

June 8, 2021

JN 21151

Dheeraj Koneru
7002 – 93rd Avenue Southeast
Mercer Island, Washington 98040
via email: dkoneru@gmail.com

Subject: **Transmittal Letter – Geotechnical Engineering Study and Critical Area Study**
Proposed Property Redevelopment
6610 East Mercer Way
Mercer Island, Washington

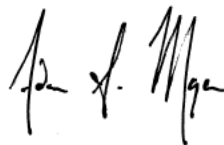
Dear Mr. Koneru:

Attached to this transmittal letter is our geotechnical engineering report and Critical Area Study related to geologic hazards for the proposed redevelopment of the property in Mercer Island. 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-10580, dated March 22, 2021.

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.



Adam S. Moyer
Geotechnical Engineer

cc: **JMK Homes** – Jed Murphey
via email: jed@jmkhomes.net

ASM/MRM:kg

GEOTECHNICAL ENGINEERING STUDY AND CRITICAL AREA STUDY
Proposed Property Redevelopment
6610 East Mercer Way
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study and Critical Area Study for proposed redevelopment of 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.

We were provided with an undated conceptual site plan of the proposed development and a topographic map of the property. The topographic map was developed by PACE and dated April 2021. Based on the conceptual site plan, and conversations with JMK Homes, we understand that the existing residence and detached garage will be demolished, and the property will likely be short-platted into two new parcels. The new, larger, eastern parcel will cover most of the property, while the new smaller western parcel will cover the western approximately 90 feet of the existing subject site. We understand a new two-story residence will be constructed near the center of the new eastern parcel and will have a lowest (first) floor near the existing ground surface. An in-ground swimming pool is proposed east of the residence, which will be surrounded by a pool deck and patio hardscaping. The grade around the pool may be raised by filling so that the bottom of the pool will not have to extend more than a few feet below the existing grade.

A smaller, detached building will be constructed on the new western parcel. While the development is early in the planning stage, we understand this detached building may be a separate residence and/or a detached garage and guest house. The detached residence will likely also be two stories with the lowest (first) floor near the existing grade. A shared driveway and/or motorcourt will be located between the two new buildings. Based on the provided conceptual plans, the proposed new residences will have minimum setbacks of 10 feet from the property lines.

Due to the proximity to Lake Washington and relatively shallow groundwater, we understand excavations for the planned redevelopment will be minimized and the final site grade will generally be kept close to, or above, the existing conditions.

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 near the south-central portion of the eastern perimeter of Mercer Island along Lake Washington. The irregularly-shaped subject site is generally trapezoidal, with a width of 170.7 feet in the north-south direction and a depth of 262 to 337 feet in the east-west direction along its southern and northern property lines, respectively. The property is bordered by Lake Washington to the east, and residential parcels containing single-family residences to the north, west, and south. The subject site is accessed by a shared private driveway that descends from East Mercer Way down to the northwestern corner of the property.

A one-story house constructed in the 1950s is located near the center of the property; the house has a floor elevation of 30.3 feet and is bordered by concrete paver patios and concrete walkways. A concrete and asphalt driveway spans between the attached garage in the northwest corner house to the shared access driveway to the northwest. A small detached garage/shop is located in the southwest corner of the property. Dense brush and shrubs surround the garage/shop and covers the southwest corner of the site, while the remainder of the property is generally covered by a grass lawn with scattered large evergreen trees. A 2- to 3-foot-tall rock bulkhead covers most of the shoreline with Lake Washington along the eastern property line; however, the northern end of the shoreline transitions into a small beach area. A wooden dock extends into Lake Washington off the shore in the southeast corner of the parcel.

The subject site slopes gently downwards from west to east to the shore of Lake Washington. The ground surface has an overall inclination ranging from 3 to 5.5 percent across the property. However, a short, 4-foot-tall moderate slope is located east of the existing house; it is apparent that this short slope was created by placing the soil generated by the house excavation to create the flat patio area along the eastern perimeter of the house.

Residential properties containing single-family residences border the site to the north, west, and south. The western and southern adjacent residences have large setbacks from the subject site. However, there is a small shed/boathouse on the southern neighboring property that is close to the south property line, and near the shore of Lake Washington. The two northern adjacent residences are offset approximately 10 and 20 feet from the shared property line with the subject site.

The City of Mercer Island's GIS tool maps the subject site within several geologic hazard areas. The entirety of the site is mapped within both a seismic hazard area and a potential landslide hazard area. There is also a mapped piped drainage that is buried along the northern property line and exits into Lake Washington to the east.

As discussed above, the subject site is very gently sloped. The western property line is at least 150 feet from the bottom of a steep-sided ravine extending in a northwesterly direction to intersect East Mercer Way. We did not observe any indications of recent slope instability on or around the site during our recent visit to the property. On the Mercer Island Landslide Hazard Map (Troost and Wisner, 2009) there is a marked "identified landslide location" near the location of the subject site. There is also a subaqueous mass wasting deposit extending east of the subject site into Lake Washington. From our observations, it is apparent that the subject site is located on an alluvial fan at the base of the ravine that rises to East Mercer Way. While landslides on the steep sideslopes of the ravine would be expected over time, the ravine itself appears to have been caused by post-glacial runoff, not large-scale slope instability.

SUBSURFACE

The subsurface conditions were explored by drilling five 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 April 30, 2021, 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 through 7.

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 silt with interbedded layers of sand, and silty sands beneath the ground surface. Dense to very dense sand was encountered beneath the loose soils at a depth of 30 feet in Test Borings 1, 2, and 3, on the eastern half of the subject site. Test Borings 4 and 5 were conducted on the western half of the subject site, and encountered medium-dense to dense sand beneath the near-surface alluvial soils at a depth of 20 feet. Medium-dense to dense clayey sand was revealed below 26 to 30 feet, and became very dense below 35 feet.

The dense to very dense soils have been compressed under the weight of glacial ice.

No obstructions were revealed by our explorations. However, debris, buried utilities, and old foundation and slab elements are commonly encountered on sites that have had previous development.

