GEOTECHNICAL ENGINEERING STUDY
PROPOSED RESIDENCE
5637 EAST MERCER WAY
MERCER ISLAND, WASHINGTON

G-3827

Prepared for

Mr. William C. Summers
Treehouse MI, LLC
P.O. Box 261
Medina, Washington 98039

March 12, 2015

GEO Group Northwest, Inc.
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March 12, 2015 G-3827

Mr. William C. Summers
MI Treehouse, LLC
P.O. Box 261
Medina, Washington 98039

Subject: Geotechnical Engineering Study
Proposed Residence
5637 East Mercer Way
Mercer Island, Washington

Dear Mr. Summers:

GEO Group Northwest, Inc., is pleased to submit this geotechnical engineering report entitled "Geotechnical Engineering Study, Proposed Residence, 5637 East Mercer Way, Mercer Island, Washington." This report presents our findings, conclusions, and recommendations from investigation activities that we have completed at the above-subject project site for your proposed construction of a single-family residence.

We explored subsurface soil conditions at the site by drilling two exploratory soil borings. Soils encountered in the borings typically consisted of loose, fine sand and silty sand underlain by medium dense to dense, unsaturated silt. Groundwater was encountered at or near the ground surface in both of the borings.

The site soils encountered in the borings will not be suitable to directly support foundations due to their loose and wet condition. Also, due to the presence of groundwater seepage from the slopes on the south part of the site, substantial excavation into the soils at the site is not recommended, particularly in the area where wet, loose soil conditions are present.

It is our opinion that the proposed residence can be supported vertically on a system of small-diameter steel pipe piles that are founded in the dense silty soils below the site. Lateral support for the residence can be achieved either by using battered pipe piles or by using helical anchors.

As an alternative, we considered the use of conventional spread footings bearing on a 3-feet thick layer of crushed rock and geotextile fabric to support the residence. Upon closer analysis, however, we have concluded that such an approach may not adequately mitigate potential soil settlement and soil liquefaction problems.

Our recommendations, along with other geotechnical aspects of the project, are discussed in more detail in the text of the attached report.

We appreciate this opportunity to have been of service to you on this project. We look forward to working with you as the project progresses. Should you have any questions regarding this report or need additional consultation, please feel free to call us.

Sincerely,

GEO Group Northwest, Inc.

William Chang, PE.
Principal

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GEOTECHNICAL ENGINEERING STUDY
PROPOSED RESIDENCE
5637 EAST MERCER WAY
MERCER ISLAND, WASHINGTON

G-3827

1.0 INTRODUCTION

1.1 Project Description

GEO Group Northwest, Inc., has completed a geotechnical engineering study for the proposed development of a single-family residence on the property at 5637 E. Mercer Way, Mercer Island, Washington.

1.2 Scope of Investigation

The tasks we completed for this study included the following:

Year 1999:

1. Conducted a subsurface investigation at the site consisting of drilling two soil borings. The borings were drilled in the approximate proposed location the proposed residence at the time of the investigation;

2. Performed laboratory testing on soil samples collected from the borings, and prepared boring logs;

3. Performed engineering analysis for foundation support, grading considerations, earthwork criteria for on-site soils and imported soils, and pavement section design; and

4. Prepared a geotechnical report of our findings, conclusions, and recommendations.

Year 2015:

1. Performed a reconnaissance of the project site to update our knowledge of current site conditions;

2. Reviewed and updated, where appropriate, the findings, conclusions, and recommendations contained in our previous reports (our 1999 report and an updated 2005 report) for the project site; and

3. Prepared this new geotechnical report of our findings, conclusions, and recommendations for the currently proposed residence for the project site.

2.0 SITE CONDITIONS

2.1 Site Description

The project site is located on the west side of the 5600 block of East Mercer Way on Mercer Island, Washington, as shown on Plate 1 - Site Location Map. The site is bordered to the south by a single family residence (5643 East Mercer Way). A small stream flows from west to east across the northern part of the site. Lake Washington is located approximately 0.2 miles east of the site.

The site consists of an irregular shaped lot that comprises about 38,700 square feet. The site generally slopes downward toward the north and northeast toward a ravine with an east-running stream on the north side of the site. Elevations on site range between approximately 158 feet at stream course in the northeast corner and approximately 226 feet at the south corner which is on a steeply rising slope (with inclinations up to approximately 75 percent). The existing conditions and topography on the site are illustrated in Plate 2 - Site Plan.

2.2 Proposed Development

We understand the proposed residence is planned to be located on the relatively less steeply sloped middle part of the site, as illustrated in Plate 3 - Proposed Residence Plan. Slopes in this area have inclinations up to approximately 28 percent. The proposed floor elevation for the residence currently are 180 feet for the basement/garage and 190 feet for the main floor of the residence, as illustrated in Plate 4 - Proposed Residence Section. Elevation views of the proposed residence are presented in Plate 5A - North & South Elevations and Plate 5B - East & West Elevations.

2.2 Geologic Overview

According to the Geologic Map of Mercer Island, Washington, by Troost, K.G. and A.P. Wisher, published October 2006, the surficial geology in the site vicinity is mapped as consisting of Quaternary-age Advance Outwash Sand (Qva) on the geologic map. These soils typically consist of fine to medium grained sand with occasional silty layers. These soils typically are underlain with a relatively impermeable silt unit, referred to as Lawton Clay on the geologic map. The map also indicates that landslide deposits are located on and in the immediate vicinity of the site.

Groundwater typically accumulates in the lower portion of the outwash sand unit where it is underlain by the impermeable silt. This water then forms springs and seeps on slopes where the contact between the units is exposed. Under these conditions, the sand soils commonly are susceptible to instability such as landslides or earthflows.

3.0 SITE INVESTIGATION

3.1 1999 Subsurface Investigation

A GEO Group Northwest geologist supervised the drilling of two exploratory soil borings (B-1 and B-2) on August 10, 1999. The borings were completed by using a manually portable drilling rig and were located in the middle portion of the site, as indicated in Plate 2 - Site Plan. The boring locations were estimated by using a roll tape and by visual reference to existing site features noted on the topographic survey that was provided to us.

Soils encountered in the borings typically consisted of a surficial layer of soft, wet, mucky fine silty sand topsoil. The topsoil was underlain with loose to medium dense, wet, fine grained, silty sand and sand. These soils were found to a depth of approximately 14 feet (equivalent to approximate elevation 173 feet in boring B-1 and approximately 20 feet (equivalent to approximately elevation 156 feet) in boring B-2. These soils were underlain with medium dense, damp to moist silt with occasional lenses of silty fine sand to the bottom depths of both borings. Logs of the soil borings are provided in Attachment 1 to this report.

Groundwater seepage was observed at the surface during our explorations at the site. Saturated soils were present approximately from ground surface to the bottom of boring B-1 at 15 feet deep, and heaving action of the wet sand into the borehole prevented further drilling of the boring. Saturated soils were encountered in boring B-2 from near ground surface to approximately 20 feet deep, but the heaving action of the wet sand was able to be mitigated.

During our activities, we also observed the presence of groundwater seepage at the base of the steep slope in the south part of the site (from southwest to southeast of the location of boring B-1).

3.2 2015 Site Reconnaissance

On March 9, 2015, we performed a reconnaissance of the site to update our knowledge of the site conditions. We observed that the site appears to have not been substantially modified since the time of our 1999 investigation activities. We observed that the ground surface conditions were similar to those we had found during the previous investigation, with presence of soft, wet, mucky sand on the middle part of the site below the base of the steep slope. We did not observe evidence of landslides on the site since the time of our previous investigation activities, such as exposed scarps, or apparent freshly exposed soils.

