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May 24, 2017

JN 15442

Scott Peyree
6059 – 77th Avenue Southeast
Mercer Island, Washington 98040

via email: speyree@gw-corp.com

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed New Multi-Purpose Building and Addition to Existing Residence
6049 and 6059 – 77th Avenue Southeast
Mercer Island, Washington

Dear Mr. Peyree:

We are pleased to present this geotechnical engineering report for the new multi-purpose building and the addition to an existing 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 and design criteria for foundations, retaining walls, and temporary shoring. This work was authorized by your acceptance of our proposal dated April 21, 2017.

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.

Thor Christensen, P.E.
Senior Engineer

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TRC/MRM:mw

GEOTECHNICAL ENGINEERING STUDY
Proposed New Multi-Purpose building and Addition to Existing Residence
6049 and 6059 – 77th Avenue Southeast
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed new multi-purpose building and the addition to the existing residence in Mercer Island.

We were provided with a topographic survey prepared by Triad dated April 16, 2015. We were also provided with a landscaping plan prepared by Dan Di Zasso dated May 18, 2017. In addition we were provided with floor plans by Gelotte Hommas dated May 10 and 19, 2017.

New #6049 Multi-Purpose Building:
Based on the provided plans, we understand that the existing #6049 house will be demolished and a detached multi-purpose building will be constructed further to the west. The building will have two stories and a basement that daylights toward the west, and will be set back at least 9 feet from the northern property line. The basement will have an elevation of 36.75 feet. An excavation of approximately 20 feet will be required for the east portion of the basement. A covered terrace will be located west of the residence. The terrace will be as much as 6 feet above the existing ground surface, and columns at the west side of the terrace will support upper level decks. A detached garage will be east of the new building. The garage will have a lower level with a floor elevation of 60 feet, as well as an upper level. The lower level will be up to 10 feet below the existing grade. Most of the surface between the building and the new garage will be a paver-covered patio. Several retaining walls with heights of up to about 6 feet are shown north, east, and south of the building; these will mostly retain fills.

A path will wrap around the north side of the new building to provide pedestrian access to the western yard. Fills of up to 4 feet and cuts of up to 2 feet will be made along and adjacent to the path. A stacked block wall is proposed along the north property line to retain the cuts and fills.

A 6-foot concrete wall will be about 25 to 35 feet east of Lake Washington and will roughly parallel the shoreline. The ground surface east of the wall will be raised with fill. A 4-foot-tall slope with an inclination of 2:1 (H:V) will extend above the top of the wall.

Addition to #6059 Residence:
The northeastern garage wing of the existing residence (#6059) will be removed and the residence will be expanded to the northeast with a two-story addition that will not be underlain by a basement. The main level of the addition will be a garage and a pantry, and the second level will be living space.

Driveway:
A new driveway will enter the #6049 property near its northeast corner and proceed to the southwest and provide access to the new building and the remodeled #6059 residence. Fill with a thickness of up to 2 feet will be placed to raise the grade at the northeast end of the driveway. A cut of up to 9 feet will be made at the low, southern portion of the driveway. Retaining walls with heights of up to 9 feet are shown east of the driveway and further to the south, and will mostly face cuts in the existing soil. A new detached garage will be constructed at the south end of the

driveway, east of the existing residence. It will have a floor elevation of 60 feet, up to 8 feet below the existing ground surface.

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 in Mercer Island. The site is bordered to the east by 77th Avenue Southeast, to the west by Lake Washington, and to the north and south by residences.

The site consists of two parcels of land, each of which is currently developed with a residence. The ground surface within the two-lot site slopes gently to moderately down toward Lake Washington to the west. There is a change in elevation of 50 feet across a distance of over 200 feet. The two lots have undergone excavation and filling associated with the previous development with the driveways and homes, and associated on-grade elements. We saw no natural steep slopes taller than 10 feet on, or around, the two properties.

A fairly steep driveway provides vehicular access to the #6059 residence (south parcel), and a somewhat flatter circular driveway accesses the northern #6049 residence (north parcel). Both houses have basements that daylight toward the west. An elevated patio with a pool extends west of the #6059 residence. A small shed is located in the northwest portion of the site, close to the northern property line.

Tiered rockeries with a total height of up to 8 feet are located east of the #6059 driveway, and support cover cuts made below 77th Avenue Southeast. A few stacked block retaining walls are located west of the #6049 residence, and appear to retain fill soils. This was confirmed by borings, as described in a subsequent section.

The western portion of the site is vegetated with grass lawn, which was very wet at the time of our site reconnaissance. Landscaping bushes, ground cover, and evergreen and deciduous trees are located in the remaining undeveloped portions of the site.

A concrete walkway and stairs are located north of the #6049 residence; a concrete retaining wall a few feet tall is located close to the northern property line and retains soil within the site.

We observed a few steel soldier piles just north of the northern property line, which appear to have been used to support an excavation for the adjacent northern residence. That residence is newer, and is located 5 feet north of the common property line.

The Mercer Island Landslide Hazard Assessment map by Kathy Troost and Aaron Wischer dated April 2009 shows that the site has been designated as a Landslide Hazard Area. That map also shows that the entire site has a slope inclination of 15 percent or steeper. Additionally, the Mercer Island Erosion Hazard Assessment map by Kathy Troost and Aaron Wischer dated April 2009 shows that the site has been designated as an Erosion Hazard Area with a slope inclination of 15 to 39 percent.

