GEOTECH
CONSULTANTS, INC.

2401 10th Ave E
Seattle, Washington 98102
(425) 747-5618 FAX (425) 747-8561

January 3, 2017

JN 16543

The Ladybug Trust
1420 – 5th Avenue, Suite 4200
PO Box 91302
Seattle, Washington 98111-9402

Attention: Michael Morgan
via email: morganm@lanepowell.com

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Ogden Point Residence
3675 West Mercer Way
Mercer Island, Washington

Dear Mr. Morgan:

We are pleased to present this geotechnical engineering report for the 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, and temporary shoring. This work was authorized by your acceptance of our proposal, P-9553, dated August 29, 2016.

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.

Thor Christensen, P.E.
Senior Engineer

cc: **Demetriou Architects** – Andrea Smith
via email: abs@demetriou.net

TRC/MRM:mw

GEOTECHNICAL ENGINEERING STUDY
Proposed Ogden Point Residence
3675 West Mercer Way
Mercer Island, Washington

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

The property is angled from True North. For the purposes of this report, we have designated Plan North as perpendicular to the shoreline of Lake Washington. This is depicted on the Site Exploration Plan, Plate 2.

We were provided with a topographic survey prepared by Terrane dated July 8, 2016. We were also provided with a site plan prepared by Demetriou Architects dated December 14, 2016. Based on these plans, we understand that the development will consist of a new residence with a garage to the northeast, a guest house to the northwest, and a swimming pool to the east of the main residence.. The residence will have a basement that daylights toward Lake Washington to the south. The existing house will be demolished but a detached building (a.k.a. Lighthouse) near the northwest corner of the site will remain.

The existing northern driveway will be lowered about 10 feet; it will expand a few feet toward the north and about 20 feet toward the east. The bulk of the driveway and the motorcourt around the garage will have an elevation of 42 feet. Cuts of 15 to 25 feet below the existing grade will be necessary along the driveway and motorcourt. The floor of the detached garage will be about 6 to 20 feet below the original ground surface.

The swimming pool will be located in the southeastern part of the site. Its pool deck will have an elevation of 34 feet, close to the ground surface to the north but about 7 feet above the ground surface to the south.

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

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site in Mercer Island. The site is bordered to the west by Lake Washington and is otherwise surrounded by residences.

The ground surface within the northern half of the site slopes steeply to moderately down toward Lake Washington to the southwest. There is a change in elevation of nearly 100 feet across the 200-foot-long eastern property edge. The elevation change lessens toward the west. The steep ground is covered with brush and trees. We did not observe any indications of recent instability on the steep slopes. The southern portion of the property slopes gently to the edge of Lake Washington. This portion of the site is primarily grass and landscaping. The ground surface on the lower, southern, portion of the property is wet and soft.

The site is accessed via a shared driveway that enters the property near its northwest corner. The driveway extends about 140 feet into the site. One 25-foot section of the driveway is wood and elevated a few feet above the ground surface. There is a concrete retaining wall about 4 feet high along most of the north edge of the driveway.

The south-central part of the site is developed with a three-story house with a basement that daylights toward the south. The lowest level of the south end of the structure contains an in-ground swimming pool. A garage is attached to the northwest corner of the house. The portion of the driveway directly west of the existing garage is underlain by finished living space. A small detached three-story building (a.k.a. Lighthouse) is downslope of the driveway near the northwest corner of the site.

The undeveloped northern and western parts of the site are vegetated with mature evergreen and deciduous trees and brush. Landscaping bushes and grass lawns are south and east of the residence. A rock bulkhead a few feet tall is along the edge of Lake Washington.

The Mercer Island Landslide Hazard Assessment map by Kathy Troost and Aaron Wischer dated April 2009 shows that the site has been designated as a Landslide Hazard Area. That map also shows landslides close by to the northwest and southeast. We are not aware of any recent deep-seated landslides in the area. However, we know that shallow slides within the looser soils on steep slopes have occurred in the area around this neighborhood. Additionally, the Mercer Island Seismic Hazard Assessment map by Kathy Troost and Aaron Wischer dated April 2009 shows that the site has been designated as a Seismic Hazard Area.

SUBSURFACE

The subsurface conditions were explored by drilling eight 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 December 15 and 16, 2016 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 10.

Soil Conditions

Loose, silty sand fill was encountered in Boring 3 to a depth of 3 feet. Fill soils can also be expected behind backfilled foundation and retaining walls, and along the downslope sides of existing structures.

The uppermost native soils encountered in the borings generally consisted of loose sand that contained varying amounts of silt and gravel. This soil became medium-dense in several of the borings. The sand soils were typically less than 10 feet thick.

Underlying the sands, the explorations found silt that was mostly non-plastic but included plastic zones. This silt was massive in appearance, but was typically loose to medium-dense to a depth of 10 to 15 feet below the existing ground surface. In Boring 7, the silt was loose to a depth of approximately 30 feet. Below the loose to medium-dense silt was very stiff to hard, or dense, silt that has been glacially-compressed.

Our firm has provided geotechnical engineering services for a new home under construction two lots to the west of this site, as well as for two house sites located immediately west of that new home. The soil conditions encountered on these lakeside properties have been similar to those found in the borings conducted on the Ladybug Trust site.

Groundwater Conditions

Perched groundwater seepage was observed at a depth of 7 to 25 feet in five of the borings. As noted above, the ground surface on the lower portion of the lot is also wet in areas, which could be the result of a leaky sprinkler system and/or water that is perched on top of the silty, low permeability soils.

