**GEOTECH**
**CONSULTANTS, INC.**

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

August 19, 2025

JN 21026

Scott Chancellor
*via email:* *scott.chancellor@gmail.com*

Subject: **New Project Update**
Proposed Single-Family Residence
4045 West Mercer Way
Mercer Island, Washington

Greetings:

We completed a geotechnical engineering study for a previous single-family residence project at the subject site dated June 18, 2021. We also prepared a letter regarding updates to the project and a review of the project plans dated August 12, 2022. In recent months we have been informed that a new client has purchased the property, and that a new residence design has been prepared. This letter is with regards to our past work done on the project, recent correspondence with the project design team, and a review of the new residence plans.

The new project plans we have reviewed in preparation of this letter include: 1) Sheets S1.0 through S4.3, which were prepared by Gary Gill, SE dated August 18, 2025, 2) Sheets A0.1 through A8.1, which were prepared by McClellan Tellone Architects dated August 12, 2022 and 3) Sheets C01 through C10, which were prepared by Facet dated July 30, 2025.

From a geotechnical engineering perspective, the design of the new residence is quite similar to the original residence design, with the residence having somewhat of an L-shape and an attached, north-south-tending garage and an entry driveway located on the upper, northwestern (project northwestern) corner of the property. The main portion of the residence will extend in the east-west direction and have a finish floor about 20 feet below the elevation of the garage floor and driveway grade. The geotechnical engineering parameters provided in the study and the August 12, 2022 letter, include, among other parameters, that the foundations be supported by concrete and/or driven steel piles, shoring be used for large excavations, and that catchment walls be used on the upslope sides of the garage and/or residence that face the steep slope on the upslope, northern side of the site. Because the currently planned residence is similar to the original residence, the parameters we provided are very appropriate for the current residence. In our professional opinion, the new plans we have reviewed conform to the recommendations in our geotechnical engineering study and also in the August 12, 2022 letter.

Per Mercer Island Code, a statement of risk for this project is needed. As such, we provide the following statement:

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

We trust that this letter is suitable for the project at this time. If there are questions regarding this letter, or if we can be of further service, please contact us.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

D. Robert Ward, P.E.
Principal

D. ROBERT WARD
STATE OF WASHINGTON
29740
REGISTERED
PROFESSIONAL ENGINEER

8/19/2025

cc: **McClellan Architects** – Joey Pasquinelli & Regan McClellan
*via email:* *joey@mccarch.com* & *regan@mccarch.com*

GEOTECH
CONSULTANTS, INC.

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

June 18, 2021

JN 21026

Mist LLC
7683 Southeast 27th Street, #418
Mercer Island, Washington 98040

Attention: Feras Alrouk
*via email:* *mist_llc@hotmail.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study and Critical Area Study**
Proposed Single-Family Residence
4045 West Mercer Way
Mercer Island, Washington

Dear Mr. Alrouk:

Attached to this transmittal letter is our geotechnical engineering report and Critical Area Study related to geologic hazards 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 excavations/shoring. This work was authorized by your acceptance of our proposal, P-10755, dated November 24, 2020.

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: **McClellan Architects** – Joey Pasquinelli & Regan McClellan
*via email: joey@mccarch.com* & *regan@mccarch.com*

ASM/DRW:kg

**GEOTECHNICAL ENGINEERING STUDY AND CRITICAL AREA STUDY**
**Proposed Single-Family Residence**
**4045 West Mercer Way**
**Mercer Island, Washington**

This report presents the findings and recommendations of our geotechnical engineering study and Critical Area Study for the site of the proposed single-family residence to be located in Mercer Island. The scope of the Critical Area Study is intended to satisfy the requirements of the recently-adopted section 19.07.110 of the Mercer Island City Code (MICC), which applies to Critical Area Studies.

We were provided with preliminary plans and a topographic map. McClellan Architects developed the provided plans, which are dated May 10, 2021. The topographic map was developed by Terrane and dated February 12, 2021. For clarity, this report will reference Project North depicted on the provided plans and our attached Exploration Plan (Plate 2), in which Lake Washington is located directly south of the subject site.

Based on the provided preliminary plans, an L-shaped single-family residence will be constructed near the center of the property. The main portion of the residence will be roughly rectangular, with a width of 18 to 26 feet, and will span east to west through the center of the parcel; this three-story portion of the residence will have a lowest floor (daylight basement) with a finished floor elevation near 28 feet. The main (second) and upper (third) floors of the residence will also extend north (stepping up the subject site's sloping ground surface) from the western perimeter of the main residence to create the building's L shape. The northern end of this northwestern wing of the residence will contain a garage with a floor slab matching the residence's upper (third) floor, the existing ground surface, and western adjacent access driveway at an elevation of 48 feet. The western and eastern ends of the building will have 7- and 10-foot setbacks from the adjacent property lines. The residence will be offset approximately 50 feet from Lake Washington along the southern property line, and the northern perimeter of the garage will have a setback of 11 feet from the northern property line. Access to the property will come from the shared access road on the tract parcel that borders the site to the west.

As discussed above, the proposed residence will generally match existing ground surface which slopes downwards to the south across the property towards Lake Washington. However, the upslope northern perimeter of the garage will be cut approximately 4 to 6 feet beneath the existing ground surface; the upslope northern perimeter of the main residence's daylight basement will be cut 8 to 14 feet into the sloping 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 on the center of the western perimeter of Mercer Island. The subject site is located on Lake Washington and is separated from West Mercer Way by several parcels. The rectangular-shaped subject site has a width of 100 feet in the east-west direction and a depth of approximately 121 to 130 feet in the north-south direction along the western and eastern property lines, respectively. The concrete foundation of a previous small residence is located in the center of the property. The remainder of the property is predominantly covered by grass and small brush.

