SFA Design Group

STRUCTURAL/GEOTECHNICAL ENGINEERING, SPECIAL INSPECTIONS

December 4, 2025

Rebecca Ebsworth
P: (206) 713-6870
E: rebebsworth@gmail.com

RE: Geotechnical Engineering Report, Proposed Foundation Underpinning
4609 Forest Avenue Southeast
Mercer Island, Washington 98040
SFA Project Number: MFR25-284

As requested via contract authorization dated and signed November 3, 2025, SFA Design Group, LLC (SFA) is providing this geotechnical report which presents our subsurface investigation findings and provides geotechnical design and earthwork recommendations for the proposed underpinning project.

We appreciate the opportunity to be of service to you on this project. If you have any questions regarding our report or if there are additional services you require, please contact us.

Introduction

The purpose of this project is to provide design and construction recommendations in support of the proposed underpinning project. Our scope of services for this project include providing information and geotechnical engineering recommendations regarding:

- Surface and subsurface conditions
- Seismicity and liquefaction
- Geologic Hazards
- Stormwater management and drainage
- Preparation of site for construction and earthwork
- Small diameter direct push pipe pile (push pier) design and construction
- Small diameter driven pile (pin pile) design and construction

Project Description

Overview

The project site is located 4609 Forest Avenue Southeast in Mercer Island, Washington (Figure-A). Based upon elevation readings provided by Groundworks, SFA understands that the foundation is experiencing unacceptable settlements and cracking of the front patio and front entrance bay window. The purpose of the proposed foundation underpinning is to mitigate further settlement of the existing foundation system and prevent future cracking. Based on the geotechnical findings, a system of small diameter pipe piles, either driven or hydraulically pushed (pin piles or push piers, respectively), is an adequate method to mitigate the settlement the structure has experienced.

Site Description

The existing residence is a single family, two-story, wood frame structure with a daylight basement. The house was built in 1931 and is approximately 3,740 sq. ft. The residence lies on an irregularly shaped 18,860 sq-ft lot. To the west of the site is Lake Washington, north and south of the site are neighboring residences, and east of the site is Forest Ave SE. Site grading is terraced for development with locally flatter areas graded at approximately 5.5H:1V (horizontal:vertical) and steeper areas approaching 1H:1V downward to the west. Surrounding the site, existing topography is generally moderately to steeply sloped downward to the west.

Plan Review
As part of SFA's review process, we have reviewed the following documents for project conformance to the geotechnical report:

- "Ebsworth Residence Underpinning, 4609 Forest Avenue SE, Mercer Island, WA 98040." SFA Design Group, dated November 12, 2025.
- "Structural Calculations, Ebsworth Residence Underpinning, 4609 Forest Avenue SE, Mercer Island, WA 98040." SFA Design Group, dated November 17, 2025.

Based on our review of the drawings and calculations, SFA agrees that the drawings and calculations have been laid out in general accordance with this Geotechnical Report.

If the project is completed in accordance with the plans as written and the geotechnical recommendations for construction are followed, the risk of damage as a result of soil instability will be minimal on the property being developed and adjacent properties. It should be noted that while the risk is minimal, there is still some risk inherent with construction, but that these risks are low as a result of following drawings and specifications.

Geotechnical Site Characterization

Geologic Conditions
The subject site is located in Mercer Island, Washington, which is located in the Puget Sound region of Washington state. The near surface soil units and land forms are predominantly governed by the deposits and effects of glaciation due to the Vashon Stade of the Fraser glaciation (Troost Et. Al.). Prominent features include very dense soils due to consolidation by glaciation, and deeply incised north-south running valleys from the recession of the glacier.