4.0 SEISMICITY

4.1 Puget Sound Seismic History

The project site is located within the Seattle metropolitan area. The greater Puget Sound region historically has experienced a number of small to moderate earthquakes and occasional strong shocks. Historical records for the region indicate that the Olympia earthquake of April 13, 1949, with a Richter magnitude of 7.1, produced ground-shaking of intensity VIII on the Modified Mercalli Scale near its epicenter. The Seattle-Tacoma earthquake of April 29, 1965, had a Richter magnitude of 6.5 and produced a ground-shaking of intensity IV to VIII near its epicenter. The most recent significant event, the Nisqually earthquake of February 28, 2001, with a Richter magnitude of 6.8, also produced ground shaking with intensities up to VIII. This level of ground-shaking is estimated to be the maximum that has occurred in the region during the approximately 160 years of the historic record.

4.2 Site Seismic Design Classification

Per the procedures specified in Section 1615 of the 2012 International Building Code (IBC), we conclude that the project site should be assigned a seismic design classification of Site Class F due to the presence of up to approximately 20 feet of potentially liquefiable soils (as discussed below in **Section 4.3 - Liquefaction Assessment**). However, the soils below a depth of approximately 20 feet are very dense and are suitable for assigning Site Class C (Very Dense Soil profile) to the proposed development of the site if the structures are fully supported on the deeper, very dense soils.

4.3 Liquefaction Assessment

Liquefaction is a phenomenon where loose granular materials below the water table temporarily behave as a liquid due to strong shaking or vibrations, such as earthquakes. Clean, loose and saturated granular materials are the soil types susceptible to liquefaction phenomena.

During our site investigation, subsurface soil consisted of wet, very loose to medium dense fine sand, silty fine sand, and silt. Water saturated loose sandy soils were encountered from ground surface to approximately 15 to 20 feet in the borings. Therefore, it is our opinion that the shallow subsurface sandy soils at the site are susceptible to liquefaction, based on the observed soil types, densities, and moisture contents. Soils at depths below approximately 20 feet are not likely to be susceptible to liquefaction, because these soils consist primarily of unsaturated silt, based on the information obtained during our investigation.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on the findings from our site investigation activities, it is our opinion that the site can be developed with a single-family residence. However, due to the presence of wet, loose sandy soils at the site and the presence of steep slopes exhibiting groundwater seepage at the site, we recommend that the residence be supported on a deep foundation system comprised o small-diameter steel pipe piles and possibly helical soil anchors that are driven into the dense underlying soils and are connected to a system of grade beams.

We also recommend that the proposed residence be designed such that the least possible amount of disturbance is made to the site soils on the steep slope area and below the steep slope area where wet, loose sands are present. For this reason, we recommend that site grading be minimized to only the amount that is necessary to achieve construction access and to construct the improvements (including the driveway) consistent with permit requirements. The residence could be built essentially at-grade or on an above-grade pile-supported deck, for example. Excavations in areas where wet, soft soils are present will need to be gently sloped or supported, and accumulation of groundwater seepage in such excavations is likely and will need to be mitigated.

Our recommendations regarding geotechnical aspects of the proposed development are presented in the following sections of this report. These subjects include site preparation and earthwork, building support, site drainage, and pavements.

5.2 Grading and Earthwork

Site Preparation

Disturbance to the site soils should be kept to a minimum, and no disturbance should occur within 25 feet of the stream in the north part of the site. Erosion control measures should be implemented around areas disturbed by construction activity to prevent sediment-laden surface runoff from being discharged off-site.

To provide equipment access to the site and to the building area, we recommend that a temporary entrance pad be used to bridge over the soft soils at the site and also provide drainage to the subgrade. To prepare working pad, the surface soils should be excavated to a depth of at least two feet below existing grade. A layer of woven geotextile filter fabric, such as Mirafi 600X or equivalent, should be placed over the subgrade prior to placing the quarry spalls, to provide separation of materials and pad reinforcement.

Site Work During Wet Weather

We understand that earthwork at the project site may be subject to a seasonal moratorium, per City of Mercer Island development regulations. Under these circumstances, earthwork at the site should not performed during the period from October 1 to March 31, and the site should be stabilized against potential development-related earth movement, erosion, or off-site sedimentation before the start of the moratorium period.

Temporary Erosion and Sediment Control

Implementing and maintaining effective temporary erosion and sediment control measures should be performed by the contractor during construction. Clearing and grading should be limited to areas where construction will occur, to the extent possible. Temporary erosion control should be installed downhill from areas disturbed by construction activity to prevent sediment-laden runoff from being discharged off site. We recommend that sediment traps, filter fabric fences, check dams, straw mulch, hay bales, stabilized construction entrances, wash pads, and other appropriate erosion control devices be used to provide temporary sediment and erosion control.

Temporary Excavation and Slopes

Under no circumstances should temporary excavation slopes be greater than the limits specified in local, state and federal government safety regulations. Temporary cuts greater than four feet in height should be sloped at an inclination no steeper than 2.5H:1V (Horizontal:Vertical) in medium dense to dense unsaturated soils, and no steeper than 1H:1V in the stiff unsaturated silt soils, unless specifically reviewed and approved by the geotechnical engineer. Excavations into saturated soils should be avoided where possible, because engineered support of such cuts (such as with shoring) will probably be required. Permanent cut and fill slopes at the site should be inclined no steeper than 2.5H:1V in non-saturated, competent soils.

We recommend that temporary and permanent cuts in the soils on or in proximity to the steep slope on the southern part of the site be avoided where possible (and not extend into saturated soils where they are necessary), due to the loose and wet soil conditions in this area.

Surface runoff should not be allowed to flow uncontrolled over the top of slopes into the excavated area. During wet weather, exposed cut slopes should be covered with plastic sheeting during construction to minimize erosion. We recommend that a GEO Group Northwest, Inc., representative be on site during excavation of cut slopes to evaluate slope stability, due to the anticipated presence of groundwater seepage and loose soil conditions.

Structural Fill

All structural fill material used to achieve design site elevations below the building area and below non-structurally supported sidewalks, driveways, and patios, should meet the requirements for structural fill. During wet weather conditions, material to be used as structural fill should have the following specifications:

1. Be free draining, granular material containing no more than five (5) percent fines (silt and clay-size particles passing the No. 200 mesh sieve);
2. Be free of organic material and other deleterious substances;
3. Have a maximum size of three (3) inches in diameter.

The fill material should be placed at or near the optimum moisture content. The optimum moisture content is the water content in soil that enables the soil to be compacted to the highest dry density for a given compaction effort.

We anticipate that the on-site material will be unsuitable in its existing condition for use as structural fill, due to its high moisture content and the presence of silt and organics in much of the material. During dry weather, however, any compactable non-organic soil may be used as structural fill, provided the material is near its optimum moisture content for compaction purposes. It should be noted that an imported granular fill material may provide more uniformity and be easier to compact to structural fill specifications.

If the on-site soils are to be used as engineered structural fill, it will be necessary to segregate the topsoil and any other organic- or debris from the soil. Also, the soil will need to be moisture conditioned to bring it near to its optimum moisture content for compaction. Once it is suitably prepared, the soil will then need to be protected from weather and from contamination with unsuitable materials until it is used.

Structural fill should be placed in thin horizontal lifts not exceeding 10 inches in loose thickness. In areas having slopes greater than 15 percent, horizontal benches should be cut to competent native soil before the fill is placed, in order to prevent possible later lateral movement. Structural fill under building areas (including foundation and slab areas), should be compacted to at least 95 percent of the maximum density, as determined by ASTM Test Designation D-1557-91 (Modified Proctor). Structural fill under pavements should be compacted to at least 90 percent of the maximum density, except for the top one foot which should be compacted to at least 95 percent. We recommend that GEO Group Northwest, Inc., be retained to evaluate the suitability of structural fill material and to monitor the compaction work during construction for quality assurance of the earthwork.

