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JN 19233

Richard and Leslie Day
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via email: rday@columbiasoft.com

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Day Residence Remodel
9843 Mercerwood Drive
Mercer Island, Washington

Greetings:

Attached to this transmittal letter is our geotechnical engineering report for the proposed remodel of your residence on Mercer Island. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design considerations for foundations, retaining walls, subsurface drainage, and temporary excavations. This work was authorized by your acceptance of our proposal, P-10370, dated May 22, 2019.

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.

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Geotechnical Engineer

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GEOTECHNICAL ENGINEERING STUDY
Proposed Day Residence Remodel
9843 Mercerwood Drive
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed residence remodel on Mercer Island.

The project is in the planning stage, and detailed plans were not available at the time of this study. We were provided with a topographic map developed by GeoDimensions and dated March 21, 2008. Based on conversations with Conard Romano Architects, we understand that the existing Day residence will be extensively remodeled; we understand that existing residence footprint will largely remain the same, but that some alterations requiring new foundations may be included. It is our understanding that the residence will probably remain one story, and no additional floors will be added to the house.

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 in the northeast quadrant of Mercer Island, upslope of East Mercer Way. The irregular-shaped subject site is located inside the curve of Mercerwood Drive. A cul-de-sac at the end of 98th Place Southeast abuts the site to the north.

A one-story residence with a main floor elevation of approximately 184.5 feet is located on the northern half of the property. A concrete driveway connects the northern adjacent cul-de-sac to an attached garage at the northern end of the house. The residence is roughly L-shaped, with a rectangular wing that extends southeast from the garage, paralleling the property's northeast property line. There are modular block walls to the north and east of this northern leg of the "L". The grade drops down to the north and east across these walls. An irregular-shaped wing extends southwest from the garage. The southwestern corner of the house overlies a basement (with a finished floor elevation of 176.3 feet) that daylights to the south-southwest. We observed several cracks in the middle of the south basement wall that could indicate excessive settlement of the shallower portion of the basement.

The ground surface slopes downwards from north to southeast across the site. A gently sloping yard is located north of the residence with a ground surface elevation ranging from 186 feet down to 182 feet. A stone-paver-covered deck is located south of the southeastern wing of the house at the main floor elevation. However, the grade drops from the deck down to a concrete patio southwest of the residence off the daylight basement; the lower patio also contains a pool. A rockery separates the lower patio/pool area and the upper deck. The southeastern edge of the property contains a steep slope that descends from the deck and patio levels, down to the Mercerwood Drive street elevation. Based on the provided topographic map, this southeastern steep slope has an inclination of 60 to 65 percent over a height of 18 to 20 feet.

The City of Mercer Island maps the subject site within several geologic hazard areas. Specifically, the subject site is mapped within both a potential landslide hazard area and an erosion hazard area. The northeast corner of the property is also mapped as a seismic hazard area. We did not observe any indications of recent slope instability on, or around the site during our recent site visit. Our review of the Landslide Hazard Assessment (Troos & Wisher, 2009) does not show any documented landslides within several blocks of the site.

SUBSURFACE

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

The borings were drilled on July 11, 2019 using a portable Acker drill. This drill system utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at approximate 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.

Soil Conditions

Test Borings 1 and 2 were conducted south of the residence at the lower patio/basement elevation and upper deck elevation respectively. Medium-dense, native, non-plastic, sandy silt was revealed below a depth of 2.5 feet in both test borings. The native soils became dense (glacially compressed) below 5 feet and became denser with depth. The silt became very dense below 15 feet in Test Boring 1, and extended to the maximum-explored depth of 21.5 feet.

Test Boring 3 was conducted near the southeast corner of the residence above the 3-foot-tall modular block wall at the top of the southeastern steep slope. Loose sandy silt fill soils with organics were encountered to a depth of 3 feet, overlying native, loose, non-plastic, sandy silt. The underlying native soils became medium-dense below 5 feet, dense below 10 feet, and very dense below 15 feet. At this location, probing indicates the bottom of the house's footing to be no more than 12 to 18 inches below the ground surface.

Near the northeast corner of the residence, Test Boring 4 encountered loose, native, non-plastic, sandy silt beneath the ground surface. The native silt became medium-dense below 5 feet and dense below 7.5 feet. The top of the footing in this area is approximately 2 feet below grade.

