GEOTECH
CONSULTANTS, INC.

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May 16, 2017

JN 16556

Jeff Sanderson
8100 Evergreen Lane
Mercer Island, Washington 98

via email: jeff@sanderson.org

Subject: **Geotechnical Engineering Study**
Proposed Landslide Repair Project
8100 Evergreen Lane
Seattle, Washington

Dear Mr. Sanderson:

We are pleased to present this geotechnical engineering report for the landslide repair project to be constructed in Mercer Island, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for repair structures. This work was authorized by your acceptance of our proposal, P-9643, dated November 29, 2016.

A landslide occurred on the north-facing slope on the northeastern portion of the Sanderson property in mid-November. This occurred following a period of significant rainfall, which in one major reason among others that the landslide occurred. This landslide has left an approximately 10-foot-tall, extremely steep slope at the northern edge of an existing, pile-supported parking structure that is located at the top of the slope. A new retaining wall will be needed in the landslide area in order for new soil and landscaping to be placed.

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

SITE CONDITIONS

Surface Conditions

The Vicinity Map, Plate 1, illustrates the general location of the Sanderson property on Mercer Island. The Sanderson property located is located on the northern side of Evergreen Lane, which is west of West Mercer Way. The property is relatively flat on its southern and central portions, but a steep slope that is in the range of 80- to 90-feet in height is located on the northern side of the property. A small creek is located at the base of the slope. The landslide, which is about 60 feet wide near the top of the slope and "necks down" to about 35 feet near the bottom, is located on the eastern portion of the northern slope.

Most notably, the top of the recent landslide and previous slope is located directly north of a parking structure mentioned earlier. This northern edge of this structure, which is nearest the top of the slope and landslide, is founded on four, 2-foot-diameter, concrete piles. As noted earlier, an extremely steep slope now exists in this area, which is very near to the piles. Recently, we obtained field notes and summary letter prepared by Terra Associates (a geotechnical engineering company who previously was involved in the property) of these concrete piles, which were installed in August/September 1993 were obtained. The information in these documents indicate that the four piles that are located under the northern edge of the parking structure (noted as Piles 12 - 15) were drilled into the ground about 20 to 21 feet. It was also noted in the documentation that the piles were embedded approximately 10 to 15 feet in to what Terra Associates believed was competent soil. We have observed the parking structure and the piles numerous times since the landslide, and we have not seen any signs of movement of the structure or the piles.

Subsurface Conditions

The subsurface conditions in the landslide area were recently explored by drilling one test boring at the approximate location shown on the Site Exploration Plan, Plate 2. However, several other test borings were drilled in this area in 1993 prior to the construction of the parking structure; these test borings were contained within a geotechnical engineering study prepared by Terra Associates. Our recent exploration program was based on the proposed construction, anticipated subsurface conditions based on the previous test borings and those encountered during exploration, and the scope of work outlined in our proposal.

The Terra Associate's study indicated that two test borings had been drilled at that time in the area of the parking structure. The location of these test borings is also shown on Plate 2. These test borings indicated that loose fill and native silty soils in the range of 10 to 12 feet were revealed overlying competent, native silty soil. On November 16, 2016, our personnel traversed the landslide area and used a shovel to expose soils in the landslide area near the northern edge of the parking structure. We observed that, in general, there was about 7 feet of loose fill soil overlying about 4 feet of loose/soft native silt soil. More competent silt soil was seen under these upper loose/soft soils; thus, the soils we observed were very similar to the soil conditions noted in the 1993 test borings. The logs of the two notable test borings are included in the Appendix of this study.

A recent test boring was drilled on December 8, 2016 using a portable Acker drill -- this drill system utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at approximate 2.5- to 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The recent Test Boring Log is attached as Plate 2. The test boring was drilled just outside of the eastern edge of the recent landslide (the landslide area was covered in plastic, so we could not drill within the landslide area). The soil revealed in the test boring was quite similar to the 1993 test borings, but not including the upper fill soil; it appears that this portion of the slope did not have fill soil placed on it. The upper, approximate 4 feet of soil revealed in the recent test boring consisted of loose, native, somewhat sandy silt soil. This loose was underlain by medium-dense to dense silt soil to a depth of approximately 13 feet. The silt below this level was either hard or dense to very dense down to a maximum explored depth of 21.5 feet.

