GEOTECH
CONSULTANTS, INC.

2401 10th Ave E
Seattle, Washington 98102
(425) 747-5618

October 17, 2018

JN 18428

Elizabeth Goodrich
1608 – 38th Avenue East
Seattle, Washington 98112
via email: *esgoodrich@gmail.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Single-Family Residence
8424 Benotho Place
Mercer Island, Washington

Dear Ms. Goodrich:

Attached to this transmittal letter is our geotechnical engineering report for the proposed single-family residence to be constructed 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 excavation shoring. This work was authorized by your acceptance of our proposal, P-10145, dated August 1, 2018.

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: **Okano Picard Studio** – Michael Picard
via email: michael@okanopicardstudio.com

ASM/MRM:kg

GEOTECHNICAL ENGINEERING STUDY
Proposed Single-Family Residence
8424 Benotho Place
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed single-family residence to be constructed on the subject lot in Mercer Island.

We were provided with conceptual architectural plans and a topographic map. Okano Picard Studio developed the plans, which are dated August 21, 2018. The topographic map was developed by Terrane and dated June 10, 2018. Based on this information, we understand that the existing residence will be demolished, and a new single-family home will be constructed near the center of the property, east of the current house's location. The western half of the site slopes steeply downwards from Benotho Place to the gently-sloped eastern half of the site and Lake Washington to the east. A two-car garage will be located on the northern end of the upper level (Level 3) with access from Benotho Place. The floor of the garage will be approximately 6 feet lower than Benotho Place. The proposed new residence will be cut into the toe of the steep slope and will daylight to the east. We understand that the lowest finished floor of the western half of the house will be the main floor (Level 2) at an elevation of 34.5 feet, and the eastern half of the residence will have a basement floor (Level 1) with an elevation of 24.5 feet. Based on the provided site plans and topographic map, cuts in the order of 9 to 12.5 feet will be required along the western perimeter of the proposed residence to reach the main floor elevation. Shallow excavations are anticipated along the eastern, downslope side of the residence. The sloped area between the garage and Benotho Place will be filled for the new driveway.

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 on the southern end of Mercer Island. The subject site has 79 feet of frontage along the eastern side of Benotho Place and extends east 217 and 286 feet along its southern and northern property lines into Lake Washington. Only approximately the western 120 feet of the property's ground surface is located above the elevation of Lake Washington. For this report, the 'subject site' will refer to the western portion of the property above Lake Washington.

The subject site is defined by two separate ground surface conditions. The eastern half, east of the existing house, contains a grass lawn that slopes gently downwards towards Lake Washington to the east. However, the grade on the western half of the site drops steeply downward toward the east from the western property line. The existing single-family residence is located on the western half of the property and cut into the steep slope. The residence has a main floor near an elevation of 55 feet and basement floors that step down to the east with finished floor elevations of 45 and 37 feet. A concrete driveway connects Benotho Place to an attached garage in the northwest corner of the house. The steep slope has an average inclination of 35 to 50 percent over a height of 18 to

24 feet to the north of the existing house. To the south of the house, the steep grade drop is broken up by some retaining walls. Concrete stairs which have experienced significant amounts of settlement follow the southern end of the residence down the steep slope. Two-tiered concrete rubble walls straddle the southern end of the site's western property line south of the residence; the grade drops approximately 9 feet across the two walls. A 4-foot-tall rock bulkhead separates the eastern grass lawn from Lake Washington to the east.

As discussed above, the subject site is bordered by Benotho Place to the west and Lake Washington to the east. The adjacent parcels to the north and south each contain single-family residences with offsets of 7.5 and 5.9 feet from the subject site respectively. The northwest corner of the southern adjacent residence contains an attached garage; the remainder of the one-story residence overlies a basement that daylights to the east. The adjacent residence to the north has one story and contains a basement floor that underlies the eastern two thirds of the residence. A small shed is located on the northern adjacent property with an offset of 2 feet from the subject site's northwest corner.

SUBSURFACE

The subsurface conditions were explored by drilling three 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 September 21, 2018 using a both track-mounted, hollow-stem auger drill and a portable Acker drill. The Acker drill system utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at approximate 2.5- and 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 5.

Soil Conditions

Test Borings 1 and 3 were conducted along the top of the steep slope in the southwest and northwest corners of the site respectively. Test Boring 1 encountered 4.5 feet of loose gravelly sand fill with asphalt debris beneath the ground surface. Loose native silt was encountered beneath the fill and extended to 20 feet below grade. The native silt became glacially-compressed and massive below 20 feet and dense below 30 feet. Test Boring 3 encountered 4.5 feet of loose silt fill soils overlying loose native silt. The native silt became glacially-compressed below 10 feet and dense below a depth of 35 feet.

Test Boring 2 was conducted on the lower, eastern, grass lawn near the toe of the steep slope. Approximately 3 feet of loose sand with gravel was encountered overlying loose native silt. The silt became dense below 15 feet and very dense below 20 feet.

The looser, near-surface soils are disturbed deposits remaining from an ancient landslide that affected this portion of Mercer Island sometime after the recession of the last glaciers over 10,000 years ago. The glacially-compressed silt was below the old landslide.

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. We encountered some asphalt debris with the fill found in Boring 1.

Groundwater Conditions

Slight groundwater seepage was observed at a depth of 2.5 to 3 feet perched in the sand above the relatively impermeable silt in Test Boring 2.

The borings were conducted following a dry summer. They were also left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself.

It should be noted that groundwater levels vary seasonally 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.

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 15 to 20 feet of uncontrolled fill and loose, disturbed native silt beneath the ground surface across the subject site. The native silt became massive and undisturbed below these depths and generally became denser with depth. Conventional shallow foundations constructed on the loose, upper, moderately-compressible silt soils would experience significant post-construction settlement as the loose soils consolidate over time. This can be observed on the site in the cracks that have formed in the existing residence foundations and surrounding hardscaping. Considering this, we recommend the proposed residence be supported on small-diameter pipe piles driven into the dense underlying silt. 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 and entryways, be supported on driven pipe piles as well to prevent differential settlement between them and the pile-supported residence. It is prudent to support garage slabs on the piles, in order to prevent noticeable settlement relative to the rest of the foundation system.

