

October 17, 2024

JN 24300

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via email: skgolf6@hotmail.com

Subject: **Transmittal Letter – Geotechnical Engineering Study and Critical Area Study**
Proposed Carport Expansion and Retaining Wall
5602 East Mercer Way
Mercer Island, Washington

Greetings:

Attached to this transmittal letter is our combined geotechnical engineering report and critical area study for the proposed expanded carport and retaining wall to be constructed on Mercer Island, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide geotechnical engineering recommendations for the project as well as a discussion of Critical Areas noted in the Mercer Island Municipal Code. This work was authorized by your acceptance of our proposal, P-11661, dated June 7, 2024.

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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MKM/DRW:kg

GEOTECHNICAL ENGINEERING STUDY AND CRITICAL AREA STUDY
Proposed Carport Expansion and Retaining Wall
5602 East Mercer Way
Mercer Island, Washington

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

We were provided with an undated site plan prepared by Bassett Larsen Design LLC. Based on this plan, we understand that an existing carport on the upslope, western side of the property will be rebuilt to be expanded to the west. To accomplish this, the existing rockery located on the western edge of the carport will be removed and replaced with a new wall that will extend further west. Based on the site plan, an excavation of up to about 9 feet is likely needed in the southwestern corner of the expanded carport to construct the new wall, with smaller excavations needed north of this corner.

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 Mercer Island. The irregular shaped site comprises a total site area of 0.43-acres. The site is bordered on all sides by residential parcels that contain residences of varying sizes; all of which are located well away from the proposed work area. Glenhome Drive runs through the southwestern corner of the site, near the proposed work area.

The grade across the site generally descends moderately to the east with a total elevation change of 50 feet. However, only about 10 to 15 feet of elevation change occurs on the western portion of the site that includes the existing/proposed carport area. An existing residence is located in the approximate center of the site. Directly west of the residence is the existing carport and a paved driveway. An existing rockery ranging in height from around 4 to 6 feet lines the western edge of the driveway and carport. This rockery has a slope above it that is inclined from 40 to almost 100 percent. The slope is generally covered with mulch, scattered decorative shrubs, and some small trees. This slope, including the rockery at its base, is steeply inclined and facilitates an elevation drop of around 13 to 15 feet. The top of the southwestern portion of the slope terminates at Glenhome Drive. The steep western slope and rockery meets Mercer Island's code criteria for a Steep Slope Area and Landslide Hazard Area where it is taller than 10 feet. It is obvious that a portion of the slope has been created during previous grading activities related to excavations at the carport/driveway and the construction of Glenhome Drive.

The City of Mercer Island maps the site within a Potential Slide Area and Erosion Hazard Area. The City of Mercer Island Landslide Hazard Assessment Map does not map any recorded landslides on, or in the direct vicinity of the site. During our times on the site, we did not observe any indications of recent shallow or deep-seated instability within the western slope and rockery.

SUBSURFACE

The subsurface conditions were explored by drilling one test boring and three test holes 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 boring was drilled on October 2, 2024 using a hand-carried drill rig, while the test holes were excavated using hand tools on the same day. Samples within the test boring 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 and 4.

Soil Conditions

The test boring was drilled on the slope above the existing rockery near the northern extent of the proposed retaining wall. Native, medium-dense silty sand was revealed at the ground surface in the test boring and continued to a depth of 2.5 feet, where the silty sand then became cemented and very dense. The very dense native soils are glacially compressed and continued to the base of the test boring at a depth of 6 feet where auger refusal was met.

The test holes were excavated nearer to the southern portion of the retaining wall and carport expansion. Test Hole 1, excavated from the top of the rockery feature, revealed at a depth of approximately 1.5 feet, the native, medium-dense silty sand was revealed. Refusal was met in dense soil at approximately 2.5 feet. The remaining test holes were excavated at the base of the rockery, and the very dense, glacially compressed, silty sand was revealed at a depth of about 1.5 feet beneath a thin layer of topsoil and medium-dense silty sand.

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

No groundwater seepage was observed in test boring or test holes. However, groundwater levels encountered during drilling can be deceptive, because seepage into a test boring can be blocked or slowed by the auger itself. It should be noted that groundwater levels vary seasonally with rainfall and other factors, with higher and greater amounts typically revealed in the winter and spring months. It is possible that groundwater could be found perched on the underlying very dense soil.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. If a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the test boring logs are interpretive descriptions based on the conditions observed during drilling.

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 C (Very Dense Soil and Soft Rock). 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 0.50g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) during an earthquake be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period). The MCE peak ground acceleration adjusted for site class effects (F_{PGA}) equals 0.74g. The soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense and very dense, glacially-compressed 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 0.49g.

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.

This report provides a summary of our geotechnical engineering conclusions and recommendations regarding the project. It also provides information regarding the mapped critical areas per the MICC.

