beneath these depths, the borings found glacially-compressed soils consisting of dense to very dense sand and silty sand with varying amounts of gravel.

Groundwater Conditions

All of the borings found groundwater below a depth of 4 to 6 feet. The borings were conducted in early spring, when groundwater levels should be near their seasonal highest. The previous fall and winter had been very wet.

Groundwater levels often fluctuate with rainfall and other factors.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. If a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the test boring logs are interpretive descriptions based on the conditions observed during drilling.

SEISMIC CONSIDERATIONS

In accordance with the 2018 International Building Code (IBC) (ASCE 7-16), the site class within 100 feet of the ground surface is best represented by Site Class Type F (Failure-Prone Site Class). However, the code allows for an exception from the F classification if the building period is less than 0.5 seconds. We anticipate the proposed residence will have a structural period of less than 0.5 seconds, and therefore, based on the SPT blowcounts from the borings, a Site Class Type D (stiff soil profile) can be used for the project. This will need to be confirmed by the project structural engineer. As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.44g and 0.50g, respectively.

The near-surface soils beneath the site consisted of saturated silty sand, sand, and silt containing various amounts of organics. Beneath the water table, these soils have a moderate to high potential for liquefaction (soil strength loss) during a large earthquake. The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period).

The glacially-compressed soils found below a depth of 13 to 18 feet are glacially compressed and are not susceptible to liquefaction in the event of the MCE.

Mitigation of potential hazards associated with seismic liquefaction and lateral spreading are discussed below in the Seismic Hazard Area section.

CRITICAL AREAS STUDY (MICC 19.07)

Potential Landslide Hazard Area: The entire subject site is located within a mapped Potential Landslide Hazard area. The site is gently sloped and the planned development is set far from any moderate or steep slopes that could experience landslides. As such, the potential for slope instability on the site or affecting the planned development is negligible. We observed no signs of landslide debris in our borings. Consistent with many lots in this area along the shore of Lake Washington, the topography is the result of alluvial deposits and sediments from Lake Washington, which covered the subject property until 1916. To our knowledge, no recent large-scale movement has been documented in this area.

The planned development is located over 150 feet any significant slopes across Butterworth Road that could be prone to instability. This setback is more than sufficient to protect the planned development from any future instability on these distant slopes. No additional measures, such as buffers or landslide catchment walls, are needed. The proposed development will not adversely impact the stability of the moderate to steep slopes to the west of Butterworth Road.

Seismic Hazard Area: The entire subject site is located within a mapped Seismic Hazard area.

> Liquefaction: The proposed development will be supported on deep foundations embedded into glacially compressed soils that are not liquefiable, due to their dense nature. However, the loose soils between the water table and the glacially-compressed soils are susceptible to liquefaction in the event of a large earthquake. The depth and lateral extent over which liquefaction could occur are impossible to accurately predict, due to unknowns related to the magnitude, duration, and predominant direction of shaking associated with future earthquakes, as well as variabilities in the soil composition.

From previous experience, as well as liquefaction analyses we have conducted for this project, we know that it at least partial liquefaction beneath the site and surrounding area is possible during the Maximum Considered Earthquake (MCE) with a 1-in-2,475-year probability. This liquefaction could occur between the groundwater table (4- to 6-foot depth) and the dense soils, which were found at an approximate depth of 13 to 18 feet.

We have utilized NovoLIQ to confirm that liquefaction of the soil underlying the water table is likely to occur in the MCE, which is a low probability event. NovoLIQ estimates that a total of approximately 4 inches of ground settlement is possible following widespread liquefaction extending to a depth of 18 feet.

The compressive capacity of pipe piles is entirely dependent on end bearing in the dense to very dense glacially-compressed soils they are driven into. The potentially liquefiable soils encountered in the borings below the water table will provide no vertical support to the pipe piles in the event of seismic liquefaction. For a 4-inch-diameter pipe with an allowable 10-ton allowable capacity, an ultimate capacity in excess of 20 tons is achievable in static conditions. This has been verified by thousands of load tests conducted in the Seattle area over the last 20 years. Conservatively assuming a skin friction of 200 psf on the pile in the upper approximately 4 to 6 feet of non-liquefiable soils, a downdrag load of approximately 1,500 pounds could be applied to the pile. This would allow a residual ultimate compressive capacity of at least 38,500 (19.3 tons). For this short-term loading condition, that would still provide a safety factor in excess of 1.9, which is acceptable for a full-scale seismic event.