The subject site is located along the toe of the slope that descends from West Mercer Way down to Lake Washington. The northern third of the parcel contains the toe of a steep slope, which continues to rise to the north onto the northern adjacent property. Based on the contour lines on the City of Mercer Island's online GIS mapping tool, this steep slope has an overall inclination of approximately 70 to 80 percent over a height of 50 to 60 feet. This area would be considered a steep slope landslide hazard area under Mercer Island's Municipal Code; this is discussed further below. However, in the northeast corner of the subject site, the steep slope rises 23 to 24 feet to a relatively flat "bench" in the center of the aforementioned steep slope.

The southern two-thirds of the subject site continues to slope moderately downwards to the south (at an inclination of approximately 20 to 25 percent) from the toe of the steep slope to a 4- to 5-foot-tall rock bulkhead along Lake Washington and the southern property line. Based on the provided topographic survey, a sewer easement passes through the southern end of the subject site and a second utility easement is located on the property along its eastern property line. The eastern utility easement is 10 feet wide, but expands to 25 feet wide along the southern end of the eastern property line.

The City of Mercer Island's GIS tool maps the subject site within several geologically hazardous areas. The majority of the site is mapped to lie within a seismic hazard area, a potential landslide hazard area and an erosion hazard area. These three hazard areas all encompass the generally vicinity between West Mercer Way and Lake Washington. The northern end of the subject site is also mapped as the toe of a steep slope which rises to the north onto the adjacent property. The southern-facing steep slope continues to the east and west onto the neighboring parcels.

We did not observe any indications of recent slope instability on or around the site during our recent visit to the property. However, on the Mercer Island Landslide Hazard Map (Troos and Wisher, 2009) the subject site is located a just downslope of a historic landslide scarp and within the downslope mass wasting deposit that extends into Lake Washington. This headscrap/mass wasting area extends continuously several blocks to the north and south of the site.

The subject site is bordered by residential properties to the north and east. As discussed above, the upper half of a steep slope covers the southern end of the northern adjacent parcel. A residence is located on the eastern adjacent property with an approximate 10-foot-setback from the subject site. A tract parcel containing an access road, small parking area, utilities, and shoreline access borders the subject site to the west. The driveway and parking area are elevated above the original ground surface; a soldier pile retaining wall is located along the downslope perimeter of the parking area. The soldier pile wall along the downslope perimeter of the raised parking area, with a height of up to 6 feet, is offset approximately 0.6 to 0.9 feet from the subject site's western property line. However, a power transformer associated with the tract parcel is located east of the parking area on the subject site. A small wooden retaining wall wraps around the downslope edges of the power transformer and extends approximately 9 feet into subject site. As discussed above, the subject site is bordered by Lake Washington to the south.

***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 February 4, 2021 using both a 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- or 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 through 5.

***Soil Conditions***

The test borings conducted on the subject site encountered variable depths of loose fill and colluvium soils beneath the ground surface; colluvium is the geologic term for landslide soils. Dense, interbedded layers of glacially compressed silty sand, sand, and silt soils were revealed at depths of 7 to 9.5 feet below the ground surface (and the loose soils). Very dense glacially compressed sandy silt was encountered below depths of 18 to 10 feet beneath the upslope northern and downslope southern ends of the site, respectively.

***Groundwater Conditions***

Perched groundwater seepage was encountered within the cleaner sand seams in our test borings. Thin perched groundwater seams were revealed at 8.5 and 17.5 feet in Test Boring 1, conducted on the upslope, northern end of the property; Test Boring 3, conducted in the southeast quadrant of the property, encountered a groundwater seam from 6.3 to 7.7 feet, perched above the underlying very dense glacially compressed sandy silt. 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. 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. We anticipate that groundwater could be found in the looser near-surface soils, perched on top of and in sand seams within the relatively impervious underlying dense glacially compressed soils. This is most likely to occur following extended 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, AND CRITICAL AREAS STUDY INFORMATION

### *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 dense to very dense, glacially compressed silty sand/sandy silt underlying the subject site. However, loose fill soils and loose colluvium in the range of 7 to 9 feet thick were encountered above these competent underlying soils. The dense to very dense soil is very competent to support building loads and is resistant to soil movement.

Based on the test borings and the depth of excavations needed at the north and northwestern portions of the lower basement and at the northern edge of the garage, the use of footing foundations could be done provided they bear on competent soil or structural fill placed over the competent soil. Some relatively minor overexcavations may be needed in some areas to reach the competent soil; other recommendations regarding the preparation of footings are given in the ***Conventional Foundations*** section of this report. However, due to the depth of loose upper soils and the sloping ground surface, overexcavations to reach competent soil for footings would be significant beneath the rest of the building's footprint; therefore, we recommend the remainder of the residence and attached garage be supported on deep foundations embedded into the dense glacially compressed soils below. Driven small-diameter pipe piles are typically the most economical deep foundation system for relatively lightly-loaded residences. We recommend any settlement sensitive elements such as floor slabs in living areas and exterior stairs, retaining walls, or patios also be supported on small-diameter pipe piles. If some risk of cracking of the garage floor slab can be tolerated, it may be constructed on grade over the existing soils; however, if no cracking due to settlement can be tolerated, it should also be supported on piles.

Excavation for the proposed residence's basement and the upslope northern perimeter of the garage will be an important geotechnical consideration for the project. Based on the provided plans, the excavation for the upslope perimeter of the proposed basement will extend approximately 8 to 14 feet beneath the existing ground surface; excavations along the northern perimeter of the garage will extend 4 to 6 feet below grade into the toe of the slope to the north. Temporary open cut slopes may be excavated at an inclination no steeper than 1:1 (Horizontal:Vertical) in the onsite soils. These cuts also cannot extend below a 2:1 (H:V) from existing retaining walls, utilities, driveways, etc. Slope cuts may be feasible along the southern and eastern perimeters of the proposed residence. It should be determined if the presence of the existing utility easements onsite will affect excavations. However, excavation along the northern perimeter of the garage, and the northern and western perimeters of the proposed basement will require temporary shoring to make the cuts into the toe of the slope and to protect the neighboring property. It will be important not to undermine the existing retaining walls on the adjacent tract parcel to the west of the subject site. The site and soil conditions make cantilevered soldier piles the most appropriate shoring system for this project. The soldier piles could also be used to provide permanent vertical and lateral support for the new residence.

