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June 8, 2022

JN 20408

Bill Summers
via email: billsummers1841@gmail.com

Subject: **Response to Geotechnical Peer Review Comments Regarding 2204-107-SUB1-PLANS**
Proposed Mercer Island Treehouse Residence
5637 East Mercer Way
Mercer Island, Washington

References: *Geotechnical Engineering Study, Proposed Residence, 5637 East Mercer Way, Mercer Island, Washington*; GeoGroup NW; March 12, 2015.

Response to September 3, 2015 Geotechnical Third Party Review Letter, Proposed Residence, 5637 East Mercer Way, Mercer Island, Washington; GeoGroup NW; October 28, 2015.

Geotechnical Report Addendum, Potential Adverse Impacts to Adjacent and Downhill Properties, 5637 East Mercer Way, Mercer Island, WA 98040; GeoGroup NW; May 3, 2017.

Geotechnical Engineering Assessment of Landslide Hazard Mitigation, Proposed Mercer Island Treehouse Residence, 5637 East Mercer Way, Mercer Island, Washington; Geotech Consultants, Inc.; December 3, 2020.

Geotech Consultants, Inc. provided geotechnical input and support during the last phase of the Reasonable Use process on this project, documenting our geotechnical conclusions about site stability and the planned development in our December 3, 2020 letter, a copy of which is attached.

Our firm is now acting as the Geotechnical Engineer of Record during the permit process, using not only the above-referenced information prepared previously by GeoGroupNW, but also our own site observations and calculations.

The following are our responses to the comments made by the City of Mercer Island's geotechnical third party reviewer:

Page: A1.0 GEN NOTES
Number: 2 *Geotechnical engineer of record to review the plan set and provide a letter confirming that the geotechnical design elements conform to their design recommendations. Provide an updated statement of risk in accordance with MICC 19.07.160.B.3*

Response: The plans that have been prepared include excavation shoring, deep foundations, and landslide catchment, which are all appropriate element for the site subsurface and topographic conditions. Considering the geotechnical-related comments we make in this letter:

> *The development practices proposed for the alteration would render the development as safe as if it were not located in a geologic hazard area.*

<u>Page: P1.0 PILE DETAILS</u>
Number: 1 *Provide geotechnical engineering study dated February 14, 2016 for review of recommendations for lateral earth pressures used in the design of the shoring and catchment walls. If the report does not provide basis for height of the catchment wall and calculation of the debris flow loading, please provide that in a letter by the geotechnical engineer of record.*

Response: From our discussions with Stoney Point Engineering, we understand that this date for the design geotechnical report they utilized is incorrect. The correct date of the GeoGroupNW report is March 12, 2015.

The note on the sheet P1.0 will be corrected accordingly. We expect that the City of Mercer Island has been provided a copy of this report, but we have attached a copy, just in case.

Soldier pile walls extending 8 feet above the existing grade have been included in the project design to provide permanent landslide catchment along the south and west, upslope, sides of the house. This steep slope to the south of the house is only 20 feet in height at its maximum, and poses the least hazard to the residence from potential soil movement. The steep slope to the west is much taller, but is comprised of more competent soils. Based on the observed conditions, and experience from observing many landslides, it is our professional opinion that the most critical design condition for the catchment wall would be a shallow skin slide occurring on the western steep slope. Typically, such a slide will affect the uppermost approximately 2 feet of looser, weathered soil. A slide length of 20 to 30 feet is not uncommon. This type of a slide is most likely to occur following extended wet weather.

Attached to this letter is an estimation of a potential slide volume assuming a 2-foot slide thickness over a 30-foot height of the slope. This amounts to a volume of approximately 2.2 cubic yards of slide material per foot width. Including a 6-foot height of accumulated slide debris, and the approximate 10-foot distance between the catchment wall and the toe of the steep slope, there is a potential catchment volume of to 7.5 cubic yards per foot width to accumulate without reaching the top of the 8-foot wall. This a factor of over 3 above the estimated slide volume. before reaching the house.

Since the mid- to late-1990's, when there were many landslides in the Puget Sound area, geotechnical engineers in the Seattle area have typically used either 80 pounds per cubic foot (pcf) or 100 pcf as the soil load for the design of the above-grade portion of catchment walls. The lower pressure was used where there was a relative short slope, or one that was not excessively steep. From our more recent discussions in the past few years with the geotechnical staff at the City of Seattle, we understand that recent publications have shown that a 100pcf loading more closely estimates impact loading from flow-type slides. As a result, we recommend that this value be assumed for the design earth pressure against the catchment walls. However, we recommend that the design catchment height be 6 feet, instead of 8. Leaving the piles extending to a heigh of 8 feet above the ground is still prudent, providing an extra measure of protection for the house.

Number: 2 *It is unclear what equivalent fluid pressure is being used to determine the passive resistance. Provide a value on the diagram. Does that value consider submerged soil conditions?*

If not, geotechnical engineer to provide recommended values. Revise structural design accordingly.

Response: For the soil conditions expected below the planned excavation level, we recommend that an ultimate (no safety factor included) passive earth resistance of 225 pounds per cubic foot (pcf) be assumed for the loose saturated sand soils below the base of the excavation. This passive resistance can be applied over two times the drilled pile diameter.

Page: P1.1 PILE DETAILS
Number: 1 *Isn't this retaining a fill? Wouldn't it be easier to provide drainage and piping behind the wall?*
Provide details on how this wall will be drained or design facing for hydrostatic pressures.

Response: This wall shown on detail 3/P1.1 will be a backfilled soldier pile wall extending along the north side of the driveway. This is also depicted on the North Elevation on sheet A3.4. The driveway area behind the wall will be covered with pavement, preventing infiltration of precipitation into the backfill zone. Groundwater will be below the backfill zone, within the native soils. As a result, the potential for any significant amounts of subsurface water behind the above-grade portion of the wall is low. Drainage of any minimal amounts of water that may accumulate in the backfill could be handled either by a footing drain installed below the existing grade behind the soldier piles, or by installing weep holes through the base of the concrete wall facing. Two-inch-diameter weep holes on 6-foot spacing would be appropriate. If the soldier pile wall is constructed without a concrete facing, the treated timber can be backfilled with clean gravel, and any small amounts of subsurface water will simply weep out through the lagging.

Page: P2.0 PILE PLAN
Number: 1 *The wall located adjacent to sloping ground conditions should be designed with reduced passive pressures. Geotechnical engineer to provide recommended values to take into consideration these sloping ground conditions. This value should also take into account saturated soil conditions.*
Revise structural design accordingly.

Response: The slope in front of the northern driveway soldier pile retaining wall is located above a short slope only 5 feet in height. Rather than utilizing a reduced passive pressure on the embedded portion of the piles, we recommend that the piles be designed for an active earth pressure acting to 2.5 feet (one-half of slope height) below the existing grade. The passive pressure below this 2.5-foot depth should be an ultimate value (no safety factor included) of 225 pounds per cubic foot (pcf) acting on 2 times the drilled pile diameter. This passive value accounts for the buoyant unit weight of the soil below the water table.

Number: 2 *Are you backfilling here as shown in Detail 1/P1.1? Are you providing a wall to return? Or permanent slope fill past the end of the shoring wall? Provide final grading if fill slope is proposed.*
Revise plan sheet.

Response: The retaining wall indicated in this comment is actually a landslide catchment wall consisting of soldier piles that extend above the ground surface. The base of the catchment wall will follow the existing grade, and will not retain fill above the existing grade.

There is no need to revise the plan sheet for this item.

Page: S2.0 FDN PLAN
Number: 1 *The geotechnical report recommended a structurally supported slab. Resolve discrepancy.*

Response: The note on S2.0 clearly indicates that the concrete floor slab would be a structural slab that will span between the pile-supported foundations. The slab it is 6 inches thick and reinforced with a mat of rebar. The term “Slab On Grade” used before “6” Concrete Structural Slab” simply refers to the fact that the structural slab will be poured on soil, rather than being elevated.

There is no need to modify the plan for this item.

Number: 2 *The geotechnical report dated 3/12/2015 indicated that the site soils are liquefiable. The geotechnical engineer should assess to what depth these soils are liquefiable. What is the estimated post-liquefaction settlement? What is the potential for lateral spreading or flow failure? What magnitude of lateral deformation is estimated?*
Include discussion of how the post-liquefaction condition impacts the proposed structural design, for example, piling embedments, lateral earth pressures associated with a liquefied soil condition, lateral loading associated with flow failure or lateral spreading, etc.
Provide mitigation recommendations for consideration by the project team.
Provide this information with supporting calculations and stability analyses in a report addendum.

Response: The potential for liquefaction and resulting ground settlement or soil bearing strength loss has been studied for many years, but it is still impossible to accurately determine where, and to what extent, liquefaction could/will occur. However, liquefaction of the loose sand soils that lie beneath the water table has a high probability during the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years.

Using NovoLIQ a maximum ground settlement of approximately 8 inches was calculated under the MCE. The results of this analysis are attached. The amount of actual ground settlement that could occur as a result of liquefaction will vary with differing soil conditions, and the magnitude, duration, and predominant direction of ground shaking associated with an earthquake.

