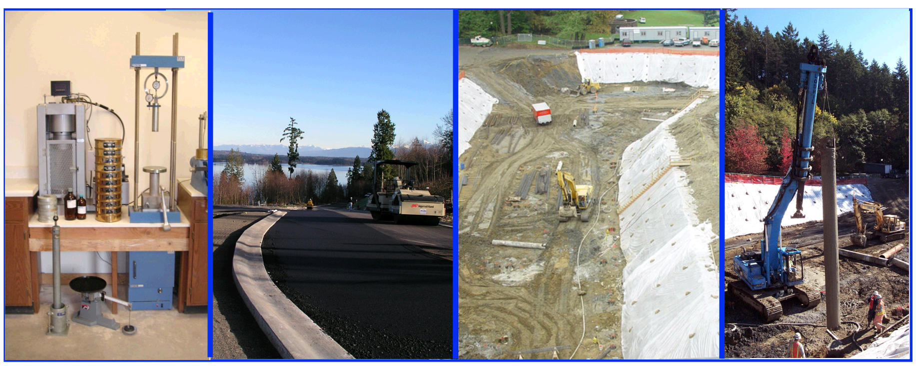
MERIT ENGINEERING INC.

GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, Washington
Parcel#: 302405-9004



Prepared For:
Buping Wang
5425 96th Ave SE
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October 5, 2023
Project No. 2EH03221024

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October 5, 2023
Project No. 2EH033221024

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Buping Wang & Wanqiu Yang
c/o Siyao Studio
siyaow@siyao.studio

Re: Geotechnical Engineering Investigation
6450 E Mercer Way
Mercer Island, WA 98040
Parcel#:302405-9004

Dear Buping:

At your request, we have conducted a geotechnical engineering investigation at the above referenced project site. The following geotechnical engineering report represents the results of our visual site reconnaissance, DCP test observations, engineering analysis, and derived conclusions on the slope stability of proposed residential building.

Thank you for this opportunity to work with you on this project. Please contact us if you have any questions about this report.

Sincerely,

Austin X. Huang, Ph.D., P.E., L.G., D.GE., F.ASCE
Principal

F.ASCE: Fellow - American Society of Civil Engineering
D.GE - Diplomate - Academy of GeoProfessionals

D.GEs provide successful projects that benefit their clients.
The D.GE certification recognizes geotechnical engineers who possess specialty education, extensive experience, integrity, and good judgment.

GEOTECHNICAL ENGINEERING REPORT

Proposed Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, Washington

Prepared for:

Buping Wang
6450 E Mercer Way
Mercer Island, Washington

by

October 5, 2023

Austin X. Huang, Ph.D., P.E., L.G., G.DE, F.ASCE
Principal

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1. INTRODUCTION

At request of Buping Wang, Merit Engineering, Inc. has conducted a geotechnical engineering investigation for the proposed development of the site, located at 6450 E Mercer Way, in Mercer Island, Washington 98040 (Parcel #302405-9004). The project area and vicinity is shown in Figure 1 and the site plan with proposed new Accessory Dwelling Unit (ADU) and test locations in Figure 2 in the Appendix.

We understand, from the information Architect Siyao provided to us, that the site is currently occupied by a single family house and a detached garage with a slope in between. The proposed project is to replace the existing detached garage with a new ADU at approximately same location. The ADU will be a 2-story structure with approximately 900 sf of conditioned space and a 2-car garage. We understand that the proposed new ADU will be at the top of the slope. The ADU will be built with minimum intrusion into the slope. Based on the topographic map provided by TERRANE, the slope at east of existing detached garage is approximately 1.57H:1V (~63.6%) with average horizontal distance 22ft with elevation loss of ~14ft, greater than 40%. The site also has Geologically Hazardous Area concerns according to the City of Mercer Island Land Development Code (LDC).

Therefore, the objective of this study specifically was to investigate surface, subsurface, and slope conditions at the site, conduct a hazard analysis, derive conclusions, and provide recommendations for site preparation, design, construction and geologic hazard mitigation of the proposed ADU. The report will, in particular, address critical area concerns of geologically hazardous area, to comply with City of Mercer Island Land Development Code 19.07.060 - Critical area maps and inventories, 19.07.090 - Critical area reviews, 19.07.100 -

Mitigation sequencing, 19.07.110 - Critical area study, and 19.07.160 - Geologically hazardous.