Based on our review of the Washington Geologic Information Portal provided by Washington Department of Natural Resources (WaDNR; DNR Website), the site is mapped as Q_{gpc} – continental glacial drift, pre Fraser. Pre-Fraser glaciation deposits have many potential sources based on geologic conditions prior to the Fraser glacial age. Additionally, glacial drift soils are soils that were deposited by glaciers and tend to be dense/hard as a direct result of the prior glaciation as well. Subsequent to the most recent (Fraser) glaciation, the weight of the glaciers compacted soils into generally dense to very dense or hard conditions based on the underlying soil type.

Subsurface Conditions
SFA Design Group performed a subsurface investigation at the site by advancing one (1) hand auger, and one (1) dynamic cone penetrometer test (DCP). Our hand exploration was extended approximately 9.3 feet below existing ground surfaces on November 11, 2025. Hand augers were performed in conjunction with DCP tests to associate in-situ relative density or consistency with the soil types as sampled from the hand auger adjacent. A DCP test uses an adjustable 10-pound or 17.6-pound, dual mass slide hammer falling a distance of 22 inches to drive a ¾" diameter cone into the ground. Specific mass used for explorations(s) are shown on the log(s). Blows are counted on 2-inch intervals and based on the number of blows per 2-inch drive, in-situ density/consistency of the soils can be estimated.

Samples collected from hand auger grab sampling were classified in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure, ASTM D2488) and logged into field boring logs attached in Appendix-A of this report. An exploration map describing where the exploration was completed is included in Figure-B at the end of this report.

Based on soils observed in our explorations and mapped conditions, we have identified the following soils at the site:

Soil Type 1 – Basement Backfill
We encountered very loose to loose clayey sand and soft lean clay extending to a depth of approximately 8 feet below ground surface. Based on the nearby basement of the structure, and loose/soft nature of the materials, we interpret this material to be backfill placed after the building was constructed to raise grades to their existing grade. Groundwater was encountered in this layer.

Soil Type 2 – Lawton Clay
Underlying the backfill materials, we encountered a medium stiff to hard lean clay. The clay exhibited low to moderate plasticity and was moist, indicating that groundwater was not present in the material. The hardening consistency of the material indicates that it has been glacially overridden and is consistent with mapped pre-Fraser Lawton Clay materials. The clay extended the full depth of our exploration.

Groundwater
Groundwater was encountered at the site during our exploration program. We observed groundwater at approximately 6.4 feet below ground surface in HA-1. Based on observed soil conditions, it is our opinion that the water table is perched. Several factors can affect groundwater levels including, but not limited to, rainfall, runoff, and other seasonal factors.

Seismic Considerations

Ground Motion Design Parameters
When reviewing the 2021 International Building Code and ASCE 7-16 the following seismic data in the table below pertain to the subject site.

Seismic Parameter	Value
Latitude (degree)	47.5635383
Longitude (degree)	-122.2307351
Estimated Site Class	C
Site Coefficient, F_a	1.200
Site Coefficient, F_v	1.500
Mapped Spectral Acceleration at 0.2-sec Period, S_s	1.437g
Mapped Spectral Acceleration at 1.0-sec Period, S_1	0.499g
Spectral Acceleration at 0.2-sec Period Adjusted for Site Class, S_{MS}	1.725g
Spectral Acceleration at 1.0-sec Period Adjusted for Site Class, S_{M1}	0.749g
Design Spectral Acceleration at 0.2-sec Period, S_{DS}	1.150g
Design Spectral Acceleration at 1.0-sec Period, S_{D1}	0.499g
Seismic Design Category	D

The structural consultant should review the above parameters and the 2021 International Building Code to evaluate the seismic design.

Conformance to the criteria presented in the above table for seismic design does not constitute any type of guarantee or assurance that significant structural damage or ground failure will not occur during a large earthquake event. The intent of the code is “life safety” and not to completely prevent damage of the structure, since such design may be economically prohibitive.

