

**GEOTECHNICAL REPORT
PROPOSED DECK RECONSTRUCTION
5336 BUTTERWORTH ROAD
MERCER ISLAND, WASHINGTON**

**Project No. 26-005
February 2026**

Prepared for:

John Anderson



3213 Eastlake Avenue East, Ste B
Seattle, Washington 98102-3513
Tel: 206.262.0370 www.pangeoinc.com

*Geotechnical & Earthquake
Engineering Consultants*

February 16, 2026
File No. 26-005

John Anderson
5336 Butterworth Road
Mercer Island, WA 98040

**Subject: Geotechnical Report
Proposed Deck Reconstruction
5336 Butterworth Road, Mercer Island, WA**

Dear John,

Attached please find our geotechnical report for the proposed deck project at the above referenced site in Mercer Island, Washington. This report documents the subsurface conditions at the site and presents our geotechnical design recommendations for the proposed project.

In general, our boring drilled near the proposed deck location generally encountered about 5 feet of loose fill overlying native, medium dense to dense silty sandy gravel (lake deposits). Based on the soil conditions encountered in the boring and previous borings drilled in the site vicinity, it is our opinion that the proposed deck should be supported by a deep foundation system, such as driven small diameter pipe piles (pin piles). Temporary unsupported excavations may be sloped as steep as 1½H:1V (Horizontal:Vertical).

We appreciate the opportunity to work on this project. Please call if there are any questions.

Sincerely,



H. Michael Xue, P.E.
Principal Geotechnical Engineer

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**GEOTECHNICAL REPORT
PROPOSED DECK RECONSTRUCTION
5336 BUTTERWORTH ROAD
MERCER ISLAND, WASHINGTON**

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering study that was undertaken to support the design of the proposed deck project in Mercer Island, Washington. Our study was performed in general accordance with our mutually agreed-upon scope of work as outlined in our proposal dated June 10, 2025, and was subsequently approved on January 5, 2026. Our service scope included reviewing readily available geologic and geotechnical data in the site vicinity, conducting a site reconnaissance, advancing one test boring, and developing the geotechnical design recommendations presented in this report.

2.0 PROJECT AND SITE DESCRIPTION

The project site is an approximately 0.58 acre waterfront lot located at 5336 Butterworth Road in Mercer Island, Washington (See Figure 1 – Vicinity Map). The site is roughly L-shaped, and borders Lake Washington to the east and existing single-family residences to the other three sides. The site is currently occupied by a 2-story single-family house with a basement. Based on our review of the GIS maps, the existing site grade generally slopes down from the west to the east with an average gradient of about 7 percent and a total vertical elevation relief of about 15 feet between the east and west property lines.

Based on the information provided to us, we understand that it is proposed to re-construct the decks on the east side of the existing house (see Figure 2). The proposed project may also include other yard improvements. We anticipate that site grading for the proposed project may involve cuts and fill on the order of 2 to 3 feet.

Based on a review of the City of Mercer Island GIS maps, the site is mapped with potential slide and seismic geologic hazards. As such, a geotechnical report will be required as part of the building permit application. The objective of our geotechnical study is to explore the site subsurface conditions, evaluate the potential geologic hazards, and provide geotechnical design recommendations pertinent to the proposed project.

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above

project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed. In any case, PanGEO should be retained to provide a review of the final design to confirm that our geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

3.0 SUBSURFACE EXPLORATION

3.1 CURRENT EXPLORATION

One test boring (PG-1) was advanced at the approximate location shown in Figure 2. The boring was drilled at the site on January 23, 2026, using a portable Acker drill rig owned and operated by CN Drilling, Inc., subcontracted to PanGEO. The test boring PG-1 was drilled to a depth of about 16½ feet below the existing grade.

The drill rig was equipped with 5-inch outside diameter hollow stem augers. Soil samples were obtained from the boring at 2½- and 5-foot intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. The completed boring was backfilled with drill cuttings and bentonite chips.

A geologist from PanGEO was present during the field exploration to observe the drilling, to assist in sampling, and to describe and document the soil samples obtained from the boring. The soils were logged in general accordance with the system summarized on Figure A-1, Terms and Symbols for Boring and Test Pit Logs. The summary test boring log is included as Figure A-2 in Appendix A.

3.2 PREVIOUS SUBSURFACE EXPLORATIONS

As part of our study, we also collected and reviewed readily-available previous geotechnical exploration data in the site vicinity. Specifically, we reviewed the following previous boring data:

- Summary test boring logs B-1 and B-2, located to the north of the subject property (GCI, 2023).

The summary logs for the previous explorations are included in Appendix B for reference. The previous explorations were advanced up to about 31.5 feet below existing grade.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

Based on our review of the *Geologic Map of Mercer Island, Washington* (Troost and Wisher, 2006), most of the sites are mapped as Pre-Olympia Nonglacial Deposits (Geologic Map Unit Qpon) with Lake Deposits (Geologic Map Unit Ql) mapped in the eastern portion of the site along the lake. The principal characteristics of these soil units are described below:

- **Lake Deposits (Ql):** This unit predominantly consists of very soft to medium stiff or very loose to medium dense horizontally bedded silt and clay derived from lake-bottom sediments exposed by the lowering of Lake Washington in 1916. Local sand layers and peat may also be present. Locally gradational with Recessional Lacustrine, Alluvium, and Peat.
- **Pre-Olympia Nonglacial Deposits (Qpon):** This unit predominantly consists of silt, clay, sand, and gravel deposits of nonglacial origin that were overridden by Olympia age glaciations, and is typically very dense and hard in its undisturbed state.

4.2 SOIL CONDITIONS

Our test boring (PG-1) generally encountered a layer of loose fill overlying medium dense native silty sandy gravel (lake deposits). The following is a generalized description of the soils encountered in the boring. For a detailed description of the subsurface conditions encountered at our exploration location, please refer to the boring log provided in Appendix A.

Fill: Boring PG-1 encountered loose, silty sand with gravel from surface to about 5 feet below existing grade. We interpret this unit as fill based on the loose condition and disturbed appearance.