The pipe piles supporting the new residence will be driven to refusal in dense, non-liquefiable soils. Small-diameter pipe piles are not displacement piles, and their compressive capacity is entirely dependent on end bearing in the dense to very dense glacially-compressed soils they are driven into. Tens of thousands of load tests have been completed throughout Seattle and the remainder of the Puget Sound region by our firm and others using ASTM D-1143, or similar testing methods. These load tests have proven that small-diameter pipe piles driven to refusal rates appropriate for the hammer size have an ultimate capacity of 200-percent, or more, of the recommended design allowable capacities.

The potentially liquefiable soils encountered in the borings below the water table provide no compressive support to the pipe piles in the event of seismic liquefaction. Therefore, any potential loss of strength in these liquefiable soils will not reduce the compressive capacity of the pipe piles.

The potential for lateral spreading is even less understood than liquefaction itself. However, some methods have been developed to estimate the potential amount of lateral ground movement that could occur where liquefiable sites lie on sloping ground. Unfortunately, these methods apply mostly to larger sites, such as waterfront facilities. Using three different methods, NovoLIQ provides estimates for this lateral movement. Unfortunately, as noted above, the potential for lateral spreading has to be evaluated for a low probability earthquake (once in 2,475 years), which has very high design ground accelerations. Also, it has been common that whenever a new code is adopted, the design earthquake motions get stronger and stronger, resulting in more theoretical ground movement.

The results of our lateral spreading analyses, which are attached, indicate that lateral ground movement of 12 to 19 feet could theoretically occur in the MCE. The theoretical lateral movements are large enough that no conventional pile system, drilled or driven, can prevent them from occurring, or can withstand the potential lateral movements without shearing off. It is important to note that these large potential movements are calculated using available methods are likely substantially overestimated for this site. The property lies at the upgradient boundary of any potential lateral spreading. The steep slope immediately west of the planned house is comprised of glacially-compressed soils that would not be affected by lateral spreading or deep instability in the event of the MCE. Also, the stability of the home site will be improved by the presence of the pipe piles and soldier pile walls, and the localized dewatering around and beneath the house, further reducing the potential for large lateral ground movements during the low probability earthquake.

From a geotechnical standpoint, preventing lateral spreading to any significant depth on a single residential lot is not possible or practical. In our professional opinion, the most appropriate mitigation against foundation collapse in the event of lateral spreading is achieved by interconnecting the piles with reinforced grade beams. In the event that the ground moves sideways a sufficient distance to bend or break the piles, the grade beams would serve to hold the structure in one piece, even if it tilts a significant amount.

Number: 3 *When the post earthquake assessment (liquefaction, settlement and lateral deformation) is provided by the geotechnical engineer, the structural engineer shall review and assess whether the proposed structural design can accommodate these settlements, lateral deformations and lateral loads without building collapse.*
Revise structural design accordingly.

Response: As discussed above, there is no way to accurately predict the amount of lateral spreading or differential ground movement that could occur. Grade beams, such as those that will interconnect the piles, have been shown to be able to span large distances without soil support. Regardless of the amount of horizontal or vertical ground movement, by holding the foundations together so that the structure can move as a unit, the intent of the Code (preventing foundation collapse) should be satisfied.

Number: 4 *The geotechnical report did not recommend shallow foundations but the use of pin piles or helical anchors for foundation support. Resolve discrepancy. Verify that retaining walls are designed for seismic loading.*

Response: The upper soils are loose, wet sands and should not be depended on to support retaining walls more than a few feet in height. Based on the expected heights of the walls to be installed along the south side of the driveway, they should be supported on piles.

The structural plans will be modified accordingly.

<u>Page: 1</u>
Number: 1 *The foundation plan does not include foundation drain locations.*
Given the seepage noted on the site there will be a need to provide proper drainage for all site shoring and retaining walls as well as perimeter drains. Provide a plan showing where these drains are anticipated and how they will connect to the storm water drainage system.

Response: Most of the foundation details show footing drains in cross-section. Subsurface drainage will definitely be important for this project, and numerous footing drains are shown for the house. However, this is common for most single-family projects and the requirement to provide a formal drainage plan is excessive. This would be more consistent with a larger multi-family or commercial development. The footing drains to be installed would simply be collected and then be tightlined around the detention tank to connect with the outlet pipe that will discharge into the stream. This is something that earthwork and drainage contractors are very familiar with.

Page: CRITICAL AREA MITIGATION
Number: 1 *Geotechnical engineer to review the proposed plantings in the steep slope areas and identify potential negative impacts of proposed plantings with respect to stability of the slopes. Provide recommendations for mitigation of potential negative impacts, if appropriate.*
Revise plan accordingly.

Response: The provided plans indicate that trees (cedars) would be planted on the steep slope areas to the west and south of the planned residence. While evergreen vegetation on steep slopes can provide some stability benefits through precipitation interception, it is our professional geotechnical opinion that this should be limited to low-growing evergreen vegetation. Trees should not be planted on steep slopes, particularly upslope of existing or proposed occupied structures. Planting the trees requires excavation that will disturb the slopes. The holes created to plant the root balls will tend to collect water, which will decrease shallow slope stability. More concerning is the dead weight that will result from the trees for a substantial length of time until they are deeply rooted into dense soil. Additionally, in the event of potential future slope instability, the trees increase the risk of substantial damage to the residence and pose a real risk to the occupants. This risk increases as the trees become larger and heavier.

It is important to note that the tall, steep slope to the west of the site is already covered with mature evergreen trees.

We recommend the following:

1. Minimize the number of trees to be included in any wetland restoration on the site, and place them only in areas away from steep slopes and the planned development area.
2. Maximize the use of low-growing evergreen plants that would be water-loving.
3. Utilize live staking as much as practical to plant amongst existing vegetation and brush outside of the areas close to the planned development area.

Please contact us if there are any questions regarding this letter.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Marc R. McGinnis, P.E.
Principal

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Attachments:

- December 3, 2020 Letter by Geotech Consultants, Inc.
- October 12, 2015 Geotechnical Report by GeoGroupNW
- Soil Liquefaction and Lateral Spreading Analysis Report from NovoLiq
- Cross-Section for Catchment Volume

cc: **Healey Alliance**
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December 3, 2020

JN 20408

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via email: billsummers1841@gmail.com

Subject: **Geotechnical Engineering Assessment of Landslide Hazard Mitigation**
Proposed Mercer Island Treehouse Residence
5637 East Mercer Way
Mercer Island, Washington

References: *Geotechnical Engineering Study, Proposed Residence, 5637 East Mercer Way, Mercer Island, Washington*; GeoGroup NW; March 12, 2015.

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Geotechnical Report Addendum, Potential Adverse Impacts to Adjacent and Downhill Properties, 5637 East Mercer Way, Mercer Island, WA 98040; GeoGroup NW; May 3, 2017.

Response to Shannon & Wilson Third Party Review, RE: Proposed Residence, 5637 East Mercer Way, Mercer Island, Washington 98040; GeoGroup NW; October 23, 2019.

Architectural Plans (The Healey Alliance AZ, June 25, 2020) and Structural Plans (Stoney Point Engineering, March 30, 2020).

Boundary and Topographic Survey, Core Design, August 31, 2020.

At your request, Geotech Consultants, Inc. has completed an independent geotechnical review of the measures that have been incorporated into the planned Mercer Island Treehouse development to mitigate the geologic hazards not only to the proposed residence, but also to the neighboring properties surrounding the site.

In order to complete this assessment, we completed the following tasks:

- Visited the site on November 3, 2020 to assess conditions on the subject property and the adjoining lots,
- Reviewed the above-referenced documents,
- Reviewed our project files for geotechnical and geologic information from previous experience on nearby sites,
- Researched the Mercer Island GIS for Critical Area mapping,
- Reviewed the Department of Natural Resources' *Geologic Information Portal* for geologic mapping of the site vicinity, and
- Reviewed the *Mercer Island Landslide Hazard Assessment* (Troost & Wisher, 2009).

Project Description

Based on the project plans, the site development will consist of a two-story residence with an east-facing daylight basement underlying approximately two-thirds of the house's footprint. This basement level will contain the garage. A new paved driveway will extend to the garage from the existing driveway that curves through the southeastern corner of the lot to serve the adjacent southern residence (#5645). The development area is constrained by an east-flowing watercourse that extends through the northern portion of the lot, and by steep slopes located along the west and south sides of the property. The planned residence will be sited in the center of the lot, where the existing ground surface slopes gently to moderately. No development, or even disturbance, is planned for of the steep slopes that rise to the west and southwest to homes along Southeast 57th Street. The provided structural plans show that significant structural considerations have been incorporated to deal with the site geologic and topographic conditions. The house to be supported on piles driven into the underlying glacially-compressed soils. Additionally, soldier pile shoring will be used to provide temporary support for the basement excavation cuts until the permanent foundation walls have been completed. Soldier piles will also be installed for the excavation to create the small motorcourt/parking area to the east of the house. These soldier piles will restrain the cuts needed into the short steep slope that rise to the neighboring southern property. The upslope (south and west) foundation walls will be extended above the surrounding ground surface to provide landslide catchment/diversion in the event of future slides moving down the neighboring steep slopes.