The test boring and test holes conducted for this study encountered native, glacially compressed, very dense, silty sand soils beneath a thin mantle of a somewhat loose soils. Therefore, it is apparent that the core of the site and slope are comprised of this competent, glacially compressed soil. This soil has a very low susceptibility to deep seated instability due to its high internal strength, and also provides excellent bearing support for the new foundations.

One of the significant features of this project is the wall needed on the western edge of the proposed expanded carport. Although the existing smaller rockery at the western edge of the existing carport appears in a stable condition and the soils of the proposed wall area are very competent, the new wall cannot be a rockery because it will not be able to achieve safety factors required by code. Therefore, we recommend that a standard, cast-in-place, footing-supported retaining wall be used for the western wall.

A significant aspect of this project is the temporary excavation needed to expand the carport and construct the new western retaining wall. It appears that the excavation needed for the footing foundations will be up to about 9 feet below the existing site grades, with the deepest excavation at the carport's southwestern corner. Constructing a temporary excavation may have some challenges, as noted further below, so we recommend that the footing of the retaining be

constructed in and “L-shape” so that all of the footing is below or inboard of the wall to limit the amount of excavation into the western slope. Based on the soils encountered in the explorations, the upper looser soils should not be excavated at an inclination steeper than a 1:1 (Horizontal:Vertical). However, a maximum vertical excavation up to 5 feet in height can be constructed at the base of the excavation where the very dense soils are encountered. In developing an excavation plan for the project, we recommend that the base of the excavation allow at least 18 inches of flat space adjacent to new footings to allow room for the concrete form and subsurface drainage installation. We further recommend that at least 2 feet of flat ground be maintained adjacent to the nearby, upper paved driveway. In addition to the constraint of the driveway, the western property line potentially constrains the excavation for the retaining wall; if the excavation cannot be kept within the property, then obtaining an excavation agreement from the adjacent property owner would be the next alternative. If an open excavation is made on and/or off the property, it should be fully covered with plastic sheeting once it has been excavated to help protect the exposed soils during periods of adverse weather. Excavation shoring is a further alternative if open excavations cannot be made or if not enough room from the driveway exists, and shifting western retaining wall eastward is also an alternative. We can provide information for shoring alternatives once an excavation plan for the project has been developed.

As competent as the very dense, glacially compressed soil is, it is sensitive to moisture if exposed. If the soils at the footing subgrade elevation are wet, or become wet during foundation construction, it may be necessary to cover the base of the excavation with a protective layer consisting of several inches of clean angular rock after scraping the excavations clean in order to provide subgrade protection during form and reinforcement construction. Excavations into glacially compressed soils are typically conducted using a toothed bucket due to the soils' density. This typically leaves at least a few inches of loose, disturbed soils atop the base of the excavation due to the excavator teeth, which is not suitable for bearing. We recommend that all final foundation excavations for the new residence occur either using a smooth bucket, flat grade bar, or flat blade shovel to ensure that the subgrades have been scraped clean of any loose or wet soil or debris.

The competent silty sand soils are dense and very dense and were observed to have been glacially compressed. These qualities leave little to no inter-particle pore spaces, making the silt soils nearly impermeable (i.e., hydraulically restrictive). Because of this, it is our professional opinion that infiltration of concentrated stormwater should not be implemented for this project. Any attempt to utilize infiltration systems at this site will likely lead to early failure of the systems due to the shallow depth to a hydraulically restrictive layer. This would cause adverse drainage impacts to the site and surrounding areas.

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.

CRITICAL AREA STUDY (MICC 19.07)

Erosion Hazard Area: The site meets the City of Mercer Island's criteria for an Erosion Hazard Area. However, excavation and construction of the planned carport expansion and retaining wall can be accomplished without adverse erosion impacts to the site and surrounding properties by exercising care and being proactive adequate erosion control measures 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. A wire-backed silt fence or straw wattles should be placed on the eastern, downslope side of the driveway/carport area. Covering the base of the excavation with a layer of clean gravel or rock may also be prudent to reduce the amount of mud and silty water generated. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Silty water accumulating in the excavation must not be allowed to flow off the site. 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.

Potential Landslide Hazard Area: There are several criteria for being a Landslide Hazard Area based on the MICC. The first of several criteria are as follow:

1. Areas of historic failures.
2. Areas with all three of the following characteristics:
 - a. Slopes steeper than 15 percent; and
 - b. Hillsides intersecting geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock; and
 - c. Springs or ground water seepage.
3. Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements.
4. Areas potentially unstable because of rapid stream incision and stream bank erosion.

In our professional opinion, based on site features and the glacially compressed silty sand soils revealed at a shallow depth in the explorations, none of these four criteria noted above are met within the site.

