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May 15, 2020

JN 20086

F. Ross Murray
7675 Northeast 14th Street
Medina, Washington 98039
via email: *f.rossmurray@gmail.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study and Critical Area Study**
Proposed Murray Residence
4803 Forest Avenue Southeast
Mercer Island, Washington

Dear Mr. Murray:

Attached to this transmittal letter is our geotechnical engineering report and Critical Area Study related to geologic hazards for the proposed single-family residence to be constructed in Mercer Island. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork, stormwater infiltration considerations, and design considerations for foundations, retaining walls, subsurface drainage, and temporary excavations/shoring. This work was authorized by your acceptance of our proposal, P-10561.

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.

Marc R. McGinnis, P.E.
Principal

cc: **Richard Flake Architecture**
via email: *richard@rfarchitecture.com*

MRM:kg

GEOTECHNICAL ENGINEERING STUDY AND CRITICAL AREA STUDY
Proposed Murray Residence
4803 Forest Avenue Southeast
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study and Critical Area Study for the site of the proposed single-family residence to be located in Mercer Island. The scope of the Critical Area Study is intended to satisfy the requirements of the recently-adopted section 19.07.110 of the Mercer Island City Code (MICC), which applies to Critical Area Studies.

We were provided with preliminary plans and a topographic map. RF Architecture developed the provided plans, and the topographic map was prepared by Terrane. Based on the provided plans, we understand that the existing house will be demolished. The new residence will be larger, but will be located in the approximate center of the site, consistent with the existing house's location. The new home will have two floors overlying a basement that will daylight to a terrace on the west located at an elevation of 47 feet. The main floor elevation will be approximately 59 feet. There will be an attached garage located off the southeast corner of the house. The floor slab elevation for the garage will be approximately 65 feet, which is approximately 6 feet above the main floor elevation. Exterior steps along the north side of the house will extend down to the front entry from the driveway/garage level. Temporary excavations of up to approximately 10 feet below existing grade will be required for the deeper, eastern, portion of the basement. A retaining wall having a height of up to approximately 15 feet will be needed to retain the cuts necessary for the eastern side of the driveway and motorcourt to the north of the garage. Temporary cuts of up to 10 to 12 feet will be needed for the eastern portion of the garage itself.

Decks will extend westward from the main and upper floors over a patio at the lower level. This patio will transition to the above-mentioned exterior terrace. A backfilled retaining wall will support the western portion of the terrace. West of this will be a grass yard at an elevation of approximately 47 feet.

A below-grade loggia building will be constructed on the western portion of the property. The finish floor elevation of this building will be approximately 28.5 feet, requiring temporary excavations of up to 14 feet below the existing ground surface.

A sidewalk with multiple landings will extend east to west along the northern side of the site to allow pedestrian access from the driveway to the house and the western portion of the property and the dock.

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 at the southern end of Forest Avenue Southeast, immediately to the south of Miller Landing, a small public open space that extends to the shore of Lake Washington. Several public utilities, including a storm drain and a sanitary sewer pump station are located within the Miller Landing parcel. The sewer main extends along the western boundary of the subject site, and extends into a manhole located in the northwest corner of the lot, just south of Miller Landing.

The subject property is a large, irregularly-shaped lot accessed via a driveway that extends south from the intersection of Forest Avenue Southeast and 81st Avenue Southeast. The property upslope, to the east, of the site contains a recently-constructed residence. Our firm provided geotechnical engineering services associated with that house's construction. The structure is supported on small-diameter pipe piles driven to refusal in underlying glacially-compressed silt. Immediately to the south is a grass-covered strip of land that provides foot access to Lake Washington. A single-family home (#4811) is located on the lot to the south of this adjoining strip of land.

The subject site is currently developed with an older residence located on the south, central portion of the lot. This house is underlain by a west-facing daylight basement. The driveway extends to the eastern side of the house, but there is currently no garage or covered parking area. Sidewalks and rockeries cascade down the sloping ground along the north side of the house to an eastern yard area. To the east of this yard is lawn and landscaping that slopes down to the shore of Lake Washington. This area contains several rockeries and there are steps providing foot access to the lake.

A storm drain pipe serving upslope properties and the subject site is located between the south wall of the on-site residence and the southern property line. This storm drain extends to the shore of Lake Washington.

The ground surface on the site, and in the surrounding area, generally slopes from east to west at a gentle to moderate inclination. The topography on the site has been extensively modified by terracing of the back yard, and landscaping to the north of the house. There are localized steeper-than-40-percent areas on the property, but these appear to be: 1) manmade, and 2) less than 10 feet in height.

The City of Mercer Island GIS indicates that the entire site lies within mapped Potential Landslide, Seismic, and Erosion Hazard Areas. We did not observe any indications of recent slope instability on or around the site during our recent visit to the property. The Forest Avenue neighborhood lies within a well-documented ancient landslide. This slide affected the area between Lake Washington and the western edge of West Mercer Way. On aerial topographic maps the arcuate shape of the ancient landslide mass is readily apparent. A large amount of the soil involved in this ancient landslide accumulated in the bottom of Lake Washington, creating one of several "sunken forests" found within the depths of the lake. The ancient slide is thought to have occurred over 10,000 years ago, possibly as the last glaciers receded from the area, or as a result of a Cascadia Subduction Earthquake. The Forest Avenue area remains covered with 15 to 20 feet of loose, disturbed soils overlying glacially-compressed silt that was not involved in the ancient slide. Since the ancient slide, numerous smaller landslides have occurred in the remaining loose, disturbed soils, typically occurring on the steeper, taller slopes. These slides have also affected oversteepened cut and/or fill slopes. One such episode of soil movement apparently occurred during the excavation for the driveway of the house (#4753) located immediately north of Miller Landing. This required the installation of drilled soldier piles to complete the permanent retention of the excavation for that driveway, which is located downslope of the driveway that serves the subject site.