Lateral Spreading: The potential for lateral spreading during a large earthquake, which is essentially a flow slide of the liquefied soil toward a free face (sloped bottom of Lake Washington), is even less understood than liquefaction itself. However, some very inexact methods have been developed to estimate the potential amount of lateral ground movement that could occur where liquefiable sites lie next to sloping free face conditions, such as the sloped bottom of Lake Washington. NovoLIQ utilizes several different methods to develop estimates for this lateral movement using five different methods. Based on these analyses, lateral ground movement of at least 5 to 10 feet could theoretically occur in the MCE. Having completed similar computations before by hand, we know that large values such as this are common for lakefront projects with more than a few feet of liquefiable soil beneath them.

Unfortunately, as with liquefaction, there is no accurate method for determining where, and to what extent, lateral spreading could occur. Even more involved methods, such as Finite Element Analyses, are still approximate at best, as they rely on a multitude of assumptions about soil properties and potential characteristics of the design earthquake.

Based on the available information, it is theoretically possible that significant lateral ground movement could occur during the MCE. The risk of this is no higher than on nearby waterfront properties that are underlain by similar loose soils and which have recently been developed with new homes. The theoretical lateral movements are large enough, and could extend to such a significant depth, that no pile system, drilled or driven, can prevent ground movement from occurring, or can withstand the potential lateral movements without shearing off.

Improving the ground beneath the site to prevent liquefaction and/or lateral spreading is infeasible for a waterfront residential site within a large area of potentially liquefiable soils, such as this one. Improving the resistance of the granular soils to liquefaction using stone columns, densification, or a similar method would involve strong ground vibrations, which would cause ground settlement and likely damage to neighboring properties, structures, and utilities. No localized ground improvement system on an isolated residential lot can resist the significant lateral soil loads that would result from liquefaction and lateral spreading of the upper approximately 20 feet of soil that could affect a large area including both the site and adjacent properties. It would be necessary to prevent liquefaction and lateral spreading in the loose soils extending far onto neighboring properties to the north, south, and west to prevent lateral movement within the house footprint on the subject site, which is not practical.

Mitigation against the life/safety hazard posed by potential foundation collapse in the event of lateral spreading can be achieved by the reinforced grade beams or mat slab that interconnects the piles. In the event that the ground moves sideways a sufficient distance to bend or break the piles, the grade beams/mat slab would serve to hold the structure in one piece, even if it tilts a significant amount.

Erosion Hazard: Due to the site's very gentle topography, it is not mapped as an Erosion Hazard area. Regardless, properly installed and maintained temporary erosion control measures will be a part of the planned construction. This is necessary to avoid adverse impacts to adjoining properties and to prevent silty runoff from flowing into Lake Washington.

Buffers and Mitigation: As noted above, the entire site lies within a mapped Potential Landslide Hazard Area. However, excluding lateral spreading, the potential for a landslide affecting the planned development is negligible. As a result, a buffer or other forms of mitigation are not necessary to protect the planned development from potential landslides. The recommendations presented in this report and this addendum letter are intended to allow the project to be constructed in the proposed configuration without adverse impacts to critical areas on the site or the neighboring properties. The geotechnical recommendations associated with foundations will mitigate any potential hazards associated with the Seismic Hazard.

Statement of Risk: In order to satisfy the City of Mercer Island's requirements, a statement of risk is needed. By following the recommendations of this report:

The geologic hazard area will be modified, or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site (development) is determined to be safe.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test borings conducted for this study encountered loose alluvial soils consisting predominantly of silt with interbedded layers of sand, peat, and silt sand beneath the ground surface across the subject site. Competent dense to very dense sands were revealed below depths of 13 to 18 feet.

