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February 28, 2023

JN 23014

Andy and Tracy Granbois c/o The Custom Home Company
PO Box 50208
Bellevue, Washington 98015

Attention: Jim Dwyer and William Fort
via email: jim.dwyer@thecustomhc.com & *William.fort@thecustomhc.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Granbois Residence
8440 Southeast 82nd Street
Mercer Island, Washington

Greetings:

Attached to this transmittal letter is our geotechnical engineering report for the proposed residence 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 and shoring. This work was authorized by your acceptance of our proposal, P-11317, dated January 19, 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

MKM/DRW:kg

GEOTECHNICAL ENGINEERING STUDY
Proposed Granbois Residence
8440 Southeast 82nd Street
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed new residence to be located in Mercer Island.

We were provided with preliminary architectural plans and a topographic map. The Custom Home Company developed these plans, which are dated November 23, 2022, and Core Design developed the site survey, dated September 28, 2022. Based on these plans we understand that an existing house on the site will be removed and replace with a new residence that will be situated in the approximate center of the property. A majority of the residence will be underlain by a full depth basement, except for beneath the garage and a covered patio located on the southwestern and northeastern corners of the residence, respectively. Finish floor elevations will be 329.5 feet and 318.5 feet for the main and basement floors, respectively. Considering the basement floor, excavations on the order of 9 to 13 feet will be needed. with shallower excavations anticipated for the on-grade structures. The residence will be set well away from the north property line, as close as 11.2 feet from the east, 20 feet from the south, and 5.6 feet from the west.

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 southern end of Mercer Island. The irregular shaped site has approximate dimensions of 140 to 150 feet in the north-south direction and 80 to 118 feet in the east-west direction. The site is bounded to the north, east, and west by single family parcels, and to the south by Southeast 82nd Street. A walking path sits between the eastern property line and adjacent eastern parcel.

The grade across the site slopes mostly gently downward from north to south, with a total elevation change of 8 to 9 feet. The northern portion of the site is populated with grass, landscaping, and hardscaping, and scattered mature trees. The existing, two-story house is located in the general center of the property and is flanked on its western side by a detached, one-story garage; both of which do not contain basement spaces. A covered breezeway links the two structures, and another covered outdoor area is situated east of the house.

The City of Mercer Island GIS does not map any Geologically Hazardous Areas on the site. No steep slopes are located within, or in the direct vicinity of the site.

The adjacent properties are all developed with single-family residences of varying construction and layout. Two-story residences are located to the north, both of which are set well away from the property line. A one-story brick residence is situated on the adjacent eastern property and is set at least 10 feet from the eastern property line. The brick residence did not appear to contain any basement space. A one- to two-story residence is located on the western adjacent parcel and is approximately 7 to 15 feet from the adjoining property line; and is not underlain by a basement. Visual observations made during our recent site visit would indicate that both adjacent residences bears on shallow foundation systems located near the ground surface.

SUBSURFACE

The subsurface conditions were explored by drilling three test borings and excavating one test hole at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The test borings were drilled on February 8, 2023 using a track-mounted, hollow-stem auger drill and the test hole was excavated on the same day with hand tools. 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 Logs are attached as Plates 3 through 6. The test hole log is attached as Plate 7.

Soil Conditions

The test borings and test hole were conducted near the perimeter of the proposed residence, and where site access and onsite utilities allowed. At the ground surface, Test Boring 3 and Test Hole 1 encountered loose fill soils to depths of 7 feet and 3 feet, respectively. The fill layer at Test Boring appeared to have resulted from previous grading to level out the front yard, while the fill at the test hole appeared to be associated with an old drain line. Beneath the fill, and at the ground surface in Test Borings 1 and 2, native silty sand, slightly silty sand and sand were revealed. These soils were initially loose, generally becoming slightly denser about 2 to 3 feet below the fill and/or ground surface. The native soils became dense at depths of 7 to 7.5 feet in all of the explorations, then very dense beneath depths of 7 to 14 feet. These native soils were observed to be glacially compressed and continued to the base of the explorations at depths ranging from 10 to 26.5 feet.

Groundwater Conditions

Perched groundwater seepage was observed at a depth of 9 to 13.5 feet in Test Borings 1 and 2 as well as Test Hole 1. While we were not able to excavate Test Hole 1 deeper than 10 feet, the groundwater layer encountered in this location is interpreted to be perched, similar to Test Boring 2. 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.

The explorations were drilled during the wet season, but these perched groundwater levels could rise slightly in other wet periods depending on precipitation amounts. Also, the levels will very likely decrease during the summer and early fall months.