2. PROJECT DESCRIPTION

The project site is a rectangular-shaped parcel of land with an area of approximately 19,270sf surrounded by single-family. The site is at east of E Mercer Way with a private shared driveway to the existing detached garage and parking space. Topography at property has generally three portions. The upper portion has a slope approximately 7.77H:1V (~12.8%) with local elevation loss ~9ft, roughly 7.3° following up with a steeper slope at middle portion approximately 1.57H:1V (~63.6%) with local elevation loss ~14ft, roughly 32°. Then gentle slope at lower portion approximately 5.85H:1V (~17%) with local elevation loss ~20ft, roughly 9.7° to property boundary at east and ultimately reaches to the Lake of Washington.

The proposed new ADU will be at the same location of upper potion of site with approximately same setback from top of steeper slope. The ADU will be built with minimum intrusion into the steeper slope.

3. SCOPE

Based on all the above information and understanding of the project and difficult access, we conducted a site exploration using Dynamic Cone Penetration Test (DCP[1]) with scope of work in compliance with our proposal No. P2EH0533827 dated August 21, 2023, in particular includes:

[1] DCP test consists of driving a 10 cm^2 (1.4″ diameter) cone into the ground. The cone is attached to steel rods and driven by a 35 pound hammer with 15″ free fall. Number of blows for each 10 cm (4″) penetration was recorded

- Conducting a site reconnaissance of the property and adjacent area;
- Conducting three (3) DCP to maximum depth of 10 feet, where penetration refusal was encountered.
- Logging soil and ground water conditions;
- Performing a slope stability analysis;
- Performing engineering analysis;
- Preparing a geotechnical engineering report with geotechnical engineering recommendations:
  - (1) surface conditions;
  - (2) subsurface soil conditions;
  - (3) groundwater conditions, and
  - (4) computer model analysis of slope stability

Recommendations for:

- (5) foundation design parameters,
- (6) structural fill and compaction criteria,
- (7) foundation retaining wall design parameters,
- (8) slab-on-grade floor,
- (9) drainage,
- (10) site grading, and

4. SITE INVESTIGATION

4.1 Surface Conditions

This subject site is located at hillside at east of E Mercer Way overlooking east to Lake Washington. Site reconnaissance was performed by a representative of Merit Engineering, Inc. on September 7, 2023 during our field DCP tests. A survey completed by TERRANE was used in this study to construct site plans showing property lines, major features, and test locations (Figures 2). The City of Mercer Island, King county online GIS map program was used to reference general topographic features at the site. Detailed slope measurements taken by Merit Engineering, Inc. were then used to note the topographic contours and location of slope to better reflect actual site conditions and create a topographic profile (Figure 9) for slope interpretation, discussed later in this report. DCP tests were performed by a representative on September 7, 2022. We also observed surface topography, surface soils, vegetation, and surface water conditions at the subject slope.

The site is situated approximately at bottom of east flank of hill at east of E Mercer Way. The top of the hill is at an elevation ~340´ with an east slope down to the lake. The rectangular-shaped parcel slopes from west to east with 19,270 square feet area. The north boundary of site is at an elevation of ~71ft with a southeast slope of 12°(~21.4%) over an average horizontal distance 70ft with elevation loss ~9ft down to the top of steeper slope at an elevation ~62ft. The existing detached garage is located at top of steeper slope with approximately 5ft setback in this site upper portion area.

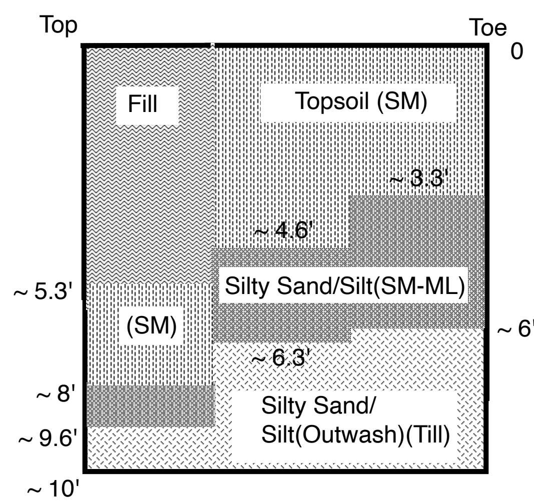
Further to east at an elevation of approximately 62ft is the top of the steeper slope. This slope angle is 32° (~63.6%) over an average horizontal distance 22ft with elevation loss of ~14ft. This site middle portion area was covered by heavy ground cover.

The steeper slope continues with a gentle slope approximately 5.85H:1V (~17%) over an average horizontal distance 117ft with elevation loss of ~20ft. The existing single family house was sit at middle of this site lower portion area with front patio area and grass land at back.

Vegetation on the middle portion slope is characterized by ground cover including shrubs and ferns. A large diameter deciduous tree is present at middle portion slope. The tree are vertically straight and do not show signs of slope movement. The landscape including patio and small trails and rockery steps are are at south of site and toe of slope. Most of the area between the house and the lake is lawn. We have not observed incline and movement of trees in the proposed parcel.

4.2 Subsurface Conditions

The site access is difficult for large machine, therefore a portable dynamic cone penetration equipment was used for testing. Surface soil and ground water conditions were investigated by conducting three (3) DCP tests along the slope at toe, middle and top to maximum 10´ depth on September 7, 2023. Tests were conducted near the adjacent to the existing detached garage foundation to generalize subsurface soil conditions. Test locations are shown on the site plan (Figure 2). DCP tests are presented in the Appendix of this report as Figures 4 through 6. Descriptions of soil symbols and classifications used in this report are also presented in the Appendix (Figure 3). Site soils are generalized in the schematic and summarized as follows:



a. Topsoil/Fill

b. Silty Sand with gravel/Silt (SM-ML)

c. Silty Fine Sand with gravel (SM-ML)

a. Topsoil/Fill

A layer of loose silty sand which varies in thickness from 3.3´ - 8´ generally blankets the slope. The soil was dry, crumbly when disturbed, and loose in situ. However, the fill soils were encountered in DCP#3 at top of slope that is ~5 feet way from foundation of detached garage to a depth of approximately 5.3 feet below the ground surface. We interpret the existing fill as being placed during the construction of the existing detached garage.

b. Silty Sand with Gravel/Silt (SM-ML)

A layer of silty sand underlying the topsoil extends to the depth of ~9.6´. The soil is medium dense and stiff. The more closer to the top of slope, the thinner of this layer is.

c. Silty Fine Sand with gravel/Silt (Outwash)(Till)

Underlying the medium dense silty sand beginning at ~9.6´ depth to the total penetration depth of ~10´ was a layer of silty sand with gravel. The soil is dense to very dense and very stiff to hard.

4.3 Geologic Background

The Lake Washington basin and Puget Sound areas were overidden by Vashon glacier around 17,600 years Cal B.P. At its maximum extent, the ice was over 3,000 feet thick in the seattle area and extended from the foothills of the Olympics to the foothills of the Cascade Mountains and from Brithsh Columbia, Canada, to south of Olympia, Washington (Booth and others, 2004). During glacial advance, sub-glacial melt water carved the Lake Washington basin and all the troughs of Puget Sound, including the Duwamish, Green, Puyallup, and Sammamish river valleys. Sub-glacially, these troughs were connected to Puget Sound, and when the ice receded, the troughs were still connected via water.

We evaluated near surface soil conditions at the site by reviewing the "Soil Survey, King County Area, Washington" by the United States Department of Agriculture (USDA) and the Geologic Map of Mercer Island by Kathy Troost and Aaron Whisher of Geologic Map of Mercer Island, Washington dated October 2006. The USDA soils map identifies the prominent soil type in the vicinity of the property as Kitsap silt loam (KpD) on slopes ranging from about 15 to 30 percent. The Soil Conservation Service (SCS now NRCS) describes the Kitsap silt loam as consisting of pre-Vashon, fine sand, silt and clay-sized particles. The SCS further indicates soils of this type are generally expected to have high strength, low permeability, and present a moderate to severe erosion hazard when exposed on the face of a steep slope.

The geologic map reviewed indicated that the soils in the project vicinity are predominantly Pre-Olympia non-glacial deposits overlying Pre-Olympia coarse-grained glacial deposits. These soils have been deposited and overridden by ice and are generally very dense/hard. The native soils encountered in the DCP tests are generally consist with the published informations.

4.4 Surface and Ground Water Conditions

No surface water was observed and no groundwater seepage was encountered during our site exploration on September 7, 2023. However, runoff or seeps of stormwater, and possibly pockets of temporary shallow groundwater, may occur locally during periods of heavy rainfall.

5. GEOLOGIC HAZARDS

5.1 Geologic Hazard Designation and Typing

Geologic hazard areas are designated by the City of Mercer Land Development Code (LDC19.07.160A) that geologically hazardous areas are lands that are susceptible to erosion, landslides, seismic events, or other factors as identified by Washington Administrative Code (WAC 365-190-120). We researched available publications for geologic hazard areas on or in the vicinity of the site.

5.2 Landslide Hazards (LDC 19.16.010)

The LDC 19.16.010 defines Landslide Hazard Areas based on a combination of geologic, topographic, and hydrologic factors. They include any areas susceptible to landslide because of any combination of bedrock, soil, slope (gradient), slope aspect, structure, hydrology, or other factors, and include, at a minimum, the following:

1. Areas of historic failures;
2. Areas with all three of the following characteristics:

   a. Slopes steeper than 15 percent; and

   b. Hillsides intersecting geologic contacts with a relatively permeable sediment

   c. overlying a relatively impermeable sediment or bedrock; and

   Springs or ground water seepage;
3. Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements;
4. Areas potentially unstable because of rapid stream incision and stream bank erosion; or
5. Steep slope. Any slope of 40 percent or greater calculated by measuring the vertical rise over any 30-foot horizontal run.

The subject site was depicted as potential landslide hazards area on City of Mercer Island GIS Map. The site dose contain a small area at direct east of existing detached garage which are 40 percent or steeper slope. However, the majority of the site is comprised of slopes inclined at gradients less than 40 percent. We observed no surficial evidence of recent or historic landslide activity or slope instability during our geologic reconnaissance of the site. As discussed above, the site contains a number of large conifer trees, none of which show any signs of past or on-going slope movement. The site is not within certain mapped intersecting geologic contacts which are prone to producing springs or groundwater seepage. Therefore, in our opinion, the risk of a potential slide in this area is considered to be low provided that the building pad are sited to minimize impacts to the extent reasonably feasible.

We compared the each criterion of the Critical Area Ordinance with site existing conditions. None of these conditions are present at the site. Base on our investigation and engineering analysis, the site is not a Landslide Hazard Area as defined by the LDC in our opinion.

5.3 Erosion Hazard Areas (LDC 19.16.010)

The LDC 19.16.010 defines those areas greater than 15 percent slope and subject to a severe risk of erosion due to wind, rain, water, slope and other natural agents including those soil types and/or areas identified by the U.S. Department of Agriculture's Natural Resources Conservation Service as having a "severe" or "very severe" rill and inter-rill erosion hazard.

The site is within a large areas mapped by the City of Mercer Island as a potential erosion hazard area. Site specific evaluations must be made in each application to determine actual site conditions. And discoverable information of actual site conditions from site-specific review is both the most accurate and reliable information to determine whether a particular site or portion of a site falls under the City's definition of an Erosion Hazard Area. The City Code should be logically read to only resort to SCS mapping as determinative if there is some ambiguity with respect to site -specific circumstances. Such is not the case for this project as we have had the opportunity to perform a complete site evaluation. It should be noted that the proposed new ADU is at approximately same location of existing detached garage.

Based on 1) the soils encountered in explorations completed at the site, 2) the condition of the existing vegetation at the site, and 3) the absence of existing erosion features at the site, in our opinion the erosion potential of the site is low. Therefore, in our opinion the site does not fall within the City's definition of an Erosion Hazard Area. However, the erosion may exist during construction from disturbed soils. Therefore we have included recommendations to mitigate erosion hazards at the site later in this report.

5.4 Seismic Hazard Areas (LDC 19.16.010)

The LDC 19.16.010 defines seismic hazard areas are areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, soil liquefaction or surface faulting.

The site and vicinity is mapped as having a low to moderate liquefaction susceptibility to seismic shaking according to the King County Liquefaction hazard map (May, 2010). And based on our field tests, the site soils consist primarily of very dense till soil at shallow depth. See the seismic hazard discussion in section 8.8 below for more details.

6. CRITICAL AREAS EVALUATION

6.1 Slope Considerations

During this study, we considered local slope conditions and focused on the specific slope area east of the proposed residence in order to evaluate the critical areas identified above and assess project feasibility. Cross-section profile (Figure 9) was constructed through locations of interest based on survey data and subsurface testing.

From site DCP tests, reconnaissance, and our knowledge of local geology, local slope stability on the site appears to be controlled by the relationship of the silty sand cover soil and hard refusal soil underneath.

Slope stability against erosion and local slumping is influenced by the interaction of the natural topography, shallow soil character, groundwater, and slope of the hardpan till or glacial deposits that may act as a failure plane. Cover soil stability must also be considered in relation to slope grading and design for the proposed development.

We understand that the proposed development is to replace a existing detached garage with new ADU at approximately same location. At the time of preparing this report, detailed site grading and building plans were not available. According to the Topographic Survey Plan prepared by TERRANE, the new ADU is proposed on the top of slope. No other development are proposed on the slope.

6.2 Hazard Analysis: Interpretations of Slope Stability

Our site reconnaissance did not find obvious visual evidence of current or past slope stability issues within the project area. We are not aware of any historic slope failures in the site vicinity. Due to the variable nature of the geology and topography in the area, the potential for such failures can vary widely between adjacent sites, and thus should be evaluated on a site-specific basis.

Surficial soils on the site slopes may exhibit some minor surface creep or erosional instability if disturbed during construction, and may be more at risk for ongoing erosion without stabilization measures such as replanting. In general, the site does not seem to be problematic in terms of large-scale mass wasting because of the moderate dense to dense soil conditions. However, care must be taken in site design and construction to avoid loading shallow soils on slopes with fill application, walls, or other landscape features that could instigate a shallow failure of the cover soil. Recommendations on practices for slope grading and construction with erosion and drainage control are provided later in this report.

We conducted an analysis on slope stability of the slope with proposed ADU, and soil parameters estimated based on the soil type from filed tests, published soil/geological information, and publication of slope stability charts by Hoek and Bray "Rock Slope

Engineering", in which they publicized computer model generated circular failure mode slope stability charts for uniform soil conditions and ground water surface at 4 times slope height behind the toe of the slope. The results show in Figure 7 and 8. Using these data, our analysis indicates that the slope projected from top to the toe of slope, which is 1.57H:1V (~63.6%), has a factor of safety over 1.5, which is the factor of safety we need for a permanent slope.

7. SLOPE STABILITY EVALUATION

We have performed a slope stability assessment using computer model analysis for factor of safety incorporating multiple methods for parameter evaluations including reconnaissance, subsurface testing, and regional interpretation. Visual reconnaissance with observations and surface mapping of slope conditions was completed during our filed test site visit.

Slope stability conditions were evaluated by using a STABLE computer model as well as by using calculations derived from literature. Soil parameters were based on field tests, soil classification, and literature values. Our model used site-specific soil parameters to determine slope conditions that have an acceptable FS (factor of safety) for static and seismic conditions.

7.1 Engineering Properties of Soils

We used values for soil unit weights (γ) based on soil classification and literature values from Hoek and Bray (1981). The values for friction angles (ϕ) of the slope soil was estimated using the correlation between ϕ and N (DCP test from geotechnical penetrating tests). The value for cohesion (c) of the soil was estimated using the correlation between c and N (DCP Test) and verified by similar values provided by Hoek and Bray (1981). In addition, the value for cohesion (c) of silty sand/silt was back-calculated using the STABLE computer model under the existing slope condition by assuming factor of safety = 1.5 for conservative.

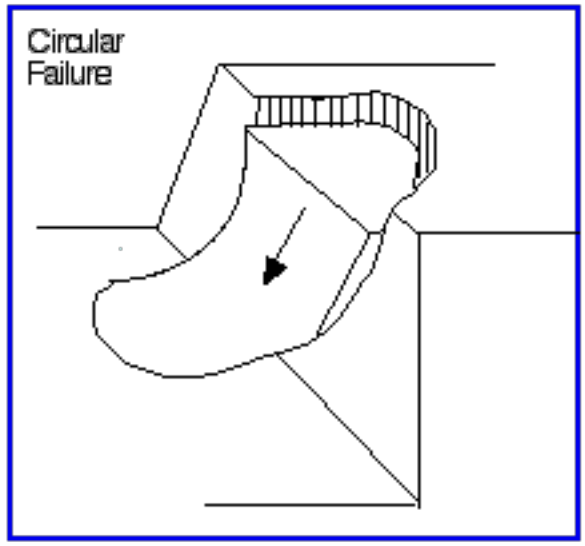
Hoek and Bray (1981) specify the friction angle of loose sand, mix grained size to be 34° to 40°. For pure sand cohesion to be 0 psf. Friction angle for glacial till (mixed grain size) to be 32° to 35° and the cohesion to be 3,000 psf to 5,000 psf. Our analysis is considered somewhat conservative due to selecting the lower bound parameters of estimated literature soil strength properties. Our analysis, therefore, should produce more practical results because we have verified parameters through multiple sources.

TABLE 1: Basic Engineering Properties for the Materials on Site

Soil	Depth (ft)			Cohesion (psf)	Friction Angleϕ	Dry Unit Wt. ϒ dry, (pcf)	Sat. a Wt. ϒ wet, (pcf)
	DCP-1	DCP-2	DCP-3				
Topsoil/Fill (SM)	0'-3.3'	0'-4.6'	0'-8'	0	32°	99	124
Silty Sand W/Silt (SM-ML)	3.3'-6'	4.6'-6.3'	8'-9.6'	250	32°	116	135
Outwash Till	6'-6.6'	6.3'-7.3'	9.6'-10'	3000	34°	130	145

7.2 Slope Failure Mode

The possible mode of slope failure at this site has to be considered for the slope stability analysis. Modes of slope failure are best categorized according to controlling geological structures. For instance, such structures can be bedrock, faults, or high strength soil layers interbedded with low strength layers. As discussed above, for this analysis we are considering the failure of the sand unit itself. Therefore, there is no significant controlling structure and the material is assumed as homogeneous soil with the internal parameters listed in Table 1. Soil slope failures are characterized to fail in a circular mode according to Hoek and Bray (1981). This will be the assumed failure mode for this site, depicted here.



7.3 STABLE Computer Model

Theoretically, a slope is acceptable if the Factor of Safety (FS) is greater than 1.0. However, in practical scenarios an FS of greater than 1.0 is always adopted to account for uncertainties that may arise from either parameter estimations or approximations of conditions imposed by the analytical model which may not be perfectly applicable to the project site. A minimum FS of 1.5 is suggested for critical slopes adjacent to haul roads or important installations that are required to remain stable for long periods of time under static conditions (Hoek and Bray, 1981) and is widely adopted in civil engineering practice.

In our analysis of the subject slope, we employed the Bishop model of slices which yields more accurate results at this site over the Janbu method. Based on the parameters described in the previous sections, the STABLE program outputs a cross-section showing the ten most critical theoretical failure planes of the cross-section and the corresponding FS values for each surface. We approached the analysis in a somewhat conservative fashion, meaning the engineering properties of the soils were more indicative of the weaker soil conditions observed on the site.

To calibrate the STABLE model, one cross-sections were created using topographic information (using survey map provided to us by Terrane), shown in Appendix. We modeled groundwater begin to flow at the surface, which is 4 times H (height of slope) horizontal distance away from toe of slope. The groundwater path was modeled conservatively by this assumption due to the lack of water table encountered when performing DCP tests. Our analysis considered both static and the pseudostatic (seismic) conditions. We considered seismic loading conditions of the subject slope by employing a horizontal earthquake coefficient of 0.31g. This acceleration is equal to one-half of the peak horizontally ground acceleration from ASCE 7 Hazard Tool with ASCE/SEI 7-16 reference document with a two-percent in 50-year probability of exceedance as defined by the 2018 International Building Code. Typically, minimum FS values employed for seismic conditions are ~75% of target static value (FS ~ 1.125).

7.4 Discussion of Results

Slope Stability With Proposed ADU

According to the preliminary building plan, the proposed building will occupy at approximately same location as existing detached garage at elevation ~63´. Based on the topographic map, the height of the slope is approximately 14´. Using these data, our analysis indicate that the slope projected from top to toe, which has slope 1.57H:1V (~63.6%), has a factor of safety over 1.5, which is the factor of safety we need for a permanent slope.

Based on our derived soil properties and site parameters, shown on the Table 1 for the entire slope from top to the toe, the ten (10) most critical surfaces were created, shown in Figure 7. The factor of safety for the critical surfaces range from 2.14 to 5.94, which is over 1.5 for representative cross-section A-A'. This gives us that at the top of slope with proposed ADU may be satisfactory.

Analysis showed that cross-sections A-A', Figure 8, do meet the minimum FS of 1.125 for seismic conditions for a horizontal earthquake coefficient of 0.31g.

8. CONCLUSIONS AND RECOMMENDATIONS

We have conducted a study of existing site and slope conditions and slope stability assessment for the proposed ADU at 6450 E Mercer Way, in Mercer Island, Washington 98040 (Parcel #302405-9004) to fulfill City of Mercer Island critical area requirements. The results of our site testing and reconnaissance are presented herein with accompanying discussion on slope stability concerns and interpretations.

We conclude, and it is our opinion based on the results and stated limitations of this study, that the site is suitable for the proposed ADU if recommendations in this report are followed. The proposed alterations to the Critical Area should not decrease slope stability on the subject or adjacent properties, adversely impact the surrounding Critical Areas, or increase surface water discharge, sedimentation, or erosion rates if the recommendations in this report are followed.

The following sections present a summary of this study and our recommendations for construction of ADU to erosion control, stormwater control, and drainage control to enhance and protect slope stability as possible within the subject property.

We understand the project location is designated as a geologically critical area. Therefore, it is our understanding that the analyses and recommendations provided in this report are based on the assumption that our firm's level of professional services will be retained for future project design and construction phase services as needed. We recommend that we be contacted to review final plans for development after the recommendations of this report are considered, to ensure they are consistent with the results and intent of recommendations herein.

8.1 Site Preparation and Grading

We recommend removing any organic topsoils and unsuitable loose and soft soils from the areas under the proposed residential structure. We anticipate that soil excavation can be accomplished with conventional equipment, although excavation into the hardpan till may be more difficult.

Any soft subgrade soils encountered during site excavation should be removed and replaced with structural fill as recommended in the Structural Fill section of this report. Based on our observations, the native silty sand subsoil may be suitable for subgrade depending on fine content, organic content, and moisture levels. If the subsoil is present at proposed subgrade level, it should be evaluated in the field, and its suitability may depend on the specific structure be installed at a given location.

Dry season construction at this site is recommended. Due to the sloping nature of the site, we recommend that care be taken to the maximum extent possible for erosion and ground control. It should be understood that significant additional costs and construction difficulty could be incurred if work proceeds in wet weather comparing with dry weather construction. Therefore, we don’t recommend wet season construction.

We recommend that we observe and verify site excavation to suitable subgrade soils, test to verify import fill materials, and observe and test compaction of structural fill materials.

8.2 Construction Recommendations

8.2.1 Temporary Excavations

Temporary soil cutslopes during earthwork without control measures should generally be no steeper than 2:1 (Horizontal:Vertical) for sandy soil, and should be evaluated for general stability after shaping and periodically in construction. For temporary cutslopes that will remain exposed for any length of time, we recommend that we review conditions on site. We also recommend that we review and consult on the proposed methods of excavation prior to earthwork, and that we verify excavation to suitable cutbank conditions and geometry during construction. As local variations may exist between locations within the site, we recommend that we evaluate all cutbanks to ensure they are consistent with the conclusions in this report, and if necessary provide alternate recommendations per location.

Maximum temporary excavation depths are expected to extend to depth of 6 to 8 feet below existing grade. Temporary excavations greater than 4 feet deep should be properly sloped or shored. All temporary excavations should be performed in accordance with Part N of WAC 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. The temporary cut slopes should be re-evaluated by a representative of Merit Engineering during construction based on actual observed soil conditions.

8.2.2 Dewatering

No surface water was observed and no groundwater seepage was encountered during our site exploration on September 7, 2023. However, it may be anticipated the footing excavation will encounter groundwater due to fluctuation of seasonal water table and unforeseen circumstance of heavy rainfall in wet season. The contractor should be prepared to provide a temporary dewatering system to control and remove seepage into the excavations. Due to the relatively fine grained nature of the soils underlying the site, we anticipate groundwater seepage in the footing excavation can be controlled by sloping the base of the excavation to drain and removing the water with sumps and pumps.

8.3 Structural Fill

Structural fill should be placed on firm, horizontal subgrade in about 10-inch thick loose lifts and compacted to at least 95% of the ASTM D-1557 maximum dry density for footings, grade slab, parking and road, and sidewalks.

We recommend import structural fill be sandy gravel or gravelly sand meeting specification - 9-03.12 (1) B, APWA/DOT 2006, that is typical in this area as base granular materials with exception that percent passing U.S. No. 200 Sieve shall not exceed 5% and all materials smaller than 4″. The specification is summarized below:

Table 2: Specification of Imported Fill Materials

Sieve Size	Percent Passing by Weight
4″ Square	100
2″ Square	75-100
U.S. No. 4	22-66
U.S. No. 200	5.0 max.
Dust Ratio $\frac{\%\ Passing\ U.S.\ No.\ 200}{\%\ Passing\ U.S.\ No.\ 40}$	⅔ max.
Sand Equivalent	30 min.

Backfill immediately behind retaining walls or adjacent to foundation stem walls should be compacted to about 90% of the ASTM D-1557 maximum dry density. Care must be taken to avoid over-compaction immediately behind walls. Backfill behind retaining walls must be free draining material.

It is important that plumbing and utility trenches be properly backfilled. Backfill in the trenches should meet the appropriate compaction criteria described above.

8.4 Foundation Design Parameters

We recommend placing foundation on native dense glacial till soils or on import structural fill installed on the native glacial till soils. Sand sub grade soils should be compacted to 95% modified proctor. If site soils are not found to be firm at a footing location and grade, we recommend excavating down to appropriately firm soils and replacing the soft/loose soil section with structural fill.

We recommend that all perimeter footings be at least 18 inches below final outside grade for frost protection. The base width of footings shall be at least 18 and 24 inches for continuous and isolated column spread footings, respectively.