Some groundwater could readily be seen on November 16, 2016 in the landslide area flowing beneath the loose/soft soils and perched on the surface of the more competent silt soil. However, no groundwater seepage in the recent test boring drilled on December 8, 2016. Therefore, it appears that the groundwater seepage that perches on the more competent silt soil occurs follows a period(s) of significant rainfall.

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

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

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

The test borings drilled in 1993 prior to the construction of the parking structure, as well as the recent test boring drilled in the hillside north of the parking structure and outside of the landslide indicate that the core of the hillside consists of competent, native silt soil. The recent test boring outside of the landslide area revealed approximately 4 feet of loose/soft, native silt soil at the ground surface; this amount of loose soil is common for native slopes in Mercer Island and the Puget Sound region. The 1993 test borings revealed up to approximately 7 feet of loose fill soil overlying the loose/soft native silt soil; we observed this fill and silt overlying competent silt in the steep soil exposure than now exists just north of the parking structure. It is apparent that, where the landslide occurred, the fill soil most of the loose/soft silt soil slid down the hillside. With the exception of the exposure, it appears that the competent silt is now close to the existing ground surface where the landslide occurred because the upper loose/soft soils slid away. We have discussed with the owner, design team, and construction team that several factors likely caused the landslide, with one being heavy precipitation that fell just prior to the landslide.

There was an approximate 4- to 5-foot-wide flat bench at the top of the slope adjacent to the northern side of the parking structure prior to the slide. Although we have seen no evidence of instability of the parking structure and its concrete columns, we believe that the bench should be put back to near its original grade to provide long-term stability to the structure and piles. To do this, fill soil and a retaining wall is needed downslope of the structure and piles. The overall new slope inclination between the edge of the top bench and the retaining wall should be no steeper 2:1 (H:V), so the height of the wall should correspond to this inclination. The lower retaining wall will be the most significant part of this project from a geotechnical engineering standpoint, and we have provided design parameters for the needed retaining wall further in this report. Some terracing and the construction of smaller "landscape" walls can be constructed between the top bench and the retaining wall provided the height of the retaining wall is such that an overall 2:1 (H:V) inclination is maintained.

Construction of the retaining wall will be difficult due to very limited access to the slope north of the parking structure. If access is possible to large equipment, it appears that a soldier-pile-installed retaining wall would be the most likely wall type; the soldier piles provide a considerable lateral strength needed for the wall depending on their depth. However, even if soldier piles can be used, it is very likely that anchors will be needed in addition to the piles to provide additional lateral strength for this project; helical anchors are likely a suitable anchor type. If large equipment cannot be used for this project, small soldier piles should that are placed into excavated holes should still be used, but cannot be relied upon for lateral strength. We recommend that the minimum depth that the soldier piles be installed to is 5 feet below the existing ground surface. A passive resistance can be included for the soldier piles in the design of the project wall below a level that is 3 feet below the ground surface. An ultimate passive pressure of 275 pcf should be used in the design; appropriate safety factors need to be included in the design of the wall using this value.

As noted earlier, an existing elevated parking structure is directly upslope and south of the landslide area. It appears that this structure was stable prior to the landslide, and as we discussed in a letter dated December 13, 2016, and based on more recent observations we have made of the structure, it appears to still be stable. Once the proposed structure is constructed below and north of the parking structure, the parking structure will be even more stable; we believe that it will be more stable than it was prior to the landslide.

We understand that part of an existing trash enclosure will likely be removed as part of the construction. When rebuilt, one new footing is needed for the structure. Provided the footing is placed on the stiff silt soil, we believe the use of a footing is very suitable. A bearing capacity of 3,000 psf can be used for the design of the footing for the rebuilt trash enclosure.