SUBSURFACE

The subsurface conditions were explored by drilling four test borings at the approximate locations shown on the Site Exploration Plan, Plate 2. We also reviewed the logs of test borings completed by our firm and others for the residence to the north of Miller Landing, and to the east of the subject lot.

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 May 13, 2020 using both a track-mounted, hollow-stem auger drill and a portable Acker drill. The Acker drill system utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at approximate 2.5- or 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

The subsurface explorations encountered soil conditions similar to those expected from previous explorations on the adjacent northern and eastern properties, and in the surrounding Forest Avenue neighborhood. The upper 10 to 15 feet of soil consisted of loose, jumbled, silty sand and silt containing varying amounts of organics. This soil appears to be colluvium resulting from the ancient landslide and subsequent slope movement and/or erosion.

In Boring 1, located in the area of the proposed loggia, we observed a layer of medium-dense sand underlying the colluvium.

Underlying these looser or disturbed soils, the borings revealed glacially-compressed silt. This soil is massive, and appears to have remained following the ancient landslide.

Groundwater Conditions

Groundwater seepage was observed only in Boring 1. This represents subsurface water perched on top of the glacially-compressed silt. It is common to encounter at least isolated zones of perched groundwater in these soil conditions, particularly following extended wet weather.

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.

CRITICAL AREAS STUDY (MICC 19.07)

Seismic Hazard and Potential Landslide Hazard Areas: The entire subject site is located within a mapped Seismic Hazard Area and a Potential Landslide Hazard area. Following the ancient landslide, no recent large-scale slope movement has been documented in this area. Shallow slope failures affecting looser fill and/or native soils on steeper slopes or oversteepened cuts have occurred on several lots in the Forest Avenue neighborhood. The recommendations presented in this report are intended to maintain, and improve, stability of the site and the neighboring properties through the use of excavation shoring, engineered retaining walls, subsurface drainage, and deep foundations. No buffers or setbacks from slopes are needed for this project to address the Potential Landslide Hazard designation.

The proposed structures for the development will be supported on deep foundations embedded into the glacially compressed soils that are not liquefiable, due to their dense nature. This mitigates the Seismic Hazard.

Steep Slope Hazard Areas: There are no steeper-than-40-percent slopes on, or near, the site that would be classified under Mercer Island Code as a Steep Slope Hazard Area. The recommendations presented in the report are intended to prevent adverse impacts to the stability of the slopes on the site and the neighboring properties.

Erosion Hazard Areas: The site meets the City of Mercer Island's criteria for an Erosion Hazard Area, due to the soils present and the slope of the ground surface. We have worked on numerous waterfront projects on Mercer Island that have avoided siltation of the lake and surrounding properties by exercising care and being proactive with the maintenance and potential upgrading of the erosion control system through the entire construction process. The temporary erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered during the site work. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil areas to prevent accumulated water or runoff from the work area from becoming silty in the first place. Silty water cannot be discharged to the lake, so a temporary holding tank should be planned for wet weather earthwork. Two wire-backed silts fence bedded in compost, not native soil or sand, should be erected as close as possible to the planned work area, and the existing vegetation between the silt fence and the lake left in place. Typically, if wet weather construction is anticipated, two parallel silt fences should be installed along the shoreline. Rocked construction access and staging areas should be established wherever trucks will have to drive off of pavement, in order reduce the amount of soil or mud carried off the property by trucks and equipment. It will also be important to cap any existing drain lines found running toward the lake until excavation is completed. This will reduce the potential for silty water finding an old pipe and flowing into the lake. Covering the base of the excavation with a layer of clean gravel or rock is also prudent to reduce the amount of mud and silty water generated. Utilities reaching between the house and the lake should not be installed during rainy weather, and any disturbed area caused by the utility installation should be minimized by using small equipment. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface.

Buffers and Mitigation: Under MICC 19.07.160(C), a prescriptive buffer of 25 feet is indicated from all sides of a shallow landslide-hazard area. As noted above, the entire site lies within a mapped Potential Landslide Hazard Area, and the prescriptive buffer would extend far beyond the boundaries of the property and the planned development area. No Steep Slope buffer would apply to this project, and no buffer is required by the MICC for an Erosion Hazard Area. We recognize that the planned development will occur within the designated critical areas. The recommendations presented in this geotechnical report are intended to allow the project to be constructed in the proposed configuration without adverse impacts to critical areas on the site or the neighboring properties. The geotechnical recommendations associated with foundations, shoring, and erosion control will mitigate any potential hazards to critical areas on the site.

Statement of Risk: In order to satisfy the City of Mercer Island's requirements, a statement of risk is needed. As such, we make the following statement:

> *Provided the recommendations in this report are followed, it is our professional opinion that the development can be constructed so that the risk to the project and adjacent property is mitigated such that the site is determined to be safe.*

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 colluvium overlying glacially-compressed silt. While the deeper portions of the excavation for the garage, house and loggia may reach the competent silt, we recommend that all of the structures be supported on driven, small-diameter pipe piles driven to refusal in the glacially-compressed silt. The piles should be used to support not only building loads, but also settlement-sensitive floor slabs, porches, decks, etc.

