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May 2, 2016

JN 16095

Derek Lee Cheshire
7615 East Mercer Way
Mercer Island, Washington 98040

via email: derek.cheshire@microsoft.com

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Residence Addition and DADU
7615 East Mercer Way
Mercer Island, Washington

Dear Mr. Cheshire:

We are pleased to present this geotechnical engineering report for the addition to the existing single-family residence and the detached accessory dwelling unit (DADU) to be constructed in Mercer Island, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for foundations and retaining walls. This work was authorized by your acceptance of our proposal, P-9398, dated February 19, 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: **Russell Architecture** – Teresa Russell
via email: teresarussell@gmail.com

TRC/DRW:mc

GEOTECHNICAL ENGINEERING STUDY
Proposed Residence Addition and DADU
7615 East Mercer Way
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed residence addition and detached accessory dwelling unit (DADU) to be located in Mercer Island.

We were provided with a topographic survey prepared by GeoDimensions dated January 26, 2016. We were also provided with a site plan prepared by Russell Architecture dated March 9, 2016. Based on this information and conversations with the project architect, we understand that the development will consist of two different structures. One is an addition to the south side of the existing residence. The addition would have one-story and a basement that daylights toward the west, matching the residence. The basement will have a floor elevation of 143 feet, which will require an excavation of up to about 7 feet. The southeast corner of the addition will be set back 5 feet from the southern property line, but the remainder of the addition will have a greater setback.

The second structure would be a DADU will be constructed about 100 feet northeast of the residence. The lowest level of the DADU would be a garage whose door in on its southern, downslope side. The basement slab will have an elevation of 129 feet. An excavation of about 18 feet will be required at the northwest corner of the DADU, and the excavation depth reduces toward the southeast. The DADU will be set back 10 feet from the northern property line.

We also understand that a residence may someday be constructed in the eastern portion of the property, but that would not be a part of the current project.

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 trapezoid-shaped site in Mercer Island. The site is long in the east-west direction, with an average length of about 600 feet. It is is bordered to the southeast by East Mercer Way and approximately the eastern 150 feet of the north property line is bordered by Southeast 76th Street, otherwise the property is surrounded by residences.

The terrain within the site generally slopes down toward the east. In the western 40 to 120 feet of the property, the ground has an inclination close to 2:1 (H:V), and then abruptly steepens to an inclination of 1:1 to 0.75:1. This steeper area has a height of 10 to 60 feet. The 100 feet of terrain east of this very steep slope has an inclination that starts close to 2:1 and flattens to about 3:1. The western slope is vegetated with trees and brush. A stream emerges from the base of the steep western slope and winds across the eastern portion of the site.

A bench about 100 feet wide is at the base of the western slope, near the center of the site. The western part of this bench is utilized as a yard and the existing residence is close to the southeast corner of this bench. The residence has one story and a basement that daylights toward the west. A tall, very-large-diameter redwood tree is located close to the southeast corner of the residence; it has a straight trunk and no bows or kinks that would indicate lateral movement. We did not observe cracks or indications of settlement-related distress in the residence foundations or the residence structure.

About 10 feet east of the residence the ground declines 20 vertical feet at an inclination of 1.5:1. Several mature evergreen trees grow on this slope, and exhibit mostly straight to only slightly bowed trunks. This indicates that the slope has been stable during the lifetime of the trees. This slope does not extend beyond the north edge of the residence, at that location it turns toward the east and flattens in inclination. One hundred and twenty feet east of the residence it has an inclination of 2:1 and 200 feet east of the residence it has an inclination of 3:1.

The north side of the central bench extends about 100 feet to the east of the residence footprint. Several mature evergreen trees grow in this northeastern part of the bench and most of those have severely bowed trunks and/or do not grow vertically, indicating recent soil movement. East of this northeast bench the ground slopes down toward the eastern corner of the property with an inclination close to 3:1. A gravel driveway accesses the site residence from Southeast 76th Street. The site stream passes below the driveway in a culvert.

There is a steep cut about 6 feet tall along the east edge of the site, and a ditch at the base of the cut. It is apparent that this cut was made for the adjacent East Mercer Way.

Landslide History

The property is located in an area that is known to have been affected by a large, ancient landslide that may have occurred soon after the last glaciers receded about 13,000 years ago. This ancient movement occurred as a large slump, resulting in a very steep headscarp that crosses the western part of the site. The majority of the soil that slumped during the landslide moved into what is now 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 that the site has a slope inclination of 15 percent or steeper, and that several landslides have been identified within and close to the site. 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. Both of these maps show that the site is underlain by landslide and mass wastage deposits.

SUBSURFACE

The subsurface conditions were explored by drilling five 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 March 31 and April 5, 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

Test Borings 1 and 2 were located near the planned residence addition. Test Boring 2, at the south side of the addition footprint, revealed medium-dense silty sand at 2.5 feet. Below that material, and close to the ground surface at Test Boring 1 close to the top of the adjacent steep slope, the explorations encountered silt that was initially medium-stiff to stiff and became medium-dense or stiff at 5 to 7.5 feet. The silt was not disturbed and continued to the base of the borings at 11.5 and 26.5 feet.