SUBSURFACE

The subsurface conditions were explored by drilling 10 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 borings were drilled on November 3 and 4, 2015 and May 8 and 9, 2017 using a track-mounted, hollow-stem auger drill and a portable Acker drill that 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 13.

Soil Conditions

In November 2000, our firm completed a geotechnical engineering report for the #6059 house prior to its construction. Three test borings completed previously by another geotechnical firm were utilized for that geotechnical report. Those explorations revealed native soils consisting of several feet of topsoil and weathered, gravelly, silty sand overlying very dense, gravelly, silty sand that has been glacially compressed. This very dense soil is locally referred to as glacial till.

The ground around both houses has undergone varying amounts of grading as a part of development. Test Borings 2, 3, and 4 were located on the sloping ground west of the #6049 residence, and encountered very loose to loose fill and buried topsoil that extended to depths of about 4.5 to 7 feet. This soil does not appear to have been compacted when it was originally placed. Fill can also be expected where flat areas were created over originally sloping ground for patios, sidewalks, landscaping, etc., and where both foundation and retaining walls have been backfilled. Loose retaining wall backfill was encountered in the upper 8 feet of Test Boring 7, located close to the basement retaining wall of the #6049 residence. Test Borings 8 and 9 were in the western yard of the #6049 residence. They revealed 2 to 3.5 feet of loose fill, with wet, loose native soils beneath.

The other borings generally found about a foot of topsoil immediately below the ground surface.

Underlying the fill and topsoil, the borings found native soils that consist of gravelly, silty sand that was generally loose to medium-dense for a few feet before becoming dense to very dense. This dense to very dense, glacially-compressed soil is the glacial till expected from our previous work on the #6059 residence. The till extended to the maximum explored depths of 5.3 to 39.5 feet. The glacial till was so dense that the portable Acker used to conduct borings B-1 through B-5 and B-8 through B-10 was unable to advance more than a few feet into the glacial till.

Boulders were not encountered in the borings. However, it is relatively common to find boulders of varying sizes scattered through glacial till soils.

Groundwater Conditions

Perched groundwater seepage was observed at a depth of 4 to 7 feet in Test Borings 1, 3, and 8 through 10, and at a depth of 24 feet in Test Boring 7. The test borings were 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, particularly following extended wet weather. It is relatively common to encounter at least localized subsurface water perched above and within glacial till soils following prolonged wet weather.

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 very dense glacial till at depths of 2.5 to 10 feet. The till was overlain by several feet of fill on the sloped ground west of the existing #6049 residence and in the lower part of the western yard of that residence. Foundations for the proposed new construction should bear on the dense to very dense native soils. Some over-excavation should be anticipated to reach those competent soils in about the western half of the multi-purpose building. Alternatively, small-diameter pipe piles could be installed into the very dense soil to support the west half of the new building. The slab for the west half of the building should be designed as a structural slab unless the existing loose soil is removed from the building footprint and replaced with structural fill. Our subsurface explorations encountered groundwater seepage at depths above the proposed excavation. The wet subgrade soils will need to be protected with at least 6 inches of clean crushed rock.

About 6 feet of fill will be placed below the proposed western terrace and a lesser amount of fill will be placed below the west edge of the new building, where loose soil is present. Instead of removing the existing loose soil and extending the foundations that support the terrace and building down to the dense native soil, small-diameter pipe piles could be installed through the fill to support the terrace and building. This may be more economical, especially if the west part of the building is supported with pipe piles. If the terrace will be supported with pipe piles the terrace floor should be constructed as a structural slab supported with pipe piles.

It will be very difficult to use the silty site soils as structural fill due to their moisture sensitivity. It is common for settlement of on-grade elements, such as patios, to result from inadequate compaction of a deep fill.

The proposed western retaining wall alignment is underlain by about 6 feet of loose soil that will need to be removed from the foundation subgrade. That over-excavation will likely encounter groundwater; it can be filled with quarry spalls that are compacted with an excavator bucket. The ***Conventional Foundations*** section provides additional recommendations for over-excavation.

The proposed northern detached garage and several retaining walls up to 5.5 feet tall will overlie the basement of the existing #6049 residence. The proposed grade over about the western two-thirds of that basement is about 6 inches above the existing slab elevation, and in the eastern third it is up to 10 feet higher. The foundations for those structures should extend down to the underlying medium-dense native soil, or be placed on compacted rock fill (quarry spall or railroad ballast rock) laid over native bearing soils.

The addition to the northeast side of the #6059 residence will partially overlie existing backfill that was placed against the retaining wall for the existing basement. This fill is likely loose and is not capable of supporting the addition. Foundations within 10 feet of existing basement retaining walls should be supported with pipe piles or be designed as grade beams to span across the loose backfill.

The glacial till soils that underlie the site have high strengths and are not landslide-prone. They are also not susceptible to seismic liquefaction. It is our opinion that the proposed development will not adversely impact other critical areas, the subject property or adjacent properties. Also, our recommendations are intended to mitigate impacts to the geologic hazard areas consistent with best available science to the maximum extent reasonably possible, such that the development area is determined to be safe. All disturbed areas around the building areas will be landscaped for permanent erosion protection.