5.3 Building Support

Based on the results from our investigation activities, it is our opinion that the proposed residence should be supported on a deep foundation system that is founded in the dense silty soils that were encountered in the borings completed for this study. Such a foundation system can consist of small-diameter steel pipe piles and possibly helical anchors to support a system of structural grade beams. The pipe piles can provide vertical support to the residence; lateral support to the residence can be provided either by battered pipe piles or by helical anchors.

Small-Diameter Pipe Piles

Pipe piles are typically are installed by driving them with a jackhammer or other pneumatic-type hammer to a condition where the resistance of the soils encountered essentially terminate the advance of the piles (this condition is called "refusal"). The depth at which refusal is achieved is dependent upon 1) the type of pipe and hammer that are used, 2) the characteristics of the subsurface soil, and 3) the allowable load-bearing capacity to be provided by the pile.

We estimate that refusal depths for the piles will be in the range of about 25 to 30 feet. These estimated depths are based on the anticipation that substantial thicknesses of very stiff to hard silt soils or dense sand soils are present below depths of about 20 feet at the site. Due to the shallow groundwater conditions at the site, we recommend that galvanized pipe be used for the piles.

The following available driving hammers, pipe sizes, allowable bearing capacities, and installation refusal criteria are recommended for supporting the residence:

Pipe Pile Design Criteria

Pipe Diameter	Pipe Specification	Hammer Weight Class	Hammer Type	Refusal Criteria*	Allowable Capacity
2 inch	Schedule 80	140 pound	jackhammer	60 sec/inch	2 tons
3 inch	Schedule 40	650 pound	TB225**	12 sec/inch	6 tons
3 inch	Schedule 40	850 pound	TB325**	10 sec/inch	6 tons
4 inch	Schedule 40	850 pound	TB325**	16 sec/inch	10 tons
4 inch	Schedule 40	1100 pound	TB425**	10 sec/inch	10 tons
6 inch	Schedule 40	1500 pound	TB425**	20 sec/inch	15 tons

* = Maximum penetration rate to be sustained through at least 3 consecutive minutes of driving
** = Teledyne pneumatic hammer model number, or equivalent

We estimate that the maximum total post-construction settlement should be one-half (1/2) inch or less. No reduction in pile capacities is required if the pile spacing is at least three times the pile diameter. A one-third increase in the above allowable pile capacities can be used when considering short-term transitory wind or seismic loads.

Vertical pipe piles do not generate significant lateral capacities. Instead, lateral forces can be resisted by passive earth pressure acting on grade beams or footings and by friction with the subgrade soils, where acceptable subgrade soil conditions are present. To fully mobilize the passive pressure resistance, the grade beams or footings must be constructed directly against competent native soil or compacted fill. For these conditions, our recommended allowable passive soil pressure for lateral resistance is 350 pcf equivalent fluid weight. A coefficient of friction of 0.35 may be used between a competent native soil or compacted fill subgrade and the foundation.

We note that the loose, wet sand soils in the proposed residence location are not acceptable for providing the above-recommended condition, and would need to be replaced with an acceptable pad of compacted fill. Other options for resisting lateral loads include using either battered pipe piles or helical anchors. Recommendations regarding helical anchors are provided below.

The performance of pipe piles is dependent on how and to what bearing stratum the piles are installed. Since a completed pile in the ground cannot be observed, it is critical that judgment and experience be used as a basis for determining the driving refusal and acceptability of a pile. Therefore, we recommend that GEO Group Northwest, Inc., be retained to monitor the pile installation operation, collect and interpret installation data and verify suitable bearing stratum. We also suggest that the contractor's equipment and installation procedures be reviewed by GEO Group Northwest, Inc., prior to pile installation to help mitigate problems which may delay the progress of the work.

Helical Anchors

The foundation for the proposed residence can be horizontally restrained by installing helical anchors into the underlying soil. Helical anchors, such as those developed by the A. B. Chance Company and Atlas Systems, Inc., consist of a steel square shaft with one or more helices on the anchor shaft. Lateral loads can be resisted by installing additional helical anchors either perpendicular to the slope face or at an inclination of 30 degrees from vertical.

The ultimate capacity for helical anchors should be determined and verified in the field by a geotechnical engineer based on the installation torque that is achieved during installation. For Chance helical anchors, the ultimate capacity can be determined by the following empirical relationship:

$$QULT = Kt * T$$

where Kt is the empirical factor (= 10 ft-1 for square shaft anchors); and T is the installation torque.

The allowable capacity of the Chance helical anchor may also be developed when sufficient torque is recorded during installation. For example, based on the empirical correlation developed by the A. B. Chance Company, an installation torque of 4,000 ft-lbs roughly correlates to an ultimate capacity of 20 tons. Thus, the allowable capacity for the installed anchor with a factor of safety of 2 with respect to its ultimate capacity is approximately 10 tons.

Based on the soil conditions encountered in the borings, we anticipate that the anchors may need to extend a minimum distance of about 15 feet into the underlying soils below the residence in order to attain acceptable load capacity. The allowable capacity of 5 tons for the anchors is based on a factor of safety of 2.0 with respect to the ultimate tensile capacities, developed behind a 15 feet long no-load zone for the anchors.

The performance of helical anchors is dependent on the method and to what bearing stratum the anchors are installed. Since a completed anchor in the ground cannot be observed, it is critical that judgment and experience be used as a basis for determining the acceptability of an anchor. Therefore, we recommend that GEO Group Northwest, Inc., be retained to monitor the anchor installation operations, collect and interpret installation data, and verify acceptable loading capacity for the anchor has been attained.

5.4 Building Floors

We recommend that building floors be structurally supported and connected to the foundation system.

5.5 Conventional Concrete Basement and Retaining Walls

GEO Group Northwest, Inc., anticipates that the proposed residence may have a daylight basement level, based on the preliminary plans we have seen for the proposed residence. Therefore, our recommendations regarding conventional concrete basement and retaining walls are provided below for your information. The following recommendations apply to walls that retain fully drained soils. If basement or retaining walls will be retaining saturated soils, then we should be consulted to provide applicable design parameters.

Conventional concrete retaining walls that are free to rotate on top should be designed for an active soil pressure. Permanent retaining walls that are restrained horizontally at the top (such as basement walls) are considered unyielding and should be designed for a lateral soil pressure under the at-rest condition. The walls should be supported on dense, native soils or structural fill. Soil parameters for the wall design are as follows:

<u>Active Earth Pressure</u>
35 pcf, equivalent fluid pressure, for level ground behind the wall;
50 pcf, equivalent fluid pressure, for 2H:1V backslope behind the wall

<u>At-Rest Earth Pressure</u>
45 pcf, equivalent fluid pressure, for level ground behind the wall;
60 pcf, equivalent fluid pressure, for 2H:1V backslope behind the wall

<u>Passive Earth Pressure</u>
350 pcf, equivalent fluid pressure, for medium dense to dense soil and structural fill.

<u>Base Friction</u>
0.35 for undisturbed, dense soil or structural fill.

Surcharge loads imposed on walls by traffic (including construction vehicles), nearby structures, or other conditions, should be added to the active and at-rest earth pressures stated above. Also, downward sloping ground in front of walls should be considered with regard to potentially reducing the value of the allowable passive earth pressure stated above.

To prevent the buildup of hydrostatic pressure behind permanent basement or conventional retaining walls, we recommend that a vertical drain mat, Miradrain 6000 or equivalent, be used to facilitate drainage behind the wall. The drain mat core is placed against the wall with the filter fabric side facing the backfill. The drain mat should extend from the finished surface grade, down to the footing drain. In addition to the vertical drain mat, a prism of clean, granular, free draining structural backfill material at least 18 inches wide should be placed against the wall. The free-draining backfill should extend downward to the footing drain.

The top 12 inches of the fill behind the wall should consist of compacted and relatively impermeable soil. This cap material can be separated from the underlying more granular drainage material by a geotextile fabric, if desired. Alternatively, the surface can be sealed with asphalt or concrete paving. The surface should be sloped to drain away from the building wall. A schematic illustration of the wall and drainage system is presented in Plate 6 - Basement and Retaining Wall Backfill and Drainage.