Groundwater Conditions

All of the borings found groundwater below a depth of 5 to 8 feet. The borings were conducted in early summer, when groundwater levels could have started to fall slightly. 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 International Building Code (IBC), 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 buildings will be less than 0.5 seconds, and therefore a Site Class Type E 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 1.45g and 0.50g, respectively.

The near surface soils beneath the site consisted of saturated silty sand, sand, silt, and peat. These soils have been demonstrated to have a moderate to high potential for liquefaction 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 recommendations presented in this report to support the proposed new structures on deep foundations embedded into the dense to very dense underlying sands not susceptible to seismic liquefaction are intended to prevent catastrophic failure of the proposed new structures in the event of a large seismic event. However, this does not imply the proposed pipe pile-supported structures will be functional and/or safe to inhabit after a large seismic event.

CRITICAL AREAS STUDY (MICC 19.07)

Seismic Hazard and Potential Landslide Hazard Areas: The entire subject site is located within a mapped Seismic Hazard Area and a Potential Landslide Hazard area. Both geologic hazard areas cover much of the general vicinity to the north and south as well. The site is essentially flat and is well set back from any steep slopes. As such, the potential for instability on the site is negligible. The mapping of the Potential Landslide Hazard Area is apparently due to the inference by geologists that the site lies within an ancient landslide. However, we observed no signs of landslide debris in our borings, and the topography in the site vicinity appears more likely to be the result of erosion by large volumes of post-glacial runoff, combined with more recent deposition of alluvial deposits. To our knowledge, no recent large-scale movement has been documented in this area.

The site is located over 150 feet from the steep slopes that occupy the adjacent western property. 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 steep slopes to the west.

The proposed development will be supported on deep foundation embedded into the glacially compressed soils which are not liquefiable, due to their dense nature. This mitigates the Seismic Hazard.

Buffers and Mitigation: Under MICC 19.07.160(C), a prescriptive buffer of 25 feet is indicated from all sides of a shallow landslide-hazard area. As noted above, the entire site lies within a mapped Potential Landslide Hazard Area, and the prescriptive buffer would extend far beyond the boundaries of the property and the planned development area. We recognize that the planned development will occur within the designated critical areas. The recommendations presented in this geotechnical report 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, shoring, and erosion control will mitigate any potential hazards to critical areas on the site.

Statement of Risk: In order to satisfy the City of Mercer Island's requirements, a statement of risk is needed. As such, we make the following statement:

Provided the recommendations in this report are followed, it is our professional opinion that the recommendations presented in this report for the planned alterations will render the development as safe as if it were not located in a geologically hazardous area, and will not adversely impact critical areas on adjacent properties.

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 and clayey sands were revealed below depths of 30 to 35 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. Considering this, we recommend the proposed houses 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 homes in the area. We recommend floor slabs and other settlement-sensitive elements, such as decks, patios, entryways, and the proposed pool and surrounding pool deck be supported on driven pipe piles as well to prevent differential settlement between them and the pile-supported residence. The utilities servicing the pool (water pipes, drainlines, etc.) should be hung from the pile-supported pool or pool deck, so that they are not broken or sheared from the pool when the underlying ground settles. It is also prudent to support garage slabs on the piles in order to prevent noticeable settlement relative to the rest of the foundation system.

The depth and elevation of the bottom of the proposed pool relative to the will be an important design consideration. Our test borings encountered groundwater at depths of 5 to 7 feet beneath the ground surface, or approximately at the elevation of Lake Washington. Therefore, it will be important to keep the bottom of the proposed pool above the groundwater table, in order to prevent the need for aggressive dewatering, and possibly shoring, to install the pool. It will be important to provide a pressure relief valve or drainage beneath the pool so that it is not lifted out of the ground when it is drained for maintenance. If the pool shell is cast directly against the surrounding fill and loose native soils, it should be reinforced to withstand relatively heavy lateral soil loads in the eventuality that the pool is drained for maintenance. An effective active earth pressure using a 45 pounds per cubic foot (pcf) equivalent fluid density should be used for design of the walls in the unsupported condition. Finally, as with any other settlement-sensitive on-grade elements, the pool deck surrounding the pool should be built as a reinforced concrete slab spanning between piles.

We understand that the proposed buildings will be constructed near the existing ground surface and will have a minimum setback of 10 feet from the property lines. Therefore, temporary excavations for the buildings and proposed pool are expected to be minimal. Temporary open cut slopes in the onsite soils above the water table 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 elevation will require temporary excavation shoring.

Considering the shallow groundwater elevation and the relatively impervious alluvial silt soils encountered underlying the subject site, we recommend an underslab drainage system be installed beneath the lowest finished floor slab of the proposed structures. The **Drainage Considerations** section of this report has an expanded discussion of underslab drainage recommendations. If the

interior spaces are to be located below the expected groundwater table, they would need to be designed to be watertight and to resist both hydrostatic pressures on walls and floors.

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.

As previously discussed, the test borings encountered loose alluvial soils consisting predominantly of relatively impervious silt beneath the ground surface. Furthermore, groundwater was encountered approximately 5 to 7 feet beneath the below grade. Considering this, it is our opinion that onsite stormwater infiltration or dispersion will not be feasible 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 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.
- 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 location of the site on the shore of Lake Washington will make proper erosion control implementation important to prevent adverse impacts to the lake. 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 between the silt fence and the lake left in place. Typically, if wet weather construction is anticipated, two parallel silt fences should be installed along the shoreline. 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 on the pipe piles.

Considering the competent nature of the underlying soils and the extensive amount of knowledge developed from pipe pile installation over the past 30 years, it is our opinion that load tests are not required to verify the above recommended capacities.

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.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using an ultimate (no safety factor included) passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. Compacted fill placed against the foundations can consist of imported soil that is tamped into place using the backhoe or is compacted using a jumping jack compactor. It is necessary for the fill to be compacted to a firm condition, but it does not need to reach even 90 percent relative compaction to develop the passive resistance recommended above.

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	300 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 $8H$ 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**, **Floor Slabs**, and **Drainage Considerations** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

FLOOR SLABS

As discussed in the **General** section, we recommend the proposed residences and garage slabs be constructed with structural slabs spanning between the pipe pile supported foundations, or with framed floors over crawlspaces.