Conventional shallow foundations constructed on the loose, moderately-compressible, alluvial soils beneath the ground surface would experience significant post-construction settlement as the loose soils consolidate over time. This is especially true for the highly-compressible organic soils found close to the ground surface. Considering this, we recommend the proposed house be supported on small-diameter pipe piles driven into the dense to very dense underlying sands. This is a typical foundation system that has been used for newer homes in the area. We recommend floor slabs, including garage slabs, and other settlement-sensitive elements, such as decks, patios, porches, etc. be supported on driven pipe piles as well to prevent differential settlement between them and the pile-supported residence. Even on-grade mechanical units, such as HVAC units and generators will settle relative to the house, if they are not carried on pipe piles. The utilities beneath the house should be hung from the grade beams that will interconnect the piles. Where the driveway meets the garage, it would be appropriate to use a reinforced concrete approach apron dowelled into the pile-supported foundation. This will allow the apron to settle without creating a downset at the garage door threshold.

We understand that the proposed building will be constructed near the existing ground surface. Short temporary open cut slopes in the onsite soils <u>above the water table</u> should be inclined no steeper than 1:5:1 (Horizontal:Vertical) from top to bottom. It appears excavations for the proposed development will be feasible using open cut slopes within the property. Excavation below the groundwater would require extensive excavation shoring and dewatering, and should be avoided.

The site soils that will be excavated have a low compacted strength and very poor drainage characteristics. We recommend against reusing the onsite soils for any wall backfill, or structural fill that will support on-grade elements, even the driveway or entryways. As a result, we expect that excavated soils will be hauled off the site, and imported granular well-draining material will be needed for structural fill for the project.

Considering the soft condition of the upper soils, it will be necessary to provide a pad of clean crushed rock, such as quarry spalls or railroad ballast rock, to establish a working surface for installation of pipe piles and construction of the foundations.

As previously discussed, the test borings encountered loose alluvial soils consisting predominantly of relatively peat and silty soils beneath the ground surface. Furthermore, groundwater was encountered approximately 4 to 6 feet beneath the below grade. Considering this, it is our opinion that onsite stormwater infiltration will be infeasible on the subject site from a geotechnical standpoint.

Projects involving small-diameter pipe piles often include the need for lateral resistance from fill placed against the foundations. If this is the case for this project, it is important that the structural engineer indicate this requirement on the plans for the general and earthwork contractor's information. Compaction requirements for this fill are discussed below in ***Pipe Piles***. The building department may require that we verify suitable compaction of this fill prior to completion of the project.

It is likely that some settlement of the ground surrounding pile-supported buildings will occur over time. In order to reduce the potential problems associated with this, we recommend the following:

- Fill to the desired site grades several months prior to constructing on-grade slabs, walkways, and pavements around the buildings. This allows the underlying soils to undergo some consolidation under the new soil loads before final grading is accomplished.

- Connect all in-ground utilities beneath the house floor slabs to the pile-supported floors or grade beams. This is intended to prevent utilities, such as sewers, from being pulled out of the floor as the underlying soils settle away from the slab. Hangers or straps can be poured into the floors and grade beams to carry the piping. The spacing of these supporting elements will depend on the distance that the pipe material can span unsupported.

- Construct all entrance walkways as reinforced slabs that are doweled into the grade beam at the door thresholds. This will allow the walkways to ramp down and away from the building as they settle, without causing a downset at the threshold. This recommendation includes the approach apron for the garage.

- Isolate on-grade elements, such as walkways or pavements, from pile-supported foundations and columns to allow differential movement.