The test 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 and are generally highest during the normally wet winter and spring months. 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 test boring logs are interpretive descriptions based on the conditions observed during drilling.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

Most of the test borings conducted for this study encountered loose to barely medium-dense soils above 10 to 15 feet, but those soils extended to 30 feet in Boring 7. Those marginal soils were followed by glacially-compressed silt. Conventional footing foundations bearing on the upper soils would experience excessive and differential settlement. As a result, we recommend that the proposed residence, guest house, and swimming pool be supported with small-diameter pipe piles driven through the upper soils and into the underlying competent soil. This is a typical foundation system for other homes constructed in the area.

Based on the conditions encountered in Boring 8, the excavation for the planned northeastern garage should reach glacially-compressed silt. As a result, it should be possible to utilize conventional foundations for this portion of the structure. Due to the moisture sensitivity of the silt soils, and the likelihood that seepage may be encountered, the excavated footing bearing surfaces should be protected with a 4- to 6-inch thickness of clean crushed rock to prevent softening under foot traffic during the placement of forms and reinforcing steel.

Small-diameter pipe piles do not have large lateral capacities. Passive resistance from compacted soil against grade beams can resist lateral loads, such as seismic and restrained earth pressures. If necessary, helical anchors installed into the glacially-compressed silt can be utilized for increased lateral resistance.

Floor slabs and other settlement-sensitive elements should be carried on piles. If the deck that will surround the pool is supported on the marginal near-surface soils it will experience differential settlement relative to the pile-supported pool. To reduce the potential for distress, the pool deck should be isolated from the pool to allow differential movement, or the pool deck could be constructed as a structural slab that is also supported with pipe piles.

We recommend installing underslab drainage and a vapor barrier below any below-grade slabs to reduce the potential for moisture to rise into finished living spaces. A typical underslab drainage detail is presented as Plate 13. The amount of seepage that will be encountered in excavations into the slope is unpredictable from the results of isolated borings. Well-constructed drainage and waterproofing should be planned to reduce the potential for post-construction seepage through these walls.

Shoring will be required to support the cuts for the north and east sides of the lowered and expanded driveway. Cantilevered soldier pile shoring should be appropriate where the excavation will be less than about 15 feet deep, but tiebacks will be necessary for the deeper excavations. We provide additional recommendations in the ***Temporary Shoring*** section. If the soldier piles are used to provide permanent retention of the cuts, they will have to be designed for the earth pressures recommended in the ***Permanent Foundation and Retaining Walls*** section.

The core of the site consists of dense native soil that has a very low potential for deep-seated landslides. However, like any steep slope in the Puget Sound region, there is a potential for shallow failures in the near-surface soils. Such failures are usually triggered by heavy rainfall and/or concentrated water flowing over the slope. The potential for failures can be reduced by maintaining vegetation on the slopes and directing water away from the slopes. The recommendations presented in this report are intended to prevent adverse impacts to the stability of the steep slopes above the site. In order to satisfy the City of Mercer Island's requirements, we make the following statement:

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

It is possible that soil and debris from any future slides on the steep slopes could travel over the top of the retaining wall located along the north side of the driveway. If desired, this wall's height could be extended 3 to 4 feet above the existing slope grade to catch or slow material that may travel down the slope in a slide. The wall would have to be designed for an increased pressure due to the potential impact load from slide debris. However, the seismic design loading can be ignored for this instance, so the overall design of the wall may not be substantially affected by the landslide loads. Design earth pressures are presented below in the ***Permanent Foundation and Retaining Walls*** section. Due to the proximity of the planned northeastern garage to the wall that will retain the cut into the base of the steep slope, we recommend that the north garage wall, and the north half of the east wall be constructed of concrete and be devoid of windows or doors. This lessens the risk to any potential occupants who may be in the garage in the event of a landslide. It is difficult to assess the pressures that this wall should be designed for, but it would be appropriate for the wall to be sufficiently reinforced to handle an active earth pressure of 100 pounds per cubic foot (pcf) acting over a 5-foot height. Siding can be extended down over the concrete walls.

The silt that underlies the site is nearly impervious, and shallow groundwater was observed in the explorations. For these reasons, stormwater infiltration is not feasible at this site.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered during the site work. The location of the site on the shore of Lake Washington will make proper erosion control implementation important to prevent adverse impacts to the lake. Exercising care and being pro-active with the maintenance and potential upgrading of the erosion control system through the entire construction process will be critical. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil to prevent accumulated water or runoff from the work area from becoming silty in the first place. 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. Silty water cannot be discharged to the lake. Seepage into the planned excavations should be expected. If possible, arrangement should be made with the City of Mercer Island to discharge accumulated water into the sanitary sewer after it has been sufficiently clarified by settling in a temporary pond or tank. Alternatively, water would have to be held in such a temporary facility until it is clean enough to discharge to the lake. It will also be important to cap any existing drain lines found running toward the lake until excavation is completed. This will reduce the potential for silty water finding an old pipe and flowing into the lake. Covering the base of the excavation with a layer of clean gravel or rock is also prudent to reduce the amount of mud and silty water generated. Utilities reaching between the house and the lake should not be installed during rainy weather, and any disturbed area caused by the utility installation should be minimized by using small equipment. Soil stockpiles should be minimized. All excavations should be kept lower than the surrounding grade, or sloped away from the street and adjacent properties. This prevents any silty water from the excavation from flowing off the site. If silty water accumulates in the excavation, it would likely have to be pumped to a temporary holding tank (i.e. Baker tank) before being disposed of properly. Discharging silty runoff to a nearby ditch or storm drain is not acceptable. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Extra straw, coarse mulch or hog fuel, and plastic sheeting should be stockpiled at the site for immediate use in the event of erosion control problems. 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.