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 our subsurface explorations. The test borings were left open for only a short time period, and were conducted at the end of summer. It should be noted that groundwater levels vary seasonally with rainfall and other factors.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. 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 conducted for this study encountered native, medium-dense sandy silt at depths of 2.5 to 5 feet beneath the ground surface across the subject site. The native silt soils became denser with depth and extended to the maximum-explored depth of 21.5 feet below grade. Due to the hardscaping alongside most of the residence, it was difficult to accurately determine what soil the existing footings bear on. However, based on our observations and test borings, it appears that most of the existing basement's footings and the northern, upslope main floor footings likely bear on the medium-dense, native, sandy silt soils suitable to support the lightly-loaded building. However, based on test holes along the southeast and northeast perimeter footings and cracks in the southwest foundation, it appears likely that the southeast wing of the residence and potentially the southern perimeter of the basement bears on loose upper soils. Considering this, we recommend for design purposes, that the southern half of the basement and the southeastern wing of the main floor be underpinned with 2-inch-diameter pipe piles. Once construction begins, it will be possible to more-closely evaluate the transition between suitable bearing soil beneath the existing footings, and where pipe piles will be needed. The remaining existing northern footings can be re-used for a design allowable bearing pressure of 2,500 pounds per square foot (psf); however, if additional loads are applied to the existing foundations, we recommend they be underpinned as well. New conventional footings could be used for the northern half of the building footprint, provided they are supported on the medium-dense or denser native silt. Due to the adjacent steep slope along the southeast end of the property, we recommend any new foundations south of the existing house footprint be supported on pipe piles as well.

As previously discussed, the majority of the subject site is mapped by the City of Mercer Island as both a potential landslide area and an erosion hazard area. The northeast corner of the site is mapped as a seismic hazard area as well. Based on the dense underlying silt soils and lack of groundwater encountered in our subsurface explorations onsite, it is our opinion that the soil is not liquefiable and the site not a seismic hazard area. The test borings conducted for this study indicate the subject site and the core of the southeastern steep slope are comprised of dense to very dense sandy silt soils not susceptible to deep-seated soil movement. However, as with any steep slope in the Puget Sound region, there is the possibility of movement of the loose near-surface soils, such as the loose fill soils found behind the small block wall along the top of the slope near the southeast corner of the residence. These shallow "skin slides" most commonly occur after extended periods of heavy precipitation. The recommendation to support the downslope, southeastern and southwestern foundations on small-diameter pipe piles embedded into the dense underlying soils that comprise the core of the slope is intended to prevent the footing from becoming undermined in the event of potential future shallow soil movement.

The silt soils that underlie the subject site have a low recompacted strength and very poor drainage characteristics. Therefore, it will not be feasible to reuse the onsite soils for structural fill beneath foundations, wall backfill, or structural fill that will support on-grade elements. It will be necessary to import granular, well-draining material for structural fill.

In order to satisfy Mercer Island's requirements, we make the following statement:

> It is our professional opinion that the development practices that we have recommended in this report would render the proposed development as safe as if it were not located in a geologic hazard area.

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

The subject site also meets the criteria for an erosion hazard area. We have been associated with numerous projects involving excavation along the top of steep slopes and landslide hazard areas. Proper erosion control implementation will be sufficient to prevent adverse impacts to the steeply sloped southeastern edge of the subject site. 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. Wherever possible, the access roads should follow the alignment of planned pavements. 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.

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 1.40g and 0.54g, 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 0.58g. The soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature and the absence of near-surface groundwater.

CONVENTIONAL FOUNDATIONS

We recommend that new, 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.

Thickened slabs are often used to support interior walls. It is important to remember that thickened slab areas support building loads, just like conventional footings do. For this reason, the subgrade below thickened slabs must be prepared in the same way as for conventional footings. All unsuitable soils have to be removed and any structural fill compacted in accordance with the recommendations of this report. We recommend against the use of thickened slabs for most projects, particularly single-family residential, as it is difficult to ensure that the subgrades have been appropriately prepared. Also, the compacted slab fill has to be protected from disturbance by the earthwork, foundation, and utility contractors.

An allowable bearing pressure of 2,500 pounds per square foot (psf) is appropriate for new or existing footings supported on competent native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil, or on structural fill up to 5 feet in thickness, will be about one inch, with differential settlements on the order of one half-inch in a distance of 50 feet along a continuous footing with a uniform load.

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

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.40
Passive Earth Pressure	300 pcf

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

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

PIPE PILES

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

Three- or 4-inch-diameter pipe piles driven with a 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 for 3- or 4-inch piles.