While the site is not located in a mapped Erosion Hazard area, appropriate temporary erosion control measures will need to be implemented to prevent silty runoff from leaving the property. We have worked on numerous waterfront projects on Mercer Island that have avoided siltation of the lake and surrounding properties by exercising care and being proactive with the maintenance and potential upgrading of the erosion control system through the entire construction process. The temporary erosion control measures needed during the site development will depend heavily on the weather and groundwater conditions that are encountered during the site work. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil areas to prevent accumulated water or runoff from the work area from becoming silty in the first place. Silty water cannot be discharged to the lake, so a temporary holding tank should be planned for wet weather earthwork. A wire-backed silt fence bedded in compost, not native soil or sand, should be erected as close as possible to the planned work area, and the existing vegetation should be left in place wherever possible outside of the silt fence. Rocked construction access and staging areas should be established wherever trucks will have to drive off of pavement, in order reduce the amount of soil or mud carried off the property by trucks and equipment. It will also be important to cap any existing drain lines found running toward the lake until excavation is completed. This will reduce the potential for silty water finding an old pipe and flowing into the lake. Covering the base of the excavation with a layer of clean gravel or rock is also prudent to reduce the amount of mud and silty water generated. Utilities reaching between the house and the lake should not be installed during rainy weather, and any disturbed area caused by the utility installation should be minimized by using small equipment. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. Wet weather construction (October 1 through March 31) on this site should be possible without adverse impacts to the surrounding properties. In preventing erosion control problems on any site, it is most important that any disturbed soil areas be immediately protected. This requires diligence and frequent communication on the part of the general contractor and earthwork subcontractor. As with all construction projects undertaken during potentially wet conditions, it is important that the contractor's on-site personnel are familiar with erosion control measures and that they monitor their performance on a regular basis. It is also appropriate for them to take immediate action to correct any erosion control problems that may develop, without waiting for input from the geotechnical engineer or representatives of the City.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking, cleaning, and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

PIPE PILES

Four-, or 6-inch-diameter pipe piles driven with 1,100-, 2,000-pound, or 3,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (1,100-pound hammer)	FINAL DRIVING RATE (2,000-pound hammer)	FINAL DRIVING RATE (3,000-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
4 inches	10 sec/inch	4 sec/inch	n/a	10 tons
6 inches	20 sec/inch	10 sec/inch	6 sec/inch	15 tons

Note: The refusal criteria indicated in the above table are valid only for pipe piles that are installed using a hydraulic impact hammer carried on leads that allow the hammer to sit on the top of the pile during driving. If the piles are installed by alternative methods, such as a vibratory hammer or a hammer that is hard-mounted to the installation machine, numerous load tests to 200 percent of the design capacity would be necessary to substantiate the allowable pile load. The appropriate number of load tests would need to be determined at the time the contractor and installation method are chosen.

As a minimum, Schedule 40 pipe should be used. Organic peat soils were encountered beneath the subject site; therefore, due to an elevated corrosion potential, it is our opinion that corrosion protection, such as galvanizing, be used for at least the uppermost 20 feet of the pipe piles.

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

It is impossible to accurately estimate the potential driven pile lengths needed to achieve refusal. We have found that the piles will penetrate into dense to very dense sandy soils a greater-than-anticipated distance. Accurately estimating potential pile depths would require installing test piles with equipment similar to what would be used during production pile driving.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. Due to the compressible nature of the upper soils, imported granular soil should be compacted against the foundations to provide lateral load resistance. In order to account for potential ground settlement over time, the passive resistance from the uppermost 6 inches of compacted fill should be neglected. We recommend using an ultimate (no safety factor included) passive earth pressure of 250 pounds per cubic foot (pcf) for this resistance.

FOUNDATION AND RETAINING WALLS

No significant retaining walls are expected for this project, and walls taller than a few feet should be avoided. Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	45 pcf
Passive Earth Pressure	250 pcf
Soil Unit Weight	130 pcf

Where: pcf is Pounds per Cubic Foot, and Active and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure. This applies only to walls with level backfill.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. The existing site retaining wall north of the proposed residence and covered walkway will likely place a surcharge onto the proposed structures' northern foundation walls. We can provide appropriate surcharge loads once more detailed plans have been developed. It may be possible for the excavation shoring to be designed to withstand this surcharge. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired.

The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The on-site soils are not free-draining, and should not be reused as wall backfill.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls at one to 2 percent to reduce the potential for surface water to percolate into the backfill.

Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. Foundation drainage and waterproofing systems are not intended to handle large volumes of infiltrated water. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The recommended wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a buildup of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

The ***General***, ***Building Floors***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

BUILDING FLOORS

As discussed in the ***General*** section, we recommend the floors of the residence and garage, as well as any other settlement sensitive-elements, be designed to span between span between the pipe pile supported foundations, using either with framed floors over crawlspaces.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI recommends a minimum 10-mil thickness vapor retarder for better durability and long term performance than is provided by 6-mil plastic sheeting that has historically been used. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection.