Surface Displacement Due to Faulting, and Regional Seismicity
The subject site is located in a seismically active zone of the Pacific Northwest and within the influence of faults that are considered to be active or potentially active. An active fault is defined as a “sufficiently active and well-defined fault” that has exhibited surface displacement within the Holocene time (about the last 11,000 years). A potentially active fault is defined as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago). We reviewed the USGS Earthquake Program Quaternary Faults and folds database online and found the site is between two traces of the Seattle fault zone, approximately 0.87 miles to the north and 0.56 miles to the south. both fault traces have a slip rate of between 0.2 and 1.0 mm/yr. We anticipate that at this distance the risk of fault rupture at the site is low.

Liquefaction
During seismic shaking, rapid pore water pressure increase can occur in granular soils, resulting in a sudden loss of soil shear strength. During this time of excess pore pressure, granular soils also experience a reordering of particles which can produce settlements as pore water pressures dissipate after shaking ends. Additional issues that may arise due to liquefaction include sand boils with additional localized settlement due to ground loss, flow failure slides, and lateral spreading of soils in and around sloping ground.

Only perched groundwater was observed at the site in our explorations. Stable groundwater was not encountered during the site-specific explorations or research. Additionally, the subject site is mapped as having a very low risk of liquefaction according to the DNR website. It is our opinion the risk of liquefaction at the site is negligible.

Geologic Hazards

Landslide Hazard

The Washington Department of Natural Resources Washington Geologic Information Portal website the site is located in a known prehistoric landslide; additionally, based on the Mercer Island GIS, the site is noted as subject to Mercer Island landslide critical area ordinance. It is our opinion that based on the relative shallowness of glacially consolidated soils, lack of shallow groundwater, and the relatively shallow slope grading observed on site that the risk of landslide at the site is relatively low. However, the scope of work proposed at the site (limited foundation underpinning of existing structures) is unlikely to affect global stability at the site.

Erosion

According to the Mercer Island GIS map, the site is listed as a potential erosion critical area. Any proposed work will require compliance with Mercer Island City Code (MICC) Chapter 15.09 – Storm Water Management Program. Surface water runoff around any open excavation or pile of soil should be redirected away from the loosened soils, or the soils should otherwise be protected according to the provisions of MICC 15.09. Additional recommendations for erosion control may be found in Earthwork Recommendations for Wet Weather section of this report, below, however, if conflicts arise between the referenced practices and documents, MICC 15.09 shall take precedence.

General Recommendations

Drainage

All drainage systems disturbed during construction should be repaired prior to final approval. In the absence of adequate drainage SFA recommends that properly functioning positive site drainage should be maintained at all times. This includes the proper routing and discharge of French and/or curtain drains. Drainage should not flow uncontrolled down any descending slope or retaining wall. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. Drainage for any hardscaped area should be directed toward the street/parking or other approved area. Roof gutters and down spouts should be utilized to control roof drainage. Down spouts should outlet a minimum of 5 feet from the proposed structure and/or into an approved drainage point. The effects of improper or non-existent drainage systems would be likely contributors to typically observed settlement of structures.

Earthwork

We anticipate that earthwork at the site will include localized demolition of landscape and hardscape adjacent to the existing structure, excavations to expose foundations for underpinning, and backfilling the excavations. Care should be taken prior to excavation to locate building utilities to avoid damage. Where existing utilities conflict with proposed underpinning efforts, contact SFA for updated design recommendations.

While excavations are open during construction, care should be taken to maintain the natural moisture content, and protect subgrade stability until completion of underpinning efforts. If water is allowed to accumulate at the bottom of any excavation, it can increase the likelihood that foot and/or construction traffic may disturb subgrade soils. Disturbed subgrades can reduce design capacities of the underpinning elements or reduce lateral resistance of the final system. If subgrades become disturbed, we recommend that disturbed soils either be removed prior to placement of any backfill materials, or the disturbed subgrade should be moisture conditioned and recompacted.