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

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test borings and test holes conducted for this study encountered native, dense, glacially compressed soils at approximately 7 feet below the ground surface. These competent, dense soils are located below relatively loose fill and native soils. For foundation consistency, all new structural loads of the new residence and its associated features should bear on this competent, dense soil. Most of the proposed residence will be underlain with a basement, and the foundation level of the basement will be into the competent soils; thus, conventional footings are very suitable as foundations for the basement. However, there are structures proposed at the southwestern and northeastern corners of the residence that will not have a basement, thus excavations just needed to reach the proposed foundation grade will not extend to the competent, dense soils. To use conventional footings for these structures, overexcavations on the order of several feet is needed; these footings could be placed directly on the competent soil or on structural fill that is placed over the competent soil up to the proposed foundation grade. Additional recommendations can be found in the ***Conventional Foundations*** section of this report regarding the overexcavation issue. However, it is our experience that, based on the amount of overexcavation (and possibly structural fill) needed, the use of driven pipe piles for those structures will likely be more cost-effective. We have provided information regarding pipe piles for those structures.

While generally sandy in composition, the underlying native soils are fine-grained, silty, and were observed to be in an elevated moisture condition while conducting our explorations. These qualities make the native soils both moisture sensitive and sometimes easily disturbed from foot or machine traffic. We recommend that all final foundation excavations occur either using a smooth bucket, grade bar, or flat blade shovel to ensure that the foundation subgrades have been scraped clean of any loose soil or debris. Depending on final foundation elevations and site conditions, especially if the soils at the base of the excavation are wet, it would be practical to cover the base of the prepared basement footing areas with a layer of clean crushed rock to help provide some subgrade protection during form and reinforcement placement. This rock layer would also help facilitate the pumping of any water that may enter the excavation, especially if the excavation extends to the levels of perched groundwater encountered in our explorations.

Excavations for the basement of the residence will be as deep as 9 to 13 feet. Based on the soils encountered in our explorations, a temporary excavation inclination of no steeper than a 1:1 (Horizontal:Vertical) is appropriate for this project. Based on the current architectural layout, it appears that a temporary excavation cannot be maintained within the western property line. Because of this, and because of the location of the adjacent western residence, shoring is necessary. This shoring system will need to be rigid in design due to the limited property line setbacks, and presence of a neighboring residence close to the western property line. Depending on final foundation depths, and layout, temporary shoring may also be needed along portions of the eastern excavation line, as corners of the basement walls extend within a 1:1 (H:V) of the property line. It may be feasible to explore the possibility of obtaining a temporary excavation easement from the owner of the walking path east of the site so that the temporary excavation could be made to avoid shoring. In developing an excavation plan for this project, a nominal 2 feet of flat space along the outside edge of the foundations should be included to account for drainage installation as well as for work room on the outside of the concrete forms. Additional recommendations can be found in the ***Temporary Shoring*** and ***Excavations and Slopes*** sections of this report.

The onsite soils that will be excavated to construct the basement foundations will consist of variable silty sand and sand soils, all of which are fine-grained and are silty in composition. Much of the onsite soils were observed to be in an elevated moisture state and are not free-draining. Considering this, the onsite soils could be difficult to reuse for structural fill applications, especially during the winter and spring months. Reuse could also be difficult if earthwork is done in times of precipitation.

The test borings confirmed that the site is underlain both by medium-dense and denser sand, slightly silty sand, and silty sand. These soils are fine-grained and were observed to be in a glacially compressed state. The density and composition of these native soils will greatly slow, and essentially stop the downward percolation of stormwater. Furthermore, perched groundwater was revealed in three of the four test borings. Considering this, we do not recommend that concentrated infiltration or dispersion of stormwater be utilized at this site. Any attempt to infiltrate at this site will only increase the chances for causing adverse drainage impacts to the adjacent residences, proposed basement living space beneath the new residence at the site, as well as the adjacent lower, southern street. All collected stormwater runoff should be conveyed to the appropriate facilities.

The basement for the proposed residence will likely be excavated close to, or below the levels of the perched groundwater encountered in our explorations. During construction, this perched groundwater will likely be able to be controlled by portable pumps. However, we strongly recommend that a robust subsurface drainage and waterproofing system be included for the basement spaces. It would be practical to retain a building envelope consultant to help recommend the most practical waterproofing systems for this project. We recommend that the basement slab be underlain by an underslab drainage system consisting of a layer of clean drain rock, in which 4-inch diameter perforated PVC pipes are buried at spacings of no more than 15-feet center-to-center. These underslab drains would tie into the foundation drainage system where the collected water would be conveyed to the appropriate facilities. The ***Subsurface Drainage*** section of this report contains further drainage recommendations.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. We anticipate that a silt fence will be needed around the downslope sides of any cleared areas. Existing pavements, ground cover, and landscaping should be left in place wherever possible to minimize the amount of exposed soil. Rocked staging areas and construction access roads should be provided to reduce the amount of soil or mud carried off the property by trucks and equipment. Trucks should not be allowed to drive off of the rock-covered areas. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Following clearing or rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. On most construction projects, it is necessary to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

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.