Lake Deposits (Ql): Below the fill, PG-1 encountered medium dense, silty, sandy gravel that extended to the maximum depth drilled at about 16½ feet below the surface. This unit appeared generally consistent with the lake deposits mapped on site.

The previous borings B1 and B-2 generally encountered soft peat, and loose to medium dense silty sand to about 15 to 20 feet overlying dense to very dense silty sand that extended to 26.5 and 31.5 feet below the ground surface.

The stratigraphic contacts indicated on the boring log represent the approximate depth to boundaries between soil units. Actual transitions between soil units may be more gradual or occur at different elevations. The descriptions of groundwater conditions and depths are likewise approximate.

4.3 GROUNDWATER CONDITIONS

Groundwater was observed at about 4 feet below existing grade in PG-1 at the time of exploration. Groundwater was recorded at about 4.5 and 6 feet below the ground surface in the previous borings drilled in March 2023. It should be noted that groundwater elevations and seepage rates may vary depending on the season, local subsurface conditions, and other factors. Groundwater levels and seepage rates are normally highest during the winter and early spring (typically October through May).

5.0 GEOLOGIC HAZARDS ASSESSMENT

5.1 LANDSLIDE HAZARDS AND STEEP SLOPES

Based on review of the Mercer Island GIS map, the eastern portion of the property is mapped as a potential slide geologic hazard area. On January 23, 2026, we conducted a reconnaissance of the site and site slopes. Based on our reconnaissance, we did not observe obvious indications of recent or historical slope movements, such as scarps, sloughs, tension cracks, uneven ground surfaces, jackstrawed trees, breaks in vegetation, water features and convergent landforms. Additionally, we observed that the adjacent properties are covered with bushes and trees. The trunks of the mature trees are observed to be straight.

We also reviewed a LiDAR image of the site and its vicinity, and the landslide inventory map from the Washington Department of Natural Resources (DNR). To the best of our knowledge, there are no reported past known slides at the site. According to the Mercer Island GIS maps, modern to ancient previous landslides are mapped on adjacent properties to the east and southeast, but no documentation is available, nor are they documented on the DNR landslide map or distinguishable on LiDAR.

In summary, based on our field observations, it is our opinion that the site appears to be globally stable in its present condition, and the landslide susceptibility at the site is considered low. It is also our opinion that the proposed deck reconstruction located to the east of the existing house as currently planned will not decrease the site stability or adversely impact the subject site and surrounding properties, provided that the proposed project is properly designed and constructed. It is our further opinion that building setback distance due to a potential landslide hazard is not needed for the proposed project.

5.2 EROSION HAZARDS

The site is not mapped within a potential erosion hazard area according to the City of Mercer Island's Geologic Hazards Map. Based on the results of our boring, it is our opinion that the site soil is anticipated to exhibit low to moderate erosion potential. The planned deck area will have limited ground disturbance for the proposed construction. In our opinion, the potential erosion hazards at the site can be effectively mitigated with the best management practice during construction and with properly designed and implemented landscaping for permanent erosion control.

During construction, the temporary erosion hazard can be effectively managed with an appropriate erosion and sediment control plan, including but not limited to installing a silt fence at the construction perimeter, limiting removal of vegetation to the construction area, placing rocks or hay bales at the disturbed/traffic areas and on the downhill side of the project, covering stockpile soil or cut slopes with plastic sheets, constructing a temporary drainage pond to control surface runoff and sediment trap if needed, placing rocks at the construction entrance, etc. Permanent erosion control measures should include establishing vegetation, landscape plants, and hardscape established at the end of project.

5.3 SEISMIC HAZARDS

Based on review of the City of Mercer Island Parcel Map, the site is mapped within a seismic/soil liquefaction hazard area.

5.3.1 Liquefaction Potential

Liquefaction is a process that can occur when soils lose shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration can result in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid. Soils with a potential for liquefaction are typically cohesionless, with a predominately silt and sand grain size, must be loose, and be below the groundwater table.

Based on our subsurface exploration, groundwater was encountered at about 4 feet. Based on the boring drilled and review of the previous borings, we anticipate that loose to medium dense soil will extend to about 20 feet deep overlying dense to very dense native soil. Due to the loose to medium dense soil conditions encountered below the groundwater, we performed liquefaction analysis based on the test boring PG-1 assuming dense to very dense soil at 20 feet, using the GeoLogismiki LiqSvs software suites. The input ground motion parameters in our analyses included a Magnitude 7.5 earthquake with a Peak Ground Acceleration (PGAm) of 0.677g to model an IBC code event. The liquefaction analyses, using the procedure proposed by the methodologies developed by Boulanger and Idriss (2014), indicate that the saturated, loose to medium dense sandy soil from groundwater table (i.e. 4 feet) to about 8 feet are potentially liquefiable during an IBC-code level earthquake. As a result of soil liquefaction, ground settlement may likely occur and the ground settlement due to soil liquefaction during a 2,475-year code level earthquake is estimated to be about one inch.

Due to the presence of relatively thick loose soil at the proposed deck location and the potential soil liquefaction during a design earthquake, we recommend that the proposed deck be supported on small diameter pipe piles (pin piles) to mitigate the potential excess post-construction foundation settlement. Recommendations for pile supported foundations are provided in [Section 6.2](#) below.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 SEISMIC SITE CLASS

The seismic design should be performed using the 2021 edition of the International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). Based on the site soil conditions, it is our opinion that Site Class D (stiff soil) should be used.