Another very significant geotechnical engineering consideration for this project is the proximity of the proposed residence to the steep slope that rises to the north; it will impact the design and construction of the residence. As previously discussed, the core of the steep slope is comprised of dense to very dense glacially compressed silty sand and sandy silt not susceptible to deep-seated movement. However, as with any steep slope on Mercer Island and in the Puget Sound region, there is the possibility of movement of the loose near-surface soils, particularly after periods of extended precipitation. Therefore, because of the potential for near-surface soil movement, we recommend a catchment wall be constructed along the full northern perimeter of the residence and garage exposed to the adjacent steep slope. A catchment wall is a heavily reinforced concrete wall extending above the finished grade along the toe of the slope to retain soil conveyed to the base of the slope by landslides. The necessary height of the catchment wall depends on the height, inclination, and soil stratigraphy of the adjacent slope, as well as the proximity of the residence to the toe of the slope. Because of this, the required height of the catchment wall varies across the northern perimeter of the residence. Based on our analysis, it is our professional opinion that the top of the catchment wall should extend a minimum of 7 feet above the existing grade along the northern perimeter of the proposed garage. The catchment wall height can be reduced to 6 feet above the existing grade along the remainder of the northern perimeter of the residence to  It will be necessary to remove any landslide soil if it accumulates behind the catchment structure -- the freeboard of the catchment wall must be maintained to provide continued protection from future landslides. The wall only needs to extend the length of the new residence. Note that the wall can only offer protection in its downslope shadow and slides can flow around the ends of the wall. The intention of the catchment wall is to provide a reasonable degree of protection from a reasonable amount of soil that descends the hillside. The catchment wall is not intended to provide protection from large trees that can descend the hillside with a slide or on their own.

## ***CRITICAL AREAS STUDY (MICC 19.07)***

**Seismic Hazard and Potential Landslide Hazard Areas:** The entire subject site is located within both a mapped Seismic Hazard Area and a Potential Landslide Hazard area. Both geologic hazard areas cover much of the general vicinity. As previously discussed, the core of the subject site consists of dense to very dense, glacially compressed, native silty sand, sand, and silt that has a low potential for deep-seated landslides. The mapping of the Potential Landslide Hazard Area is due to the inference by geologists that the site lies within an ancient landslide, which most likely occurred following the recession of the last glaciers approximately 13,000 years ago.  No recent large-scale movement has been documented in this area.  The proposed development will be supported on conventional footings or deep foundations embedded into the glacially compressed soils which are not liquefiable, due to their dense nature and the absence of near-surface groundwater.  This mitigates the Seismic Hazard.

**Steep Slope Hazard Areas:** The southern two-thirds of the site slopes downwards towards Lake Washington to the south at an inclination of 20 to 25 percent. However, based on the provided topographic map of the subject site and the contour lines on the City of Mercer Island's GIS mapping tool, approximately the northern third of the subject site has an inclination of at least 40 percent over a horizontal distance of 30 feet (which the City of Mercer Island code defines as a Steep Slope). A Steep Slope is a qualification as a Landslide Hazard Area under the Mercer Island Code. The Steep Slope continues onto the northern adjacent property, and has an overall height of approximately 50 to 60 feet based on the contour lines on Mercer Island's GIS map. The new residence will be constructed into the sloping ground surface along the toe of the steep slope to the north, and will be supported on either conventional footings bearing directly on the underlying dense to very dense glacially compressed soils, or on deep foundations embedded into these competent glacially compressed soils, which are not susceptible to deep-seated movement per the recommendations in this report. It is our opinion that no buffers or setbacks are needed from this Steep Slope, provided the recommendations presented in this report are followed. The recommendations presented in the report are intended to prevent adverse impacts to the stability of the slope onsite.

**Erosion Hazard Areas:** The site also meets the City of Mercer Island's criteria for an Erosion Hazard Area. We have worked on numerous waterfront projects on Mercer Island that have avoided siltation of the lake and surrounding properties by exercising care and being proactive with the maintenance and potential upgrading of the erosion control system through the entire construction process. The location of the site on the shore of Lake Washington will make proper erosion control implementation important to prevent adverse impacts to the lake. The temporary erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered during the site work. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil areas to prevent accumulated water or runoff from the work area from becoming silty in the first place. Silty water cannot be discharged to the lake, so a temporary holding tank should be planned for wet weather earthwork. A wire-backed silt fence bedded in compost, not native soil or sand, should be erected as close as possible to the planned work area, and the existing vegetation between the silt fence and the lake left in place. Typically, if wet weather construction is anticipated, two parallel silt fences should be installed along the shoreline. Rocked construction access and staging areas should be established wherever trucks will have to drive off of pavement, in order reduce the amount of soil or mud carried off the property by trucks and equipment. It will also be important to cap any existing drain lines found running toward the lake until excavation is completed. This will reduce the potential for silty water finding an old pipe and flowing into the lake. Covering the base of the excavation with a layer of clean gravel or rock is also prudent to reduce the amount of mud and silty water generated. Utilities reaching between the house and the lake should not be installed during rainy weather, and any disturbed area caused by the utility installation should be minimized by using small equipment. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface.