As with any project that involves demolition of existing site buildings and/or extensive excavation and shoring, there is a potential risk of movement on surrounding properties. This can potentially translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. However, the demolition, shoring, and/or excavation work could just translate into *perceived* damage on adjacent properties. Unfortunately, it is becoming more and more common for adjacent property owners to make unsubstantiated damage claims on new projects that occur close to their developed lots. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring, and/or commencing with the excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners. Additionally, any adjacent structures should be monitored during demolition and construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods.

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 1.46g 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 0.68g. The soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature and lack 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 0.46g.

CONVENTIONAL FOUNDATIONS

The proposed residence can be supported on conventional continuous and spread footings bearing on undisturbed, dense, native soil, or on structural fill placed above this competent native soil. See the section entitled ***General Earthwork and Structural Fill*** for recommendations regarding the placement and compaction of structural fill beneath structures. Prior to placing structural fill beneath foundations, the excavation should be observed by the geotechnical engineer to document that adequate bearing soils have been exposed.

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

It is apparent that some overexcavation will be needed below the footings to expose competent native soil, especially beneath the shallower, on-grade structures. Unless lean concrete is used to fill an overexcavated hole, the overexcavation must be at least as wide at the bottom as the sum of the depth of the overexcavation and the footing width. For example, an overexcavation extending 2 feet below the bottom of a 2-foot-wide footing must be at least 4 feet wide at the base of the excavation. If lean concrete is used, the overexcavation need only extend 6 inches beyond the edges of the footing. A typical detail for overexcavation beneath footings is attached as Plate 7.

<u>An allowable bearing pressure of 3,500 pounds per square foot (psf) is appropriate for footings bearing directly on the competent, dense native soil or on lean-mix concrete placed over the dense soil. However, for any footing on soil structural fill, the allowable bearing pressure should only be 1,500 psf.</u> A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil, or on structural fill up to 5 feet in thickness, will be about one-half-inch, with differential settlements on the order of one-half-inch in a distance of 25 feet along a continuous footing with a uniform load.

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

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.50
Passive Earth Pressure	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, pipe piles may be more economical for the on-grade structures that flank the deeper basement of the residence. 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.

As a minimum, Schedule 40 pipe should be used. The site soils are not highly organic and are not located near salt water. As a result, they do not have an elevated corrosion potential. Considering this, it is our opinion that standard “black” pipe can be used, and corrosion protection, such as galvanizing, is not necessary for the pipe piles.

Mercer Island, like other jurisdictions, has adopted Seattle Director’s Rule 10-2009. Seattle Director’s Rule 10-2009 contains several prescriptive requirements related to the use of pipe piles having a diameter of less than 10 inches. Under Director’s Rule 10-2009, load tests are required on 3 percent of the installed piles up to a maximum of 5 piles, with a minimum of one pile load test on each project. Additionally, full-time observation of the pile installation by the geotechnical engineer-of-record is required by Director’s Rule 10-2009.

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. This is an ultimate value that does not include a safety factor. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate.

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
Coefficient of Friction	0.50
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 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 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. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls at one to 2 percent to reduce the potential for surface water to percolate into the backfill.

Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. Foundation drainage and waterproofing systems are not intended to handle large volumes of infiltrated water. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The recommended wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew, or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a buildup of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

The ***General***, ***Slabs-On-Grade***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop competent native soil, or on structural fill placed atop the competent native soils. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill. Building floors could also be constructed as a framed floor atop a crawlspace where desired.

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, near existing utilities and structures. It is important that vertical cuts not be made at the base of sloped cuts of this project. 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.

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

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

TEMPORARY SHORING

Cantilevered shoring has proven to be an efficient method for providing rigid excavation shoring where the total excavation depth does not exceed 15 feet. The shoring design should be submitted to Geotech Consultants, Inc. for review prior to beginning site excavation. We are available and would be pleased to assist in this design effort.

Soldier Pile Installation

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. We anticipate that the holes could be drilled without casing, but the contractor should be prepared to case the holes or use the slurry method if caving soil is encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered or restrained by one row of tiebacks, and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 35 pounds per cubic foot (pcf). The active pressure should act on the pile spacing above the bottom of excavation, and on the pile diameter below the base of the excavation. If needed, traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Existing adjacent western residence may exert surcharges on the proposed shoring wall, depending on the location of the shoring wall with respect to the adjacent residence. Slopes above the shoring walls will exert additional surcharge pressures. These surcharge pressures will vary, depending on the configuration of the cut slope and shoring wall. We can provide recommendations regarding slope and building surcharge pressures when the preliminary shoring design is completed.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 525 pcf. For temporary shoring, we recommend that a minimum factor of safety of 1.2 be included to overturning and sliding calculations when using this ultimate value. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on three times the grouted pile diameter. Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

EXCAVATION AND SHORING MONITORING

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. This can translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring or commencing excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners.

Additionally, the shoring walls and any adjacent foundations should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least every third installed soldier pile should be monitored by taking readings at the top of the pile. Additionally, benchmarks installed on the surrounding buildings should be monitored for at least vertical movement. We suggest taking the readings at least once a week, until it is established that no deflections are occurring. The initial readings for this monitoring should be taken before starting any demolition or excavation on the site.

