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September 12, 2025
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Project No. 23-017

Mr. David Do
4649 Forest Ave SE
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Subject: Geotechnical Services Report Addendum
Beachfront DADU
4649 Forest Ave SE, Mercer Island, Washington

This letter addresses the city request for additional subsurface information and characterization to further evaluate potential seismic hazard issues with the project site.

Background/Existing Conditions

The area of the subject property where a new DADU is proposed is currently developed with a decades old small beach cottage that is quite debilitated and currently used as a storage shed. According to the Mercer Island geological hazard maps the property is classified as a critical area due to landslide, erosion and seismic hazards. Based on the current plans the existing beach cottage/shed will be demolished and a new 864 square foot DADU will be constructed on the site. Access to the property is via a trail and stairs down the slope from the existing main residence to the beach.

The subject building site is located at beach level on the far western portion of the overall property. The property begins along the west side of the Forest Ave SE right of way and extends approximately 425 feet to the west, well beyond the existing shoreline. An existing residence is located on the upper portion of the property adjacent to the street. Landscaping and stairs and a portion of a steep concrete driveway occupy the midportion of the parcel. Near beach level the parcel is nearly flat and contains the old cottage/shed and a boat dock. The shoreline is protected from erosion with a rockery. The steep concrete driveway leads to an existing residence that is on the adjacent property to the south. The adjacent residence to the south and the adjacent residence to the north, are both located at beach level and presumably on similar soil conditions.

Subsurface Exploration

On August 1, 2025, a limited access Acker drill rig was mobilized to the site by Geologic Drill Partners, Inc. This small, limited access drill rig was necessary as access for any larger type of drill rig was inordinately difficult.

Two borings were drilled for this small project and small site. The approximate boring locations were measured from existing features at the site and are indicated on the attached Site & Exploration Plan. The borings were drilled to the maximum capacity of the Acker drill rig.

Soil samples were obtained from the borings at 2 ½ and 5-foot intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1556) in which the samples are obtained using a 2 inch outside diameter split spoon sampler. The sampler was driven into the soil 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 sampler 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. After auger withdrawal the borings were backfilled with drill cuttings and bentonite chips in accordance with Department of Ecology guidelines. A licensed engineering geologist was present throughout the field exploration to observe the drilling, assist in sampling, and to document the soil samples obtained from the borings.

Boring EB-1 was drilled to 20 feet. The upper 15 feet consisted of medium stiff to stiff, silt with some sand to sandy silt. At a depth of 20 feet about 2 ½ feet of heaving sand into the auger was encountered and a soil sample was unable to be taken at that depth. The sample above at 15-16 ½ feet, and observations from the driller, indicated a contact between fine grained and granular soil at a depth of about 16 feet. Stiff to very stiff, brown silt to sandy silt was observed to be overlying medium dense to dense, gray, gravelly sand.

Boring EB-2 was drilled to 20 feet and sampled to 21 ½ feet. No further drilling could be done as the hard silt/clay soil bound up the auger such that it could not be turned. Observed soils above the hard silt/clay consisted entirely of stiff to very stiff, silt with trace sand to sandy silt.

Groundwater

No significant amount of ground water was observed in either boring. Saturated soil was observed about 6 feet deep in EB-1 and minor seepage was observed at about 8 feet depth in EB-2. The fine grained soil likely smeared the sides of the borings and restricted ground water flow into the borings. Given the nearness to the lake, ground water will ultimately reflect the lake level which is about 5 feet below ground surface on the west side of the site.

Other Nearby Studies

There have been multiple geotechnical studies performed all along the properties on Forest Ave SE. These studies were previously reviewed during our original geotechnical evaluation of the subject site. The most meaningful studies were for the property immediately adjacent to the site (4651 Forest Ave SE) and the next property to the south (4661 Forest Ave SE) as they are very near the subject site and have at least one exploration boring or pit at the same elevation as our current borings for the site. These explorations confirm our findings.

Site Geology

According to the Geology Map of Mercer Island, by Troost and Wisher, 2006, the site is mapped as Pre-Olympia fine grained non-glacial deposits (Qponf) overlying Pre-Olympia non-glacial deposits. The beach area where the planned DADU will be located is mapped as Holocene lake deposits (Ql). As stated in our previous geotechnical report, dated January 30, 2024, it is our opinion that the mapped lake deposits are reworked old landslide deposits. Also as previously stated, nearby studies where exploration borings have been placed into the near shore environment (lake deposits) have indicated that the materials are generally disturbed silt sediments overlying glacially consolidated silt/clay sediments.

