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October 6, 2020

JN 20279

John Kahan
8167 West Mercer Way
Mercer Island, Washington

*via email: jbkahan@msn.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study and Critical Area Study**
Proposed New Residence
8163 West Mercer Way
Mercer Island, Washington

Dear Mr. Kahan,

Attached to this transmittal letter is our geotechnical engineering report for the proposed 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-10654, dated July 30, 2020.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Marc R. McGinnis, P.E.
Principal

cc: **Brandt Design Group** – Bree Medley
*via email: bree@brandtdesigngroup.com*

MKM/MRM:kg

# GEOTECHNICAL ENGINEERING STUDY
## Proposed New Residence
## 8163 West Mercer Way
## Mercer Island, Washington

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

Development of the property is in the planning stage, and detailed plans were not available at the time of this study. Preliminary plans were provided to us, prepared by Brandt Design Group, dated May 20, 2020. A topographic survey had also been prepared by CORE Design, dated April 20, 2015. For the purposes of this report, Project North is parallel to West Mercer Way, which is the same orientation used on Brandt Design's plans. Based on the plans, the property will be developed with a new residence located on the eastern extent of the irregular shaped lot. The new house will be two stories in height and will contain a western facing daylight basement. A two-story deck is shown extending off the western side of the main and upper levels, and a patio will likely be constructed off the west side of the house at the basement level. Currently, the existing driveway apron is proposed for use in the new construction, and will provide access to an attached, two-car garage. An accessory parking spot is shown east of the driveway. Preliminary property line setbacks of 10 feet from the north, 20 feet from the east, and 7.5 feet from the south are shown. Based on the preliminary cross sections, finish floor elevations of 119.25 feet and 109 feet are proposed for the main floor and basement. Based on these elevations, we can anticipate that excavations for the garage are planned to be relatively minimal, and excavations for the daylight basement and accessory parking area will be on the order of 10 feet.

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 near the southern tip of Mercer Island. The irregularly-shaped site encompasses approximately 0.4 acres between two other single-family properties connected by a winding, private driveway. The site is bordered to the north and south by driveways, and to the east and west by single family properties containing large single-family residences. The western adjacent property is owned by the client and is located well away from the subject site. The upslope, eastern property contains a newer two-story residence with a daylight basement situated greater than 10 feet from the property line. The neighbor's yard appears to extend past the surveyed property lines based on observations made in the field.

The site is currently undeveloped and is covered with blackberry bushes and tall grass. As mentioned previously, a hammerhead apron was installed into the expected building area when the driveway was originally constructed. What appears to be an old house foundation is located west of the development area, near where the lot necks down as the driveway meanders to the south to the adjacent western house. Previous documentation would indicate that this structure functioned as a barn prior to the property being subdivided in the late 1990's. An old, 6 to 12-foot tall concrete retaining wall lines the eastern side of the driveway downslope of the planned development area and the old foundation. This wall was observed to lean downslope slightly and had several visible cracks in its face. The grade across the lot and the surrounding properties generally slopes downward from east to west, with a total elevation change of 92 feet across the total property bounds. A relatively flat plateau has been created just south of the driveway and is likely composed of fill that was stockpiled on this lot during previous construction activity. This plateau is located approximately 6 to 8 feet higher than the driveway apron grade. Atop this plateau, and from the eastern property line, the grade carries out gently to moderately across the proposed development area, before dropping steeply downward to the west at an inclination of approximately 50 percent, with shorter portions of the steep slope inclined from 50 to 70 percent. Parts of this steep slope terminate along the irregular driveway alignment and may have been oversteepened when the driveway was built. This moderate to steep slope continues westward across the remainder of the property, and continues into the neighboring downslope, western property.

Research conducted on the City of Mercer Island GIS 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 planned development area meets the criteria for a Steep Slope and most of the site would meet the criteria for an Erosion Hazard. The *Mercer Island Landslide Hazard Assessment Map* (Troost and Wisher, 2009) indicates that several documented landslides have occurred in the general vicinity of the site. As a whole, the southern point of Mercer Island is well known for its landslide history, and the area is known to be underlain by slide debris and mass wasting deposits. We did not observe any indications of recent instability on the site during our field work. Past slides have occurred as shallow movement (less than 5 feet in depth) and larger movements up to 20 feet in depth. These slides affect the loose soils overlying the glacially-compressed silt, and are typically triggered following extended periods of rainfall.

While historic landslide records are difficult to obtain in this area of King County, we are aware that a large landslide affected the property two lots south of the subject site (#8235) in 1996. Our firm provided geotechnical services related to the investigation and stabilization of this landslide. The slope failure occurred on the downslope side of the existing residence and extended to a depth of approximately 20 feet. The slide mass extended through loose fill and colluvium, failing along the underlying dense silt, subsequently, soldier pile walls were constructed along the western side of the house and extending west along the northern side of the site to prevent another deep failure. These soldier pile walls are embedded into the dense silt.

We have not been able to find structural documents associated with the recently-constructed house upslope of the site. Considering the relatively poor surface soils known to underlie this area, it should have been supported on piles embedded into the underlying dense soils.

## *SUBSURFACE*

The subsurface conditions on the subject lot were explored by drilling three 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 borings were drilled on September 14, 2020 using a track-mounted, hollow-stem auger drill. 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***

Test Boring 1 was drilled off the edge of the hammerhead in the concrete driveway, and Test Boring 2 was drilled near the western edge of the proposed residence. Beneath the ground surface, approximately 7 feet of fill was encountered in Test Boring 2, which was drilled near the top of the steep slope. A thick layer of asphalt was encountered at 5 feet in Test Boring 2 and may have been a part of an old driveway associated with the foundation remains found to the west of this location. Beneath the fill in Test Boring 2, and beneath the ground surface in Test Boring 1, medium-stiff, and stiff silt was encountered. This silt was observed to contain organics, and was jumbled, fractured and blocky. This upper, looser silt deposit appears to be colluvium which would have been deposited during an older landslide event. This loose, fractured silt extended to a depth of approximately 22 feet in Test Boring 1, and 25 feet in Test Boring 2, where the silt became massive and hard. This hard silt extended to the base of the two test borings at a depth of 36.5 feet.

Test Boring 3 was attempted several times close to the approximate mapped location. This exploration met auger refusal at depths of 2 to 4 feet in this general area on what felt like large cobbles or quarry spalls. We were not able to hand clear the holes past this depth to try to advance the auger deeper. The presence of asphalt in Test Boring 2, as well as the rocks found at a shallow depth in Test Boring 3 could be an indicator that this site was used as a construction laydown area during the construction of either upslope or downslope residences. Regardless, we were not able to advance this boring to a useful depth in this location, no matter how much effort the driller made. We anticipate that an approximate 8-foot thick layer of fill would be encountered in this location beneath the existing ground surface, before aligning with the subsurface soil profile found in the location of Test Boring 1.

As stated above, obstructions in the form of what appeared to be quarry spalls or heavy cobbles were encountered in the vicinity of Test Boring 3, making it impossible to drill a boring. Debris, buried utilities, and old foundation and slab elements are commonly encountered on sites that have had previous development.

We conducted several borings for the development of the property to the south in the Fall of 1999 (currently addressed 8177 West Mercer Way, but the street address was 8229 West Mercer Way at the time of that report). Three borings were drilled across the upper lot, near the level of the subject site. These borings encountered approximately 20 to 25 feet of colluvium that was underlain by very dense silt. Even within the very dense silt, some moderate fracturing was observed. The results of these test borings are in general conformance with the subsurface conditions encountered in the test borings conducted for the proposed new residence.

We understand that the upslope (#8159) and downslope residences (#8167) were constructed less than 5 years ago. No recent public geotechnical information was attainable for the upslope property outside of the original short-plat report for the property conducted in 1996. However, PanGEO, Incorporated conducted the study for the downslope property in 2016. We were able to find the logs of their test borings on Mercer Island's GIS website. As part of their study, three borings were drilled within the residence footprint, as well as near the alignments of proposed permanent shoring walls near the eastern extent of the planned development area. These borings were located within a more moderately sloped area and encountered mass wasting (aka colluvium) and beach deposits to depths of approximately 10 to 15 feet beneath the ground surface. Medium-dense and denser sand, silty sand and silt were encountered beneath the mass wasting deposits, extending to the maximum drill depths attainable by the small, hand-carried Acker drill used. It is our understanding that the downslope residence is supported by small diameter pipe piles. No construction information was available for the upslope house, but we anticipate that the upslope residence is also supported on deep foundations.

Logs of the adjacent borings discussed above can be found in Appendix A. These borings are also publicly available on the City of Mercer Island GIS Tool.

### ***Groundwater Conditions***

No groundwater seepage was observed at a depth during drilling. 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, fracture zones in the silt, and between the looser near-surface soil and the underlying hard silt.

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

## ***CRITICAL 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 hard, glacially compressed, native soil that has a low potential for deep-seated landslides. However, this competent soil is overlain by looser fill and colluvial 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 Potential Landslide Hazard risk.

The foundations for the new construction will be supported on hard, non-liquefiable soils, which will mitigate 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 proposed 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. From our observations, and the findings of our explorations, this slope has been created by placement of fill on the lot, and previous excavation for the driveway extending to the adjacent western lot. 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 erosion impacts 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, particularly if grading and construction occurs during the wet season. 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 west of the silt fence be 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. Silty water accumulating in the excavation must not be allowed to flow off the site. In wet conditions, this can require the use of temporary holding tanks (aka Baker tanks). 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), the code-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 geologic 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 test borings conducted for this study encountered loose fill and colluvium beneath the ground surface. Hard silt was not encountered until a depth of 22 feet on the eastern, upslope side of the site, and until 25 feet on the western, downslope side of the site. Any new foundations constructed atop the loose, unconsolidated fill soil and colluvium would result in excessive post-construction settlement. All new foundation loads need to bear on, or into the suitable bearing soils. Due to the depth of the loose and medium-dense soils (22 to 25 feet), the excavations needed to reach competent soils would be near impossible to achieve without extensive excavation shoring. Therefore, we recommend that the new residence be supported 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 foundations system could consist of small diameter pipe piles that are driven to refusal in the underlying dense soil. An expanded discussion can be found in the ***Pipe Piles*** section of this report.

It is likely that some settlement of the ground surrounding pile-supported buildings will occur over time. In order to reduce the potential problems associated with this, we recommend the following:

- Fill to the desired site grades several months prior to constructing on-grade slabs, walkways, and pavements around the buildings. This allows the underlying soils to undergo some consolidation under the new soil loads before final grading is accomplished.