DRAINAGE CONSIDERATIONS

If permanent foundation walls may be constructed against the shoring walls, a plastic-backed drainage composite, such as Miradrain, Battledrain, or similar, should be placed against the entire surface of the shoring prior to pouring the foundation wall. Weep pipes located no more than 6 feet on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the inside of the perimeter footing will be used to collect and carry the water discharged by the weep pipes to the storm system. Isolated zones of moisture or seepage can still reach the permanent wall where groundwater finds leaks or joints in the drainage composite. This is often an acceptable risk in unoccupied below-grade spaces, such as parking garages. However, formal waterproofing is typically necessary in areas where wet conditions at the face of the permanent wall will not be tolerable. If this is a concern, the permanent drainage and waterproofing system should be designed by a specialty consultant familiar with the expected subsurface conditions and proposed construction. A typical drainage detail for foundations against shoring piles is attached to this report as Plate 8.

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 9. In addition, underslab drainage should also be provided under the basement. A typical underslab drainage detail is attached to this report as Plate 10 for reference. 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 ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing 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.

Perched groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to 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 fills 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 onsite sand can be used as structural fill, but it will have to be in a suitable moisture condition. It appears that the sand will be naturally wet in the wetter periods of the year.

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

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath 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 TCHC LLC, the Granbois Family, 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.

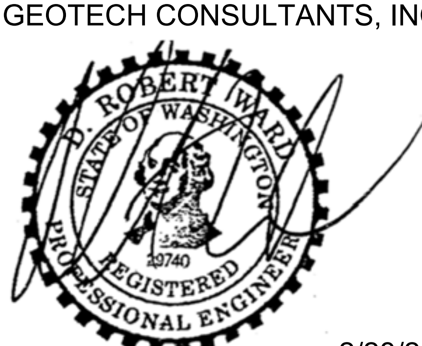
The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 6	Test Boring and Test Hole Logs
Plate 7	Typical Footing Overexcavation
Plate 8	Typical Shoring Drain Detail
Plate 9	Typical Footing Drain Detail
Plate 10	Typical Underslab Drainage 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.



2/28/2023

D. Robert Ward, P.E.
Principal

MKM/DRW:kg

NORTH

(Source: King County iMap)

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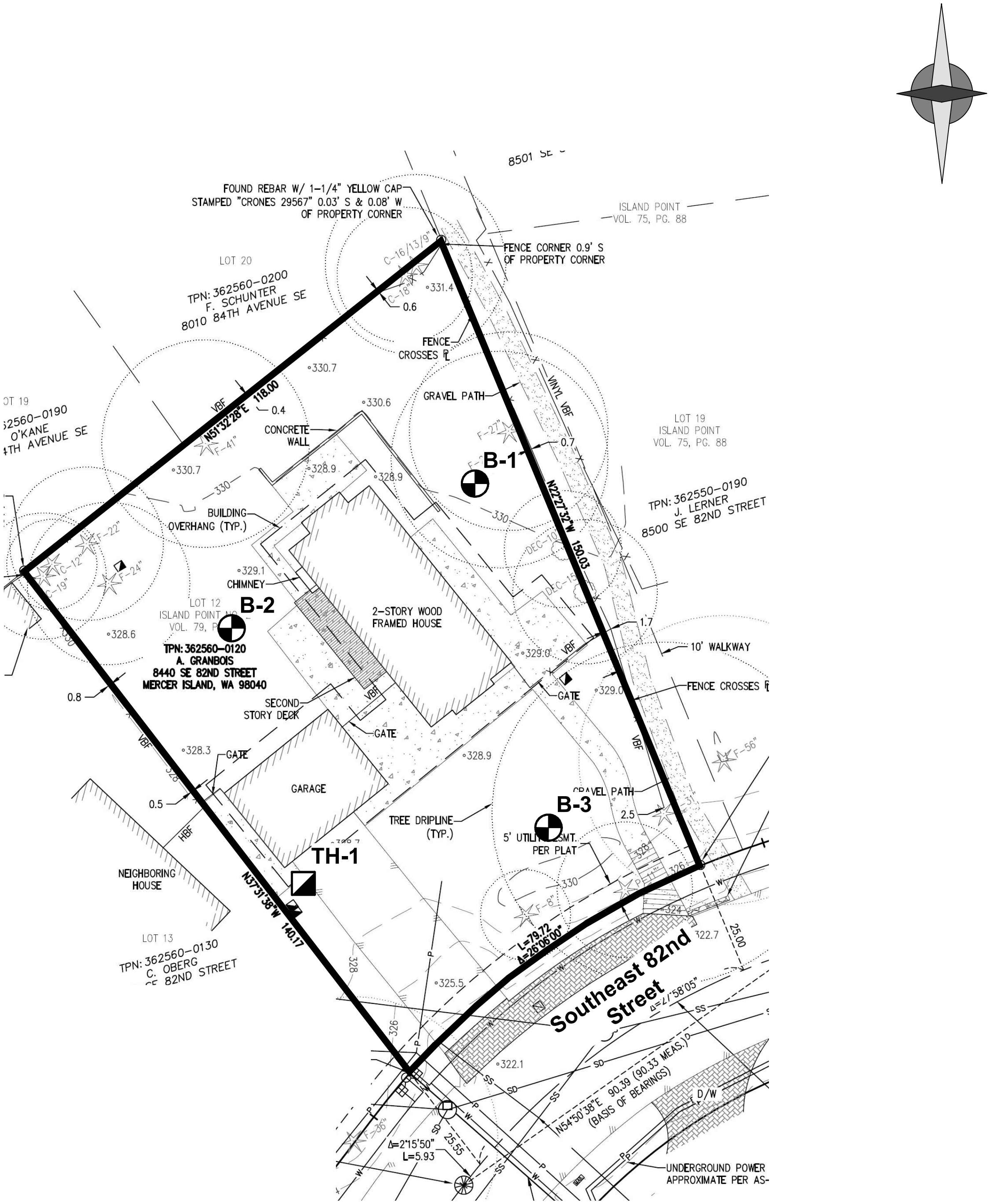
VICINITY MAP