Due to the relatively shallow groundwater encountered beneath the site, we recommend an underslab drainage system be installed beneath the lowest finished floor. An expanded discussion on underslab drainage is presented in the **Drainage Considerations** section of this report.

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 8. 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 discussed in the **General** section, we recommend an underslab drainage system be installed beneath the lowest finished floors. This system should consist of at least 9 inches of clean gravel or rock beneath the entire slab footprint. Perforated, 4-inch-diameter, PVC pipe should be embedded in the gravel/rock layer at 15- to 20-foot spacing and the gravel layer and piping should then be covered with a vapor retarder/barrier. The perforated pipe can be laid essentially level and at the same elevation as the perimeter footing drains. However, a solid outlet pipe should be sloped for gravity drainage to an approved discharge point. A typical underslab drainage detail is attached to this report as Plate 9.

As a minimum, a vapor retarder, as defined in the **Floor Slabs** 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. Even temporary dewatering 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 Dheeraj Koneru and his 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.

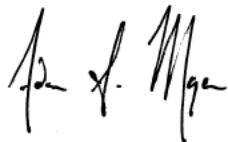
The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 7	Test Boring Logs
Plate 8	Typical Footing Drain Detail
Plate 9	Typical Underslab 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.



Adam S. Moyer
Geotechnical Engineer

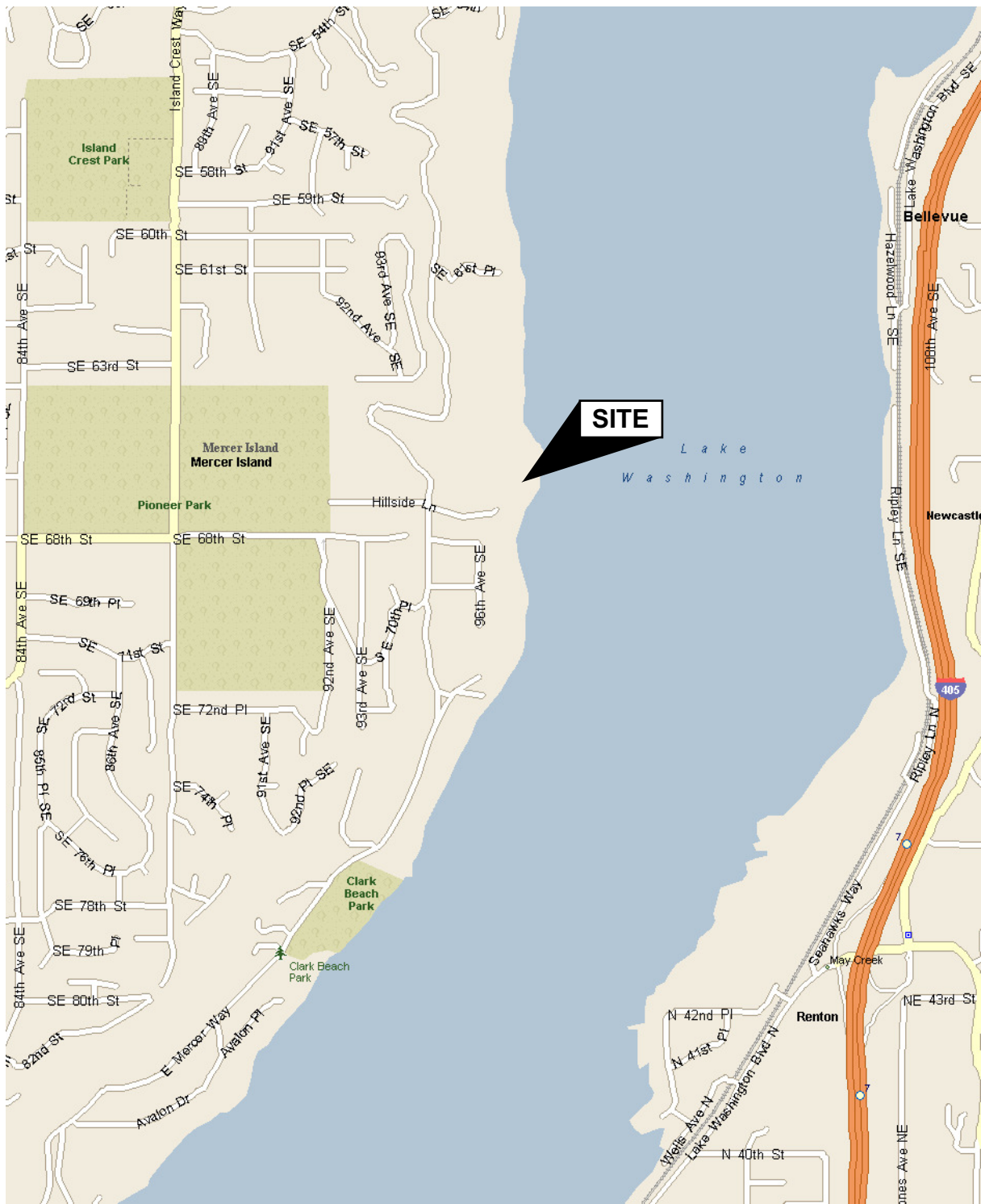
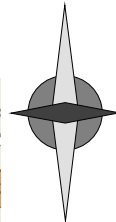


06/08/2021

Marc R. McGinnis, P.E.
Principal

ASM/MRM:kg

NORTH



(Source: Microsoft MapPoint, 2013)