The conditions near the proposed DADU were explored with Test Borings 3 and 4 and were significantly different from the soils revealed in the area of the proposed addition. The ground surface at Test Boring 3 was about 12 feet higher in elevation than Test Boring 4. The upper soil in Test Boring 3 was loose silty sand with gravel that had a thickness of about 9 feet. Below that material, and close to the surface at Test Boring 4, we observed 8 to 14 feet of medium-dense sand with gravel. In Test Boring 4 this was followed by about 5 feet of medium-dense silty sand with gravel. The sand with gravel and silty sand with gravel were underlain by silt that was medium-dense to medium-stiff and jumbled because of past slope movement (ancient landslide soil). The jumbled silt continued to 40.5 feet in Test Boring 3, and to about 33 feet in Test Boring 4. That silt contained an approximately 8-foot layer of wet sand with silt in Test Boring 4. The ancient landslide soil was underlain by layers of very stiff clayey silt and medium-dense sand with silt that extended to 41.5 and 51.5 feet.

Test Boring 5, located in the lower, eastern part of the property, was somewhat similar to Test Borings 3 and 4, with loose to medium-dense silty sand with gravel underlain by loose sand to about 23 feet. Below the sand, the test boring revealed medium-stiff, ancient landslide silt to a depth of about 29 feet. The next soil layer was intact silt that was loose to stiff at 30 feet, stiff from 35 to 40 feet, and very stiff at 45 feet.

Groundwater Conditions

Groundwater seepage was observed at a depth of 7.5 and 10 feet in Test Borings 3 and 5. In Test Boring 4 we observed artesian water that rose to approximately 15 feet below the ground surface. 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 is generally highest during the normally wet winter and spring months.

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.

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 site soils have a very low potential for seismic liquefaction because of their clayey or stiff nature, and/or the lack of saturated sandy 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 0.56g, respectively. The IBC states that a site-specific seismic study need not be performed provided that the peak ground acceleration be equal to $S_{DS}/2.5$, where S_{DS} is determined in ASCE 7. It is noted that S_{DS} is equal to $2/3S_{MS}$. S_{MS} equals F_a times S_S, where F_a is determined in Table 11.4-1. For our site, $F_a = 1.0$. The calculated peak ground acceleration that we utilized for the seismic-related parameters (earth pressures and seismic surcharges) of this report equals 0.39g.

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 at the proposed residence addition location encountered medium-stiff to medium-dense native soil that increased in relative density with depth. We have not observed indications of recent slope movement at the proposed residence addition footprint; in fact the mature straight evergreen trees just east of the residence indicate no significant slope movement has occurred in at least the last 50 years. In addition, the subsurface soils in our two test borings there were not disturbed. Furthermore, the existing residence foundation does not exhibit distress due to settlement or slope movement. For these reasons, it is our opinion that the addition can be supported with conventional footing foundations that bear on the medium-dense/medium-stiff native soils.

The soils that underlie the proposed DADU are disturbed to depths of about 34 to 41 feet, and evergreen trees close to the DADU are bowed. We believe that the very near-surface soils in that area have recently experienced slow creep-type movement (evidenced by bowing trees, etc), and we also believe based on the test borings and geologic information that an ancient landslide has occurred in that area of the DADU to a depth of about 34 to 41 feet. Research has shown that the ancient landslide likely occurred approximately 1100 to 1200 years ago with a high magnitude. More typically, smaller earthquakes with less magnitude or a different location than the one that occurred 1100 to 1200 years ago are likely in the Puget Sound region (such as the ones that have occurred in 1949, 1965, and 2001). The seismic considerations noted earlier regarding the IBC address the smaller earthquakes, and our recommendations and conclusions regarding the construction of the garage is with respect to the these considerations. As noted in the following paragraphs and other sections of this report, we recommend that the DADU be supported on piles that embed into the competent soil below about 34 to 41 feet. The use of such piles will prevent catastrophic settlement or movement of the foundations, and the safety of the occupants should be protected. The intent is not to prevent any damage to the foundation or ensure continued function of the structure if movement of the ancient landslide soil were to again occur. Thus, we believe that the DADU will be "safe" if the recommendations in this report are followed as discussed in the statement of risk that is included later in this section of the report.

Conventional footings are not appropriate for the DADU because of the potential for movement of the very upper soils and possibly the ancient landslide soils, and because of the marginal bearing capability of the upper soils. Therefore, we recommend that the DADU be supported with driven pipe piles or drilled concrete piles that are embedded into competent soil below these soils. There is a need for excavation shoring to provide lateral support of the residence that includes the used of drilled concrete piles. The concrete piles could be used to not only provide lateral support as part of the shoring, but also provide vertical support. Driven steel piles that are embedded well into the competent lower soil could also be used for vertical support of the garage. Recommendations regarding shoring piles and driven steel piles (pipe piles) are provided in later sections of this report. Piles will likely need to extend 50 to 60 feet below the ground surface. Regardless of which pile system is used, all piles should be connected via concrete grade beams, would maintain rigidity of the structure's foundation.

The site stream is located about 60 feet west of the DADU footprint, the lowest floor of that structure will be 10 feet below the elevation of the nearby part of the stream, and groundwater was encountered in the western DADU exploration (Test Boring 3) well above the base of the excavation. It appears that dewatering will be needed during construction to reduce the amount of water that enters the excavation.

The relatively shallow groundwater level creates several issues for both temporary and permanent conditions at the DADU. Well-constructed drainage and waterproofing will be important beneath and around the planned building. In addition to waterproofing and drainage measures placed against foundation walls, we recommend installing underslab drainage and a vapor barrier below the basement slab to reduce the potential for moisture to rise into finished living spaces. A typical underslab drainage detail is presented as Plate 13.

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

> In our judgment, provided the recommendations in this study are followed, the development will be designed so that the risk to the lot and adjacent properties is mitigated such that the site is determined to be safe.

If the site driveway is to be widened it should be toward the north, where the ground surface is generally higher than the driveway. The ground south of the driveway should not be raised by fill placement; that would decrease stability of slope and could contribute to slope failures.

We understand that potentially a new residence may be constructed on the lower reaches of the property in the future. Such a residence would be subject to the same recommendations of the proposed DADU because similar soils were encountered in the lower reaches of the site as the area of the DADU.

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. Wherever possible, the access roads should follow the alignment of planned pavements. 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/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.

CONVENTIONAL FOUNDATIONS FOR THE RESIDENCE ADDITION

The proposed residence addition 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 2,000 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.

SHORING AND DRILLED CONCRETE PILES

We have just considered a cantilevered soldier pile shoring system for this project because it has proven to be an efficient and economical method for providing excavation shoring where the depth of excavation is less than about 18 feet. For temporary shoring, it is common to include a safety factor of 1.2 in the design. If tie-back anchors are needed, we could provide information for those at a later date.

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. Normally, the drilling method for shoring is to constructed and “open hole”. Because wet sand was encountered in our test borings near the DADU, the contractor should be prepared to case open holes or use the slurry method due to the strong likelihood of caving of the drilled hole. Alternatively, an augercast method of drilled piles could be used. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 35 pounds per cubic foot (pcf). An extra 5 pcf should be added if the shoring is permanent.

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 300 pcf. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

The vertical capacity of soldier piles to carry the downward component of the tieback forces will be developed by a combination of frictional shaft resistance along the embedded length and pile end-bearing. The embedded length is only considered in the competent, non-landslide soil that is approximately 40 feet below the ground surface.

PARAMETER	DESIGN
Pile Shaft Friction	1,000 psf
Pile End-Bearing	10,000 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. For the pile end-bearing to be appropriate, the bottom of the drilled holes must be cleaned of loosened soil. The shoring contractor should be made aware of this, as it may affect their installation procedures. 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.

PIPE PILES

Six-inch-diameter pipe piles driven with a 650- or 800- or 1,100-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (1,100-pound hammer)	FINAL DRIVING RATE (2,700-pound hammer)	FINAL DRIVING RATE (3,000-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
6 inches	20 sec/inch	10 sec/inch	6 sec/inch	15 tons

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

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

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using a passive earth pressure of 250 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate passive value.

If lateral resistance from fill placed against the foundations is required for this project, the structural engineer should indicate this requirement on the plans for the general and earthwork contractor's information. Compacted fill placed against the foundations can consist of imported soil that is tamped into place using the backhoe or is compacted using a jumping jack compactor. It is necessary for the fill to be compacted to a firm condition, but it does not need to reach even 90 percent relative compaction to develop the passive resistance recommended above. Due to their small diameter, the lateral capacity of vertical pipe piles is relatively small. However, if lateral resistance in addition to passive soil resistance is required, we recommend driving battered piles in the same direction as the applied lateral load. 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. The allowable vertical capacity of battered piles does not need to be reduced if the piles are battered steeper than 1:5 (Horizontal: Vertical).

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 *	40 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction**	0.35
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 is only suitable for the addition where footings can be used.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired. The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. No frictional resistance should be used for pile-supported structures. 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 9**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 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.

SLABS-ON-GRADE

The building floors of the addition can be constructed as slabs-on-grade atop competent native soil, or on structural fill. However, a slab-on-grade for the garage needs to be on at least 12 inches of imported material needed for an underslab drain as described in the ***Drainage Considerations*** section of this report. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

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.

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. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil in the area of the residence addition would generally be classified as a Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated steeper than 1:1 (H:V). However, the soils revealed in Test Borings 3, 4, and 5 would generally be classified as Type C. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1.5:1 (Horizontal: Vertical), extending continuously between the top and the bottom of a cut.

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

All permanent cuts into native soil should be inclined no steeper than 2.5:1 (H:V). Flatter inclinations would be necessary where soil is loose and wet. 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.

DRAINAGE CONSIDERATIONS

If permanent foundation walls are constructed against the shoring walls, 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 noted in the ***General*** section of this report, underslab drainage should also be provided for the proposed DADU. Plate 13 provides several recommendations for the underslab drainage 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. 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).

Onsite soils are not suitable for use as structural fill for this project. 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 structures 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. At additional cost, we can provide recommendations for reducing the risk of future movement on the steep slopes, which could involve regrading the slopes or installing subsurface drains or costly retaining structures. 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 building .

This report has been prepared for the exclusive use of Derek Lee Cheshire 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

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:

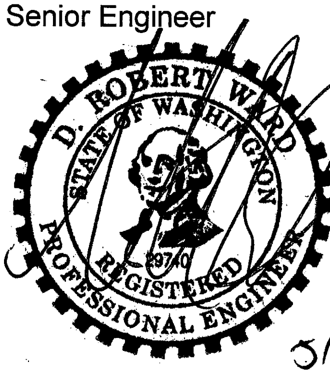
Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 10	Test Boring Logs
Plate 11	Typical Footing Drain Detail
Plate 12	Typical Shoring Drain Detail
Plate 13	Typical Underslab Drainage Detail

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.

Thor Christensen, P.E.
Senior Engineer

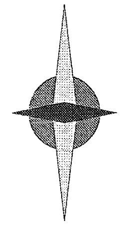


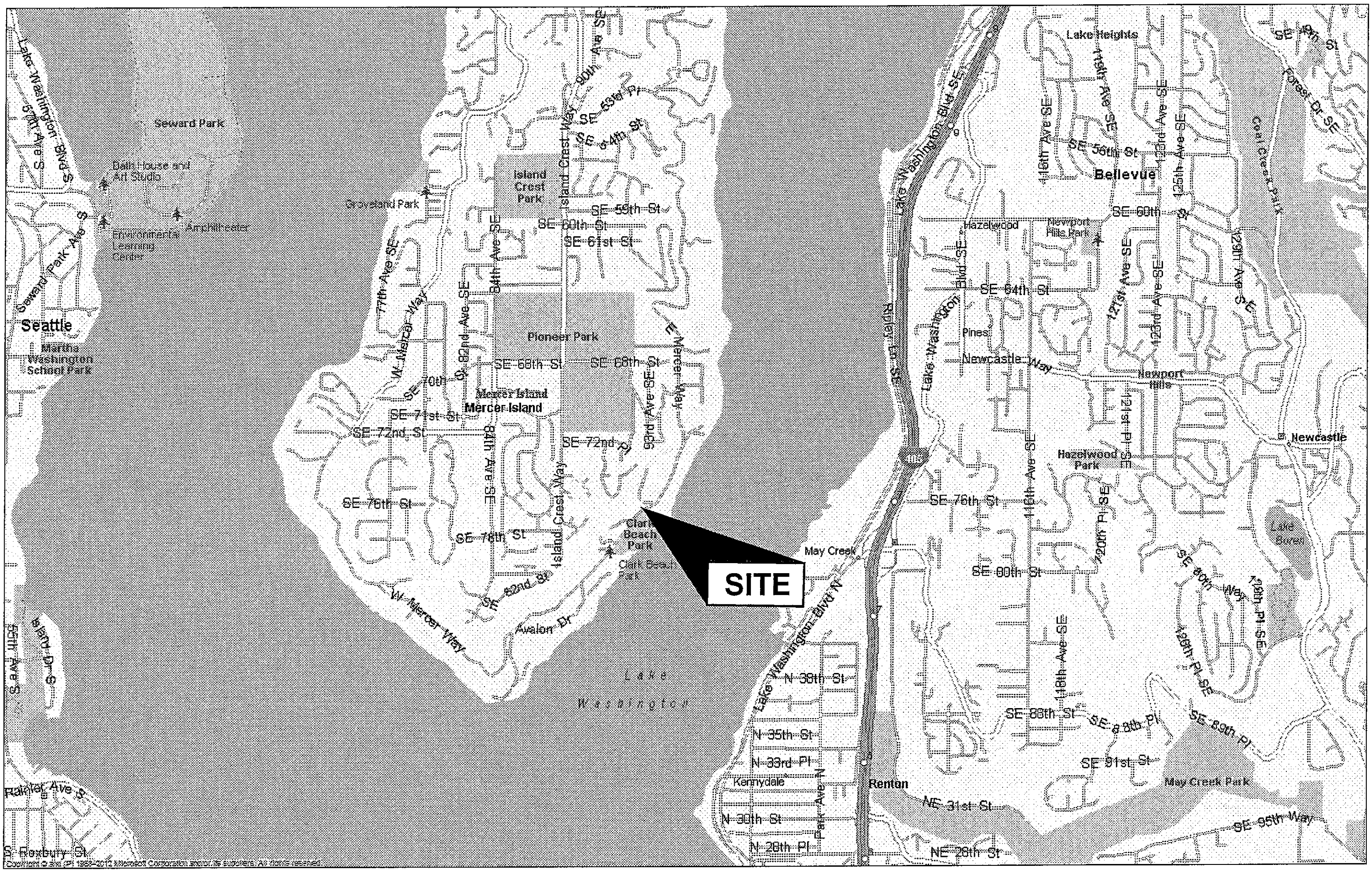
5/2/16

D. Robert Ward, P.E.
Principal

TRC/DRW:mc

NORTH





(Source: Microsoft MapPoint, 2013)

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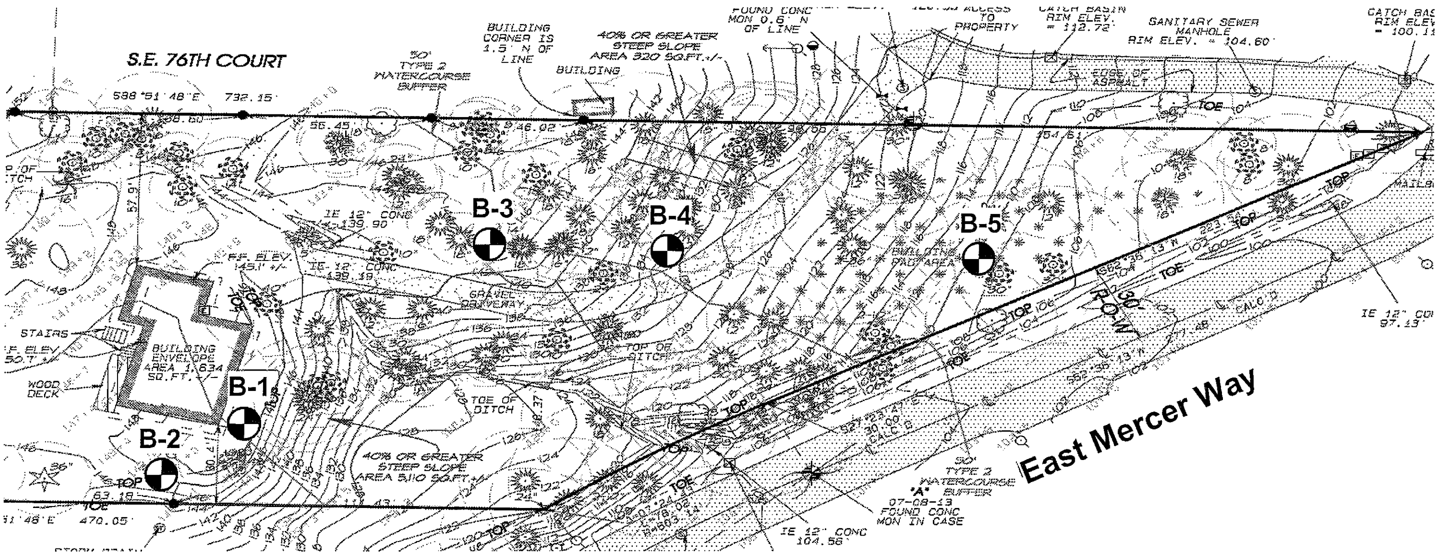
VICINITY MAP

7216 East Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16095	April 2016		1

NORTH



Legend:

Boring Location

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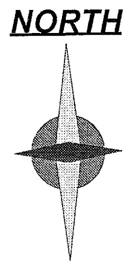
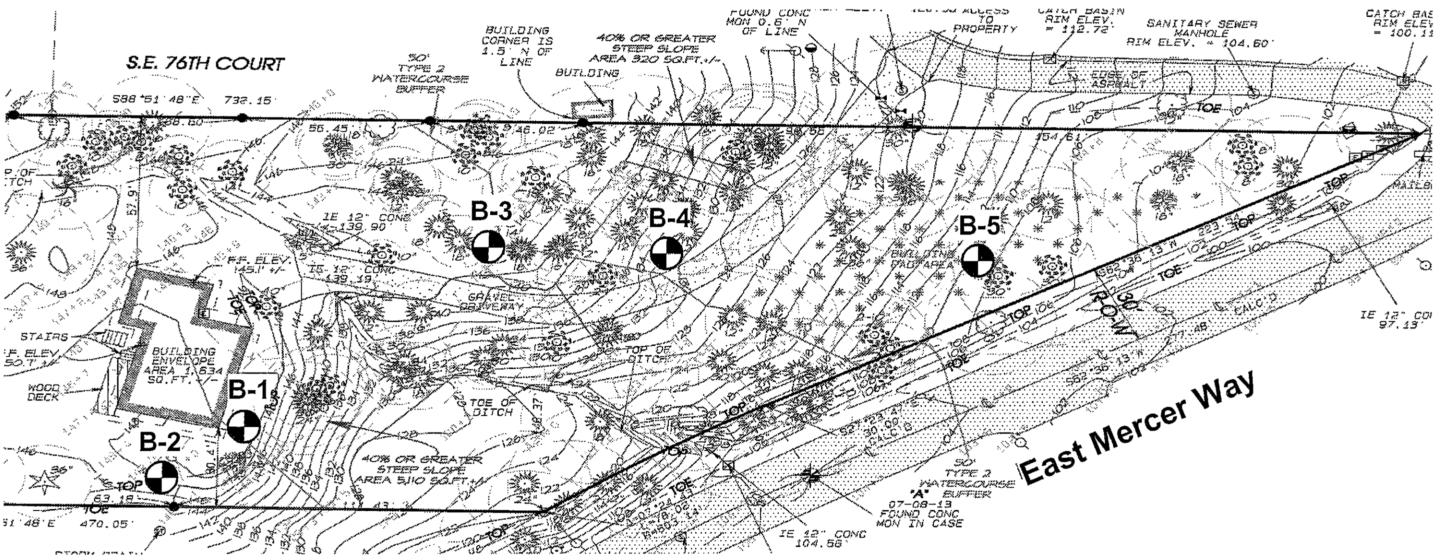
SITE EXPLORATION PLAN

7216 East Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16095	April 2016	No Scale	2





Legend:

Boring Location

Test Pit Location

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SITE EXPLORATION PLAN

7216 East Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16095	April 2016	No Scale	2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					ML	Topsoil over;
						Gray-brown SILT with clay, slightly plastic, moist, stiff
			9	1		
5			13	2		
			13	3		
10			18	4		-becomes medium-dense and non-plastic, reduced clay content
15			17	5		
20			21	6		-becomes gray, increased sand content
25			18	7		
30						

*** Test boring was terminated at 26.5 feet on March 31, 2016.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 2016	TRC	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					SM	Grass and topsoil over; Brown silty SAND, fine to coarse-grained, moist, medium-dense
			19	1		
5			7	2	ML	Gray-brown SILT with clay, slightly plastic, moist, medium-stiff
			18	3		-becomes stiff
10			25	4		-reduced clay content
15						
20						
25						
30						

*** Test boring was terminated at 11.5 feet on March 31, 2016.**
*** No groundwater was encountered during drilling.**

GEOTECH CONSULTANTS, INC.

TEST BORING LOG

7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 2016	TRC	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Topsoil over;
					SM	Brown silty SAND with gravel, fine to coarse-grained, moist, loose
			8	1		
5			7	2		-becomes wet
		▼	7	3		-becomes gray
10			13	4	SW	Gray SAND with gravel, fine to coarse-grained, wet, medium-dense
15			22	5		
20			25	6		
25			11	7	ML	Gray jumbled SILT with clay, slightly plastic, moist, medium-stiff **(Ancient Landslide Soil)**
30						

* **Continued on Plate 6.**

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TEST BORING LOG

7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 2016	TRC	5

BORING 3 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			12	8	ML	Gray jumbled SILT with clay, slightly plastic, moist, stiff **(Ancient Landslide Soil)**
35			14	9		
40			9	10	ML	Gray intact SILT, non-plastic, moist, loose
45			14	11	SP	Gray SAND with silt, very fine-grained, wet, medium-dense
50			23	12	MH	Gray clayey SILT, plastic, moist, very stiff
55						
60						

*** Test boring was terminated at 51.5 feet on March 31, 2016.**
*** Groundwater was encountered at 7.5 feet during drilling.**

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TEST BORING LOG

7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 2016	TRC	6

BORING 4

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0						Topsoil over;
					SW	Gray SAND with gravel, fine to coarse-grained, moist, medium-dense
5			11	1		
10			15	2	SM	Rust-brown mottled gray-brown silty SAND with gravel, fine to coarse-grained, moist, medium-dense
15			14	3	ML	Gray SILT, non-plastic, moist, medium-dense
20			11	4		-becomes jumbled **(Ancient Landslide Debris)**
25			14	5	SP	Gray SAND with silt, fine-grained, wet, medium-dense
30						

* **Continued on Plate 8.**

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TEST BORING LOG

7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 2016	TRC	7

BORING 4 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			18	6	SP	
					ML	Gray jumbled SILT, non-plastic, moist, medium-dense **(Ancient Landslide Soil)**
35			17	7	MH	Gray intact clayey SILT, plastic, moist, very stiff
40			25	8		
45						
50						
55						
60						

*** Test boring was terminated at 41.5 feet on April 5, 2016.**
*** Artesian groundwater was encountered at 15 feet during drilling.**

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TEST BORING LOG
7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 2016	TRC	8

BORING 5

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0						Topsoil over;
					SM	Brown silty SAND with gravel, fine to coarse-grained, wet, loose to medium-dense
5			11	1		
10		▼	7	2	SW	Brown to gray SAND with gravel, fine to coarse-grained, wet, loose
15			11	3		-becomes medium-dense and gray
20			8	4		-becomes loose
25			10	5	ML	Gray jumbled clayey SILT, plastic, moist, medium-stiff **(Ancient Landslide Soil)**
30						

* **Continued on Plate 8.**

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TEST BORING LOG
7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 2016	TRC	9

BORING 5 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			9	6	ML	Gray SILT, non-plastic to slightly plastic, moist, loose/medium-stiff
35			11	7	MH	Gray clayey SILT, plastic, moist, stiff
40			15	8		
45			22	9		-becomes very stiff
50						
55						
60						

*** Test boring was terminated at 46.5 feet on April 5, 2016.**
*** Groundwater was encountered at 10 feet during drilling.**

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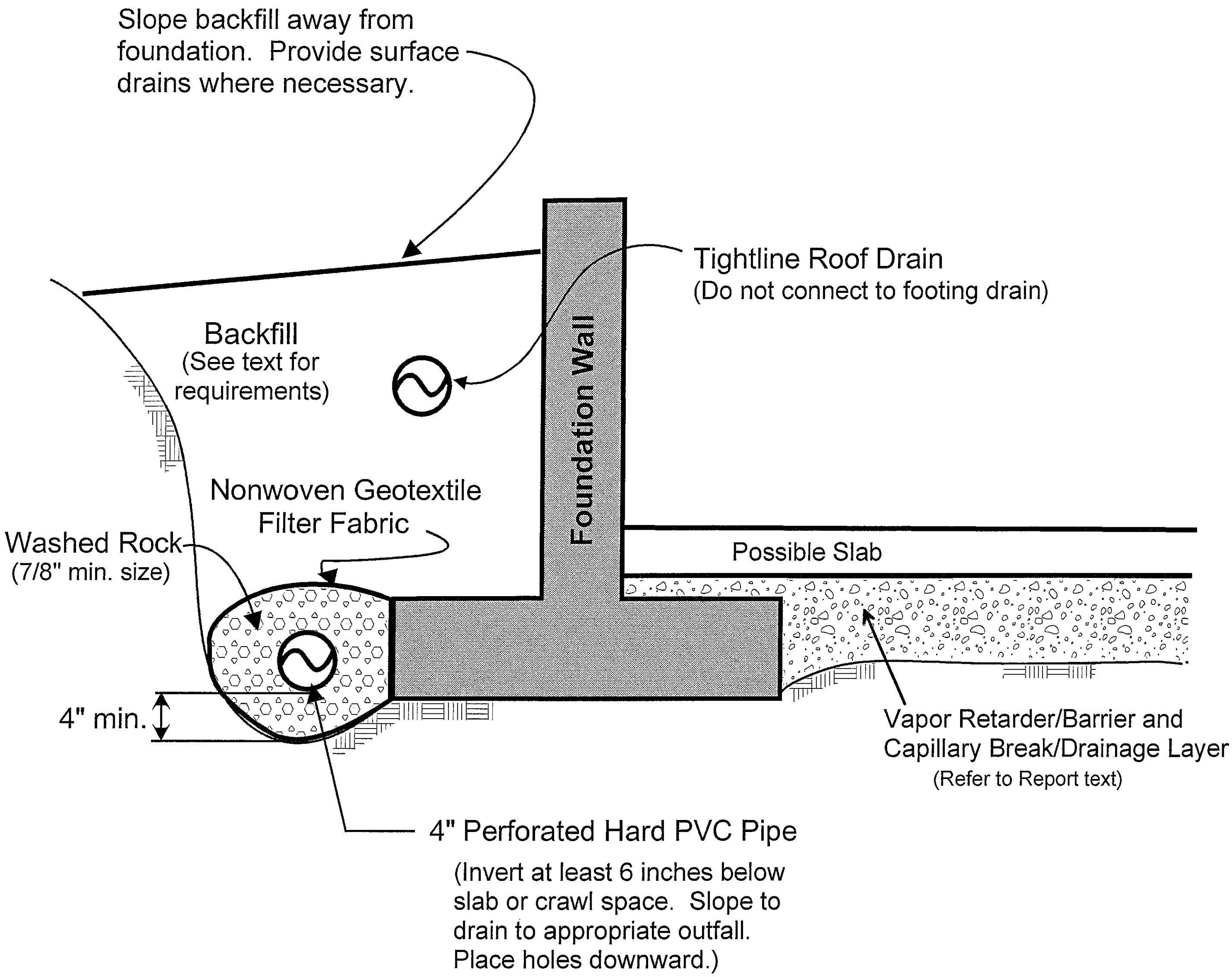
TEST BORING LOG

7615 East Mercer Way
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16095	March 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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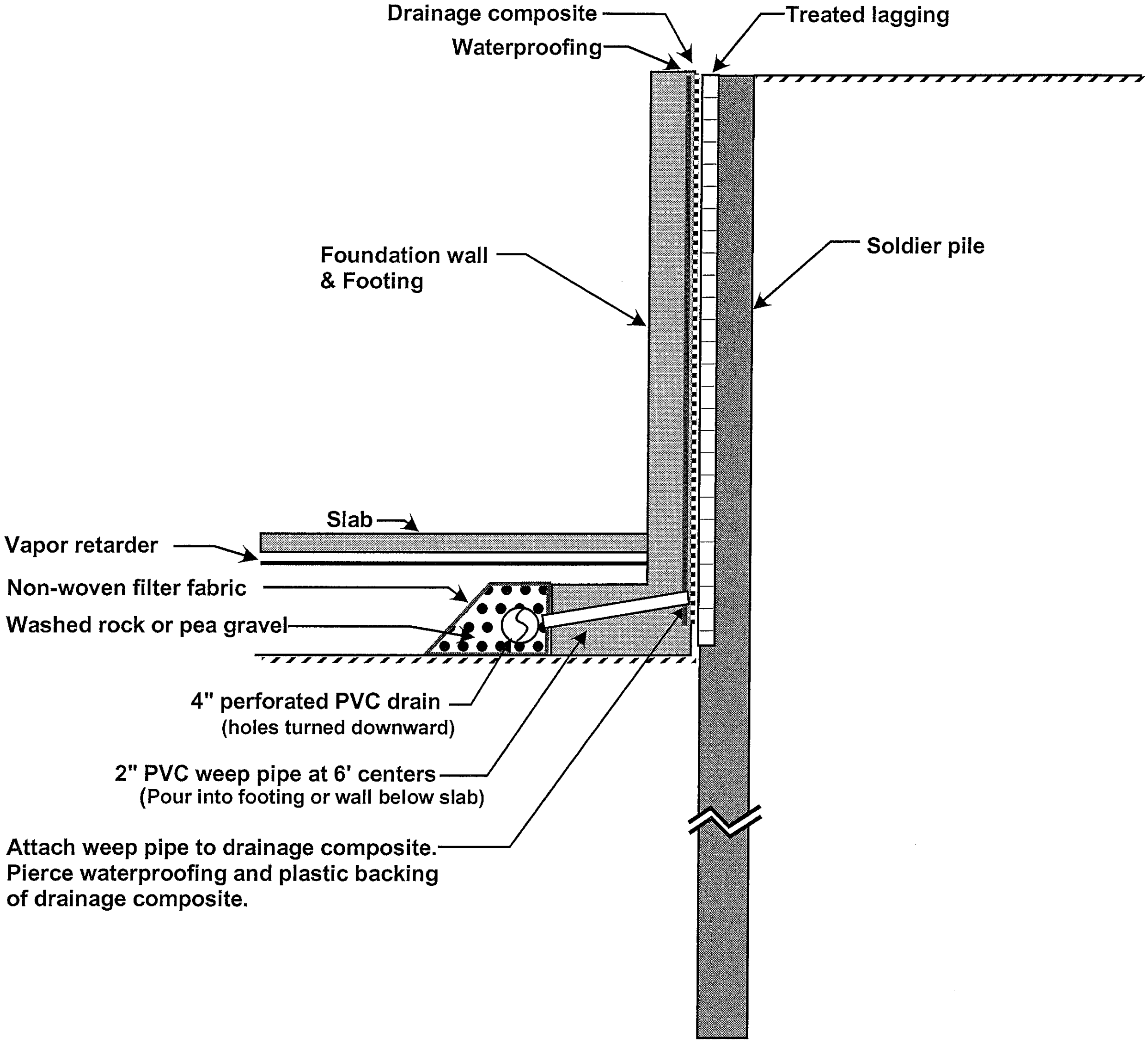
FOOTING DRAIN DETAIL

7216 East Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16095	April 2016		11



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

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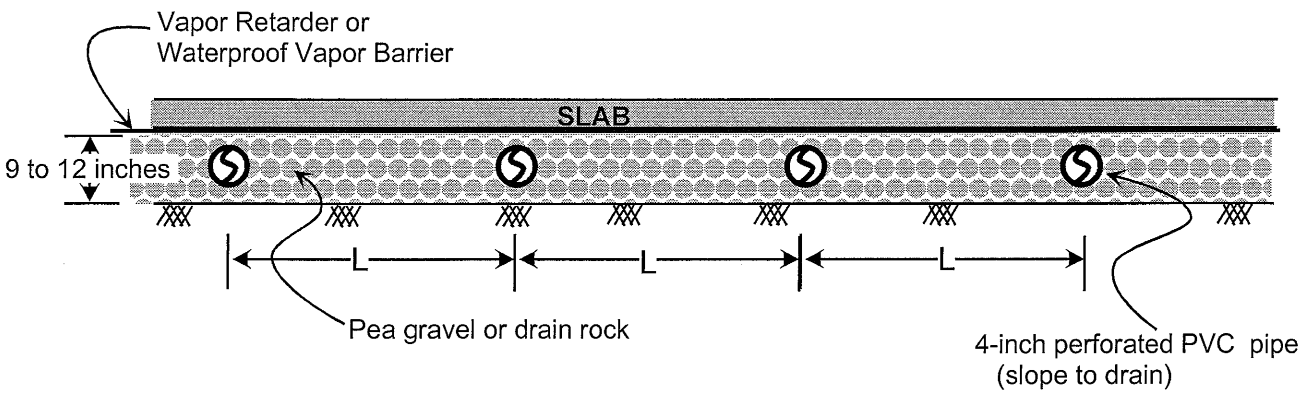
SHORING DRAIN DETAIL
7216 East Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16095	April 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

7615 East Mercer Way
Mercer Island, Washington

Job No:	Date:		Plate:
16095	April 2016		13