There are no steep natural slopes on the site taller than 10 feet. The recommendations of this report are intended to prevent adverse impacts to slope stability on the site or the adjoining properties. The proposed development is located in the most suitable spot from a geologic standpoint and to minimize site disturbance. 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 recommended in this report for the new development should render the development as safe as if it were not located in a geologic hazard area."

Temporary excavations in existing fill and loose to medium-dense native soils can have an inclination as steep as 1:1 (H:V). Cuts into the underlying dense glacial till can have an inclination of 0.75:1. These cuts should be continuously sloped from top to bottom. Where excavations cannot be contained within the site, and where temporary easements to extend excavations into adjacent properties cannot be obtained, shoring will be necessary. Additional shoring may be desired to limit the volume of excavated soil and the resulting retaining wall backfill, and to allow increased construction access. We anticipate that shoring will need to be installed along portions of the northern side of the #6049 building.

The glacial till soils that underlie the site have extremely low permeability and are not suitable for infiltration of stormwater; this is illustrated by the very wet lawn in the lower, west portion of the site, and the presence of perched groundwater in the borings. The wet lawn condition could be improved by installing extensive drainage below the lawn. A landscape architect could provide consultation concerning reducing the amount of water in the yard.

We recommend installing underslab drainage and a vapor barrier below the basement slab to reduce the potential for moisture to rise into finished living spaces. A typical underslab drainage detail is presented as Plate 16.

A significant geotechnical consideration for development of this site is the moisture sensitivity of the silty soils. These fine-grained, silty soils are sensitive to moisture, which makes them impossible to adequately compact when they have moisture contents even 2 to 3 percent above their optimum moisture content. The reuse of these soils as structural fill to level the site will only be successful during dry weather. Imported granular fill will be needed wherever it is not possible to use the on-site soils.

It is likely that some settlement of the ground surrounding pile-supported portions of 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.

- 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.

The glacial till soils are not highly erodible. The erosion control measures needed for the disturbed areas during the site development will depend heavily on the weather conditions that are encountered during the site work. 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. However, we have been associated with numerous waterfront projects 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. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil 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. One or two wire-backed silt fences 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.

Seepage into the site excavation should be expected. As a result, excavation dewatering will be needed. Until the base of the excavation is covered with a layer of clean crushed rock, the water pumped from the excavation will likely be silty. It may be possible to discharge this silty water to the sanitary sewer, with permission from the governing agency. Alternatively, the water would have to be held in a storage tank until it is clear enough to discharge to the storm sewer.

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 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.

As with any project that involves demolition of existing site buildings and/or extensive excavation and shoring, there is a potential risk of movement on surrounding properties. This can potentially translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. However, the demolition, shoring, and/or excavation work could just translate into *perceived* damage on adjacent properties. 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. Therefore, 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, any adjacent structures 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.

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 soil profile within 100 feet of the ground surface is best represented by Site Class Type C (Very dense 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 site soils are not susceptible to seismic liquefaction under the strong ground shaking assumed for the Maximum Considered Earthquake (MCE) because of their dense nature.

CONVENTIONAL FOUNDATIONS

We recommend that continuous and individual spread footings have minimum widths of 12 and 16 inches, respectively. Exterior footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface for protection against frost and erosion. The local building codes should be reviewed to determine if different footing widths or embedment depths are required. Footing subgrades must be cleaned of loose or disturbed soil prior to pouring concrete. Depending upon site and equipment constraints, this may require removing the disturbed soil by hand.

Depending on the final site grades, overexcavation may be required below the footings to expose competent native soil. Unless lean concrete is used to fill an overexcavated hole, the overexcavation must be at least as wide at the bottom as the sum of the depth of the overexcavation and the footing width. For example, an overexcavation extending 2 feet below the bottom of a 2-foot-wide footing must be at least 4 feet wide at the base of the excavation. If lean concrete is used, the overexcavation need only extend 6 inches beyond the edges of the footing.

An allowable bearing pressure of 3,000 pounds per square foot (psf) is appropriate for footings supported on competent native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be about one-inch, with differential settlements on the order of one-half-inch in a distance of 30 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level, well-compacted fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.50
Passive Earth Pressure	350 pcf

Where: pcf is Pounds per Cubic Foot, and Passive Earth Pressure is computed using the Equivalent Fluid Density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at

PIPE PILES

Three- or 4-inch-diameter pipe piles driven with a 650- or 800- or 1,100-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (650-pound hammer)	FINAL DRIVING RATE (800-pound hammer)	FINAL DRIVING RATE (1,100-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
3 inches	12 sec/inch	10 sec/inch	6 sec/inch	6 tons
4 inches	20 sec/inch	15 sec/inch	10 sec/inch	10 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 200 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. We recommend a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate passive value.

If lateral resistance from fill placed against the foundations is required for this project, the structural engineer should indicate this requirement on the plans for the general and earthwork contractor's information. Compacted fill placed against the foundations can consist of on-site or 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

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 backfill:

PARAMETER	VALUE
Active Earth Pressure * -level backslope	35 pcf
Active Earth Pressure * -backslope inclined steeper than 3:1 (H:V)	55 pcf
Passive Earth Pressure	350 pcf
Coefficient of Friction	0.50
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.**

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 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 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 for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

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

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. If the native silty sand is used as backfill, a minimum 12-inch width of free-draining gravel should be placed against the backfilled 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 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. 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 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 build up 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***, ***Slabs-On-Grade***, 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

The building floors can be constructed as slabs-on-grade atop competent native soil, or on structural fill, or they can be pile supported. Where loose soil is present and will not be removed, as at the west edge of the proposed multi-purpose building, the slab should be designed as a structural slab supported with pipe piles or conventional foundations. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

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 also notes that vapor *retarders* such as 6-mil plastic sheeting have been used in the past, but are now recommending a minimum 10-mil thickness for better durability and long term performance. 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.