- Construct all entrance walkways as reinforced slabs that are doweled into the grade beam at the door thresholds. This will allow the walkways to ramp down and away from the building as they settle, without causing a downset at the threshold.

- Isolate on-grade elements, such as walkways or pavements, from pile-supported foundations and columns to allow differential movement.

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 hard native silt that has a low potential for deep-seated landslides. However, slides within the near-surface fill and colluvial soil can, and have, occurred in this area. The hazard to the residence associated with a potential failure of the onsite steep slope, or even of the leaning concrete retaining wall on the adjacent western property, will be mitigated by proper retention of the loose soils within the development area. Removal of any of the existing fill located in the western portion of the site on the steep slope will reduce the potential for future instability. Even so, due to the loose conditions of the surficial soils, and the steep slope west and northwest of the residence location, a stabilization wall will be needed along the downslope side of the residence to retain the upper, loose fill and colluvium 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, hard silt. The wall will need to run along the entire western perimeter of the house, and will need to return along the north side of the living space that will occupy the western half of the house. As discussed below, removal of at least some of the fill located on the western side of the site will reduce the depth and structural reinforcing necessary for the stabilization wall. Further discussion regarding the stabilization wall can be found in the ***Stabilization Wall*** section of this report. The western and northern foundation wall of the house could be supported on the drilled piles, but it is typically better to locate these piles outside the footprint of the structure, in order to provide stabilization for at least a narrow strip of ground alongside the house.

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. It should be noted that the proposed stabilization wall will not increase the stability of the slope west of the development area outside of transferring the weight of the new residence through the potentially unstable soils down to a competent soil strata. The future property owners should be made well aware that there always exists at least some risk with owning property on, or near steep slopes.

As previously discussed, the new residence will contain a basement constructed into the slope located west of the garage footprint. We anticipate that excavations of approximately 10 feet will be needed to reach the basement level in areas. It is not known if the adjacent western house incorporated a substantial pile foundation or a stabilization wall in its design. Considering this, and the low strength of the near-surface soils, we recommend that any temporary cuts along the east and south sides of the basement be shored using soldier piles. Based on the preliminary plan, we anticipate that shoring will be needed along the south and eastern sides of the basement excavation as well as along the perimeter of the driveway parking area along the eastern extent of the site. These soldier piles must consider the loose conditions of the near-surface soils. Driven soldier piles can cause excessive ground vibrations in the loose upper soils and should not be utilized for this project.

The proposed parking area on the eastern side of the driveway shows a retaining wall lining its perimeter. As discussed above, excavation shoring will be needed in this area as well due to the location of this wall to the property line, and the depth of the excavation that will be needed to create a flat area at the same elevation as the driveway. The shoring system in this area could consist of permanent soldier piles that remain exposed following construction, or a concrete retaining wall could be cast in front of the wall. Both temporary and permanent shoring design recommendations can be found in the ***Shoring*** section of this report. We recommend that the driveway slab be poured up to the face of this wall for additional lateral support.

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 older 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. Installation of driven pipe piles is a loud process, but does not result in strong ground vibrations. 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, such as hoepacks, 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.

The excavated soil will be unusable as fill for the project and should be hauled off the site. 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, the steep inclination of the slope to the west of the proposed residence, and the Potential Landslide Hazard, it is our professional opinion that onsite infiltration or dispersion of stormwater is not feasible for this project. All collected stormwater, even from paved surfaces, should be discharged to an approved stormwater system. Pervious pavements should not be used for this project.

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 buildup 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 1.46g and 0.55g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) during an earthquake be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period). The MCE peak ground acceleration adjusted for site class effects ($F_{PGA}$) equals 0.61g. The soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE (because of their dense nature and the absence of near-surface groundwater.

Sections 1803.5 of the IBC and 11.8 of ASCE 7 require that other seismic-related geotechnical design parameters (seismic surcharge for retaining wall design and slope stability) include the potential effects of the Design Earthquake. The peak ground acceleration for the Design Earthquake is defined in Section 11.2 of ASCE 7 as two-thirds (2/3) of the MCE peak ground acceleration, or 0.41g.

## *PIPE PILES*

Three- or 4-inch-diameter pipe piles driven with an 850- or 1,100- or 2,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

| INSIDE PILE DIAMETER | FINAL DRIVING RATE (850-pound hammer) | FINAL DRIVING RATE (1,100-pound hammer) | FINAL DRIVING RATE (2,000-pound hammer) | ALLOWABLE COMPRESSIVE CAPACITY |
|---|---|---|---|---|
| 3 inches | 10 sec/inch | 6 sec/inch | 2 sec/inch | 6 tons |
| 4 inches | 16 sec/inch | 10 sec/inch | 4 sec/inch | 10 tons |

**Note:** The refusal criteria indicated in the above table are valid only for pipe piles that are installed using a hydraulic impact hammer carried on leads that allow the hammer to sit on the top of the pile during driving. If the piles are installed by alternative methods, such as a vibratory hammer or a hammer that is hard mounted to the installation machine, numerous load tests to 200 percent of the design capacity would be necessary to substantiate the allowable pile load. The appropriate number of load tests would need to be determined at the time the contractor and installation method are chosen.

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

We expect that the City of Mercer Island will require geotechnical observation of the pile installation. Considering this, the recommendations we have made above for minimum refusal criteria, and our previous experience with pile projects in close proximity to the site, it is our professional opinion that the recommended capacities do not need to be verified by load testing.

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, in the event that the ground downslope of the piles moves away.

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 at least 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 extending to the very stiff to hard silt encountered below the fill and colluvium in our test borings. This correlates to an approximate elevation of 89 feet along the western wall of the proposed development area. Along the northern side of the planned house, the elevation of the very stiff to hard silt can be assumed to step up from elevation 89 feet to 94 feet along the north wall of the living area, west of the garage. The total stabilization depth will be influenced by the final elevation of the basement slab and foundations. The depth of the lateral active soil load acting on the piles could be reduced by removing some of the fill and colluvium underneath the western portion of the house, and using either a deepened crawl space, or a structural slab constructed over geofoam. An ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 375 pcf will resist the lateral movement of the piles below the recommended the stabilization elevation.

If the stabilization piles will be used to support the foundation of the structure, a seismic surcharge pressure of 10H pounds per square foot should be added to the active design earth pressure. H is the design retention depth of the stabilization piles.

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 hard silt 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 level backfill:

| PARAMETER | VALUE |
|---|---|
| Active Earth Pressure * | 45 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 values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

### ***Wall Pressures Due to Seismic Forces***

Per IBC Section 1803.5.12, a seismic surcharge load need only be considered in the design of walls over 6 feet in height. A seismic surcharge load would be imposed by adding a uniform lateral pressure to the above-recommended active pressure. The recommended seismic surcharge pressure for this project is 10**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, and cannot be adequately compacted to avoid excessive post-construction settlement. We recommend that the native soils not be reused as retaining 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.

## *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. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type C. Temporary cut slopes up to approximately 4 feet in height could be excavated at an inclination no steeper than 1.5:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. Taller cuts, especially along the east and south sides, should be shored with soldier piles.

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 onsite soil should be inclined no steeper than 3:1 (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.

## SOLDIER PILE SHORING

As discussed in the ***General*** section, the excavation for the proposed basement and eastern parking area will 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. We anticipate that at least a top casing extending to the underlying competent silt will be needed to prevent caving in the upper, loose soils. 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 45 pounds per cubic foot (pcf). For a permanent wall east of the driveway and parking space, an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 60 pounds per cubic foot (pcf) should be used to account for the sloped condition. Traffic surcharges, such as from the adjacent southern driveway, can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Existing adjacent structures such as the neighboring southern driveway will exert surcharges on the proposed shoring wall. 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.

The soils below the excavation level for the shoring will be loose, and have a low strength. 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 250 pcf. This value is low due to the very low internal strength of the fill and colluvial soils that the soldier piles will be embedded into. 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*

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 6.

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

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 8.

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 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 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 John Kahan 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 - 5 | Test Boring Logs |
| Plate 6 | Typical Shoring Drainage Detail |
| Plate 7 | Typical Footing Drain Detail |
| Plate 8 | Typical Underslab Drainage Detail |
| Appendix A | Adjacent Boring Logs |

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.