8440 Southeast 82nd Street
Mercer Island, Washington

Job No:	Date:		Plate:
23014	Feb. 2023		1

NORTH



Legend:

Test Boring Location

Test Hole Location

SITE EXPLORATION PLAN

8440 Southeast 82nd Street
Mercer Island, Washington

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Job No:	Date:		Plate:
23014	Feb. 2023	No Scale	2

BORING 1

Elevation ±330 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Brown slightly gravelly, silty SAND with gravel and organics, fine-grained, moist, loose
			14	1		-becomes loose to medium-dense
5			14	2		-with trace roots, becomes gray-brown mottled orange
			41	3	SP	Gray-brown SAND with silt, fine-grained, moist, dense
10			35	4		-becomes gray with bedded rusting planes, very moist, slightly silty in seams
			28	5		-becomes wet
					SM/SP	Gray-brown mottled orange, silty SAND with gravel, very fine-grained, very moist, medium-dense to dense
15			50/6"	6		-becomes gray, slightly silty to silty, very dense
20			58	7	SP	Gray SAND, fine-grained, moist to very moist, very dense
25						
30						

* **Test boring was terminated at 21.5 feet on February 8, 2023.**

* **Perched groundwater was encountered from 12 to 13.5 feet during drilling.**

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TEST BORING LOG

8440 Southeast 82nd Street
Mercer Island, Washington

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23014	Feb. 2023	MKM	3

BORING 2

Elevation ±329 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Grass and topsoil over;
			11	1		Brown with trace mottling, slightly silty SAND with trace roots, fine-grained, moist, loose to medium-dense
5			12	2	SP SM	-becomes rusted with roots
			32	3		-becomes gray, very fine-grained to fine-grained, very moist, dense -becomes wet
10			42	4		-with thin lenses of clean sand and silt, becomes moist -becomes very moist to wet
			58	5		Gray with rusting, gravelly SAND with thin seams of cemented silty sand, fine to medium-grained, moist, very dense
15			83	6		-increased gravel content
					SP	
20			58	7		-becomes very moist
25			81	8		-grades to slightly silty sand, becomes very fine-grained
30						

*** Test boring was terminated at 26.5 feet on February 8, 2023.**

*** Slight perched groundwater was encountered from 9 to 10 feet during drilling.**

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TEST BORING LOG

8440 Southeast 82nd Street
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
23014	Feb. 2023	MKM	4

BORING 3

Elevation ±328 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0–7					FILL	Brown slightly silty SAND with gravel, fine to medium-grained, moist, loose **(FILL)**
5			11	1		
7–9.5			77	2	SM	Gray silty SAND, fine-grained, moist, very dense
9.5–18			71	3	SP	Gray gravelly SAND with rusted lenses, fine to medium-grained, moist, very dense
15			80/11"	4		-becomes slightly silty
			50/6"	5		-becomes fine to coarse-grained

*** Test boring was terminated at 18 feet on February 8. 2023 due to auger refusal.**

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

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TEST BORING LOG

8440 Southeast 82nd Street
Mercer Island, Washington

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TEST HOLE 1

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			FILL	Brown to drk-brown, slightly gravelly, silty SAND with roots, fine-grained, moist, loose **(FILL)**
			SP/SM	Brown slightly gravelly, SAND with silt and roots, fine-grained, moist, loose -becomes gray-brown mottled orange
5				
				-becomes slightly silty, dense
		▽		-becomes wet
10				

*** Test Hole terminated at 10 feet on February 8, 2023.**
*** Perched groundwater seepage was encountered at 9 feet during excavation.**
*** No caving observed during excavation.**