Given the soil conditions, it is likely that some settlement of the ground surrounding the pile-supported house will occur over time. In order to reduce the potential problems associated with this, we suggest 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 lowest floors 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 and the garage approach apron as reinforced slabs that are doweled into the grade beam at the door thresholds. This will allow the exterior concrete slabs to ramp down and away from the building as they settle, without causing a downset at the door thresholds.

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

The depth of the excavation into the western slope will be an important geotechnical consideration. Based on the provided preliminary plans, the western perimeter of the proposed residence will require excavations up to 9 to 12.5 feet in height into the toe of the western slope to reach the anticipated bottom-of-excavation elevation of approximately 32.5 feet. We understand that the residence will have setbacks of approximately 10 feet from the northern and southern property line. The near-surface soils have a very low strength. Considering the presence of the street to the west of the site, the steep inclination of the ground surface, and the low strength of the soils, it will be important that excavation shoring in the form of soldier piles be planned for the temporary excavations. This will be necessary to maintain the stability of the sloping ground until the foundation walls of the new house are completed. The ***Cantilevered Soldier Pile Shoring*** section of this report provides an expanded discussion of temporary excavation shoring recommendations.

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 front entry walks.

The subject site is located within a potential landslide hazard area that encompasses the entire general vicinity. The core of the subject site consists of dense native soil that has a very low potential for deep-seated landslides. However, any slope in the Puget Sound area has some potential for shallow soil movement in the near-surface soils, particularly after extended periods of concentrated precipitation. The potential for failures of the onsite steep slope will be mitigated by proper retention. The recommendations presented in this report are intended to prevent adverse impacts to the stability of the slope onsite.

The site also meets the City of Mercer Island's criteria for an erosion hazard area. We have been associated with numerous waterfront projects involving excavation into steep slopes that have avoided siltation of the lake and surrounding properties by exercising care and being pro-active 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 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. 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 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.

In order to satisfy the City of Mercer Island's requirements, we make the following statement:

> It is our professional opinion that the development practices that we have recommended in this report would render the proposed development as safe as if it were not located in a geologic hazard area.

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.

Considering the very low permeability of the silt encountered in our test borings, it is our professional opinion that onsite stormwater infiltration is not feasible of the subject site.

The adjacent houses are likely supported on conventional foundations that bear on compressible soils. As a result, it is likely that they have undergone excessive settlement already. There is always some risk associated with demolition and foundation construction near structures such as this. Unfortunately, it is becoming more and more common for adjacent property owners to make unsubstantiated damage claims on new projects that occur close to their developed lots. It is imperative that un-shored excavations do not extend below a 3:1 (Horizontal:Vertical) imaginary bearing zone sloping downward from existing footings. Contractors working on the demolition and construction of your home must be cautioned to avoid strong ground vibrations, which could cause additional settlement in the neighboring foundations. Installation of pipe piles is a loud process, but it does not cause strong ground vibrations. During demolition, strong pounding on the ground with the excavator, which is often used to break up debris and concrete, should not occur. Large equipment and vibratory compactors should not be used close to the north and south property lines. We recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring, and/or commencing with the excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners. Additionally, in order to protect yourselves from unsubstantiated damage claims from the adjacent owners, 1) the existing condition of the foundation should be documented before starting demolition, and 2) the footings should be monitored for vertical movement during the demolition, excavation, and construction process. These are common recommendations for projects located close to existing structures that may bear on loose soil and have already experienced excessive settlement. We can provide additional recommendations for documentation and monitoring of the adjacent structures, if desired.

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.

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 D (Stiff Soil). 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 1.46g and 0.56g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) during an earthquake 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 proposed residence will be supported on small-diameter pipe piles embedded into the dense underlying soils which are not susceptible to seismic liquefaction.

PIPE PILES

Four- or 6-inch-diameter pipe piles driven with a 850- or 1,100- or 2,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacity.

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	25 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. The site soils are not highly organic, and are not located near salt water. As a result, they do not have an elevated corrosion potential. Considering this, it is our opinion that standard "black" pipe can be used, and corrosion protection, such as galvanizing, is not necessary for 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.

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 a passive earth pressure of 250 pounds per cubic foot (pcf) for this resistance. This passive earth pressure is an ultimate value; it does not include a safety factor. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate.

FOUNDATION AND RETAINING WALLS

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 * - Level Backfill - 2.5:1 (H:V) Sloped Backfill	 40 pcf 55 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. 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 10**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 that will be excavated have a low strength and poor drainage properties. They should not be reused as wall backfill. We expect that most or all backfill will have to be imported.

Drainage composite similar to Miradrain 6000 should be placed against the backfilled retaining walls. The drainage composites should be hydraulically connected to the foundation drain system. Free-draining backfill should be used for the entire width of the backfill where seepage is encountered. For increased protection, drainage composites should be placed along cut slope faces, and the walls should be backfilled entirely with free-draining soil. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

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

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. This capillary break/drainage layer is not necessary if an underslab drainage system is installed.

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***, ***Permanent 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.

CANTILEVERED SOLDIER PILE SHORING

Cantilevered soldier pile systems have proven to be an efficient and economical method for providing excavation shoring where the depth of excavation is less than 15 feet.

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. Based on the perched groundwater and loose soils encountered in our test borings, the contractor should be prepared to case the holes or use the slurry method if caving soil is encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

If permanent building walls are to be constructed against the shoring walls, drainage should be provided by attaching a geotextile drainage composite with a solid plastic backing, similar to Miradrain 6000, to the entire face of the lagging, prior to placing waterproofing and pouring the foundation wall. These drainage composites should be hydraulically connected to the foundation drainage system through weep holes placed in the foundation walls.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered, and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 40 pounds per cubic foot (pcf). If soldier pile walls will permanently retain soil, they should be designed for earth pressures presented in the ***Permanent Foundations and Retaining Walls*** section.

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Existing adjacent buildings may exert surcharges on the proposed shoring wall, unless the buildings are underpinned. Slopes above the shoring walls will exert additional surcharge pressures. These surcharge pressures will vary, depending on the configuration of the cut slope and shoring wall. We can provide recommendations regarding slope and building surcharge pressures when the preliminary shoring design is completed.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 300 pcf. No safety factor is included in the given value. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up." Plate 6 attached to this report presents a typical cantilevered soldier pile wall detail.