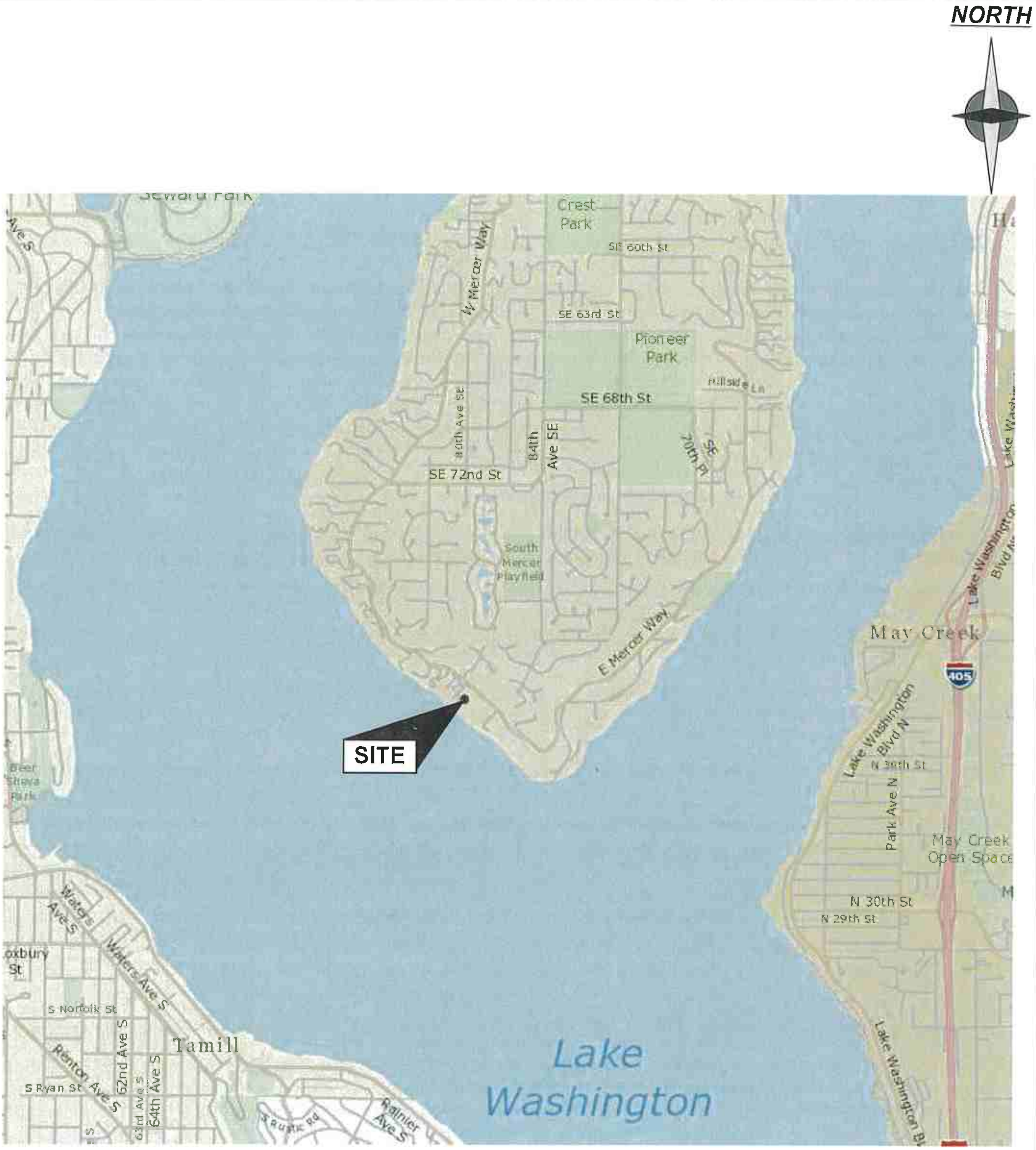
Respectfully submitted,

GEOTECH CONSULTANTS, INC.

MARC R. McGINNIS
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER

10/06/2020

Marc R. McGinnis, P.E.
Principal



(Source: King County iMap)

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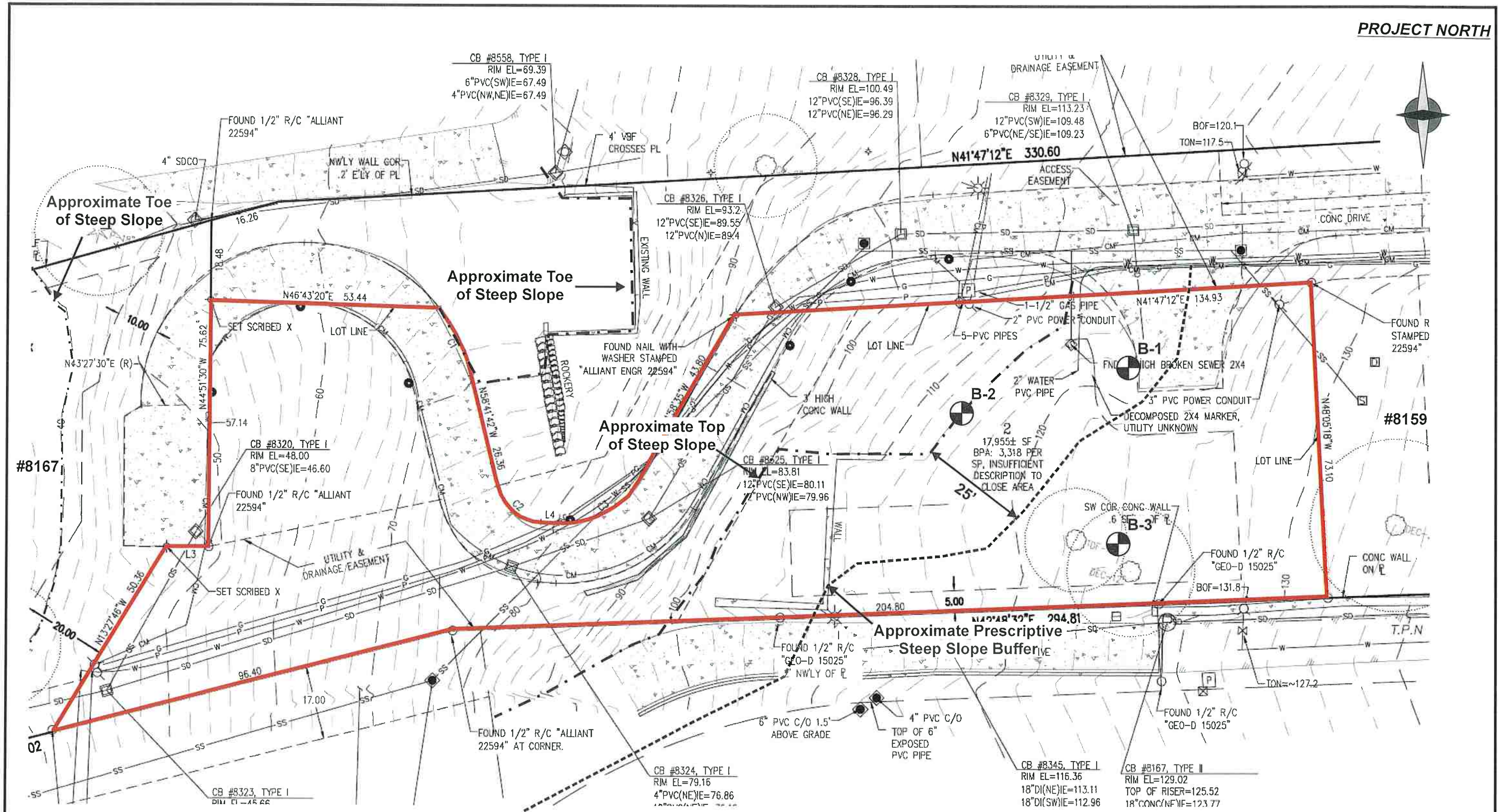
# VICINITY MAP

8163 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 20279 | Oct. 2020 | | 1 |



**Legend:**

- ◐ Test Boring Location
- ╲ Lot Boundary

* The City of Mercer Island 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 property boundaries.

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# SITE EXPLORATION PLAN

8163 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 20279 | Sept. 2020 | | 2 |

# BORING 1

| | Moisture | Water Table | Blows per Foot | Sample | USCS | Description |
|---|---|---|---|---|---|---|
| | | | | | | Topsoil |
| | | | 6 | 1 | ML | Gray and brown, heavily mottled SILT with organics, low plasticity, moist, fractured, medium-stiff **(Colluvium)** |
| 5 | | | 6 | 2 | | -with pockets of brown, gravelly sand |
| | | | 16 | 3 | | -becomes gray mottled orange and dark-brown, soft |
| | | | | | SM | Gray-brown mottled orange, gravelly, silty SAND, fine to medium-grained, very moist, medium-dense |
| 10 | | | 23 | 4 | | Gray mottled orange, SILT with seams of very gravelly sand, low plasticity, moist, fractured, very stiff **(Colluvium?)** |
| 15 | | | 14 | 5 | ML | -becomes gray, slightly gravelly, slightly plastic, stiff |
| 20 | | | 26 | 6 | | -becomes gray-brown mottled orange and gray, slightly sandy, slightly fractured, very stiff<br>-less fractured at tip of sampler |
| | | | | | | -driller noted stiffer drilling action at 22 feet |
| 25 | | | 33 | 7 | | Gray with light-gray streaks, slightly sandy SILT, low plasticity, moist, massive, hard |
| | | | | | ML | |
| 30 | | | 47 | 8 | | -becomes bluish-gray, horizontally bedded |
| 35 | | | 32 | 9 | | |
| 40 | | | | | | |

*** Test boring was terminated at 36.5 feet on September 14, 2020.**

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

GEOTECH
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## BORING LOG

8163 West Mercer Way
Mercer Island, Washington

| Job | Date: | Logged by: | Plate: |
|---|---|---|---|
| 20279 | Sept. 2020 | MKM | 3 |

# BORING 2

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40

| Moisture | Water Table | Blows per Foot | Sample | USCS | Description |
|---|---|---|---|---|---|
| | | | | | Grass over; |
| | | 15 | 1 | FILL | Brown silty SAND with decayed organics and roots, fine-grained, moist to dry, loose **(FILL)** |
| | | ** | 2 | | 4-inch layer of asphalt |
| | | 5 | 3 | | Gray-brown mottled orange, slightly clayey SILT with organics and pieces of hard silt, slightly plastic, moist, fractured and blocky, soft **(Colluvium)** |
| | | 10 | 4 | | -becomes mottled orange, brown, and gray with thin gravelly seams, medium-stiff |
| | | 11 | 5 | ML | -becomes gray-brown, slightly gravelly with a lense of very moist to wet, gravelly, silty sand |
| | | **44 | 6 | | -becomes very stiff with a 6-inch layer of wet, gravelly, silty sand (**Overstated blow counts - rock in tip of sampler) |
| | | 34 | 7 | | Bluish-gray SILT, slightly plastic, moist, massive, bedded, hard |
| | | 94/11" | 8 | ML | -becomes gray-brown mottled orange, slightly blocky<br>-layer of gray, gravelly, silty SAND, fine-grained, moist, very dense |
| | | 50/5" | 9 | | -becomes gray with trace rusting, low plasticity, very dense |

