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Subject: **Geotechnical Report**
Proposed Remodel and Expansion of Existing Residence
5419 – 96th Avenue S.E.
Mercer Island, Washington

Greetings:

This report presents our geotechnical engineering findings and recommendations for the proposed remodel and expansion of the existing residence 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 considerations for foundations, slope stability, retaining walls, subsurface drainage, and temporary excavations. This work was authorized by your acceptance of our proposal, P-11971.

We were provided with architectural drawings prepared by Ripple Design Studio dated 23, July 2025, and a topographic survey prepared by Terrane dated 6/20/25. Based on the provided information, we expect that the existing home will be remodeled and expanded. Off the south end of the house, the existing exterior main floor deck will be replaced by a two-story addition containing a studio, pantry, and mudroom on the main floor, and a bedroom and exercise room on the basement level. The footprint of this addition will extend eastward of the existing deck footprint. Outdoor storage space will be created between the basement of this southern addition and the existing concrete retaining wall that currently supports the existing two-car garage and the exterior parking pad. The main floor deck on the east side of the house will be rebuilt in a different configuration. The plans also indicate that the west side of the existing garage will be expanded by a few feet, and a third garage bay will be created in the area currently occupied by the exterior parking pad.

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 eastern side of Mercer Island. The subject property is irregular in shaped, with the curved western and eastern property lines formed by a private driveway and 96th Avenue S.E., respectively. The private drive extends off 96th Avenue S.E. to serve both the subject residence and the house to the south (#5425). The existing residence on the site consists of one level overlying an east-facing daylight basement. This structure is located on the western half of the site. An attached, two-car garage is located at the southwest corner of the house. A tall backfilled retaining wall extends along the east side of the garage, and appears to provide support for the eastern garage wall. The remainder of the garage was likely built on compacted fill placed behind the retaining wall. This retaining wall continues south of the garage, and then returns to the west to provide an exterior parking area immediately to the south of the garage. The parking area is underlain by the backfill placed behind the retaining wall. We noted diagonal cracking in this wall near the southeast corner, potentially resulting from outward rotation of both the east and south legs of the wall. To the east of this retaining wall is a large main floor deck, with paved storage and patio space beneath it. The area between the house and the private drive is covered with landscaping, the driveway, and a front entry walk. Immediately east of the house is a relatively level landscaped area that has been created by grading behind several retaining structures. East of these retaining walls, the ground slopes down to 96th Avenue S.E. This sloped area is heavily vegetated and contains trees of varying sizes. A set of steps providing access to 96th Avenue S.E. extends down the slope on the north edge of the site.

The ground surface on the western, developed portion of the lot is generally flat to gently-sloped. Along the north and south side of the house, the ground drops at a moderate inclination over the height of the basement. East of the rear landscaped area, the ground slopes steeply down toward 96th Avenue S.E. over an elevation change of 28 to 30 feet. We saw no indications of recent instability on this steep slope.

From our review of the Mercer Island GIS, the site is mapped as an Erosion Hazard and a Potential Landslide Hazard. Also, the slope on the eastern portion of the property is mapped as a Steep Slope. The closest mapped landslides documented on the *Mercer Island Landslide Hazard Assessment* are located over 400 feet west of the site, along the upslope side of East Mercer Way.

SUBSURFACE

The subsurface conditions were explored by drilling three test borings and hand excavating a test hole 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 October 2, 2025 using a track-mounted, hollow-stem auger drill. The test hole was excavated on the same day. Samples in the borings were taken at approximate 2.5- to 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 and Test Hole Logs are attached as Plates 3 through 7.

In addition to this field work, we also reviewed the results of borings that we conducted for the remodel of the house immediately to the north (#5411). As a part of our involvement with that project, we also monitored the installation of deep foundations for the improvements to that residence.

Soil Conditions

The test borings all found an upper layer of fill extending to a depth of 5 to 7 feet below the existing grade. The fill encountered in Borings 1 and 2 likely originated from the excavation for the house, and was placed to create the flat landscaped area east of the house. The fill in Boring 3 appears to be backfill placed behind the retaining wall that lines the east and south sides of the exterior parking pad.