Excavation for the proposed residence's basement will extend approximately 10 to 13 feet beneath the surrounding ground surface. Temporary open cut slopes may be excavated at an inclination no steeper than 1.5:1 (Horizontal:Vertical) in the onsite soils. Slope cuts should not be taller than approximately 8 feet. These cuts also cannot extend below a 2:1 (H:V) from existing retaining walls, utilities, driveways, etc. We expected that much of the excavation for the driveway, garage, house, and loggia will require temporary shoring to make the cuts and protect the surrounding properties. The site and soil conditions make cantilevered soldier piles the most appropriate shoring system for this project. The soldier piles could be designed to permanently resist lateral earth pressures, substantially reducing the lateral loads that will have to be resisted by the foundations. If the space between the soldier piles and the permanent basement walls is backfilled with geofoam, the lateral earth pressure acting on the basement walls will only be 5 pounds per cubic foot (pcf). Geofoam is self-supporting, and the 5 pcf earth pressure accounts for the need to install a 6-inch width of free-draining crushed rock between the geofoam and the concrete wall.

Any on-grade elements, such as driveways, patios, sidewalks, etc. that are placed on the native soils will tend to settle slightly over time. The only way to prevent this is to support them structurally on piles.

Wet weather construction (October 1 through March 31) on this site should be possible without adverse impacts to the surrounding properties. The above section entitled **Erosion Hazard Areas** covers temporary erosion control measures that would be prudent. In preventing erosion control problems on any site, it is most important that any disturbed soil areas be immediately protected.

This requires diligence and frequent communication on the part of the general contractor and earthwork subcontractor. As with all construction projects undertaken during potentially wet conditions, it is important that the contractor's on-site personnel are familiar with erosion control measures and that they monitor their performance on a regular basis. It is also appropriate for them to take immediate action to correct any erosion control problems that may develop, without waiting for input from the geotechnical engineer or representatives of the City.

All, or the vast majority, of the excavated soil will be unsuitable for reuse on the site. These soils will be silty and fine-grained, and will have a high moisture content. They have poor drainage characteristics and low compacted strength, and will present an erosion control problem. As a result, we expect that excavated soils will have to be hauled off the site, and imported granular fill will be needed for the project.

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, retaining walls, 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, including the existing retaining walls, 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 E (Soft Soil). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.44g and 0.55g, 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 layer of wet, medium-dense sand encountered in Boring 1 is potentially susceptible to seismic liquefaction. However, this layer is relatively thin. The use of deep foundations embedded into dense, non-liquefiable soils mitigates the risk of potential foundation collapse in the event of seismic liquefaction of the overlying loose soils.

PIPE PILES

Four- or 6-inch-diameter pipe piles driven with 1,100-, 2,000-pound, or 3,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (1,100-pound hammer)	FINAL DRIVING RATE (2,000-pound hammer)	FINAL DRIVING RATE (3,000-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
4 inches	10 sec/inch	4 sec/inch	n/a	10 tons
6 inches	20 sec/inch	10 sec/inch	6 sec/inch	15 tons

These capacities have been well documented over the past 20 years for piles installed using hydraulic hammers on free-swinging leads. In our professional opinion, load tests are not required to verify the above-recommended allowable capacities for piles installed to refusal under the observation of the geotechnical engineer of record.

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.

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 an ultimate passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance. 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 compacted backfill:

PARAMETER	VALUE
Active Earth Pressure *	45 pcf
Passive Earth Pressure	300 pcf
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. The existing site retaining wall north of the proposed residence and covered walkway will likely place a surcharge onto the proposed structures' northern foundation walls. We can provide appropriate surcharge loads once more detailed plans have been developed. It may be possible for the excavation shoring to be designed to withstand this surcharge. 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.

Earth pressures for design of walls with geofoam backfill are discussed in the ***General*** section.

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. The on-site soils are not free-draining, and would have a low compacted strength. They should not be reused as wall backfill.

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

FLOOR SLABS

As discussed in the ***General*** section, the floors of the structures should be designed to span between the pile-supported foundations.

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. or crushed rock are typically used for this layer.

We recommend that underslab drainage be provided below floor slabs that are located more than approximately 5 feet below the surrounding grade. A typical underslab drainage detail is attached as Plate 7.

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***, ***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. 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 C. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1.5:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. We recommend a maximum unshored slope cut height of 8 feet. Additional considerations for temporary cuts, especially those near existing structures, including walls, are discussed in the ***General*** section.

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 sand or 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). Fill slopes should not be constructed with an inclination greater than 2.5:1 (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.

EXCAVATION SHORING

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. 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 should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 40 pounds per cubic foot (pcf). If the soldier piles will permanently restrain soil loads, an active fluid density of 45 pcf should be assumed, and the above-recommended seismic surcharge should be included in the design.

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. The existing site retaining wall north of the proposed residence and covered walkway may place a surcharge onto the proposed northern shoring walls. We can provide appropriate surcharge loads once more detailed plans have been developed. 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 retaining wall 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 400 pcf. No safety factor is included in the given value. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two 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." A typical Cantilevered Soldier Pile Shoring detail is attached to this report as Plate 8.