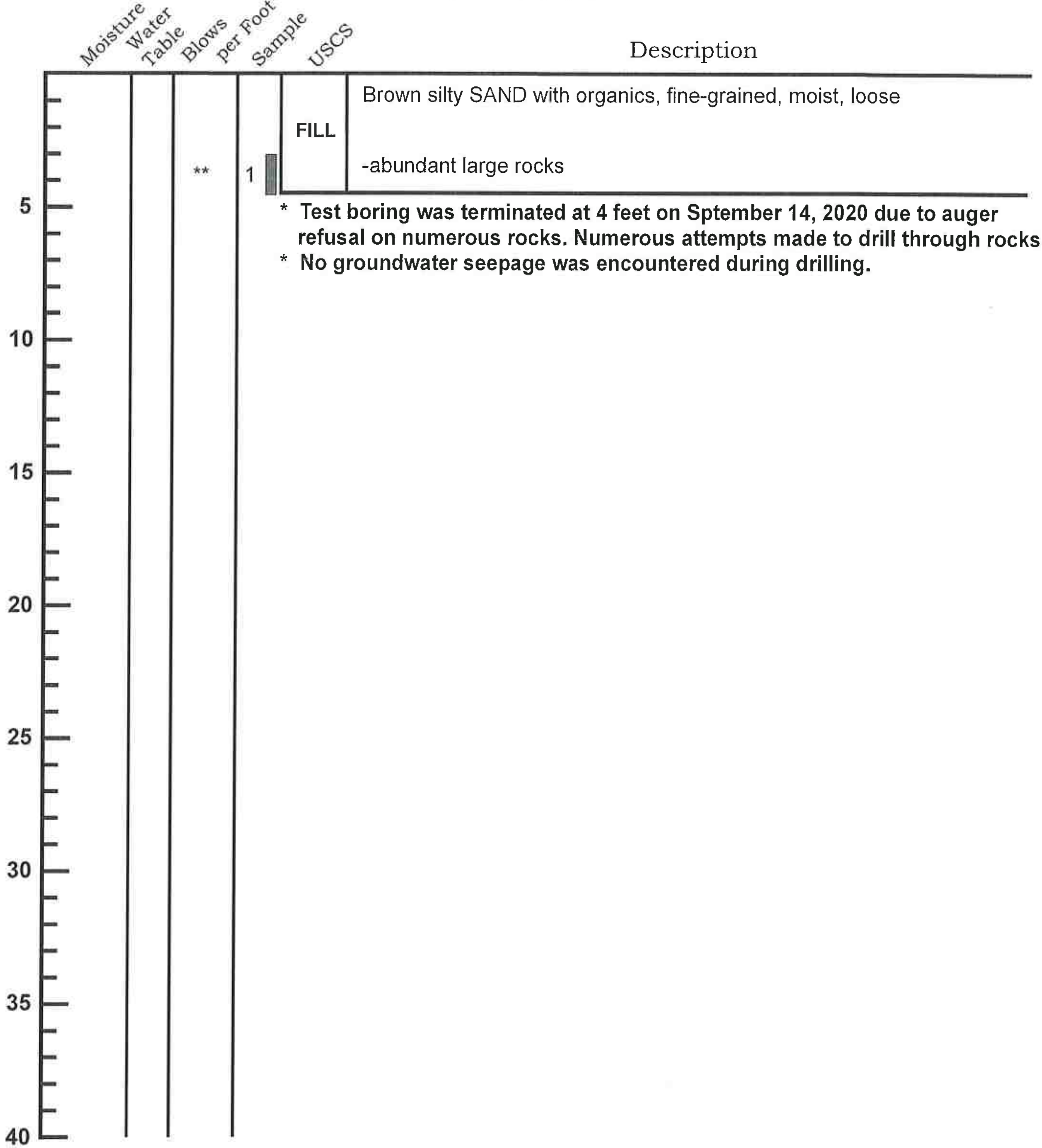
- **Test boring was terminated at 36.5 feet on September 14, 2020.**
- **No groundwater seepage was encountered during drilling.**

**GEOTECH**
CONSULTANTS, INC.

**BORING LOG**
8163 West Mercer Way
Mercer Island, Washington

| Job | Date: | Logged by: | Plate: |
|---|---|---|---|
| 20279 | Sept. 2020 | MKM | 4 |

# BORING 3



Brown silty SAND with organics, fine-grained, moist, loose

-abundant large rocks

*** Test boring was terminated at 4 feet on Sptember 14, 2020 due to auger refusal on numerous rocks. Numerous attempts made to drill through rocks.**

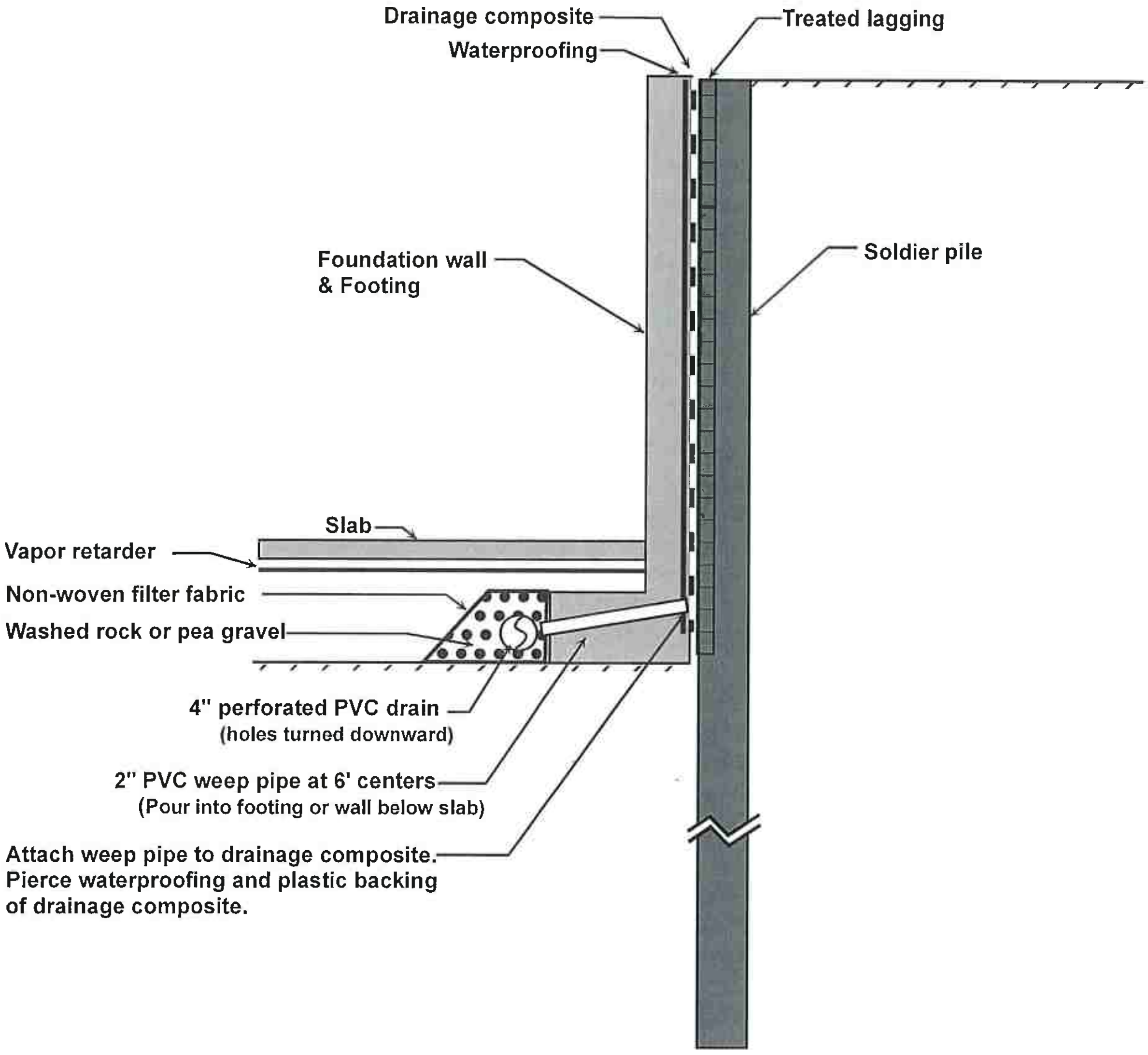
*** No groundwater seepage was encountered during drilling.**

GEOTECH
CONSULTANTS, INC.

## BORING LOG

8163 West Mercer Way
Mercer Island, Washington

| Job | Date: | Logged by: | Plate: |
|---|---|---|---|
| 20279 | Sept. 2020 | MKM | 5 |



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

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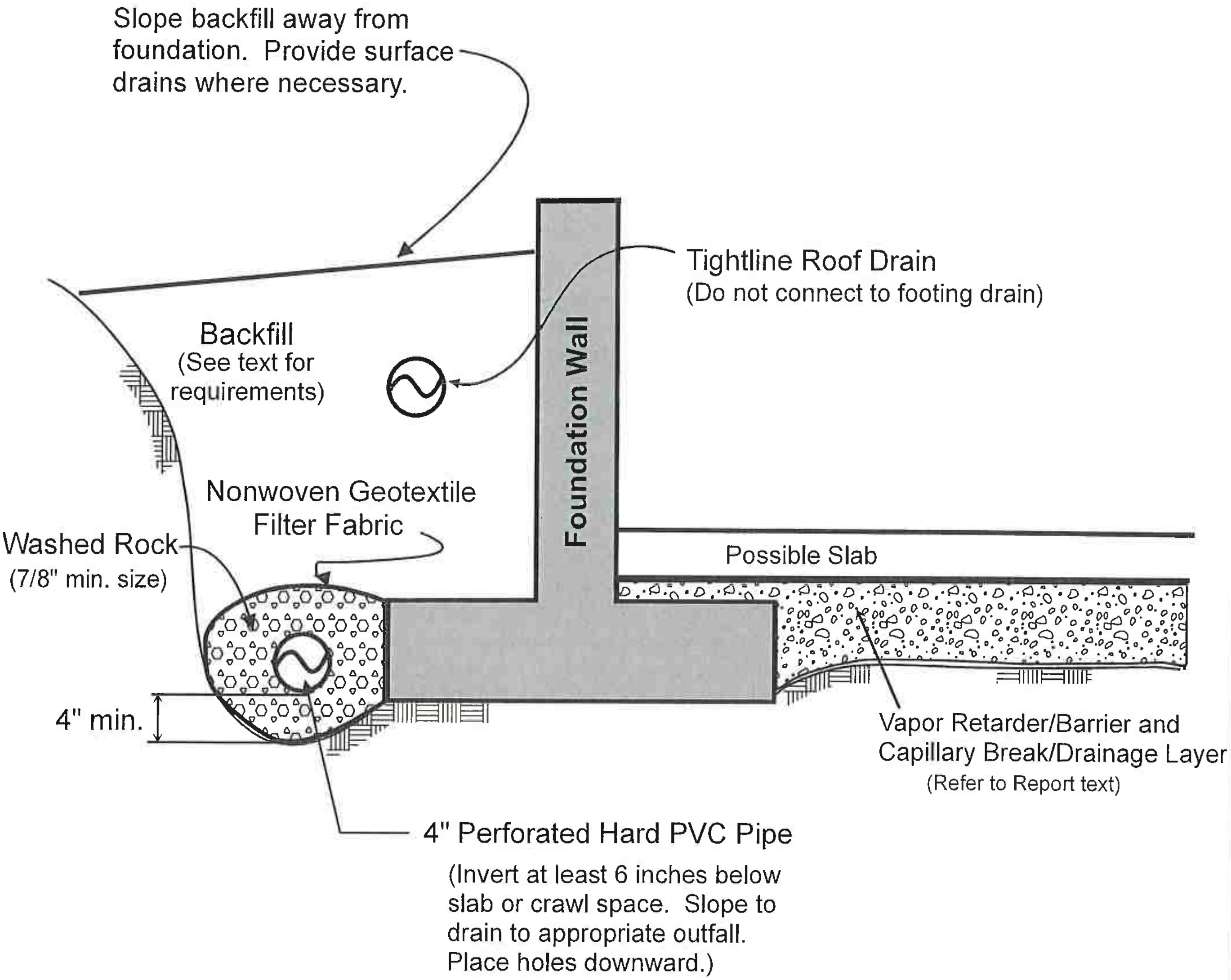
CONSULTANTS, INC.