Beneath the fill, the borings found native soils consisting of sand and silty sand that was initially loose. This native soil became medium-dense to dense within a depth of approximately 7 feet below existing grade to the east of the existing house, and at a depth of approximately 15 feet below the southern parking pad. Based on the results of Boring 3 and our hand-excavated test hole, it appears that the tall retaining wall was constructed on the upper, loose to medium-dense sand.

The borings that we conducted previously for the adjacent northern project encountered similar fill and native soil conditions.

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

Groundwater Conditions

Our explorations were conducted in early fall, before the onset of heavy precipitation. No groundwater seepage was encountered in the borings. We previously found shallow perched seepage in one of the borings conducted on the adjacent northern lot. This seepage appeared to represent subsurface water that could not percolate downward through the glacially-compressed sands.

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. Where 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 loose fill and native soils overlying competent, glacially-compressed sands. The dense to very dense, glacially-compressed sands have a high internal strength and are not susceptible to deep-seated instability. This is confirmed by the lack of a history of documented landslides in this area. However, the upper fill is too loose and variable in composition to adequately support the building loads of the proposed addition or new foundations, without the risk of significant post-construction settlement. Considering this, we recommend that all new building loads from the southern addition that extends east of the existing deck footprint be supported on deep foundations embedded into the competent, dense to very dense soils. Based on our experience with numerous similar projects, it is our professional opinion that driven, small-diameter pipe piles will likely provide the most efficient deep foundation system for this expansion. Any new floor slabs for the basement level of the new addition should also be structurally supported on pipe piles either as structural slabs or framed floors overlying crawl spaces.

New foundations for the addition within the footprint of the existing southern deck could be supported on conventional foundations. It will be necessary to excavate these footings to reach medium-dense to dense, native soils. If this becomes impractical during construction, small-diameter pipe piles could be used instead.

It appears that the eastern and southern sides of the new third garage bay can be supported on the existing retaining walls. However, the condition of the retaining wall should first be assessed by the project structural engineer. Alternatively, new foundations supported on driven pipe piles can be installed on the inside of the existing retaining wall. Any other new foundations for the westward expansion of the existing garage or for the new third garage bay should be supported on driven pipe piles. The existing parking area was constructed over fill placed behind the existing retaining walls. If an on-grade slab is poured on this fill for the floor of the third garage bay, or for the western expansion of the existing garage, the new slab may undergo slight settlement relative to the pile-supported foundations.

The slope to the east of the house is not susceptible to instability extending into the glacially-compressed soil. However, future slope movement in the existing fill or loose native soils is possible, especially during a large earthquake. In order to protect the planned eastern addition the from potential damage in the event of foreseeable future shallow slope movement, we recommend: 1) a perimeter grade beam be used to interconnect the pipe piles for the addition extending east from the existing deck, and 2) the eastern foundation of this addition is supported on closely-spaced, driven, small-section soldier piles. These soldier piles would laterally restrain the soil to a depth of 7 feet below the ground surface (approximately 5.5 feet below the perimeter grade beam) in the unlikely event that the fill at the crest of the eastern slope would fail during the low probability earthquake. The driven soldier piles can also provide vertical support for the eastern foundation of the addition, in place of pipe piles.

The isolated posts for the reconfigured eastern deck should also be supported on driven soldier piles. These piles should be designed to retain soil to a depth of 7 feet acting over two times the width of the soldier pile.

We recommend against placing any additional fill above the existing grades east of the existing house. If the basement floor of the new addition is built as a pile-supported slab, only lightweight geofoam can be used to reach as backfill inside the additions to reach the slab subgrade elevations. Use of a framed floor over a crawl space would avoid the need for this geofoam fill. The existing eastern landscape walls are not structural, and provide no benefit to the stability of the site. While they can remain in their current configuration, they should not be raised or modified. The slope east of the landscape walls should remain undisturbed. If utilities need to be extended down this slope, the trenches would either need to be excavated and backfilled by hand, or they would need to be installed using directional drilling.

We anticipate that the existing basement walls and slabs may remain as a part of the planned remodel. It is likely that only limited subsurface drainage, waterproofing, or moisture protection measures were installed as a part of the original house construction. Therefore, it would be appropriate for the design team to consider measures that should be incorporated in the remodel to provide protection against future moisture or water intrusion. An envelope consultant who specializes in these matters could be beneficial to the long-term performance of the remodeled home.

CRITICAL AREAS STUDY (MICC 19.07)

This site is designated on Mercer Island GIS website as being located in several Geologic Hazard Areas. The following sections contain our discussion of each of the Hazard Areas with regard to geotechnical aspects of this project.

Potential Landslide Hazard: The entire subject site is mapped as a Potential Landslide Hazard area. As previously discussed, the core of the subject site consists of competent, dense glacially-compressed, native sands that have a low potential for deep-seated landslides. As noted earlier, no landslides are mapped on or near the subject site. Shallow landslides affecting the existing fill are possible, and the recommendations of this report address that potential hazard.

The proposed development will be supported on deep foundations embedded into the dense to very dense soils that underlie the site. The addition located near the steep slope on the eastern side of the site will be supported both vertically and laterally by the deep foundations. Closely-spaced driven piles will stabilize the loose fill soils underneath the addition. The slope itself will not be disturbed by the planned construction. These measures will prevent the new development from adversely affecting the stability of the slope, and will protect the new construction from damage in the event of future shallow slope movement.

Steep Slope Hazard: **T**he eastern steep slope would be classified as a Steep Slope Hazard where it is inclined steeper than 40 percent and its height is more than 10 feet. While this slope meets those criteria, it is neither excessively steep or very tall. The potential for future instability in the dense to very dense glacially-compressed soil is negligible. The recommendations presented above will mitigate the potential impacts from future instability in the looser, near-surface soils. Because of this, it is our opinion that the planned development configuration with setbacks of only a few feet from the top of the steep slope can be constructed while protecting the stability of the slope and the development itself, provided the recommendations presented in this report are followed.

Erosion Hazard: The sloping eastern portion of the site meets the City of Mercer Island's criteria for an Erosion Hazard. This slope will not be disturbed by the planned development. As a result, the erosion potential within the planned development area is actually low. 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. A wire-backed silt fence bedded in compost, not native soil, or sand, should be erected as close as possible to the planned work area, and the existing vegetation between the silt fence and the lake left in place. Rocked construction access and staging areas should be established wherever trucks will have to drive off of pavement, in order reduce the amount of soil or mud carried off the property by trucks and equipment. 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. The potential for a deep-seated landslide on the eastern slope is negligible. As noted above, the entire site lies within a mapped Potential Landslide Hazard, and the prescriptive buffer would extend far beyond the boundaries of the property and the planned development area. However, it is our professional opinion that the proposed development can be constructed extending to within a few feet of the landscape walls that define the top of the steep eastern slope, provided the recommendations presented in this report are followed.

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.

We can only provide the verbatim statement of risk required by the Mercer Island Code in the future, after we have reviewed the submitted permit drawings.

The underlying glacially-compressed soils are essentially impervious and will stop downward percolation of large volumes of water infiltrated above it. Also, the upper soils on, and near, the steep slope could be destabilized by infiltrating stormwater that would naturally migrate downgradient toward the slope. Considering this, it is our professional opinion that onsite infiltration of stormwater is infeasible 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.

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 *ASCE7 Hazard Tool website*, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.44g and 0.5g, respectively, under ASCE 7-16.

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 soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature and the absence of near-surface groundwater.

PIPE PILES

As noted in the ***General*** section the addition, including the western garage expansion and the reconstructed eastern deck, should be vertically supported on small-diameter pipe piles. Depending on access, a combination of 2-inch-diameter up to 4-inch-diameter pipe piles could be used. Two-inch diameter pipe piles can be installed using hand-carried equipment. The 3- and 4-inch-diameter piles could be used where larger installation equipment can assess an area.

A 2-inch-diameter pipe pile driven with a minimum 90-pound jackhammer or a 140-pound Rhino hammer to a final penetration rate of 1-inch or less for one minute of continuous driving may be assigned an allowable compressive load of 3 tons.

Three- or 4-inch-diameter pipe piles driven with an 850- or 1,100- or 2,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (850-pound hammer)	FINAL DRIVING RATE (1,100-pound hammer)	FINAL DRIVING RATE (2,000-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
3 inches	10 sec/inch	6 sec/inch	2 sec/inch	6 tons
4 inches	16 sec/inch	10 sec/inch	4 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.

Schedule 80 pipe should be used for any 2-inch diameter pipe pile. As a minimum, Schedule 40 pipe should be used for pipe piles 3- or 4-inches in diameter. 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.

We expect that Mercer Island will require formal load testing on 3- or 4-inch pipe piles, with 200-percent load tests conducted on at least 3 percent of the 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. <u>However, if the ground in front of a foundation is loose or sloping, such as on the eastern side of the proposed east additions and reconstructed deck, the passive earth pressure given above should be neglected.</u> 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.

STABILIZATION PILES

As discussed in the ***General*** section, a stabilization wall is recommended below the eastern foundation of the eastern additions. The stabilization wall should consist of closely spaced, driven soldier piles spaced no further apart than 2 feet edge-to-edge. The soil within the stabilization zone will arch between the piles if a failure does in fact occur on the eastern slope. The piles could be installed by driving them to depth with a large hydraulic impact hammer. These driven beams are typically cheaper to install than drilled piles, but are limited in steel section size, length, and pile-to-pile spacings.

The stabilization wall should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 40 pcf. This pressure acts over the center-to-center spacing of the wide-flange beams. Assuming that the grade beam will be bottomed at least 1.5 feet below grade, the stabilization piles should be designed to resist this active earth pressure to a depth of 5.5 feet below the grade beam. The depth of retention can be reduced by removing some of the existing fill at the top of the slope and building taller foundation walls for the additions. Any fill placed above this depth underneath the floor of the addition would have to consist of geofoam, which is lightweight and self-supporting. Under the IBC, a seismic surcharge does not need to be applied to the design of the wall, as even following a shallow slide, the depth of retention for the stabilization wall would be less than 6 feet.

Each isolated post for the reconfigured eastern deck should be supported on a driven, wide-flange soldier pile. These piles should be designed for a 40 pcf soil pressure acting on two times the width of the beam to a depth of 7 feet below the existing or final grade, whichever is highest. As discussed above, the depth of retention can be reduced by removing some of the fill, which would require sloping the grade beneath the deck.

An ultimate (no safety factor included) passive soil pressure equal to that pressure exerted by a fluid with a density of 350 pcf will resist the lateral movement of the piles below the stabilization depth. This passive resistance can be assumed to act over twice the wide of the wide-flange beams. Typically, a safety factor of 1.5 is applied to the ultimate passive resistance for static conditions.

If the wide-flange beams are driven into the dense soils, to a depth of at least 15 feet below existing grade they can support an allowable compressive capacity of at least 10 tons.

CONVENTIONAL FOUNDATIONS

New footings located within the footprint of the existing house and southern deck can be supported on medium-dense to dense, native soils. We recommend that continuous and individual spread footings have minimum widths of 16 and 24 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.

An allowable bearing pressure of 2,000 pounds per square foot (psf) is appropriate for footings supported on competent native soil. A one-third increase in this design bearing pressure can 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 less than one-inch.

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.

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
Lateral Earth Pressure *	40 pcf
Passive Earth Pressure	300 pcf
Soil Unit Weight	130 pcf

Where: pcf is Pounds per Cubic Foot, and Lateral 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 lateral equivalent fluid pressure. This applies only to walls with level backfill.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above lateral 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 the design of these types of walls, if desired.

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

Wall Pressures Due to Seismic Forces

Per IBC Section 1803.5.12, a seismic surcharge load need only be considered in the design of walls over 6 feet in height. A seismic surcharge load would be imposed by adding a uniform lateral pressure to the above-recommended lateral pressure. The recommended seismic surcharge pressure for this project 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.

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. A minimum 12-inch width of free-draining gravel or a drainage composite similar to Miradrain 6000 should be placed against the backfilled retaining walls. The gravel or drainage composites should be hydraulically connected to the foundation drain system. Free-draining backfill should be used for the entire width of the backfill where seepage is encountered. For increased protection, drainage composites should be placed along cut slope faces, and the walls should be backfilled entirely with free-draining soil. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

The ***General*** section also contains recommendations related to drainage and moisture protection for the basement walls and slab floors.

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 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.

BUILDING FLOORS

New lower (basement) floors should be supported by the piled foundations, either as a structural slab or as a framed floor over a crawlspace. Both flooring systems would be designed to span between the pile supported foundations without any reliance on soil bearing.

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.

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. Temporary cuts to a maximum overall 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 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 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.

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.

DRAINAGE CONSIDERATIONS

Footing drains should be used where: (1) crawl spaces or basements will be below a structure; (2) a slab is below the outside grade; or (3) the outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock that is encircled with non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space. The discharge pipe for subsurface drains should be sloped for flow to the outlet point. Roof and surface water drains must not discharge into the foundation drain system. A typical footing drain detail is attached to this report as Plate 7. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

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

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

GENERAL EARTHWORK AND STRUCTURAL FILL

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

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

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 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 and test holes 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 the proposed additions 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, resulting in possible loss of ground or damage to the facilities around the proposed construction.

This report has been prepared for the exclusive use of the Alpays 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 - 6	Test Boring and Test Hole Logs
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.

10/24/2025

Marc R. McGinnis, P.E.
Principal

cc: **Ripple Design Studio** – Jeffrey Almeter
via email: *jeffrey@rippledesignstudio.com*

MRM:kg

NORTH

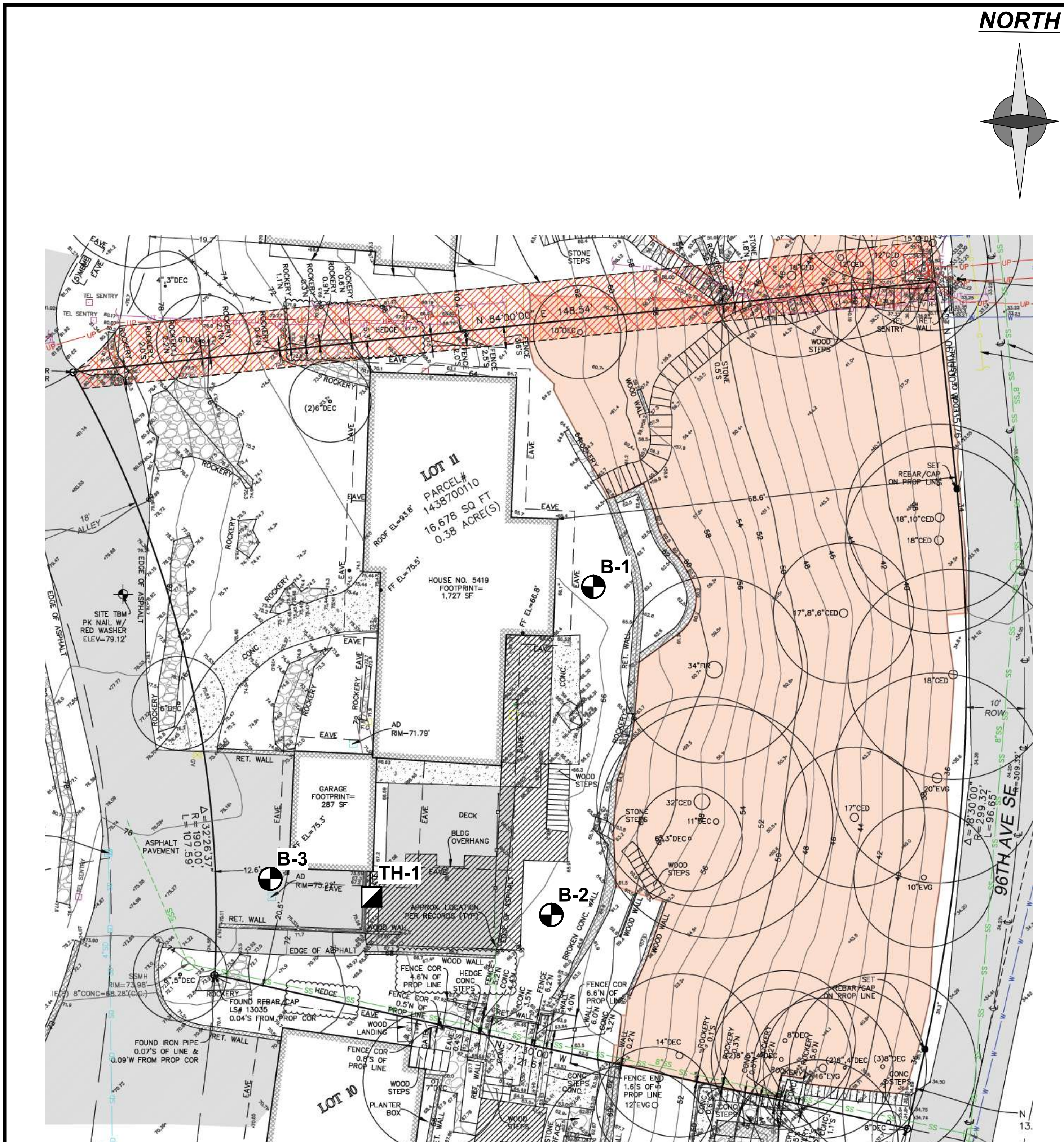
(Source: King County iMap)

GEOTECH
CONSULTANTS, INC.

VICINITY MAP

5419 - 96th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
25288	Oct. 2025		1



Legend:

- Test Boring Location
- Test Hole Location

GEOTECH CONSULTANTS, INC.

SITE EXPLORATION PLAN

5419 - 96th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
25288	Oct. 2025	No Scale	2

BORING 1

Elevation ±66 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Brown silty SAND with abundant roots, fine-grained, moist, loose **(FILL)**
			19	1		
5			9	2		-becomes black with abundant organics (old topsoil)
					SP	Brown SAND, fine-grained, dry, loose
						-with a large root
			38	3		-becomes gray-brown with rusting, gravelly, dense
10			42	4		-with abundant gravel
15			35	5	SP	
20			62	6		-becomes gray, very dense
25			47	7		
30						

* **Test boring was terminated at 26.5 feet on October 2, 2025.**
* **No groundwater was encountered during drilling.**

GEOTECH CONSULTANTS, INC.

TEST BORING LOG

5419 - 96th Avenue East
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
25288	Oct. 2025	MKM	3

BORING 2

Elevation ±66 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0–7					FILL	Dark-brown to black, silty SAND fine-grained, moist, loose **(FILL)**
			4	1		
5			16	2		
7–21					SP	Brown slightly gravelly, SAND, fine-grained, moist, medium-dense
			11	3		
10			46	4		-becomes rusted, gravelly, dense
15			51	5		-becomes gray with rusting, very dense
20			54	6		

* **Test boring was terminated at 21 feet on October 2, 2025.**
* **No groundwater was encountered during drilling.**

GEOTECH
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TEST BORING LOG

5419 - 96th Avenue East
Mercer Island, Washington

Job	*Date:*	*Logged by:*	*Plate:*
25288	Oct. 2025	MKM	4

BORING 3

Elevation ±75 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	2-inches of asphalt over; crushed rock and Type 17
			19	1		
5			9	2	SP	Rust-brown SAND, fine-grained, moist, loose
			22	3		-becomes rusted, medium-dense
10			11	4	SM	Gray very silty SAND, fine-grained, moist, medium-dense -becomes rusted, very moist
15			82	5		-becomes very dense
20						
25						
30						

*** Test boring was terminated at 16 feet on October 2, 2025 due to auger refusal.**

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

GEOTECH CONSULTANTS, INC.