**Buffers and Mitigation:** Under MICC 19.07.160(C), a prescriptive buffer of 25 feet is indicated from all sides of a shallow landslide-hazard area (we believe that the landslide hazard on this site is only shallow). As noted above, the entire site lies within a mapped Potential Landslide Hazard Area, and the prescriptive buffer would extend far beyond the boundaries of the property and the planned development area. It is our professional opinion that the location of the proposed development is sufficient without any buffer from the Steep Slope, provided the recommendations presented in this report are followed. In particular, the inclusion of a catchment wall along the upslope perimeter of the proposed residence and the use of foundations that bear on or into dense to very dense core soils are intended to address the proximity to the Steep Slope. We recognize that the planned development will occur within the Landslide Hazard area; the recommendations presented in this geotechnical report are intended to allow the project to be constructed in the proposed configuration without adverse impacts to critical areas on the site or the neighboring properties. The geotechnical recommendations associated with foundations, shoring, and erosion control will mitigate any potential hazards to critical areas on the site.

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

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

Wet weather construction (October 1 through March 31) on this site should be possible without adverse impacts to the surrounding properties. The above section entitled **Erosion Hazard Areas** covers temporary erosion control measures that would be prudent. 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.

All, or the vast majority, of the excavated soil will be unsuitable for reuse on the site because they are very silty, fine-grained, and overly moist. Because of this, they have poor drainage characteristics and low compacted strength, and will present an erosion control problem. As a result, we expect that excavated soils will be hauled off the site, and imported granular fill will be needed for the project.

We anticipate that onsite stormwater infiltration will be considered for the project. However, the underlying glacially compressed soils are essentially impervious and will stop downward percolation of large volumes of water infiltrated above it. This is a common problem throughout the Pacific Northwest. Also, the upper soils at the site are loose and could be destabilized by the infiltration of stormwater into them. Considering this, it is our professional opinion that onsite infiltration of stormwater is not feasible for the subject site.

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.

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

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 1.42g and 0.50g, 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 MCE peak ground acceleration adjusted for site class effects ($F_{PGA}$) equals 0.67g. The proposed residence will be supported on conventional footings or on deep foundations embedded into the dense to very dense underlying glacially compressed soils beneath the subject site, which are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature.

## *CONVENTIONAL FOUNDATIONS*

The northern perimeter foundation of the proposed basement and attached garage may be supported on conventional continuous footings bearing on native, undisturbed, dense, silty sand/sandy silt, or on structural fill. Prior to placing structural fill beneath foundations, the excavation should be observed by the geotechnical engineer to document that adequate bearing soils have been exposed. We recommend any structural fill placed beneath the footings consist of 2- to 4-inch quarry spalls placed in maximum 12-inch lifts and compacted with either a hoe pack or tamping with a large excavator bucket. Excavation using a toothed bucket usually leaves several inches of disturbed soils. The loosened soil must be entirely scraped out of the base of the footing excavations. This should be accomplished with a flat-bladed bucket, a grade bar that is dragged with the bucket, or by hand-shoveling the loose soil out of the excavation. The competent bearing soils beneath the site are very moisture sensitive due to their high fines content; thus, if the footing subgrade soil is wet, or becomes wet at the time of foundation construction, we recommend covering the bearing surfaces with several inches of clean crushed rock immediately after the excavation is completed. This is intended to protect the footing subgrade soils from becoming softened by foot traffic during the footing forms and rebar placement, which will be a particular concern during wet conditions.

We recommend that continuous footings have a minimum width of 16 inches. 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 native, dense silty sand/sandy silt or on structural fill (consisting of compacted 2- to 4-inch quarry spalls) placed on top of these c0mpetent native soils. 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, or on structural fill up to 5 feet in thickness, will be about one inch, with differential settlements on the order of one-half inch in a distance of 50 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 | 0.40 |
| Passive Earth Pressure | 300 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. The above ultimate values for passive earth pressure and coefficient of friction do not include a safety factor.

***PIPE PILES***

Three-, 4-, or 6-inch-diameter pipe piles driven with a 1,100-, 2,000-, or 3,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

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

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

As a minimum, Schedule 40 pipe should be used. 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 300 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. 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 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. Due to their small diameter, the lateral capacity of vertical pipe piles is relatively small. However, if lateral resistance in addition to passive soil resistance is required, we recommend driving battered piles in the same direction as the applied lateral load. The lateral capacity of a battered pile is equal to one-half of the lateral component of the allowable compressive load. The allowable vertical capacity of battered piles does not need to be reduced if the piles are battered steeper than 1:5 (Horizontal:Vertical).

## *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 *<br>- Northern perimeter foundation walls (sloped backfill)<br>- All other walls (level backfill) | <br>60 pcf<br><br>40 pcf |
| Passive Earth Pressure | 300 pcf |
| Coefficient of Friction | 0.40 |
| 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. We can provide appropriate surcharge loads once more detailed plans have been developed. It may be possible for the excavation shoring to be designed to withstand this surcharge. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

### ***Wall Pressures Due to Landslides***

As discussed in the ***General*** section, we recommend the design of the northern perimeter of the proposed residence be designed with a catchment wall; a permanent shoring wall could be used for this purpose. Based on our analysis, it is our professional opinion that the top of the catchment wall should extend a minimum of 7 feet above the existing grade along the northern perimeter of the proposed garage and 6 above the existing along the remainder of the northern perimeter of the residence. We anticipate that future slides would occur as a mudflow striking the catchment wall. We recommend that an active equivalent fluid pressure of 80 pounds per cubic foot (pcf) should be used in the design of the catchment wall to account for the impact force. Because this impact load is temporary, the safety factor against sliding and overturning of 1.2 is needed for the catchment wall.

***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 9**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.

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.

***Retaining Wall Backfill and Waterproofing***

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

As discussed in the ***General*** section, the floor slabs of the pile-supported residence and the exterior patios should be designed as structural slabs spanning between the pile-supported grade beams. If some risk of cracking of the garage slab can be tolerated, the garage slab may be constructed on grade over existing soils. We do recommend placing these concrete slabs over at least 1 foot of structural fill to provide more uniform support for the slab where the subgrade is soft or settles more rapidly than the surrounding ground. Also, slabs in the pipe pile areas should be reinforced with a minimum mat of number 4 rebar at 16 inches each way to reduce slab cracking and breakage. Isolation joints should be provided where the slabs intersect columns and walls. Control and expansion joints should also be used to control cracking from expansion and contraction. Saw cuts or preformed strip joints used to control shrinkage cracking should extend through the upper one-fourth of the slab. The spacing of control or expansion joints depends on the slab shape and the amount of steel placed in it. Reducing the water-to-cement ratio of the concrete and curing the concrete, by preventing the evaporation of free water until cement hydration occurs, will also reduce shrinkage cracking. If no cracking due to settlement can be tolerated in these, slabs, they would need to be structurally supported on the piles.