We expect that extensive temporary and permanent drainage will be installed as a part of this project. The provided project plans indicate that runoff from impervious surfaces in the development area will initially be collected in a detention tank, and then will be discharged at a reduced rate. The natural discharge point for this water is the watercourse that runs along the north side of the development area. All precipitation falling within the planned development area currently infiltrates into the ground to add to the flow in the watercourse.

Geologic Setting and Landslide Hazard Assessment

From our site observations, and review of topographic information provided not only in the project plans, but also on Mercer Island's GIS system, it is apparent that the subject site occupies the base of an east-trending ravine. This ravine feature starts many lots to the west, near 91st Avenue Southeast, and extends east to the old shore of Lake Washington. There are numerous similar ravines along the eastern side of Mercer Island, and they were formed largely from heavy flows of post-glacial runoff traveling down the sideslopes of Mercer Island when the last glaciers receded over 10,000 years ago. Now, this ravine serves to carry surface runoff and groundwater seepage, as well as runoff from impervious surfaces (roads, roofs, driveways, etc.) that are generally located in the same storm drainage basin. Downstream of the site, the watercourse flows through a culvert underneath East Mercer Way to continue eastward to Lake Washington.

The soft/loose upper soils found in GeoGroup NW's borings are consistent with alluvial soils that have been deposited in the base of the ravine by water flow and erosion, and potentially previous slides on the steep sideslopes of the ravine. The unconsolidated condition of these soils is evident simply from walking around the development area, where we could easily push our T-probe into the soil to its full 4-foot length with minimal effort. As verified by GeoGroup's borings, these alluvial soils are underlain by glacially-compressed soils. This is consistent with the geologic mapping of the area, which shows glacial drift or glacial outwash soils.

It was not necessary for us to cross onto the adjacent western and southwestern properties to observe the conditions on the slope. We could assess the slope conditions from the western property line of the Mercer Island Treehouse property, and from the trail in the adjacent northern Parkwood Ridge Open Space. The steep slopes rising to the west and southwest from the building site on the Mercer Island Treehouse property are 90 to 100 feet in height. Based on available topographic information from the *Boundary and Topographic Survey*, and our on-site measurements with a hand-held clinometer, the steep slopes within the property boundaries are inclined at approximately 50 percent. However, the heavily-treed, steeper slope to the west southwest is inclined at 65 to 75 percent. The slopes to the west and southwest of the site are heavily treed with large evergreen trees. We were able to observe the steep slope west and southwest of the site over its full height. Based on anecdotal information provided, and review of the *Mercer Island Landslide Hazard Assessment*, there has been previous landsliding behind the adjacent western homes, likely near the top of the steep slope. There were no obvious indications of recent instability that we could observe. While deciduous trees on the slope displayed their typical curved trunks, there were no signs that this curvature was related to slope movement. The evergreen trees, which will typically grow with straight trunks, did not display the multiple curves in their trunks that would be indicative of deeper slope movement. In fact, there are some very large evergreen trees on the slope that have no curvature to their trunks at all. We did observe some of the typical “pistol butting” of the base of some of the trees. This is typical on steep slopes, where seedlings can be tipped sideways by shallow soil creep, falling branches, etc. before they are bigger and deeply rooted. This causes a curve or “pistol butt” in the base of the trunk, while the remainder of the evergreen tree then grows straight upward. We also saw stumps of old growth evergreen trees in, and around, the planned development area, a further testament to the deep stability of the area.

It is important to realize that the soil conditions comprising the steep slopes rising to the west and southwest of the site are substantially different, and more stable, that those found in the development area in the base of the ravine. The geologic mapping found on the *Geologic Information Portal* confirms that the upland area along Southeast 57th Street, as well as the steep slopes below the homes on that street, is underlain by Glacial Till. This soil is a glacially-compressed mixture of gravel, silt, and fine-grained sand. It is cemented, and is often referred to as hardpan. Glacial Till has a very high internal strength, often allowing tall vertical banks to stand for many, many years with only limited spalling off the face of the bank. This is evident throughout the Pacific Northwest not only in marine bluffs, but also in manmade excavations, such as those made for roads. Our observation of the conditions on the steep slopes extending west and south of the development site showed established underbrush and numerous mature trees on the slopes. Glacial Till soils are not susceptible to deep-seated instability, even on the steeply-inclined natural slopes around the site.

That is not to say that landslides cannot occur on steep slopes underlain by Glacial Till. Over time, which can take 30+ years, the near-surface few feet (typically 2 feet) of soil naturally weathers and loosens by freeze-thaw effects. This loosened layer, combined with the topsoil and duff that can accumulate, periodically slides down a steep slope, usually following extended wet weather. Unfortunately, man’s actions (improper discharge of runoff, placement of uncontrolled fill on or near a slope, or leaking utilities) can increase the likelihood, or be the sole cause, of landslides in these soil conditions. We have been associated with numerous slides on Mercer Island steep slopes that were directly related to improper development practices used when properties were developed above steep slopes. These often revolved around the common, and improper, practice of placing uncompacted and unretained soil over steep slopes to create flatter areas for yards and landscaping. Our review of the *Mercer Island Landslide Hazard Assessment* confirms that there have been documented slides on the steep slopes to the west and south of the planned development, and that is no surprise. However, for the reasons discussed above, we expect the natural slides to have been relatively localized and confined to the near-surface few feet of weathered soil. Larger slides, especially those that may have affected rear yards, decks, landscaping, etc. of the upslope homes, likely involved improperly placed or unretained fill.

The undersigned project engineer has also been associated with the recent slide that affected the eastern slope below East Mercer Way at 5368 East Mercer Way, approximately 400 feet to the east of the Mercer Island Treehouse property. This slide occurred on November 28, 2020. Similar to the slides discussed above, this recent landslide was shallow, affecting uncontrolled fill and weathered soils above the dense, glacially-compressed soil. It appears to have been triggered by excessive water within the looser soils.

Geotechnical Conclusions

Development of the subject property, while challenging, can be accomplished safely, without risk to surrounding properties. Anyone familiar with development on Mercer Island is aware of numerous sites that have been successfully developed in, and near, ravines and steep slopes. Our firm has been involved with many such projects over its 34+ year history. The geotechnical measures of shoring, slide catchment, and foundation piles recommended by GeoGroup NW which have been included in the project are appropriate to protect the planned residence and its occupants from the geologic hazards associated with the site.

The geotechnical measures incorporated into the plans at the recommendation of GeoGroup NW are appropriate to prevent adverse impacts to the stability of the site and the surrounding properties. These measures are significant and costly, but are needed to accommodate the geologic constraints of the property and surrounding lots. The planned shoring is necessary to support the unconsolidated, loose soils for the excavation of the house. The loose soils in the building area provide no significant lateral support for the glacially-compressed materials that comprise the steep slopes to the west and south. Removal of the loose sediments would not cause instability in the glacially-compressed soils of the steep slopes. Even so, the excavation shoring that will be installed to facilitate the excavation of the below-grade portion of the structure will provide lateral support for the base of the steep slopes that exceeds what currently exists. This shoring will also minimize the amount of excavation necessary for the project by preventing the need for temporary cut slopes extending outside the footprint of the structure.

Including the slide catchment wall into the design of the house will provide protection against damage that could result from slide debris reaching the structure. Also, by eliminating the need for a separate, free-standing wall, the amount of site disturbance and excavation will be reduced.

The potential for future shallow instability on the steep slopes that extend up to the neighboring west and south properties will not be increased by the planned development. The slopes are comprised of competent, glacially-compressed soils. The trees and underbrush on these slopes will remain, and no excavation into the steep slopes themselves will occur. Again, as discussed above, support for the loose soils at the bottom of the slope will be improved by the shoring and permanent below-grade walls of the new residence.

The planned development will not pose a risk to the neighboring houses. The excavation for the new house will be quite distant from all neighboring houses, even the one immediately south at #5645. These structures do not count on lateral support from the soft/loose soils that will be removed for the new house's construction. From a practical standpoint, if these houses were, in fact, supported by the loose/soft soils at the base of the slope, they would have long ago experienced excessive settlement and lateral movement to the point that they would require foundation underpinning and stabilization measures. Driving of the small-diameter foundation piles to be used for the new house does not cause strong ground vibrations and will not cause settlement in the foundations of the neighboring homes.

The subsurface drainage system that will be installed for the house will not decrease the stability of the steep slopes. Removal of water from soil, especially near slopes, does not have a negative impact on slope stability. In many cases, the removal of water will actually improve stability of slopes.

Under the Mercer Island Municipal Code, the subject property meets the criteria for the following geologic hazards: Potential Landslide Hazard, Steep Hazard, Seismic Hazard and Erosion Hazard.

Potential Landslide Hazard: Under Mercer Island Code (MICC) 19.07.160.C.2, a prescriptive minimum buffer of 25 feet is to be maintained from Shallow Landslide Hazard areas, and 75 feet from Deep-seated Landslide Hazard areas. Considering the competent glacial till soils that comprise the steep slopes to the west and southwest of the site, and the lack of evidence of deep-seated slides, it is our professional opinion that this slope would be a Shallow Landslide Hazard Area.