Wet weather construction on this site should be possible without adverse impacts to the surrounding properties. In preventing erosion control problems on any site, it is most important that any disturbed soil areas be immediately protected. This requires diligence and frequent communication on the part of the general contractor and earthwork subcontractor. As with all construction projects undertaken during potentially wet conditions, it is important that the contractor's on-site personnel are familiar with erosion control measures and that they monitor their performance on a regular basis. It is also appropriate for them to take immediate action to correct any erosion control problems that may develop, without waiting for input from the geotechnical engineer or representatives of the City.

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

> In our judgment, the development practices that we have recommended in this report should render the anticipated new construction as safe with regards to the erosion hazard as if it were not located in a geologic hazard area.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking 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 Site Class). The glacially-compressed site soils that will support the foundations are not susceptible to seismic liquefaction under the Maximum Considered Earthquake (MCE) because of their dense nature.

PIPE PILES

Three-, 4-, or 6-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
6 inches	n/a	n/a	10 sec/inch	20 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 300 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. The recommended passive resistance does not include a safety factor.

Due to their small diameter, the lateral capacity of vertical pipe piles is relatively small. However, if some additional lateral resistance in addition to passive soil resistance is required, driving battered piles in the same direction as the applied lateral load can help. The lateral capacity of a battered pile is equal to one-half of the lateral component of the allowable compressive load, with a maximum allowable lateral capacity of 1,000 pounds for 3- and 4-inch piles and 2,000 pounds for 6-inch piles. The allowable vertical capacity of battered piles does not need to be reduced if the piles are battered steeper than 1:5 (Horizontal:Vertical).

CONVENTIONAL FOUNDATIONS

The proposed northeastern garage can be supported on conventional continuous and spread footings bearing on undisturbed, medium-dense, native soil. We recommend that continuous and individual spread footings have minimum widths of 16 and 24 inches, respectively. Exterior footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface for protection against frost and erosion. The local building codes should be reviewed to determine if different footing widths or embedment depths are required. Footing subgrades must be cleaned of loose or disturbed soil prior to pouring concrete. Depending upon site and equipment constraints, this may require removing the disturbed soil by hand.

An allowable bearing pressure of 2,500 pounds per square foot (psf) is appropriate for footings supported on competent native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be about one-inch, with differential settlements on the order of one-half-inch in a distance of 30 feet along a continuous footing with a uniform load.

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

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

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

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

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 backfill:

PARAMETER	VALUE
Active Earth Pressure *	40 pcf (level backslope) 60 pcf (below steep north slope)
Passive Earth Pressure	300 pcf
Coefficient of Friction	0.40
Soil Unit Weight	130 pcf

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

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

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.

If a slide catchment extension is added on top of the northern driveway retaining wall, this section of wall should be designed for an active equivalent fluid density of 100 pcf to model the potential impact load from slide debris.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired. The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Frictional resistance should not be considered for pile-supported walls. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The onsite soils generally have too high of silt and moisture contents to be reused as wall backfill. They are not free-draining.

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 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. 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 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 build up of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

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

HELICAL ANCHORS

Helical anchors consist of single or multiple helixes that are rotated into the ground on the end of round or square metal shafts. These anchors can be used to support both compression and tension loads, but the lateral capacity of vertical anchors is negligible due to the relatively small diameter of the metal shafts. The design capacity of single helix anchors is the allowable soil bearing capacity on the helix area. Multiple-helix anchors are typically assumed to have a design capacity equal to the sum of the allowable bearing capacity on each helix, if they are separated more than three helix diameters. Buckling of the shaft may limit the design load, and we recommend that the shaft be analyzed for buckling assuming no lateral soil support, as the installation of the anchor will disturb the soil in the vicinity of the shaft.

The minimum diameter of a single helix anchor is 8 inches. The ultimate capacity of the anchor in tension or compression can be estimated roughly by multiplying the installation torque by 10. We recommend that the helix be installed at least 5 feet into competent native soil. A typical anchor capacity for small to mid-size anchors in the site soils is 15 to 20 kips, but multiple helices may be needed on the anchors to achieve these capacities.

The anchors should be installed by a specialty contractor familiar with design and installation of chance systems. The contractor can assist with refining the anchor design and details and estimating capacities for different soil and anchor conditions. At least one anchor should be load tested to at least 200 percent of the design load to verify the allowable capacity.

FLOOR SLABS

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer. This capillary break/drainage layer is not necessary if an underslab drainage system is installed. The ***General*** section should be reviewed for recommendations related to underslab drainage.

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 also notes that vapor *retarders* such as 6-mil plastic sheeting have been used in the past, but are now recommending a minimum 10-mil thickness for better durability and long term performance. 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.

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

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. 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. No vertical cuts should be planned for the on-site soils where workers will have to enter an excavation. Also, cuts at the base of the steep northern slope, such as those for the driveway, should be shored over their full height.

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

All permanent cuts into native soil should be inclined no steeper than 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 steep slopes 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

Cantilevered and tied-back soldier pile systems have proven to be an efficient and economical method for providing excavation shoring. Tied-back walls are typically more economical than cantilevered walls where the depth of excavation is greater than 15 feet.

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. The contractor should be prepared to case the holes or use the slurry method if caving soil or heavy seepage 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 clean crushed rock, 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.

If permanent building walls are to be constructed against the shoring walls, drainage should be provided by attaching a geotextile drainage composite with a solid plastic backing, similar to Miradrain 6000, to the entire face of the lagging, prior to placing waterproofing and pouring the foundation wall. These drainage composites should be hydraulically connected to the foundation drainage system through weep holes placed in the foundation walls.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered or restrained by one row of tiebacks, 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). Below the steep northern slope this pressure should increase to 55 pcf. If the soldier piles with permanently retain soil, they should be designed for the earth pressures given abofee in ***Permanent Foundation and Retaining Walls***.