The backfill in areas adjacent to concrete retaining walls should be compacted with hand held equipment or a hoe-pack. Heavy compacting machines (such as a vibratory roller) should not be allowed within a horizontal distance to the wall equivalent to one half the wall height, unless the walls are designed with the added surcharge.

5.6 Drainage

The finished ground at the site should be graded such that surface water is directed off the site. Water should not be allowed to stand in any area where footings, slabs or pavements are to be constructed. During construction, loose surfaces should be sealed at night by compacting the surface to reduce the potential for moisture infiltration into the soils. Final site grades should allow drainage away from the building. We suggest that the ground be sloped at a gradient of three percent for a distance of at least ten feet away from the building except in areas that are to be paved.

5.7 Pavement Subgrade

We recommend that the driveway for the new residence be supported on a thickened base of compacted ballast rock (at least 24" thick) that is underlain and overlain with a layer of woven geotextile fabric, such as Mirafi 500X or equivalent. The pavement section can then be constructed over the upper layer of geotextile. The pavement section can consist of at least 6 inches of base course overlain with at least 2 inches of asphalt.

6.0 LIMITATIONS

This report has been prepared for the specific application to the proposed development of the site decsribed herein, and for the exclusive use of Mr. William C. Summers of MI Treehouse, LLC, and his authorized representatives or agents. We recommend that this report be included in its entirety in the project contract documents for reference during construction.

Our findings and recommendations stated herein are based on field observations, our experience and judgment. The recommendations are our professional opinion derived in a manner consistent with the level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area and within the budget constraint. No warranty is expressed or implied. In the event the soil condition vary during site work, GEO Group Northwest, Inc. should be notified and the above recommendation should be re-evaluated.

7.0 ADDITIONAL SERVICES

We recommend that GEO Group Northwest Inc. be retained to perform a general review of the final design and specifications of the proposed development to verify that the earthwork, foundation, drainage, pavement, and other geotechnical recommendations are properly interpreted and incorporated into the design and construction documents and are appropriate for the finalized layout of the proposed development.

We also recommend that GEO Group Northwest Inc. be retained to provide monitoring and testing services for geotechnically-related work during construction. A GEO Group Northwest, Inc., representative should observe geotechnically-related construction work for compliance with the geotechnical recommendations in this report, and should be available to discuss and recommend design changes, if needed, in the event substance conditions differ from those anticipated prior to the start of construction.

GEO Group Northwest, Inc.

Respectfully Submitted,

GEO Group Northwest, Inc.

State of Washington
Engineering Geologist
1005
Licensed Geologist

Keith Johnson
Geologist

KEITH A. JOHNSON

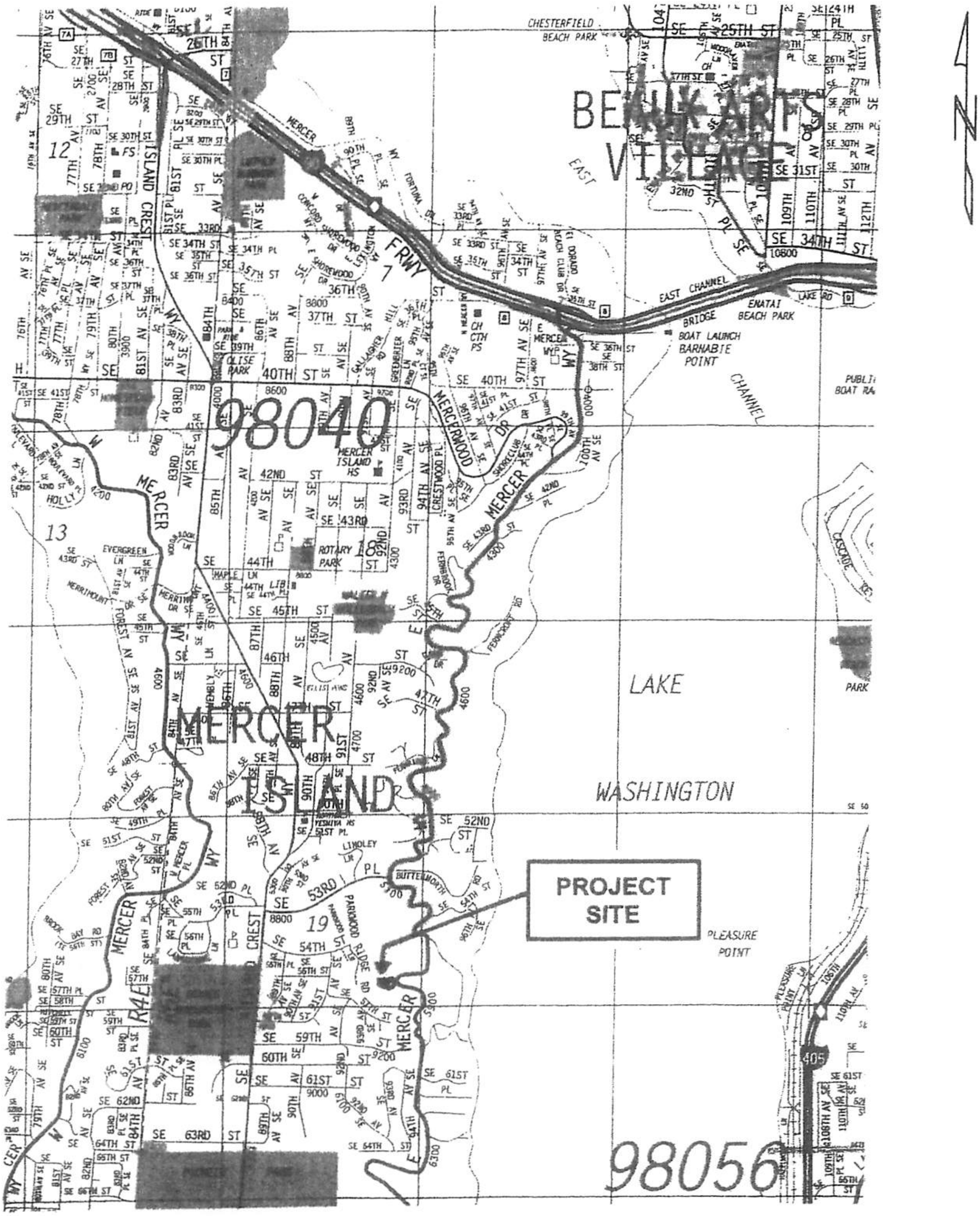
William Chang, PE
Principal

WILLIAM CHANG
STATE OF WASHINGTON
20114
REGISTERED
PROFESSIONAL ENGINEER

PLATES

G-3827

GEO Group Northwest, Inc.



Source: Thomas Guide, 2007.