The planned residence will extend into the minimum prescriptive buffer. Considering the measures that have been included in the home design, a buffer is not necessary to mitigate the landslide hazard to the site or the neighboring properties. The excavation for the new home will not adversely impact the stability of the surrounding properties, as it will be shored with substantial engineered soldier pile walls that will maintain temporary support for the excavation at the toe of the steep slope. Also, the permanent basement walls will provide appropriate long-term support that will, in fact, provide more stability for the slope's toe than the loose soils currently do. The hazard to the occupants of the planned Mercer Island Treehouse residence from the buffer reduction will be mitigated by constructing the upslope walls of the house to catch or deflect landslide debris from potential future slides on the steep slopes.

Steep Slope Hazard: Under MICC 19.07.160.C.2.a, a minimum prescriptive buffer equal to the height of the steep slope, not to exceed 75 feet, shall be applied to the top and toe of the steep slope. Considering the height of the steep slope to the west and southwest, the 75-foot maximum prescriptive buffer would apply.

The planned residence will encroach into this prescriptive buffer, extending to the toe of the steep slope areas located within the site boundaries. However, from a geotechnical standpoint, this buffer encroachment will not adversely impact the stability of the steep slopes, for the same reasons discussed above. The excavation will be temporarily shored with an engineered soldier pile wall that will maintain support for the toe of the steep slope, and the permanent basement walls will provide increased lateral support for the toe of the steep slope. These measures will prevent adverse impacts to the stability of the steep slopes within the site, and on the surrounding properties.

Seismic Hazard: MICC 19.07.160.D addresses development considerations for Seismic Hazard areas. There is no information indicating that the site lies on, or near, an active fault. As a result, no buffer associated with the Seismic Hazard designation is required.

However, the loose soils underlying the groundwater table could undergo liquefaction (soil strength loss) in the event of strong ground shaking during a large earthquake. This is a typical risk associated with sites located in ravines or valleys, and along lake shores. The Seismic Hazard related to potential foundation bearing loss under shallow foundations from seismic liquefaction will be mitigated for this project by the use of deep pile foundations that will be embedded into dense to very dense soils that are not liquefiable. This will maintain vertical support for the piles in the event of an earthquake, and the grade beams that will interconnect the piles will provide added protection against foundation collapse.

Erosion Hazard: Under the criteria of the Mercer Island Code, much of the island falls under the designation of an Erosion Hazard area. This is based mostly on the presence of silty, fine-grained soils, and ground that slopes at 15 percent or more. Not only the site, but all of the adjoining properties, including those upslope to the west and southwest, fall under the classification of Erosion Hazard areas.

MICC 19.07.160.E requires that:

1. All development proposals within erosion hazard areas shall comply with Chapter 15.09 of the MICC for the Storm Water Management Program, and
2. The planned development or activity within an erosion hazard area cannot increase the potential for instability on or off the site.

To satisfy condition 1, during the design and permitting process, the City of Mercer Island will require that the project meets the requirements of the stormwater code. We expect that this will include preparing a detailed Temporary Erosion and Sedimentation Control (TESC) plan, which is a requirement for any project located within an Erosion Hazard area. Additionally, the City will require that the site stormwater design complies with their stormwater code.

For condition 2, as discussed above, in the Landslide Hazard and Steep Slope Hazard sections, the proposed project will incorporate measures that will prevent an increase in the potential for instability both on, and of, the site.

In their October 23, 2019 letter, GeoGroup NW provided the "statement of risk" required by the City of Mercer Island code (MICC 19.07.160.C.3) for geologically hazardous areas. This statement, which addresses risks to both the site and the adjacent property, is appropriate, and is consistent with statements of risk we have had to provide in our company's 34+ years of geotechnical engineering on Mercer Island. From a geotechnical standpoint, an alternative statement of risk,

> "Construction practices are proposed for the alteration that would render the development as safe as if it were not located in a geologically hazardous area and do not adversely impact adjacent properties"

would also apply to the project, and technically be more appropriate. However, this does not change the conclusions we have reached about the appropriateness of the planned development and the mitigation measures that will be included.

From a geotechnical standpoint, it is worth noting that the upslope properties actually pose more of a hazard to the subject property than the other way around. The homes along the top of the steep slope are well within the minimum prescriptive buffer for steep slope hazard areas, and were constructed well before the implementation of Critical Area codes on Mercer Island. Past practices, such as placement of uncontrolled fills and/or walls on or near steep slopes for yards and landscaping, would not be allowed under current codes. Improper fill placement and grading, excessive clearing or poorly-managed tree removal, or ineffective or malfunctioning drainage systems above a steep slope increase the potential for future slope movement. While the hazard of potential future slope movement has been addressed for the planned Mercer Island Treehouse residence by the planned slide catchment wall to be incorporated into the house, it is still the responsibility of upslope property owners to avoid increasing the potential for instability on the steep slopes.

Please contact us if there are any questions regarding this letter.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

12/03/2020

Marc R. McGinnis, P.E.
Principal

cc: **Mccullough Hill Leary** – Courtney Kaylor
via email: courtney@mhseattle.com

MRM:kg

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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GEO Group Northwest, Inc.

Geotechnical Engineers, Geologists
& Environmental Scientists

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

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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

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.

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Engineering Geologist
1005
Licensed Geologist

Keith Johnson
Geologist

KEITH A. JOHNSON

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Principal

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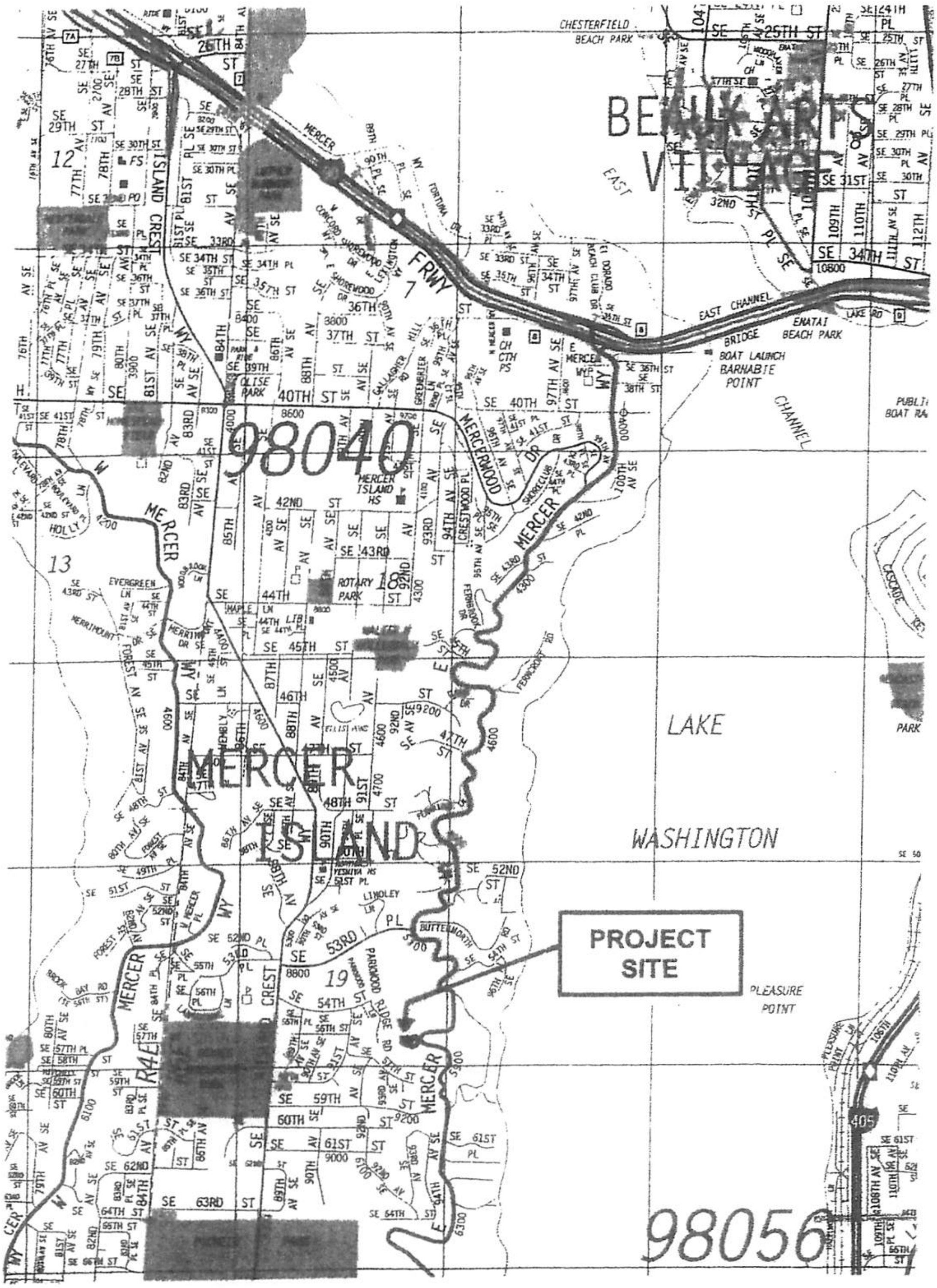
PLATES

G-3827

GEO Group Northwest, Inc.



Source: Thomas Guide, 2007.