# SHORING DRAIN DETAIL

8163 West Mercer Way
Mercer Island, Washington

| | | | |
|---|---|---|---|
| *Job No:* 20279 | *Date:* Oct. 2020 | | *Plate:* 6 |



**NOTES:**

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

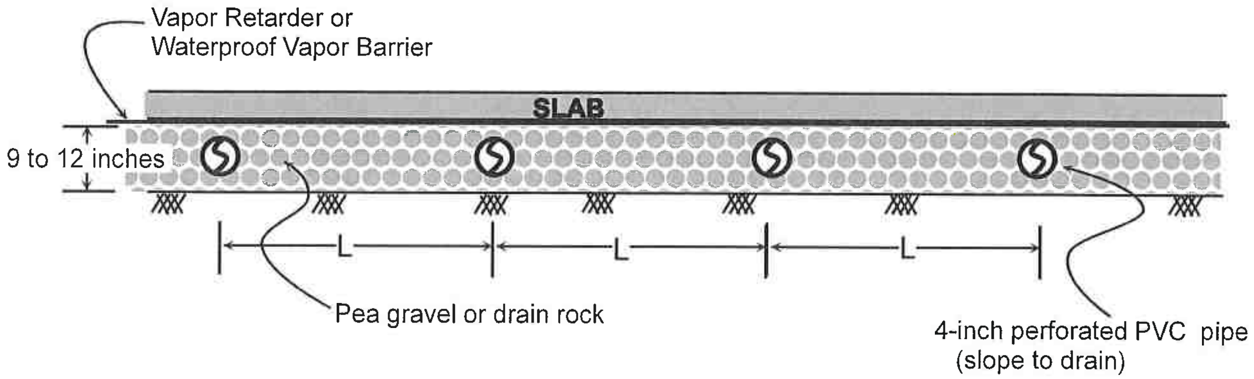
(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

**GEOTECH**
CONSULTANTS, INC.

# FOOTING DRAIN DETAIL

8163 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 20279 | Oct. 2020 | | 7 |



**NOTES:**

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

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**CONSULTANTS, INC.**

## TYPICAL UNDERSLAB DRAINAGE

8163 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 20279 | Oct. 2020 | | 8 |

# Appendix A

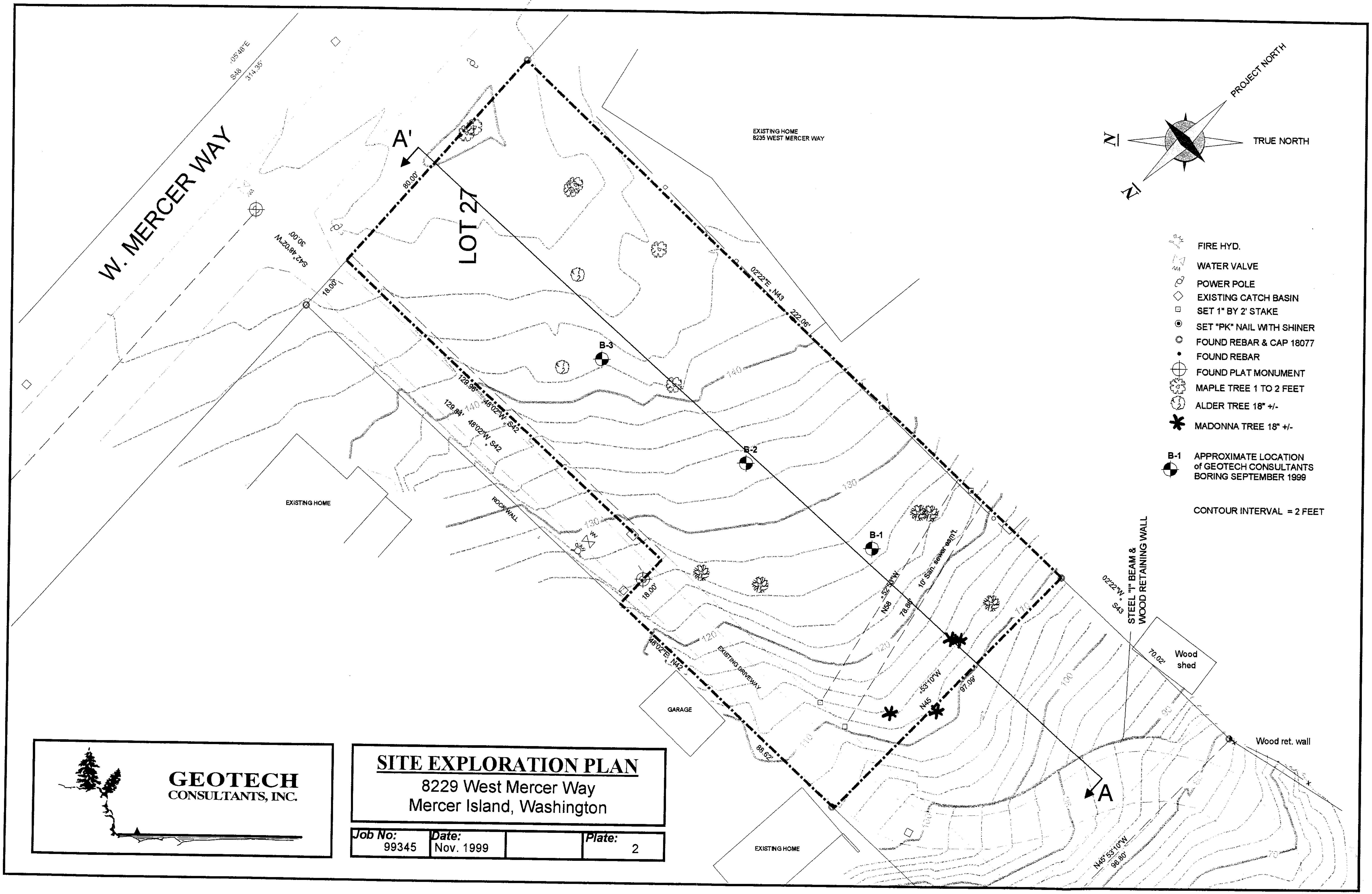
# Nearby Explorations

## Geotech Consultants, Inc.

## November 15, 1999 Proposed Residence – 8229 West Mercer Way

## PanGEO, Inc. March 15, 2016

## 8167 West Mercer Way



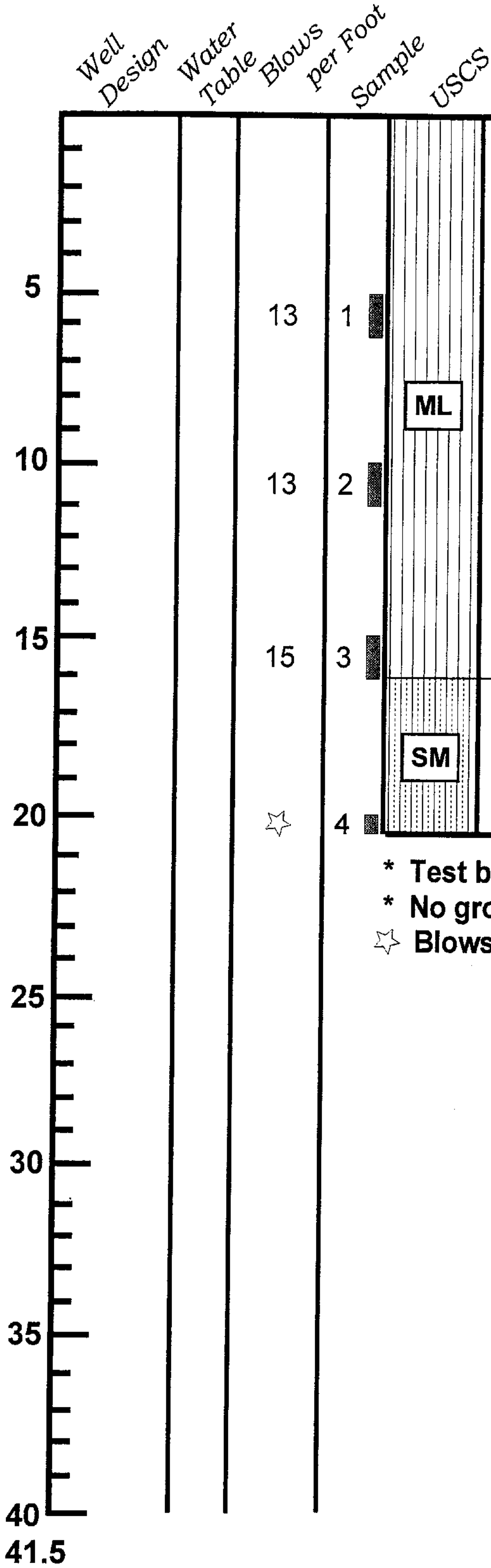
**GEOTECH**
**CONSULTANTS, INC.**

# SITE EXPLORATION PLAN

8229 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | | Plate: |
|---|---|---|---|
| 99345 | Nov. 1999 | | 2 |

# BORING 1



*Description*

Olive-brown, sandy SILT, with decayed organics and shards of hard silt, low plasticity, loose to medium-dense ***(Colluvium)***

Brown, gravelly, silty SAND, fine-grained, moist, medium-dense ***(Colluvium)***

- *** Test boring was terminated at 20.5 feet during drilling on September 15, 1999.**
- *** No groundwater seepage was encountered during drilling.**
- **☆ Blows not representative. Sampler pounded on a rock.**