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

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

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

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

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

## *EXCAVATIONS AND SLOPES*

Temporary excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Also, temporary cuts should be planned to provide a minimum 2 to 3 feet of space for construction of foundations, walls, and drainage. Unless approved by the geotechnical engineer of record, it is important that vertical cuts not be made at the base of sloped cuts. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut.

The above-recommended temporary slope inclination is 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 greater than 2.5:1 (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.

## *EXCAVATION SHORING*

Information regarding a cantilevered soldier pile system is given in this section. This system has proven to be an efficient and economical method for providing excavation shoring where the depth of excavation is less than 15 feet.

### ***Soldier Pile Installation***

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 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.

***Soldier Pile Wall Design***

Temporary or permanent 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). However, for shoring walls along the northern perimeter of the residence and garage along toe of the slope to the north, this active pressure should be increased to 60 pcf to account for the slope. If the soldier piles will permanently restrain soil loads, a uniform seismic surcharge load of 9**H** pounds per square foot (psf), where **H** is the design retention height of the wall should also be included in the design. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis. Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet.

As discussed in the ***General*** section, we recommend the design of the northern perimeter of the proposed residence be designed with a catchment wall; a permanent shoring wall could be used for this purpose. Based on our analysis, it is our professional opinion that the top of the catchment wall should extend a minimum of 7 feet above the existing grade along the northern perimeter of the proposed garage and 6 above the existing along the remainder of the northern perimeter of the residence. We anticipate that future slides would occur as a mudflow striking the catchment wall. We recommend that an active equivalent fluid pressure of 80 pounds per cubic foot (pcf) should be used in the design of the catchment wall to account for the impact force. Because this impact load is temporary, the safety factor against sliding and overturning of 1.2 is needed for the catchment wall.

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. For permanent walls, we recommend a minimum factor of safety of 1.5 be applied to overturning and sliding calculations when using this ultimate value (temporary installations may use a factor of safety of 1.2) 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."

The soldier piles embedded into the dense underlying glacially compressed soils may also be used to provide permanent vertical support of the residence. The vertical capacity of soldier piles will be developed by frictional shaft resistance along the embedded length.

| PARAMETER | DESIGN VALUE (Allowable) |
|---|---|
| Pile Shaft Friction | 2,000 psf |

**Where: psf is Pounds per Square Foot.**

The above value is for the portion of the pile embedded into the dense underlying glacially compressed soils. The concrete surrounding the embedded portion of the pile must have sufficient bond and strength to transfer the vertical load from the steel section through the concrete into the soil.

***Tieback Anchors***

We recommend installing tieback anchors at inclinations between 20 and 30 degrees below horizontal. The tieback will derive its capacity from the soil-grout strength developed in the soil behind the no-load zone. The minimum grouted anchor length should be 10 feet. The no-load zone is the area behind which the entire length of each tieback anchor should be located. To prevent excessive loss-of-ground in a drilled hole, the no-load section of the drilled tieback hole should be backfilled with a sand and fly ash slurry, after protecting the anchor with a bond breaker, such as plastic casing, to prevent loads from being transferred to the soil in the no-load zone. The no-load section could be filled with grout after anchor testing is completed.

During the design process, the possible presence of foundations or utilities close to the shoring wall must be evaluated to determine if they will affect the configuration and length of the tiebacks.

Based on the results of our analyses and our experience at other construction sites, we suggest using an adhesion value of 2000 psf in the dense native soil to design temporary anchors, if the mid-point of the grouted portion of the anchor is more than 10 feet below the overlying ground surface. For permanent anchors, this adhesion value should be decreased to 1700 psf. These values apply to non-pressure-grouted anchors. Pressure-grouted or post-grouted anchors can often develop adhesion values that are two to three times higher than that for non-pressure-grouted anchors. These higher adhesion values must be verified by load testing.

Soil conditions, soil-grout adhesion strengths, and installation techniques typically vary over any site. This sometimes results in adhesion values that are lower than anticipated. Therefore, we recommend substantiating the anchor design values by load-testing all tieback anchors. At least two anchors in each soil type encountered should be performance-tested to 200 percent of the design anchor load to evaluate possible anchor creep. Wherever possible, the no-load section of these tiebacks should not be grouted until the performance tests are completed. Unfavorable results from these performance tests could require increasing the lengths of the tiebacks. The remaining anchors should be proof-tested to at least 135 percent of their design value before being "locked off." After testing, each anchor should be locked off at a prestress load of 80 to 100 percent of its design load.

If caving or water-bearing soil is encountered, the installation of tieback anchors will be hampered by caving and soil flowing into the holes. It will be necessary to case the holes, if such conditions are encountered. Alternatively, the use of a hollow-stem auger with grout pumped through the stem as the auger is withdrawn would be satisfactory, provided that the injection pressure and grout volumes pumped are carefully monitored.

All drilled installations should be grouted and backfilled immediately after drilling. No drilled holes should be left open overnight.

## *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 6 presents typical considerations for foundation drains at shoring walls.

Footing drains should be placed inside the building for shored areas noted above, outside of the building in non-shored areas, or behind backfilled walls. 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. The footing drains 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. A typical footing drain detail is attached to this report as Plate 7. 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.

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. 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 native on-site soils are not suitable for reuse as structural fill, due to their high fines content and moisture sensitivity. The onsite gravelly, slightly silty sand fill soils could potentially be re-used as structural fill provided they can be placed and compacted near their optimum moisture content.

Fills placed on sloping ground should be keyed into the medium-dense to dense native soils. This is typically accomplished by placing and compacting the structural fill on level benches that are cut into the competent soils. The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches, but should be thinner if small, hand-operated compactors are used. We recommend testing structural fill as it is placed. If the fill is not sufficiently compacted, it should be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction.

The following table presents recommended levels of relative compaction for compacted fill:

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

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

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

## ***LIMITATIONS***

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the subsurface explorations are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings and test holes. 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.

The recommendations presented in this report are directed toward the protection of only proposed structures from damage due to slope movement. Predicting the future behavior of steep slopes and the potential effects of development on their stability is an inexact and imperfect science that is currently based mostly on the past behavior of slopes with similar characteristics. Landslides and soil movement can occur on steep slopes before, during, or after the development of property. The owner of any property containing, or located close to steep slopes must ultimately accept the possibility that some slope movement could occur. However, such movement will not affect the development if the recommendations in this report are followed.

This report has been prepared for the exclusive use of Mist LLC 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 - 5 | Test Boring Logs |
| Plate 6 | Shoring Foundation Drain Detail |
| Plate 7 | 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.

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Geotechnical Engineer

06/18/2021
D. Robert Ward, P.E.
Principal

ASM/DRW:kg

NORTH


(Source: Microsoft MapPoint, 2013)

**GEOTECH**
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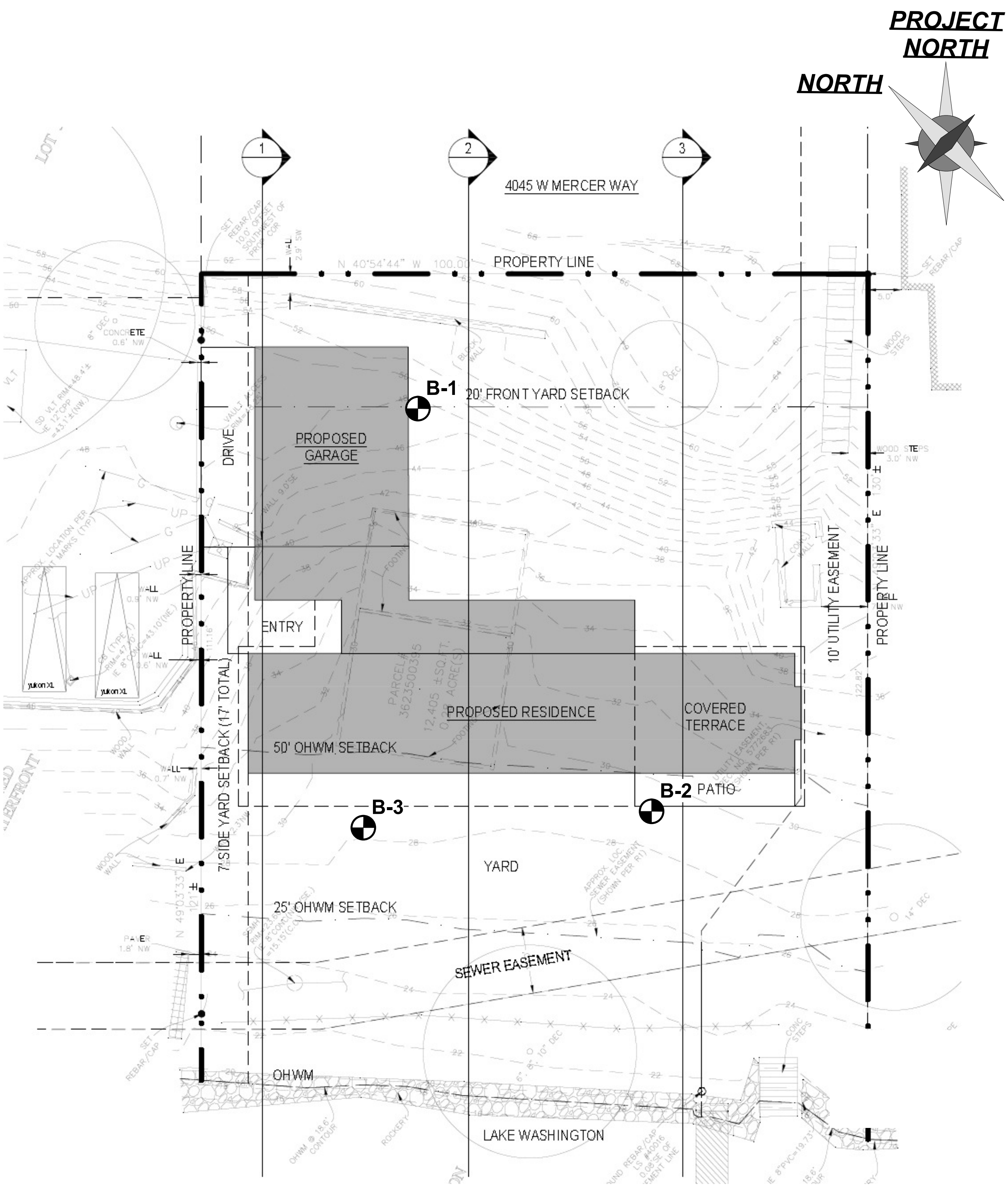
# VICINITY MAP

4045 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 21026 | May 2021 | | 1 |




**Legend:**

- Test Boring Location

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# SITE EXPLORATION PLAN

4045 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 21026 | May 2021 | No Scale | 2 |