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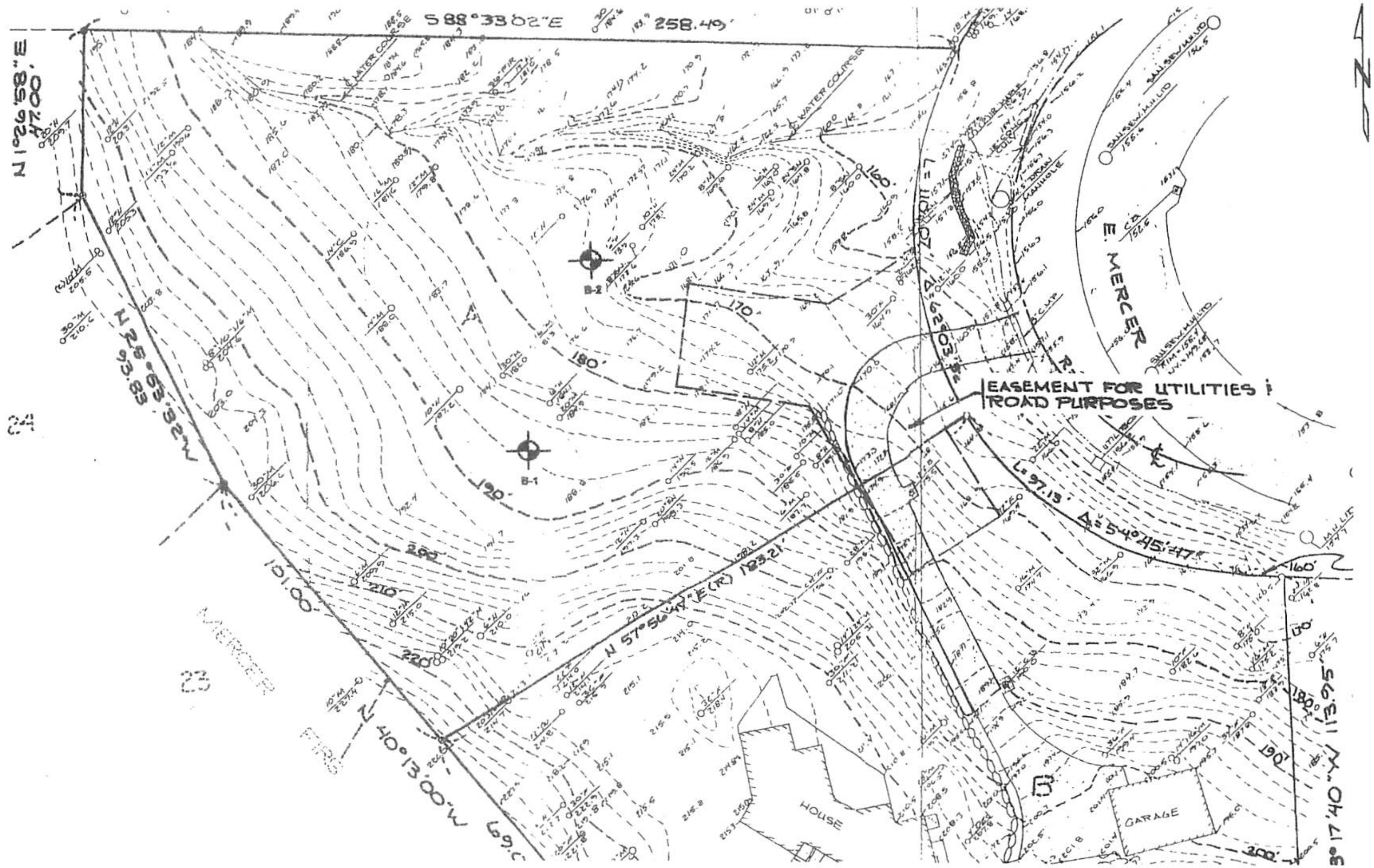
Geotechnical Engineers, Geologists, & Environmental Scientists

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.

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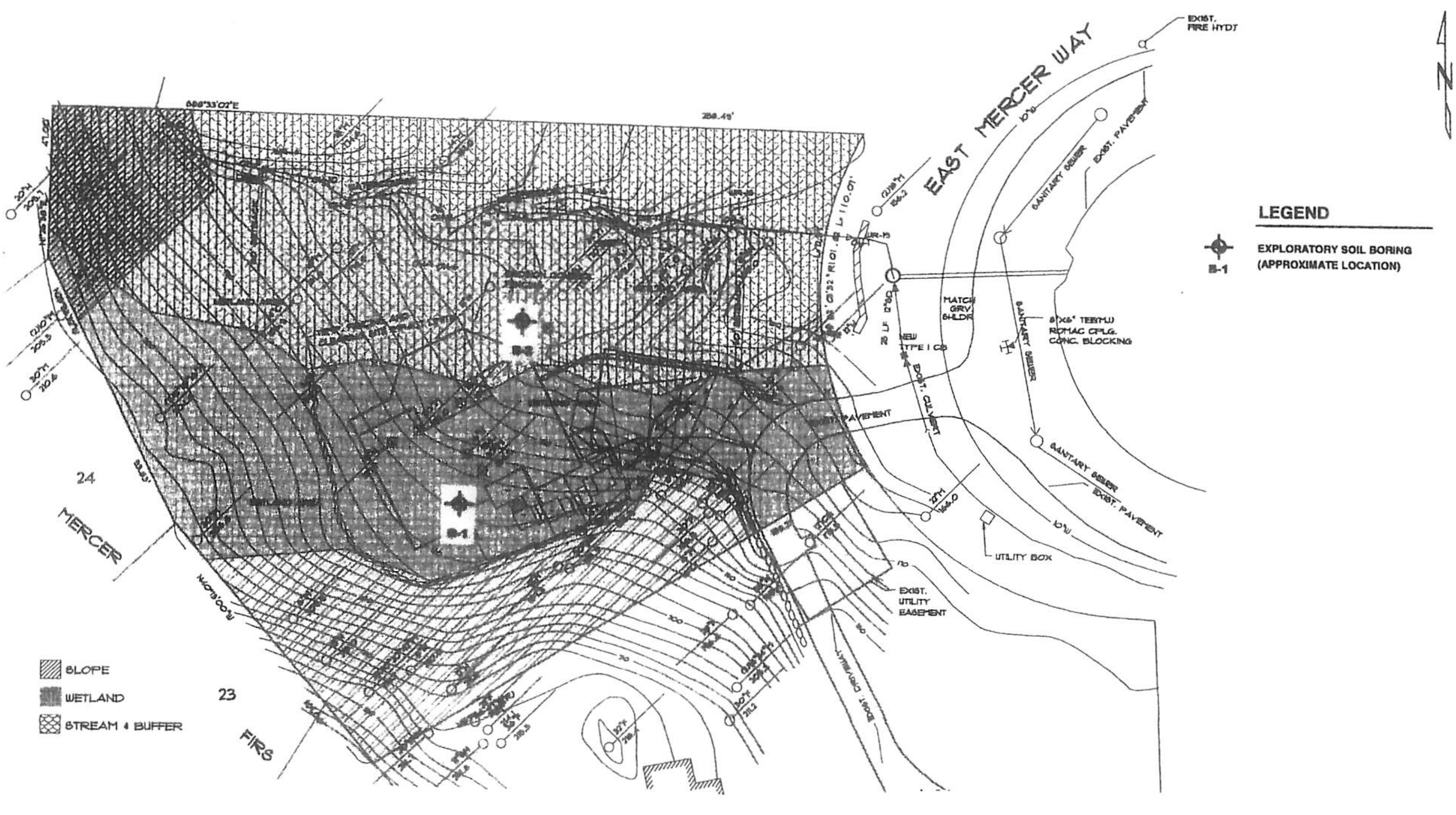
Geotechnical Engineers, Geologists, & Environmental Scientists

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.

