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JN 19011

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via email: janerosenstein@hotmail.com

Subject: **Transmittal Letter – Geotechnical Engineering Study and Critical Area Study**
Proposed New Residence
2035 – 81st Avenue Southeast
Mercer Island, Washington

Greetings,

Attached to this transmittal letter is our geotechnical engineering report for your new 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 and design considerations for foundations, retaining walls, subsurface drainage, slope stability, and temporary excavations and shoring. This work was authorized by your acceptance of our proposal, P-10260, dated December 31, 2018.

This report is an update of our previous August 6, 2019 report and 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.

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

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GEOTECHNICAL ENGINEERING STUDY AND CRITICAL AREA STUDY
Proposed New Residence
2035 – 81st 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 new residence to be located in Mercer Island.

Development of the property is in the planning stage, and detailed plans were not available at the time of this study. The preliminary architectural sketches provided to us by Brandt Design Group indicate that the existing home is to be demolished, and a new residence with a larger footprint is planned. The new residence will be irregular shaped and will consist of three parts. The main, central portion of the residence will be located in the rough middle of the site, and will be flanked by two "wings" that are rotated slightly off center from the main area to run mostly parallel to the north and south property lines. The new residence is shown to be one-story in height, with a basement daylighting to the west extending underneath the central portion of the residence and the western side of the southern wing. An attached, two-car garage is shown on the northern side of the residence and will be accessed from the private drive that currently serves the property. A large, elevated deck is shown to extend from the western side of the main level of the residence and a smaller deck is shown to extend off the southwestern side of the lower level and a covered patio is shown to the west of the basement. Based on the preliminary drawings, excavation depths of 10 feet are anticipated to reach the basement level.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site in the Beaumont neighborhood of Mercer Island. The parallelogram-shaped site has approximate dimensions of 108 to 136 feet in the north-south direction and 139 to 171 feet in the east-west direction. The subject site is bounded to the north, south, and west by other single family residences of similar construction, and to the east by 81st Avenue Southeast. The adjacent northern and southern residences are shown to be relatively close to the shared property lines. The adjacent southern residence is shown to contain a basement and the house to the north does not appear to contain a basement.

The site is currently developed with an older, one story house that occupies much of the upper bench of the site. The residence is underlain by a basement that daylights to the west and appears to encompass most of the main level footprint of the house, save for the covered carport to the north of the house. A large, elevated deck extends to the west of the main level of the house, out onto the western slope. A small patio and walkway extend from the basement level and stairs provide access from the deck down to the basement grade. A 4 to 8-foot tall rockery lines the eastern side of the site, covering the grade drop from the private driveway down to the entryway to the house. Several small railroad tie walls retain small portions of areas surrounding the house. The remainder of the lot is undeveloped, with scattered landscaping beds located on the east side of the site. The remaining western half of the lot is covered with underbrush and several mature trees.

The grade across the site drops moderately to steeply downward from east to west, with a total elevation change of 45 feet across the site. Much of this grade change occurs both to the east and west of the house footprint. The eastern side of the property has been excavated into the original ground surface for the driveway and front entry, creating steep manmade slopes that are generally less than 10 feet in height. There are rockeries protecting all, or portions, of these oversteepended cuts. The grade continues to drop moderately across the house footprint approximately 10 feet to the level of the daylight basement. Past the west edge of the house, the grade is inclined much steeper, with average inclinations in excess of 40 to 60 percent over an elevation change of approximately 18 to 24 feet. The upper portion of this slope has likely been oversteepened by grading associated with various landscape features when the house was originally constructed. Below this steel slope, the grade then flattens out to the western property line before sloping downward to the neighboring lot.

Research conducted on the City of Mercer Island GIS Mapping Portal indicates that the subject site is mapped within a Potential Landslide Hazard Area. The site is also mapped within an Erosion Hazard Area and Seismic Hazard Area. The slope to the west of the house would generally be classified as a steep slope and much of the site meets the criteria for an erosion hazard. The *Mercer Island Landslide Hazard Assessment* map (Troost & Wisher, 2009) indicates that a documented landslide may have previously occurred in the vicinity of the site. We did not observe any indications of recent instability on the site during our field work.

SUBSURFACE

The subsurface conditions were explored by drilling 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 borings were drilled on February 14, 2019 using a portable Acker drill. This drill system utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at approximate 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 through 5.

Soil Conditions

Borings 1 and 2 were drilled on the lower, western side of the existing residence. Beneath the ground surface, both borings encountered 6 to 6.5 feet of loose fill soil consisting of jumbled silt and silty sand with organics. Beneath the fill, native silt was encountered that was initially loose, becoming medium-dense at a depth of 7.5 feet in Boring 2. The silt was underlain by medium-dense silty sand and silt at depths of 9 to 12.5 feet that became dense at 17.5 feet in Boring 2 and very dense at 12.5 feet in Boring 1. The western borings were terminated at depths of 14 to 19 feet where auger refusal was met in the dense to very dense soils.

Boring 3 was conducted to the east of the existing house near the toe of the cut rockery. Native, medium-dense silt was encountered beneath the pavers, extending to a depth of 7 feet. Dense silty sand to sandy silt was encountered beneath the silt, extending to the base of the boring fat a depth of 9 feet where auger refusal was met.

Similar glacially-compressed silty and silty sands have been encountered in other borings completed on lots upslope (east) and downslope (northwest) of the subject site.

No obstructions were revealed by our explorations. However, debris, buried utilities, and old foundation and slab elements are commonly encountered on sites that have had previous development.

Although our explorations did not encounter cobbles or boulders, they are often found in soils that have been deposited by glaciers or fast-moving water.

Groundwater Conditions

Slight perched groundwater seepage was observed at a depth of 18 to 18.5 feet in Boring 2 within a sand seam. The borings were left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself.

It should be noted that groundwater levels vary seasonally with rainfall and other factors. We anticipate that groundwater could be found in more permeable soil layers and between the looser near-surface soil and the underlying denser soil.

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

CRITICAL AREA 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. This is noted on the attached Site Exploration Plan.

Both geologic hazard areas cover much of the general vicinity to the north and south as well. The core of the subject site consists of dense to very dense, glacially compressed, native soil that has a low potential for deep-seated landslides. However, this competent soil is overlain by looser soils that could experience slope movement, particularly during a large earthquake. The recommendations presented in our report are intended to stabilize the development area in the event of foreseeable slope movement, thereby mitigating the landslide hazard risk.

The foundations for the new construction will be supported on dense, non-liquefiable soils, mitigating the seismic hazard.

Steep Slope Hazard Areas: Based on the provided topographic map of the subject site, and our site observations, the slope immediately west of the existing house is over 10 feet in height, and exceeds an inclination of 40 percent. This slope would meet the definition of a steep slope under the MICC. The approximate top and toe of the steep slope is indicated on the attached Site Exploration Plan. It is our opinion that no buffers or setbacks are needed from this Steep Slope, provided the recommendations presented in this report are followed. The recommendations presented in the report are intended to prevent adverse impacts to the stability of the steep slope, and to protect the planned development from foreseeable future soil movement on the steep slope.

Erosion Hazard Area: The site also meets the City of Mercer Island's criteria for an Erosion Hazard Area. This has also been indicated on the attached Site Exploration Plan.

Excavation and construction of the planned residence can be accomplished without adverse to the site 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. Proper erosion control implementation will be important to prevent adverse impacts to the site and neighboring properties. 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 off the site, so a temporary holding tank should be planned for wet weather earthwork. A wire-backed silt 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. 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. 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. 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: The attached Site Exploration Plan (Plate 2) denotes the extents of the critical areas that cover the site. 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. The prescriptive Steep Slope buffers from the top and toe of the western steep slope are indicated on the Plan. 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 and their applicable prescriptive buffers. 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 recommendations presented in this report for the planned alteration will render the development as safe as if it were not located in a geologically hazardous area, and will not adversely impact critical areas on adjacent properties.*

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 borings conducted for this study encountered loose fill and native silt beneath the site. Suitable dense to very dense bearing soils were not encountered until depths of 12.5 and 15 feet on the western, low side of the proposed residence and 7.5 feet on the eastern, upslope side of the proposed residence. Any new foundations constructed on the unconsolidated fill and loose, native soils would result in excessive, differential post-construction settlement. All new foundation loads need to bear on, or into, the dense soil. Due to the depth of the loose and medium-dense soils (12.5 to 17.5 feet), the excavations needed to reach competent bearing soil impossible to achieve. Therefore, we believe that it is most appropriate to support the residence on deep foundations that are embedded into the dense soil. This includes any building floors or settlement-sensitive elements, such as entryways or stairways. For most of the residence, the deep foundation system could consist of small diameter pipe piles. An expanded discussion can be found in the ***Pipe Piles*** section of this report.

Due to the loose condition of the soils at the top of the steep slope to the west of the residence, a stabilization wall will likely be needed along the western, downslope side of the residence to retain the upper loose soils and stabilize the development area. The stabilization wall should consist of closely-spaced, heavily reinforced, drilled concrete piles that will embed significantly into the underlying, dense and very dense soil. Further discussion regarding the stabilization wall can be found in the ***Stabilization Wall*** section of this report. The western foundation wall of the house could be supported on the drilled piles. However, the piles should be located along the western edge of the area that is to be permanently stabilized against the potential for future slope instability.

As previously discussed, the subject site is located within a potential landslide hazard area that encompasses much of the general vicinity. The core of the subject site consists of dense native soil that has a low potential for deep-seated landslides. However, any slope in the Puget Sound area has some potential for shallow soil movement in the near-surface soils, particularly after extended periods of concentrated precipitation. The potential for failures of the onsite steep slope will be mitigated by proper retention of the loose soils within the development area. As discussed above in the ***Critical Area Study*** section, the recommendations presented in this report are intended to prevent adverse impacts to the stability of the slope onsite, protect the planned development from damage in the event of future instability, and prevent the development from adversely affecting the stability of surrounding properties.

As previously discussed, the new residence will contain a basement constructed into the slope. We can assume that excavations of approximately 10 feet will be needed to reach the basement level. Due to the loose conditions of the upper fill and native soil, temporary excavations with inclinations of no steeper than 1.5:1 (Horizontal:Vertical) are appropriate. If temporary excavations of this inclination cannot be kept within the property, or excavation easements are not able to be obtained from the adjacent property owners, temporary shoring will be needed. Based on the preliminary plan, we anticipate that shoring will be needed along the south side and southeastern half of the excavation. Driven soldier piles can cause excessive ground vibrations in the loose upper soils and should not be utilized for this project. Additional recommendations are presented in the ***Temporary Shoring*** section of this report.

The new basement will be excavated in close proximity to the base of the tall rockery on the eastern side of the site. the soils encountered in our boring near the base of the rockery indicated that approximately 7.5 feet of loose soil was present beneath the toe of the rockery. We recommend that no unshored excavations be made within a 3:1 (Horizontal:Vertical) inclination of the base of the rockery. These recommendations are presented in order to prevent the existing, undocumented rockery from becoming less stable and undermining the private driveway above.

The basement for the new residence will be excavated into soil with a low permeability. We recommend installing an underslab drainage system beneath the basement slab of the new residence. this system would consist of a layer of clean crushed rock beneath the interior slab or crawlspace. The rock layer should be at least 12 inches thick and contain 4-inch diameter, perforated PVC pipes at no more than 15-foot center-to-center spacings. The entire rock layer and pipe system should be covered with a thick vapor retarder/barrier. The perforated pipes should tie into the exterior footing drains. The ***Drainage Considerations*** section of this report contains an expanded discussion of our subsurface drainage recommendations.

The adjacent houses are likely supported on conventional foundations that bear on compressible soils. As a result, it is likely that they have undergone excessive settlement already. There is always some risk associated with demolition and foundation construction near structures such as this. It is imperative that unshored excavations do not extend below a 2:1 (Horizontal:Vertical) imaginary bearing zone sloping downward from existing footings. Contractors working on the demolition and construction of your home must be cautioned to avoid strong ground vibrations, which could cause additional settlement in the neighboring foundations. During demolition, strong pounding on the ground with the excavator, which is often used to break up debris and concrete, should not occur. Large equipment and vibratory compactors should not be used close to the property lines. Additionally, in order to protect yourselves from unsubstantiated damage claims from the adjacent owners, 1) the existing condition of the foundation should be documented before starting demolition, and 2) the footings should be monitored for vertical movement during the demolition, excavation, and construction process. These are common recommendations for projects located close to existing structures that may bear on loose soil and have already experienced excessive settlement. We can provide additional recommendations for documentation and monitoring of the adjacent structures, if desired.

No soil generated from the project excavation or new structural fill should be placed on, or near the steep slope, as the surcharge from the additional soils could reduce the stability of the slope. No water should be directed towards the steep slope along the western side of the development. Poorly managed stormwater runoff is a common cause of slope instability that is well documented in the Puget Sound area. Due to the silty, fine-grained nature of the upper fill and native soils onsite and the steep inclination of the slope to the west of the proposed residence, it is our professional opinion that onsite infiltration of stormwater is not feasible for this project. All collected stormwater should be discharged to an approved stormwater system.

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

The drainage and 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 1.36g and 0.52g, 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 0.56g. The dense soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature.

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.

STABILIZATION WALL

As discussed in the ***General*** section, a retaining structure consisting of closely spaced, reinforced concrete piles is needed along the downslope, western edge of the residence to provide complete stabilization. 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. We anticipate that the upper portions of the pile shafts will require casing due to the presence of loose, wet soil. 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 for a total depth of 15 feet beginning at the existing ground surface on the downslope side of the development area. An ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 400 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.

Typical design considerations for a stabilization wall are depicted on Plate 9.

If the drilled piles for the stabilization wall are used to support the downslope side of the structure or deck, the vertical capacity of the piles will be developed by skin friction between the concrete and the dense to very dense soils found in Borings 1 and 2. An allowable skin friction of 900 pounds per square foot can be assumed for this embedded portion of the piles.

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 <u>level</u> backfill:

PARAMETER	VALUE
Active Earth Pressure *	40 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.

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 site soils are fine-grained and have a high silt content. As a result, they are not free-draining. We recommend that the native soils not be reused as retaining walls 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.

BUILDING FLOORS

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

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

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

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

EXCAVATIONS AND SLOPES

Temporary excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Also, temporary cuts should be planned to provide a minimum 2 to 3 feet of space for construction of foundations, walls, and drainage. Temporary cuts to a maximum overall depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. 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.

The above-recommended temporary slope inclination is based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger. These recommendations may need to be modified if the area near the potential cuts has been disturbed in the past by utility installation, or if settlement-sensitive utilities are located nearby.

All permanent cuts into native soil should be inclined no steeper than 2.5:1 (H:V). Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

Any disturbance to the existing slope outside of the building limits may reduce the stability of the slope. Damage to the existing vegetation and ground should be minimized, and any disturbed areas should be revegetated as soon as possible. Soil from the excavation should not be placed on the slope, and this may require the off-site disposal of any surplus soil.

TEMPORARY SHORING

As discussed in the ***General*** section, the excavation for the proposed basement will likely require temporary shoring. Cantilevered soldier pile systems have proven to be an efficient method for providing excavation shoring. The shoring design should be submitted to Geotech Consultants, Inc. for review prior to beginning site excavation. We are available and would be pleased to assist in this design effort. A safety factor of either 1.2 or 1.5 should be included in the design of the shoring depending on whether the shoring is temporary or permanent.

Soldier Pile Installation

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. 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, and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 35 pounds per cubic foot (pcf). Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Existing adjacent buildings or structures such as the adjacent rockery and driveway will exert surcharges on the proposed shoring wall, unless the buildings are underpinned. Slopes above the shoring walls will exert additional surcharge pressures. These surcharge pressures will vary, depending on the configuration of the cut slope and shoring wall. We can provide recommendations regarding slope and building surcharge pressures when the preliminary shoring design is completed.

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

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 350 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. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

EXCAVATION AND SHORING MONITORING

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

Additionally, the shoring walls and any adjacent foundations should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least one third of the installed soldier piles 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

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

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. A typical shoring drainage detail is attached to this report as Plate 8.

Footing drains placed inside 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).

Drainage inside the building's footprint should also be provided where (1) a crawl space or slab will slope or be lower than the surrounding ground surface, (2) an excavation encounters significant seepage, or (3) an excavation for a building will be close to the expected high groundwater elevations. We can provide recommendations for interior drains, should they become necessary, during excavation and foundation construction. A typical underslab drainage detail is attached to this report as Plate 7.

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

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

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

GENERAL EARTHWORK AND STRUCTURAL FILL

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

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

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches, but should be thinner if small, hand-operated compactors are used. We recommend testing structural fill as it is placed. If the fill is not sufficiently compacted, it should be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction.

The following table presents recommended levels of relative compaction for compacted fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

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

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the 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 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 residence from damage due to slope movement. Predicting the future behavior of steep slopes and the potential effects of development on their stability is an inexact and imperfect science that is currently based mostly on the past behavior of slopes with similar characteristics. Landslides and soil movement can occur on steep slopes before, during, or after the development of property. The owner of any property containing, or located close to steep slopes must ultimately accept the possibility that some slope movement could occur, resulting in possible loss of ground or damage to the facilities around the proposed residence.

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

ADDITIONAL SERVICES

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

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

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 5	Boring Logs
Plate 6	Typical Footing Drain Detail
Plate 7	Typical Underslab Drainage Detail
Plate 8	Typical Shoring Drain Detail
Plate 9	Typical Stabilization Wall Detail

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

Respectfully submitted,

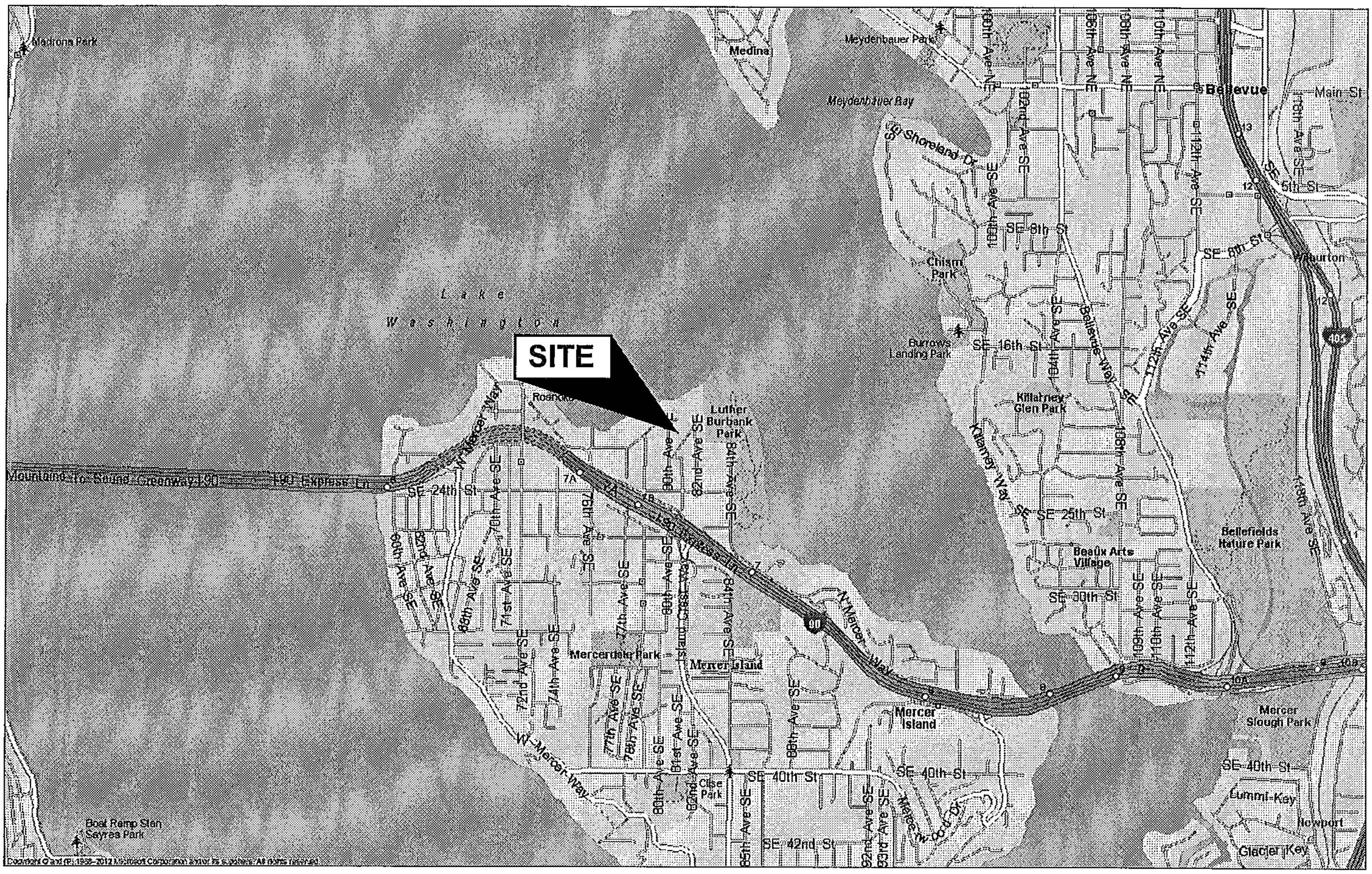
GEOTECH CONSULTANTS, INC.

10/29/19

Marc R. McGinnis, P.E.
Principal

MKM/MRM:kg

NORTH



(Source: Microsoft MapPoint, 2013)

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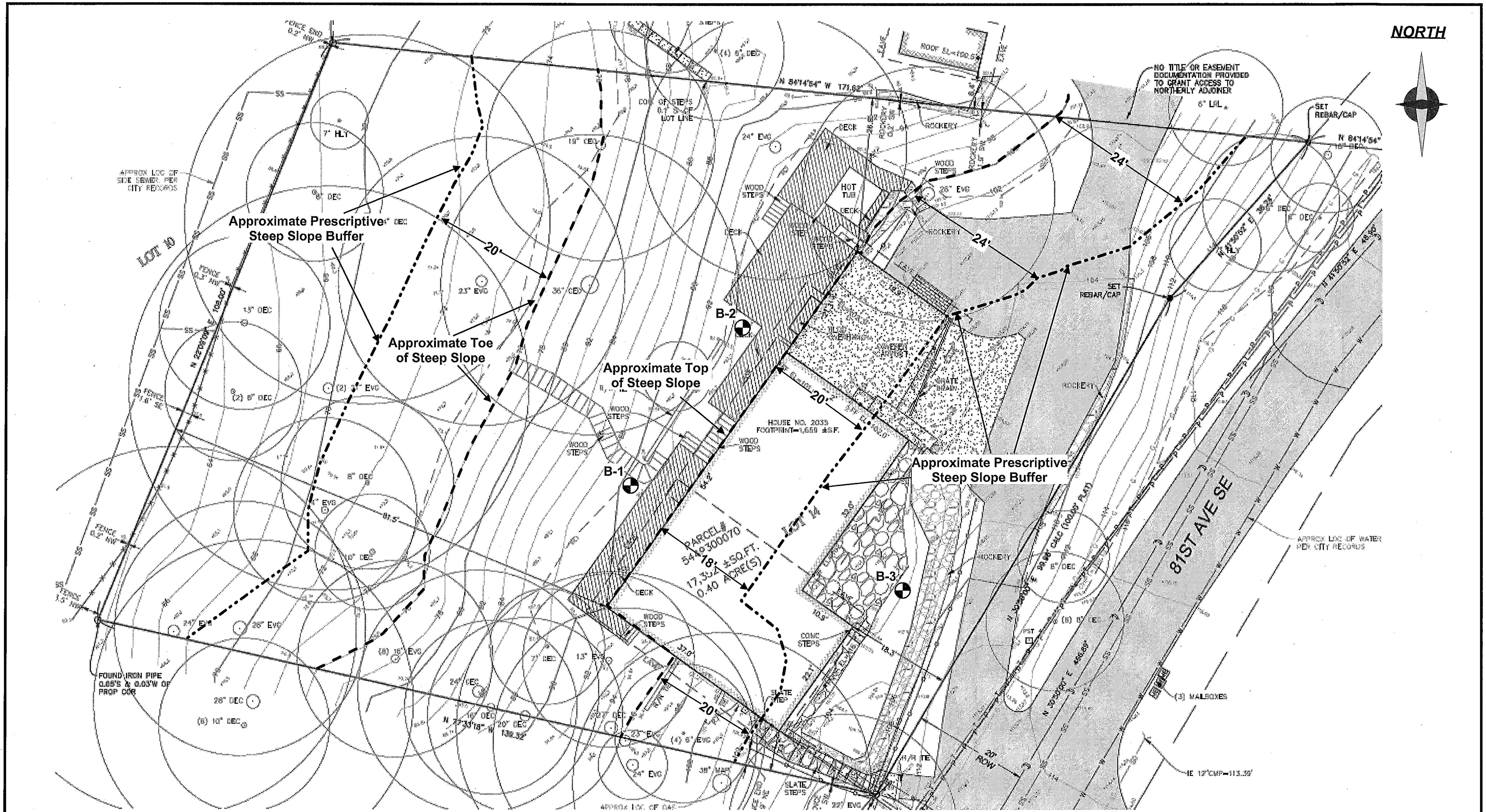
VICINITY MAP

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
19011	Mar. 2019		1



* The City of Mercer Island's GIS tool maps the subject site as a Seismic Hazard Area, Potential Landslide Hazard Area, and an Erosion Hazard Area in its entirety. The prescriptive buffers for shallow Potential Landslide Hazard Areas under MICC 19.07 extend beyond the propert boundaries

Legend:

Test boring location

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SITE EXPLORATION PLAN

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
19011	Oct. 2019		2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			6	1	FILL	Brown slightly gravelly, silty SAND with decayed organics, fine-grained, very moist to wet, loose **(FILL)**
5			8	2		-with a small piece of concrete
					ML	Brown mottled orange and blue, SILT with trace organics, non-plastic, moist, loose
			8	3		-becomes gray-brown mottled orange, slightly gravelly
10			10	4	SM / ML	Gray-brown mottled orange, slightly gravelly, silty SAND to sandy SILT, fine-grained, very moist, medium-dense
			81 / 11"	5		-becomes very dense
15						
20						

*** Test boring was terminated at 14 feet on February 14, 2019 due to auger refusal.**

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

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TEST BORING LOG

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
19011	Mar. 2019	MKM	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			5	1	FILL	Gray-brown SILT with organics, non-plastic, moist, jumbled, loose **(FILL)**
5			4	2		-with abundant roots and organics
					ML	Gray-brown mottled orange, slightly sandy SILT with roots, non-plastic, moist, fractured, loose
			12	3		-becomes brown and gray-brown with organics, medium-dense
10			12	4		-becomes brown mottled orange, moist to very moist
			20	5	SM/ML	Gray-brown mottled orange, slightly gravelly, silty SAND and SILT, fine-grained, moist, medium-dense
15			25	6		
		▼	40	7		-becomes gray, dense -becomes very fine-grained, wet -becomes moist
20						

* **Test boring was terminated at 19 feet on February 14, 2019 due to auger refusal.**

* **Slight perched groundwater was encountered from 18-18.5 feet during drilling.**

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TEST BORING LOG

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
19011	Mar. 2019	MKM	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Flagstone paver over;
			12	1	ML	Gray-brown mottled orange, slightly gravelly, sandy SILT, slightly plastic, moist, medium-stiff
5			11	2		-becomes brown mottled orange with thin gray streaks and organics, non-plastic, medium-dense
			48	3	SM ML	Gray and gray-brown, slightly gravelly, very silty SAND and sandy SILT with thin clean sand seams, fine-grained, non-plastic, cemented, dense to very dense
10						
15						
20						

*** Test boring was terminated at 9 feet on February 14, 2019 due to auger refusal.**

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

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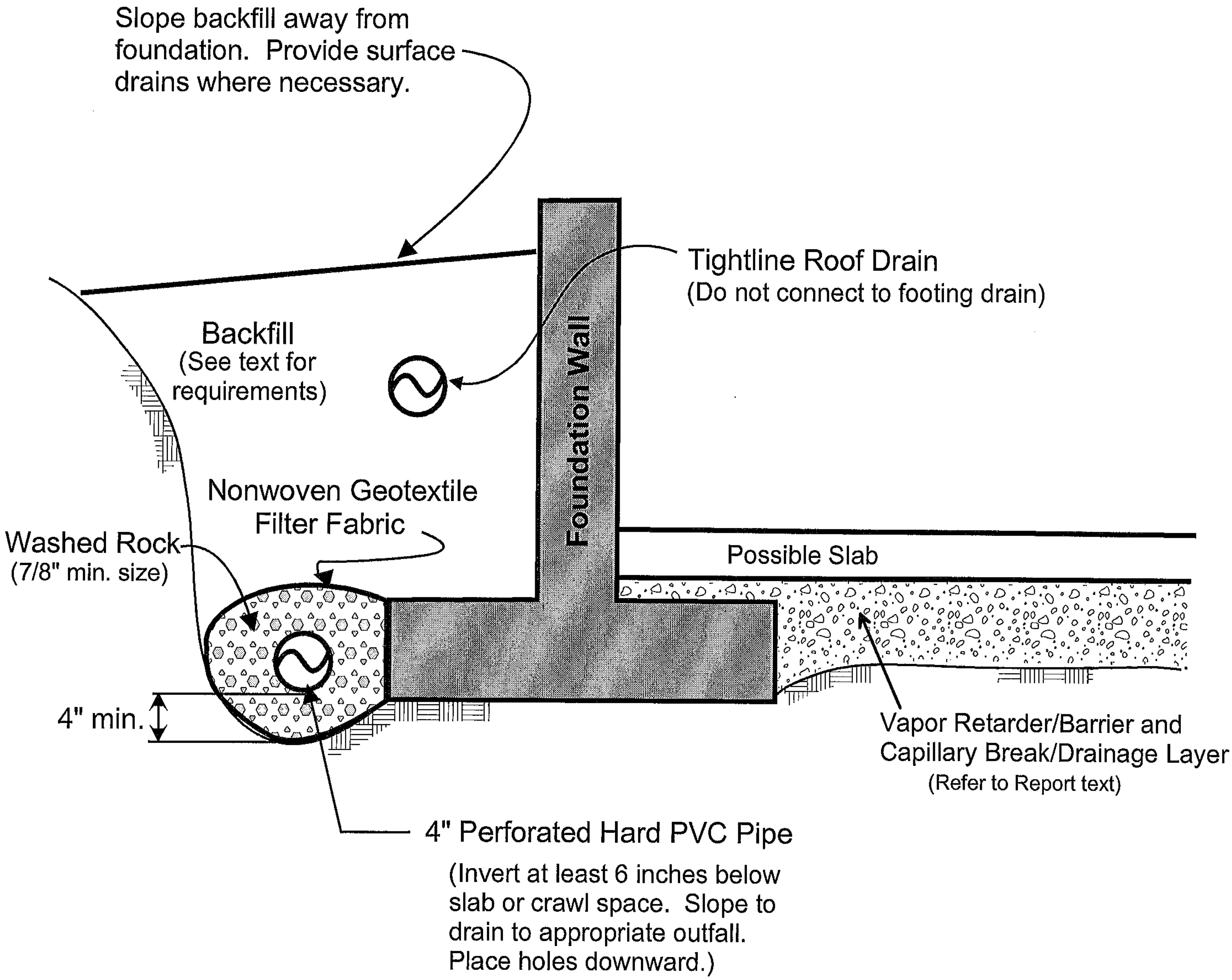
TEST BORING LOG
2035 - 81st Avenue Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
19011	Mar. 2019	MKM	5



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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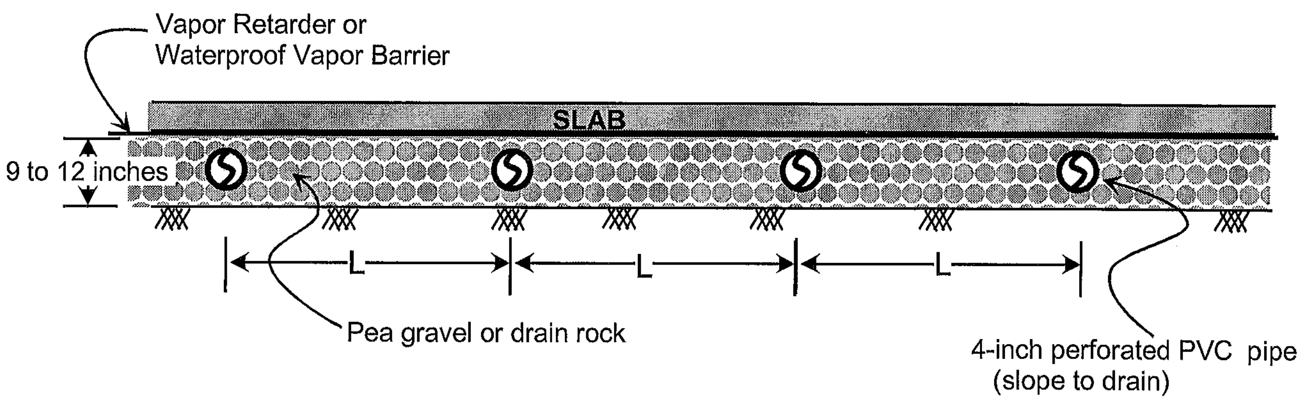
FOOTING DRAIN DETAIL

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
19011	Mar. 2019		6



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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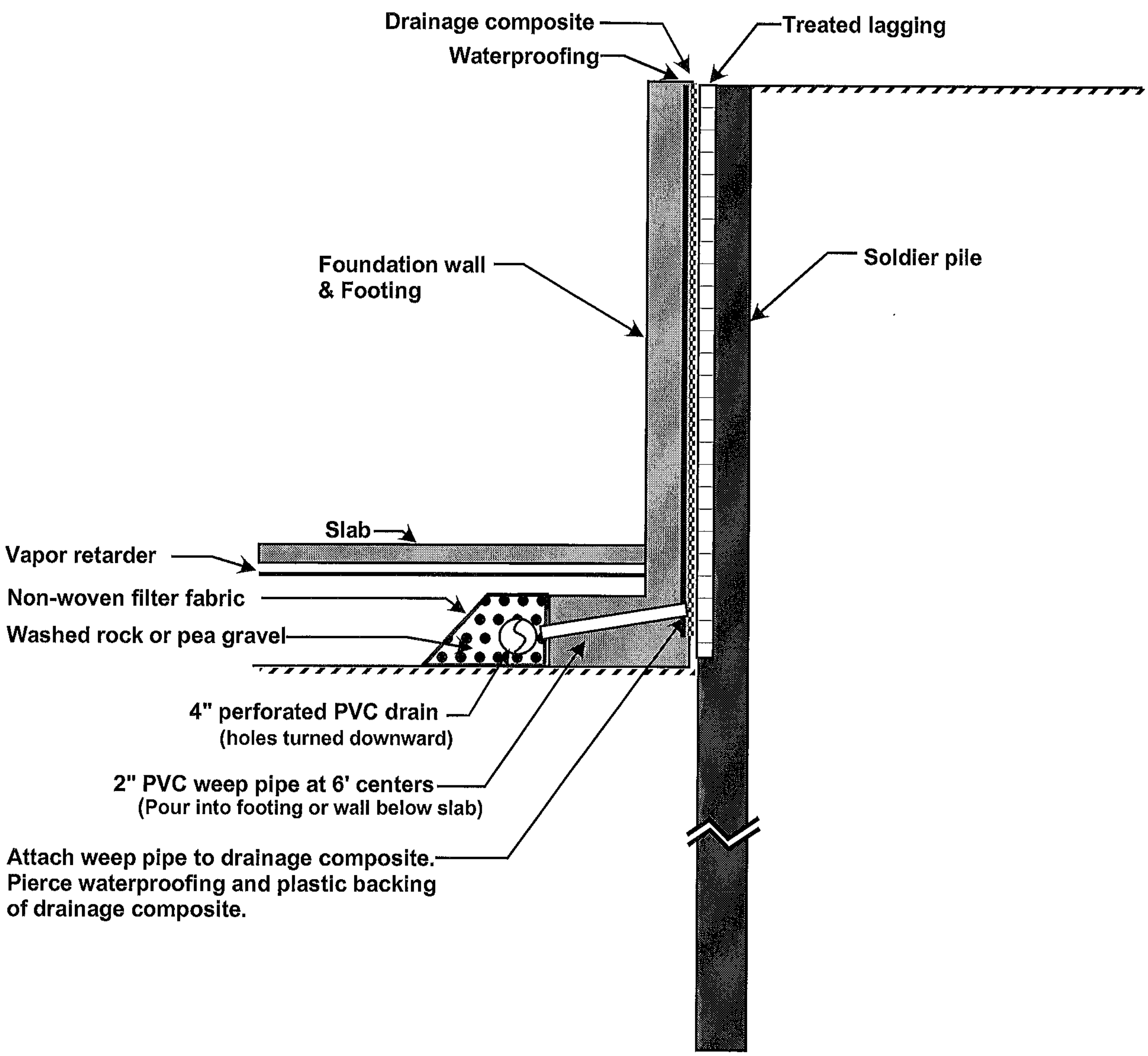
TYPICAL UNDERSLAB DRAINAGE

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
19011	Mar. 2019		7



<u>Note</u> - Refer to the report for additional considerations related to drainage and waterproofing.

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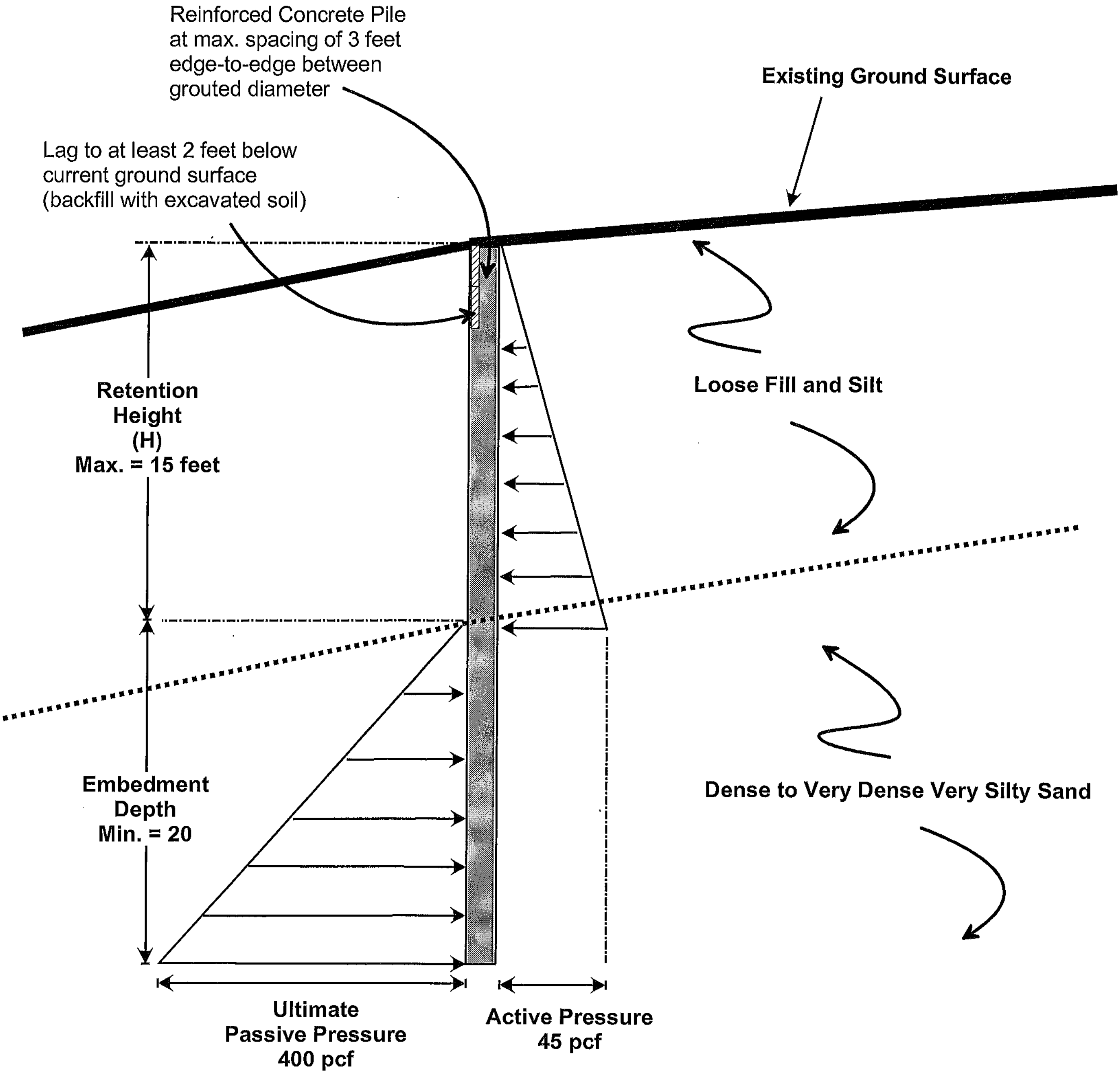
SHORING DRAIN DETAIL

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
19011	Mar. 2019		8



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

2035 - 81st Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
19011	Mar. 2019		9