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. 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 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." Tied-back soldier piles should be embedded no less than 10 feet below the lowest point of the excavation, including footing and utility excavations.

The vertical capacity of soldier piles will be developed by frictional shaft resistance along the embedded length and pile end-bearing.

PARAMETER	*DESIGN VALUE*
Pile Shaft Friction	750 psf

Where: psf is Pounds per Square Foot.

The above values assume that the excavation is level in front of the soldier pile and that the bottom of the pile is embedded a minimum of 10 feet below the floor of the excavation. The concrete surrounding the embedded portion of the pile must have sufficient bond and strength to transfer the vertical load from the steel section through the concrete into the soil.

Drilled and Grouted Tieback Anchors

We recommend installing tieback anchors at inclinations between 20 and 30 degrees below horizontal. If drilled and grouted tieback anchors are used instead of helical anchors, the tieback will derive its capacity from the soil-grout strength developed in the soil behind the no-load zone. The minimum grouted anchor length should be 10 feet. The no-load zone is the area behind which the entire length of each tieback anchor should be located. To prevent excessive loss-of-ground in a drilled hole, the no-load section of the drilled tieback hole should be backfilled with a sand and fly ash slurry, after protecting the anchor with a bond breaker, such as plastic casing, to prevent loads from being transferred to the soil in the no-load zone. The no-load section could be filled with grout after anchor testing is completed.

During the design process, the possible presence of foundations or utilities close to the shoring wall must be evaluated to determine if they will affect the configuration and length of the tiebacks.

Based on the results of our analyses and our experience at other construction sites, we suggest using an adhesion value of 900 psf to design temporary anchors, if the mid-point of the grouted portion of the anchor is more than 10 feet below the overlying ground surface. An allowable adhesion value of 750 psf should be used for permanent anchors. This value applies to non-pressure-grouted anchors. Pressure-grouted or post-grouted anchors can often develop adhesion values that are two to three times higher than that for non-pressure-grouted anchors. These higher adhesion values must be verified by load testing.

Soil conditions, soil-grout adhesion strengths, and installation techniques typically vary over any site. This sometimes results in adhesion values that are lower than anticipated. Therefore, we recommend substantiating the anchor design values by load-testing all tieback anchors. At least two anchors in each soil type encountered should be performance-tested to 200 percent of the design anchor load to evaluate possible anchor creep. Wherever possible, the no-load section of these tiebacks should not be grouted until the performance tests are completed. Unfavorable results from these performance tests could require increasing the lengths of the tiebacks. The remaining anchors should be proof-tested to at least 135 percent of their design value before being "locked off." After testing, each anchor should be locked off at a prestress load of 80 to 100 percent of its design load.

If caving or water-bearing soil is encountered, the installation of tieback anchors will be hampered by caving and soil flowing into the holes. It will be necessary to case the holes, if such conditions are encountered. Alternatively, the use of a hollow-stem auger with grout pumped through the stem as the auger is withdrawn would be satisfactory, provided that the injection pressure and grout volumes pumped are carefully monitored.

All drilled installations should be grouted and backfilled immediately after drilling. No drilled holes should be left open overnight.

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. As a result, the shoring walls should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least every fourth soldier pile should be monitored by taking readings at the top of the pile. 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 excavation in front of the shoring wall.

DRAINAGE CONSIDERATIONS

We anticipate that permanent foundation walls will 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.

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). At its highest point, a perforated pipe invert should be at least 6 inches below the level of a crawl space or the bottom of a floor slab, and it should be sloped slightly for drainage. Plate 11 presents typical considerations for footing drains and Plate 12 presents a typical shoring drain detail. All roof and surface water drains must be kept separate from the foundation drain system.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing even a few inches of free draining gravel underneath the vapor retarder limits 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 a building should slope away at least 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. Water from roof, storm water, and foundation drains should not be discharged onto slopes; it should be tightlined to a suitable outfall located away from any slopes.

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, behind permanent retaining or foundation walls, 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. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can 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 relative compactions for structural fill:

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

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

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

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test borings are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

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

This report has been prepared for the exclusive use of The Ladybug Trust and its 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 - 10	Test Boring Logs
Plate 11	Typical Footing Drain Detail
Plate 12	Typical Shoring Drain Detail
Plate 13	Typical Underslab Drainage 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.