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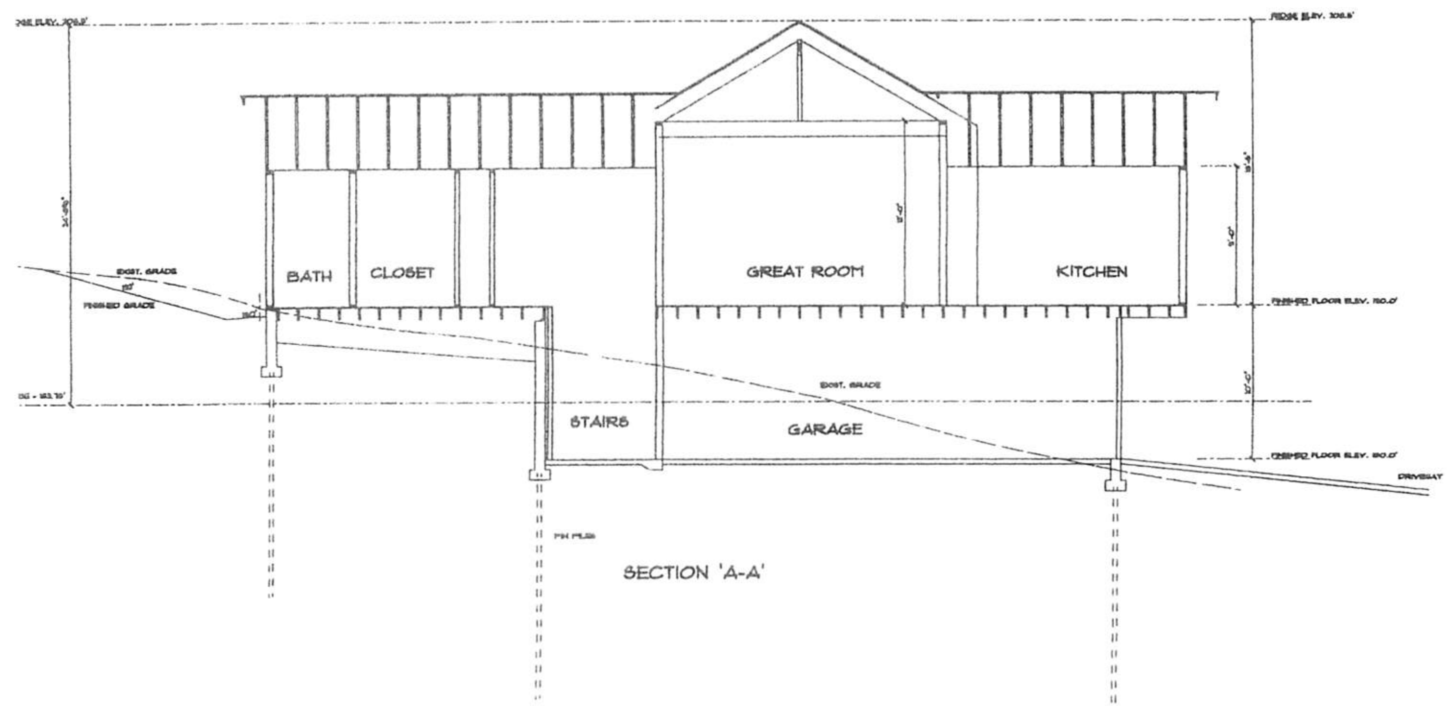
PROPOSED DEVELOPMENT PLAN

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.

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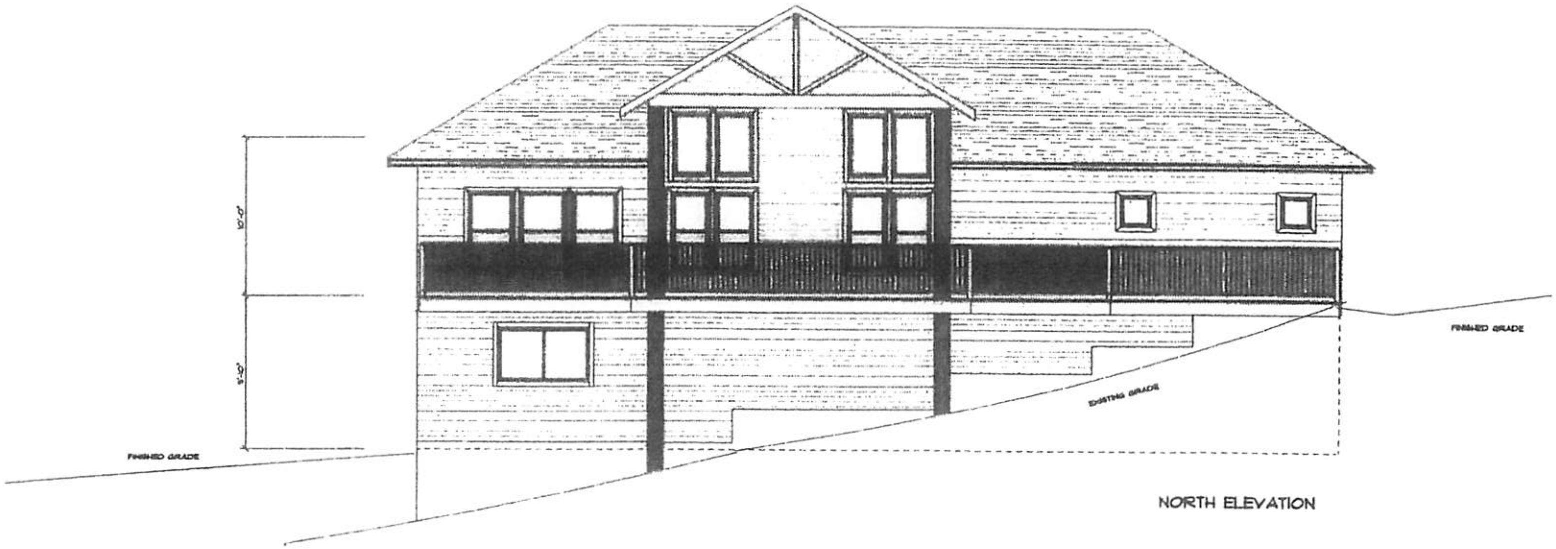
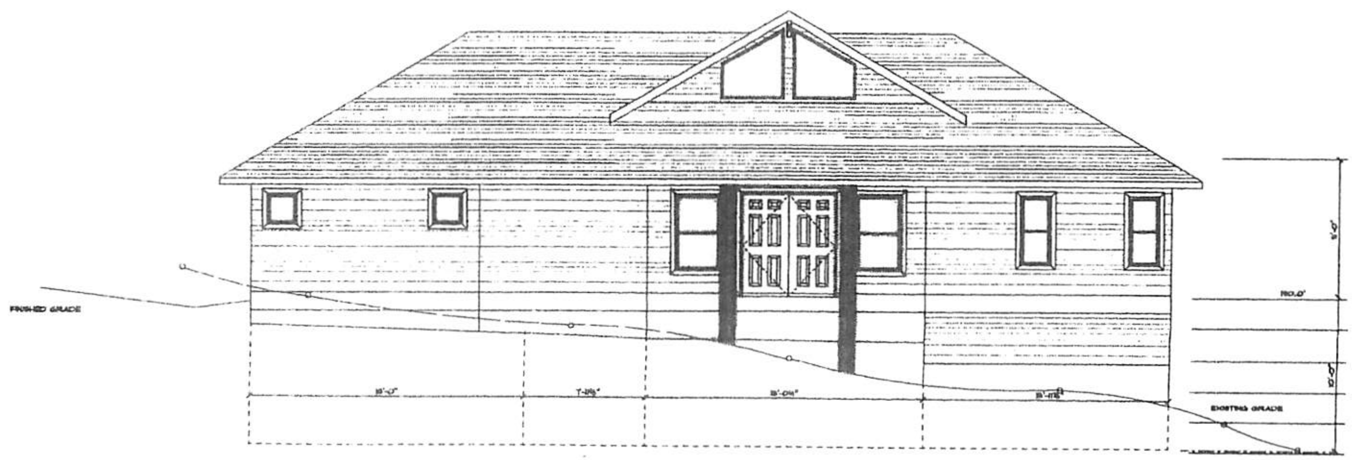
PROPOSED RESIDENCE SECTION

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.

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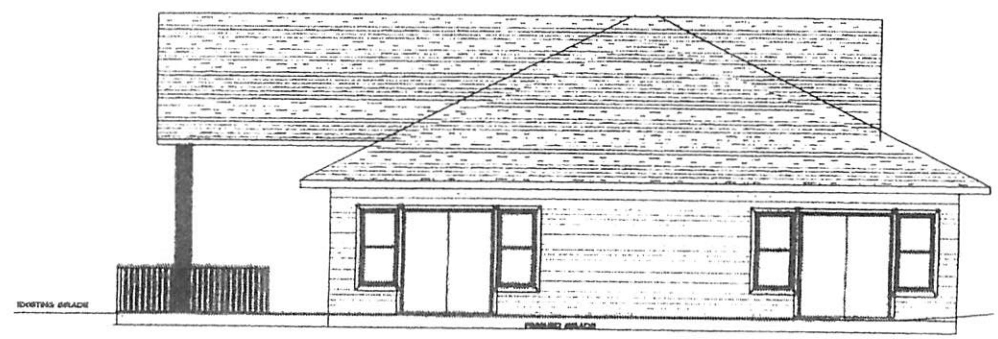
NORTH & SOUTH ELEVATIONS

