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March 5, 2019

JN 18587

Harris Klein
5360 Lansdowne Lane
Mercer Island, Washington 98040
*via email: popnum@live.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Residence and Driveway
74XX Southeast 38th Street
Vacant Lot - Tax Parcel No. 362350-0226
Mercer Island, Washington

Dear Mr. Klein:

Attached to this transmittal letter is our geotechnical engineering report for the proposed residence and driveway to be constructed in Mercer Island, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork, stormwater infiltration considerations, and design considerations for foundations, retaining walls, subsurface drainage, and temporary excavations. This work was authorized by your acceptance of our proposal, P-10253, dated December 26, 2018.

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.

Adam S. Moyer
Geotechnical Engineer

cc: **Plan to Permit, LLC** – George Steirer
*via email: george@plantopermit.com*

ASM/MRM:kg

**GEOTECHNICAL ENGINEERING STUDY**
**Proposed Residence and Driveway**
**74XX Southeast 38th Street**
**Vacant Lot – Tax Parcel No. 362350-0226**
**Mercer Island, Washington**

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

Development of the property is in the planning stage, and detailed plans were not available to us at the time of this study. We were provided with a topographic map which was developed by Encompass Engineering & Surveying and is dated January 3, 2018. Based on conversations with George Steirer of Plan to Permit LLC, we understand that a single-family residence will be constructed near the center of the vacant lot. We anticipate that the residence may include a basement that daylights south-southwest.

The subject site is bordered by 74th Avenue Southeast to the west and Southeast 38th Street to the south; however, neither of these right-of-ways are improved. The closest developed street is 73rd Avenue Southeast, which is 280 feet west of the subject site. The portion of Southeast 38th Street between the subject site and 73rd Avenue Southeast contains a shared driveway on its southern half that is used by two residences southwest of the subject site. The ground surface on the northern half of the right-of-way rises steeply to the north approximately 10 to 14 feet to the yard areas of the adjacent properties to the north. We understand that development of the subject site will require improving the Southeast 38th Street right-of-way. We anticipate that this may require cutting into the toe of the short slope on the northern half of the right-of-way. The ground surface within the right-of-way begins to drop to the southeast near the subject site's southwest corner. Improvements to the right-of-way to provide access to the subject site will likely require permanently-retained structural fill along the downslope side of the new access road in this area.

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 Mercer Island. The undeveloped, rectangular-shaped subject site is located on the northeast corner of the Southeast 38th Street and 74th Avenue Southeast right-of-ways. The subject lot has dimensions of 80 feet in the north-south direction and 110 feet in the east-west direction. The undeveloped parcel is covered by underbrush and scattered mature evergreen and deciduous trees. The ground surface slopes on the site downwards from northwest to southeast with a total change in elevation of 38 feet across the site. The northwest half of the site is moderately sloped with an inclination of 20 to 27 percent. However, the grade steepens to an inclination of 44 to 60 percent across the southeast half of site with a drop in elevation of 18 to 24 feet to the southeast corner of the property.

As previously discussed, the adjacent Southeast 38th Street and 74th Avenue Southeast right-of-ways to the south and west are unimproved. The subject site is bordered to the north and east by a

3-acre parcel owned by the City of Mercer Island which is used as public open undeveloped space. The 74th Avenue Southeast right-of-way currently contains a walking trail from the public open space to the shared driveway within the Southeast 38th Street right-of-way southwest of the subject site.

From our review of the Mercer Island GIS, we can see that the site lies within a mapped potential landslide hazard, seismic hazard, and erosion hazard area. Additionally, a wedge of steep slope area is mapped in the southeastern portion of the lot.

We did not observe any indications of recent slope instability on, or around, the site during our visits to the property. On the Mercer Island Landslide Hazard Map (Troos and Wisher, 2009) a landslide is mapped as having occurred to the south of the site.

## *SUBSURFACE*

The subsurface conditions were explored by drilling two 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.

The test borings were drilled on January 30, 2018 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 2.5- and 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 and 4.

### ***Soil Conditions***

The test borings encountered medium-dense silt with sand beneath the ground surface. However, the silt became heavily fractured within a softer matrix at depths of 7.5 to 12.5 feet in both test borings. Massive, medium-dense to very dense silt was revealed beneath depths of 10 to 15 feet. The upper, disturbed, fractured layers of silt indicate a potential interface of prior soil movement above the denser underlying soils.

We did not observe any other indications of obvious landslide debris in the borings or signs or soils movement at the ground surface. The fracturing may be the result of past soil creep of the near-surface soils.

Test Boring 1 was terminated at a depth of 16.4 feet due to refusal in the very dense underlying silt. In Test Boring 2, the silt became slightly plastic and very stiff below 28 feet and extended to the maximum-explored depth of 31.5 feet.

Our firm completed two test pits on the subject site in 1997 which encountered slickenslides within the silt at depths of 10 to 12 feet. This is consistent with our recent test borings.