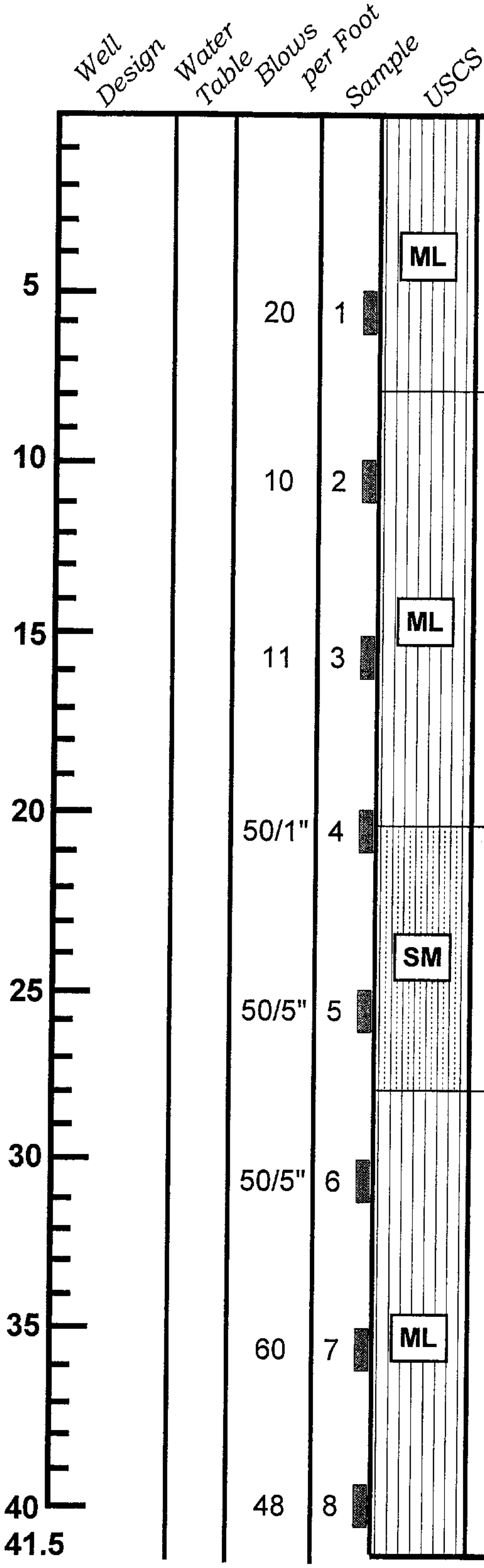
**GEOTECH**
**CONSULTANTS, INC.**

## BORING LOG

8229 West Mercer Way
Mercer Island, Washington

| ***Job No:*** | ***Date:*** | ***Logged by:*** | ***Plate:*** |
|---|---|---|---|
| 99345 | 9-99 | SES | 3 |

# BORING 2



*Description*

Light brown, sandy SILT, with some organics, medium-dense, moist, ***(Colluvium)***

- becomes gravelly

Olive-brown, clayey SILT, with trace sand, medium-plasticity, very moist, loose ***(Colluvium)***

- becomes gravelly

Brown with gray and orange mottling, silty SAND, with gravel, fine-grained, very moist, very dense

Gray SILT, massive, low plasticity, moist, very dense

*** Test boring was terminated at 41.5 feet during drilling on September 15, 1999.**
*** No groundwater seepage was encountered during drilling.**

**GEOTECH**
**CONSULTANTS, INC.**

**BORING LOG**

8229 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | Logged by: | Plate: |
|---|---|---|---|
| 99345 | 9-99 | SES | 4 |

# BORING 3

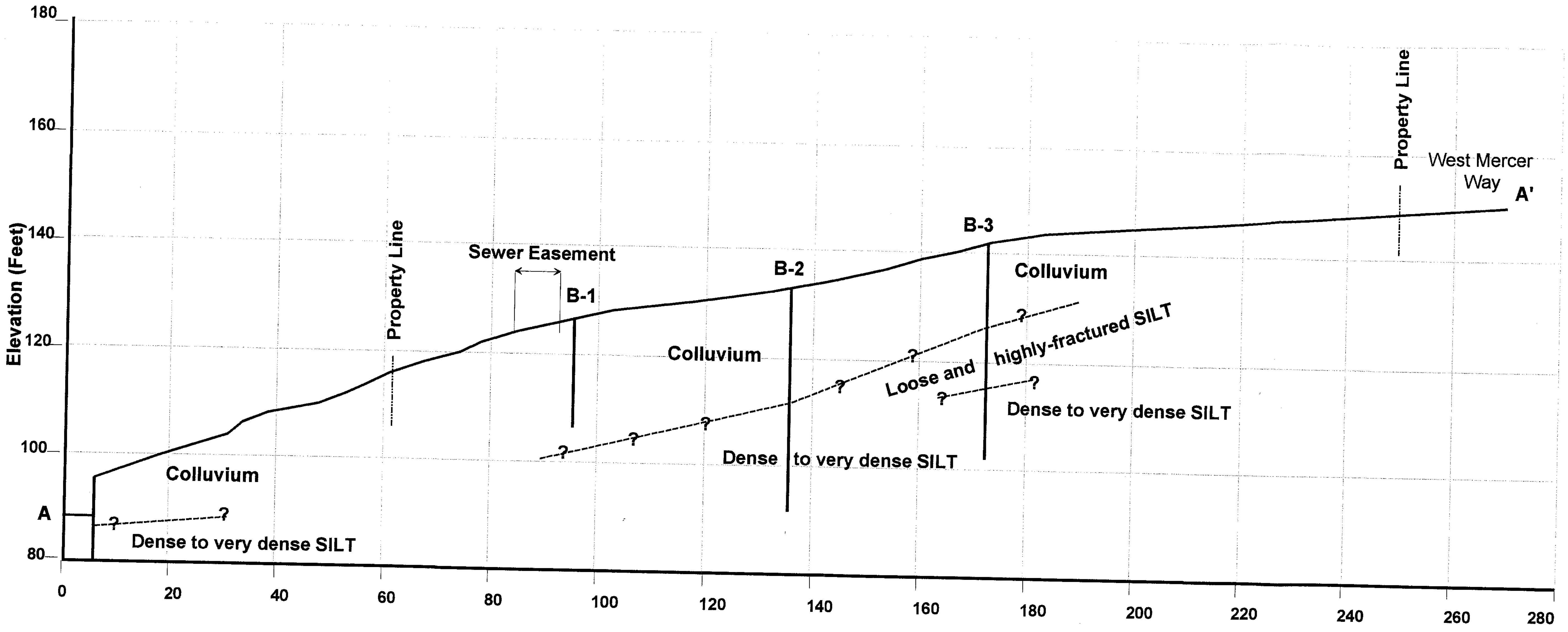
| | Well Design | Water Table | Blows per Foot | Sample | USCS | Description |
|---|---|---|---|---|---|---|
| | | | | | | Brown, gravelly, sandy SILT, non-plastic, moist, loose ***(Colluvium)*** |
| | | | | | ML | |
| 5 | | | 73 | 1 | | - gravelly layer |
| | | | | | | Olive-brown SILT, low plasticity, very moist, loose ***(Colluvium)*** |
| 10 | | | 11 | 2 | ML | |
| | | | | | | Olive-brown SILT, massive, non-plastic, very moist, loose |
| 15 | | | 10 | 3 | ML | |
| | | | | | | Dark gray, slightly clayey SILT, heavily-fractured, low plasticity, stiff to very stiff |
| 20 | | | 16 | 4 | | |
| 25 | | | 26 | 5 | ML | - becomes thinly bedded, slightly fractured |
| 30 | | | 53 | 6 | | - becomes olive-brown, moderately fractured |
| | | | | | | Dark gray SILT, massive, low plasticity, very moist, dense to very dense |
| 35 | | | 32 | 7 | | |
| | | | | | ML | |
| 40 | | | 64 | 8 | | |
| 41.5 | | | | | | |

*** Test boring was terminated at 41.5 feet during drilling on September 15, 1999.**
*** No groundwater seepage was encountered during drilling.**

GEOTECH
CONSULTANTS, INC.

**BORING LOG**
8229 West Mercer Way
Mercer Island, Washington

| Job No: | Date: | Logged by: | Plate: |
|---|---|---|---|
| 99345 | 9-99 | SES | 5 |



**LEGEND:**

- ▼ **Approximate water table**
- ▌ **Test Borings completed by Geotech Consultants on September 15, 1999**

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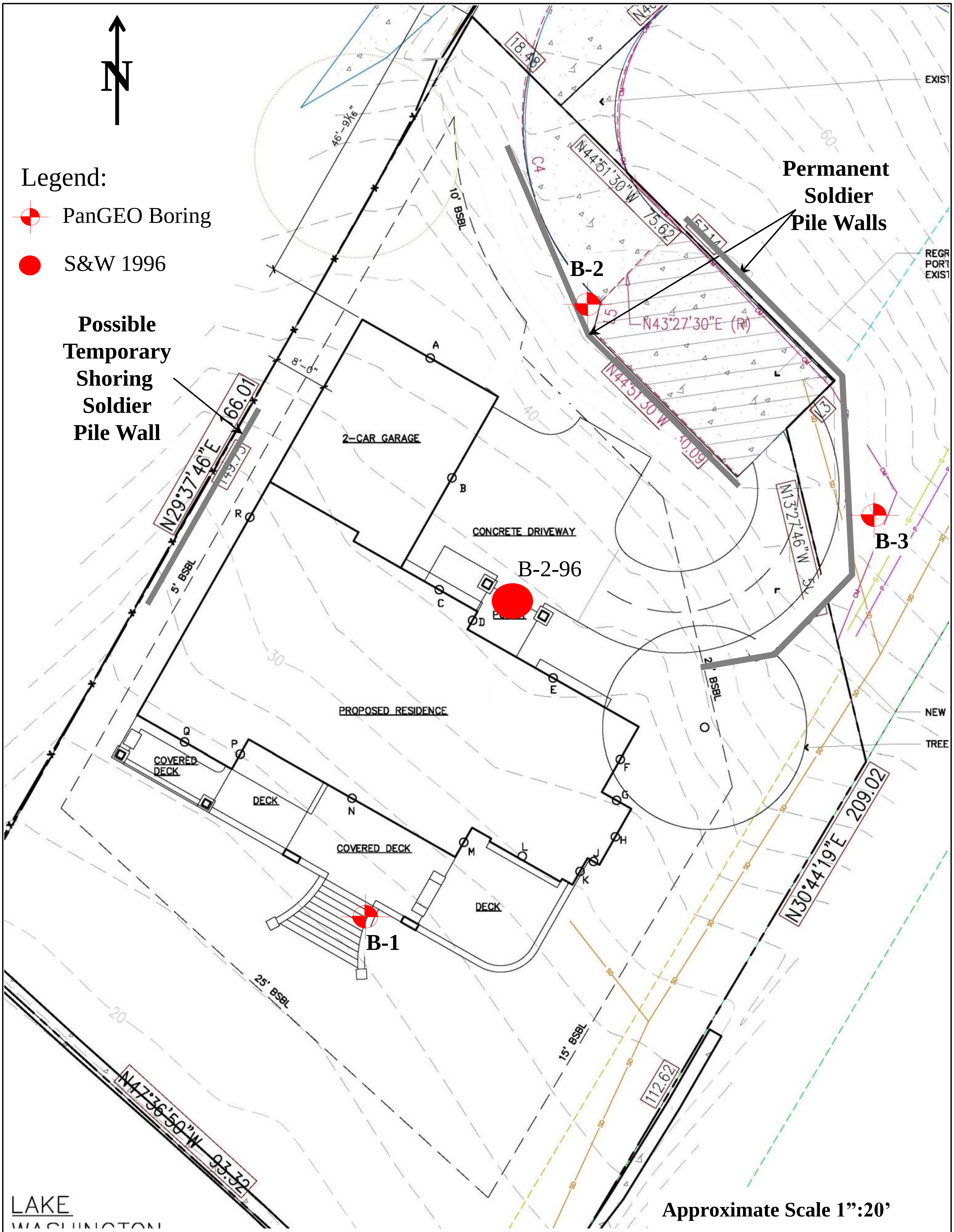
CONSULTANTS, INC.

## SUBSURFACE CROSS SECTION

8229 West Mercer Way
Mercer Island, Washington

| | | | |
|---|---|---|---|
| *Job No:* 99345 | *Date:* Nov. 1999 | | *Plate:* 6 |



PanGEO
INCORPORATED

**Proposed New Residence**
**8167 W. Mercer Way**
**Mercer Island, Washington**

**SITE AND EXPLORATION PLAN**

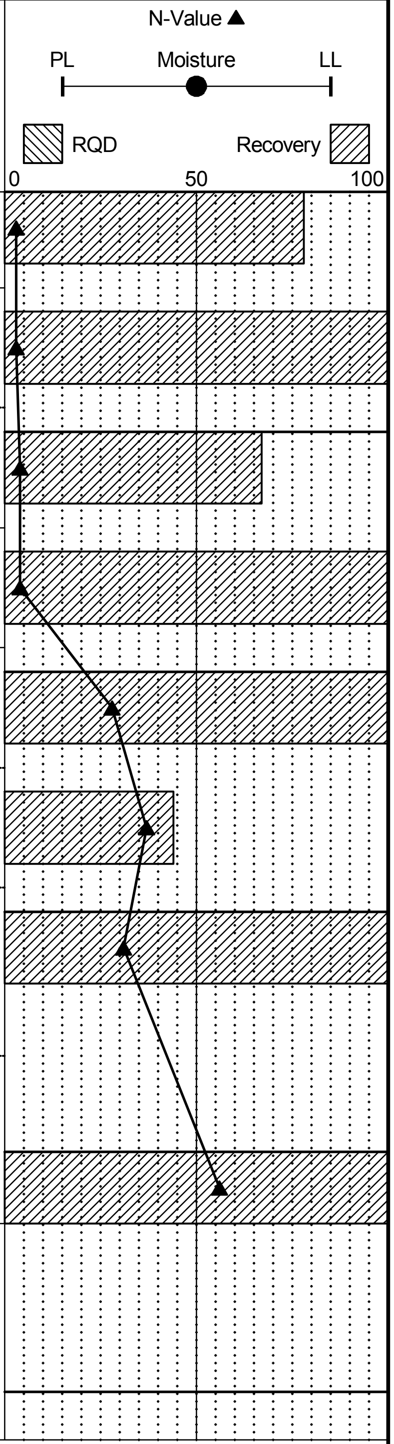
Project No. 15-099.100

Figure No. 2

15-099.100_Figure-1 - 2.ppt 3/4/2016(10:12 AM) SHE

| | | | |
|---|---|---|---|
| Project: | Proposed New Residence | Surface Elevation: | 26.0ft |
| Job Number: | 15-099.100 | Top of Casing Elev.: | |
| Location: | 8167 West Mercer Way, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: , Easting: | Sampling Method: | SPT |

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0 | S-1 | ☒ | 1<br>1<br>2 | | ▼ | Loose, dark brown, fine sandy SILT with organics: very moist, mixed texture, massive. (Topsoil / Beach and Mass Wasting Deposit). |
| | S-2 | ☒ | 2<br>1<br>2 | | | Soft, speckled dark gray and rusty brown, silty CLAY: very moist, low to medium plastic, abundant organics, fine mottles, mixed and broken texture, scattered fine gravel and sand. (Mass Wasting Deposit). |
| 5 | S-3 | ☒ | 3<br>2<br>2 | | | Soft and loose, interbedded, dark gray, silty CLAY with gravel and silty, fine to medium SAND with gravel: wet, low plastic and non-plastic, trace rusty weathering, trace organics, broken and mixed textures, massive. (Mass Wasting and Lake Beds). |
| | S-4 | ☒ | 2<br>1<br>3 | | | Loose, green gray, silty, fine SAND: wet, low plastic fines with slow dilatancy, homogeneous, laminated, occasional fine gravel bands. (Lake Deposit). |
| 10 | S-5 | ☒ | 13<br>13<br>15 | | | Medium dense, green gray, interbedded silty, fine to medium SAND and sandy, fine GRAVEL: very moist, slightly to low plastic fines, fine bedded (3 to 4 inches), trace weathering, laminated, silty clay bed in tip. (pre-Olympia Beds). |
| | S-6 | ☒ | 18<br>20<br>17 | | | Dense, brown to green gray, silty, fine SAND: very moist, non-plastic to slightly plastic fines, fine, sub-rounded, tabular to prismatic gravel beds, laminated. (pre-Olympia Beds). |
| 15 | S-7 | ☒ | 8<br>14<br>17 | | | Dense, brown, fine sandy SILT: moist, non-plastic, homogeneous, green gray vein at 60°, indistinct bedding. (pre-Olympia Bed). |
| 20 | S-8 | ☒ | 16<br>20<br>36 | | | Very dense, brown, silty, fine to medium SAND: very moist, homogeneous, non-plastic fines, indistinctly laminated to massive. (pre-Olympia Bed). |
| 25 | | | | | | Bottom of Boring. |



| | |
|---|---|
| Completion Depth: | 21.5ft |
| Date Borehole Started: | 2/12/16 |
| Date Borehole Completed: | 2/12/16 |
| Logged By: | S. Evans |
| Drilling Company: | CN Drilling |

Remarks: Groundwater measured in open boring roughly 3 hours following extraction of augers. May not represent static groundwater level.

# LOG OF TEST BORING B-1

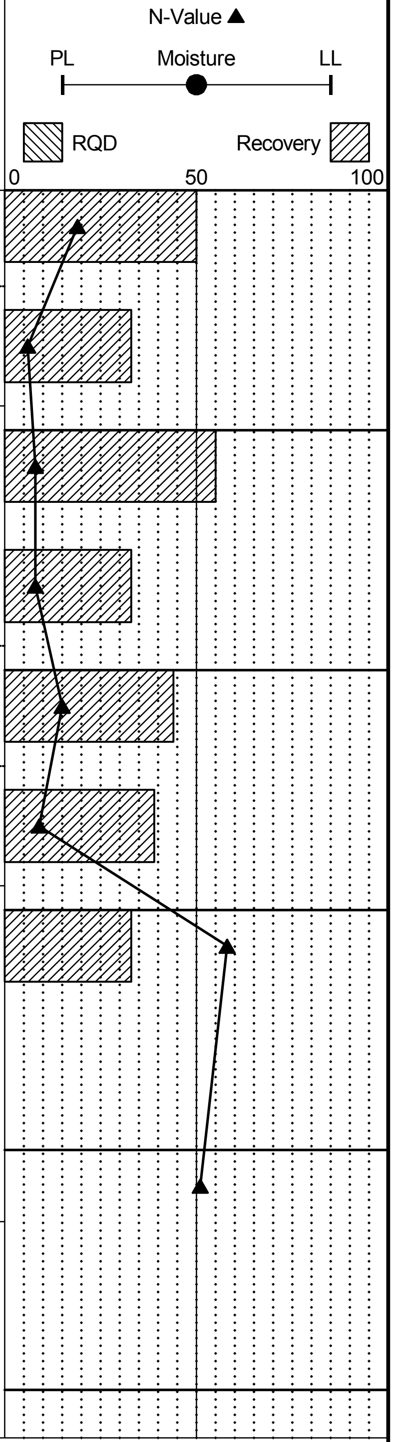
**Figure 4**

The stratification lines represent approximate boundaries. The transition may be gradual.

Project: Proposed New Residence
Job Number: 15-099.100
Location: 8167 West Mercer Way, Mercer Island, WA
Coordinates: Northing: , Easting:

Surface Elevation: 48.0ft
Top of Casing Elev.:
Drilling Method: HSA
Sampling Method: SPT

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0 | S-1 | | 4<br>5<br>14 | | | Medium dense, brown, gravelly, silty, fine to coarse SAND; very moist, sub-angular and blocky gravel, slightly plastic fines, trace of organics, mixed texture. (Fill). |
| | S-2 | | 4<br>3<br>3 | | | Loose, brown, silty/clayey GRAVEL with sand: very moist, slightly to low plastic fines, sub-angular to sub-rounded and blocky gravel, mixed texture, 50% clast supported. (Fill). |
| 5 | S-3 | | 3<br>3<br>5 | | | Medium stiff, brown, silty CLAY with gravel: very moist, low plastic, some sand, scattered charcoal, sub-angular to sub-rounded gravel, mixed and massive. (Mass Wasting Deposits). |
| | S-4 | | 2<br>2<br>6 | | | Grading to sandy, silty CLAY with gravel, some charcoal seams. |
| 10 | S-5 | | 9<br>7<br>8 | | | Medium dense, yellow brown, gravel with silt and sand: wet, fine to coarse, sub-rounded and blocky gravel, slightly to low plastic fines, massive. (Mass Wasting Deposits). |
| | S-6 | | 6<br>5<br>4 | | | Stiff, mixed brown and gray, silty CLAY with gravel: very moist, low plastic, some sand, mixed and massive. (Mass Wasting Deposits). |
| 15 | S-7 | | 20<br>36<br>22 | | | Dense to very dense, silty, sandy GRAVEL: wet, low plastic fines, massive. (pre-Olympia Beds). |
| 20 | S-8 | | 16<br>21<br>30 | | | |
| | | | | | | Bottom of Boring. |
| 25 | | | | | | |



Completion Depth: 21.5ft
Date Borehole Started: 2/12/16
Date Borehole Completed: 2/12/16
Logged By: S. Evans
Drilling Company: CN Drilling

Remarks: Groundwater measured in open boring roughly 2.5 hours following extraction of augers. May not represent static groundwater level.