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the fill and loose soil at the subject site would generally be classified as Type B, and the dense glacial till would be classified as Type A. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical) in the loose soils and 0.75:1 (H:V) in the glacial till soils. These temporary cut slopes should be inclined continuously from top to bottom, with no vertical cut faces. Flatter cut slopes or additional drainage measures would be needed if caving or substantial seepage was observed in the looser, near-surface soils.

The above-recommended temporary slope inclinations are based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that sand or loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger. These recommendations may need to be modified if the area near the potential cuts has been disturbed in the past by utility installation, or if settlement-sensitive utilities are located nearby.

All permanent cuts into native soil should be inclined no steeper than 2:1 (H:V). Fill slopes should not be constructed with an inclination steeper than 2:1 (H:V). To reduce the potential for shallow sloughing, fill must be compacted to the face of these slopes. This can be accomplished by overbuilding the compacted fill and then trimming it back to its final inclination. Adequate compaction of the slope face is important for long-term stability and is necessary to prevent excessive settlement of patios, slabs, foundations, or other improvements that may be placed near the edge of the slope.

Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

TEMPORARY SHORING

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. The contractor should be prepared to case the holes or use the slurry method if caving soil or heavy seepage 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 30 pounds per cubic foot (pcf).

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. 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 450 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. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

DRAINAGE CONSIDERATIONS

We anticipate that permanent foundation walls will 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.

Footing drains placed inside 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. Plate 14 presents typical considerations for footing drains and Plate 15 presents a shoring drain detail. All roof and surface water drains must be kept separate from the foundation drain system.

If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** 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 even a few inches of free draining gravel underneath the vapor retarder limits the potential for seepage to build up on top of the vapor retarder.

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 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. Water from roof, storm water, and foundation drains should not be discharged onto slopes; it should be tightlined to a suitable outfall located away from any slopes.

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, behind permanent retaining or foundation walls, 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.

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. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can 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 relative compactions for structural 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).

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

ADDITIONAL SERVICES

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

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

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 13	Test Boring Logs
Plate 14	Typical Footing Drain Detail
Plate 15	Typical Shoring Drain Detail
Plate 16	Typical Underslab Drainage Detail

Marc McGinnis

From: Ed Kim <edkim@hotelconcepts.us>
Sent: Tuesday, May 23, 2017 1:52 PM
To: Marc McGinnis
Cc: Swank
Subject: 708 S 288th Lane Federal Way

swank@sbcglobal.net

Hi Marc,
The address is shown above. You can go there anytime to take a look and we can talk afterwards. Feel free to just walk around the house to the area in question.

Our neighbor to the left as you see from the street (Ian Swank) want to either meet with you at least have you look at his side. I'll leave it to him to contact you directly. He's copied on this email. thanks.

--
Ed Kim 206-441-5155

Ian Swank
(661) 505-5855 (c)

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.