### ***Groundwater Conditions***

No groundwater seepage was observed in our subsurface explorations. 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 medium-dense silt beneath the ground surface. However, a layer of softer silt with fracturing and hard silt inclusions was encountered at depths of 7.5 to 12.5 feet below grade. Previous test pits we conducted in 1997 on the subject site encountered similar subsurface conditions with fracturing and slickenslides below 10 feet and extending to the maximum-explored depth of 12 feet. In our recent test borings, the silt became massive, and medium-dense to very dense below depths of 10 to 15 feet. Based on these subsurface explorations, it appears there may have been past movement or creep of the upper 7.5 to 12.5 feet of soil on the site. There were no indications in our test borings of soil movement extending into the underlying, massive silt, and no signs of recent soil movement were observed on the ground surface. The recommendations presented in this report are intended to prevent the planned development from increasing the risk of instability within, and outside of, the development area, and to protect the new residence from damage in the event of future shallow soil movement.

The loads from the proposed residence, including any basement or garage slabs, should be entirely supported on a deep foundation system embedded into the denser underlying silt soils. To accomplish this, we recommend the following:

1. A stabilization wall should be constructed along the downslope, southeast face of the proposed residence designed to retain the loose upper soils beneath the residence in the event of future movement of the upper site soils on the slope below. A stabilization wall is a below-grade retaining wall which consists of closely-spaced, drilled, heavily-reinforced concrete piles embedded into the medium-dense to very dense underlying silt. At a minimum, the stabilization wall should span the width of the residence. However, the wall could span the full width of the subject site to increase the stabilized area. The stabilization piles could also be designed to support the downslope, southeast edge of the residence. Alternatively, the stabilization wall could be constructed offset to the southeast from the residence to provide a larger stabilized yard area downslope of the new residence.

Expanded discussions regarding stabilization wall and driven pipe piles can be found in later sections of this report.

2. The remainder of the residence upslope of the stabilization wall can be supported on small-diameter pipe piles driven into the dense underlying soils. We recommend the residence be constructed with crawlspaces or with structural slabs spanning between pile-supported grade beams.

Another major geotechnical consideration for the project will be the excavation for the residence and driveway. We anticipate that the residence will include a daylight basement; thus, cuts up to 10 to 12 feet in height are anticipated along the north-northwestern side of the residence. Considering the fractured nature of the upper silt soils, we recommend any excavations deeper than 4 feet be supported with cantilevered soldier pile shoring. Rockeries could be used to support single, shallow cuts having a height of up to 4 feet. This will be necessary to maintain the stability of the sloping ground surface upslope of the residence until the permanent residence foundation walls can be constructed. This will also reduce the amount of excavation and soil removal from the site. The ***Temporary Cantilevered Soldier Pile Shoring*** section of this report presents additional temporary shoring recommendations.

The site soils that will be excavated have a low compacted strength and very poor drainage characteristics. We recommend against using the onsite soils for any wall backfill, or structural fill that will support on-grade elements, even the driveway or front entry walkways.

Given the low permeability of the site soils, and the potential for future instability within the near-surface soils, it is our professional opinion that subsurface infiltration or dispersion are not feasible methods for disposing of runoff from impervious areas.

As previously discussed, we understand that development of the subject site will require improvement of the Southeast 38$^{th}$ Street right-of-way to the west of the subject site. This will likely require a cut into the toe of the existing southern-facing slope that covers a large portion of the northern half of the right-of-way. Assuming the subsurface conditions are consistent with the borings conducted on the subject site, it is our opinion that permanent cuts 4 feet in height or less can be completed with the construction of a rockery. No tiered rockeries may be used. Temporary cuts should be sloped no steeper than 2.51 (Horizontal:Vertical). Any permanent cuts taller than 4 feet should be permanently supported with a drilled soldier pile retaining wall. Furthermore, the existing ground surface in the right-of-way drops to the southeast near the southwest corner of the subject site. A closely-spaced soldier pile stabilization wall may be necessary along the downslope side of the new street improvements where the grade is to be raised. We also anticipate that the design of the downslope retaining wall and the new pavement surface will likely need to be designed for fire truck access and loading. Considering this and the relatively low strength of the upper silt soils, we recommend the new street pavement section consist of minimum 8 inches of crushed rock base course underlying a minimum 6 inches of asphalt pavement surface course. The silt soils encountered in our explorations are very moisture sensitive; therefore, it will be very important that roadway subgrade earthwork preparation be completed during dry conditions. This will be very challenging to complete during the wetter winter and spring months.

The subject site is located within both a potential landslide hazard area and a seismic hazard area, each encompassing much of the general vicinity. The core of the subject site consists of medium-dense to very dense native silt that has a low potential for deep-seated landslides. The soils that will support the foundations are not liquefiable, due to their glacially-compressed nature. However, as discussed above, our subsurface explorations indicate the loose upper soils have previously experience movement/creep. The recommendations presented in the report are intended to prevent adverse impacts to the stability of the slope onsite and to prevent future soil movement beneath the proposed development during a large seismic event.

The subject site also meets the criteria for an erosion hazard area. We have been associated with numerous projects in involving excavating into steep slopes and landslide hazard areas. Proper erosion control implementation will be sufficient to prevent adverse impacts to the steeply sloped subject site. The temporary erosion control measures needed during the site development will depend heavily on the weather conditions that occur during the site work. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil areas to prevent accumulated water or runoff from the work area from becoming silty in the first place. A silt fence bedded in compost, not native soil or sand, should be erected as close as possible to the planned work area. The fence located immediately downslope of the stabilization pile wall will have to be wire backed and be struct between metal fence posts in the event that some of the drilling spoils come to rest against the fence. Rocked construction access and staging areas should be established wherever trucks will have to drive off of pavement in order to reduce the amount of soil or mud carried off the property. Covering the base of the excavation with a layer of clean crushed rock is also prude to reduce the amount of mud and silty water generated. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or imperious surfaces.

Wet weather construction on this site should be possible without adverse impacts to the surrounding properties. In preventing erosion control problems on any site, it is most important that any disturbed soil areas be immediately protected. This required 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 warning or input from the geotechnical engineer or representatives of the City.

In order to satisfy the City of 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.

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 the absence of near-surface groundwater.

## *PIPE 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. 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 250 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. 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 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.

### *STABILIZATION WALL*

As discussed in the ***General*** section, a retaining structure consisting of closely spaced, reinforced concrete piles is recommended along the downslope edge of the residence/development to retain the loose soils beneath the proposed residence. The piles should be spaced no further apart than 3 feet edge-to-edge so that the soil will arch between them. The piles would be constructed by setting steel H-beams or rebar cages in drilled holes and grouting the spaces between the steel reinforcements and the soil with concrete for the entire height of the hole. No groundwater was encountered in our test borings; however, we recommend that the shoring contractor be prepared to case the drilled shafts if loose caving soils are encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement of adjacent structures. If water is present in a hole at the time of construction, concrete must be tremied to the bottom of the hole. The contractor should be well prepared for this and have at least one casing and a tremie pipe of sufficient length prior to starting drilling.

The stabilization wall should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 45 pcf (assuming level backfill) for a total depth of 15 feet beginning at the existing ground surface on the downslope side of the proposed residence. An ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 350 pcf will resist the lateral movement of the piles below the 15-foot depth. For long term conditions, a safety factor of 1.5 should be applied to lateral design of this stabilization wall. Plate 6 depicts a typical stabilization wall detail.

***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 *<br>- Level Backfill<br>- 2.5:1 (H:V) Sloped Backfill | <br>45 pcf<br>55 pcf |
| Passive Earth Pressure | 250 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. 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. The onsite soils are not free-draining, and 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*

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.

## *TEMPORARY CANTILEVERED SOLDIER PILE SHORING*

Cantilevered soldier pile systems have proven to be an efficient and economical method for providing excavation shoring where the depth of excavation is less than approximately 15 feet.

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. Considering the groundwater encountered in our test borings, the shoring 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 and that has a level backslope should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 40 pounds per cubic foot (pcf). If the soldier piles will be used for permanent lateral support of the excavation, they should be designed for the active and seismic pressures recommend in ***Permanent Foundation and Retaining Walls***.

Slopes above the shoring walls will exert additional surcharge pressures. Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. We can review the initial shoring design to verify our preliminary surcharge considerations are still appropriate for the design layout.

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 350 pcf. 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." Plate 5 depicts a typical cantilevered soldier pile shoring detail.

## *EXCAVATION AND SHORING MONITORING*

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. The shoring walls should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least every 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 7 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 8. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

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.

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. As discussed in the ***General*** section, the on-site soils are not suitable for reuse as structural fill, due to high fines content and moisture-sensitivity.

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

This report has been prepared for the exclusive use of Harris Klein 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 - 4 | Test Boring Logs |
| Plate 5 | Cantilevered Soldier Pile Shoring |
| Plate 6 | Stabilization Wall Detail |
| Plate 7 | Typical Shoring Drain Detail |
| Plate 8 | 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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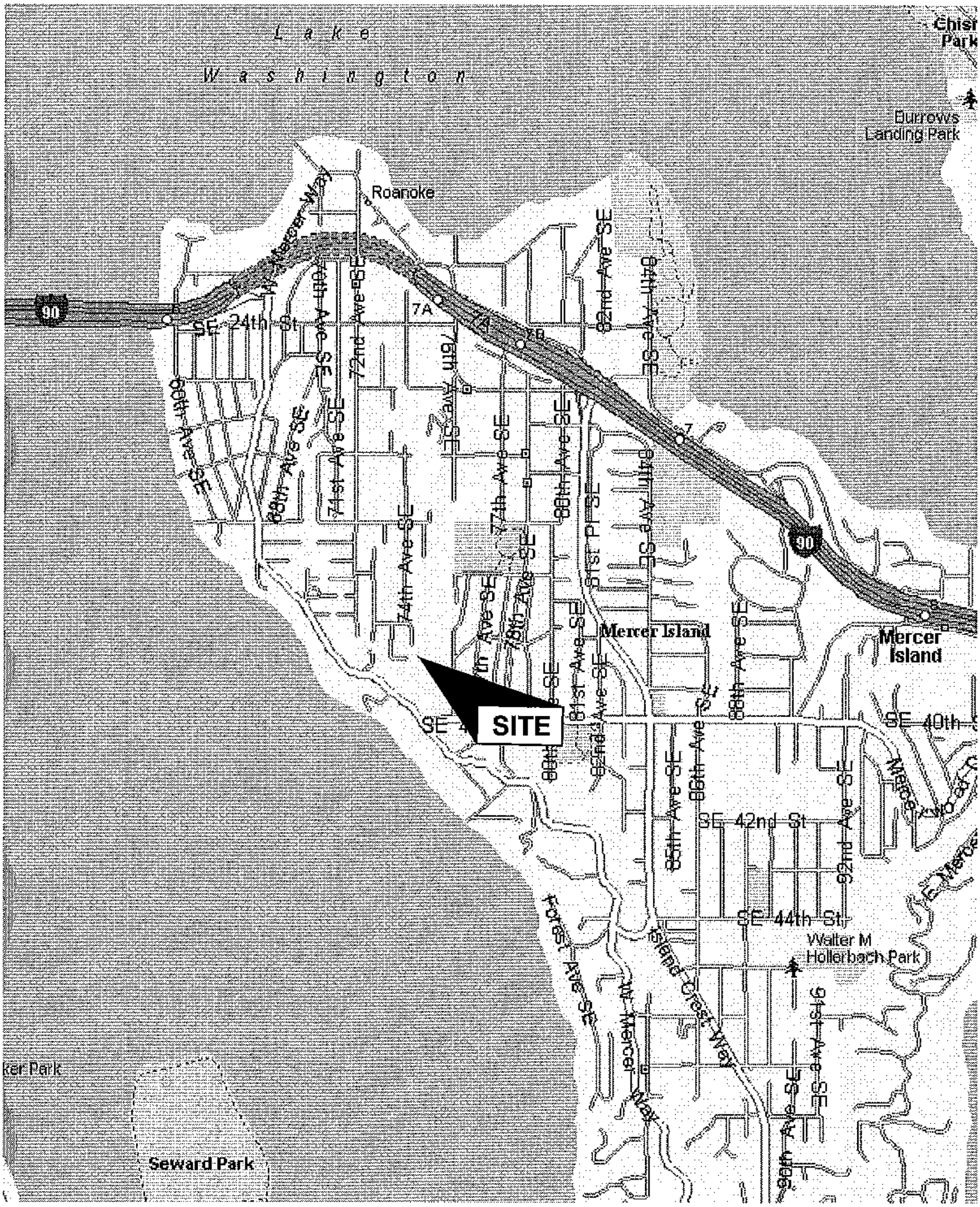
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER

3/5/19

Marc R. McGinnis, P.E.
Principal

ASM/MRM:kg

**NORTH**




(Source: Microsoft MapPoint, 2013)

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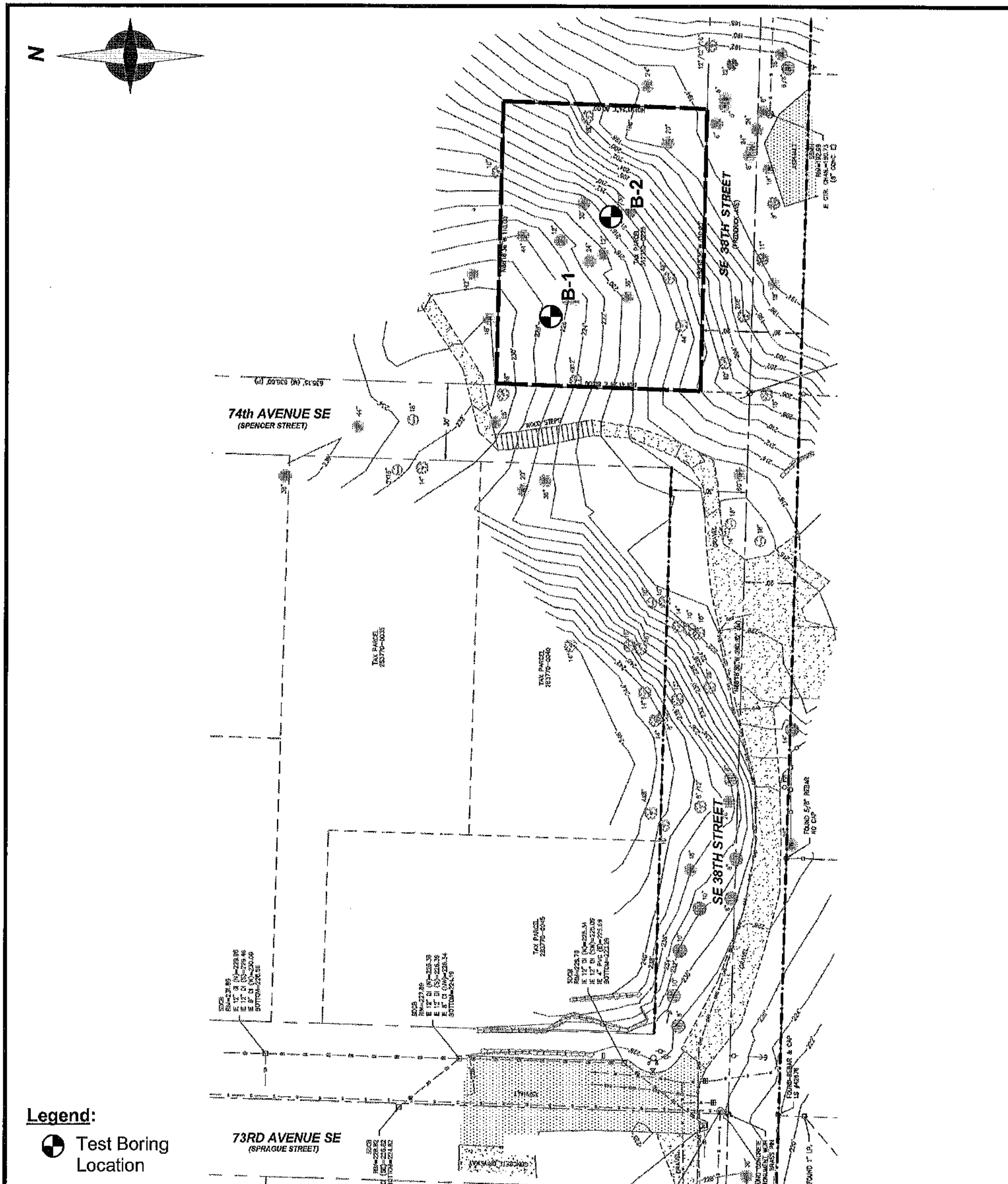
## VICINITY MAP

74XX Southeast 38th Street
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 18587 | Mar. 2019 | | 1 |

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# SITE EXPLORATION PLAN

74XX Southeast 38th Street
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 18587 | Mar. 2019 | No Scale | 2 |

# BORING 1

| Depth (ft.) | Moisture | Water Table | Blows per Foot | Sample | USCS | Description |
|---|---|---|---|---|---|---|
| | | | | | | Vines and brambles over: |
| | | | | | | Gray SILT, non-plastic, moderately fractured, blocky, with occasional dark-gray hard inclusions, moist, medium dense |
| | | | 12 | 1 | | -with very fine-grained sand and occasional rootlets |
| 5 | | | 17 | 2 | | |
| | | | 11 | 3 | ML | - with dark-gray, blocky, hard silt inclusions in softer matrix |
| 10 | | | 54 | 4 | | - becomes massive, very dense |
| 15 | | | 85/ 11" | 5 | | |
| 20 | | | | | | |
| 25 | | | | | | |
| 30 | | | | | | |
| 35 | | | | | | |

* **Test boring was terminated at 16.4 feet on January 30, 2019.**
* **No groundwater was encountered during drilling.**

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**TEST BORING LOG**
74XX Southeast 38th Street
Mercer Island, Washington

| *Job* | *Date:* | *Logged by:* | *Plate:* |
|---|---|---|---|
| 18587 | Mar. 2019 | ASM | 3 |

# BORING 2

| Depth (ft.) | Moisture | Water Table | Blows per Foot | Sample | USCS |
|---|---|---|---|---|---|
| | | | 18 | 1 | ML |
| 5 | | | 26 | 2 | |
| | | | 14 | 3 | |
| 10 | | | 14 | 4 | |
| | | | 38 | 5 | |
| 15 | | | 20 | 6 | |
| 20 | | | 27 | 7 | |
| 25 | | | 12 | 8 | |
| 30 | | | 21 | 9 | |
| 35 | | | | | |

**Description**

Brambles over:

Gray-brown sandy SILT with trace rootlets, non-plastic, very fine-grained, dry to moist, medium dense

- becomes moist

- with trace hard, blocky inclusions

- 8-inch zone with occasional hard, blocky, fractured, inclusions in a softer matrix

- becomes massive

- becomes very moist with rust mottling

- reduced sand content, becomes slightly plastic, very stiff

* **Test boring was terminated at 31.5 feet on January 30, 2019.**

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

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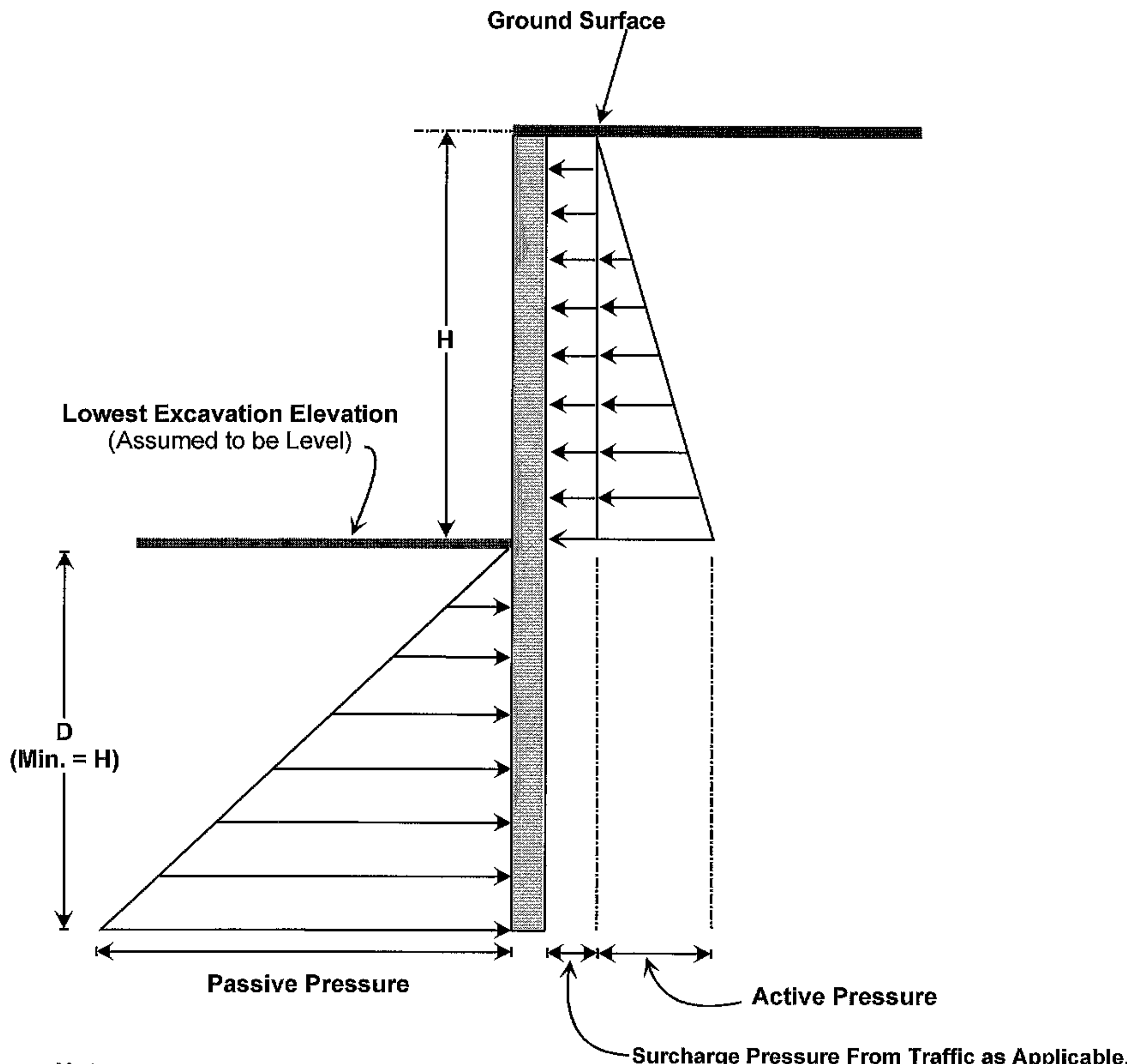
## TEST BORING LOG

74XX Southeast 38th Street
Mercer Island, Washington

| Job | Date: | Logged by: | Plate: |
|---|---|---|---|
| 18587 | Mar. 2019 | ASM | 4 |




**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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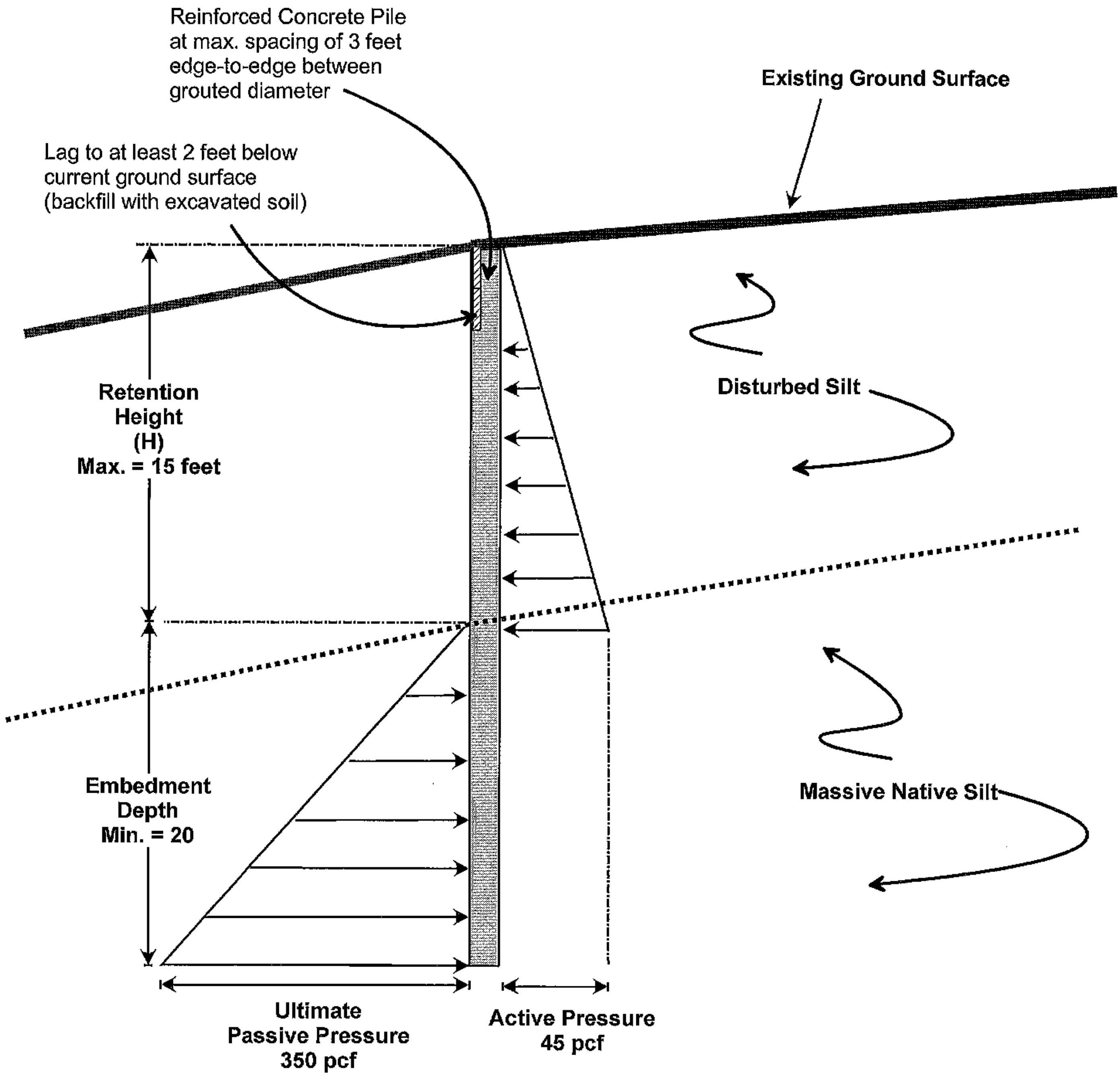
**CANTILEVERED SOLDIER PILE SHORING**