If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

We recommend that the contractor, the project materials engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material.

The ***General***, ***Foundation and Retaining Walls***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

DRAINAGE CONSIDERATIONS

Footing drains should be used where: (1) crawl spaces or basements will be below a structure; (2) a slab is below the outside grade; or, (3) the outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock that is encircled with non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space. The discharge pipe for subsurface drains should be sloped for flow to the outlet point. Roof and surface water drains must not discharge into the foundation drain system. A typical footing drain detail is attached to this report as Plate 5. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

As a minimum, a vapor retarder, as defined in the ***Building Floors*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing a few inches of free draining gravel underneath the vapor retarder is also prudent to limit the potential for seepage to build up on top of the vapor retarder.

Shallow groundwater was observed during our field work. As discussed above, excavations below the water table would require extensive dewatering and shoring, and should be avoided. Pumping a substantial amount of groundwater could cause ground settlement on adjacent properties, if the drawdown influence zone from the dewatering extends outside of the property boundaries.

Final site grading in areas adjacent to a building should slope away at least one to 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process. As discussed in the ***General*** section, the on-site soils are not suitable for reuse as structural fill, due to their high fines content and moisture sensitivity.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches, but should be thinner if small, hand-operated compactors are used. We recommend testing structural fill as it is placed. If the fill is not sufficiently compacted, it should be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended levels of relative compaction for compacted fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath footings, slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the subsurface explorations are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Ryan and Ashley Asdourian and their representatives, for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

In addition to reviewing the final plans, Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 and 4	Test Boring Logs
Plate 5	Typical Footing Drain Detail

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

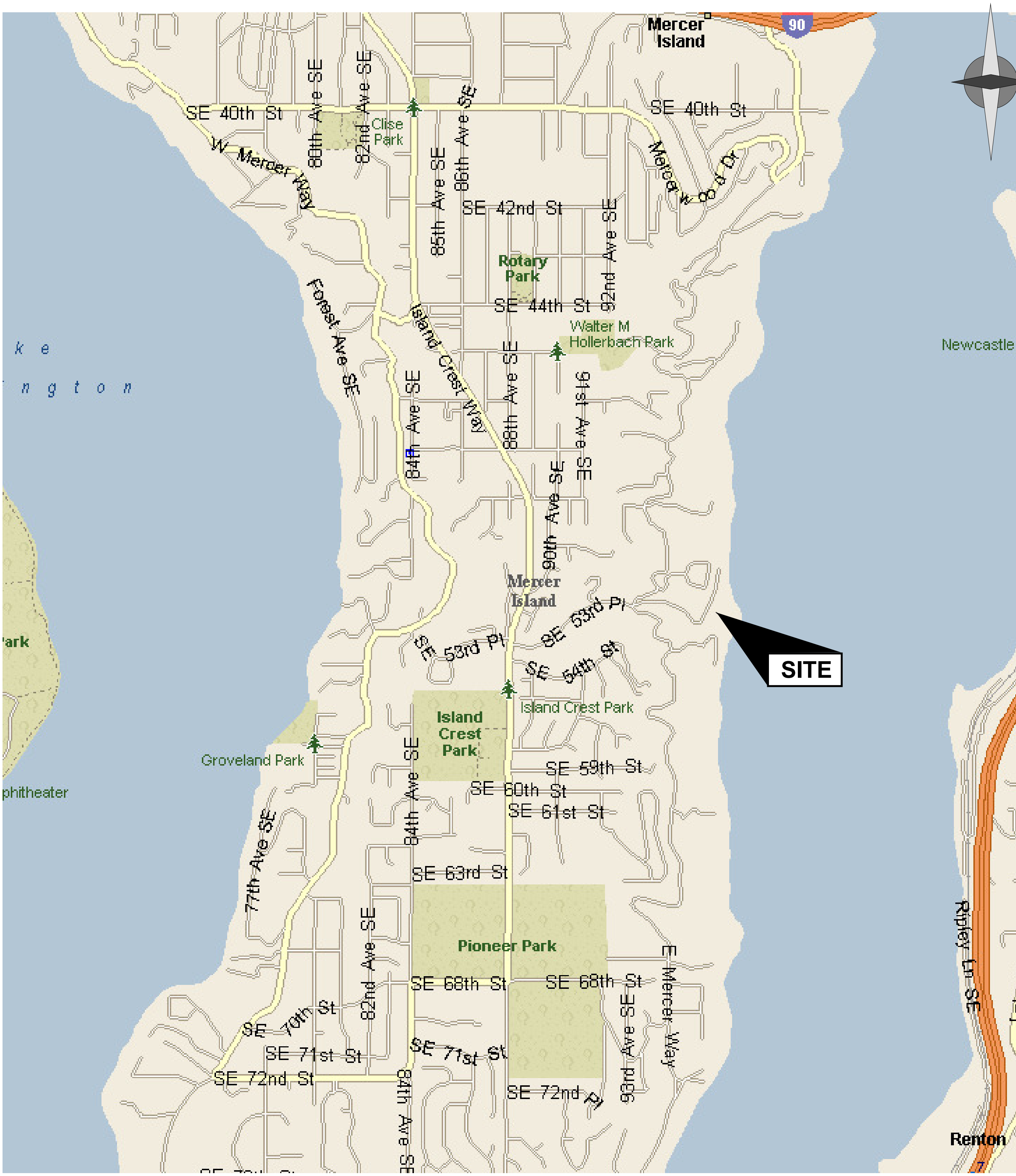
GEOTECH CONSULTANTS, INC.