Thor Christensen, P.E.
Senior Engineer

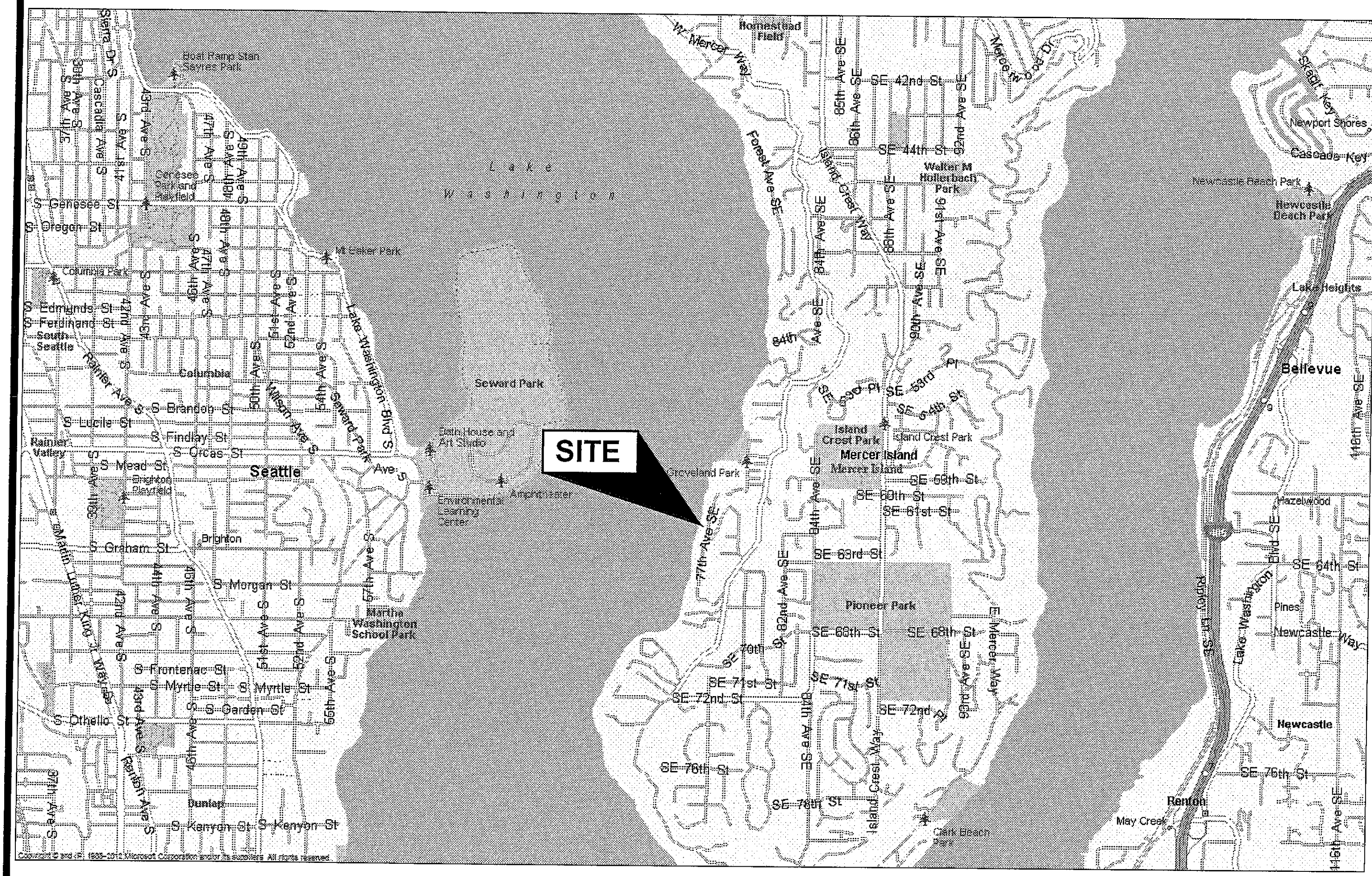
MARC R. McGINNIS
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER

5/24/17

Marc R. McGinnis, P.E.
Principal

TRC/MRM:mw

NORTH



(Source: Microsoft MapPoint, 2013)

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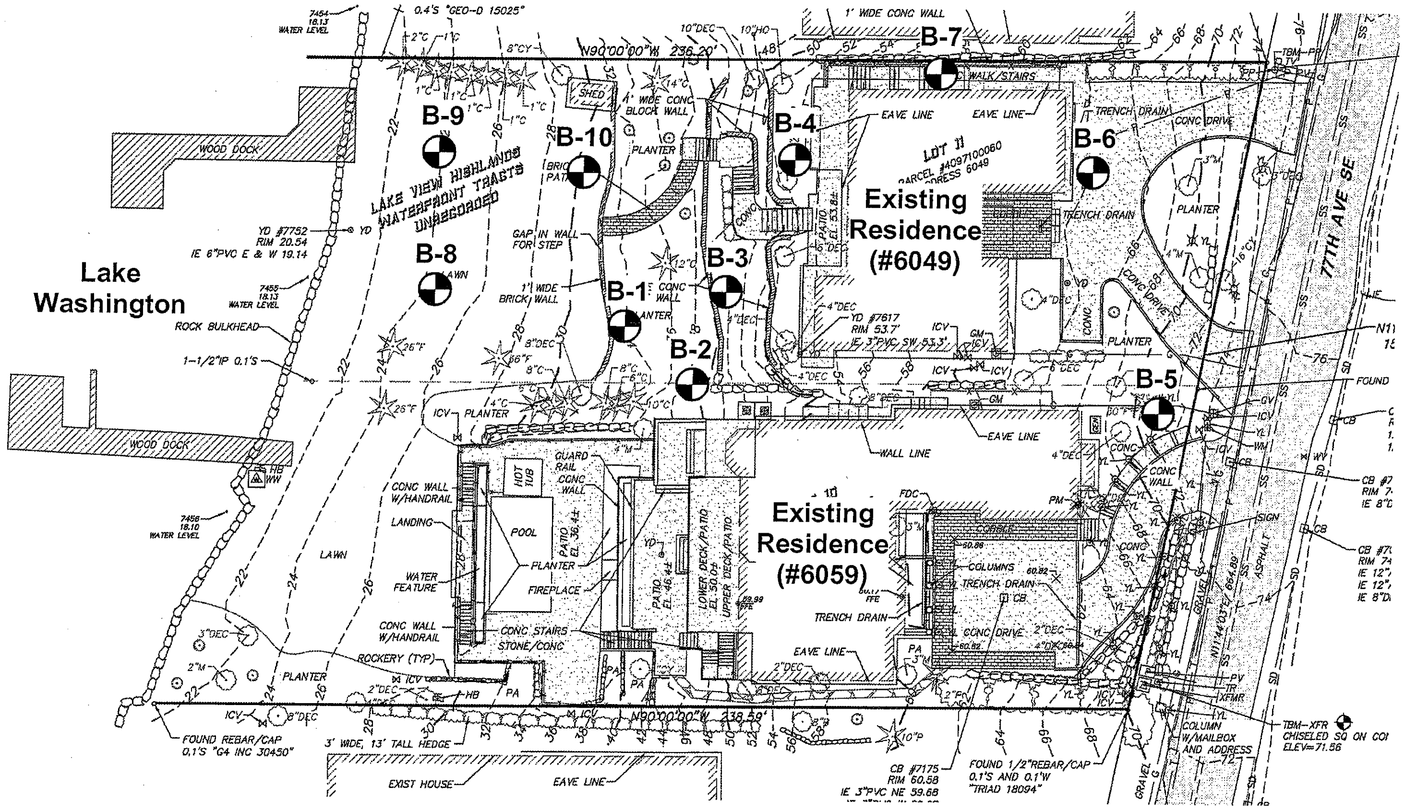
VICINITY MAP