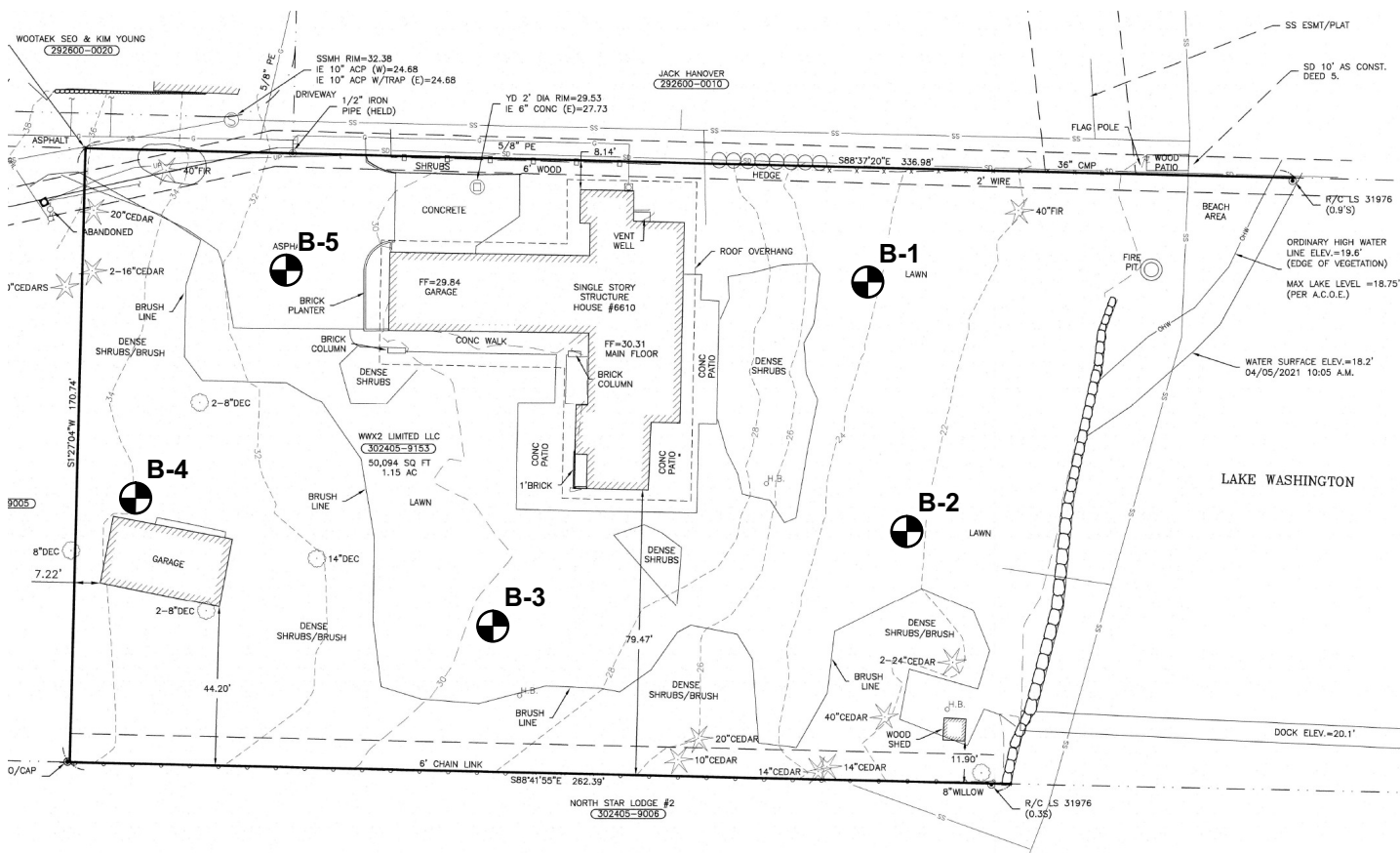
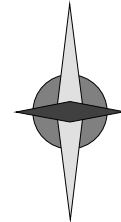
**GEOTECH
CONSULTANTS, INC.**

VICINITY MAP


**6610 East Mercer Way
Mercer Island, Washington**

<i>Job No:</i> 21151	<i>Date:</i> June 2021	<i>Plate:</i> 1
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NORTH



Legend:

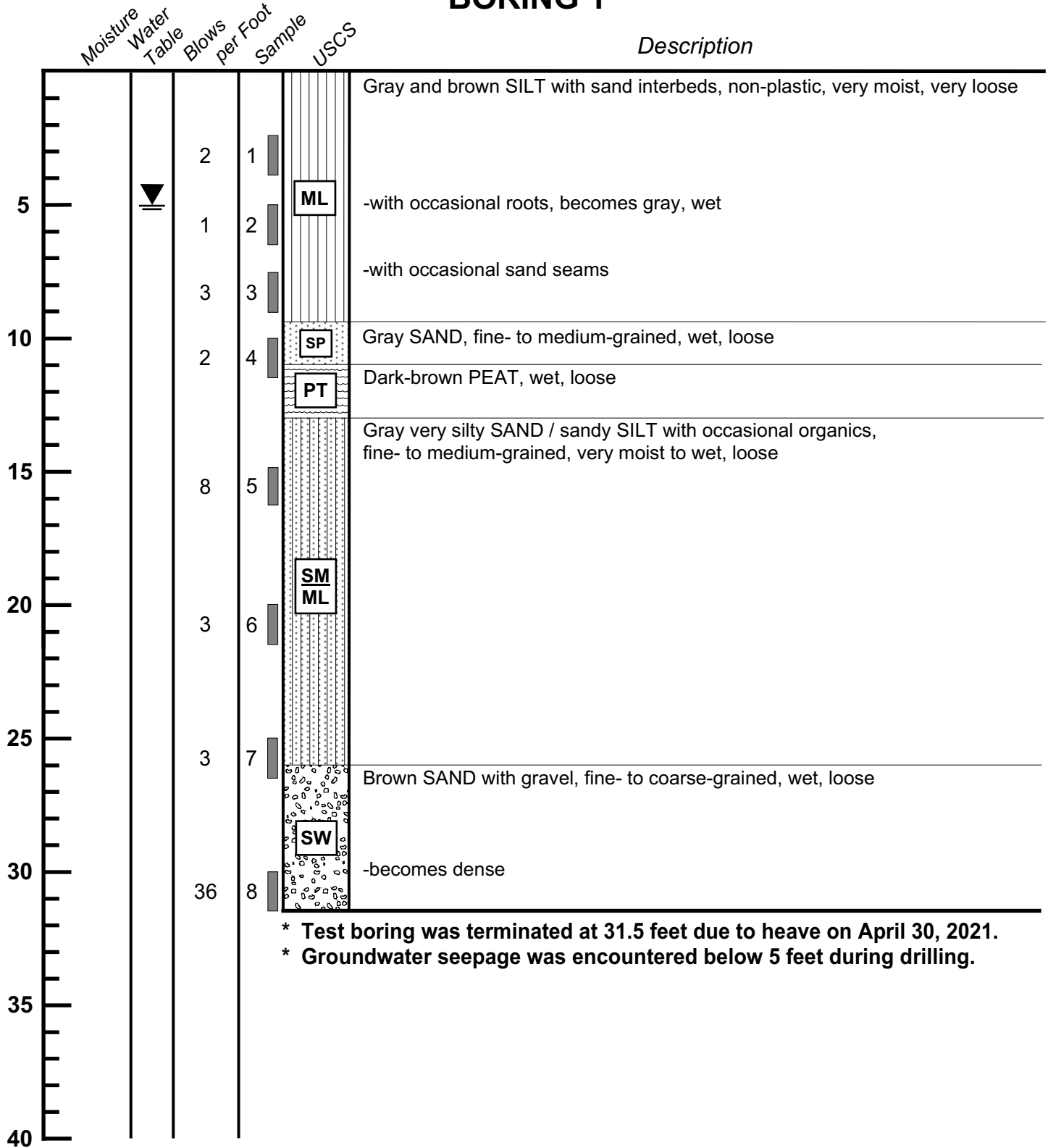
 Test Boring Location



SITE EXPLORATION PLAN
 6610 East Mercer Way
 Mercer Island, Washington

Job No: 21151	Date: June 2021	No Scale	Plate: 2
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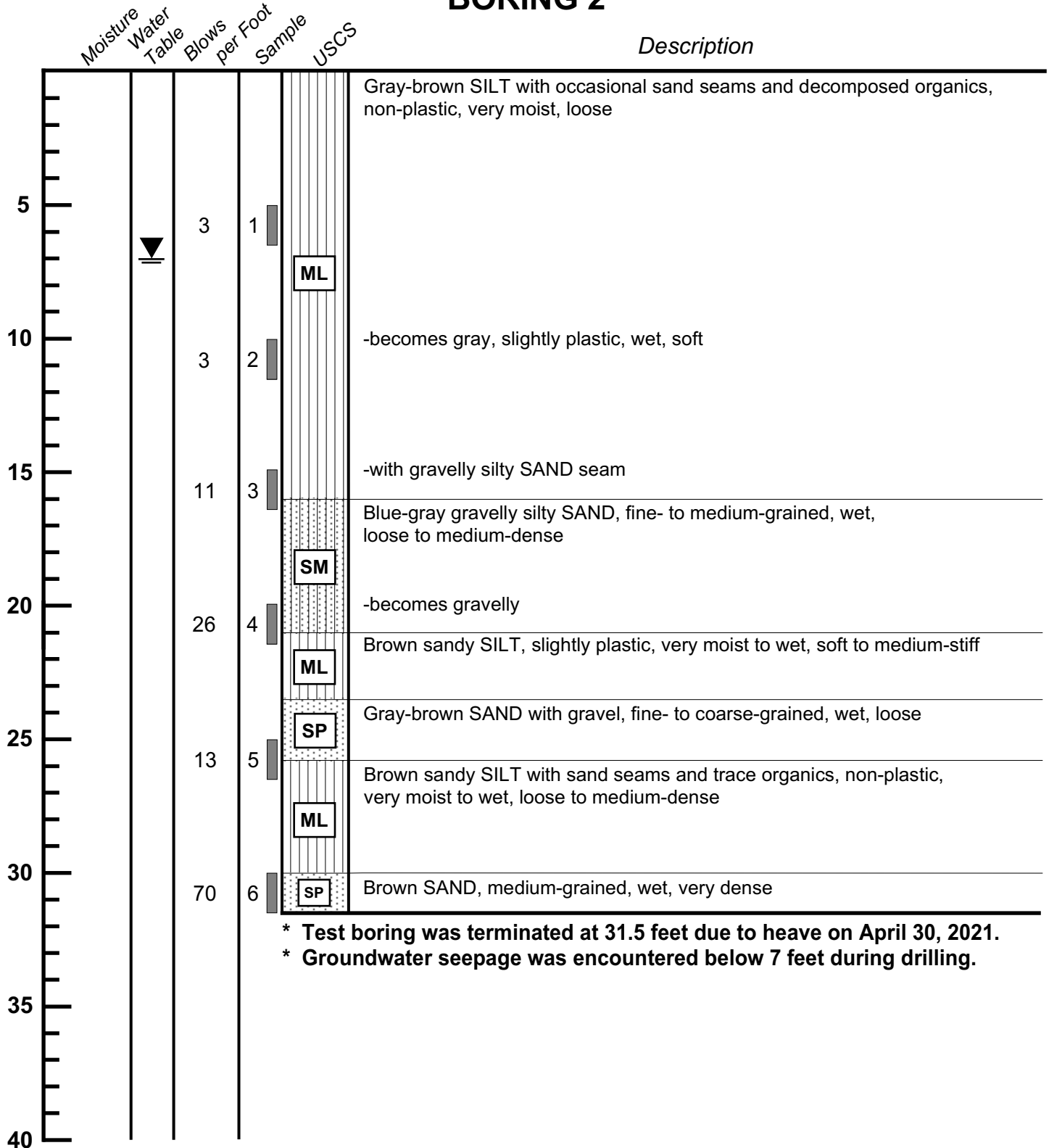
BORING 1



BORING LOG
 6610 East Mercer Way
 Mercer Way, Washington

Job No: 21151	Date: June 2021	Logged by: ASM	Plate: 3
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BORING 2



* Test boring was terminated at 31.5 feet due to heave on April 30, 2021.
 * Groundwater seepage was encountered below 7 feet during drilling.