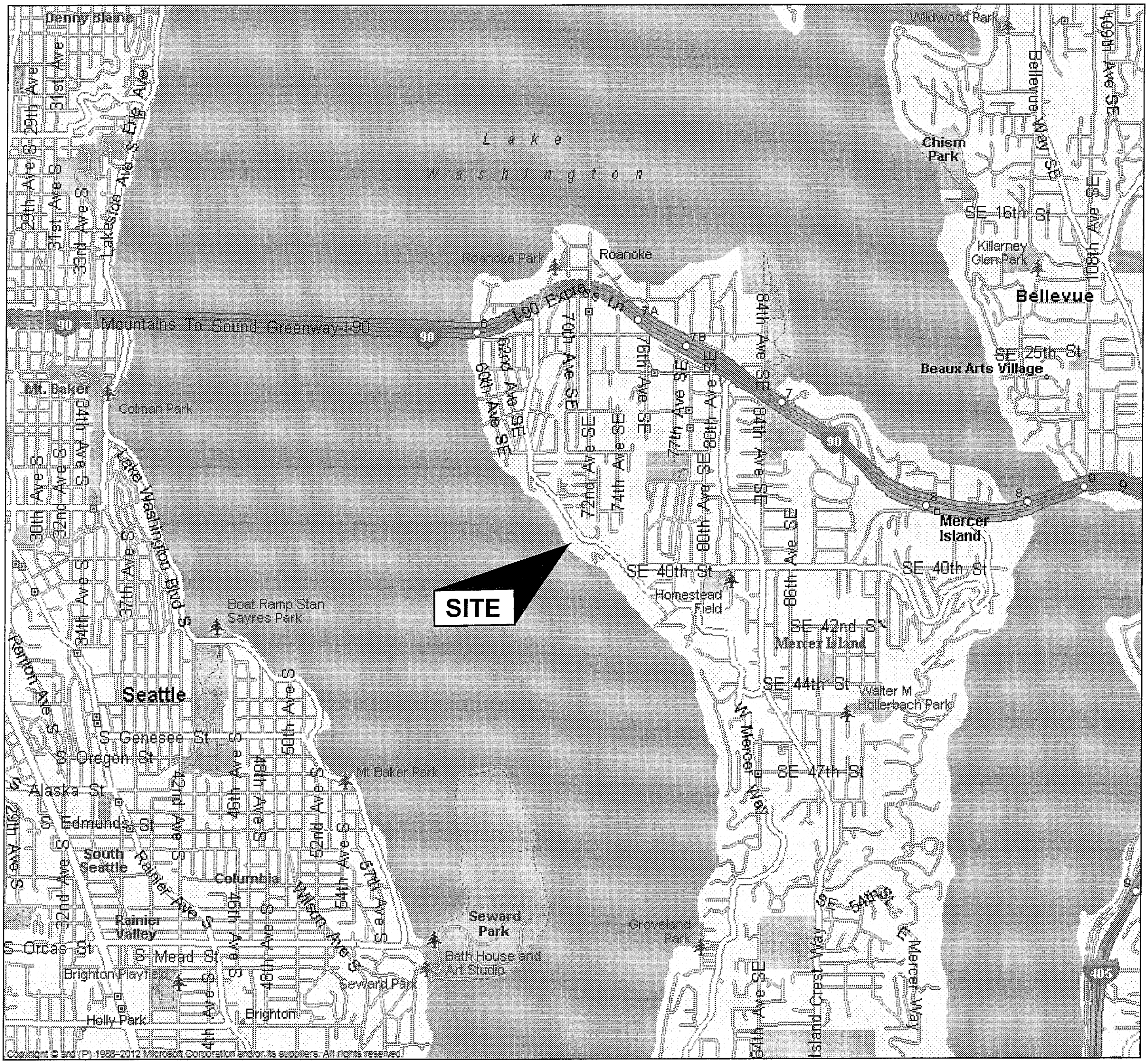
MARC R. McGINNIS — STATE OF WASHINGTON — 27845 — REGISTERED PROFESSIONAL ENGINEER

1/3/17

Thor Christensen, P.E.
Senior Engineer

Marc R. McGinnis, P.E.
Principal

NORTH



(Source: Microsoft MapPoint, 2013)

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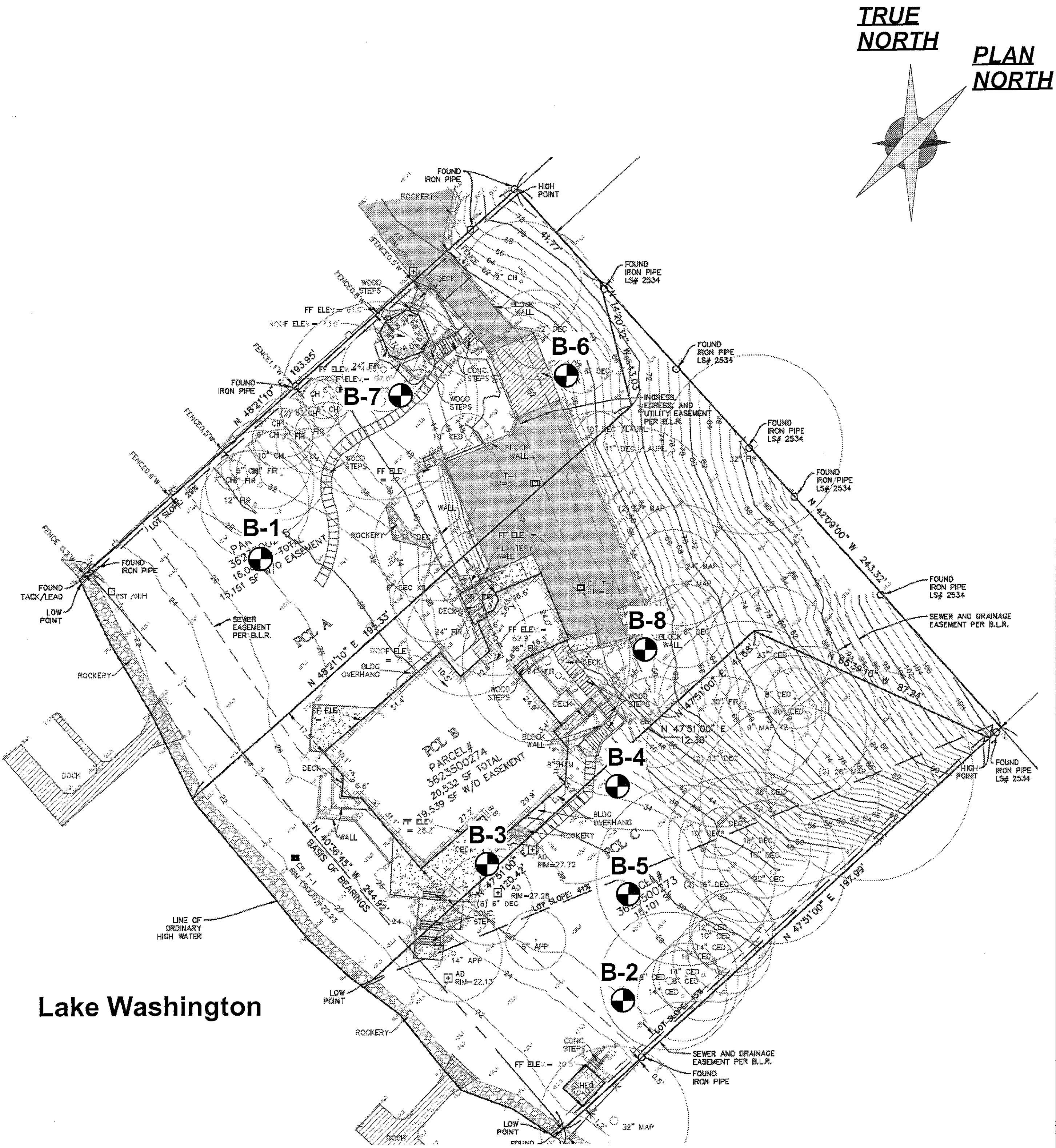
VICINITY MAP

3675 West Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16543	Dec. 2016		1



Legend:

Test Boring Location

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SITE EXPLORATION PLAN

3675 West Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16543	Dec. 2016	No Scale	2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SW	Grass and topsoil over; Rust-brown mottled brown SAND with silt, fine to coarse-grained, moist, loose
			6	1		
5			8	2		-with gravel, becomes gray-brown
		▼				
			14	3	SM	Gray-brown silty SAND, fine-grained, moist, medium-dense -with gravel, becomes fine to coarse-grained
10			10	4	ML	Gray-brown SILT, non-plastic, moist, medium-dense
15			13	5		-becomes stiff, plastic, and gray
20			18	6		-becomes very stiff
25			35	7		-becomes dense and non-plastic
30						
35						

*** Test boring was terminated at 26.5 feet on December 15, 2016.**
*** Perched groundwater was encountered at 7 feet during drilling.**

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TEST BORING LOG
3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SW	Grass and topsoil over; Brown SAND with gravel, fine to coarse-grained, moist, very loose
			3	1		
5						-becomes loose and wet
			5	2	ML	Gray-brown SILT, non-plastic, moist, loose
			9	3		
10			14	4		-becomes medium-dense
15			17	5		-becomes gray
20			30	6		-becomes dense
25						

*** Test boring was terminated at 21.5 feet on December 15, 2016.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Grass and topsoil over; Gray silty SAND with gravel, fine to coarse-grained, moist, loose **(FILL)**
			4	1	SW	Brown SAND, fine to coarse-grained, moist, loose
5			4	2		-increased silt content, becomes gray-brown
			15	3		-becomes medium-dense, reduced coarse sand -with a 5 inch silty layer
10			31	4	ML	Gray-brown SILT, non-plastic, moist, dense
15			18	5		-becomes very stiff and plastic
20			19	6		
25			32	7		-becomes dense and non-plastic
30						
35						

*** Test boring was terminated at 26.5 feet on December 15, 2016.**
*** Some groundwater was encountered below 15 feet during drilling, likely from seams of sand.**

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TEST BORING LOG

3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	5

BORING 4

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SW	Topsoil over; Brown SAND with gravel, fine to coarse-grained, moist, loose to medium-dense
			13	1		-becomes medium-dense
5			16	2	ML	Gray-brown SILT, non-plastic, moist, medium-dense
			19	3		-becomes rust-brown mottled
10			11	4		-increased fine-grained sand content
15			15	5		
20			19	6		
25			20	7		-becomes very stiff and plastic, reduced sand content
30						
35						

*** Test boring was terminated at 26.5 feet on December 15, 2016.**

*** Some groundwater was encountered below 20 feet during drilling, likely from seams of sand.**

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TEST BORING LOG

3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	6

BORING 5

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Topsoil over; Gray-brown silty SAND, fine-grained, moist, medium-dense
			19	1	SW	Brown SAND, fine to coarse-grained, moist, medium-dense
5			15	2		-becomes wet
			25	3	ML	Gray-brown clayey SILT, slightly plastic, moist, very stiff
10			22	4		-becomes medium-dense and non-plastic, with fine to medium-grained sand
15			21	5		-becomes very stiff and plastic, reduced sand content
20			22	6		-becomes medium-dense and non-plastic
25						

* **Test boring was terminated at 26.5 feet on December 15, 2016.**
* **Groundwater was encountered at approximately 12 feet during drilling.**

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TEST BORING LOG
3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	7

BORING 6

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Topsoil over; Gray-brown silty SAND, fine to coarse-grained, moist to wet, loose
			8	1		
5			19	2		-becomes medium-dense and moist
			13	3	ML	Gray-brown SILT, non-plastic, moist, medium-dense -with a one-inch layer of fine to coarse-grained sand
10			24	4		-becomes gray-brown to gray
15			31	5		-becomes dense and gray
20			28	6		-becomes medium-dense
25			35	7		-becomes dense
30						
35						

*** Test boring was terminated at 26.5 feet on December 16, 2016.**
*** Groundwater was not encountered during drilling.**

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TEST BORING LOG

3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	8

BORING 7

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Topsoil over; Gray-brown silty SAND, fine to medium-grained, moist, loose
5			7	1		
			24	2		-becomes medium-dense and rust-brown mottled gray-brown
10					SW	Gray-brown SAND with gravel, fine to coarse-grained, wet, medium-dense
			11	3	ML	Gray-brown SILT, non-plastic, wet, medium-dense
15			13	4		-becomes stiff, plastic, and moist
20			15	5		-becomes medium-dense and non-plastic, with fine-grained sand
25		▼	11	6		-becomes stiff and slightly plastic, reduced sand content
30			9	7		-becomes loose and non-plastic, with fine-grained sand
35			31	8		-becomes hard and plastic

* **Test boring was terminated at 36.5 feet on December 16, 2016.**
* **Groundwater was encountered at 25 feet during drilling.**

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TEST BORING LOG

3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	9

BORING 8

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Topsoil over; Brown silty SAND, fine to medium-grained, moist, loose
			3	1		
5			15	2		-becomes medium-dense
			14	3	SW	Gray-brown SAND, fine to coarse-grained, moist, medium-dense
10					SM	Gray-brown silty SAND, fine to coarse-grained, moist, medium-dense
			23	4	ML	Gray-brown clayey SILT, plastic, moist, very stiff
15			37	5	SM	Gray-brown silty SAND with gravel, fine to coarse-grained, moist, dense
20			59	6	ML	Gray-brown sandy SILT, non-plastic, fine-grained, moist, very dense
25			41	7		-becomes dense and gray
30						
35						

*** Test boring was terminated at 26.5 feet on December 16, 2016.**
*** No groundwater was encountered during drilling.**

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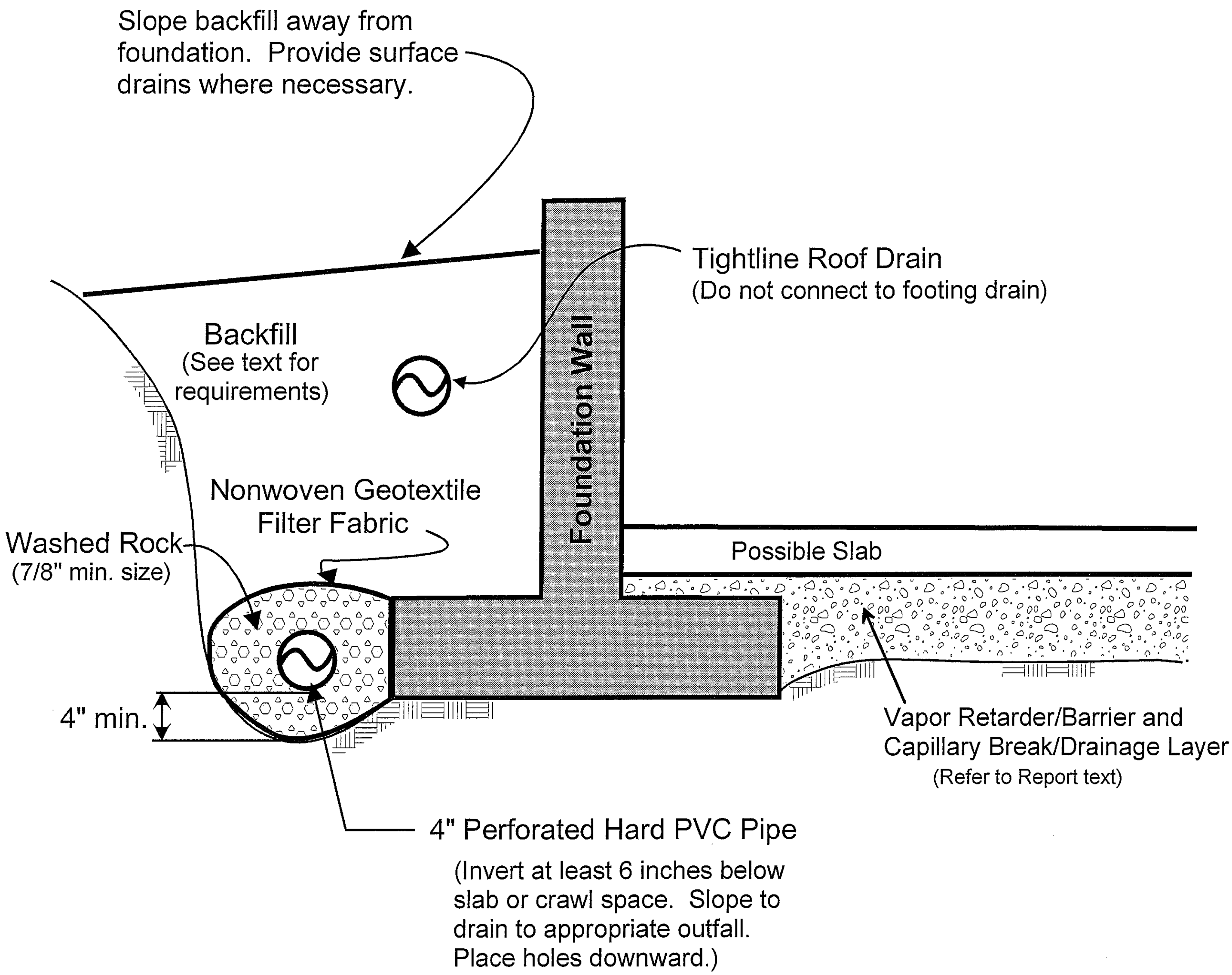
TEST BORING LOG

3675 West Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16543	Dec. 2016	TRC	10



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

3675 West Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
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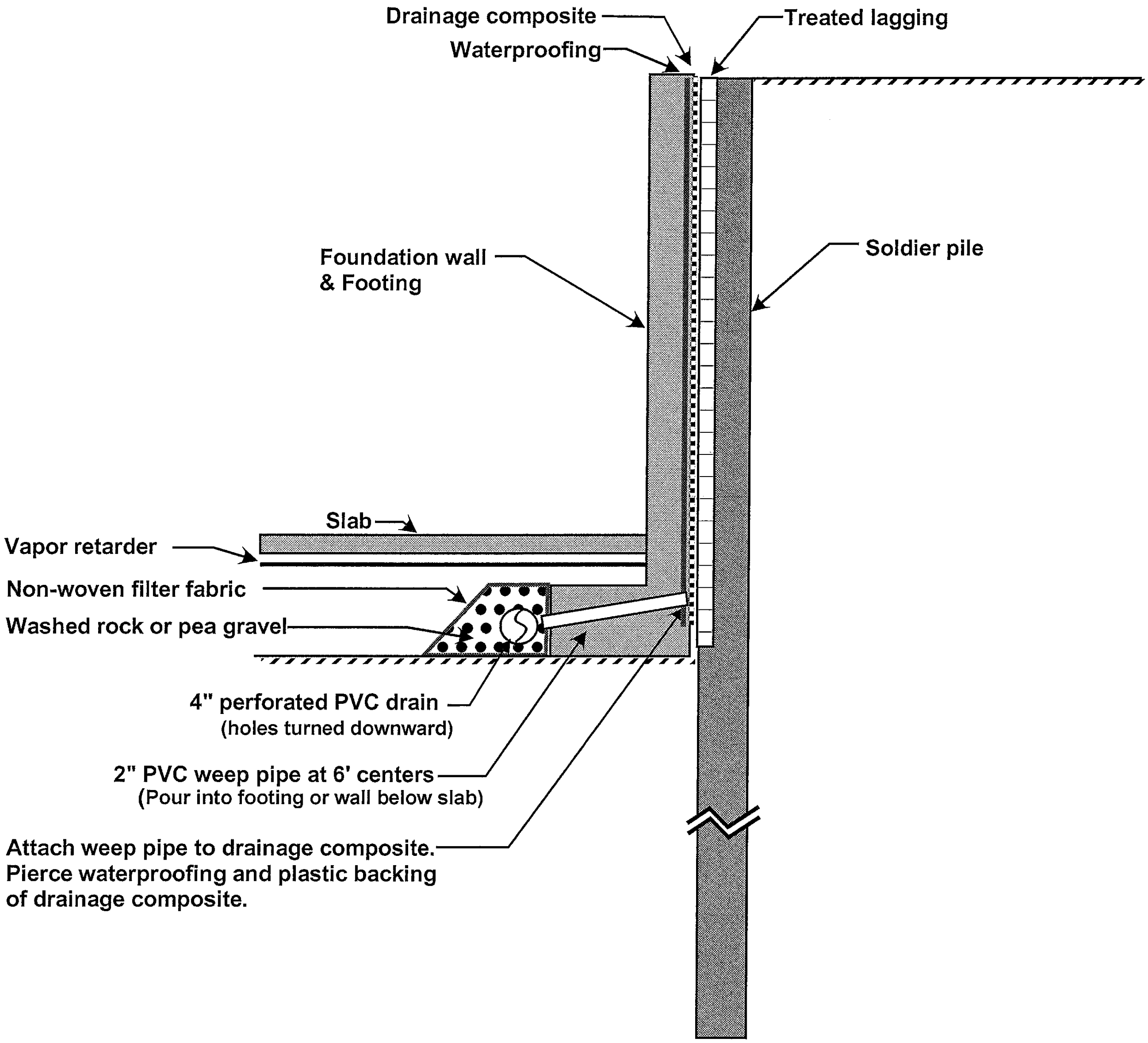

Note - Refer to the report for additional considerations related to drainage and waterproofing.

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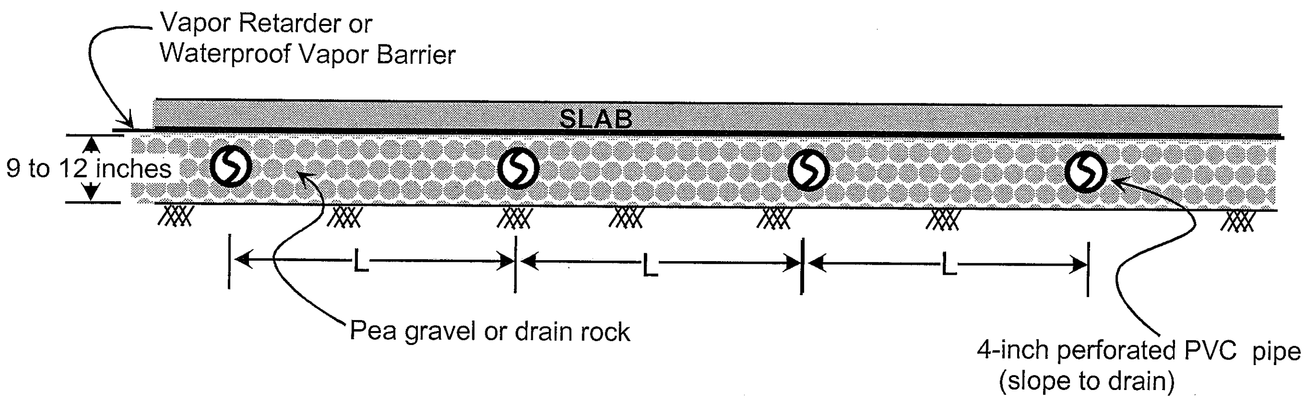
SHORING DRAIN DETAIL

3675 West Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16543	Dec. 2016		12



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

3675 West Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16543	Dec. 2016		13