EXCAVATION AND SHORING MONITORING

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. This can translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring or commencing excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners.

Additionally, the shoring walls and any adjacent foundations should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least every other soldier pile should be monitored by taking readings at the top of the pile. Additionally, benchmarks installed on the surrounding buildings should be monitored for at least vertical movement. We suggest taking the readings at least once a week, until it is established that no deflections are occurring. The initial readings for this monitoring should be taken before starting any demolition or excavation on the site.

DRAINAGE CONSIDERATIONS

We anticipate that permanent foundation walls may be constructed against the shoring walls. Where this occurs, a plastic-backed drainage composite, such as Miradrain, Battledrain, or similar, should be placed against the entire surface of the shoring prior to pouring the foundation wall. Weep pipes located no more than 6 feet on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the inside of the perimeter footing will be used to collect and carry the water discharged by the weep pipes to the storm system. Isolated zones of moisture or seepage can still reach the permanent wall where groundwater finds leaks or joints in the drainage composite. This is often an acceptable risk in unoccupied below-grade spaces, such as parking garages. However, formal waterproofing is typically necessary in areas where wet conditions at the face of the permanent wall will not be tolerable. If this is a concern, the permanent drainage and waterproofing system should be designed by a specialty consultant familiar with the expected subsurface conditions and proposed construction. Plate 7 presents typical considerations for a shoring drainage system.

Footing drains placed inside the building, outside of the building, or behind backfilled walls should consist of 4-inch, perforated PVC pipe surrounded by at least 6 inches of 1-inch-minus, washed rock wrapped in a 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 level of a crawl space or the bottom of a floor slab, and it should be sloped slightly for drainage. All roof and surface water drains must be kept separate from the foundation drain system.

Footing drains outside of the building 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. A typical footing drain detail is attached to this report as Plate 8. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

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.

Slight perched groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. 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. It is important that existing foundations be removed before site development. 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.

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 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 test borings 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 pits 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 Elizabeth Goodrich and her 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 - 5	Test Boring Logs
Plate 6	Cantilevered Soldier Pile Wall Detail
Plate 7	Typical Shoring Drain Detail
Plate 8	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,

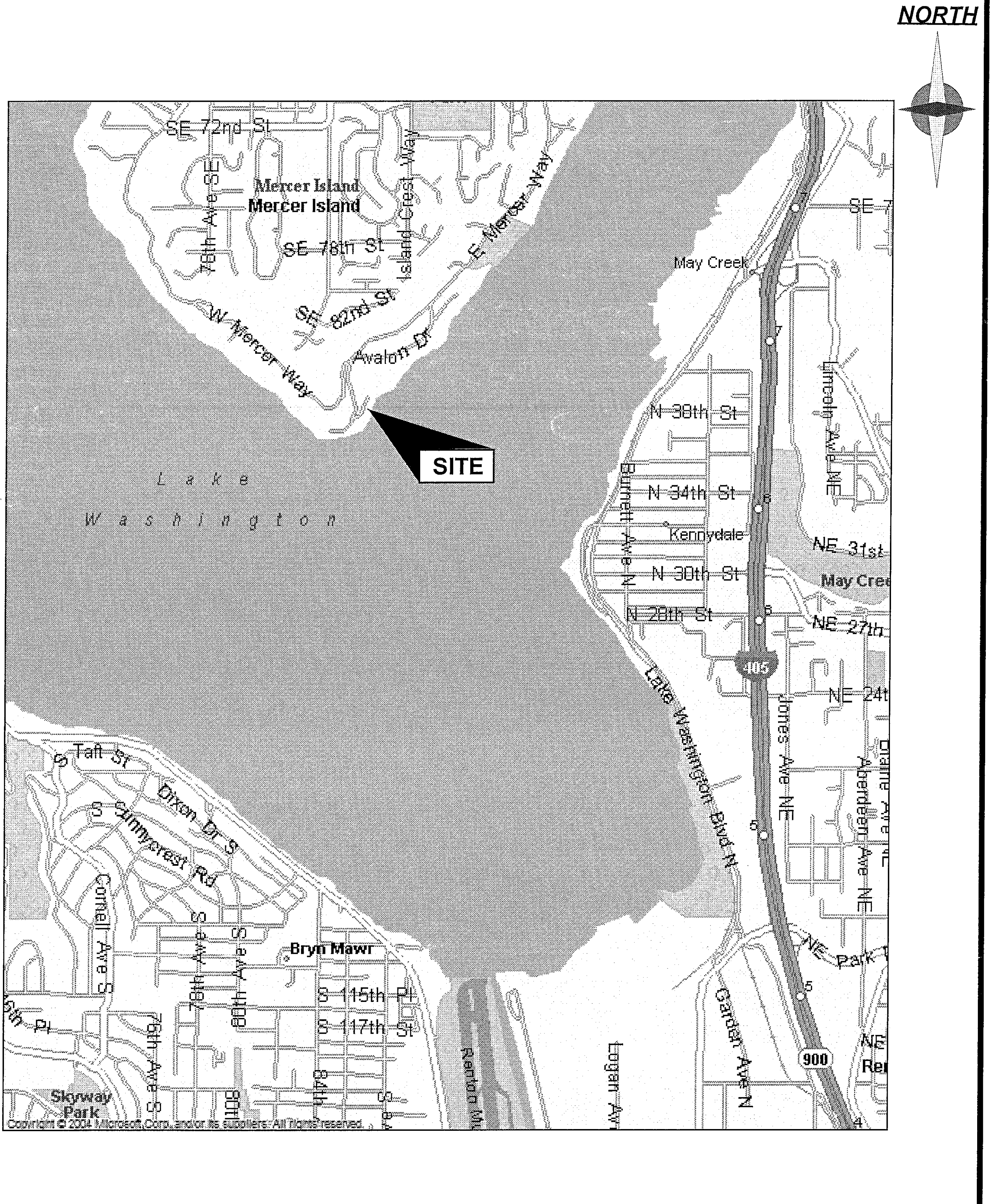
GEOTECH CONSULTANTS, INC.