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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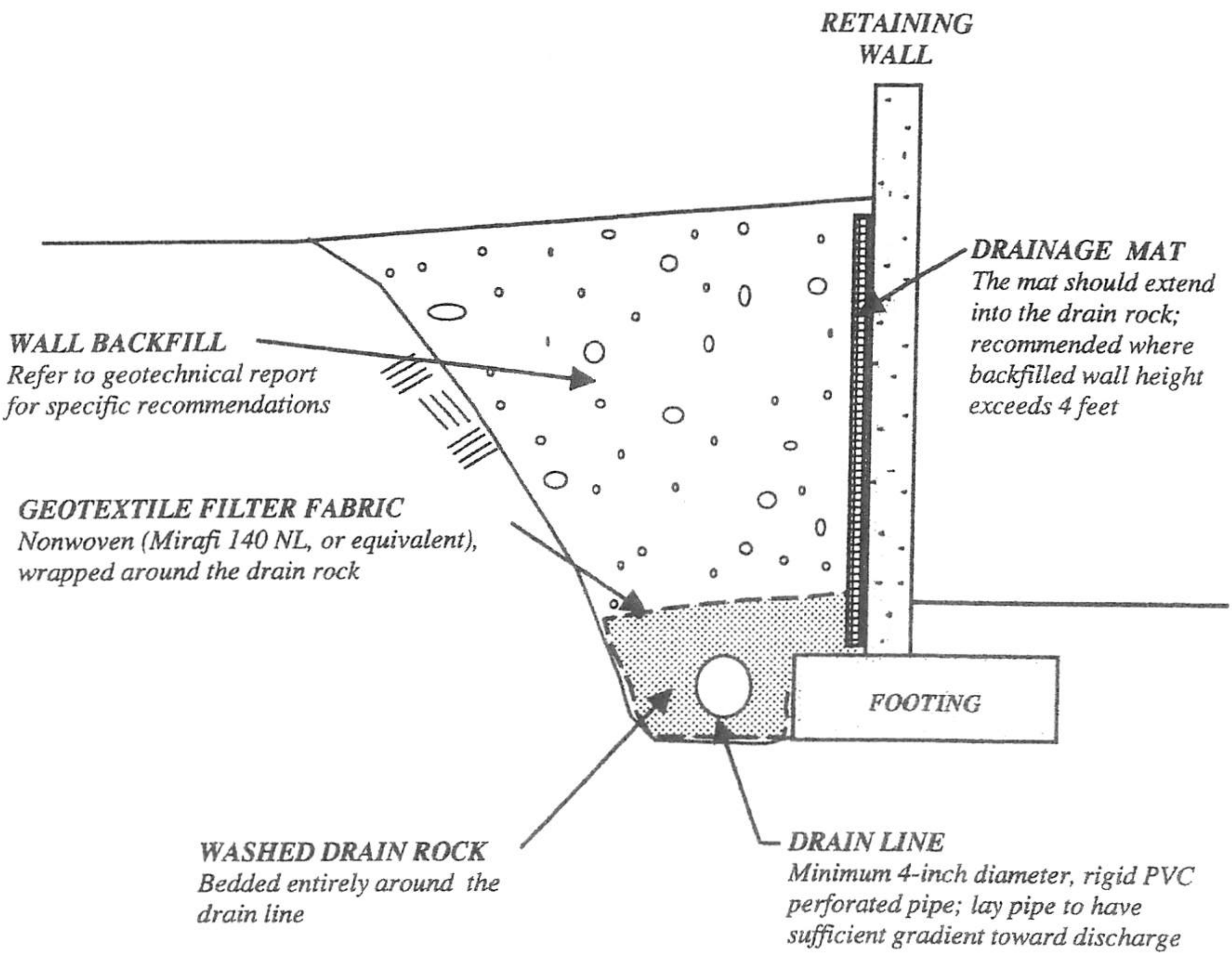
EAST & WEST ELEVATIONS

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.

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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

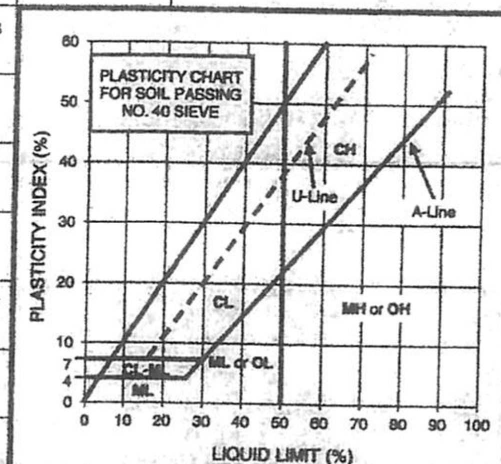
G-3827

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 More Than Half by Weight Larger Than No. 200 Sieve	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
	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 Less Than Half by Weight Larger Than No. 200 Sieve	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		
	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 Qu, 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.
Geotechnical Engineers, Geologists, & Environmental Scientists
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							
25							
30							
35							
40							

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.

LEGEND: 2" O.D. Split-Spoon Sampler; 3" O.D. Shelby-Tube Sampler; 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.
	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	
5	SP-SM	As above, loose.	I	S2	4,3,5 (N=8)	23.6	
	SP-SM	As above, medium dense, trace coarse sand.	I	S3	4,7,9 (N=16)	21.4	
10	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)

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** A3

Soil Liquefaction Analysis Report

Geotech Consultants, Inc.

Project : 20408 MI Treehouse LLC
Project No. : 20408
Client : MI Treehouse LLC
Site Address : 5637 East Mercer Way, Mercer Island, Wa 98040

Borehole : B-2
Total Depth : 27 ft
Water Level : 1 ft
Calculated By : MKM
Reviewed By :

Table i : Input Data and Assumptions

Input Assumption	Setting
Field Test Type :	Standard Penetration Test (SPT)
Apply All Corrections to SPT?	True
Groundwater Level (ft) =	1
Earthquake Magnitude M =	7.1
Magnitude Scaling Factor (MSF) :	1.15 (Idriss, 1997 -NCEER)
Fines Content Correction :	(according to user settings)
Depth Reduction Factor (Rd) :	Idriss 1999, Golesorkhi 1989
Relative Density (Dr) Estimation :	Idriss & Boulanger, 2003
Site Topography :	Gently Sloped : 20 %
Ground Improvement Feature :	None
Peak Ground Acceleration PGA (g) =	0.682

Table ii : CRR Calculation Methods

CRR Formula	Selected?
NCEER Workshop (1997)	True
Boulanger & Idriss (2014)	True
Vancouver Task Force (2007)	False
Cetin et al. (2004)	False
Chinese Code	False
Seed et al. (1983)	False
Japanese Highway Bridge Code	False
Tokimatsu and Yoshimi (1983)	False
Shibata (1981)	False
Kokusho et al. (1983)	False

Table iv : Field Tests

Depth (ft)	SPT Blow Counts(N)
2.5	4
5	8
7.5	16
10	4
15	5
20	23
25	19

Table iii : Subsurface Soil Layers

Layer Thickness (ft)	Soil Type	Unit Weight (lb/ft3)	Fines Content (%)	D50 (mm)	Check Liquefaction	Su (ksf)
7	Sand	110	15	0.25	True	0
7	Sand	115	5	0.3	True	0
5	Sand	120	20	0.25	True	0
8	Silt	115	5	0.02	False	0

Table v : Post-Liquefaction Displacements

Type	Method	Movement (inch)
Lateral Spreading	Youd et al., 2002	144
Lateral Spreading	Barlett & Youd, 1992	167
Lateral Spreading	Hamada et al., 1986	226
Lateral Spreading	Youd & Perkins, 1987	LSI ~41 see details for LSI=50
Vertical Settlement	Ishihara & Yoshimine, 1992	8

Soil Liquefaction Analysis Report

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Borehole : B-2
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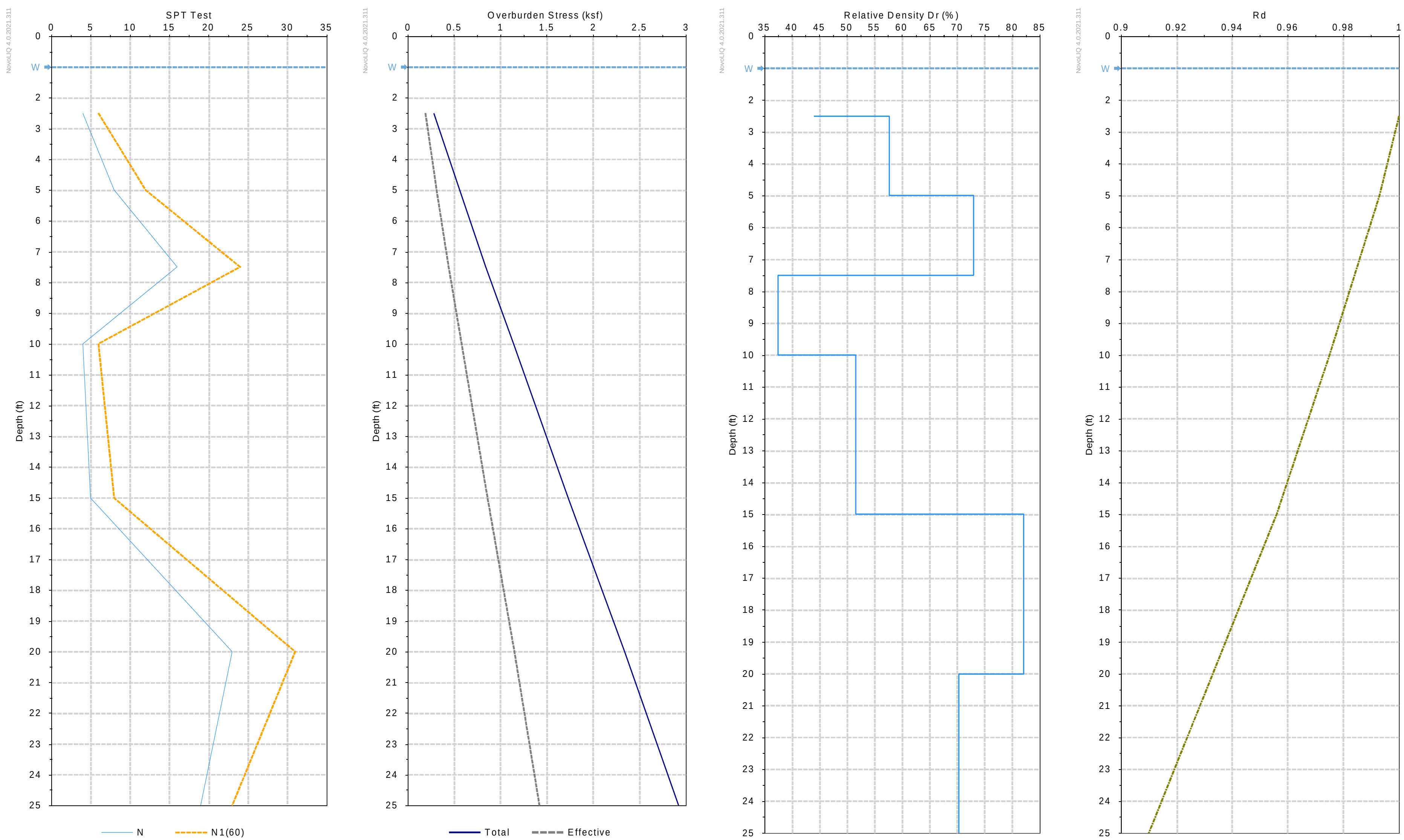
Calculated By : MKM
Reviewed By :