EXCAVATION AND SHORING MONITORING

The house upslope to the east of the site is supported on piles, and there are no other settlement-sensitive structures close to the property lines. As a result, the potential for excavation and/or shoring having adverse impacts on the surrounding properties is low. Even so, 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 vertical movement. 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 other 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

We anticipate that permanent foundation walls may be constructed against the shoring walls. Where this occurs, 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. Plate 9 presents typical considerations for foundation drains at shoring walls.

Footing drains placed inside the building, outside of the building, or behind backfilled walls should consist of 4-inch, perforated PVC pipe surrounded by at least 6 inches of 1-inch-minus, washed rock wrapped in a 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 level of a crawl space or the bottom of a floor slab, and it should be sloped slightly for drainage. All roof and surface water drains must be kept separate from the foundation drain system.

Footing drains outside of the building 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. A typical footing drain detail is attached to this report as Plate 10. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

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

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 excavations 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. As discussed in the ***General*** section, the native on-site soils are not suitable for reuse as structural fill, due to their high fines content and moisture sensitivity. The onsite gravelly, slightly silty sand fill soils could potentially be re-used as structural fill provided they can be placed and compacted near their optimum moisture content.

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).

The onsite soils will have high silt and moisture contents, making them impossible to accurately recompact for wall backfill or other structural uses. They also will have poor drainage characteristics. 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 subsurface explorations are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings and test holes. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of F. Ross Murray, and his 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	Cantilevered Soldier Pile Shoring
Plate 8	Underslab Drainage Detail
Plate 9	Shoring Drain Detail
Plate 10	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.

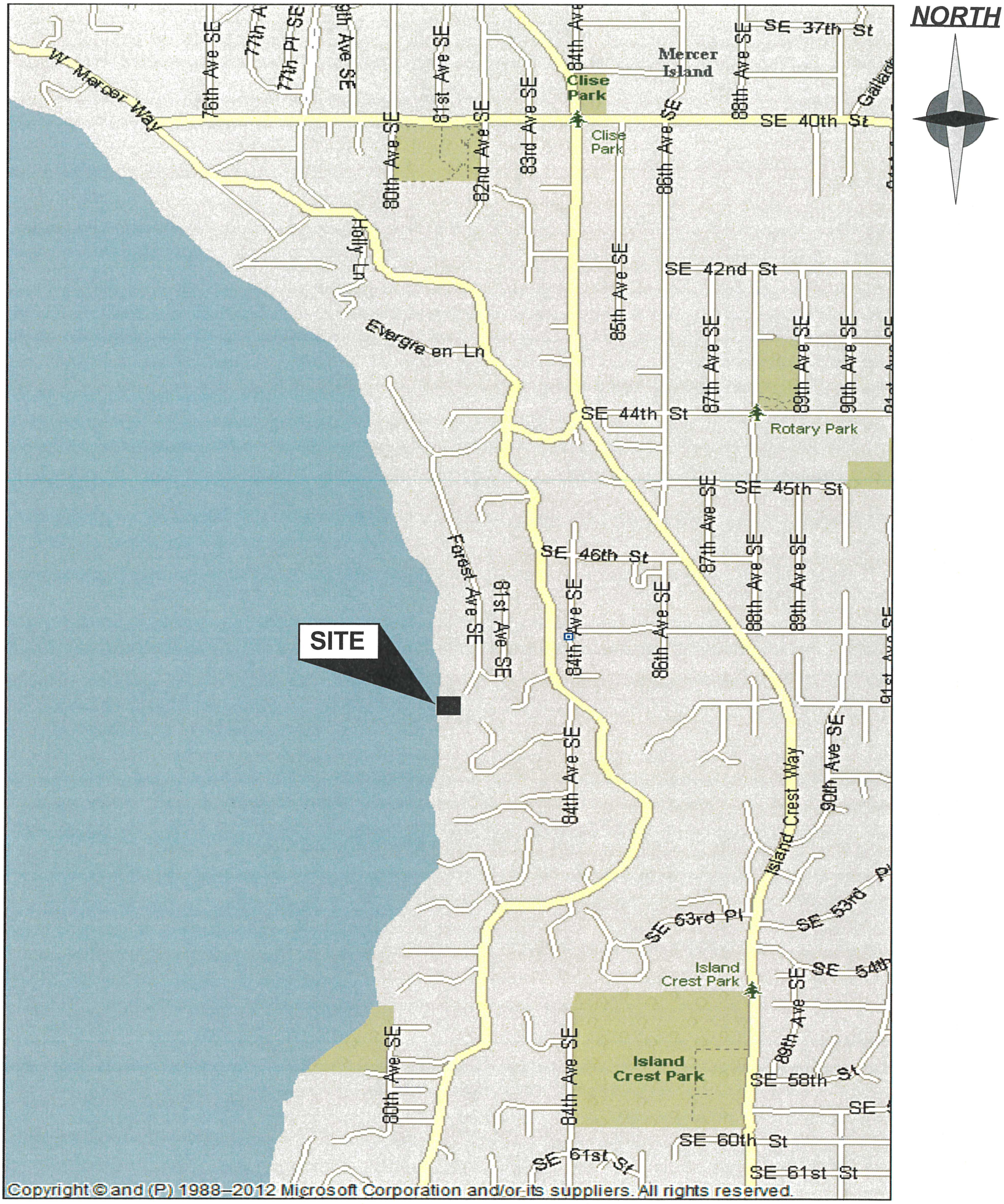
Respectfully submitted,

GEOTECH CONSULTANTS, INC.