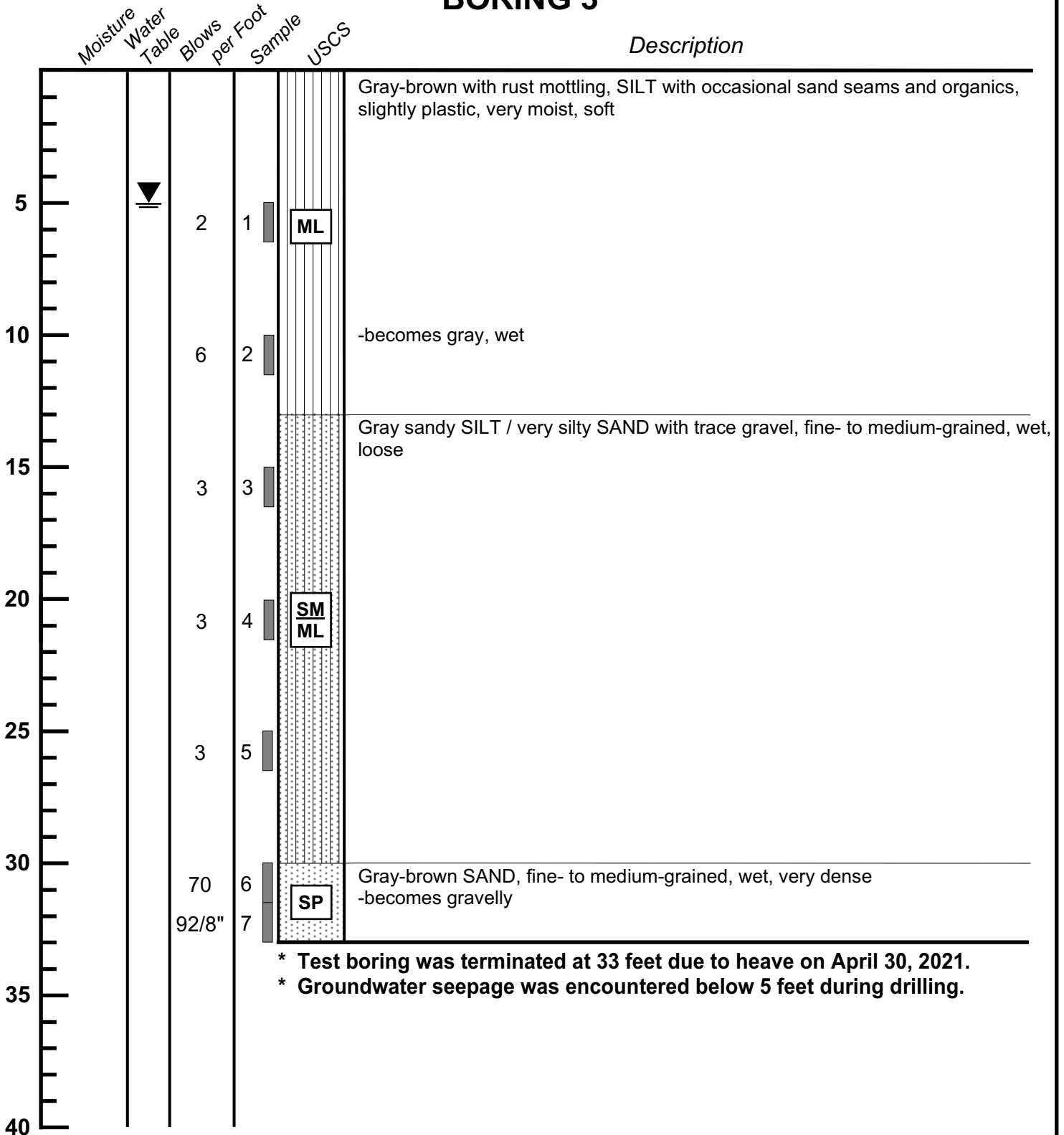


BORING LOG
 6610 East Mercer Way
 Mercer Way, Washington

Job No: 21151	Date: June 2021	Logged by: ASM	Plate: 4
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BORING 3

Description



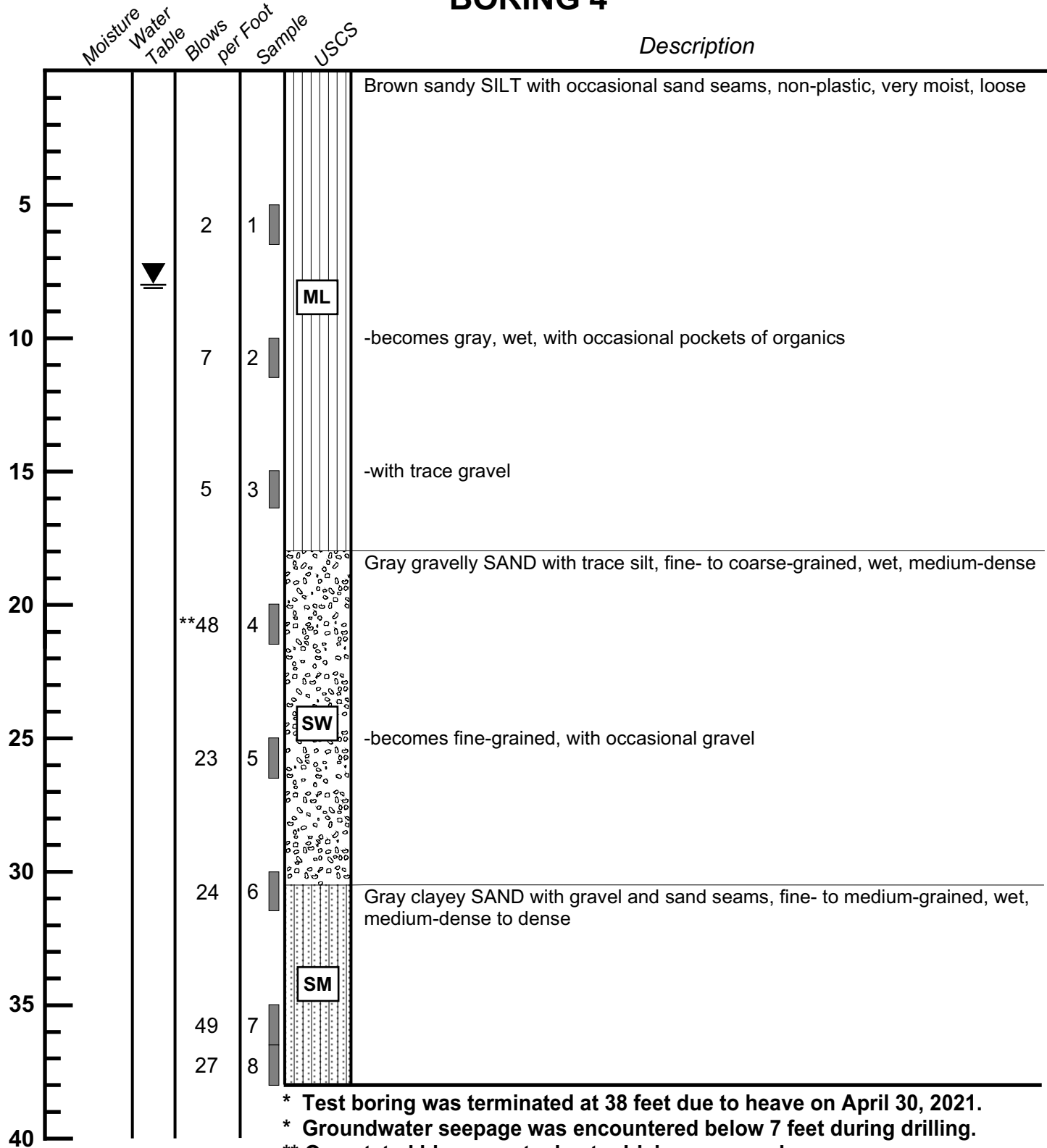
* Test boring was terminated at 33 feet due to heave on April 30, 2021.
 * Groundwater seepage was encountered below 5 feet during drilling.