In general, any excavation should observe OSHA safety requirements. On site soils may be considered soil type C for temporary excavations. Excavations should be no more than 4 feet deep relative to surrounding surfaces. Deeper excavations should be sloped at 1.5H:1V (horizontal:vertical) or flatter, or benched at an equivalent slope of 1.5H:1V with no benches taller than 4 feet high.

Based on the observed soil types at the site, we anticipate that conventional earth moving equipment will be adequate to achieve the planned excavations. The bottom of the excavations should be cleared of any loose soil or gravel, and any disturbed soils prior to backfill placement.

If foundation cracks are observed during excavation at the site, we recommend sealing cracks to prevent water from passing through the foundation. Similarly, if the structure is to undergo lifting to relevel the foundation, cracks may appear in flooring and/or the surrounding foundation elements. Any cracking should be repaired after installation is complete.

Fill Type and Placement
Materials on site are considered acceptable for re-use and fill as general fill, e.g. anywhere not to be supported by retaining walls or under shallow foundations. Fill should be placed in level lifts and be compacted at least 90% of the maximum dry density as defined by ASTM D1557. Using hand held equipment such as jumping jacks or plate compactors, the loose lift thickness should not exceed 6 inches. If larger equipment, such as a hoe-pack is used to compact backfill, loose lifts up to 12 inches thick may be used.

Earthwork Recommendations for Wet Weather
Near surface soils appear to have relatively high fines content (greater than approximately 5% by weight) based on our visual-manual observations. These soils will be sensitive to increases in moisture content and will be difficult to compact to a firm, unyielding condition if the moisture content is more than approximately 4 percentage points above or below the optimum moisture content as defined by the point at which the soil reaches its peak dry density in an ASTM D1557 test. If earthwork is planned during times of prolonged inclement weather, we recommend importing a clean, free-draining material, or backfilling excavations with lean-mix concrete.

Measures for temporary erosion control may be required if work will occur during periods of wet weather. If necessary, guidelines can be found in Section III-1 – Choosing Your Source Control BMPs of the Stormwater Management Manual for Western Washington (June 2024).

Site Specific Recommendations

Executive Summary
Based on our research, site visit, explorations, and calculations, the settlement observed at the site is likely the result of settlement of fill placed during construction. To cease this settlement, extending loads down to the underlying very stiff to hard Lawton Clay materials is recommended. To this end, we recommend use of small diameter hydraulically pushed or driven pipe piles (push piers or pin piles, respectively) to transfer the loads to the very stiff to hard clay.

Small Diameter Direct Push Pile Underpinning (Push Piers)
Small diameter direct push piles, also known as push piers, generally consist of approximately 2- to 4.5-inch nominal diameter steel pipes hydraulically pushed to end bearing refusal using the existing structure weight as the counterweight for the hydraulic ram. Push piers are installed segmentally using interlocking pipe components to extend the length of the pile as it is pushed into the soil, thus producing a small footprint for construction. Additionally, since the existing structure is used as a counterweight to help drive the piers, the capacity of the piers is tested during installation.

Push Pier Design Recommendations
The following table summarizes the anticipated capacity of push piles driven to refusal as a function of the nominal diameter of the pile.

Pipe Diameter	Allowable Capacity (FS = 2)
2 inches	6,000 lbs
3 inches	12,000 lbs
4 inches	20,000 lbs
6 inches	30,000 lbs

Push piers, to achieve the specified allowable capacities, should be spaced a minimum of 3 pile diameters clear space between piles. Push piers spaced closer together may not achieve full capacity, if closer spacings are required, please contact SFA for updated design parameters. Push piers should expect to be driven to a minimum depth of 5 feet before achieving refusal. This assumes that the skin friction reducer is installed correctly at the tip of the pier. Specific termination criteria for push pier underpinning will be a function of the specific design

capacities of the piers and the installation equipment to determine final driving forces. The structural engineer will need to work with the contractor to establish termination criteria for push piers.

It should be noted that the lateral resistances of the structure be checked after completion of underpinning due to the reduction of sliding resistance through base friction of the structure at the push pile location(s).