05/15/20

Marc R. McGinnis, P.E.
Principal

MRM:kg



(Source: Microsoft MapPoint, 2013)

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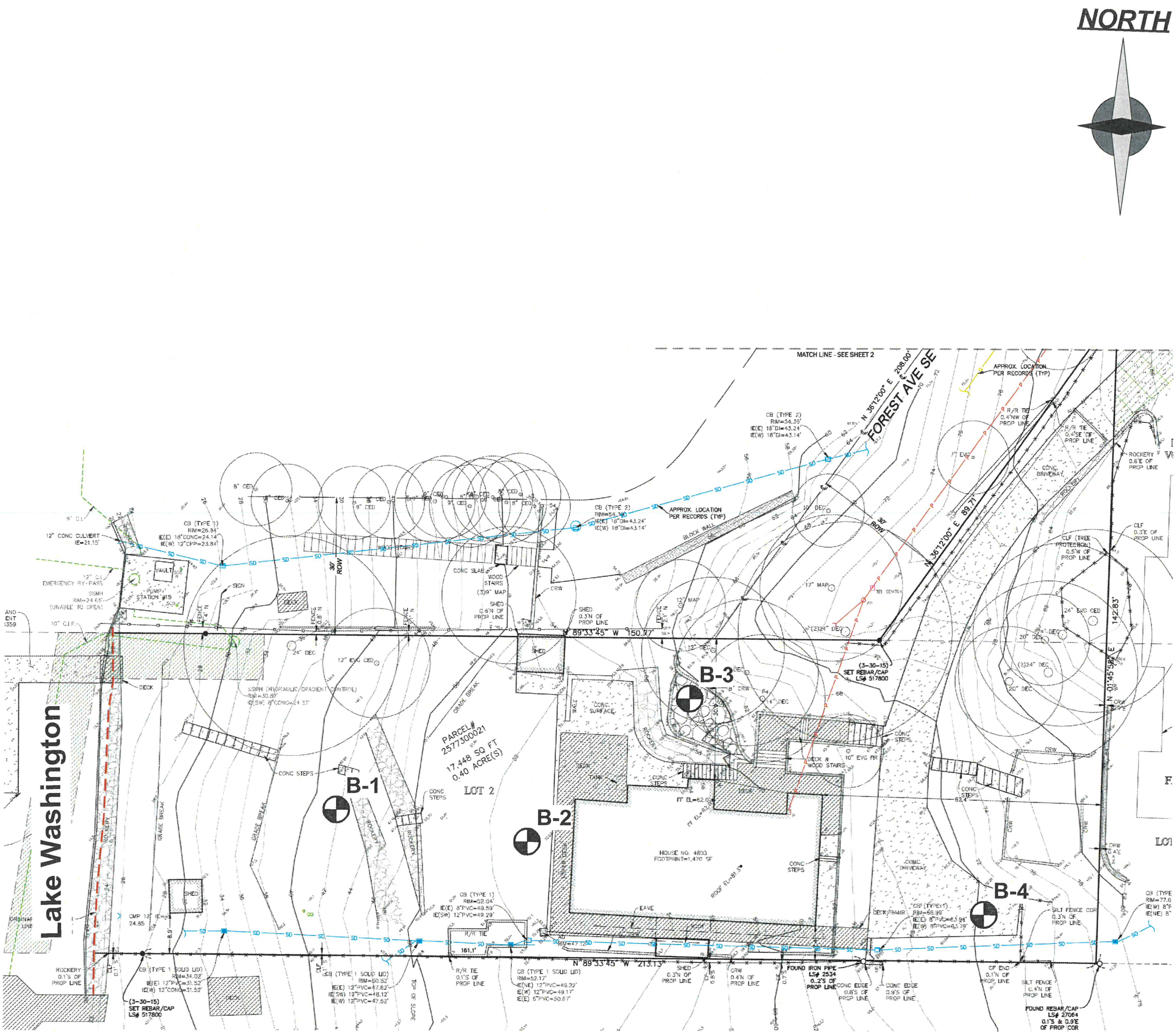
VICINITY MAP

4803 Forest Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
20086	May 2020		1



Legend:

Test Boring Location

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SITE EXPLORATION PLAN

4803 Forest Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
20086	May 2020	No Scale	2

BORING 1

Elevation ±42 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SP	Brown gravelly SAND with silt, fine- to medium-grained, moist, loose (Colluvium)
			12	1		
5			11	2		-becomes loose to medium-dense, with rust-brown mottling
			16	3		-with occasional gray silt inclusions
10			11	4	ML	Gray with rust mottling, SILT with occasional wood and organics, slightly plastic, jumbled, medium stiff to stiff (Colluvium)
			10	5		-becomes blue-gray, stiff
15			15	6		-interbedded with gravelly SAND, fine- to medium-grained, moist to very moist, medium-dense
					SP	Gray-brown gravelly SAND, fine- to medium-grained, wet, medium-dense
20			20	7		
			17	8		
			24/10"	9	ML	Gray-brown sandy SILT, non-plastic, moist, dense
25						
30						