74XX Southeast 38th Street
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 18587 | Mar. 2019 | | 5 |




**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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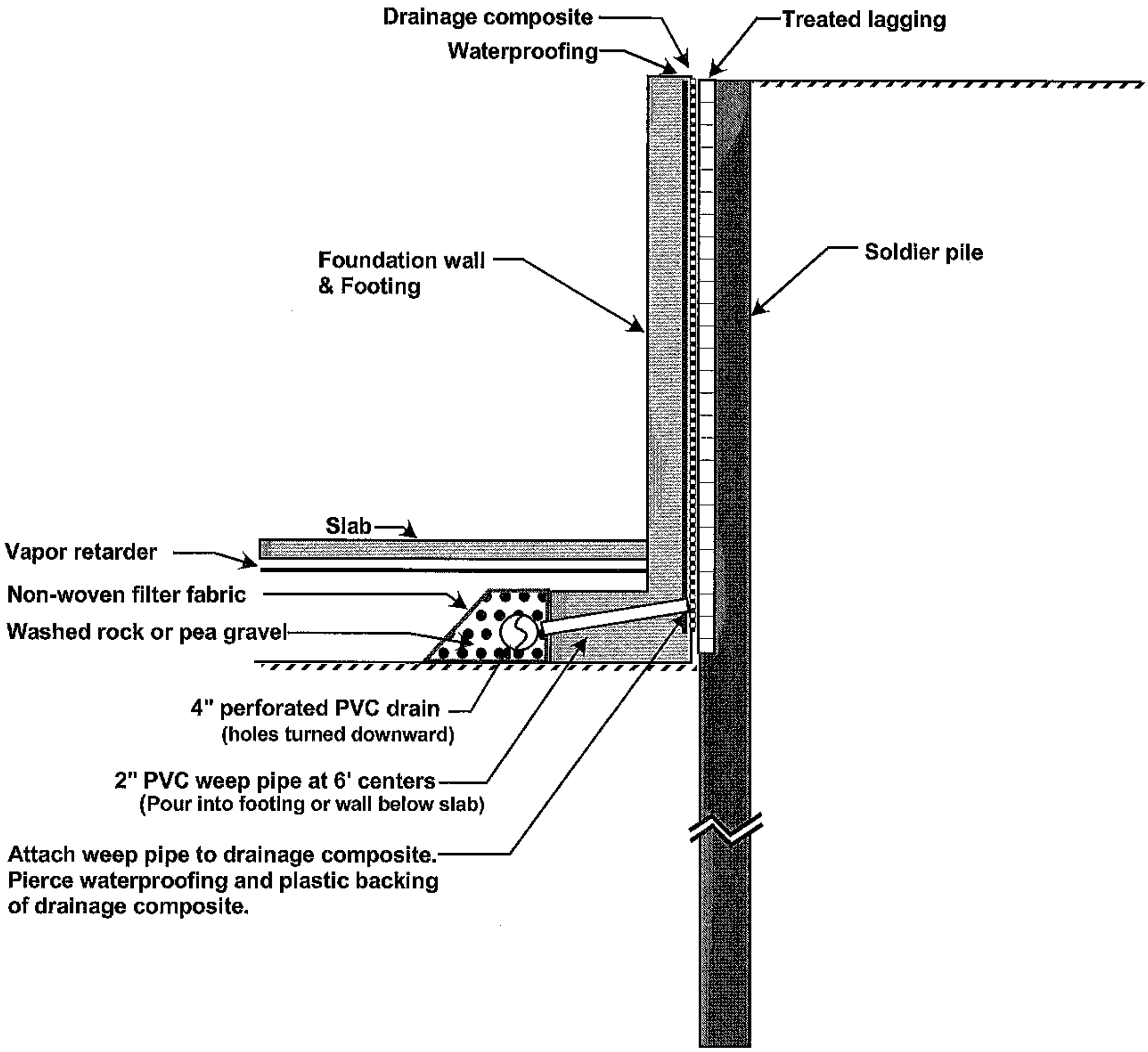
# STABILIZATION WALL DETAIL