BORING LOG
 6610 East Mercer Way
 Mercer Way, Washington

Job No: 21151	Date: June 2021	Logged by: ASM	Plate: 5
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BORING 4



* Test boring was terminated at 38 feet due to heave on April 30, 2021.
 * Groundwater seepage was encountered below 7 feet during drilling.
 ** Overstated blow counts due to driving on gravels.

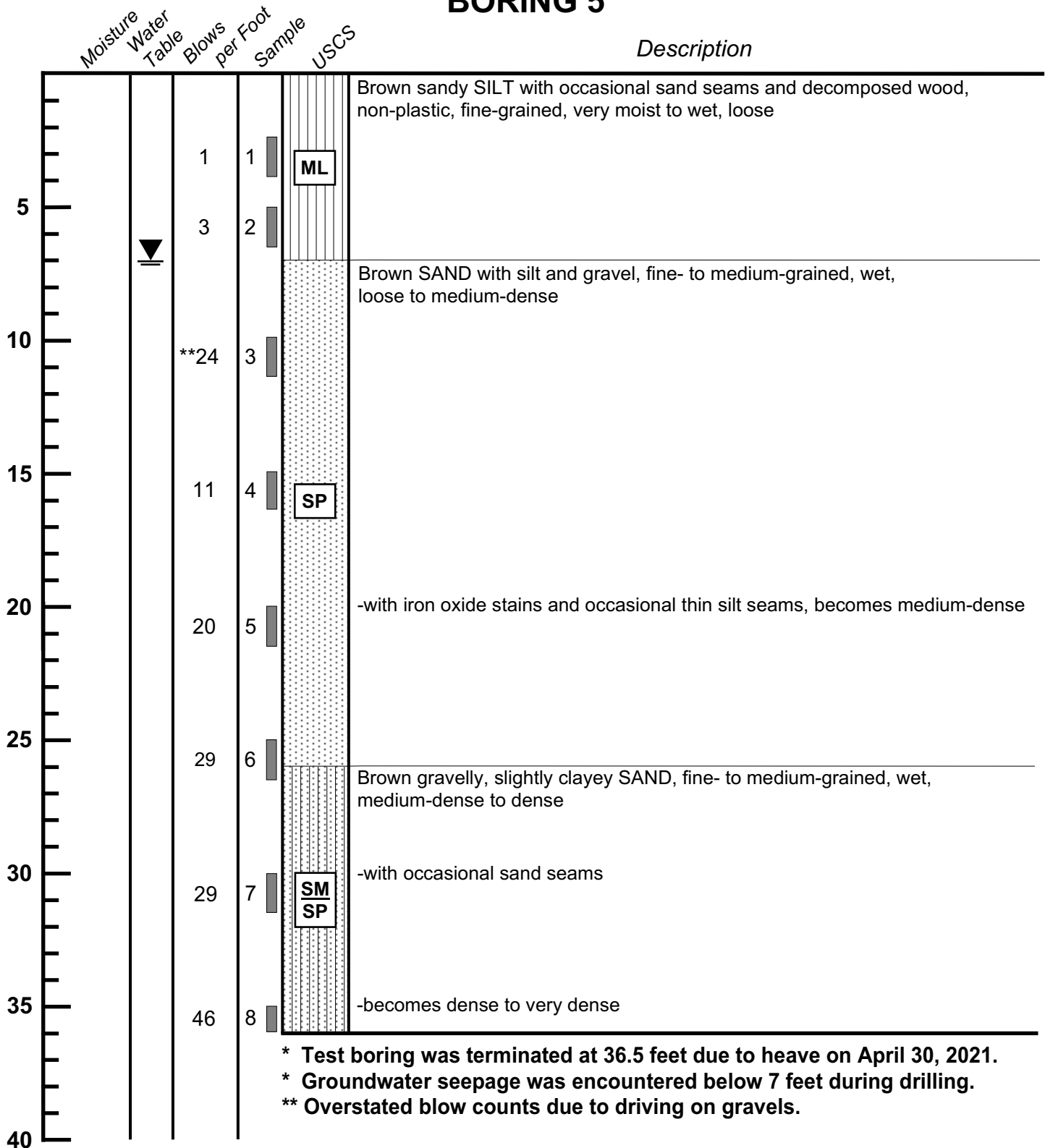


BORING LOG
 6610 East Mercer Way
 Mercer Way, Washington

Job No: 21151	Date: June 2021	Logged by: ASM	Plate: 6
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BORING 5