3/07/2023

Marc R. McGinnis, P.E.
Principal

MRM:kg

NORTH



(Source: Microsoft MapPoint, 2013)

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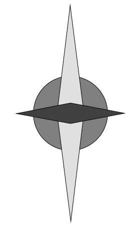
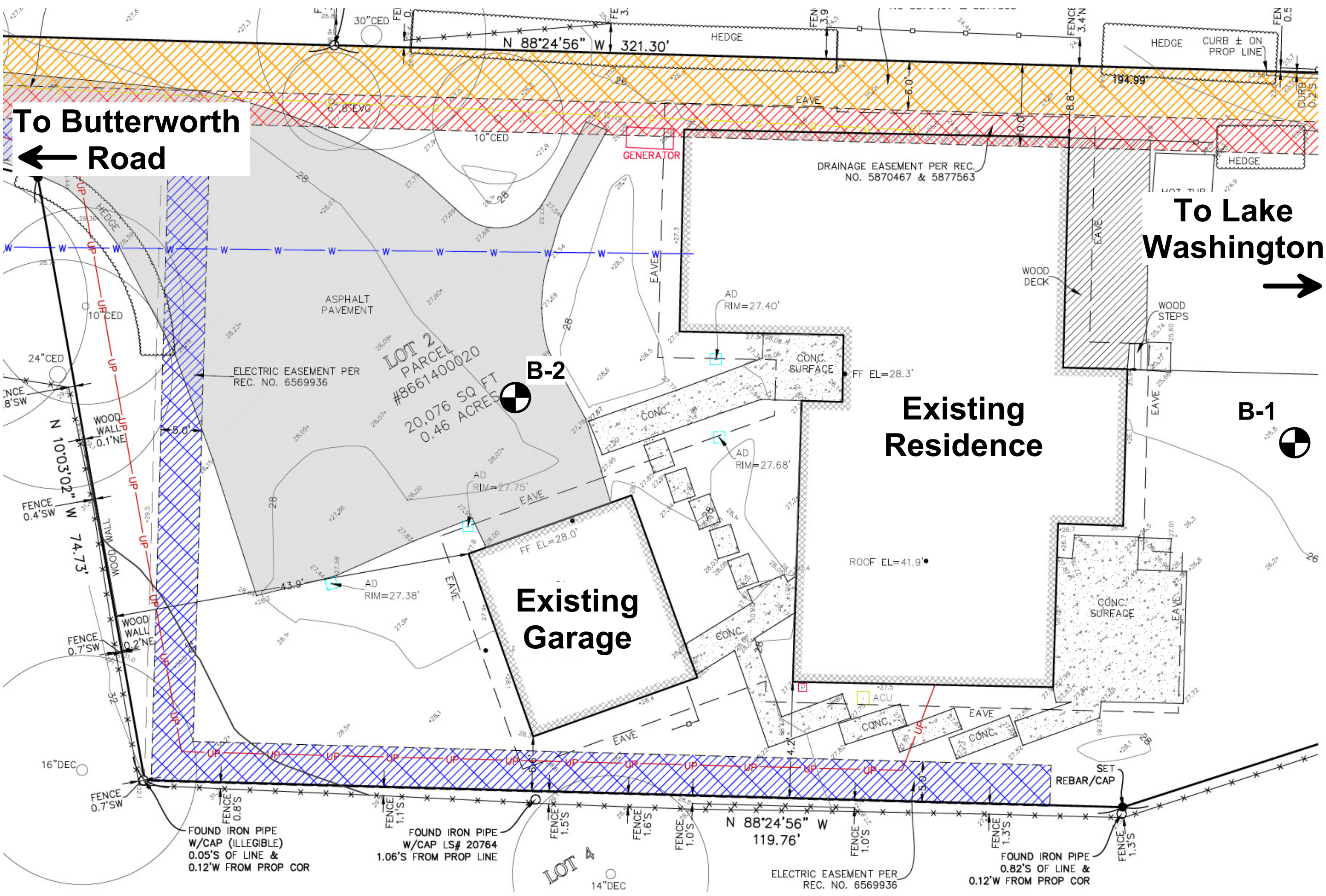
VICINITY MAP