6.2 DECK FOUNDATIONS

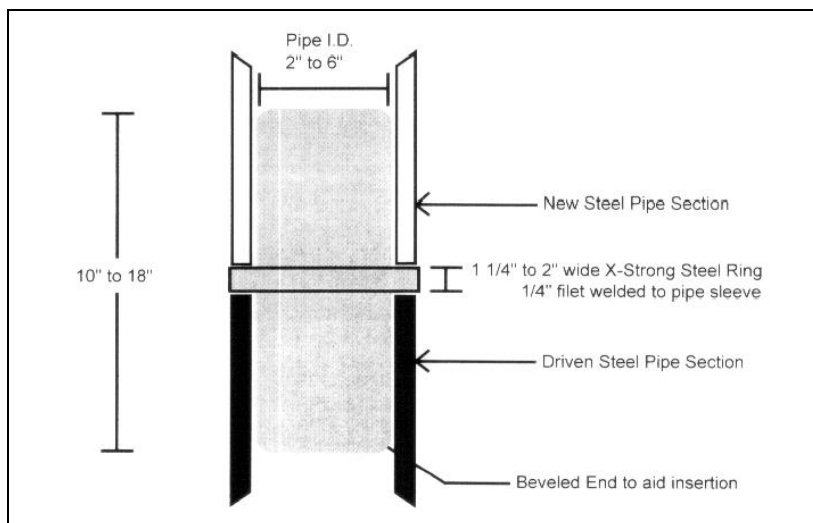
Based on the subsurface conditions at the site, our understanding of the current design and the results of our engineering analysis, it is our opinion that the proposed deck should be supported by a deep foundation system, such as small diameter driven steel pipes (pin piles), to avoid the excessive post-construction foundation settlement. Our pin pile design recommendations are presented in the sections below. Alternatively, helical piles may also be used to support the proposed deck and PanGEO can provide helical pile design recommendations if requested.

Based on the site access conditions, it is our opinion that two-inch diameter pin piles are the most appropriate to support the proposed deck. The 2-inch diameter should consist of galvanized, schedule-80 steel pipes. The piles should be driven to refusal, which is defined as less than one inch of penetration during one minute of continuous driving, with either a 90-lb jackhammer or a 140-lb pneumatic hammer. Piles driven to refusal are considered adequate for supporting an allowable axial compression load of 3 tons per pile.

We estimate that settlement is anticipated to be less than ½ inch under anticipated loading conditions for the pile supported foundations. The tensile and lateral capacities of pin piles should be ignored in design calculations.

The number of pin piles needed, and pile layout should be determined by the project structural engineer. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see detail below). We discourage welding of pipe joints, particularly when galvanized pipe is used, as we have frequently observed welds broken during driving.

The quality of a pin pile foundation is dependent in part on the experience and professionalism of the installation company. Therefore, a qualified contractor with pin pile driving experience of similar projects should be selected to install the piles. We also recommend that PanGEO be retained to provide full time monitoring of the pin pile installation to confirm the capacity of the piles.



Lateral Forces – The capacity of pin pipes to resist lateral loads is very limited and should not be used in design. Therefore, lateral forces should be resisted by the passive earth pressures acting against the pile caps and below-grade walls or from battered piles (batter no

steeper than 3(H):12(V)). ***Friction at the base of pile-supported concrete grade beam should be ignored in the design calculations.*** Passive resistance values may be determined using an equivalent fluid weight of 100 pounds per cubic foot (pcf). This value includes a safety factor of about 1.5 assuming that properly compacted granular fill will be placed adjacent to and surrounding the foundations.

Estimated Pile Length – The required pile length in order to develop the recommended pile capacity may vary depending on the actual subsurface and driving conditions encountered. For planning and cost estimating purposes, we estimate that pile lengths will likely be on the order of 15 to 20 feet. It is our opinion that a minimum pile depth of 15 feet is considered appropriate.

6.3 RETAINING WALL DESIGN PARAMETERS

Retaining walls, if needed, should be designed to resist the lateral earth pressures exerted by the soils behind the wall. Proper drainage provisions should also be provided behind the walls to intercept and remove groundwater that may be present behind the wall. Our geotechnical recommendations for the design and construction of the retaining wall are presented below.

6.3.1 Lateral Earth Pressures

Cantilever walls should be designed for an equivalent fluid pressure of 40 pcf for a level backfill condition behind the walls assuming the walls are free to rotate. If the walls are restrained at the top from free movement, such as below-grade walls, an equivalent fluid pressure of 55 pcf should be used for a level backfill condition behind the walls.

Permanent walls should be designed for an additional uniform lateral pressure of 10H psf for seismic loading, where H corresponds to the buried depth of the wall.

The recommended lateral pressures assume the backfill behind the walls consists of a free draining and properly compacted fill with adequate drainage provisions.

6.3.2 Surcharge

Surcharge loads, where present, should also be included in the design of retaining walls. We recommend a lateral load coefficient of 0.35 be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half the wall height.

6.3.3 Lateral Resistance

Lateral forces from wind or seismic loading and unbalanced lateral earth pressures may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and by friction acting on the base of the foundations. Passive resistance values may be determined using an equivalent fluid weight of 150 pounds per cubic foot (pcf), assuming a level ground in front of the foundations. This value includes a factor of safety of 1.5, assuming the footing is poured against recompacted native soil or properly compacted structural fill adjacent to the sides of footing. A friction coefficient of 0.30 may be used to determine the frictional resistance at the base of the footings. This coefficient also includes a safety factor of approximately 1.5.

6.3.4 Wall Drainage

Provisions for wall drainage should consist of a 4-inch diameter perforated drainpipe placed behind and at the base of the wall footings, embedded in 12 to 18 inches of clean crushed rock or pea gravel wrapped with a layer of filter fabric. A minimum 18-inch-wide zone of free draining granular soils (i.e., clean crushed rocks or washed gravel) is recommended to be placed adjacent to the wall for the full height of the wall. Alternatively, a composite drainage material, such as Miradrain 6000, may be used in lieu of the clean crushed rock or washed gravel. The drainpipe at the base of the wall should be graded to direct water to a suitable outlet.

6.3.5 Wall Backfill

Based on the field exploration, the on-site soil would not be suitable for wall backfill due to its high fines content. As such, wall backfill should consist of imported, free draining, granular materials, such as WSDOT Gravel Borrow or approved equivalent.

Wall backfill should be properly moisture conditioned, placed in loose, horizontal lifts less than about 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. Within 5 feet of the wall, care should be exercised during compaction so that no extra pressure is exerted on the walls and the walls are damaged.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 SITE PREPARATION

Site preparation for the proposed project includes stripping and clearing of surface vegetation and excavations to the design subgrade for the proposed deck. All stripped surface materials should be properly disposed off-site or be “wasted” on site in non-structural landscaping areas.