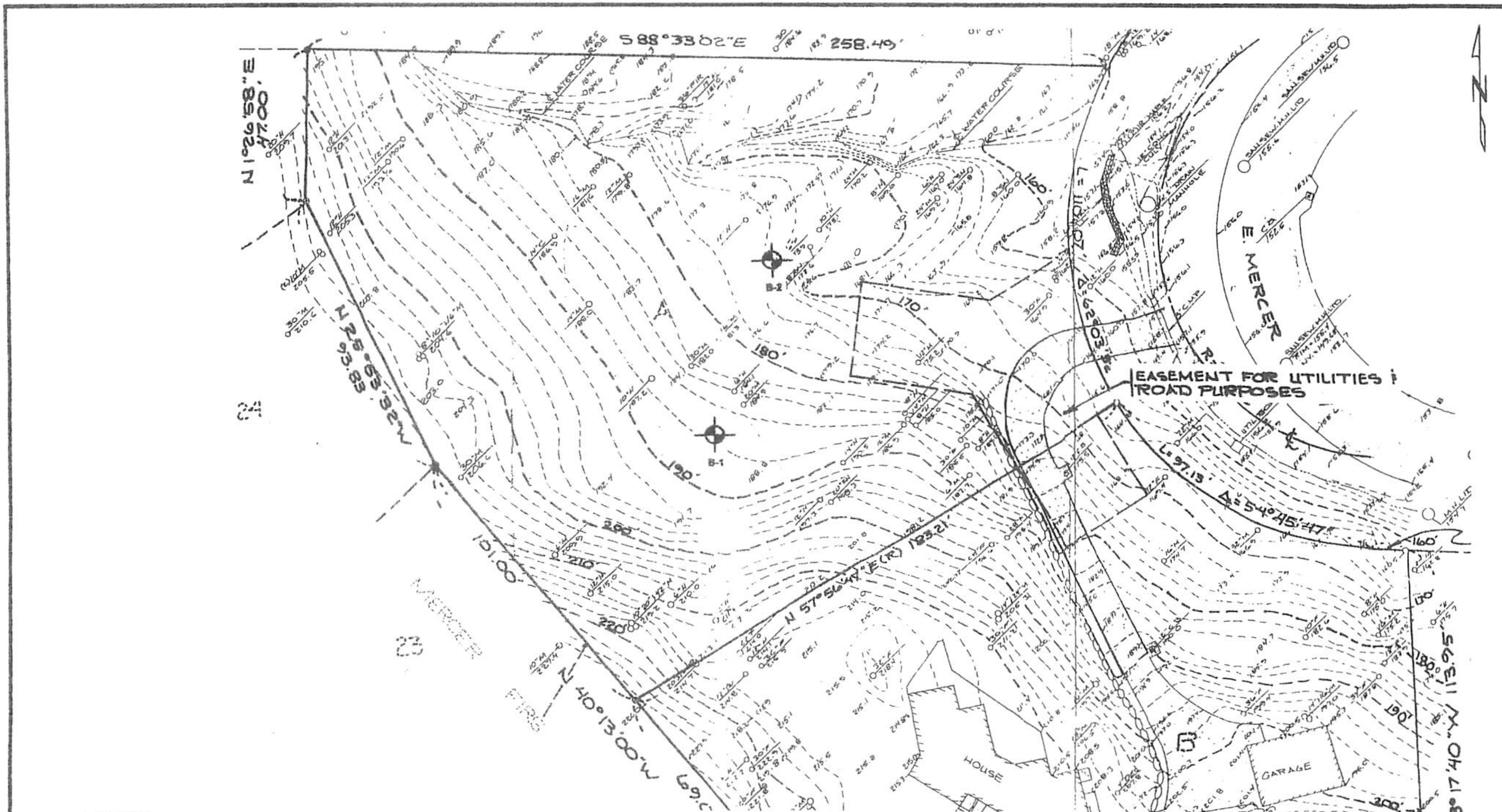
GEO Group Northwest, Inc.

Geotechnical Engineers, Geologists, & Environmental Scientists

SITE LOCATION MAP

PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE: 1" = 2,000'	DATE: 3/11/2015	MADE: KJ	CHKD: WC	JOB NO: G-3827	PLATE 1



LEGEND

B-1 EXPLORATORY SOIL BORING DRILLED BY GEO GROUP NORTHWEST (APPROXIMATE LOCATION)

0 30 60
SCALE: 1 INCH = 30 FEET

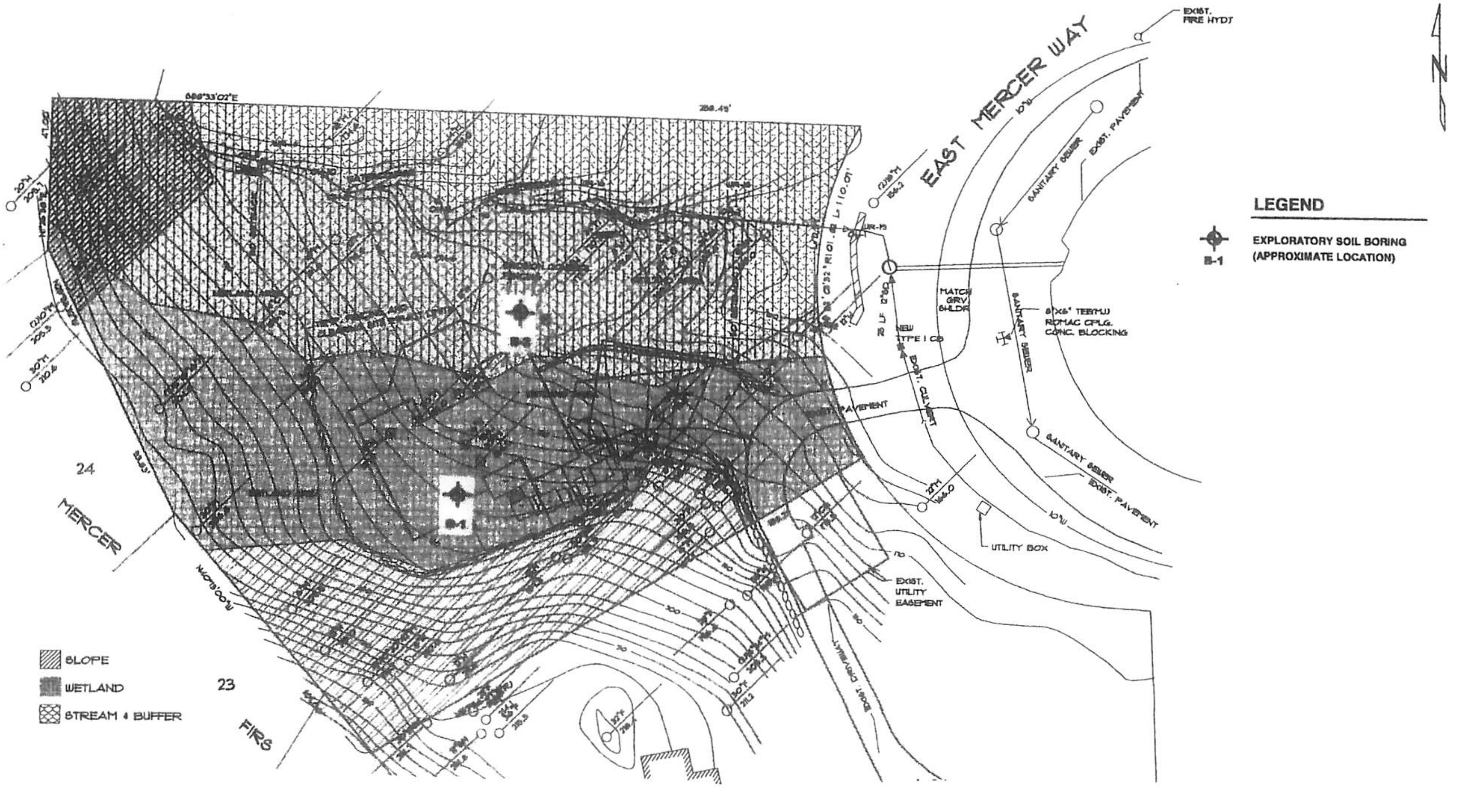
Source: Survey of a Portion of Lots A & B, Greg Newitt Short Plat (M.I.S.P. No. 77-1-010), by M.W. Marshall, Professional Land Surveyor, dated 1/16/98.

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

SITE PLAN

PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE	1" = 30'	DRAWN BY	KJ	CHECKED BY	WC	DATE	3/10/2015	PROJECT NO.	G-3827	PLATE	2



Source: Master Site Plan by Healy-Jorgensen Architects, dated 3/2/15.