Soil Liquefaction Analysis Report

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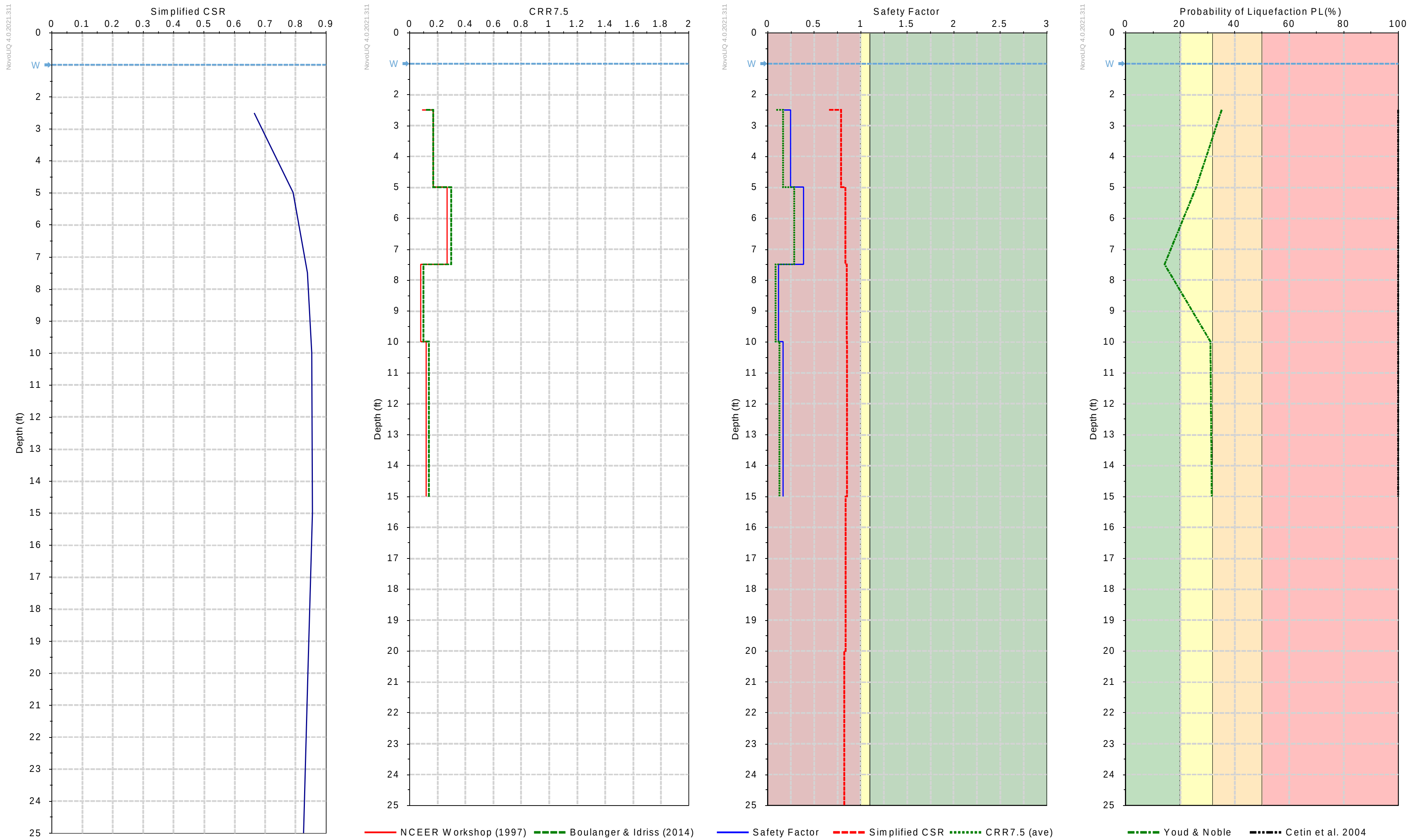

Soil Liquefaction Analysis Report

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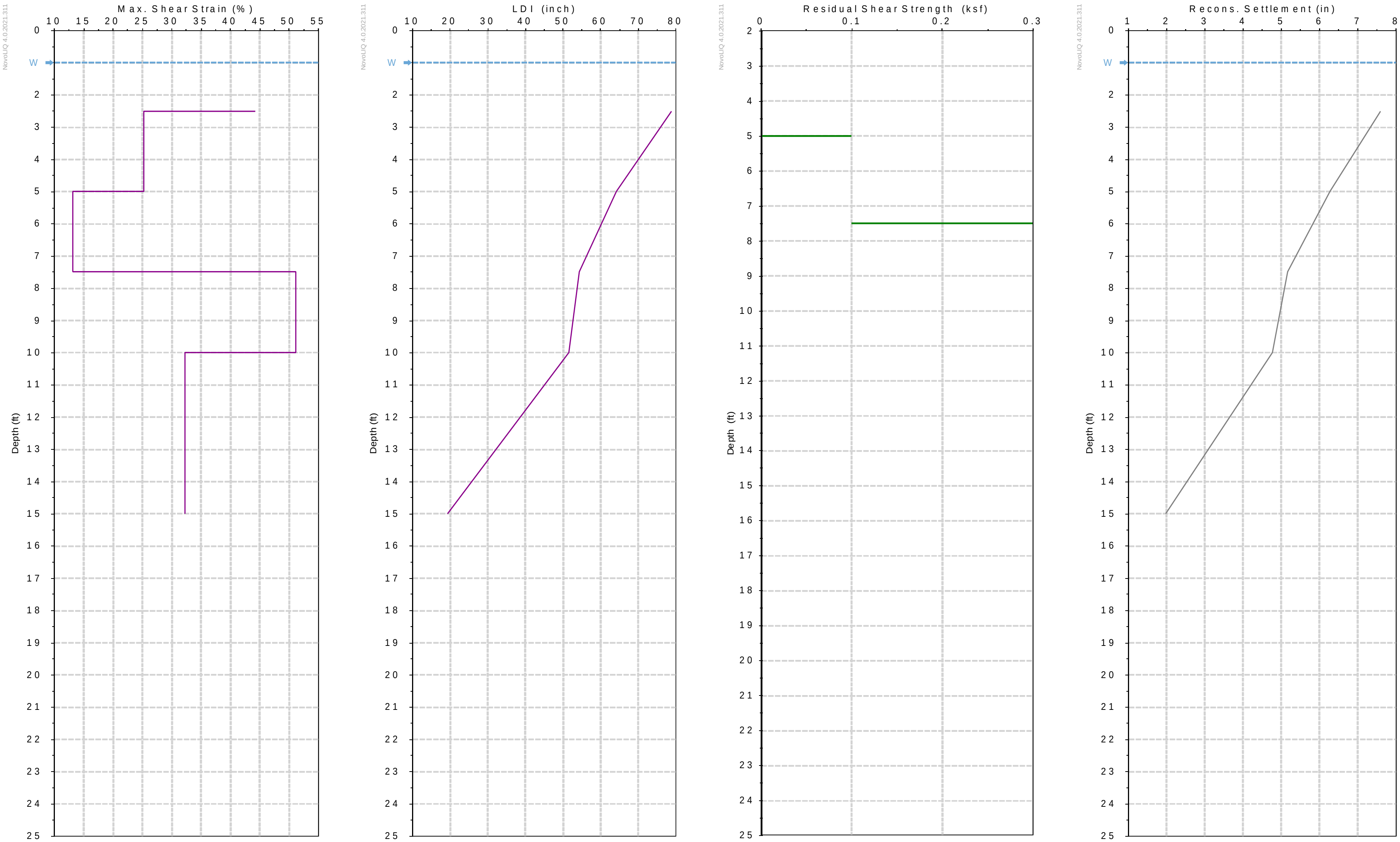
Reviewed By :



GEOTECH
CONSULTANTS, INC.
2401 - 10th Avenue East
Seattle, WA 98102
(425) 747-5618

Job No. 20408

Project Treehouse

Subject Catchment Volumes

Made By MRM Checked By

Date 6/8/2022 Page 1 Of 1