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

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using a passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance. We recommend a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate passive value. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. As discussed in the ***General*** section, no passive pressure should be used along the southeast perimeter foundation due to the ground surface sloping away from the house to the southeast towards Mercerwood Drive.

As discussed above in the ***General*** section, if lateral resistance from fill placed against the foundations is required for this project, the structural engineer should indicate this requirement on the plans for the general and earthwork contractor's information. Compacted fill placed against the foundations can consist of imported soil that is tamped into place using the backhoe or is compacted using a jumping jack compactor. It is necessary for the fill to be compacted to a firm condition, but it does not need to reach even 90 percent relative compaction to develop the passive resistance recommended above. Due to their small diameter, the lateral capacity of vertical pipe piles is relatively small. However, if lateral resistance in addition to passive soil resistance is required, we recommend driving battered piles in the same direction as the applied lateral load. The lateral capacity of a battered pile is equal to one-half of the lateral component of the allowable compressive load, with a maximum allowable lateral capacity of 500 pounds. The allowable vertical capacity of battered piles does not need to be reduced if the piles are battered steeper than 1:5 (Horizontal:Vertical).

FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	40 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction	0.40
Soil Unit Weight	130 pcf

Where: pcf is Pounds per Cubic Foot, and Active and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure. This applies only to walls with level backfill.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

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

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**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. Free-draining backfill should be used for the entire width of the backfill where seepage is encountered. For increased protection, drainage composites should be placed along cut slope faces, and the walls should be backfilled entirely with free-draining soil. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

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

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

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

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

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 non-organic native soil, or on structural fill. 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.

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

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

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

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

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

EXCAVATIONS AND SLOPES

Temporary excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Also, temporary cuts should be planned to provide a minimum 2 to 3 feet of space for construction of foundations, walls, and drainage. Temporary cuts to a maximum overall depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Unless approved by the geotechnical engineer of record, it is important that vertical cuts not be made at the base of sloped cuts. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut.

The above-recommended temporary slope inclination is based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that 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 2.5:1 (H:V). 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 slope, and this may require the off-site disposal of any surplus soil.

DRAINAGE CONSIDERATIONS

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

As a minimum, a vapor retarder, as defined in the ***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.

No groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to a building should slope away at least one to 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section.

GENERAL EARTHWORK AND STRUCTURAL FILL

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

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

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).

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

LIMITATIONS

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

The recommendations presented in this report are directed toward the protection of only the residence from damage due to slope movement. Predicting the future behavior of steep slopes and the potential effects of development on their stability is an inexact and imperfect science that is currently based mostly on the past behavior of slopes with similar characteristics. Landslides and soil movement can occur on steep slopes before, during, or after the development of property. The owner of any property containing, or located close to steep slopes must ultimately accept the possibility that some slope movement could occur, resulting in possible loss of ground or damage to the facilities around the residence.

This report has been prepared for the exclusive use of Richard and Leslie Day 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 Logs
Plate 7	Typical Footing Drain Detail

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Adam S. Moyer
Geotechnical Engineer

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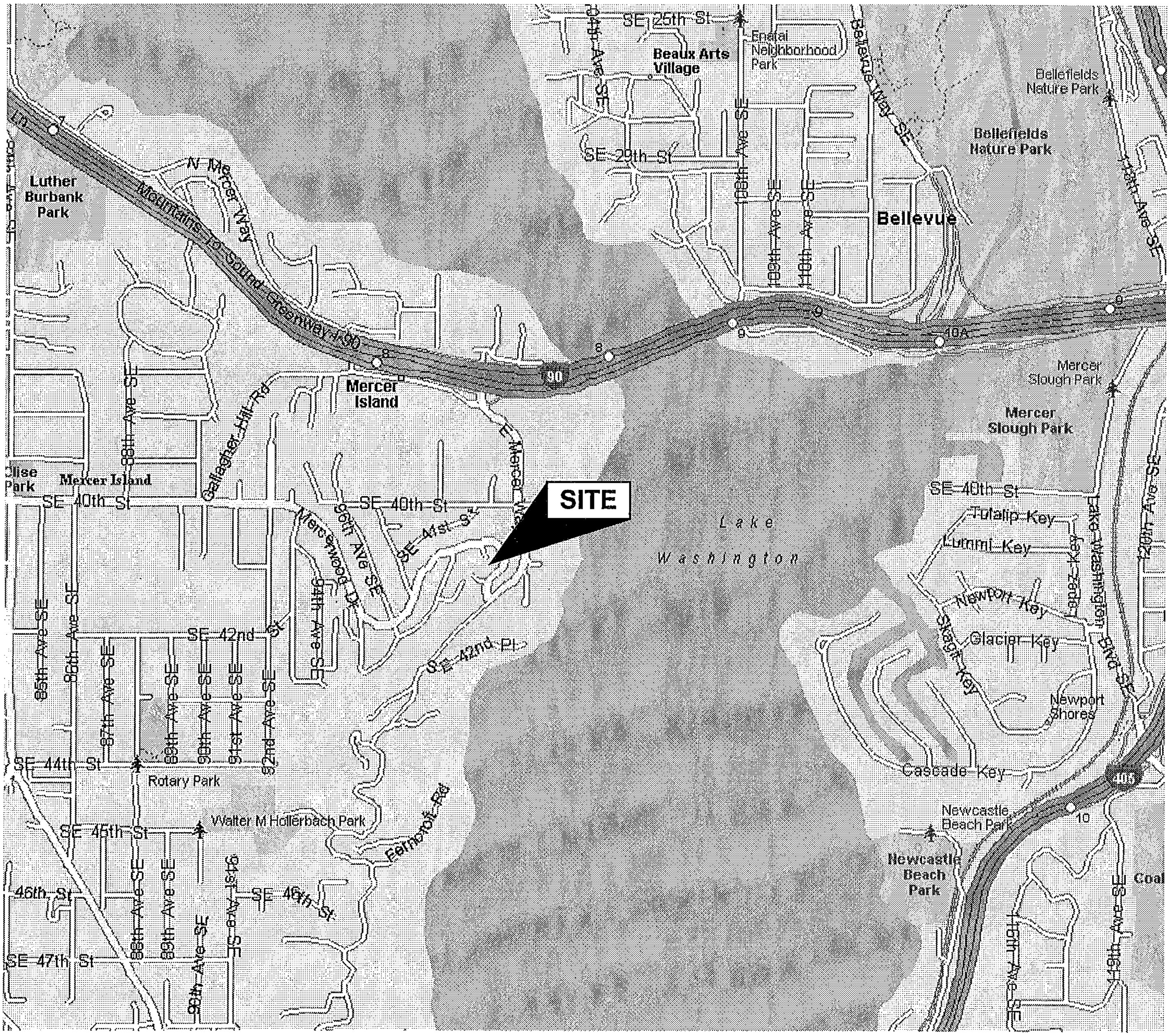
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER
08/19/19

Marc R. McGinnis, P.E.
Principal

ASM/MRM:kg

NORTH



(Source: Microsoft MapPoint, 2013)

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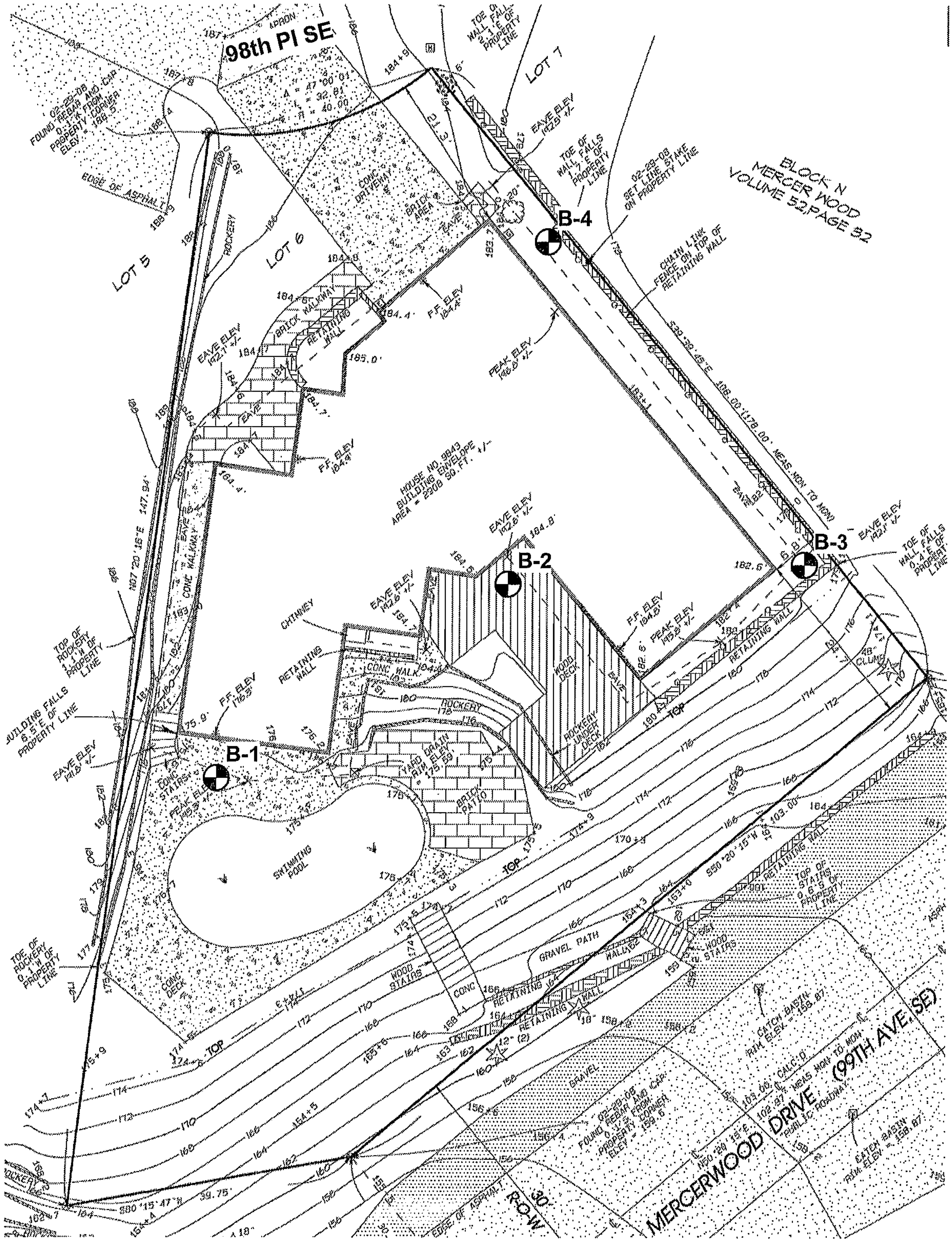
VICINITY MAP

9843 Mercerwood Drive
Mercer Island, Washington

Job No:	Date:		Plate:
19233	Aug. 2019		1

NORTH



Legend:

Test Boring Location

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SITE EXPLORATION PLAN

9843 Mercerwood Drive
Mercer Island, Washington

Job No:	Date:		Plate:
19233	Aug. 2019	No Scale	2

BORING 1

Elevation ± 175 Feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					ML	Tan sandy SILT, non-plastic, very fine-grained, moist, medium-dense
			22	1		
5						-becomes dense, slightly reduced sand content
			32	2		
						-becomes brown
			43	3		
10			37	4		
15						-becomes very dense
			67	5		
20			59	6		
25						

*** Test boring was terminated at 21.5 feet on July 11, 2019.**
*** No groundwater seepage was observed during drilling.**

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TEST BORING LOG

9843 Mercerwood Drive
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
19233	Aug. 2019	ASM	3

BORING 2

Elevation ± 184.5 Feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					ML	Tan sandy SILT, non-plastic, very fine-grained, thin laminations, moist, medium-dense
			14	1		
5			34	2		-becomes dense
			40	3		
10			47	4		
15						
20						
25						

*** Test boring was terminated at 11.5 feet on July 11, 2019.**
*** No groundwater seepage was observed during drilling.**

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TEST BORING LOG

9843 Mercerwood Drive
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
19233	Aug. 2019	ASM	4

BORING 3

Elevation ± 182 Feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Dark brown sandy SILT with organics, non-plastic, dry, loose **(FILL)**
			10	1	ML SM	Tan sandy SILT with occasional roots, non-plastic, dry, loose
5			15	2		-becomes brown, with trace fine-grained gravel, medium-dense
			22	3		-grades to very silty SAND, reduced gravel and roots content
10			35	4		-becomes dense
			35	5		
15			53	6		-becomes very dense
20						
25						

*** Test boring was terminated at 16.5 feet on July 11, 2019.**
*** No groundwater seepage was observed during drilling.**

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TEST BORING LOG

9843 Mercerwood Drive
Mercer Island, Washington

Job	*Date:*	*Logged by:*	*Plate:*
19233	Aug. 2019	ASM	5

BORING 4

Elevation ± 184 Feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					ML	Brown sandy SILT with trace gravel and roots, non-plastic, fine-grained, moist, loose
			4	1		
5			26	2		-becomes medium-dense, reduced gravel and roots content
			40	3		-becomes dense, with occasional thin vertical laninations
10			35	4		
15						
20						
25						

* **Test boring was terminated at 11.5 feet on July 11, 2019.**
* **No groundwater seepage was observed during drilling.**

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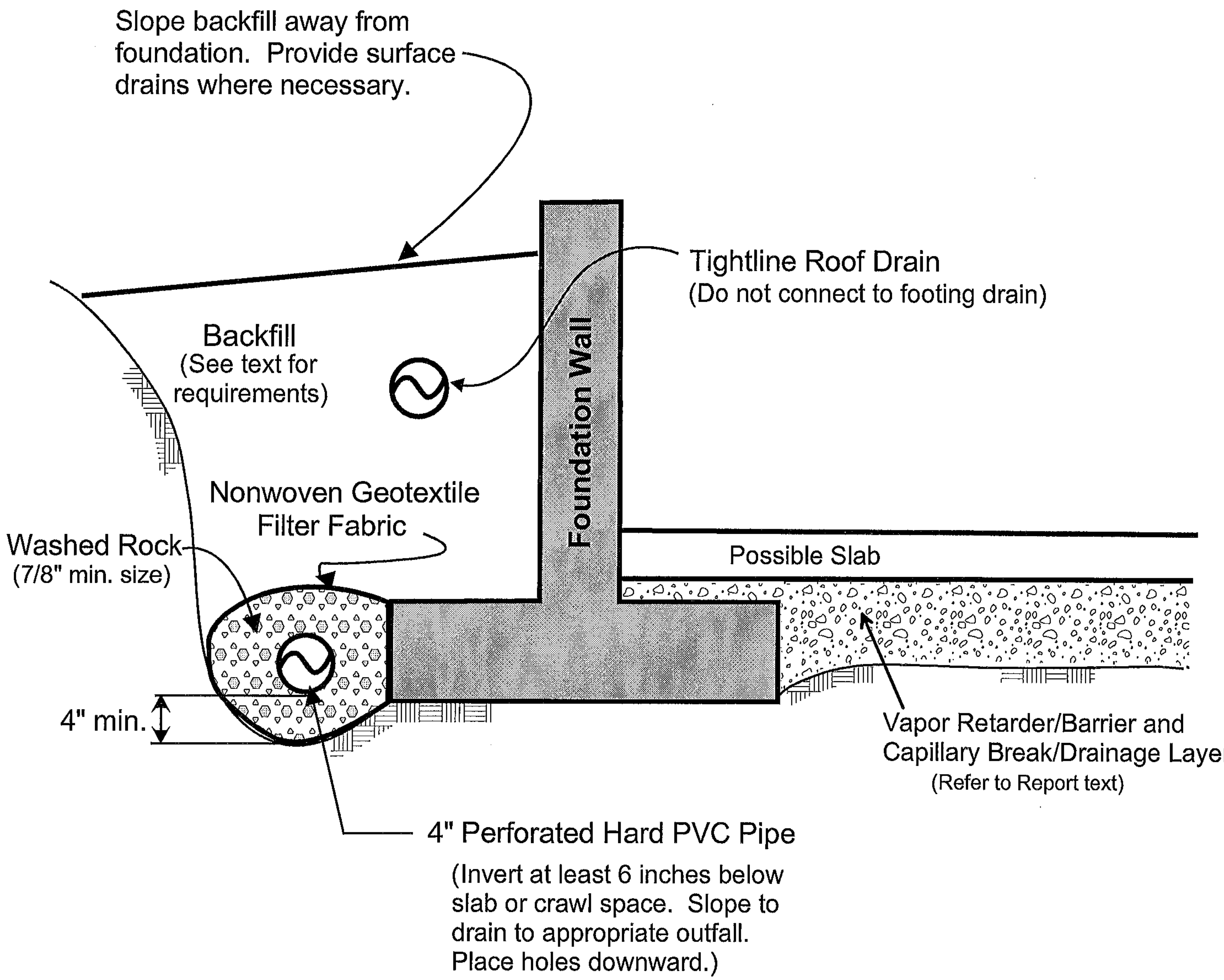
TEST BORING LOG
9843 Mercerwood Drive
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
19233	Aug. 2019	ASM	6



NOTES:

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

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

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FOOTING DRAIN DETAIL

9843 Mercerwood Drive
Mercer Island, Washington

Job No:	Date:		Plate:
19233	Aug. 2019		7