5300 Butterworth Road
Mercer Island, Washington

Job No:	Date:		Plate:
23030	Mar. 2023		1

NORTH





Legend:

Test Boring Location

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SITE EXPLORATION PLAN

5300 Butterworth Road
Mercer Island, Washington

Job No:	Date:		Plate:
23030	Mar. 2023	No Scale	2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			4	1	PT	Dark-brown PEAT with occasional sand seams, very moist, soft
5		∇	3	2		-becomes wet
			20	3	SM/SP	Gray to gray-brown, silty SAND, fine to medium-grained, wet, medium-dense
10			23	4	SP	Gray-brown SAND with gravel, fine to medium-grained, wet, medium-dense
15			19	5	SM	Gray to gray-brown, silty SAND with gravel and occasional sandy seams, fine-grained, wet, medium-dense
					SP/SM	Dark-gray-brown SAND with trace gravel, fine-grained, wet, dense
20			35	6		-becomes interbedded with silty SAND with gravel
25			59	7		-becomes very dense
30			61	8	SP	Dark-gray-brown SAND with gravel and occasional thin, silty sand seams, fine-grained, wet, very dense

*** Test boring was terminated at 31.5 feet on March 2, 2023.**
*** Groundwater was encountered below 6 feet during drilling.**

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TEST BORING LOG

5300 Butterworth Road
Mercer Island, Washington

Job No:	Date:	Logged by:	Plate:
23030	Mar. 2023	ASM	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Asphalt over crushed rock
						Dark-brown PEAT, fibrous, soft
		∇	20	1	SM	Gray-brown with rust mottling, silty SAND, fine-grained, moist, medium-dense
5			9	2		Rust-brown SAND with silt and gravel, fine to medium-grained, wet, loose
			11	3	SP	
10			8	4		-with occasional silty sand seams
15			50/6"	5	SP/SM	Gray-brown, silty SAND with gravel and sand seams, fine-grained, wet, very dense
20			45	6	SP	Gray, gravelly SAND, fine to coarse-grained, wet, dense to very dense
25			56	7		-becomes fine-grained, with trace thin silt lenses
30						

*** Test boring was terminated at 26.5 feet on March 2, 2023.**
*** Groundwater was encountered below 4.5 feet during drilling.**