There is also a fifth criteria with regards to Landslide Hazard areas: Any slope that is 40 percent or greater measured over a 30-foot horizontal run (Steep Slope). The western slope and rockery area qualifies under this criterion. However, because the core of the slope and site are comprised of very dense, glacially-compressed silty sand, which is not susceptible to deep seated instability, this area will be stable during constructed and be more stable than in its current condition because of the new retaining wall. Because of these reasons, and provided the recommendations in this report are

followed, we strongly believe that the excavations and new retaining wall can extend into this landslide hazard area and no buffers are necessary.

Buffers and Mitigation: As noted above, no buffer(s) are needed for this project and no special mitigation measures beyond what is recommended in this report are needed.

Statement of Risk: We can provide the statement of risk required by the City of Mercer Island once we have reviewed the geotechnical engineering aspects of the final plans that have been submitted for permit.

CONVENTIONAL FOUNDATIONS

The proposed new carport foundations and retaining wall can be supported on conventional continuous and spread footings bearing on undisturbed, very dense, glacially-compressed silty sand 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 3,500 pounds per square foot (psf) is appropriate for footings supported on competent soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads.

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

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

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

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. The above ultimate values for passive earth pressure and coefficient of friction do not include a safety factor.

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 the western retaining wall:

PARAMETER	VALUE
Lateral Earth Pressure *	35 pcf
Passive Earth Pressure	350 pcf
Coefficient of Friction	0.50
Soil Unit Weight	135 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 residential 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 a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

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 $9H$ pounds per square foot (psf), where H is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The 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.

Wall backfill should 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.

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 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 over 2 feet should not be made at property boundaries. Based upon Washington Administrative Code (WAC) 296, Part N, the upper, looser soils 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 a 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. The underlying Glacial Till would generally be classified as Type A, and temporary cut slopes greater than 5 feet in height should not be excavated steeper than a 0.75:1 (H:V) extending continuously from top to bottom of a cut. However, a vertical excavation of up to 5 feet can be made at the base of the 1:1 (H:V) excavation below the sloped excavation where Glacial Till soil is exposed. Please refer to the **General** section of this report for additional discussions and recommendations regarding the proposed excavation.

The above-recommended temporary slope inclinations are based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that 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.

Any disturbance to the existing slope outside of the building limits may reduce the stability of the slope. Damage to the existing vegetation and ground should be minimized, and any disturbed areas should be revegetated as soon as possible. Soil from the excavation should not be placed on the southern slope.

DRAINAGE CONSIDERATIONS

A footing drain, or simple weep holes, should also be placed at the base of earth-retaining walls. The 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 5. 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. For a weep pipe system, 2-inch-diameter

pipes should be placed through the base of the wall at 6-foot intervals. The filter fabric and washed rock recommendation made above still applies.

The excavation and site should be graded so that surface water is directed away from buildings and 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. 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 footings, slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

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

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions

encountered in the test borings are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

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

ADDITIONAL SERVICES

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

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

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 4	Test Boring and Test Hole Logs
Plate 5	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.

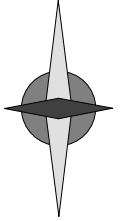


10/17/2024

D. Robert Ward, P.E.
Principal

MKM/DRW:kg

NORTH



(Source: King County iMap)



GEOTECH
CONSULTANTS, INC.

VICINITY MAP
5602 East Mercer Way
Mercer Island, Washington

Job No: 24300	Date: Oct. 2024	Plate: 1
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BORING 1

Depth (ft.)	Moisture Water Table	Blows per Foot	Sample	USCS	Description	Elevation ±86 feet
0					Topsoil	
0					Brown silty SAND with roots, fine-grained, dry, dessicated, medium-dense	
1		50	1	SM	-becomes gray-brown mottled orange, cemented, very dense	
6		50	2		-becomes gray with rusting	

* Test boring was terminated at 6 feet on October 2, 2024 due to auger refusal.
 * No groundwater was encountered during drilling.



TEST BORING LOG
 5602 East Mercer Way
 Mercer Island, Washington

Job No: 24300	Date: Oct. 2024	Logged by: MKM	Plate: 3
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TEST HOLE 1

Depth (feet)	Soil Description
0 – 2.5	Brown slightly gravelly, silty SAND with roots, fine-grained, dry, loose [SM] <ul style="list-style-type: none">- 1.5', becomes slightly gravelly, medium-dense- 2.5', abundant gravels

Test Hole was terminated at a depth of 2.5 feet on October 2, 2024 due to hand auger refusal. No groundwater seepage was observed.

TEST HOLE 2

Depth (feet)	Soil Description
0 – 1.5	TOPSOIL
1.5 – 2.5	Gray with rusting, silty SAND, fine-grained, moist, dense [SM]

Test Hole was terminated at a depth of 2.5 feet on October 2, 2024. No groundwater seepage was observed.