Push Pier Construction Recommendations
Push piers require skin friction reduction collars during installation to achieve the full bearing capacity described in this report. Without a skin friction reduction collar, piers may stop before reaching the target depth of embedment described in Push Pier Design Recommendations. If minimum depth required cannot be reached, pre-drilling of push pier locations may be required.

Continuous special inspection is required during installation per 2021 SBC Section 1705.1.1. A representative of SFA should be on-site during installation of all push pier underpinning elements.

Push Pier Lateral Capacity
The scope of work is limited in nature and, as such, unlikely to greatly affect lateral capacities of the remainder of the structure. Lateral capacities do not need to be verified for this project.

Push Pier Observations and Testing
Continuous special inspection is required during installation per 2021 IBC Section 1810.4.12. Load testing shall be performed in accordance with ASTM Method D1143 (Quick Method) on 3 percent of helical piers but not less than one (1) pier and will be selected and observed by the special inspector and that the maximum load of the test be twice the design load. An alignment load (AL) shall be applied to the pile prior to setting the deflection measuring equipment to zero or a reference position. The AL shall be no more than 10% of the DL. Incremental loading shall be in accordance with the following schedule.

Test Loading Schedule	Hold Time	Max Deflection
AL (.10 DL Max)	0 min.	N/A
0.25 DL	Until Stable	N/A
0.50 DL	Until Stable	N/A
0.75 DL	Until Stable	N/A
1.00 DL	Until Stable	N/A
1.25 DL	Until Stable	N/A
1.50 DL	Hold for Creep Test (See Below)	0.04 inches
1.25 DL	Until Stable	N/A
1.00 DL	Until Stable	N/A
0.75 DL	Until Stable	N/A
0.50 DL	Until Stable	N/A
0.25 DL	Until Stable	N/A
AL	Until Stable	N/A

Load testing creep acceptance criteria shall be no greater than 0.04 inches within a 10-minute period. Pier movements shall be measured at 0, 1, 2, 3, 5, 6, and 10 minutes. If movement is observed greater than 0.04 inches within the 10-minute period the load test shall be held for an additional 50 minutes, the pier is to be deepened and re-tested, or the pier is to be abandoned and replaced with a new pier. If the load test is to be held the pier movements shall be measured at 15, 20, 30, 40, 50, and 60 minutes. The creep versus the logarithm of time shall be plotted. If the creep rate is less than 0.080 inches between 6 and 60 minutes, the load test shall be considered successful.

Small Diameter Driven Piles (Pin Piles)
Small diameter driven piles, also known as pin piles, generally consist of 2- to 6-inch nominal diameter steel pipes driven to refusal by a small percussive hammer. The hammer may be handheld or mounted to machinery such as a skid-steer or small excavator, depending on the size of the pile.

Pin Pile Design Recommendations
The following table summarizes the anticipated nominal capacity of pin piles driven to refusal (defined as less than 1" of movement observed over a 1 minute time span with a suitable hammer for the pipe size) as a function of the nominal diameter of the pile.

Pipe Diameter	Allowable Capacity (FS = 2)
2 inches	6,000 lbs
3 inches	12,000 lbs
4 inches	20,000 lbs
6 inches	30,000 lbs

Pin piles should be expected to reach a minimum depth of 5 feet before achieving refusal.

Pin Pile Construction Recommendations
Piles should be driven to refusal, defined as 1 inch or less of penetration by the pile over a 15-second period of constant driving. The depth to refusal may vary between piles based on local conditions at the time of construction. A representative of SFA Design Group should be on site during installation of pin piles to confirm pile refusal criteria.

Pin Pile Testing and Lateral Capacities
The recommendations made for push piers testing and lateral capacities may also be used for pin pile testing and lateral capacities.