Adam S. Moyer
Geotechnical Engineer

27845

10/17/18
Marc R. McGinnis, P.E.
Principal

ASM/MRM:kg



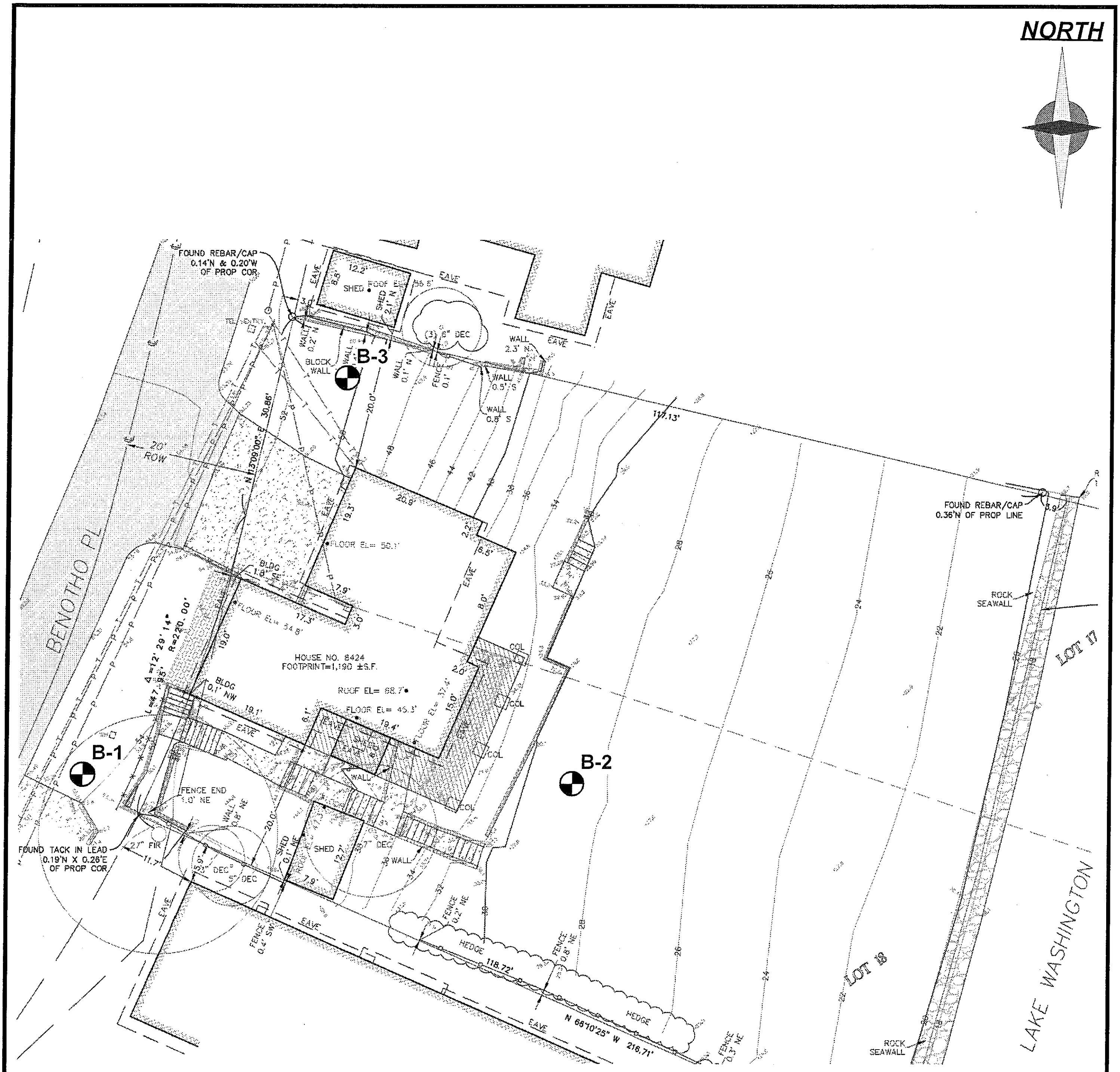
(Source: Microsoft MapPoint, 2013)

GEOTECH
CONSULTANTS, INC.

VICINITY MAP

8424 Benotho Place
Mercer Island, Washington

Job No:	Date:		Plate:
18428	Oct. 2018		1



Legend:

⊕ Test Boring Location

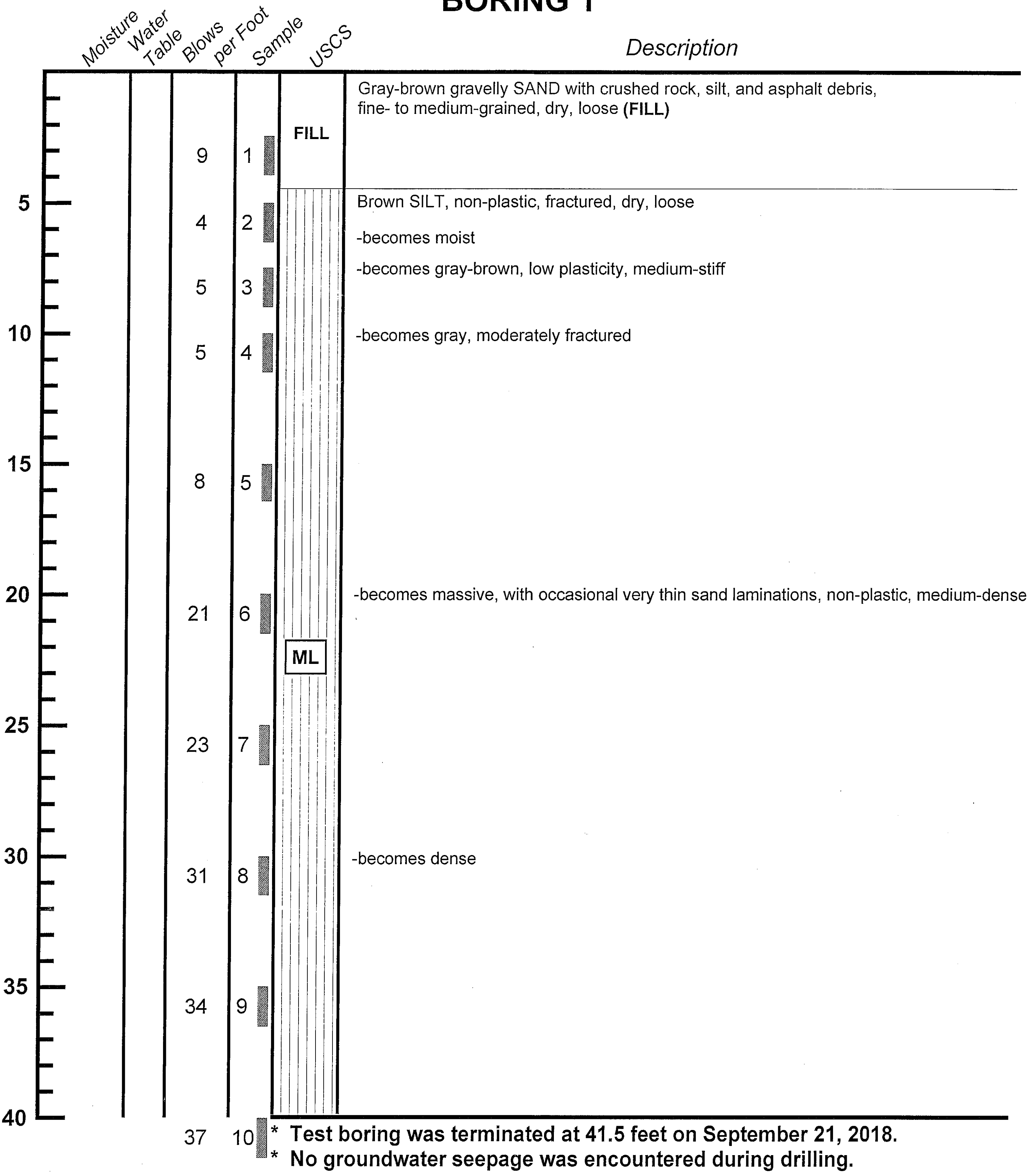
GEOTECH
CONSULTANTS, INC.