TEST HOLE 3

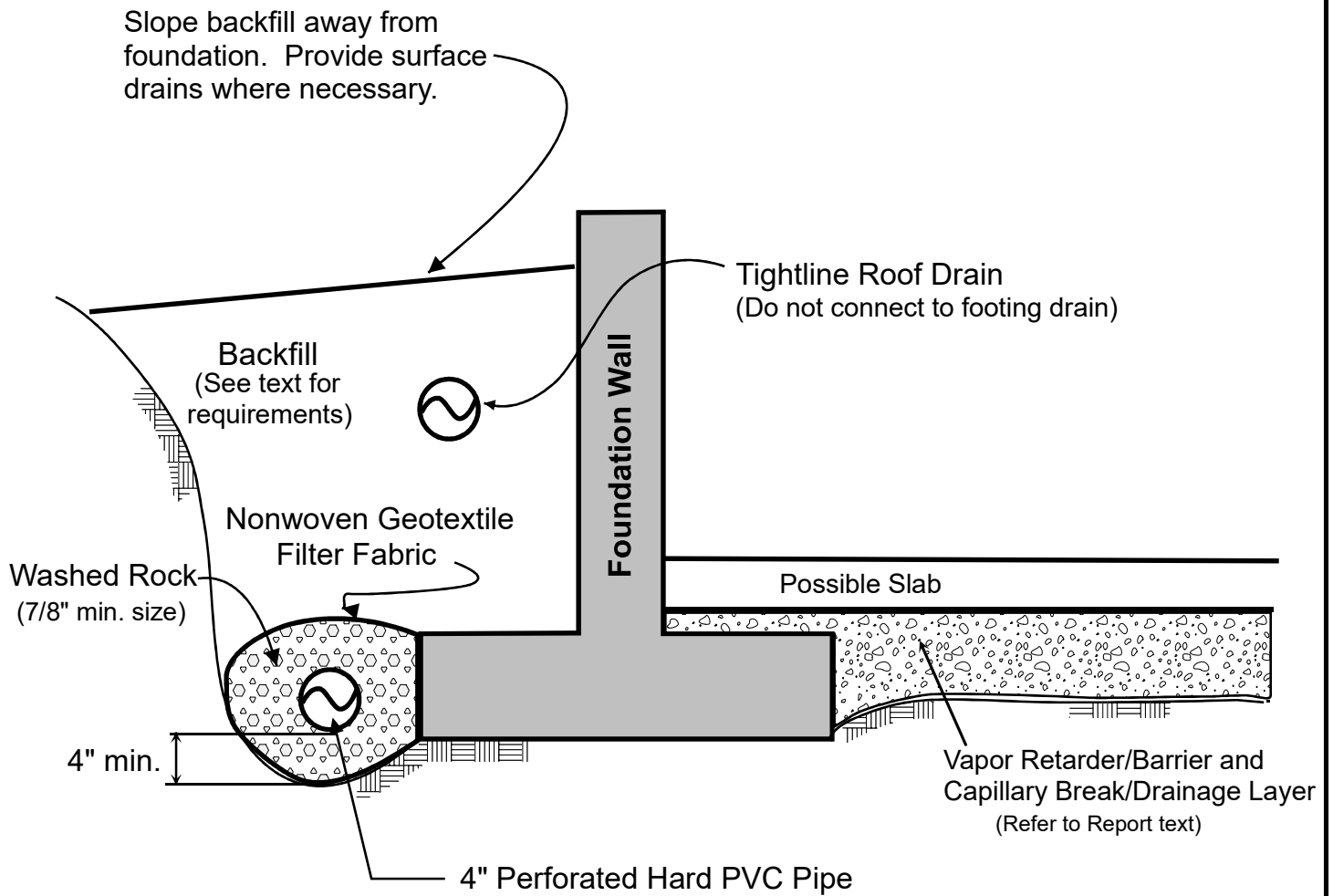
Depth (feet)	Soil Description
0 – 1.0	TOPSOIL
1.0 – 2.5	Brown silty SAND with roots, fine-grained, moist, loose [SM] <ul style="list-style-type: none">- 1.5', becomes gray with rusting, dense

Test Hole was terminated at a depth of 2.5 feet on October 2, 2024. No groundwater seepage was observed.



TEST HOLE LOGS
5602 East Mercer Way
Mercer Island, Washington

Job No: 24300	Date: Oct. 2024	Plate: 4
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(Invert at least 6 inches below slab or crawl space.
 Footing drain pipes can be laid flat with no slope, however, the
 non-perforated discharge pipes that connect to the footing
 drains should be sloped for flow to the outlet point.
 Place holes downward.)

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.



FOOTING DRAIN DETAIL
 5602 East Mercer Way
 Mercer Island, Washington

Job No: 24300	Date: Oct. 2024	Plate: 5
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