The City of Mercer Island requires a "statement of risk" with regards to the project because it is located in a Geologic Hazard Area. As such we make the following statement:

> The proposed development has been designed so that the risk to the lot and adjacent property is mitigated such that the project is determined to be safe.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type D (Stiff Site Soil Class). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.42g and 0.44g, respectively. The site soils are not susceptible to seismic liquefaction because of their dense nature (and/or the absence of near-surface groundwater.

RETAINING WALL DESIGN

This section discusses a new retaining wall that will retrain fill soil with an overall inclination of 2:1 (H:V). As noted earlier, anchors are very likely needed for lateral restraint of the wall. The most important parameter for the design of the retaining wall is active pressure. If one anchor is used for the wall, the wall is not considered restrained and the active pressure would be triangular. However, if two or more anchors are used, the wall would be considered restrained and a rectangular pressure of should be used. The following recommended parameters should be used:

PARAMETER	VALUE
Active Earth Pressure * - one anchor	60 pcf
Active Earth Pressure ** - two or more anchors	39H psf
Soil Unit Weight	140 pcf

*** pcf is Pounds per Cubic Foot, and the Active Earth Pressure is computed using the Equivalent Fluid Pressures.**
**** psf is Pounds per Square Foot, and H is the Wall Height**

As noted in the ***General*** section of this report, for the soldier piles used in the wall design, we recommend that the minimum depth that the soldier piles be installed to is 5 feet below the existing ground surface. A passive resistance can be included for the soldier piles in the design of the project wall below a level that is 3 feet below the ground surface. An ultimate passive pressure of 275 pcf should be used in the design. A safety factor of 1.5 should be used in the static design of the retaining wall.

Wall Pressures Due to Seismic Forces

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 9**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

HELICAL ANCHORS

Helical anchors are a very suitable alternative to provide lateral resistance/strength to the retaining wall where only hand or very small equipment can be used. Helical anchors consist of single or multiple helixes that are rotated into the ground on the end of round or square metal shafts. The design capacity of single helix anchors is the allowable soil bearing capacity on the helix area. Multiple-helix anchors are typically assumed to have a design capacity equal to the sum of the allowable bearing capacity on each helix, if they are separated more than three helix diameters.

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

The anchors should be installed at an angle ranging from approximately 15 to 25 degrees from horizontal. Anchors in the lower portion of the wall should extend at least 7 feet behind the wall, while any upper anchors should extend at least 10 feet behind the wall.

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

RETAINING WALL BACKFILL AND WALL DRAINAGE

We understand that organic, topsoil is needed at the surface of portions of the new slope above the new retaining wall to allow new plant and trees to grow. This is suitable in our opinion. However, structural fill backfill near the existing ground surface needs to consist of coarse, free-draining material containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter.

As it is possible that groundwater will continue to flow perched on the competent silt soil in the future following significant rainfall events. This water needs to be able to continue to flow through the area in the future. It is important that weep holes be placed near the bottom of the retaining wall to allow water to pass through the base of the wall; we believe that these will be sufficient and no formal footing drain is needed. Depending the material used to face the new retaining wall, a drainage mat may also be needed on the inside of the wall facing. If wood lagging is used, the mat is not needed as small gaps in the lagging can be included in the lagging installation.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining walls.

Compacted fill slopes should not be constructed with an inclination greater than 2:1 (H:V). To reduce the potential for shallow sloughing, fill must be compacted to the face of these slopes. This can be accomplished by overbuilding the compacted fill and then trimming it back to its final inclination. Adequate compaction of the slope face is important for long-term stability and is necessary to prevent excessive settlement of patios, slabs, foundations, or other improvements that may be placed near the edge of the slope.

All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil. Topsoil will be placed on regraded slopes to promote growth of vegetation. Proper preparation of the regraded surface, and use of appropriate topsoil is necessary to prevent the topsoil from sliding off the slope. This is most likely to occur following extended wet weather if a silty topsoil is used. On steeper slopes, it may be necessary to “track walk” the slope or cut small grooves across the slope prior to placing the topsoil.

GENERAL EARTHWORK AND STRUCTURAL FILL

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process. The onsite soils should be used as structural fill.

Structural fills placed on sloping ground should be keyed into the competent silt soils. This is typically accomplished by placing and compacting the structural fill on level benches that are cut into the competent soils. The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction.