Exploration borings EB-1 and EB-2 both encountered about 4 feet of disturbed material in the upper portion of the borings. In B-1 the soil had a disturbed matrix and in B-2 there was a slight blocky texture. This soil was medium stiff (B-1) closest to the lake and very stiff (B-2) furthest from the lake. Both samples consisted of moist, gray silt with some sand. This is most likely old fill soil but could also be colluvium from upslope sources.

Underlying this disturbed appearing soil was 9 to 12 feet of medium stiff to very stiff, gray silt with some sand to sandy silt with occasional pebbles and several small (1/2 inch thick) sand interbeds. Based on the density of this soil unit and lack of any organic material, we interpret this soil unit to be old landslide deposits from upslope sources and not lake deposits, as mapped on the published map for the area. This is in agreement with multiple studies in the area by others.

Below the old landslide deposits, the material in each boring was significantly different. At a depth of 15 ½ feet in EB1 the soil changed from silt to well sorted, medium sand with gravel. This material was medium dense (nearly dense) and based on drilling action appeared to extend to the bottom of the boring at a depth of 20 feet. As stated above, heaving sand into the auger prevented additional sampling of this material.

In EB-2, on the east side of the planned building pad, the soil changed at a depth of 13 feet from sandy silt to very stiff, becoming hard with depth, silty clay to clayey silt. Although a sample was able to be attained from 20 to 21 ½ feet, the auger could not be further advanced into the hard silt/clay with the limited access drill rig.

This lower material in each boring is consistent with the pre-Olympia nonglacial deposits mapped at the site. The high SPT-N values indicate that this material has been glacially overridden and consolidated.

CONCLUSIONS AND RECOMMENDATIONS

The findings in the two recently drilled exploration borings confirm the soil conditions encountered in the Hand Borings previously placed on the site for our January 30, 2024, geotechnical report. Soil with a potential for liquefaction is typically cohesionless, poorly sorted, fine to medium sand and must be loose and below the groundwater table. Although most of the soil underlying the planned building site will be below water table as influenced by the nearby lake level, the soil is predominantly fine-grained deposits. In the upper 15 feet only very thin (1/2 inch thick) sand seams were observed. Below a depth of 15 feet in exploration boring EB-1, nearest the lake, medium dense to dense, well graded sand with gravel was observed. However, the well graded soil matrix and density of this material is sufficient to mitigate liquefaction potential.

Based on these conditions, in our professional opinion, the liquefaction potential of the site is negligible and neither additional liquefaction analysis nor design considerations related to soil liquefaction are necessary for this project. The geotechnical recommendations provided in our January 30, 2024, technical report should be followed for design of the project except for the following:

Foundation Recommendations

Due to the presence of old landslide deposits on the site which may include variable soil that differs from that encountered in the exploration borings, it is our opinion that the new structure should be supported upon a pile foundation that extends into the underlying dense or hard native sediments. Small diameter pipe piles are generally the most cost-effective methodology for these conditions and the limited access of the site.

Based on the pile installation records from nearby sites and the exploration borings completed as described above, suitable bearing soils for small diameter pipe piles will be encountered at a depth of about 15 feet to 20 feet below existing ground surface. However, the piles may penetrate to a greater depth depending upon the exact subsurface conditions at the pile location.

<u>Small Diameter Pipe Piles</u>

The piles should be Schedule 80, 2-inch diameter, galvanized steel pipe that is driven to a minimum embedment depth of 5 feet into bearing soil and encounters effective refusal into the underlying natural, dense sediments with a 140 pound jackhammer (Rhinohammer). Current industry standards allow 2-inch diameter pin piles to have a load carrying capacity of 3 tons per pile when driven to effective refusal with the above jackhammer. Also 2-inch diameter pin piles only require geotechnical special inspection for approval and generally do not require load testing. However, if any unusual conditions are observed during installation, the geotechnical professional may recommend load testing of 1 or more piles. Effective refusal criteria is a maximum of 1 inch of penetration for 60 seconds of continuous driving over a minimum of 3 cycles.

The underlying medium dense to dense granular sediments and hard silt sediments do not present a liquefaction hazard during a seismic event and there is no reduction in pile capacity due to potential down drag from the surrounding soils.

Each pile or group of piles should be connected to other piles using a grade beam designed by a structural engineer to span between the piles. Resistance to lateral loads would be provided either by passive soil resistance against the grade beams or installation of batter piles. A passive equivalent fluid pressure equal to 100 pounds per cubic foot (pcf) can be used for this purpose provided the grade beams are backfilled with soil that is compacted to a medium dense or better condition. There should be no allowance for friction between the grade beams, slabs and the soil.

We do not recommend assigning any lateral resistance to a small diameter pipe pile that is installed vertically. If batter piles are necessary to resist lateral loads for the new footings the lateral resistance would be equal to the horizontal component of the axial pile load. The batter pile may be assumed to carry the full vertical design load of 6 kips. The maximum recommended batter is 1H:4V or about 15 degrees.

The actual total length of each pile will be determined based on conditions encountered during installation. However, based on the underlying soil conditions it appears that the piles should penetrate to a minimum depth of 20 feet in order to be embedded into the underlying bearing soil. These small diameter pipe piles are only allowed to a maximum installed depth of 30 feet. Should any piles penetrate deeper than 30 feet it will be necessary to re-evaluate and either drive 3 inch diameter piles or switch to helical anchors/piles.

Since completion of the pile takes place below ground, the judgment and experience of the geotechnical professional must be used as a basis for determining the required penetration and acceptability of each pile. Consequently, use of the presented capacity in the design requires that a qualified geotechnical professional from our firm, who will interpret and collect the installation data and examine the contractor's operations, inspect all pile installations. A final summary report would then be distributed following completion of pile installation. This will also be required by the city and should be indicated on the plans.

Anticipated settlement of footings founded on a pipe piles should be on the order of ½ inch or less with differential settlements of approximately one-half that amount between comparably loaded footings.

Our findings and recommendations provided in this report were prepared in accordance with generally accepted principles of engineering geology and geotechnical engineering as practiced in the Puget Sound area at the time this report was submitted. We make no other warranty, either express or implied.

Respectfully submitted,

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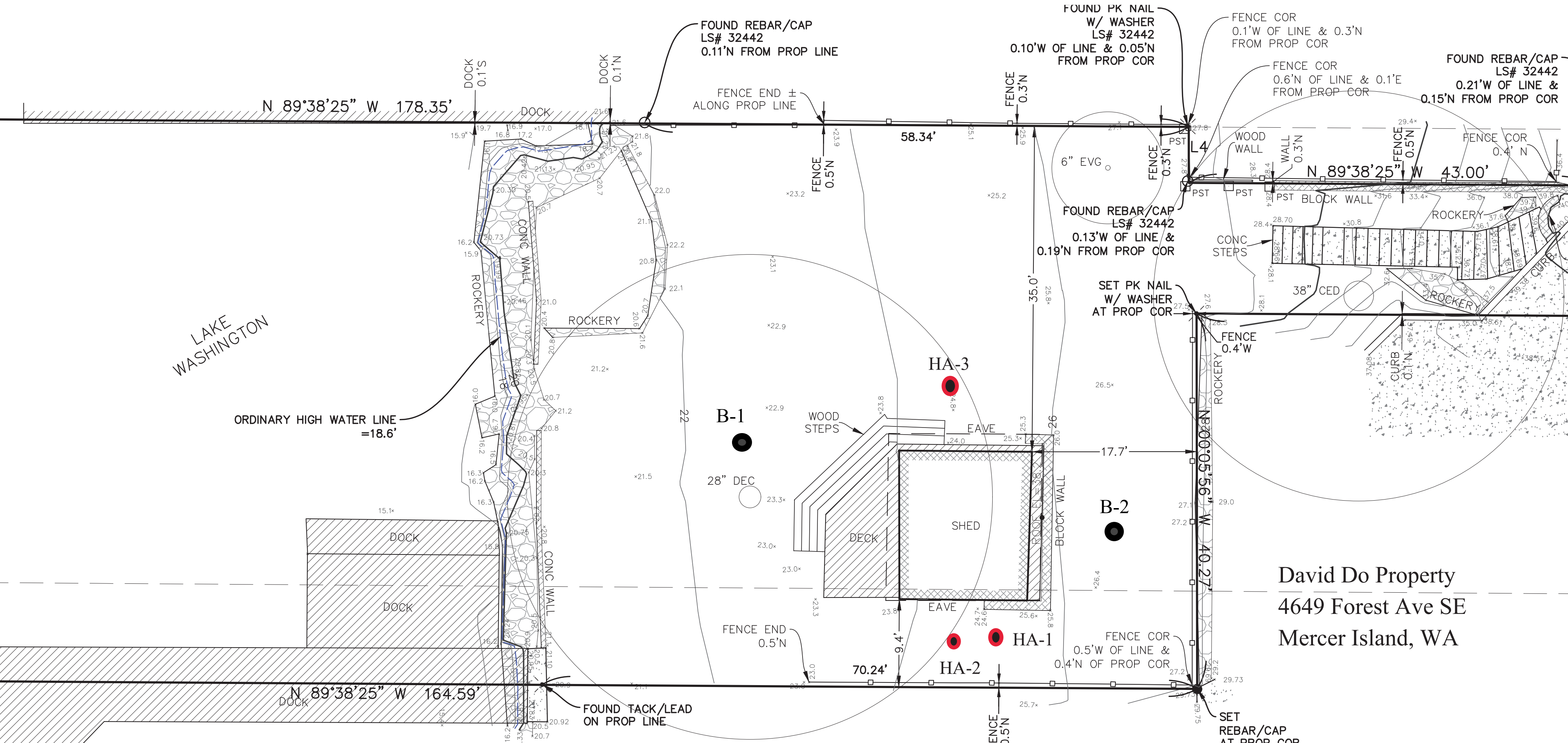
Attachments: Site & Exploration Plan
Exploration Borings EB-1 & EB-2

SITE & EXPLORATION PLAN



David Do Property
4649 Forest Ave SE
Mercer Island, WA

Reference: Terrane - Topographic & Boundary Survey, February 2023

Legend: Approximate Location of Hand Boring

Approximate Location of Drilled Boring

EXPLORATION BORING LOG

Number **EB-1** PAGE 1 OF 1

SEDIMENT DESCRIPTION	DEPTH	SAMPLE	GROUND WATER	STANDARD PENETRATION RESISTANCE Blows/Foot
Medium stiff, moist, gray silt with some sand, disturbed matrix (fill?).		3 4 4		8
Medium stiff, moist, gray silt with some sand, slightly plastic.	5	3 3 3	▼	6
Stiff, saturated, gray sandy silt with occasional pebbles.		3 4 6	A.T.D	10
Very stiff, saturated, brown, sandy silt.	10	6 9 10		19
Upper 6″ - Very stiff, saturated, brown sandy silt. Lower 12″ Medium dense, saturated, gray, medium sand with gravel, well sorted.	15	7 12 15		27
Drilled to 20 ft. 2 1/2 feet of heave. Cannot sample. Bottom of boring at 20 feet.	20			

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by geologic Interpretations, engineering analysis, and judgment. They are not necessarily representative of other times and locations. We will not accept responsibility for the use or interpretation by others of information presented on this log.

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AUGUST 2025
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EXPLORATION BORING LOG

Number **EB-2** PAGE 1 OF 1

SEDIMENT DESCRIPTION	DEPTH	SAMPLE	GROUND WATER	STANDARD PENETRATION RESISTANCE Blows/Foot
Stiff to very stiff, moist, gray silt with some sand, slight blocky texture.		5 7 8		15
Medium stiff, moist, tannish gray silt, with some sand.	5	4 6 7		13
Very stiff, moist, gray silt, with some sand.		7 8 8		16
Very stiff, very moist, gray, sandy silt, small 1/2-inch sand interbeds.	10	9 10 10		20
Very stiff, very moist, gray, silty clay to clayey silt.	15	9 12 14		26
Hard, very moist, gray, silty clay to clayey silt.	20	18 22 24		46
Auger unable to turn further. Bottom of boring at 21-1/2 feet. Free water was not observed in the boring at the time of drilling. Very small groundwater seep observed at 8 feet.				

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