SITE EXPLORATION PLAN

8424 Benotho Place
Mercer Island, Washington

Job No:	Date:		Plate:
18428	Oct. 2018	No Scale	2

BORING 1



Gray-brown gravelly SAND with crushed rock, silt, and asphalt debris, fine- to medium-grained, dry, loose **(FILL)**

Brown SILT, non-plastic, fractured, dry, loose

-becomes moist

-becomes gray-brown, low plasticity, medium-stiff

-becomes gray, moderately fractured

-becomes massive, with occasional very thin sand laminations, non-plastic, medium-dense

-becomes dense

*** Test boring was terminated at 41.5 feet on September 21, 2018.**

*** No groundwater seepage was encountered during drilling.**

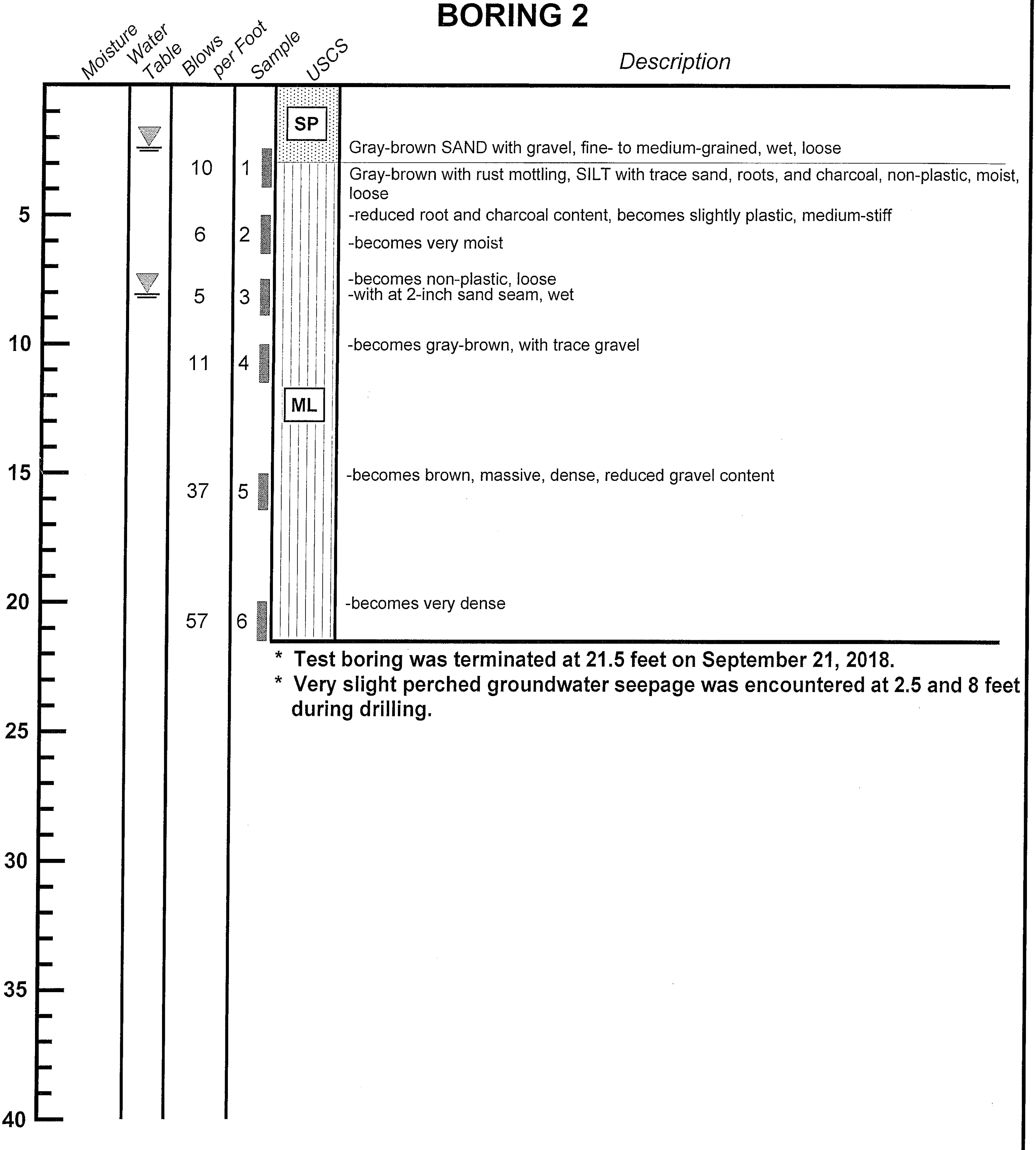
GEOTECH
CONSULTANTS, INC.