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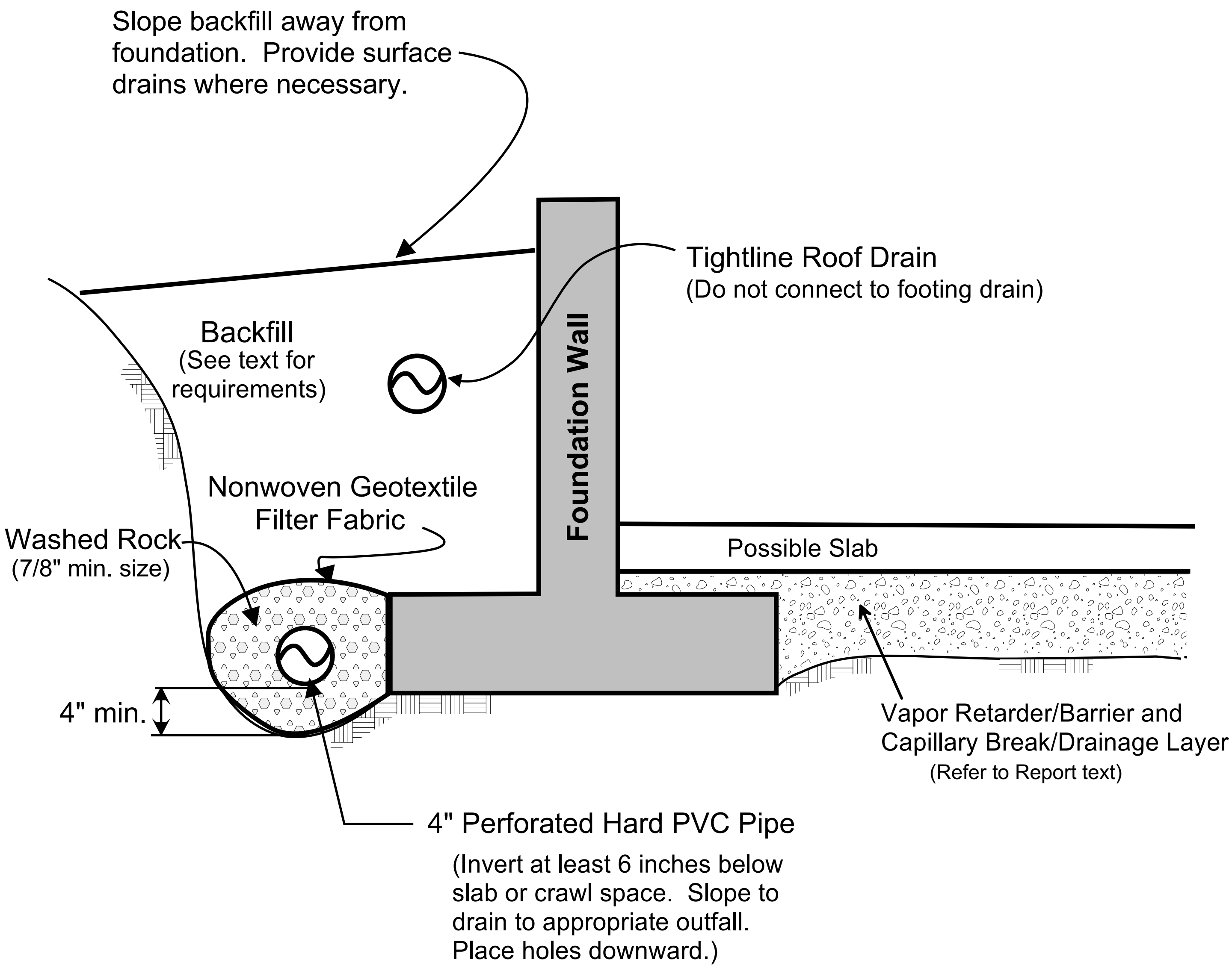
TEST BORING LOG
5300 Butterworth Road
Mercer Island, Washington

Job No:	Date:	Logged by:	Plate:
23030	Mar. 2023	ASM	4



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

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FOOTING DRAIN DETAIL

5300 Butterworth Road
Mercer Island, Washington

Job No:	Date:		Plate:
23030	Mar. 2023		5