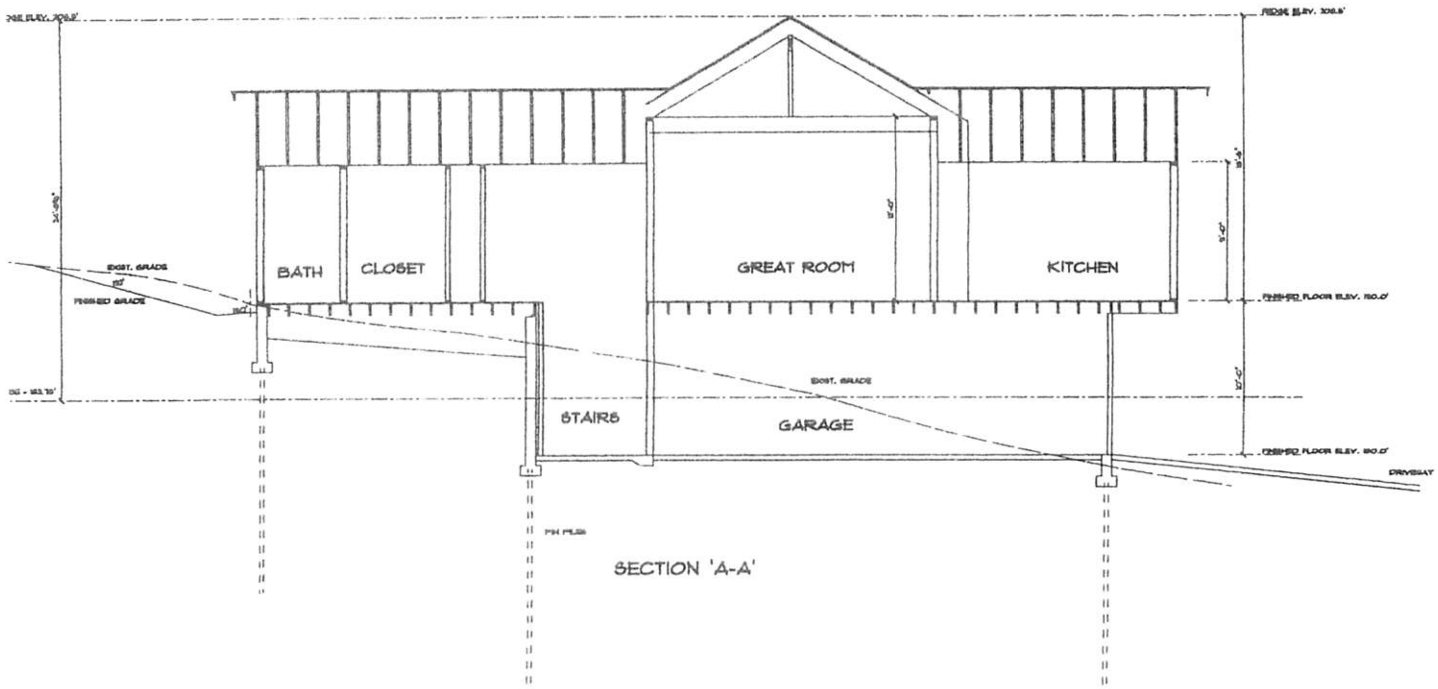
GEO Group Northwest, Inc.

Geotechnical Engineers, Geologists, & Environmental Scientists

PROPOSED DEVELOPMENT PLAN

PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE	1" = 30'	DRAWN BY	KJ	CHECKED BY	WC	DATE	3/10/2015	PROJECT NO.	G-3827	PLATE	3



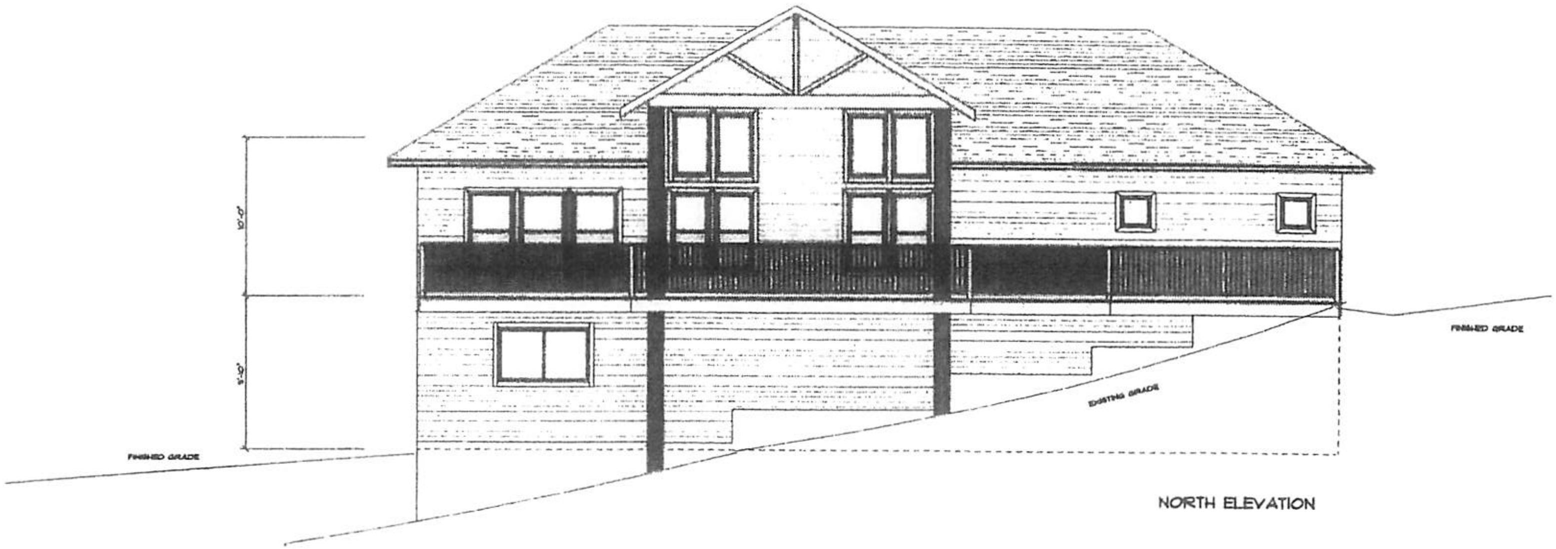
Source: Section A - A' by Healy-Jorgensen Architects, dated 10/1/14.

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

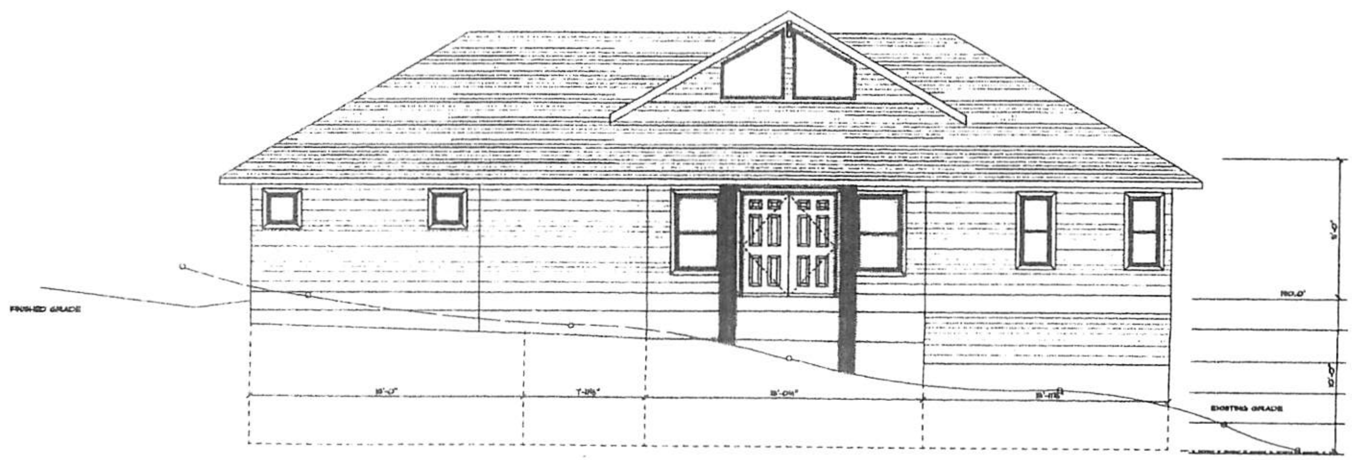
PROPOSED RESIDENCE SECTION

PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE	1" = 8'	DRAWN BY	KJ	CHECKED BY	WC	DATE	3/10/2015	PROJECT NO.	G-3827	PLATE	4



NORTH ELEVATION



SOUTH ELEVATION

Source: North - South Elevations, by Healy-Jorgensen Architects, dated 10/1/14.