* **Test boring was terminated at 24.3 feet on May 13, 2020.**

* **Groundwater seepage was encountered from 18 to 24 feet during drilling.**

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TEST BORING LOG

4803 Forest Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
20086	May 2020	ASM	3

BORING 2

Elevation ±52.5 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Gray with rust mottling, sandy SILT with occasional gravel and decomposed wood, non-plastic, moist, loose (FILL/Colluvium)
			5	1	ML	
5			6	2		
			7	3		Gray-brown with rust fracturing, SILT with sand and occasional decomposed wood, slightly plastic, moist, medium stiff (Colluvium)
10						-becomes blue-gray
			8	4	ML	-becomes gray-brown, slightly fractured, stiff, reduced sand and wood content, with trace roots
15			19	5		Gray-brown SILT with trace gravel, slightly plastic, slightly fractured, moist, stiff to very stiff
					ML	
20						-becomes very stiff to hard, no fracturing, with occasional thin fine-grained sand seams
			32/10"	6		
25						
30						

* **Test boring was terminated at 21.5 feet on May 13, 2020.**
* **No groundwater was encountered during drilling.**

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TEST BORING LOG

4803 Forest Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
20086	May 2020	ASM	4

BORING 3

Elevation ±60 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Gray-brown SILT with occasional gravel, slightly plastic, blocky, moderately fractured, moist, stiff (Colluvium)
			9	1		
5					ML	-reduced gravel content, with vertical bedding, becomes very stiff
			14	2		
			18	3		
10						Gray-brown SILT, non-plastic, very slightly fractured, with horizontal bedding, moist medium-dense
			26	4		
15			24	5	ML	
20			60	6		-becomes gray, massive, very dense
25						
30						

*** Test boring was terminated at 21.5 feet on May 13, 2020.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

4803 Forest Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
20086	May 2020	ASM	5

BORING 4

Elevation ±71 feet

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Gray-brown gravelly SAND, fine- to coarse-grained, moist, loose **(FILL)**
			4	1	ML	Gray SILT with occasional roots, slightly plastic, jumbled, moist, medium stiff (Colluvium)
5			5	2		-becomes plastic
			9	3		-reduced roots content, slightly fractured, stiff
10			22	4	ML	Gray SILT, plastic, massive, moist, very stiff
15			32	5		-becomes very stiff to hard
20			32	6		-with sand, becomes non-plastic, dense
25			36	7		
30						

*** Test boring was terminated at 26.5 feet on May 13, 2020.**
*** No groundwater was encountered during drilling.**

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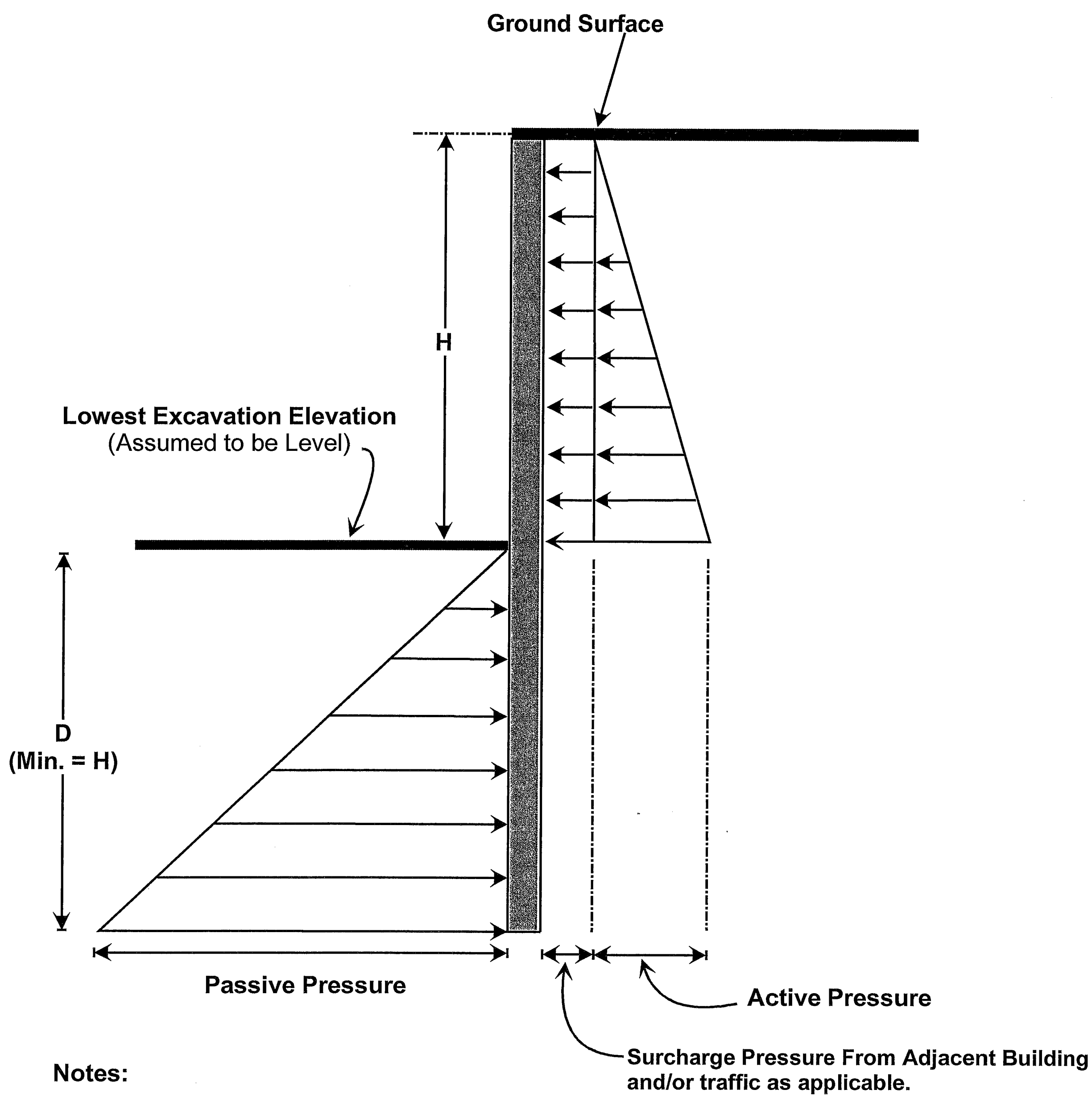
TEST BORING LOG