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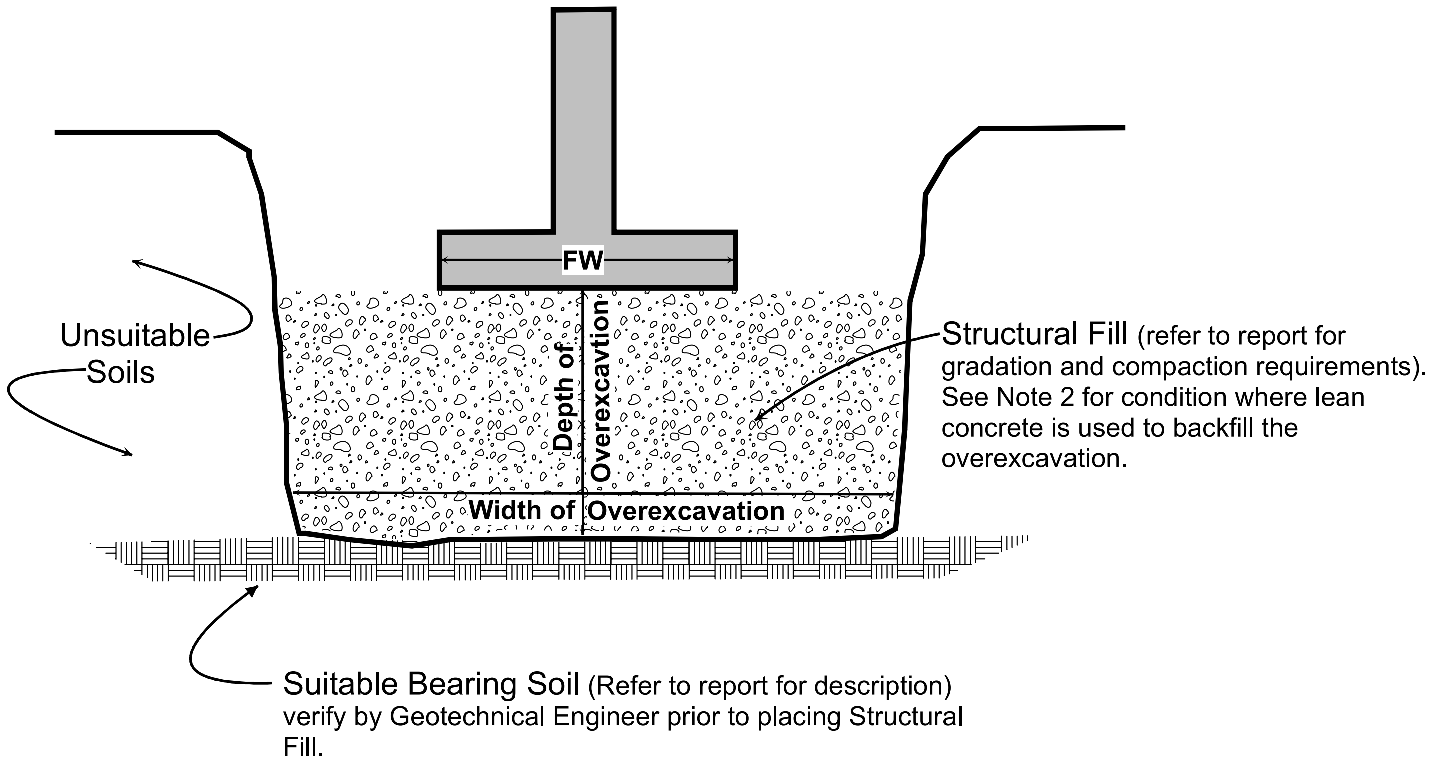
TEST HOLE LOG

8440 Southeast 82nd Street
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
23014	Feb. 2023	MKM	6



Width of Overexcavation = Footing Width (FW) + Depth of Overexcavation

NOTES:

1. Refer to report text for additional overexcavation, foundation, and structural fill considerations.
2. Where lean concrete (minimum 1-1/2 sacks of cement per cubic yard) is used to backfill the overexcavation, the overexcavation must extend only 6 inches beyond the edges of the footing.

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TYPICAL FOOTING OVEREXCAVATION

8440 Southeast 82nd Street
Mercer Island, Washington

Job No:	Date:		Plate:
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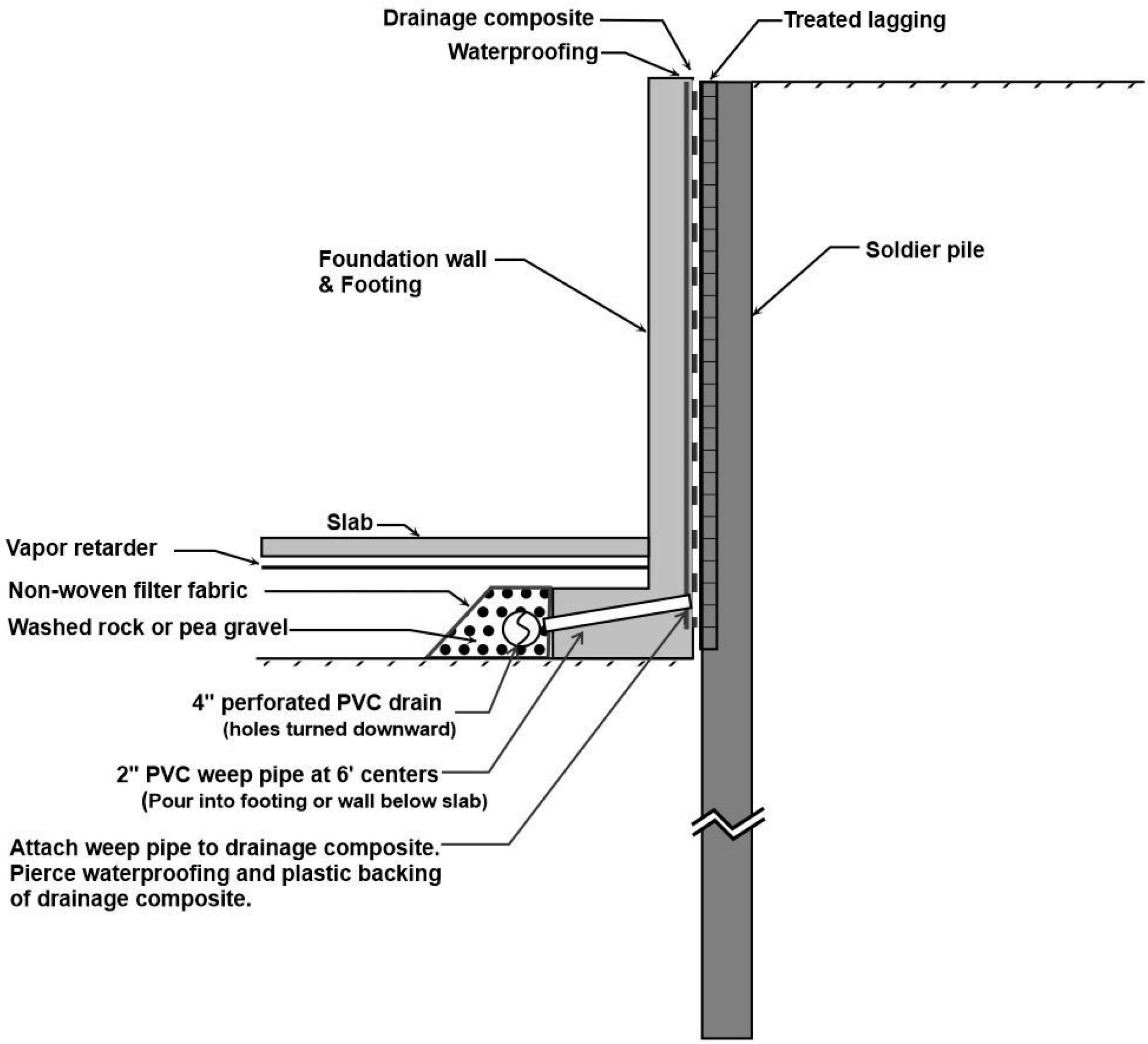

Note - Refer to the report for additional considerations related to drainage and waterproofing.

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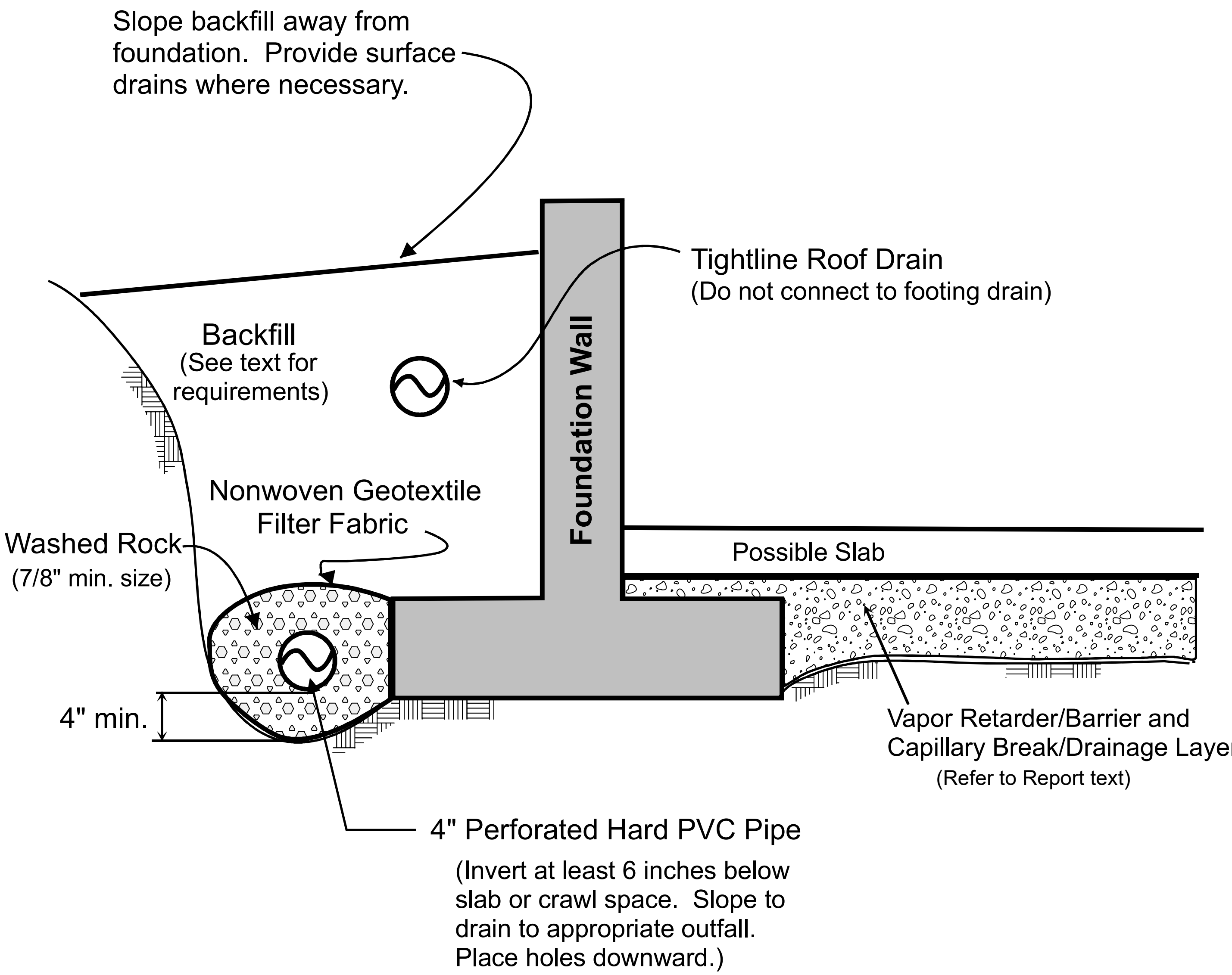
SHORING DRAIN DETAIL

8440 Southeast 82nd Street
Mercer Island, Washington

Job No:	Date:		Plate:
23014	Feb. 2023		8



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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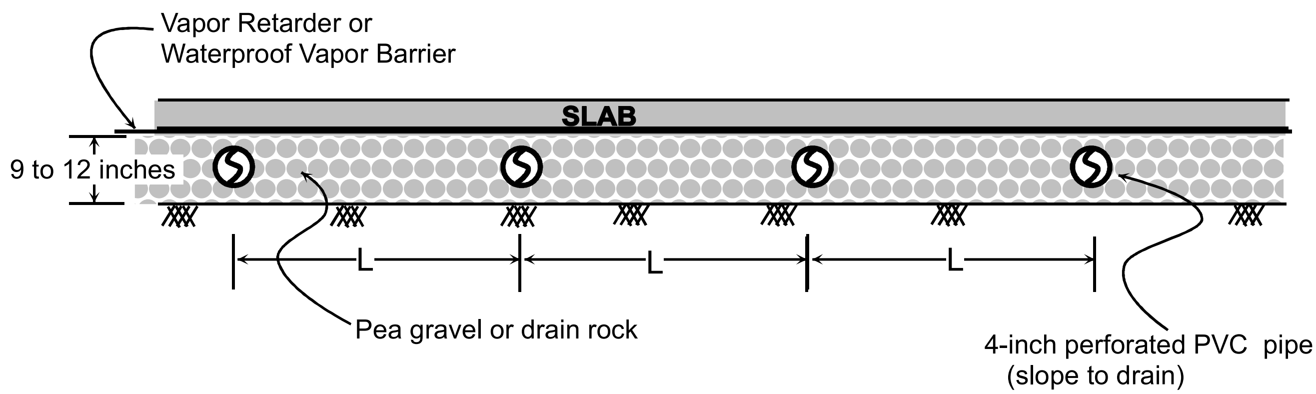
FOOTING DRAIN DETAIL

8440 Southeast 82nd Street
Mercer Island, Washington

Job No:	Date:		Plate:
23014	Feb. 2023		9



NOTES:

(1) Refer to the report text for additional drainage and waterproofing considerations.
(2) The typical maximum underslab drain separation (L) is 15 to 20 feet.
(3) No filter fabric is necessary beneath the pipes as long as a minimum thickness of 4 inches of rock is maintained beneath the pipes.
(4) The underslab drains and foundation drains should discharge to a suitable outfall.

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TYPICAL UNDERSLAB DRAINAGE

8440 Southeast 82nd Street
Mercer Island, Washington

Job No:	Date:		Plate:
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