GEO Group Northwest, Inc.

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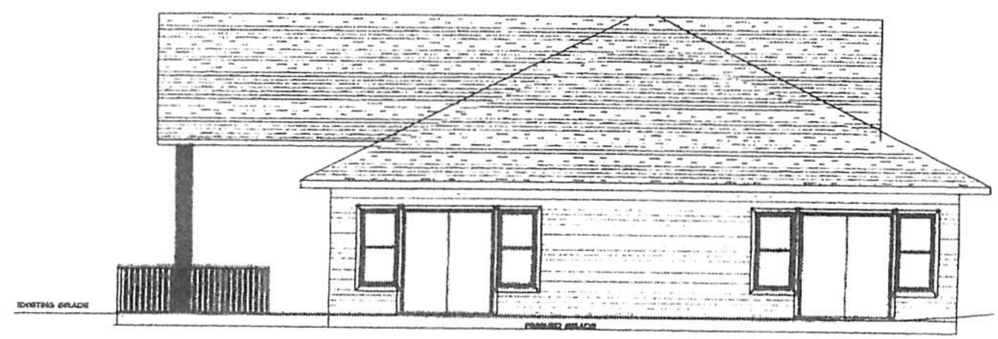
NORTH & SOUTH ELEVATIONS

PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE	1" = 8'	DRAWN BY	KJ	CHECKED BY	WC	DATE	3/10/2015	PROJECT NO.	G-3827	PLATE	5A

EAST ELEVATION



WEST ELEVATION

Source: East - West Elevations, by Healy-Jorgensen Architects, dated 10/1/14.

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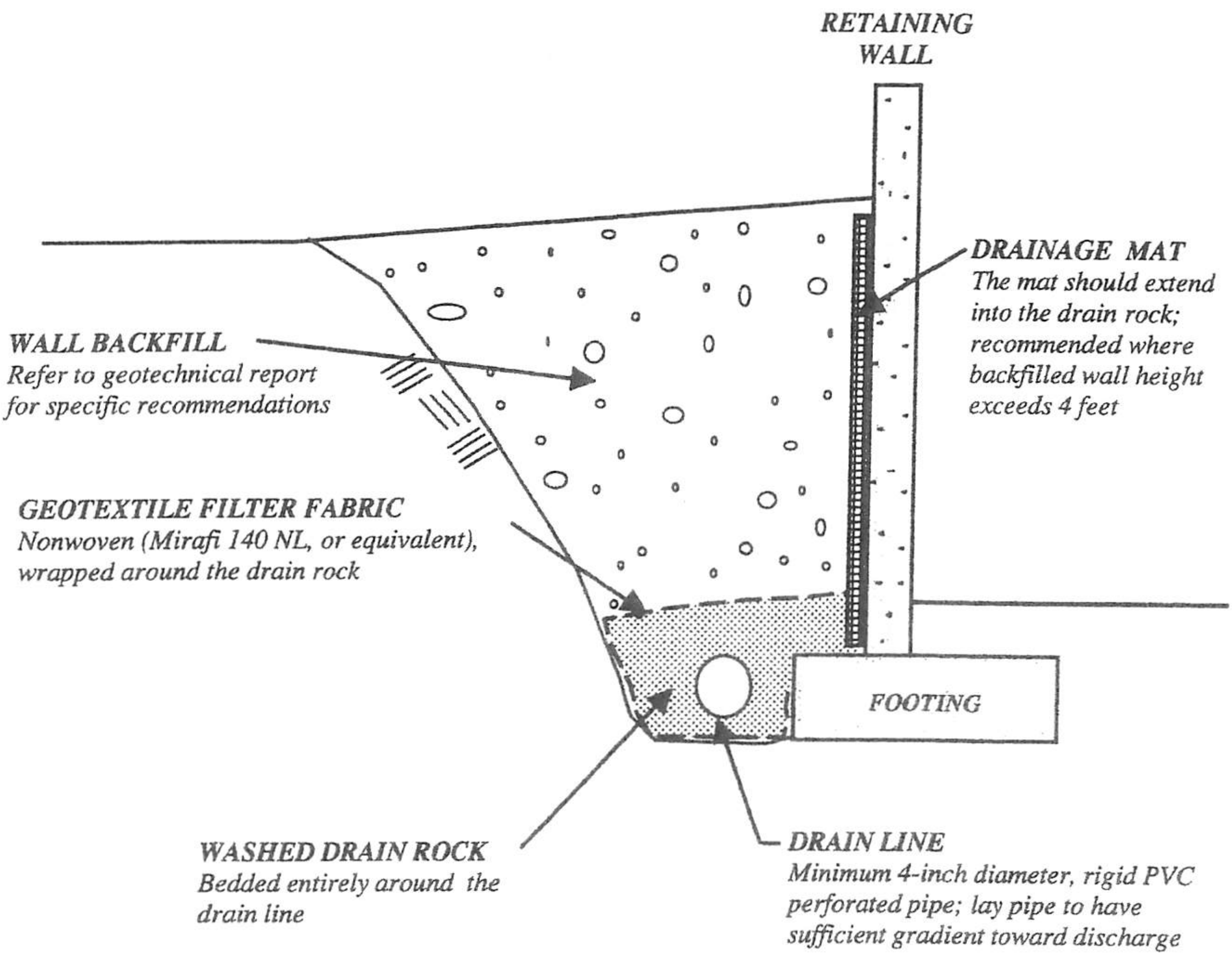
EAST & WEST ELEVATIONS
PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE	1" = 8'	DRAWN BY	KJ	CHECKED BY	WC	DATE	3/10/2015	PROJECT NO.	G-3827

PLATE 5B



NOT TO SCALE

NOTES:

1.) Do not replace rigid PVC pipe with flexible corrugated plastic pipe.

2.) Perforated PVC pipe should be tight jointed and laid with perforations oriented downward. The pipe should be gently sloped to provide flow toward the tightline or discharge location.

3.) Do not connect other drain lines into the footing drain system.

4.) Backfill should meet structural fill specifications if it will support driveways, sidewalks, patios, or other structures. Refer to the geotechnical engineering report for structural fill recommendations.