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
15442	May 2017		1



Legend:

Test Boring Location

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SITE EXPLORATION PLAN

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
15442	May 2017	No Scale	2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Topsoil over: Gray-brown silty SAND, fine to coarse-grained, moist, loose to medium-dense -becomes very dense **(TILL)**
		▼	80/ 11.5"	1		
5			50/3"	2		
10						

* **Test boring was terminated on November 3, 2015 at 5.3 feet.**
* **Perched groundwater was encountered at 4 feet during drilling.**

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	April 2016	TRC	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Topsoil over:
						Brown silty SAND with gravel, fine to coarse-grained, wet, very loose **(FILL)**
			3	1		
5			77/ 9"	2	SM	Gray-brown silty SAND with gravel, fine to coarse-grained, moist, very dense **(TILL)**
			50/5"	3		
10						

*** Test boring was terminated on November 3, 2015 at 7.5 feet.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	April 2016	TRC	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Topsoil over: Gray-brown silty SAND with gravel, fine to coarse-grained, wet, very loose **(FILL)**
			3	1		
5			2	2		
		▼				
			82/ 11"	3	SM	Gray-brown silty SAND with gravel, fine to coarse-grained, moist, very dense **(TILL)**
10			50/5"	4		

* **Test boring was terminated on November 3, 2015 at 10 feet.**
* **Perched groundwater was encountered at 7 feet during drilling.**

GEOTECH CONSULTANTS, INC.

TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	April 2016	TRC	5

BORING 4

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Topsoil over: Gray-brown silty SAND with gravel, fine to coarse-grained, wet, loose **(FILL)**
			7	1		
						Topsoil
5			8	2	SM	Brown silty SAND with gravel, fine to coarse-grained, moist to wet, loose
			72	3		-becomes very dense, gray, and moist **(TILL)**
10						

*** Test boring was terminated on November 3, 2015 at 9 feet.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	April 2016	TRC	6

BORING 5

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Topsoil over: Brown silty SAND with gravel, fine to coarse-grained, wet, loose
			6	1		
5			18	2		-becomes medium-dense and rust-brown mottled gray-brown
			81	3		-becomes very dense and gray-brown **(TILL)**
10						

*** Test boring was terminated on November 3, 2015 at 9 feet.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG
6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	April 2016	TRC	7

BORING 6

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	6 inches of concrete over: Rust-brown mottled gray-brown silty SAND with gravel, fine to coarse-grained, moist, medium-dense
			17	1		
5			62	2		-becomes very dense **(TILL)**
10			50/5"	3		-becomes gray-brown
15			97/ 9"	4		
			50/2"	5		-becomes gray
20						

*** Test boring was terminated on November 4, 2015 at 18.2 feet.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	April 2016	TRC	8

BORING 7

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						10 inches of concrete over:
					FILL	Gray SAND with gravel, coarse-grained, moist, very loose **(FILL)**
5			3	1		
10			40	2	SM	Gray-brown silty SAND with gravel, fine to coarse-grained, moist, dense
15			74/ 11.5"	3		-becomes very dense **(TILL)**
20						

Continued on Plate 10

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	May 2017	TRC	9

BORING 7 (cont'd)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
20			28	4	SM	Gray-brown silty SAND with gravel, fine to coarse-grained, moist, medium-dense to dense **(TILL)**
		▼			SW	Gray-brown SAND with gravel, fine to coarse-grained, very wet, very dense
25			83/ 10"	5	SM	Gray-brown silty SAND with gravel, fine to coarse-grained, moist, very dense **(TILL)**
			100/ 7"	6		
30						

*** Test boring was terminated on May 8, 2017 at 39.5 feet.**
*** Perched groundwater was encountered at 24 feet during drilling.**

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	May 2017	TRC	10

BORING 8

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Grass and topsoil over: Brown silty SAND with gravel, fine to coarse-grained, very moist, loose **(FILL)**
		▼	7	1	SM	Brown slightly silty SAND with gravel, fine to coarse-grained, very moist, loose
5						-becomes rust-brown mottled gray-brown
			16	2		-becomes medium-dense at 6 feet
			37	3		-becomes dense **(TILL)**
10			66	4		-becomes very dense
15						

*** Test boring was terminated on May 9, 2017 at 11.5 feet.**
*** Perched groundwater was encountered at 4 feet during drilling.**

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	*Date:*	*Logged by:*	*Plate:*
15442	May 2017	TRC	11

BORING 9

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Grass and topsoil over: Brown slightly silty SAND with gravel, fine to coarse-grained, wet, loose **(FILL)**
		▼	7	1	SM	Brown slightly silty SAND with gravel, fine to coarse-grained, wet, loose
5			29	2		-becomes medium-dense, very moist, and gray-brown
			47	3		-becomes dense **(TILL)**
10			78	4		-becomes very dense
15						