74XX Southeast 38th Street
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 18587 | Mar. 2019 | | 6 |




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

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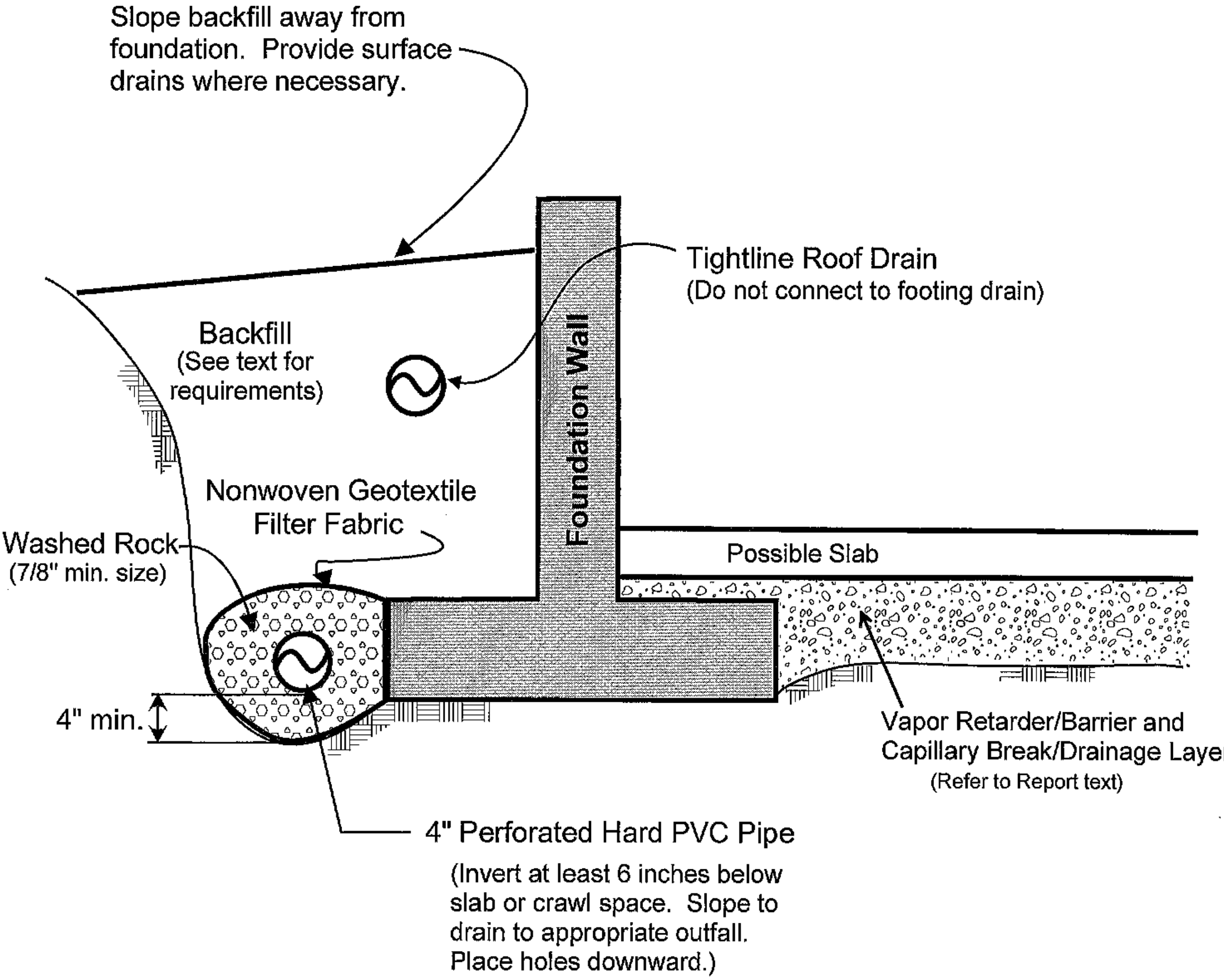
# FOUNDATION DRAIN DETAIL

74XX Southeast 38th Street
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 18587 | Mar. 2019 | | 7 |




**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**

74XX Southeast 38th Street
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 18587 | Mar. 2019 | | 8 |