Limitations and Closure

This report has been prepared for the exclusive use of Rebecca Ebsworth and their design consultants relative to maintaining the subject site. No portion of this report may be used by other parties or for other purposes. The exploratory work was performed at a limited number of locations on the perimeter of the existing building. SFA considered a number of unique, project-specific factors when establishing the scope of services for this report. This report has not been prepared for use by other parties, and may not contain sufficient information for the purposes of other parties.

Our findings were obtained in accordance with generally accepted current professional principles and local practice in geotechnical engineering and reflect our best professional judgment based on experience and gathered data. It is to be understood that geotechnical information is characterized by a degree of uncertainty. Judgments rendered meet current professional standards; no other warranty is issued, either expressed or implied. The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the limited number of test borings and the results of laboratory testing and engineering analysis. As part of the engineering analysis it has been assumed, and is expected, that the geotechnical conditions that exist across the area of study are similar to those encountered in the borings. However, no warranty is expressed or implied as to the conditions at locations or depths other than those explored. If a period of one year elapses since preparation of this report, the geotechnical consultant should verify the current site conditions, and provide any additional recommendations (if necessary) prior to construction.

Thank you for the opportunity of providing our services to you on this project.

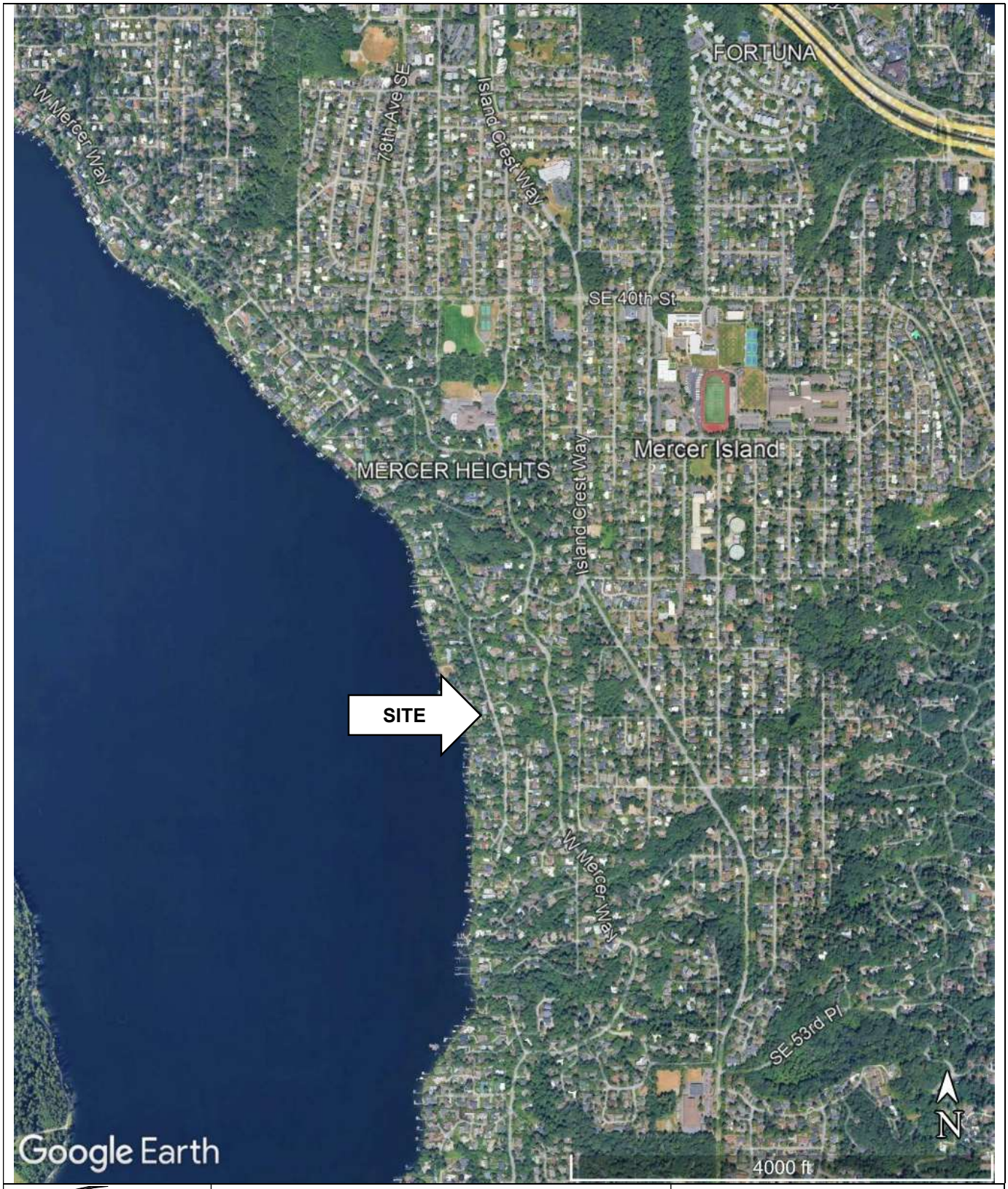
Sincerely,

SFA Design Group, LLC

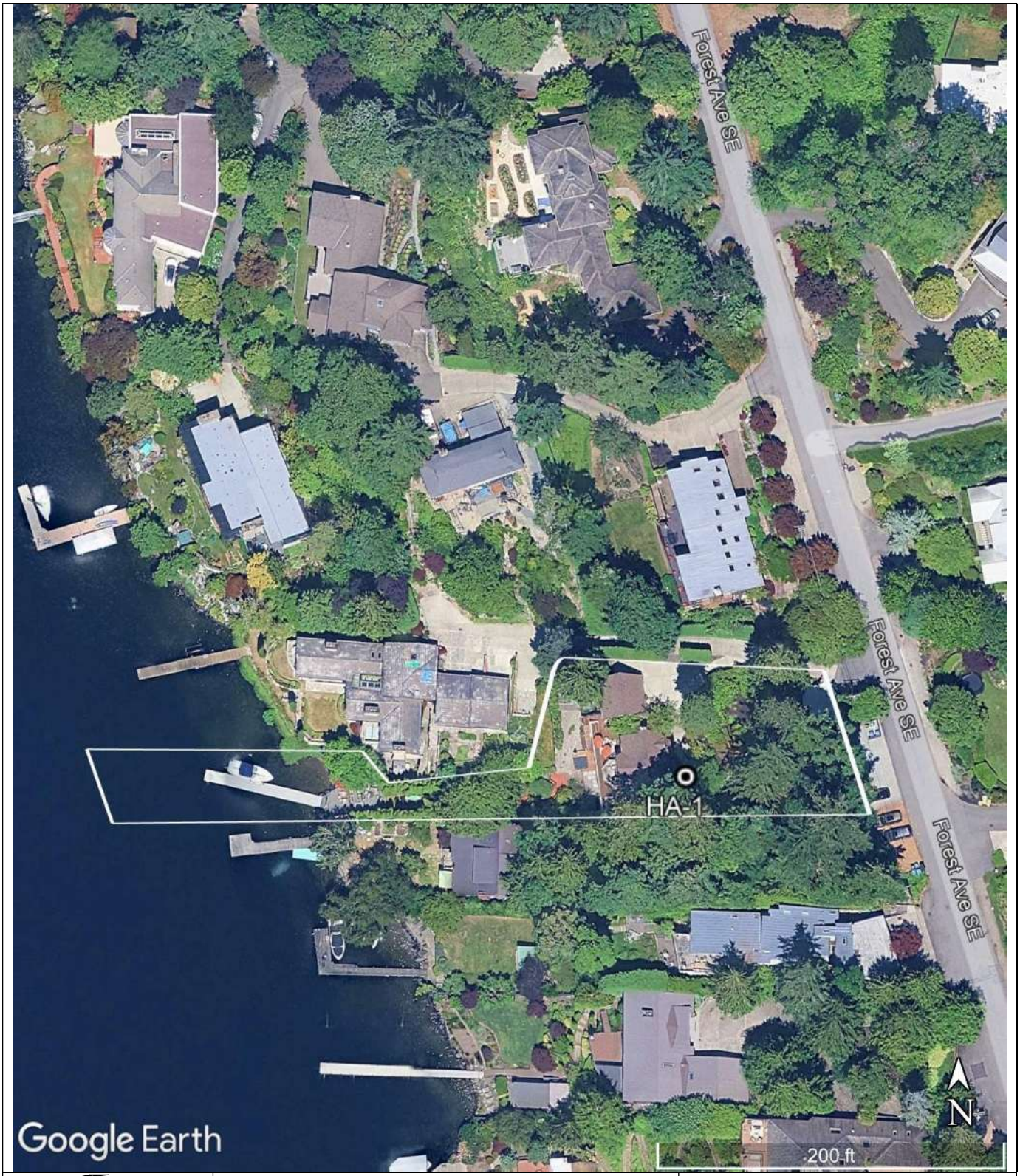
12/4/2025

Tristan T. Anderson, P.E.
Principal

Attachments:
Figure-A – Site Location Map
Figure-B – Exploration Plan
Appendix A – Field Exploration Logs



sfa	**SITE LOCATION** **4609 Forest Ave SE,** **Mercer Island, WA 98040**	Project Number: MFR25-284 FIGURE-A



sfa	**EXPLORATION PLAN** **4609 Forest Ave SE,** **Mercer Island, WA 98040**	Project Number: MFR25-284 FIGURE-B

Appendix A
Field Exploration Logs

Date Started:	Date Completed:	SFA Project No.		Boring Log:
11/11/2025	11/11/2025	MFR25-284		**HA-1**

Location:	Drilling Subcontractor:	Drill Rig:	
Near front entrance of house	SFA Design Group	Hand Auger	
Logged by:	**Checked by:**	**Drill Bit:**	**Drill Diameter:**
D. Hawthorne	TTA	Hand Auger	2 inches