BORING LOG

8424 Benotho Place
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
18428	Oct. 2018	ASM	3

BORING 2



Gray-brown SAND with gravel, fine- to medium-grained, wet, loose

Gray-brown with rust mottling, SILT with trace sand, roots, and charcoal, non-plastic, moist, loose

-reduced root and charcoal content, becomes slightly plastic, medium-stiff

-becomes very moist

-becomes non-plastic, loose

-with at 2-inch sand seam, wet

-becomes gray-brown, with trace gravel

-becomes brown, massive, dense, reduced gravel content

-becomes very dense

* **Test boring was terminated at 21.5 feet on September 21, 2018.**

* **Very slight perched groundwater seepage was encountered at 2.5 and 8 feet during drilling.**

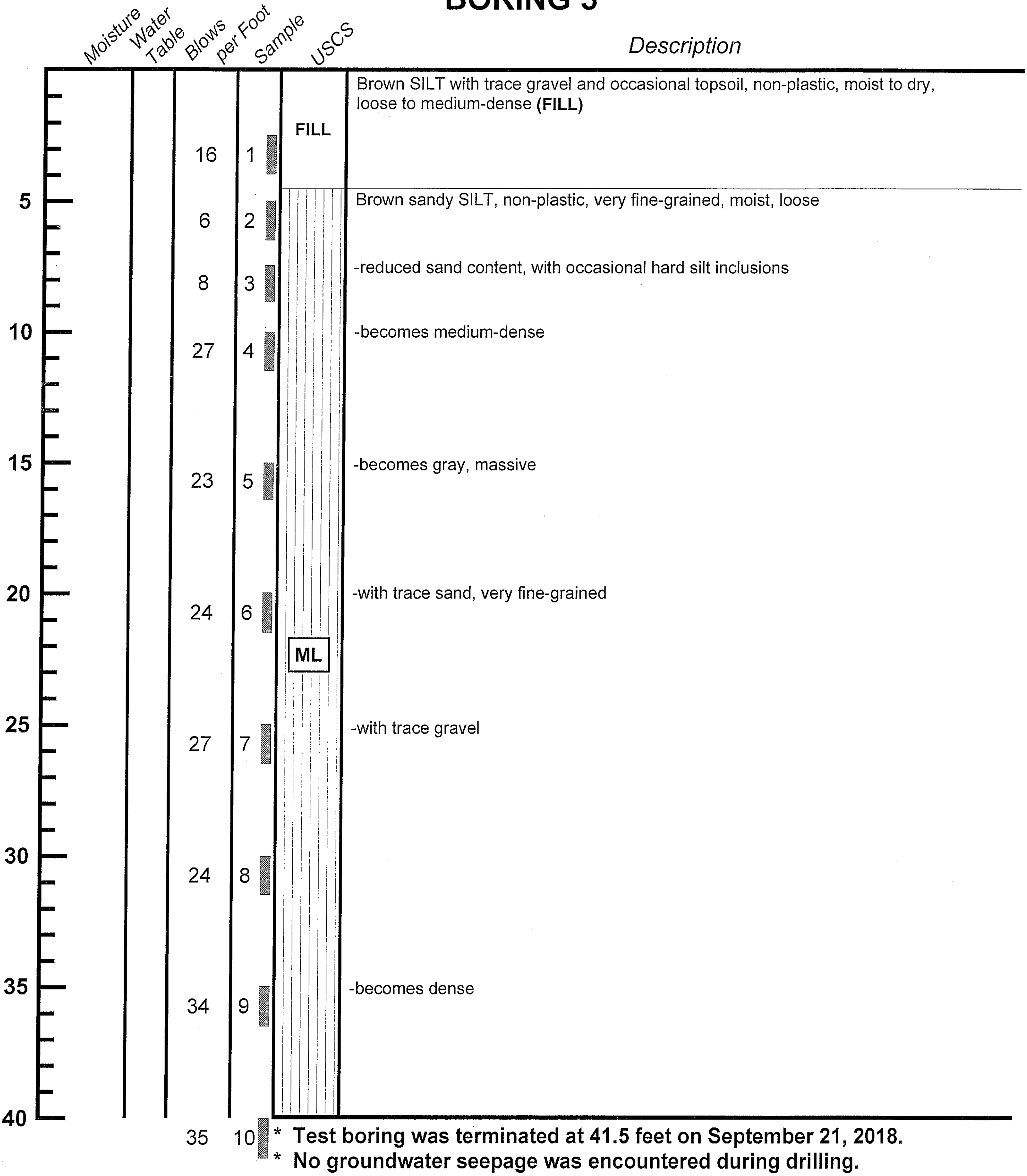
GEOTECH
CONSULTANTS, INC.

BORING LOG

8424 Benotho Place
Mercer Island, Washington

Job	**Date:**	**Logged by:**	**Plate:**
18428	Oct. 2018	ASM	4

BORING 3



Brown SILT with trace gravel and occasional topsoil, non-plastic, moist to dry, loose to medium-dense **(FILL)**