Description

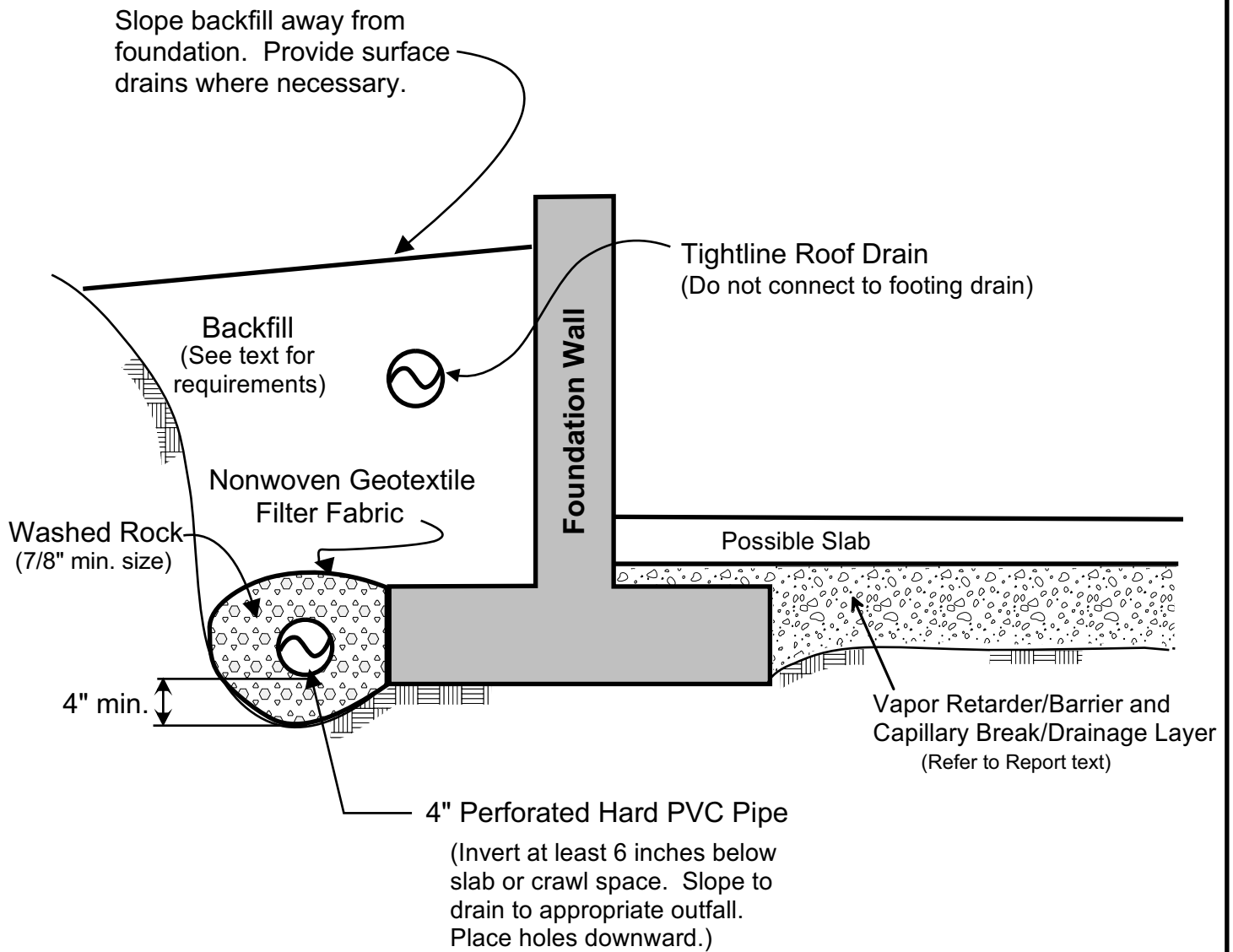


- * Test boring was terminated at 36.5 feet due to heave on April 30, 2021.
- * Groundwater seepage was encountered below 7 feet during drilling.
- ** Overstated blow counts due to driving on gravels.



BORING LOG
6610 East Mercer Way
Mercer Way, Washington

Job No: 21151	Date: June 2021	Logged by: ASM	Plate: 7
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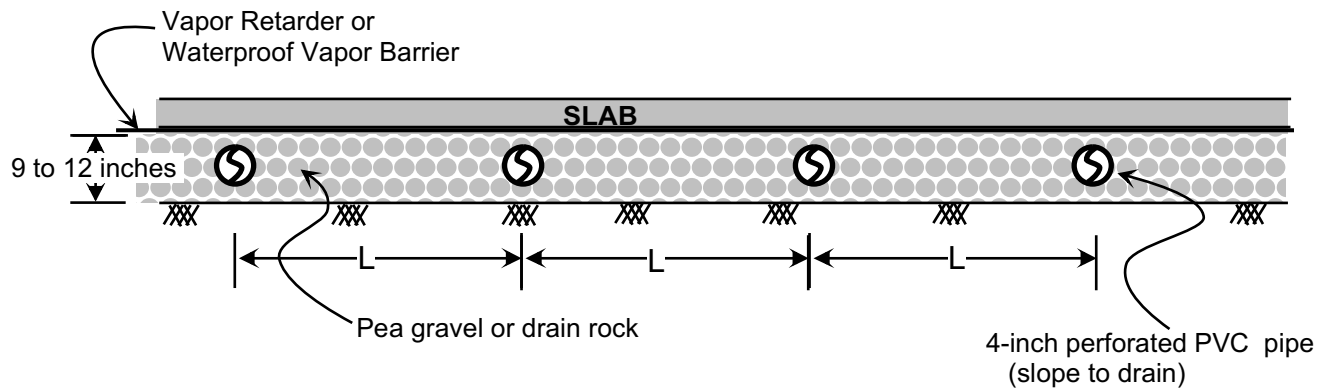
NOTES:

- (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.



FOOTING DRAIN DETAIL
6610 East Mercer Way
Mercer Island, Washington

Job No: 21151	Date: June 2021	Plate: 8
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NOTES:

- (1) Refer to the report text for additional drainage and waterproofing considerations.
- (2) The typical maximum underslab drain separation (L) is 15 to 20 feet.
- (3) No filter fabric is necessary beneath the pipes as long as a minimum thickness of 4 inches of rock is maintained beneath the pipes.
- (4) The underslab drains and foundation drains should discharge to a suitable outfall.



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TYPICAL UNDERSLAB DRAINAGE

6610 East Mercer Way
Mercer Island, Washington

Job No:
21151

Date:
June 2021

Plate: