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via email: jimteadie@gmail.com

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Eadie Residence Remodel and Addition Project
5411 – 96th Avenue Southeast
Mercer Island, Washington

Greetings:

Attached to this transmittal letter is our geotechnical engineering report for the proposed residence remodel and addition project to be constructed in Mercer Island, Washington. 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, retaining walls, subsurface drainage, and temporary excavations. This work was authorized by your acceptance of our proposal, P-11539, dated December 14, 2023.

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.

Matthew K. McGinnis
Geotechnical Engineer

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GEOTECHNICAL ENGINEERING STUDY
Proposed Eadie Residence Remodel and Addition Project
5411 – 96th Avenue Southeast
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed remodel and addition project to be located on Mercer Island.

We were provided with site plans and a topographic map. Sturman Architects developed these plans, which are dated October 31, 2023. Based on these plans, we understand that the existing residence will undergo an interior remodel. Additions are proposed to be constructed off the southeast and south sides of the residence at the main and upper floors, and also reconstructing and adding a second story atop the existing garage. The western garage wall may be reused to brace the excavation during construction. A second story is also proposed to be added over the southern half of the existing residence and will extend eastward out over the existing elevated deck. New foundations will be needed for the additions, and new loads will be introduced to the existing foundations where the additions tie in and new floor area is proposed. It appears that only shallow excavations are proposed for this project; the additions will generally match the floor elevations of the existing residence, although the garage slab will be situated slightly below the elevation of the main floor of the residence.

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 irregular shaped site comprises a total site area of 0.35-acres. The site is bounded to the north, east, and west by 96th Avenue Southeast, and to the south by a residential parcel. Other, extensively developed parcels are set adjacent to 96th Avenue Southeast.

The grade across the property slopes downward from west to east with a total elevation change of about 46 feet within the site bounds. The grade is initially gently to moderately sloped from the western property line, continuing into the site across a landscaped area and driveway. A short rockery lines the western side of the driveway, which leads to a detached garage near the southwestern corner of the site. The existing, two-story residence is located east of the driveway, and is underlain by an east-facing daylight basement. The areas around the residence are extensively landscaped and hardscaped, and facilitate a moderate grade drop between the western main floor entrance, and daylight basement level; a patio and elevated deck are situated to the south and southeast of the residence. Small gravel pathways and sloped landscaped areas exist both to the north of the residence, as well as to the east of the residence, and encompass the remainder of the property. Many of these sloped areas are only moderately inclined, but a steeply inclined slope exists along the eastern perimeter of the site. The slope due east of the eastern perimeter of the residence is inclined just over 40 percent, and has an elevation change of up to about 20 feet before the grade flattens out at the street level. Based on the City of Mercer Island's code, the portions of these steeply inclined slopes that are greater than 10 feet in height would meet the criteria for a Steep Slope Area. These steeply inclined slopes are landscaped with scattered shrubs, trees, and landscape mulch.

The City of Mercer Island GIS maps the site to lie within an Erosion Hazard Area. Much of the site is inclined in excess of 15 percent and would meet the general criteria for an erosion hazard. In addition, the site is mapped to lie within a Potential Landslide Hazard Area. The *Mercer Island Landslide Hazard Assessment* map (Troost & Wisher, 2009) does not indicate any documented landslides on or near the site.

SUBSURFACE

The subsurface conditions were explored by drilling three test borings and excavating four test holes with hand equipment 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 March 4, 2022 and January 25, 2024 using a track-mounted, hollow-stem auger drill and a portable Acker drill. The test holes were excavated on January 25, 2024 using hand tools to assess soil conditions near the existing foundations. Samples 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 6.

Soil Conditions

The test borings encountered approximately 3.5 to 7 feet of loose soil at the ground surface. Native, loose to medium-dense sand and silty sand was revealed beneath the fill, extending to a depth of about 5 to 7.5 feet. Medium-dense silty sand was revealed beneath a depth of 5 feet in Test Boring 3, and dense to very dense sand and slightly silty sand was encountered at a depth of 7.5 to 10 feet in the test borings. These dense and very dense native soils continued to depths of 10.5 to 16.5 feet where the test borings were terminated generally due to auger refusal in the dense soils.

The test holes were excavated near the existing garage and residence foundations. Beneath the ground surface, the top of the residence and garage footings were revealed at depths of approximately 4 to 6 inches, and the base of the footings, where it was possible to expose int the test holes, was generally revealed at a depth of 12 inches. Loose and medium-dense fill soil was revealed beneath the footings in Test Holes 1, 2, and 3, and generally continued to the base of the shallow test holes. Test Hole 4, excavated near the northeast corner of the garage, encountered native, medium-dense silty sand, similar to what was encountered in Test Boring 3, at a depth of 1 foot beneath the ground surface.

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

Slight perched groundwater seepage was observed at a depth of 7.5 to 10 feet in Test Boring 3. 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.

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 soils at the ground surface that overlie medium-dense and denser, native sand and silty sand soil at depths ranging from about 5 to 7.5 feet. These dense native soils are glacially consolidated, and they are the core soil of the site. These soils have a high internal strength and are not susceptible to deep seated instability. However, the upper fill and native soils are too loose and variable in composition to adequately support the building loads of the proposed additions, as well as the newly introduced loads on the existing foundations, without the risk of significant post-construction settlement; they also are susceptible to shallow slope movement. Considering this, we recommend that all building loads be supported on, or into the competent, medium-dense and denser native soils. Based on the test borings and test holes, it appears that differing bearing conditions exist beneath the existing residence and garage foundations. It appears that the upslope, western wall of the garage and upslope, western basement wall of the residence were constructed on the competent native soils and could be reused as part of the proposed development. However, a deep excavation would be needed to reach these soils across the majority of the addition area. In addition, based on the shallow test holes, it appears that a majority of the existing garage and residence foundations were constructed as shallow foundations atop both loose fill/backfill soil, and loose native sand. New foundations and existing foundations outside of the western perimeter garage and basement walls will need to be supported on deep foundations in order to prevent post construction settlement of the looser surficial soils. This includes any interior foundations that will need to support new loads. For this project, small diameter pipe piles would likely provide the most efficient deep foundation system for most of the project. However, because the eastern side of the additions are located near the top of a steep slope which has up to about 7 feet of loose soil, we recommend that the deep foundation system at the eastern side of the additions includes helical anchors to support potential lateral loads if any potential lateral movement of the loose soil were ever to occur. We also recommend that all foundations on the eastern side of the additions be connected with a continuous grade beam that is supported with the piles and the anchors. Further recommendations are listed in the ***Pipe Piles*** and ***Helical Anchor*** sections of this report.

CRITICAL AREAS STUDY (MICC 19.07)

This site is designated on Mercer Island GIS website as being located in several Geologic Hazard Areas. Our discussion of the Hazard Areas with regards to the geotechnical engineering aspect of this project are discussed below.

Seismic Hazard and Potential Landslide Hazard Areas: The entire subject site is located within 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 competent, dense glacially compressed, native silty sand, sand, and silt that has a low potential for deep-seated landslides. As noted earlier, no landslides are mapped on or near the subject site. The site is not mapped to lie within a Seismic Hazard Area per the GIS.

The proposed development will be supported on conventional footings or deep foundations that embed on or into the competent soils that underlie the site. All of the additions located near the steep slopes located on the eastern side of the site will be supported both vertically and laterally by the deep foundations. Therefore, it is our professional opinion that the potential issues posed by potential movement of loose soil on or near the eastern steep slope are mitigated. In addition, soils at the site are not liquefiable, due to their dense nature and the absence of near-surface groundwater.

Steep Slope Hazard Areas: As noted earlier, some steep slopes exist along the eastern side of the edge of the site; these qualify as a Steep Slope Hazard Area per Mercer Island Code. The slopes only slope about 40 to 50 percent, so they are only moderately steep slopes. Also as noted above, no landslides are mapped on or near the site. As noted earlier, the core soil of the site is dense sand and silty sand, and all loads for the development will be supported on or into competent native soils. Because of this, it is our opinion that no buffers or setbacks are needed from this Steep Slope, provided the recommendations presented in this report are followed. Although some movement of the loose surface soil is possible on the eastern slopes, it is our professional opinion that, if the recommendations presented in the report are followed, such movement will not impact the proposed development.

Erosion Hazard Areas: The site also meets the City of Mercer Island's criteria for an Erosion Hazard Area. We have worked on numerous sloping site in 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 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. 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 potential landslide hazard on this site is only shallow because of the presence of medium-dense silty sand and dense sand within 5 to 10 feet of the ground surface as revealed in the test borings. 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. However, it is our professional opinion that the location of the proposed development is sufficient without any buffer from the eastern steep slope provided the recommendations presented in this report are followed. In particular, the inclusion of foundations that bear on or into the core soil is intended to address geotechnical engineering issues in 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.

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.

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 1.44g and 0.5g, 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 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.

Sections 1803.5 of the IBC and 11.8 of ASCE 7 require that other seismic-related geotechnical design parameters (seismic surcharge for retaining wall design and slope stability) include the potential effects of the Design Earthquake. The peak ground acceleration for the Design Earthquake is defined in Section 11.2 of ASCE 7 as two-thirds (2/3) of the MCE peak ground acceleration, or 0.45g.

CONVENTIONAL FOUNDATIONS

As noted earlier in this report, the existing upslope, western footings of the garage and basement can be used to support loads of the proposed residence project. However, a low bearing capacity of 2,000 psf should be used in the analysis of these footings. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. 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.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level, well-compacted fill.

We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.50
Passive Earth Pressure	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

As noted in the ***General*** section, the majority of the additions, and where the existing residence and garage will experience new loads outside of their western foundation walls 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 larger, 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 over 3-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.

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 additions, the passive earth pressure given above will not be appropriate.</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.

We recommend that the piles be driven to at least a depth of 8 feet below the existing ground. However, it is very likely that the piles will be driven deeper, with depths of up to about 20 feet possible.

HELICAL ANCHORS

Helical anchors consist of single or multiple helixes that are rotated into the ground on the end of round or square metal shafts. The minimum diameter of a single helix anchor is 8 inches. The ultimate capacity of the anchor in tension or compression can be estimated roughly by multiplying the installation torque by 10. We recommend that the uppermost helix be installed at least 5 feet behind the grade beam and into the very dense soils that comprise the core of the site. A typical anchor capacity for a single 8- to 10-inch helix is 10 to 15 kips, but multiple options for the number of helixes and capacity are available. The anchors should be installed by a specialty contractor familiar with the design and installation of helical anchor systems. The anchor contractor can assist with refining the anchor design and details and estimating capacities for different soil and anchor conditions. All anchors should be installed to a torque rate that is estimated to be at least 200 percent of the estimated design capacity. We also recommend that at least one of the anchors be load tested to 200 percent of its design capacity.

The anchors need to restrain lateral pressures of the potential movement of the looser upper soils. The lateral pressure can be calculated by applying an active earth pressure of 40 pounds per cubic foot (pcf) over a depth of 7 feet measured from the existing grade. A safety factor of 1.5 should be applied in determining the necessary capacity of the anchors needed to mitigate the forces of this potential movement.

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 *	35 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 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.

BUILDING FLOORS

New building 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 6. 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 Jim and Jessica Eadie 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.

2/13/2024

D. Robert Ward, P.E.
Principal

MKM/DRW:kg

NORTH

(Source: King County iMap)

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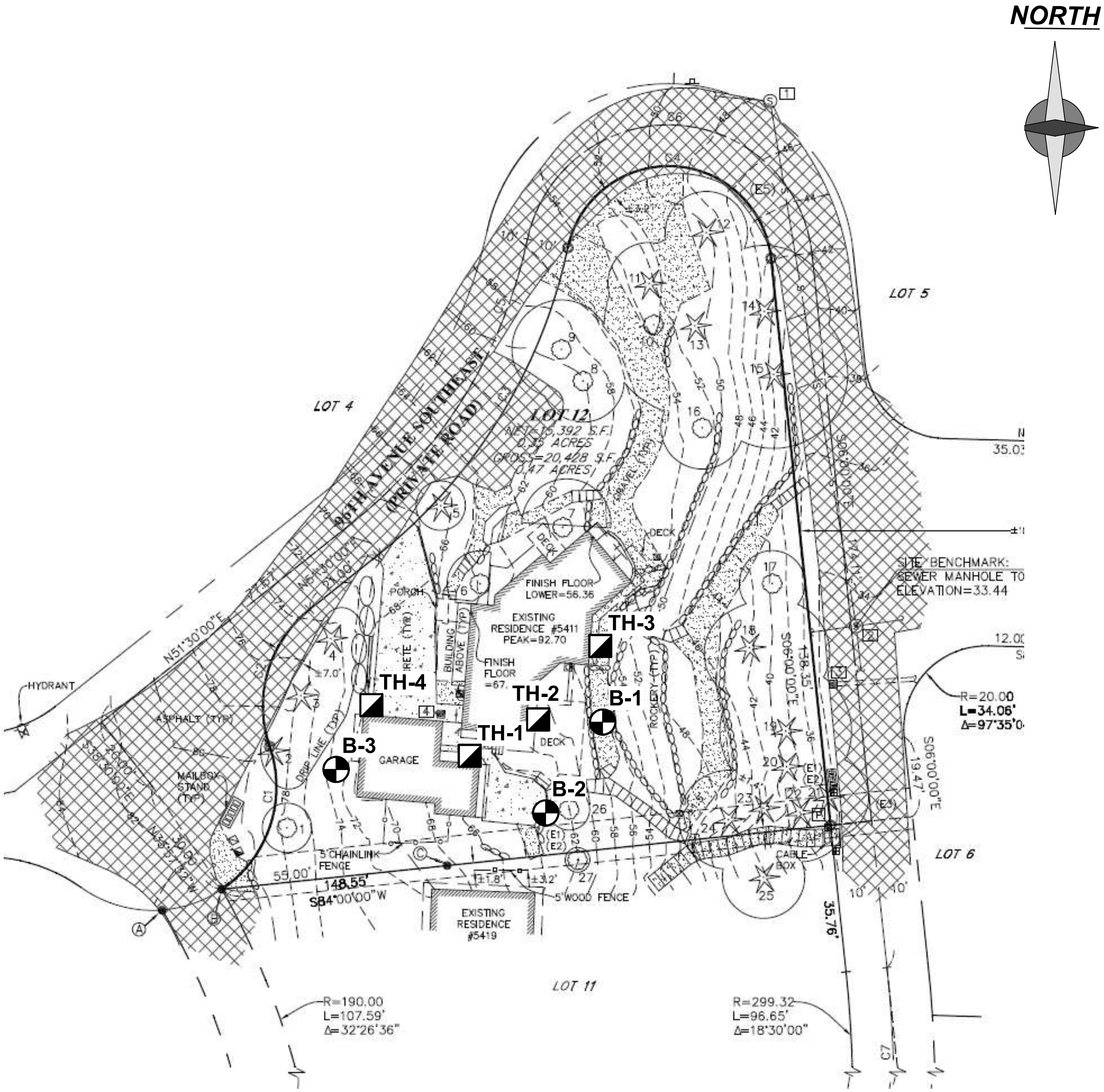
VICINITY MAP

5411 - 96th Avenue Southeast
Mercer Island, Washington

Job	Date:		Plate:
23457	Feb. 2024		1



Legend:

- Test Boring Location
- Test Hole Location

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SITE EXPLORATION PLAN

5411 - 96th Avenue Southeast
Mercer Island, Washington

Job	Date:		Plate:
23457	Feb. 2024	No Scale	2

BORING 1

Elevation ±56 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Crushed rock over;
			13	1	FILL	Dark-brown, gravelly, silty SAND with roots, fine-grained, moist, loose **(FILL)**
5			9	2		-becomes grayish-brown
			30	3	SP	Grayish-brown, slightly gravelly, SAND with rusting, fine-grained, moist, dense
10			50/6"	4		-becomes gray, very dense
15			73	5		-increased silt content

Depth scale: 5, 10, 15, 20, 25, 30

* **Test boring was terminated at 16.5 feet on March 4, 2022 due to auger refusal.**
* **No groundwater was encountered during drilling.**

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TEST BORING LOG

5411 - 96th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
23457	Feb. 2024	MKM	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0–4					FILL	Crushed rock over brown and dark brown SAND with roots, fine-grained, moist, loose
4					SP	Brown gravelly SAND, fine-grained, moist, loose
5			9	1	SP	Brown to gray-brown with trace rusting gravelly SAND with trace organics, fine to medium-grained, moist, loose to medium-dense
			59	2	SP	- becomes gray-brown, very gravelly, very dense
10			50/6"	3	SP	
			50/6"	4	SP	- becomes very moist to wet
15						

*** Test boring was terminated at 14.5 feet on January 25, 2024.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

5411 - 96th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
23457	Feb. 2024	MKM	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Brown sandy GRAVEL with roots, fine to medium-grained, moist, loose
			7	1		
5			24	2	SM	Brown heavily mottled silty SAND with gravel and roots, fine-grained, very moist, loose - becomes gray with rusted sand seams, slighly silty to silty, medium-dense
		▽	25	3	SP/SM	Interbedded layers of gray SAND and slighly silty SAND, fine-grained, wet, medium-dense
10			50/6”	4		- becomes very gravelly, very moist, very dense
15						

*** Test boring was terminated at 10.5 feet on January 25, 2024.**
*** Groundwater was encountered at 7.5 feet during drilling.**

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TEST BORING LOG

5411 - 96th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
23457	Feb. 2024	MKM	5

TEST HOLE 1

Depth (Feet)	Soil Description
0.0 – 1.5	Brown gravelly SAND with trace organics, fine-grained, moist, loose **[FILL]** - 6”, top of footing
1.5 – 4.0	Gray-brown gravelly SAND, fine-grained, moist, medium-dense **[FILL?]** - 3.5’, becomes gray, very gravelly

Test Hole was terminated at 4.0 feet on January 25, 2024, due to refusal on rocks.
No groundwater seepage was encountered in the test hole.
No caving was observed during excavation.

TEST HOLE 2

Depth (Feet)	Soil Description
0.0 – 4.5	Brown gravelly SAND, fine to medium-grained, moist, loose **[SP]** - 4”, top of footing - 12”, bottom of footing - 2’, becomes gray-brown, very gravelly - 4.5’, becomes medium-dense

Test Hole was terminated at 4.5 feet on January 25, 2024.
No groundwater seepage was encountered in the test hole.
No caving was observed during excavation.

TEST HOLE 3

Depth (Feet)	Soil Description
0.0 – 4.5	Dark brown gravelly, silty SAND, fine-grained, moist, loose **[FILL]** - 4”, top of footing - 12”, bottom of footing

Test Hole was terminated at 4.5 feet on January 25, 2024.
No groundwater seepage was encountered in the test hole.
No caving was observed during excavation.

TEST HOLE 4

Depth (Feet)	Soil Description
0.0 – 1.0	Topsoil
1.0 – 2.5	Gray slightly silty SAND, fine-grained, very moist, medium-dense **[SM]**

Test Hole was terminated at 2.5 feet on January 25, 2024.
No groundwater seepage was encountered in the test hole.
No caving was observed during excavation.

***NOTE** – Letters in brackets **[ ]** denote the USCS soil classification.

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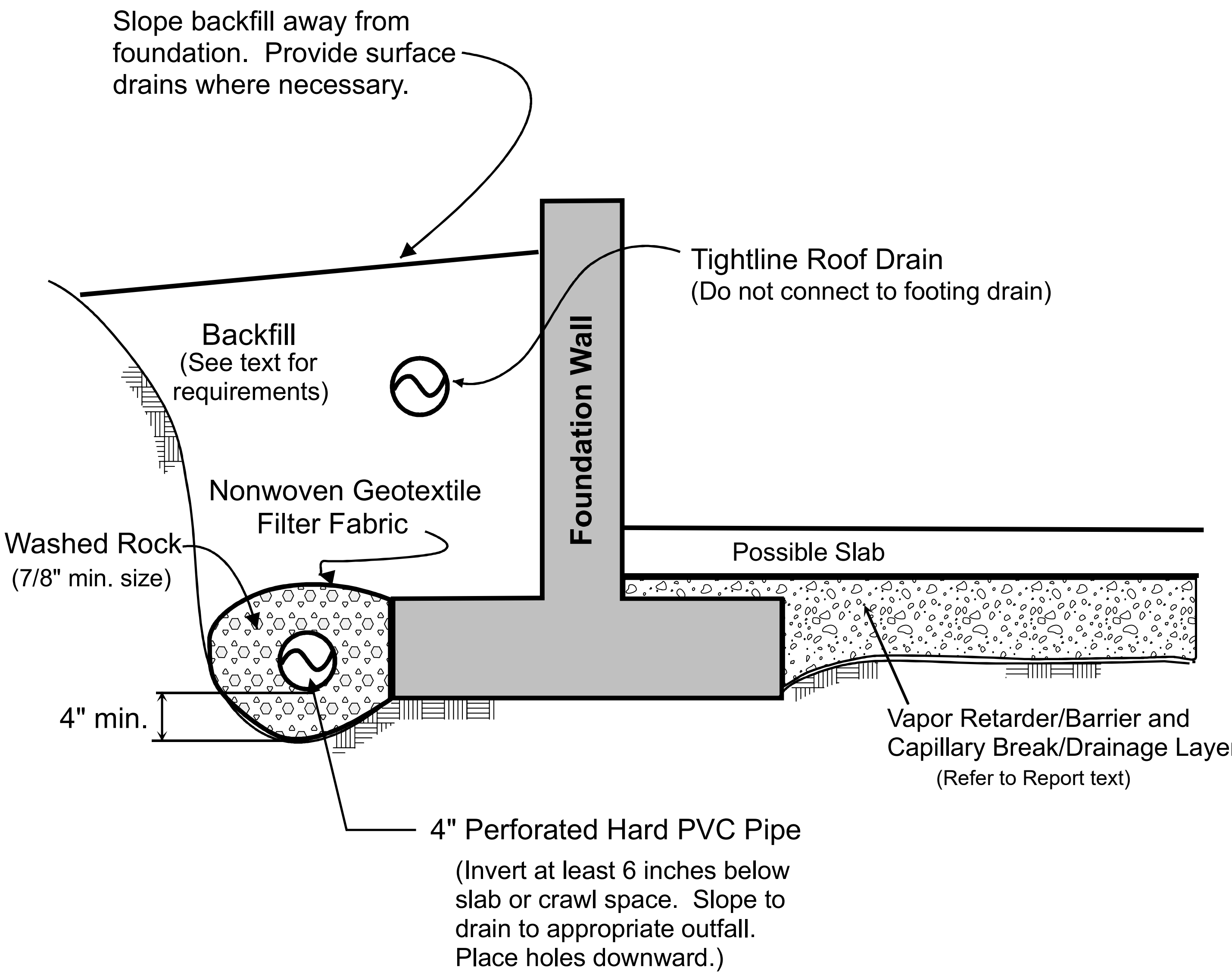
TEST HOLE LOG

5411 - 96th Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
23457	Feb. 2024	MKM	6



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

5411 - 96th Avenue Southeast
Mercer Island, Washington

Job	Date:		Plate:
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