GEO Group Northwest, Inc.
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TYPICAL BASEMENT AND RETAINING WALL BACKFILL AND DRAINAGE
PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE NONE	DATE 3/11/2015	MADE KJ	CHKD WC	JOB NO. G-3827	PLATE 6

ATTACHMENT A

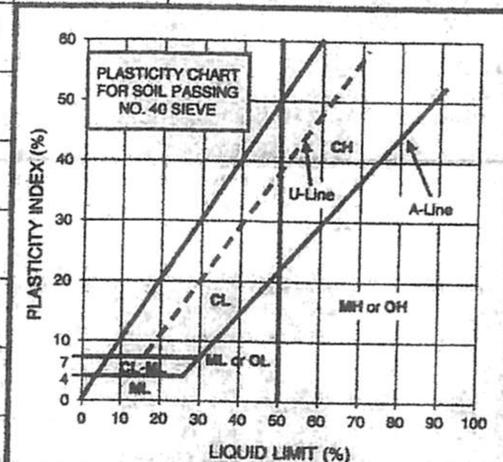
G-3827

BORING LOGS

SOIL CLASSIFICATION & PENETRATION TEST DATA EXPLANATION

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS	GRAVELS (More Than Half Coarse Fraction is Larger Than No. 4 Sieve)	CLEAN GRAVELS (little or no fines)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURE, LITTLE OR NO FINES	CONTENT OF FINES BELOW 5%	Cu = (D60 / D10) greater than 4 Cc = $(D30)^2$ / (D10 * D60) between 1 and 3
			GP	POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES LITTLE OR NO FINES		CLEAN GRAVELS NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (with some fines)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	GM: ATTERBERG LIMITS BELOW "A" LINE, or P.I. LESS THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		GC: ATTERBERG LIMITS ABOVE "A" LINE, or P.I. MORE THAN 7
More Than Half by Weight Larger Than No. 200 Sieve	SANDS (More Than Half Coarse Fraction is Smaller Than No. 4 Sieve)	CLEAN SANDS (little or no fines)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	CONTENT OF FINES BELOW 5%	Cu = (D60 / D10) greater than 6 Cc = $(D30)^2$ / (D10 * D60) between 1 and 3
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		CLEAN SANDS NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (with some fines)	SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE with P.I. LESS THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE with P.I. MORE THAN 7
FINE-GRAINED SOILS	SILTS (Below A-Line on Plasticity Chart, Negligible Organics)	Liquid Limit < 50%	ML	INORGANIC SILTS, ROCK FLOUR, SANDY SILTS OF SLIGHT PLASTICITY		
		Liquid Limit > 50%	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOIL		
	CLAYS (Above A-Line on Plasticity Chart, Negligible Organics)	Liquid Limit < 50%	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, CLEAN CLAYS		
		Liquid Limit > 50%	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
Less Than Half by Weight Larger Than No. 200 Sieve	ORGANIC SILTS & CLAYS (Below A-Line on Plasticity Chart)	Liquid Limit < 50%	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		Liquid Limit > 50%	OH	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS		



SOIL PARTICLE SIZE

FRACTION	U.S. STANDARD SIEVE			
	Passing		Retained	
	Sieve	Size (mm)	Sieve	Size (mm)
SILT / CLAY	#200	0.075		
SAND				
FINE	#40	0.425	#200	0.075
MEDIUM	#10	2.00	#40	0.425
COARSE	#4	4.75	#10	2.00
GRAVEL				
FINE	0.75"	19	#4	4.75
COARSE	3"	76	0.75"	19
COBBLES	76 mm to 203 mm			
BOULDERS	> 203 mm			
ROCK FRAGMENTS	> 76 mm			
ROCK	>0.76 cubic meter in volume			

GENERAL GUIDANCE FOR ENGINEERING PROPERTIES OF SOILS, BASED ON STANDARD PENETRATION TEST (SPT) DATA

SANDY SOILS				SILTY & CLAYEY SOILS		
Blow Counts N	Relative Density, %	Friction Angle ϕ, degrees	Description	Blow Counts N	Unconfined Strength q_u, tsf	Description
0 - 4	0 -15		Very Loose	< 2	< 0.25	Very soft
4 - 10	15 - 35	26 - 30	Loose	2 - 4	0.25 - 0.50	Soft
10 - 30	35 - 65	28 - 35	Medium Dense	4 - 8	0.50 - 1.00	Medium Stiff
30 - 50	65 - 85	35 - 42	Dense	8 - 15	1.00 - 2.00	Stiff
> 50	85 - 100	38 - 46	Very Dense	15 - 30	2.00 - 4.00	Very Stiff
				> 30	> 4.00	Hard

GEO Group Northwest, Inc.
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13240 NE 20th Street, Suite 10 Bellevue, WA 98005
Phone (425) 649-8757 Fax (425) 649-8758

PLATE A1

BORING NO. B-1

Logged By: KJ Date Drilled: 8/10/1999 Surface Elev. 187 feet +/-

Depth ft.	USCS Code	Description	Sample Type	Sample No.	Blow Count per 6-inches	Water Content %	Other Tests & Comments
	OL	Organic topsoil, very soft, wet, black.	I	S1	1,1,1 (N=2)	44.4	
	SM	SILTY SAND, very loose, wet, fine grained sand, 20-25% fines, trace black organics, occasional gray lenses, brown.	I	S2	1/12",1 (N=1)	27.0	
5	SP-SM	SAND, loose, wet, 10% fines, fine grained, mottled gray and brown.	I	S3	1,2,3 (N=5)	28.0	
	SP-SM	As above, medium dense, 5-10% fines.	I	S4	5,6,6 (N=12)	29.2	
10	SP-SM	As above, 2.5 feet of sand heave into hole.	I	S5	5,6,9 (N=15)	27.9	
15	SM	SILTY SAND, medium dense to dense, moist to wet, 20% fines, very fine to fine grained sand, brownish gray.	I	S6	9,15, 16,28 (N=31*)	25.8	* = Blow counts may be affected by sand heave.
20		Bottom of boring: 17 feet. Drilling Method: Hollow-stem auger 0 to 17 feet. Sampling Method: 2-inch-O.D. standard penetration sampler driven using a 140 lb. hammer with a 30-inch drop. Groundwater encountered near ground surface during drilling. Boring backfilled with bentonite chips.					
25							
30							
35							
40							

LEGEND:
- I 2" O.D. Split-Spoon Sampler
- II 3" O.D. Shelby-Tube Sampler
- III 3" O.D. California Sampler

GROUNDWATER OBSERVATION WELL: seal; measured water level; well tip (screen)

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG
PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

JOB NO. G-3827 | DATE 3/11/2015 | PLATE A2

BORING NO. B-2

Logged By: KJ   Date Drilled: 8/10/1999   Surface Elev. 176 feet +/-

Depth ft.	USCS Code	Description	Sample Type	Sample No.	Blow Count per 6-inches	Water Content %	Other Tests & Comments
	OL	Very soft, moist, black, organic topsoil and red decomposed wood, poor sample recovery.	I		1/18" (N=0)		Poor recovery.
5	SP-SM	SAND, loose, wet, fine to medium grained, 10-15% fines, rust-colored oxide staining, some black organics, brown.	I	S1	1,2,2 (N=4)	34.6	
	SP-SM	As above, loose.	I	S2	4,3,5 (N=8)	23.6	
10	SP-SM	As above, medium dense, trace coarse sand.	I	S3	4,7,9 (N=16)	21.4	
	SP	As above, loose, 5% fines, fine grained, grayish brown.	I	S4	4,4,4 (N=8)	27.4	
15	SM	SILTY SAND, loose, wet, fine to medium grained sand, 20-25% fines, trace small wood chips, rare coarse sand, trace reddish oxide staining, dark gray.	I	S5	3,2,3 (N=5)	23.8	
20	ML	SILT, stiff, damp to moist, trace fine sand, contains wet sand lenses, dark gray.	I	S6	5,11,12 (N=23)	30.6	
25	ML	As above, occasionally laminated (some brown laminae and organics, some wet sand lenses.	I	S7	5,9,10 (N=19)	28.1	
30		Bottom of boring: 27 feet. Drilling Method: Hollow-stem auger 0 to 27 feet. Sampling Method: 2-inch-O.D. standard penetration sampler driven using a 140 lb. hammer with a 30-inch drop. Groundwater encountered near ground surface during drilling. Boring backfilled with bentonite chips.					
35							
40							

LEGEND:
- I 2" O.D. Split-Spoon Sampler
- II 3" O.D. Shelby-Tube Sampler
- III 3" O.D. California Sampler

GROUNDWATER OBSERVATION WELL: seal; measured water level; well tip (screen)

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BORING LOG

PROPOSED RESIDENCE
5637 E. MERCER WAY
MERCER ISLAND, WASHINGTON

JOB NO. G-3827 **DATE** 3/11/2015 **PLATE** A3