Drill Crew:	**Total Depth:**	**Hammer Type:**	**Hammer Wt (lb)** / **Hammer Drop (in)**
N/A	9.3 feet	Army Corps DCP	17.6 / 22

Depth to Groundwater	Latitude:	Longitude:
6.4 feet	47.56353	-122.23066

Depth (feet)	Sample Type	DCP Blows (8kg hammer)	Sample No.	Graphic Log	Material Description
0		0			Bare earth and topsoil - approximately 2 inches thick
		0			(SC) Clayey Sand, very loose to loose, grayish brown, moist, trace rootlets
1		0			
		0			
		0			
		0			
		0			
		0			
		2			
		1			
		0	S-1		Grades to gray,
2		1			
		1			
		1			
		2			
		2			
		2			
3		1			
		1			
		1			
		1			
		1			
		2			
		1			
		2	S-2		Grades sandier, brownish gray
4		1			
		2			
		3			
		2			
		2			
		2			
5		2			
		3			
		3			
		2	S-3		(CL) Sandy Lean Clay, soft to medium stiff, gray, moist to wet
6		2			
		2			Grades wet
		3			
		2			
		1			
		2			
7		2			
		2			
		1			
		2	S-4		Grades less sandy
		2	S-5		(CL) Lean Clay, medium stiff to stiff, gray, dry to moist, low to moderate plasticity
8		1			
		3			
		5			Auger refusal at approximately 8.5 feet below ground surface on gravelly material
		6			Grades very stiff to hard
		7			DCP terminated at approximately 9.3 feet below ground surface
9		9			
		11			
		12			
10					
11					
12					
13					

Project:	Ebsworth Residence
Location:	4609 Forest Ave SE Mercer Island, WA 98040