4803 Forest Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
20086	May 2020	ASM	6



Notes:

(1) The report should be referenced for specifics regarding design and installation.
(2) Active pressures act over the pile spacing.
(3) Passive pressures act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller.
(4) It is assumed that no hydrostatic pressures act on the back of the shoring walls.
(5) Cut slopes or adjacent structures positioned above or behind shoring will exert additional pressures on the shoring wall.

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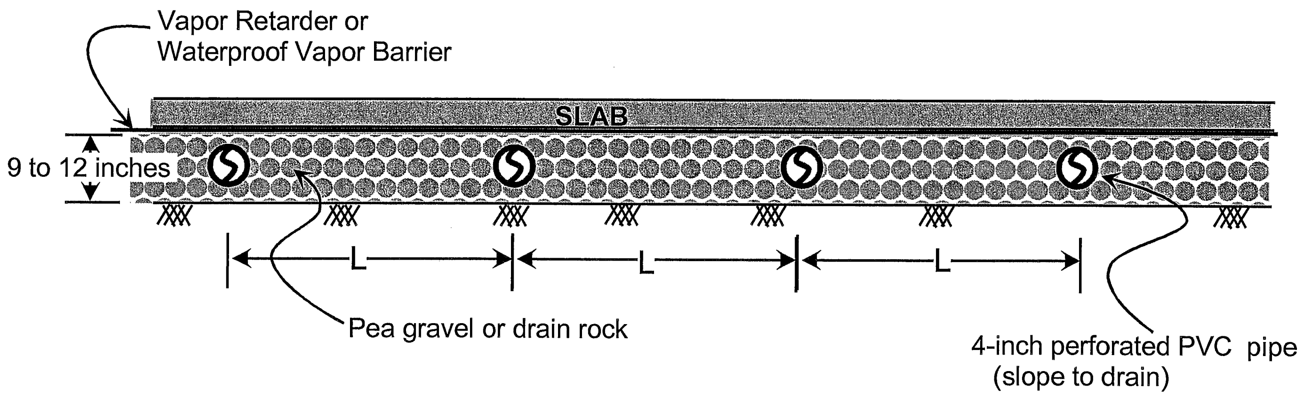
CANTILEVERED SOLDIER PILE SHORING

4803 Forest Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
20086	May 2020		7



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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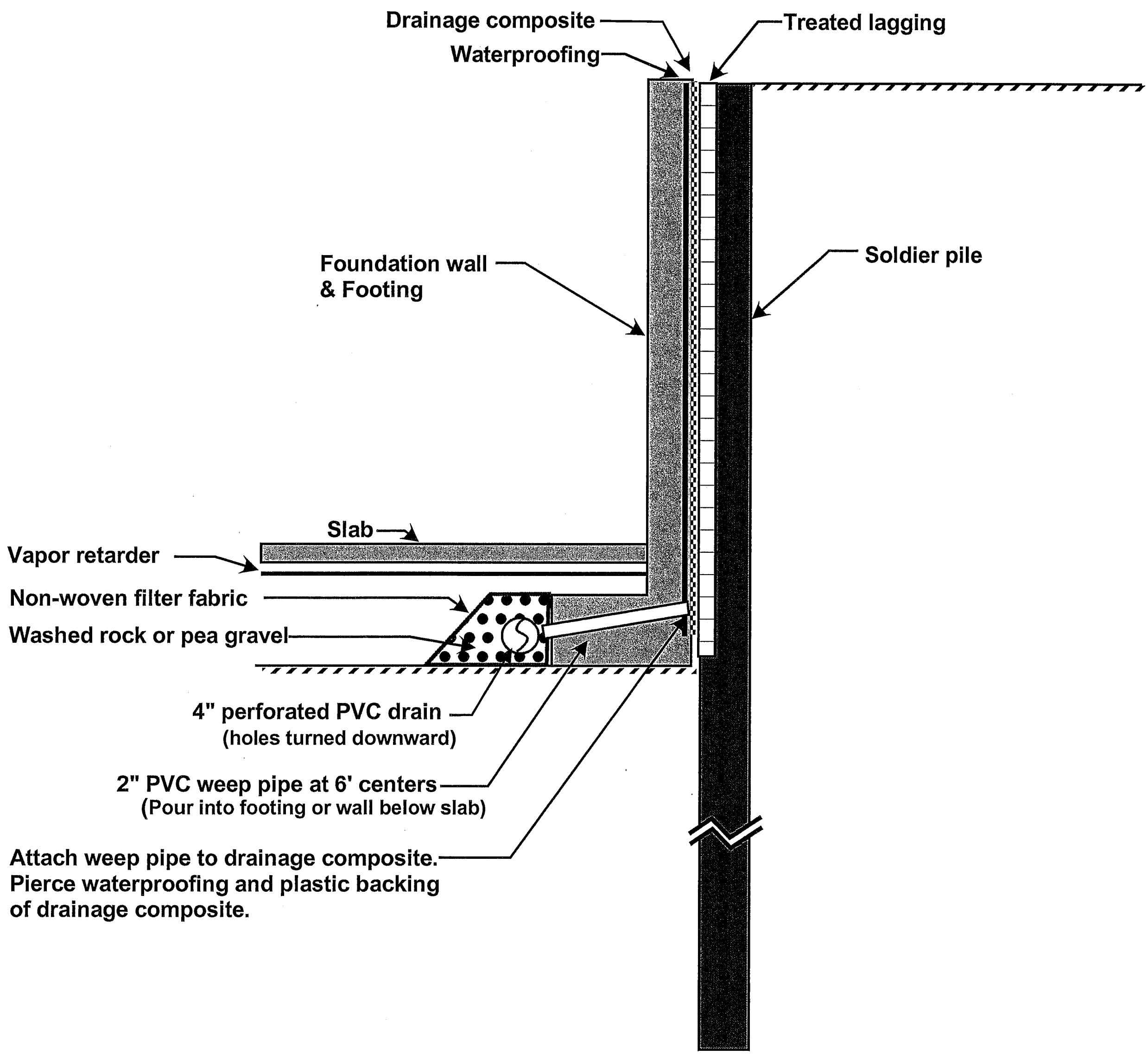
TYPICAL UNDERSLAB DRAINAGE