The non-organic structural fill used for this project should be compacted to at least 95 percent of the Minimum Relative Compaction, where Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut.

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

LIMITATIONS

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

The recommendations presented in this report are directed toward the protection of only the area directly above the new retaining wall. Predicting the future behavior of steep slopes and the potential effects of development on their stability is an inexact and imperfect science that is currently based mostly on the past behavior of slopes with similar characteristics. Landslides and soil movement can occur on unrestrained steep slopes before, during, or after the development of property.

This report has been prepared for the exclusive use of Jeff Sanderson and his representatives, for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

In addition to reviewing the final plans, Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3	Recent Test Boring Log
Appendix	1999 Test Boring Logs

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

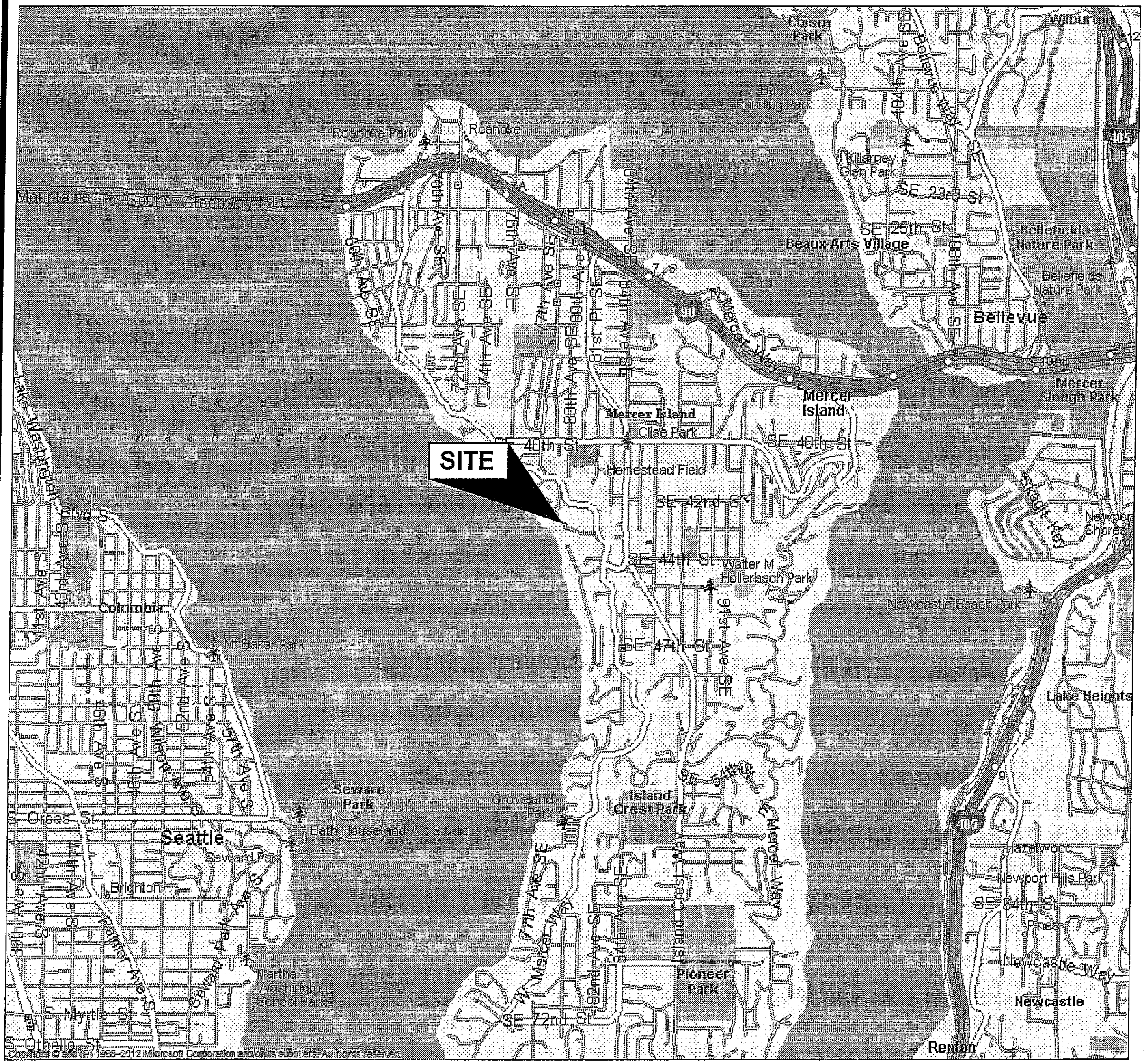
D. ROBERT WARD
STATE OF WASHINGTON
19740
REGISTERED
PROFESSIONAL ENGINEER

5/16/17

D. Robert Ward, P.E.
Principal

cc: **SHKS Architects** – Jonathon Hartung
via email to: jh@shksarchitects.com

NORTH



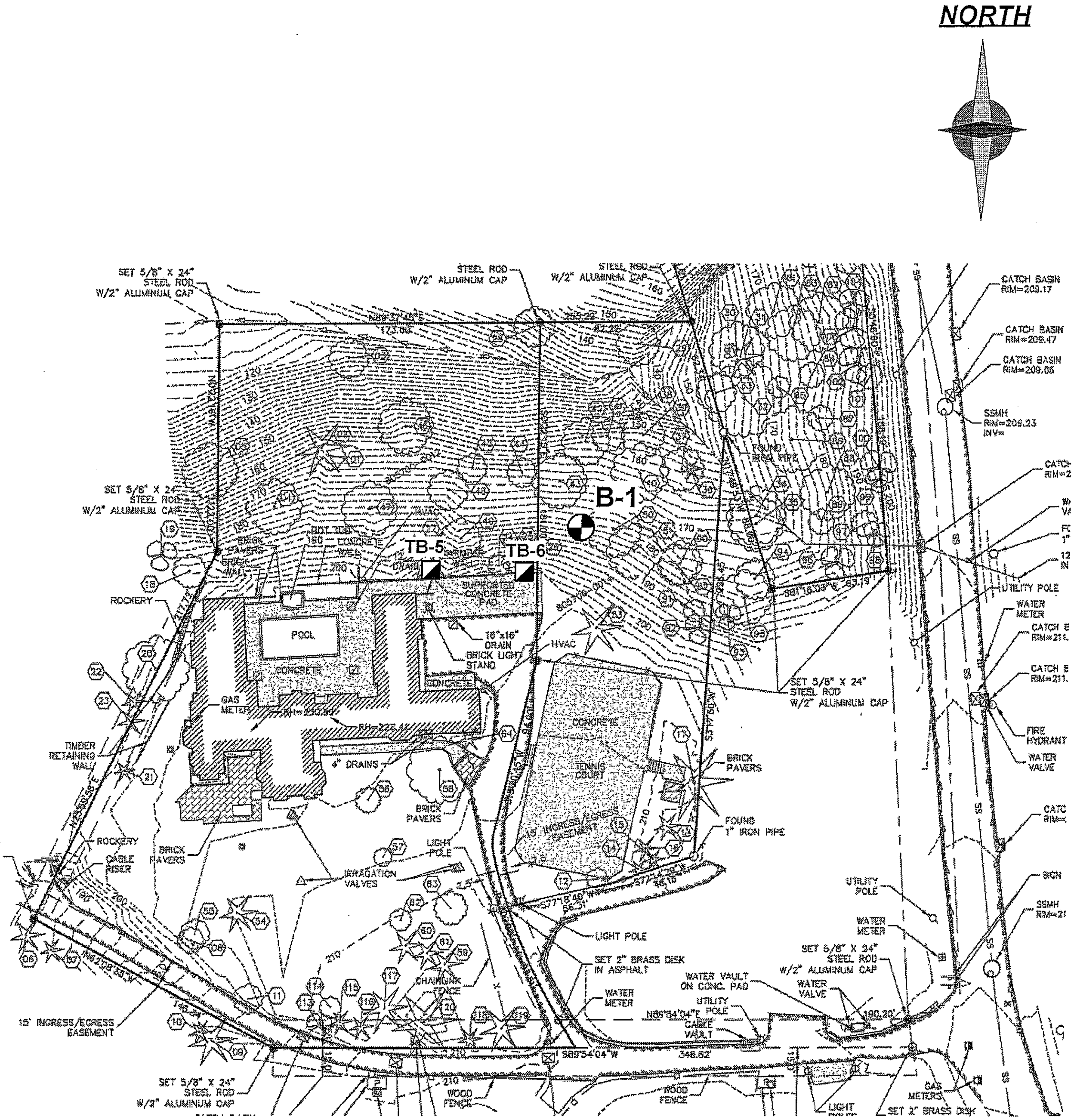
(Source: Microsoft MapPoint, 2013)

GEOTECH CONSULTANTS, INC.

VICINITY MAP

8100 Evergreen Lane
Mercer Island, Washington

Job No:	Date:		Plate:
16556	Dec. 2016		1



Legend:

- Recent Test Boring Location (approximate)
- 1993 Test Boring Locations (approximate)

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SITE EXPLORATION PLAN

8100 Evergreen Lane
Mercer Island, Washington

Job No:	Date:		Plate:
16556	Jan 2017	No Scale	2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						Topsoil over;
			4	1		Gray-brown SILT to sandy SILT, non-plastic, fine-grained, moist to wet, loose
5			13	2	ML	Gray-brown sandy SILT, non-plastic, fine-grained, moist, medium-dense
						-reduced sand content
			34	3		-increased sand content, becomes dense
10			41	4		
					ML/CL	
15						Gray-brown clayey SILT, plastic, moist, hard
			50	5		Gray-brown sandy SILT, non-plastic, fine-grained, moist, dense
					ML	
20			46	6		Gray-brown SILT to sandy SILT, non-plastic, fine-grained, moist, dense
25						

*** Test boring was terminated at 21.5 feet on December 8, 2016.**
*** No groundwater seepage was observed during drilling.**

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TEST BORING LOG

8100 Evergreen Lane
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
16556	Dec. 2016	TRC	3

APPENDIX – 1993 TEST BORINGS

Boring No. B-5

Logged by: DRK

Dated: 3-12-93

Approximate Elev. +197

Graph/ USCS	Soil Description	Consistency	Depth (ft.)	Sample	(N) Blows (ft)	Water Content (%)
SP SM	FILL - Black to brown, fine to medium SAND with silt and wood debris, moist.	Loose			17*	20.1
(CL)	Brown, sandy CLAY with considerable organics (disturbed), damp.	Medium Stiff to Stiff	5		10	20.5
ML CL	Brown changing to gray, sandy SILT to CLAY with interbedded layers of sand and silty sand, damp.				20	20.1
			10		18	26.8
			15		52	26.2

Boring terminated at 16.5 feet.
No groundwater seepage encountered.

*Not representative due to debris.

TERRA ASSOCIATES
Geotechnical Consultants

Boring Log
GAMORAN RESIDENCE
MERCER ISLAND, WASHINGTON

Proj. No. T-2295 Date 3/93 Figure 8

Boring No. B-6

Logged by: DRK

Dated: 3-12-93

Approximate Elev. +198

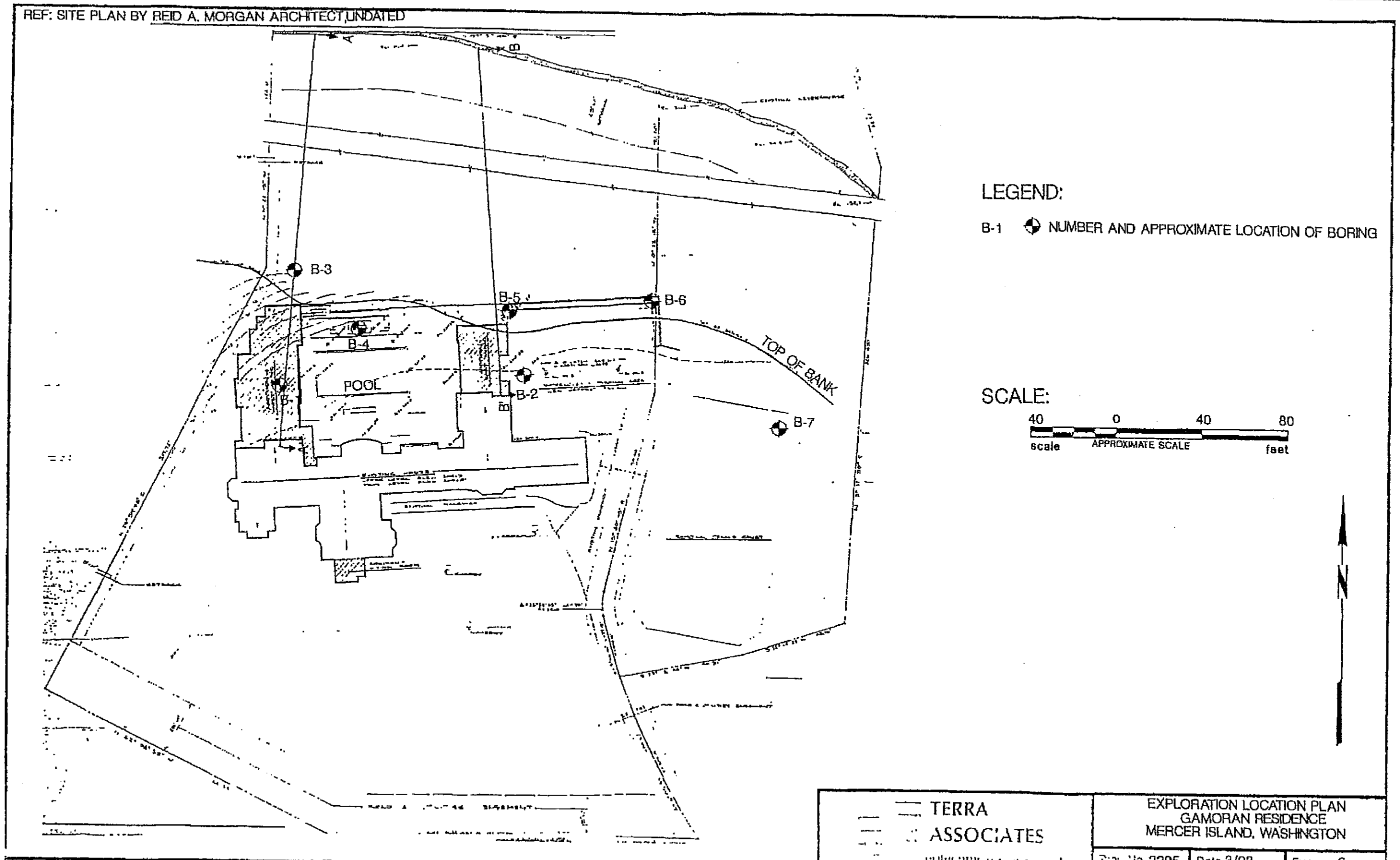
Graph/ USCS	Soil Description	Consistency	Depth (ft.)	Sample	(N) Blows (ft)	Water Content (%)
SM	FILL - Dark brown, silty SAND with medium to coarse gravel, wood and glass debris, moist.	Loose			10	21.8
			5		13	8.8
ML CL	Brown, sandy SILT to CLAY with interbedded layers of fine sand and silty sand, dry to damp.	Very Stiff to Hard			23	15.8
			10		58	22.6
					48	23.3

Boring terminated at 13.5 feet.
No groundwater seepage encountered.
Note: Water used to cool bit while drilling from 8 to 13.5 feet.

TERRA ASSOCIATES
Geotechnical Consultants

Boring Log
GAMORAN RESIDENCE
MERCER ISLAND, WASHINGTON

Proj. No. T-2295 Date 3/93 Figure 9



TERRA ASSOCIATES

EXPLORATION LOCATION PLAN
GAMORAN RESIDENCE
MERCER ISLAND, WASHINGTON

Proj. No 2295	Date 3/93	Figure 2