# BORING 1

Elevation ±48 feet

| Depth (ft.) | Moisture | Water Table | Blows per Foot | Sample | USCS | Description |
|---|---|---|---|---|---|---|
| | | | | | FILL | Dark-brown gravelly silty SAND with organics and wood debris, fine- to medium-grained, moist, loose **(FILL)** |
| | | | 6 | 1 | SP | Gray-brown SAND with silt, fine-grained, moist to very moist, loose (Colluvium) |
| 5 | | | 19 | 2 | ML | Gray sandy SILT with trace fine-grained gravel, non-plastic, very fine-grained, moist, medium-dense **(Colluvium)**<br>-with occasional sand lenses<br>-with piece of wood |
| | | ▽ | 41 | 3 | SM | Gray silty SAND with gravel, fine- to medium-grained, moist, dense<br>-with wet sand seam |
| 10 | | | 45 | 4 | SP | Gray SAND with silt and occasional gravel, fine- to medium-grained, very moist, dense<br>-with decomposed wood fragments |
| | | | 29 | 5 | SM | Gray silty SAND with gravel, fine-grained, moist, dense<br>-with occasional rust mottling |
| 15 | | | 37 | 6 | | |
| | | ▽ | | | | -with thin wet sand seam |
| | | | 68 | 7 | ML | Gray SILT, non-plastic, moist, very dense |
| 20 | | | 50/5" | 8 | | -becomes sandy in part |
| 25 | | | 83 | 9 | | |

*** Test boring was terminated at 26.5 feet on February 4, 2021.**

*** Slight perched groundwater seepage was observed at 8.5 and 17.5 feet during drilling.**

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## TEST BORING LOG

4045 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | Logged by: | Plate: |
|---|---|---|---|
| 21026 | May 2021 | ASM | 3 |

# BORING 2

Elevation ±29 feet

| Depth (ft.) | Moisture | Water Table | Blows per Foot | Sample | USCS | Description |
|---|---|---|---|---|---|---|
| | | | | | FILL | Dark-brown gravelly silty SAND with organics, fine- to medium-grained, moist, loose **(FILL)** |
| | | | 5 | 1 | FILL | Brown gravelly SAND, fine- to coarse-grained, moist, loose **(FILL)** |
| 5 | | | 17 | 2 | ML | Gray-brown sandy SILT with decomposed wood, non-plastic, jumbled, moist, loose **(Colluvium)**<br>-becomes very moist to wet, with occasional gravel and reduced wood content |
| | | | 12 | 3 | | -with occasional rootlets<br>-becomes fractured, reduced sand content<br>-becomes loose, with occasional gravel, increased sand content |
| 10 | | | 27 | 4 | ML | Gray slightly sandy SILT, non-plastic, moist, medium-dense to dense |
| 15 | | | | | | |
| 20 | | | | | | |
| 25 | | | | | | |

* **Test boring was terminated at 11.5 feet due to refusal on February 11, 2021.**
* **No groundwater seepage was observed during drilling.**

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## TEST BORING LOG

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Mercer Island, Washington

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|---|---|---|---|
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# BORING 3

Elevation ±29 feet

| Depth (ft.) | Moisture | Water Table | Blows per Foot | Sample | USCS | Description |
|---|---|---|---|---|---|---|
| | | | 7 | 1 | FILL | Brown SAND with trace gravel and organics, fine- to medium-grained, moist, loose **(FILL)** |
| 5 | | | 7 | 2 | SP/SM | Gray-brown with rust mottling, slightly silty SAND, fine-grained, very moist, loose **(Colluvium)**<br>-grades to SAND with silt and gravel, becomes wet |
| 10 | | | 43 | 3 | ML | Gray sandy SILT with occasional organics, non-plastic, fine-grained, jumbled, moist, medium-dense<br>-becomes dense, massive, reduced organics and sand content |
| | | | 72 | 4 | | -becomes very dense |
| 15 | | | | | | |
| 20 | | | | | | |
| 25 | | | | | | |

*** Test boring was terminated at 11.5 feet due to refusal on February 11, 2021.**

*** Slight perched groundwater seepage was observed from 6.3 to 7.7 feet during drilling.**

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## TEST BORING LOG

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Mercer Island, Washington

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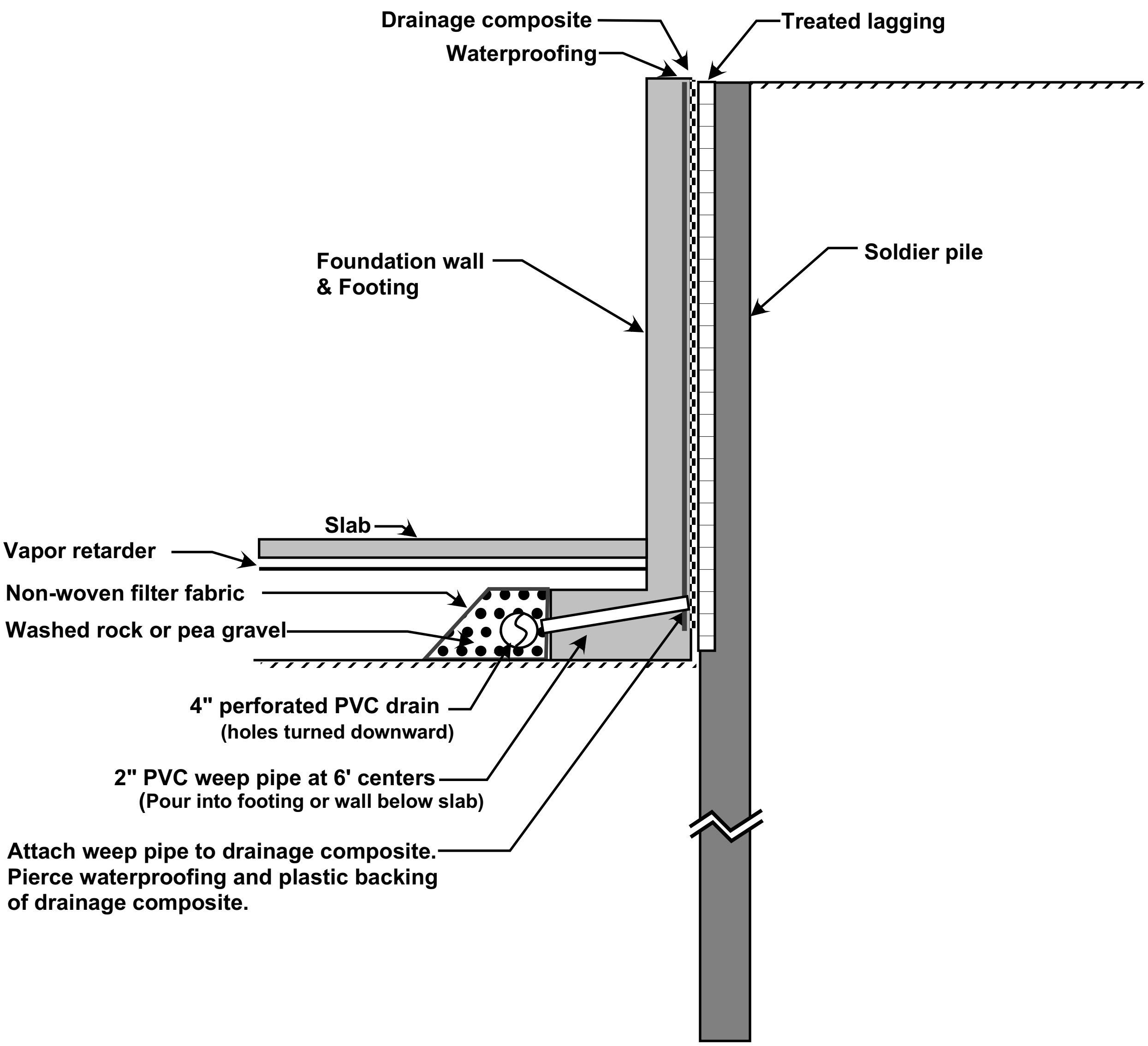