4803 Forest Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
20086	May 2020		8



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

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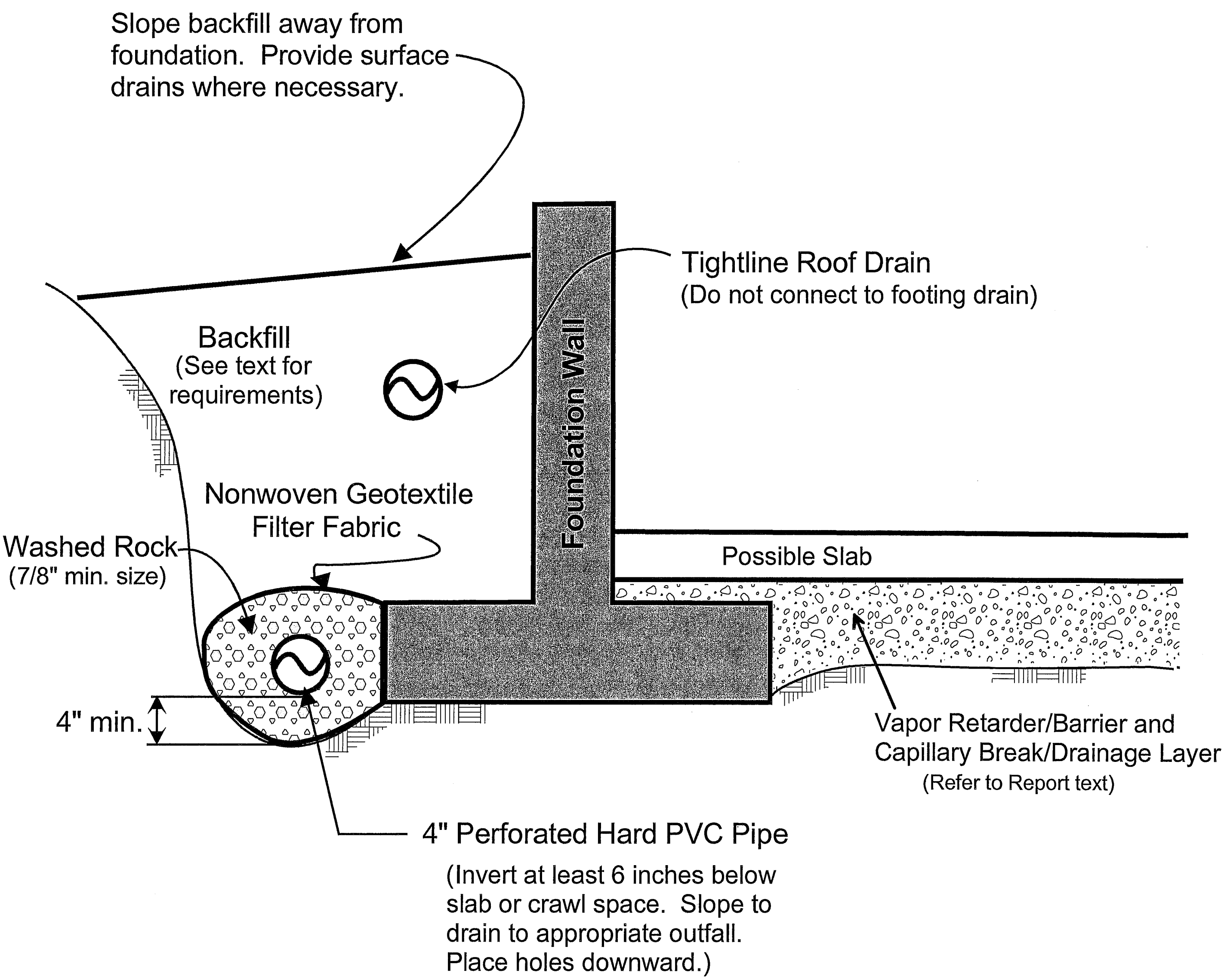
SHORING DRAIN DETAIL
4803 Forest Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
20086	May 2020		9



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

4803 Forest Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
20086	May 2020		10