* **Test boring was terminated on May 9, 2015 at 11 feet.**
* **Perched groundwater was encountered at 4 feet during drilling.**

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TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	*Date:*	*Logged by:*	*Plate:*
15442	May 2017	TRC	12

BORING 10

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Grass and topsoil over: Rust-brown mottled gray-brown slightly silty SAND with gravel, fine to coarse-grained, very moist, loose to medium-dense
			30	1		-becomes dense **(TILL)**
		▼				
5			50/ 3"	2		-becomes very dense and gray-brown
10						

* **Test boring was terminated on May 9, 2017 at 5.3 feet.**

* **Perched groundwater was encountered at 4 feet during drilling.**

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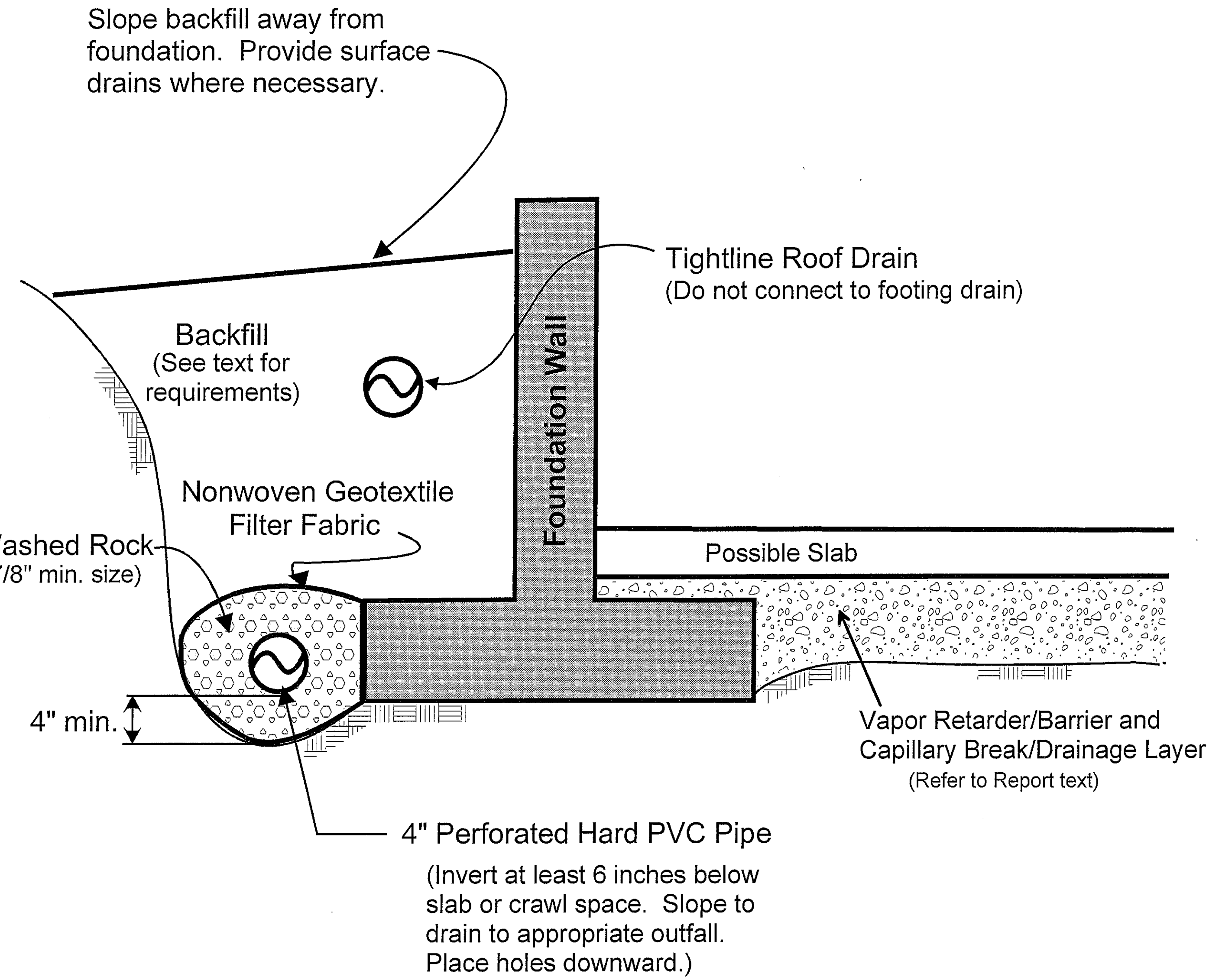
TEST BORING LOG

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
15442	May 2017	TRC	13



NOTES:

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

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

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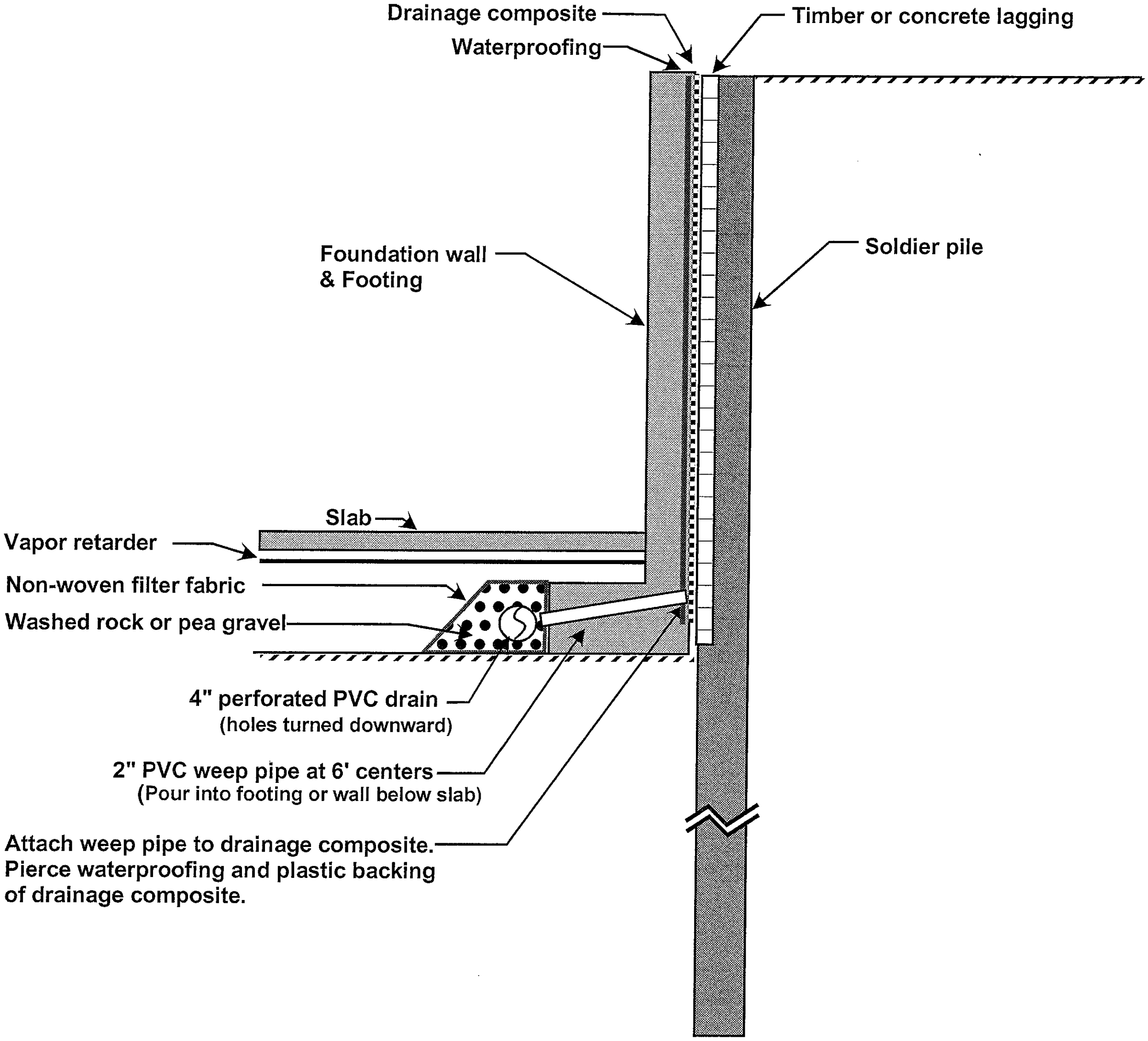
FOOTING DRAIN DETAIL

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
15442	May 2017		14



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

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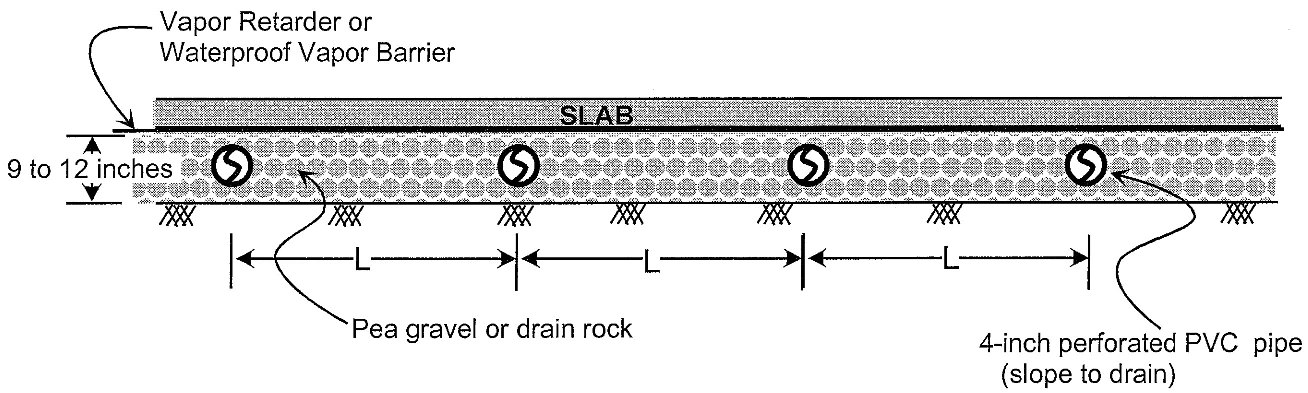
SHORING DRAIN DETAIL
6059 - 77th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
15442	May 2017		15



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

6059 - 77th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
15442	May 2017		16