PanGEO INCORPORATED

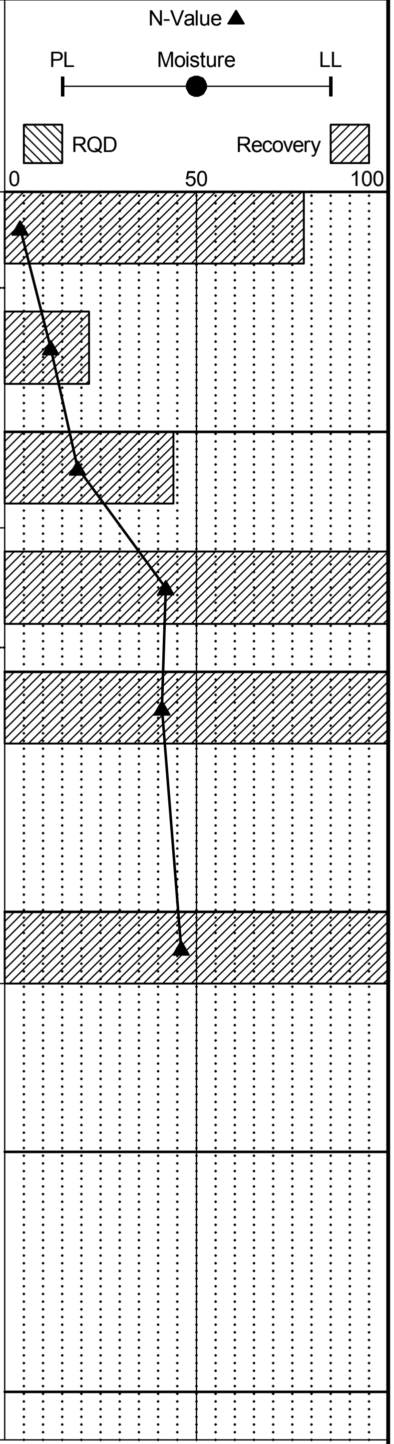
## LOG OF TEST BORING B-2

**Figure 5**

The stratification lines represent approximate boundaries. The transition may be gradual.

|  |  |  |  |
|---|---|---|---|
| Project: | Proposed New Residence | Surface Elevation: | 50.0ft |
| Job Number: | 15-099.100 | Top of Casing Elev.: |  |
| Location: | 8167 West Mercer Way, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: , Easting: | Sampling Method: | SPT |

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0 | S-1 | ☒ | 1<br>2<br>2 |  |  | Medium stiff, brown, silty CLAY with gravel: very moist, low plastic, trace organics, mixed and broken textures, massive. (Mass Wasting Deposits). |
|  | S-2 | ☒ | 5<br>5<br>7 |  |  | Medium dense, brown gray, fine to coarse GRAVEL with sand and silt: wet, slightly to low plastic fines, sub-rounded and blocky, massive. (Mass Wasting Deposits). |
| 5 | S-3 | ☒ | 12<br>11<br>8 |  |  |  |
|  | S-4 | ☒ | 9<br>16<br>26 |  | ▼ | Dense, brown, fine sandy SILT: wet, non-plastic, rapid dilatancy, occasional gravel, homogeneous, laminated. (pre-Olympia Beds). |
| 10 | S-5 | ☒ | 13<br>17<br>24 |  |  | Hard, green gray, silty CLAY: moist, trace to some gravel, slightly to low plastic, some fine sand, massive, homogeneous. (pre-Olympia Beds). |
| 15 | S-6 | ☒ | 10<br>20<br>26 |  |  | Grading to low plastic, slow dilatancy. |
|  |  |  |  |  |  | Bottom of Boring. |
| 20 |  |  |  |  |  |  |
| 25 |  |  |  |  |  |  |



|  |  |
|---|---|
| Completion Depth: | 16.5ft |
| Date Borehole Started: | 2/12/16 |
| Date Borehole Completed: | 2/12/16 |
| Logged By: | S. Evans |
| Drilling Company: | CN Drilling |

Remarks: Groundwater measured in open boring roughly 0.5 hours following extraction of augers. May not represent static groundwater level.

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INCORPORATED

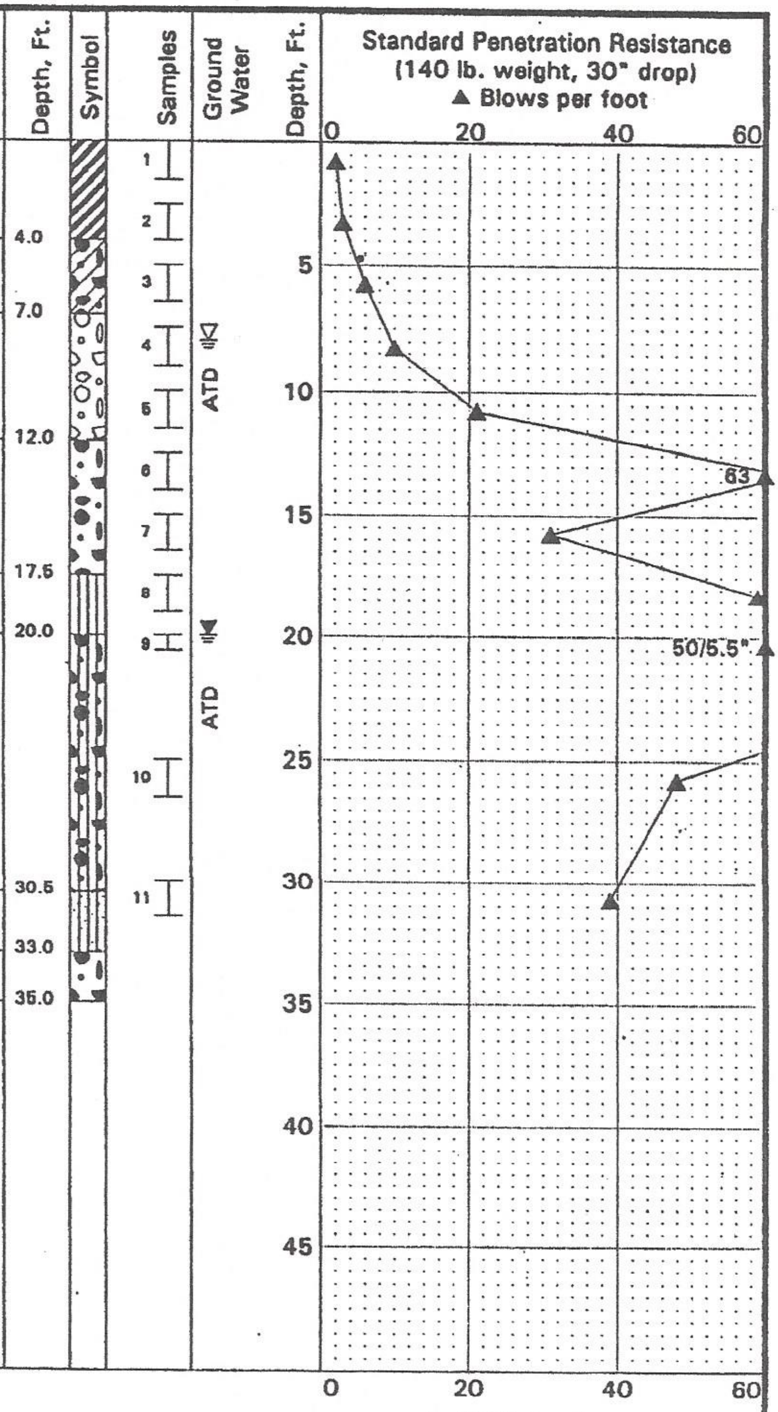
# LOG OF TEST BORING B-3

**Figure 6**

The stratification lines represent approximate boundaries. The transition may be gradual.

| MATERIAL DESCRIPTION<br>Surface Elevation: Approx. 31 Feet | Depth, Ft. |
|---|---|
| Soft, gray, slightly sandy, silty CLAY; moist; scattered gravel and organics; iron-oxide stains; slightly blocky structure; (Fill) CH. | 4.0 |
| Loose, gray, silty, sandy, clayey GRAVEL; moist; scattered wood debris, organics, and debris (brick); iron-oxide stains; (Fill) GC. | 7.0 |
| Medium dense, tan to gray, slightly clayey, slightly silty to silty, sandy GRAVEL; moist to wet; GW. | 12.0 |
| Dense to very dense, gray, clean to slightly silty SAND and GRAVEL; wet; GP/SP. | 17.5 |
| Dense, tan and brown, layered sandy SILT, silty SAND and silty, gravelly SAND; moist; ML/SM. | 20.0 |
| Dense to very dense, light brown, silty, sandy GRAVEL; wet; layers of silt and sand; GM. | 30.5 |
| Dense, tan, layered, silty SAND to sandy SILT; wet; SM/ML. | 33.0 |
| Dense, light brown, slightly silty, sandy GRAVEL; wet; GP. | 35.0 |

BOTTOM OF BORING
COMPLETED 7/2/96



● % Water Content

Plastic Limit |—●—| Liquid Limit
Natural Water Content

LEGEND

- • Sample Not Recovered
- ⌶ 2" O.D. Split Spoon Sample
- Ⅱ 3" O.D. Shelby Tube Sample
- ▨ Surface Seal
- ▩ Annular Sealant
- ▤ Piezometer Screen
- ▨ Grout
- ⊽ Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USC letter symbol based on visual classification.

8225 W. Mercer Way Short Plat
Mercer Island, Washington

## LOG OF BORING B-2 -96

October 1996
W-7358-01

**SHANNON & WILSON, INC.**
Geotechnical and Environmental Consultants

Fig. 7