Brown sandy SILT, non-plastic, very fine-grained, moist, loose

-reduced sand content, with occasional hard silt inclusions

-becomes medium-dense

-becomes gray, massive

-with trace sand, very fine-grained

-with trace gravel

-becomes dense

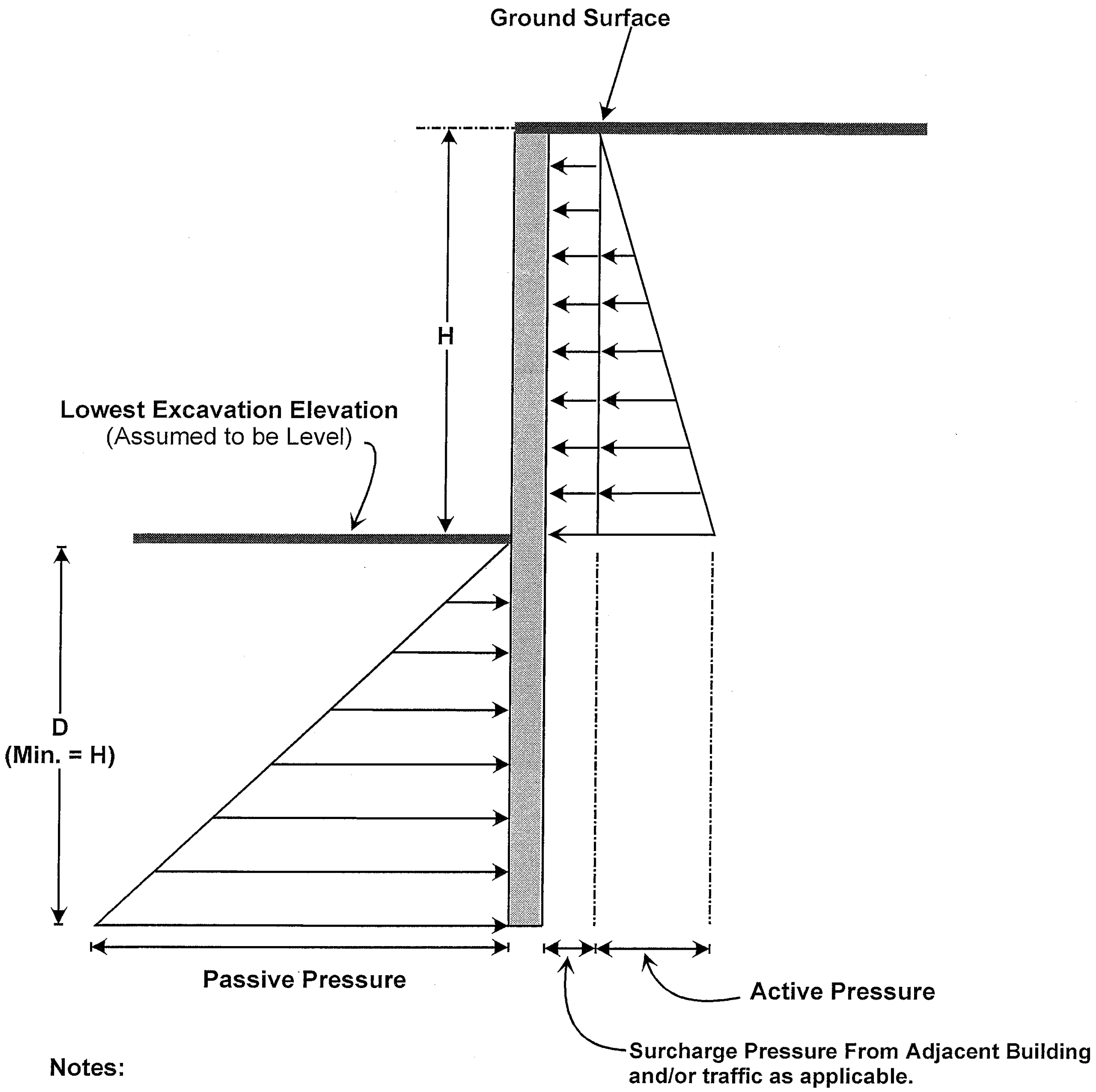
*** Test boring was terminated at 41.5 feet on September 21, 2018.**
*** No groundwater seepage was encountered during drilling.**

GEOTECH
CONSULTANTS, INC.

BORING LOG

8424 Benotho Place
Mercer Island, Washington

Job	*Date:*	*Logged by:*	*Plate:*
18428	Oct. 2018	ASM	5



Notes:

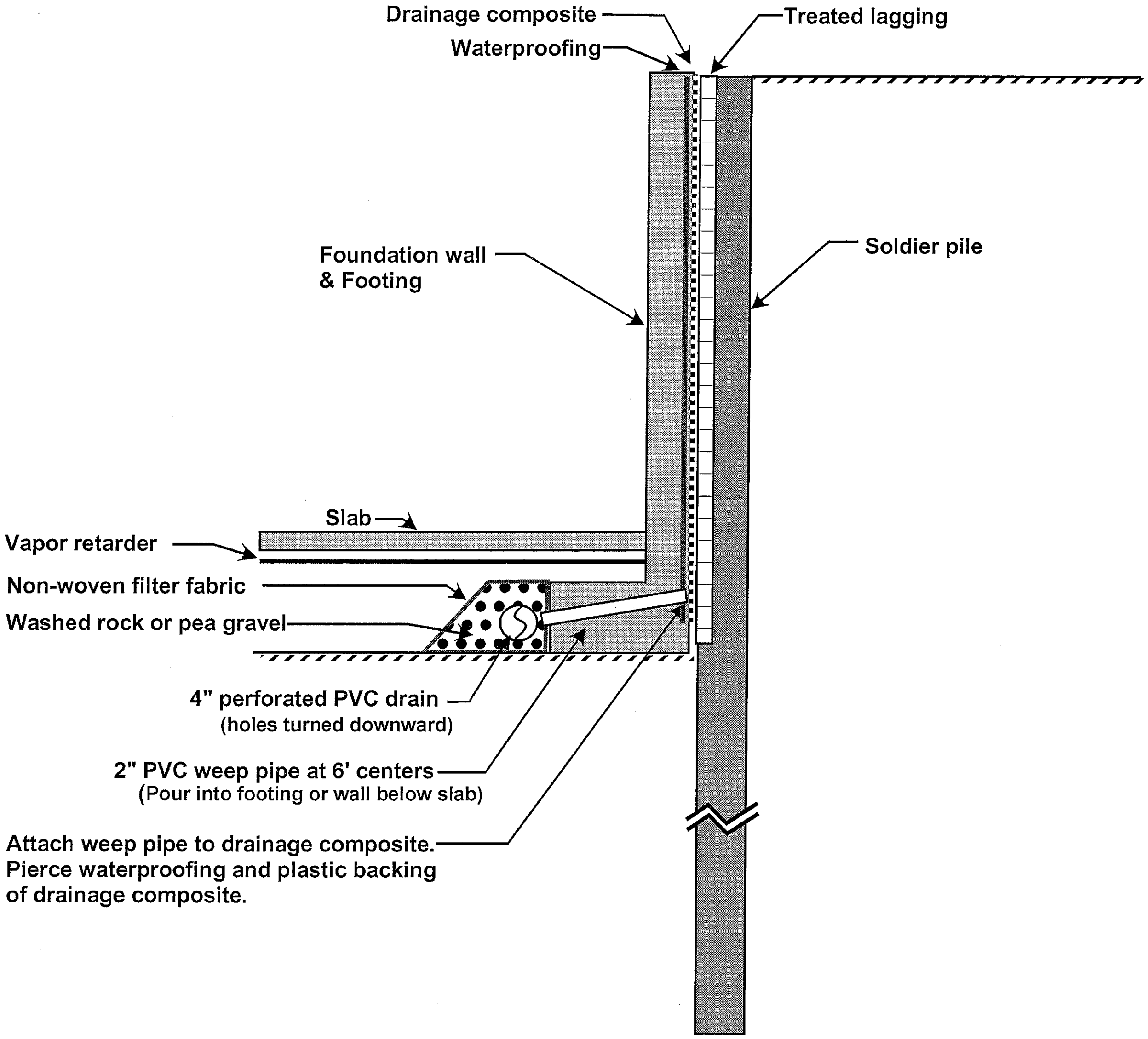
(1) The report should be referenced for specifics regarding design and installation.
(2) Active pressures act over the pile spacing.
(3) Passive pressures act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller.
(4) It is assumed that no hydrostatic pressures act on the back of the shoring walls.
(5) Cut slopes or adjacent structures positioned above or behind shoring will exert additional pressures on the shoring wall.