__Note__ - Refer to the report for additional considerations related to drainage and waterproofing.

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# SHORING DRAIN DETAIL

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Mercer Island, Washington

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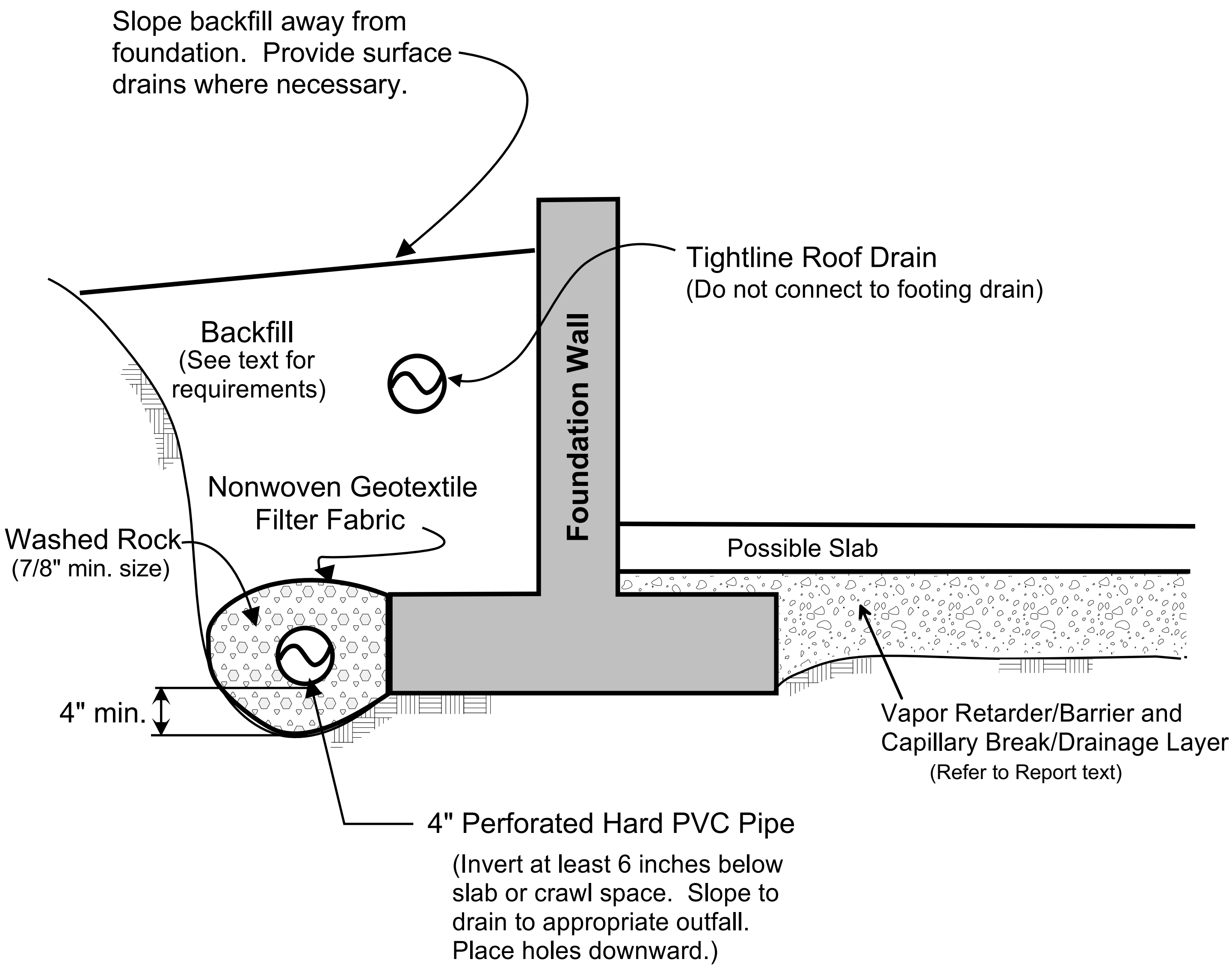


**NOTES:**

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

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

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**FOOTING DRAIN DETAIL**

4045 West Mercer Way
Mercer Island, Washington

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