TEST BORING LOG

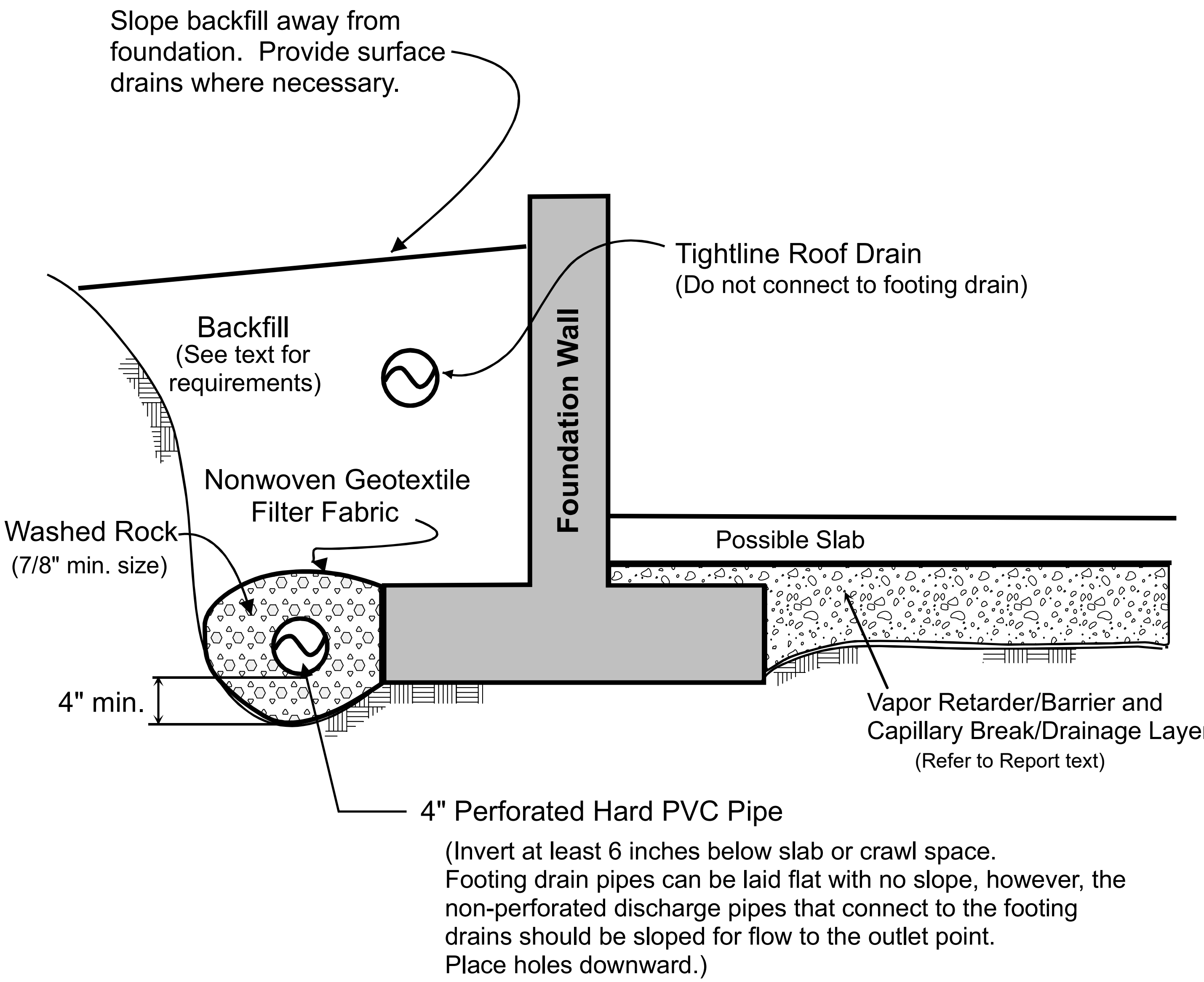
5419 - 96th Avenue East
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
25288	Oct. 2025	MKM	5

TEST HOLE 1

Depth (feet)	Soil Description
0 – 1.5	Washed rock and dark-brown silty SAND, fine-grained, moist, loose **[FILL]** - Top of footing measured at 8 inches below ground surface
1.5 – 3.0	Brown SAND, fine-grained, moist, loose to medium-dense **[SP]**

Test Hole was terminated at a depth of 3.0 feet on October 2, 2025.
No groundwater seepage was observed.



NOTES:

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

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

GEOTECH CONSULTANTS, INC.

FOOTING DRAIN DETAIL

5419 - 96th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
25288	Oct. 2025		7