GEOTECH
CONSULTANTS, INC.

CANTILEVERED SOLDIER PILE WALL DETAIL

8424 Benotho Place
Mercer Island, Washington

Job No: 18428	*Date:* Oct. 2018		*Plate:* 6



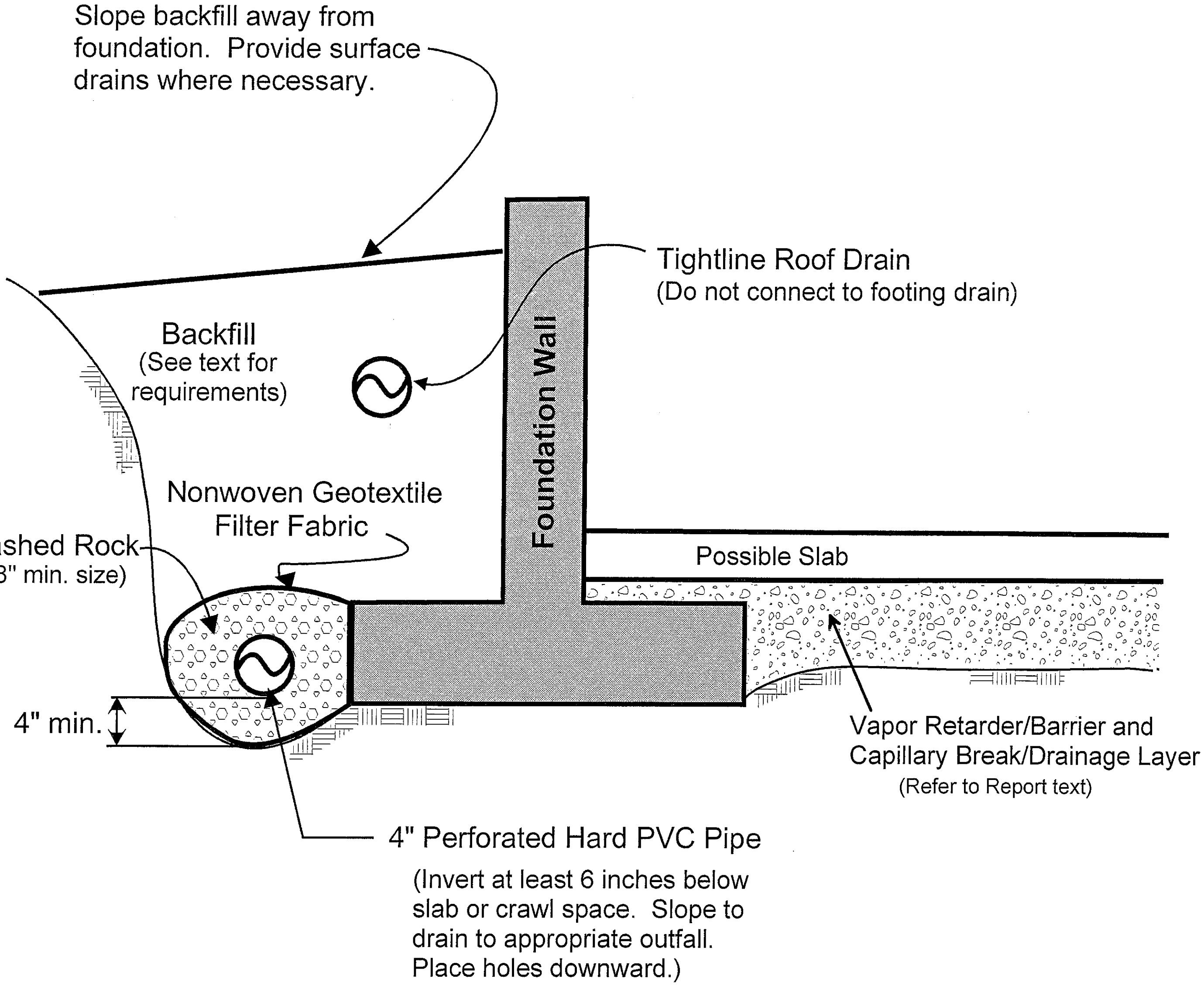
<u>Note</u> - Refer to the report for additional considerations related to drainage and waterproofing.

GEOTECH
CONSULTANTS, INC.

SHORING DRAIN DETAIL

8424 Benotho Place
Mercer Island, Washington

Job No: 18428	*Date:* Oct. 2018		*Plate:* 7



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.

GEOTECH
CONSULTANTS, INC.

FOOTING DRAIN DETAIL

8424 Benotho Place
Mercer Island, Washington

Job No:	Date:		Plate:
18428	Oct. 2018		8