Following site clearing and excavations, the adequacy of the subgrade where structural fill, foundations, slabs, or pavements are to be placed should be verified by a representative of PanGEO. The subgrade soil in the improvement areas, if recompacted and still yielding, should also be over-excavated and replaced with compacted structural fill or CDF/lean-mix concrete.

7.2 TEMPORARY EXCAVATION

Based on our understanding of the project, we anticipate that temporary excavations on the order of about 2 to 3 feet will be needed for the new deck foundation construction. As such, it is our opinion that unsupported slope cut excavations are feasible at the site. Based on the soil conditions at the site, for planning purposes, it is our opinion that temporary excavations may be sloped as steep as 1½H:1V.

All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. In general, temporary excavations deeper than a total of 4 feet should be sloped or shored. However, excavations less than 4 feet deep, if located along or near property lines, will also need to be sloped or supported if sufficient space is not available to lay back the excavations without encroaching into neighboring properties.

The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions and may need to be flattened in the wet seasons and should be covered with plastic sheets. The cut slopes should be covered with plastic sheets in the raining season. We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

7.3 MATERIAL REUSE

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. The contractor should be aware that the site soils contain high fines content, and may be difficult to compact to the

requirements of structural fill. As a result, for planning purposes, we do not recommend the on-site soils be re-used as structural backfill for the project. Structural fill should consist of well-graded granular material, such as WSDOT Gravel Borrow or CSBC, or approved equivalent.

7.4 STRUCTURAL FILL PLACEMENT AND COMPACTION

Structural fill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557.

Depending on the type of compaction equipment used and depending on the type of fill material, it may be necessary to decrease the thickness of each lift in order to achieve adequate compaction. PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

7.5 EROSION AND DRAINAGE CONSIDERATIONS

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area from leaving the immediate work site. Temporary erosion control may require the use of hay bales on the downhill side of the project to prevent water from leaving the site and potential storm water detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to a positive and permanent discharge system.

Permanent control of surface water should be incorporated in the final grading design. Adequate surface gradients and drainage systems should be incorporated into the design such that surface runoff is directed away from structures. Potential problems associated with erosion may also be reduced by establishing vegetation within disturbed areas immediately following grading operations.

7.6 WET EARTHWORK RECOMMENDATIONS

It is our opinion that construction of the project can be accomplished during the wet season. However, performing earthwork activities during wet season is anticipated to be more costly than during dry weather conditions. Based on the anticipated soil conditions and topography in

the proposed construction area, it is our opinion that potential for erosion at the site can be adequately mitigated by employing sediment control best management practices (BMPs). Additional information and details of the BMPs discussed in this section can be found in the Washington State Department of Ecology's *Stormwater Management Manual for Western Washington, Volume II* (<http://www.ecy.wa.gov/programs/wq/stormwater/manual.html>). Sediment control BMPs should be installed/constructed and functional prior to land disturbing activities.

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below:

- All footing surfaces should be protected against inclement weather unless the footings can be poured immediately after the subgrade is exposed. It is the contractor's responsibility to protect the footing subgrade from disturbance. If needed, one option is to place 2 to 3 inches of lean-mix concrete or 4 to 6 inches of crushed surfacing base course (CSBC) on the exposed foundation subgrade as soon as the subgrade is exposed.
- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- Where practical, maintain vegetation buffers around cleared areas (BMP C101).
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing ¾-inch sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Geotextile silt fences should be strategically located to control erosion and the movement of soil. Erosion control measures should be installed along all the property boundaries.

- Excavation slopes and soils stockpiled on site should also be covered with plastic sheets.

8.0 STATEMENT OF RISK

We understand that the site is mapped with geologic hazard areas, specifically potential slide, seismic, and erosion geologic hazards. Per Mercer Island City Code Section 19.07.060.D.2, development within geologic hazard areas and critical slopes may occur if the geotechnical engineer provides a statement of risk with supporting documentation indicating that one of the following conditions can be met:

- a. The geologic hazard area will be modified, or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe; or
- b. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area; or
- c. Development practices are proposed for the alteration that would render the development as safe as if it were not located in a geologic hazard area; or
- d. The alteration is so minor as not to pose a threat to the public health, safety, and welfare.

It is our opinion that the proposed development meets the criteria (a), (c), and (d), above.

9.0 ADDITIONAL SERVICES

To confirm that our recommendations are properly incorporated into the design and construction of the proposed sauna, PanGEO should be retained to conduct a review of the final project plans and specifications, and to monitor the construction of geotechnical elements. Modifications to our recommendations presented in this report may be necessary, based on the actual conditions encountered during construction.

10.0 LIMITATIONS

We have prepared this report for use by John Anderson and the project design team. Recommendations contained in this report are based on a site reconnaissance, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

This report has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use of this report.

We appreciate the opportunity to be of service.

Sincerely,

PanGEO, Inc.

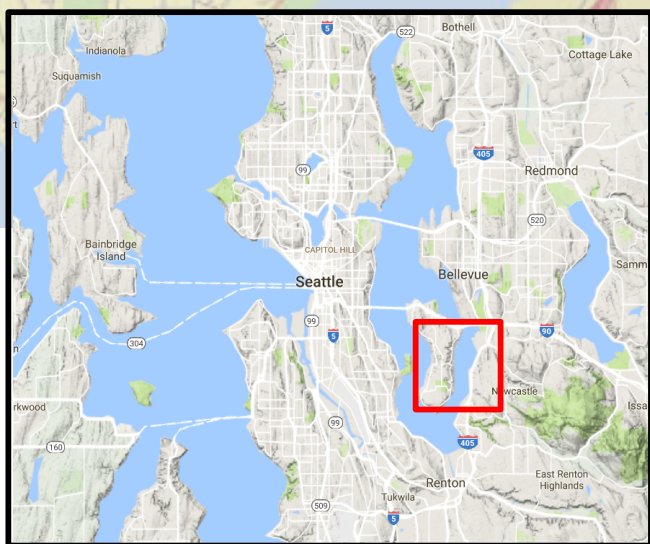


2/16/2026

H. Michael Xue, P.E.
Principal Geotechnical Engineer
(mxue@pangeoinc.com)

11.0 REFERENCES

- ASCE, 2016, *ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.
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Not to Scale

Base Map: King County iMap



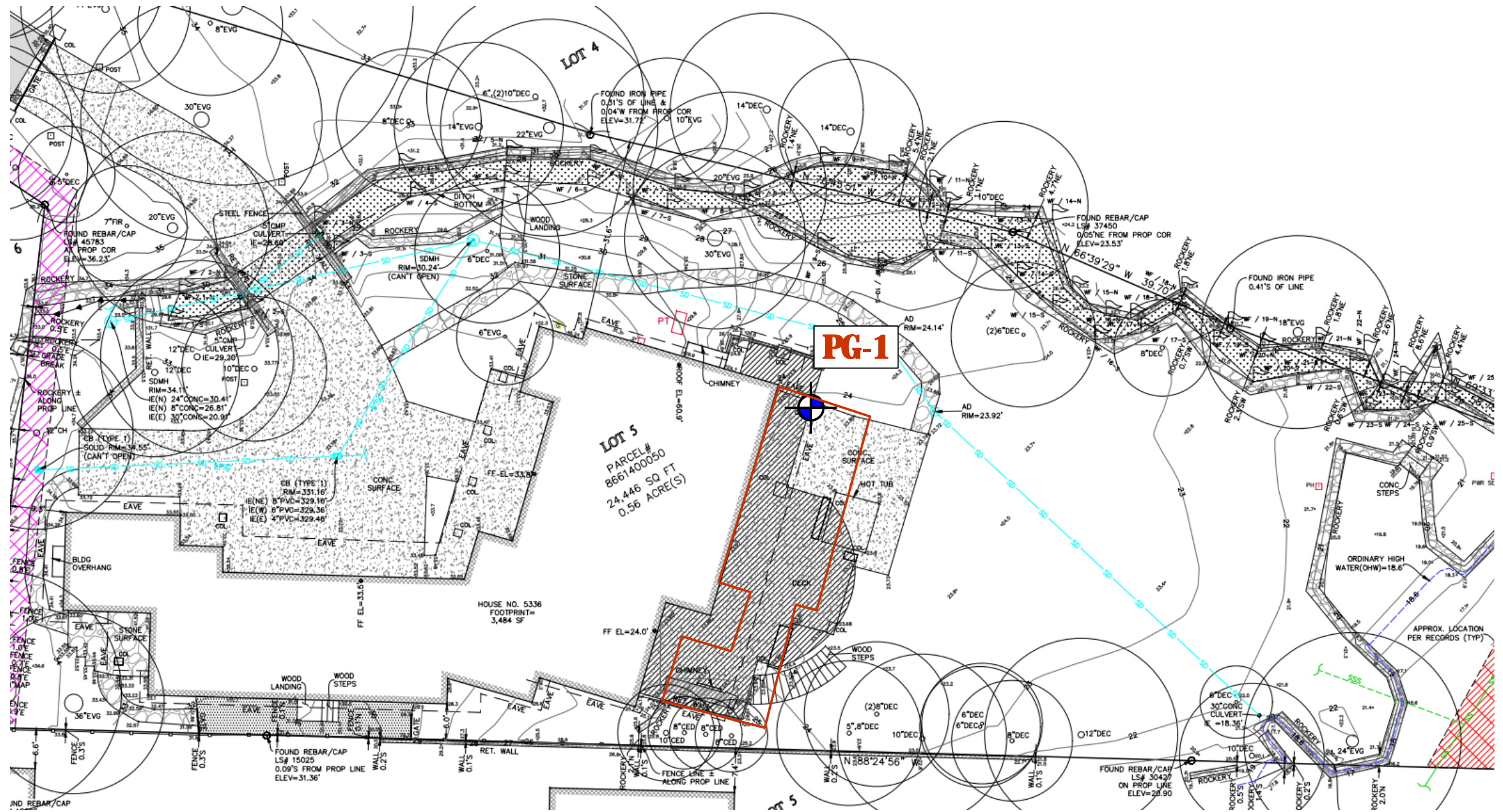
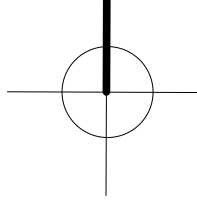
**Proposed Deck
Reconstruction
5336 Butterworth Road
Mercer Island, WA**

VICINITY MAP

Project No. **26-005**


Figure No. **1**

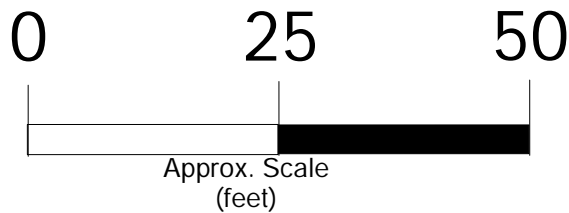
NORTH




LEGEND:

 Approx. Proposed Deck Area

 Approximate Test Boring Location
PanGEO, Inc., January 2025



	Proposed Deck Reconstruction 5336 Butterworth Road Mercer Island, Washington	SITE AND EXPLORATION PLAN	
		Project No. 26-005	Figure No. 2

APPENDIX A

SUMMARY TEST BORING LOG

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)		GW: Well-graded GRAVEL
	GRAVEL (>12% fines)		GP: Poorly-graded GRAVEL
			GM: Silty GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)		GC: Clayey GRAVEL
	SAND (>12% fines)		SW: Well-graded SAND
			SP: Poorly-graded SAND
			SM: Silty SAND
Silt and Clay 50% or more passing #200 sieve	Liquid Limit < 50		SC: Clayey SAND
			ML: SILT
			CL: Lean CLAY
	Liquid Limit > 50		OL: Organic SILT or CLAY
			MH: Elastic SILT
			CH: Fat CLAY
Highly Organic Soils			OH: Organic SILT or CLAY
			PT: PEAT

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

- ATT Atterberg Limit Test
- Comp Compaction Tests
- Con Consolidation
- DD Dry Density
- DS Direct Shear
- %F Fines Content
- GS Grain Size
- Perm Permeability
- PP Pocket Penetrometer
- R R-value
- SG Specific Gravity
- TV Torvane
- TXC Triaxial Compression
- UCC Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Split Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

- Notes:**
- Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.
 - The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered: Units of material distinguished by color and/or composition from material units above and below	Fissured: Breaks along defined planes
Laminated: Layers of soil typically 0.05 to 1mm thick, max. 1 cm	Slickensided: Fracture planes that are polished or glossy
Lens: Layer of soil that pinches out laterally	Blocky: Angular soil lumps that resist breakdown
Interlayered: Alternating layers of differing soil material	Disrupted: Soil that is broken and mixed
Pocket: Erratic, discontinuous deposit of limited extent	Scattered: Less than one per foot
Homogeneous: Soil with uniform color and composition throughout	Numerous: More than one per foot
	BCN: Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel	3 to 3/4 inches	Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
		Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Coarse Gravel:	3 to 3/4 inches	Silt	0.074 to 0.002 mm
Fine Gravel:	3/4 inches to #4 sieve	Clay	<0.002 mm

MONITORING WELL

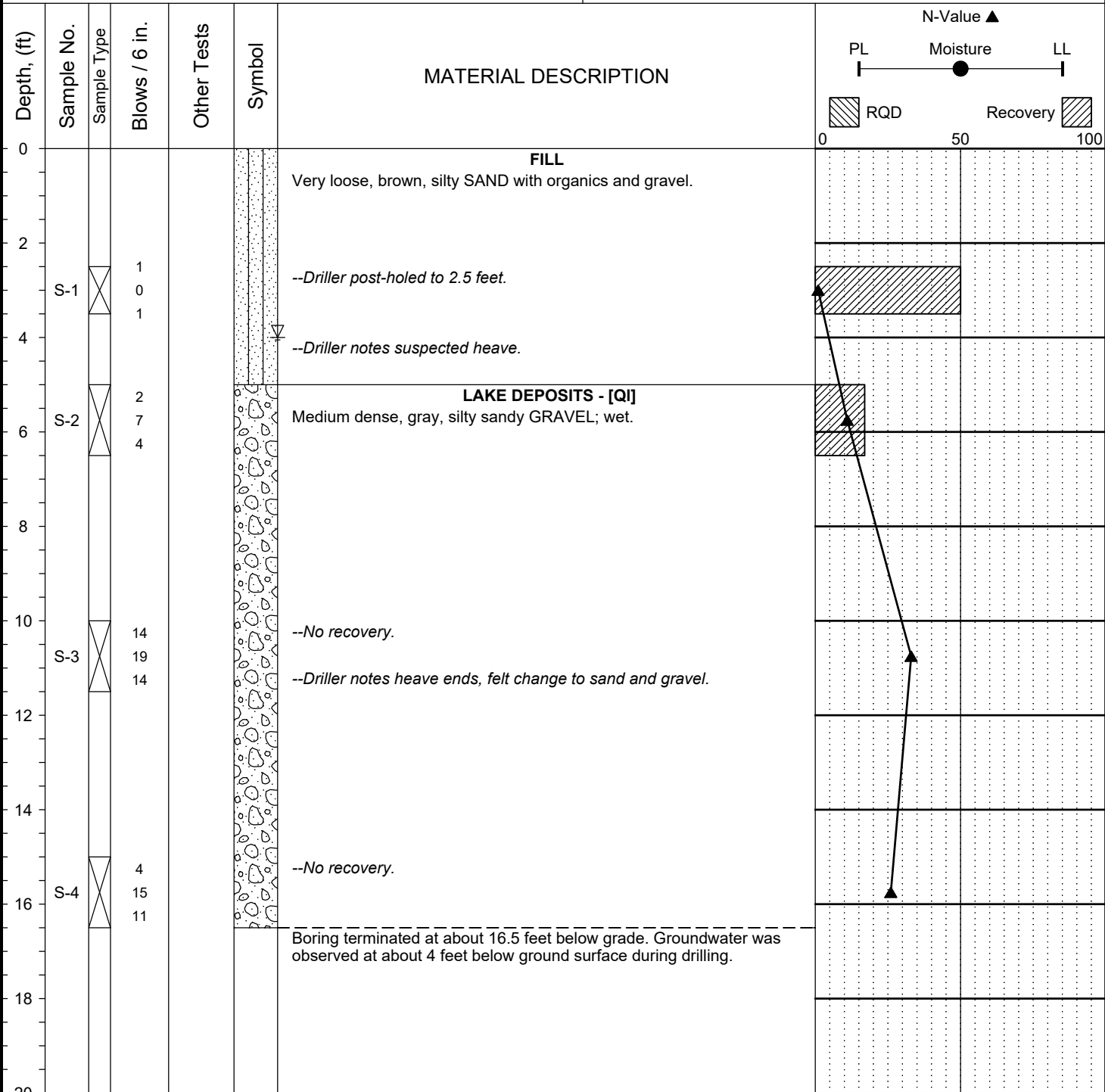
- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

LOG KEY 13-104 LOGS.GPJ_PANGEO.GDT 6/18/13

Project:	Proposed Deck Reconstruction	Surface Elevation:	25.0ft
Job Number:	26-005	Top of Casing Elev.:	N/A
Location:	5336 Butterworth Road, Mercer Island, WA	Drilling Method:	Portable Acker, Hollow stem auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

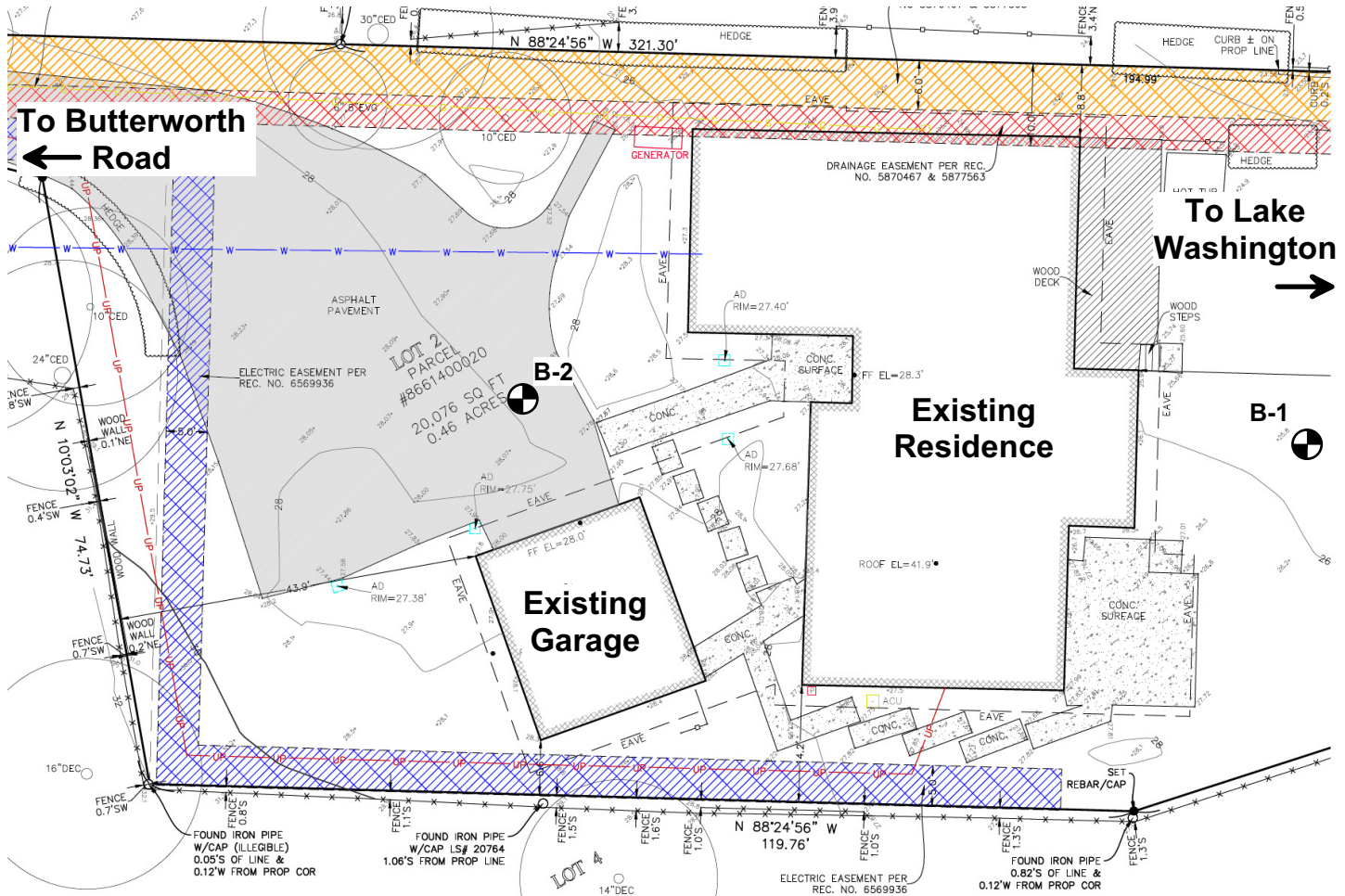
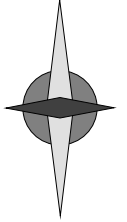


Completion Depth:	16.5ft	Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated from King County iMap topography
Date Borehole Started:	1/23/26	
Date Borehole Completed:	1/23/26	
Logged By:	J. Meissner	
Drilling Company:	CN Drilling	

APPENDIX B

SUMMARY PREVIOUS BORING LOGS

NORTH



Legend:

Test Boring Location

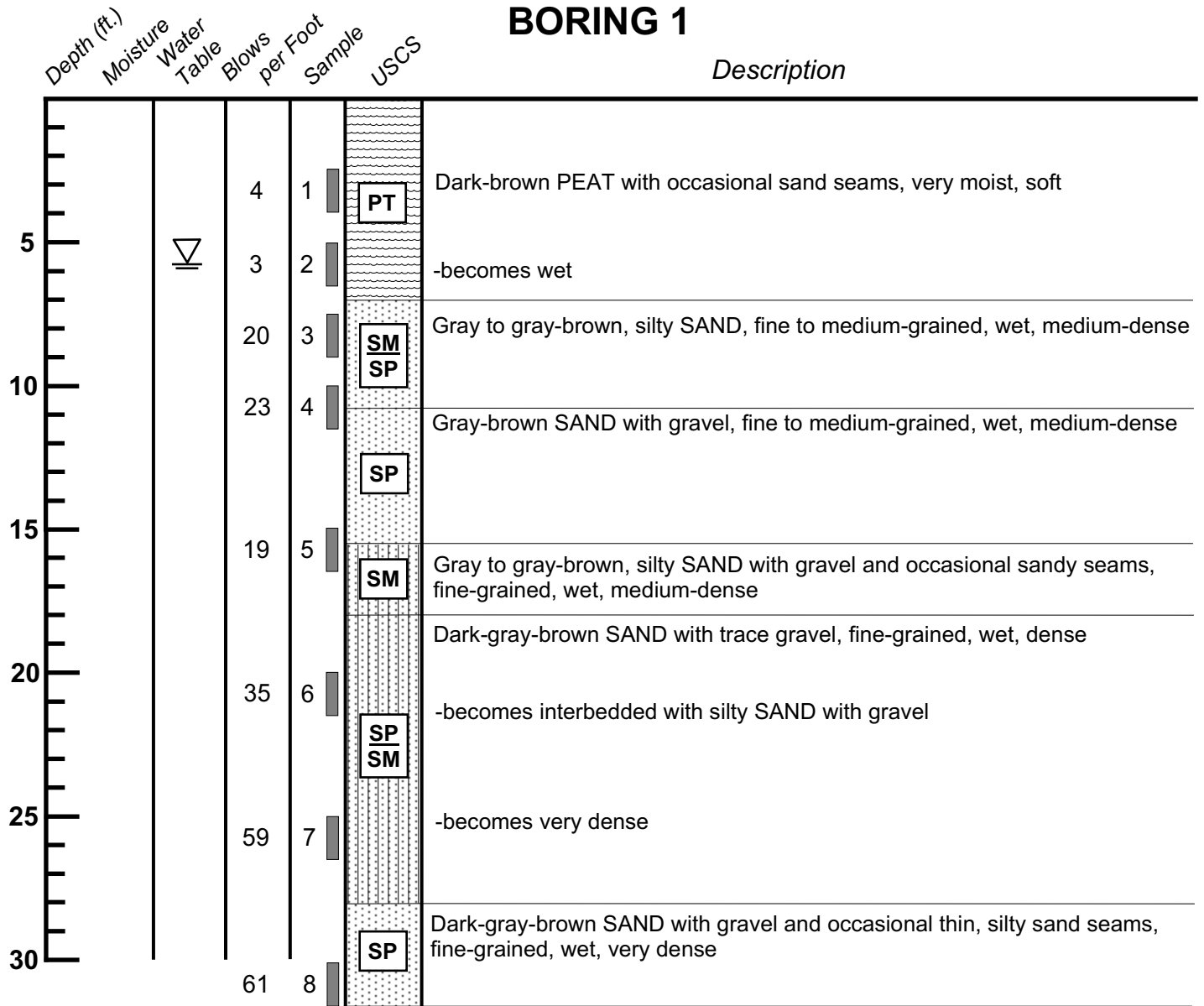


SITE EXPLORATION PLAN

5300 Butterworth Road
Mercer Island, Washington

<i>Job No:</i> 23030	<i>Date:</i> Mar. 2023	<i>No Scale</i>	<i>Plate:</i> 2
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BORING 1



* Test boring was terminated at 31.5 feet on March 2, 2023.

* Groundwater was encountered below 6 feet during drilling.



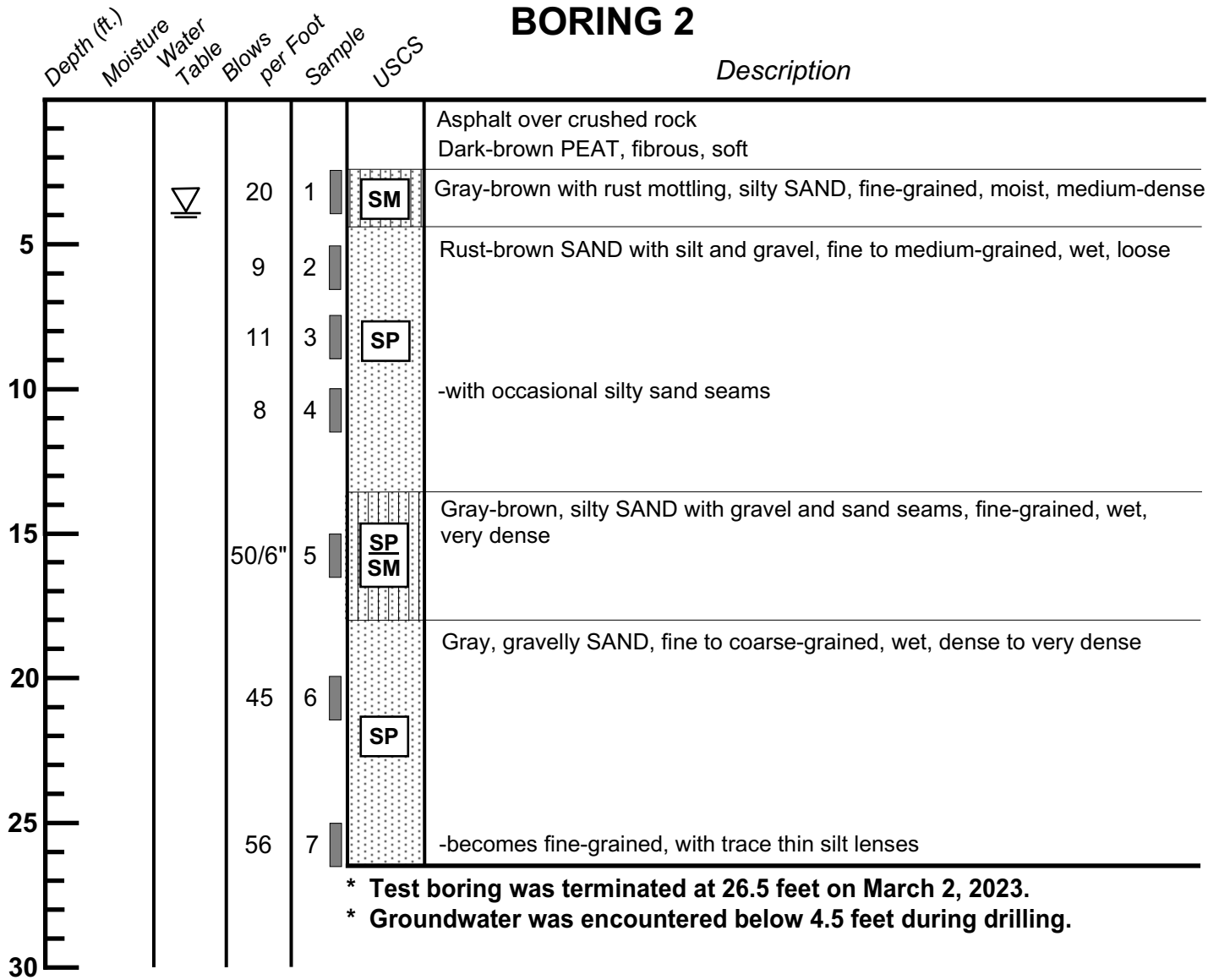
GEOTECH
CONSULTANTS, INC.

TEST BORING LOG

5300 Butterworth Road
Mercer Island, Washington

Job No: 23030	Date: Mar. 2023	Logged by: ASM	Plate: 3
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BORING 2



- * Test boring was terminated at 26.5 feet on March 2, 2023.
- * Groundwater was encountered below 4.5 feet during drilling.



TEST BORING LOG			
5300 Butterworth Road Mercer Island, Washington			
Job No: 23030	Date: Mar. 2023	Logged by: ASM	Plate: 4