Under condition of satisfying the above recommended footing dimensions, a soil bearing pressure of 2,500 psf (*pounds per square foot*) is recommended. Bearing pressure may be increased by 1/3 for transient wind or seismic loads. This bearing recommendation is preliminary pending building design details. We recommend that we be contacted in the design phase to evaluate building details with our soils and slope condition and revise bearing allowances accordingly.

With the above recommended soil bearing capacity, the anticipated load on the footings, and the soil conditions from the tests, we estimate that the total potential settlement of the foundations should be less than 1 inch. While most settlement will occur in the short term as loads are applied, some settlement may occur over a long period of time after construction.

We recommend proof-rolling building pads before placement of footings with a loaded dump truck to reveal soft or yielding surficial soils. Any soft subgrade soils encountered during site excavation or exposed during proof-rolling should be re-compacted.

We recommend that we review portions of plans and specifications pertaining to earthwork and foundations to ensure they are consistent with recommendations in this report.

We also recommend that we observe and verify site excavation to suitable soil stratum, a proof roll test to verify imported fill materials, and observe and test compaction of structural fill materials.

8.5 Foundation and Site Drainage

A perimeter footing drainage system should consist of at least 6-inch diameter, perforated, rigid pipe. Pipes should be placed along the exterior base of the foundation perimeter and tightlined to a storm drain system or natural drain course. Pipe should be bedded on 2 inches, and backfilled with a minimum of 12-inches, of pea gravel.

Under-slab cross-drains may be helpful to maintain a dry slab floor to facilitate drainage. A cross-drain system should be overlain by drain rock beneath the slab.

Roof downspouts should be tightlined to a storm drain system separately from footing drains. In addition, the site should be graded so that surface water runoff is directed to catch basins attached to a storm sewer drain.

In addition, the site general perimeter drain shall be installed for general slope stability protection. And we recommend that we be retained to consult and review on the drainage installation work.

8.6 Slab-On-Grade Floor

A slab-on-grade floor may be supported on building pads that are prepared with firm native subgrade soils, or import structure fill compacted over firm native soils. At least 4-inches of drain rock of ¾″ maximum size should be placed between the slab and slab subgrade.

A vapor barrier visquine should be placed between the slab and capillary break material. An additional 1 to 2 inches of sand may be placed on top of the vapor barrier if desired to aid in concrete curing. In addition, use of a commercial concrete slab sealant for moisture protection may prove to be very helpful.

Floor slabs reinforced with 6 x 6 wire mesh may help reduce potential crack separation and vertical offsets at cracks. Reinforcement should be set at or above the mid-depth of the slabs. To reduce cracking potential we suggest exterior patios and other flatworks contain reinforcement as recommended above for floor slabs. Any flatwork subgrades should be watered thoroughly prior to concrete placement to close soil shrinkage cracks. Flatworks should have frequent joint controls.

Additional measures to reduce potential cracking are considered warranted at critical areas where slab movement could impair use; such critical areas include any exterior patio slabs that meet the interior floor level at doorways. For such areas we recommend that recommend that the upper 12-inches of native soil be over excavated and replace with import structural materials as specified in the Structural Fill section of this report.

8.7 Lateral Earth Pressure

We recommend that we be contacted for consultation and evaluation engineered retaining walls or walls with a surcharge loads are considered in site design. Addition of a retaining wall and backfill near the top or on a slope may cause undue loading and therefore surficial soil-slope instability, and it is preferable to limit or avoid such load application if possible. Retaining walls against the base of an existing slope or cut are acceptable given the wall will not change the current topography significantly.

We recommend placing structural fill behind subsurface and retaining wall. The horizontal thickness of the fill should be at least the height of the wall. For structural fill, as recommended in the Structural Fill section of this report with a level ground, the parameters of lateral earth pressures are listed in Table 3

Soil	Active K_a	Passive, K_p	At Rest, K_o
Structural Fills	0.28	3.54	0.44
Equivalent Fluid Pressures*			
Structural Fills	34	425	53

*Equivalent fluid pressure is the product of lateral earth pressure coefficient and the unit weight of the soil.

The soil parameters of lateral earth pressure for the on-site soils may be much stronger than those in the above table, however, it must be evaluated to confirm on site during construction when site excavation opens up the ground for visual observation.

Design of subsurface walls should include appropriate lateral load due to adjacent surcharge. Under uniform surcharge qo, lateral load due to a uniformly distributed lateral pressure σ, should be added to active and at rest soil lateral pressure, respectively as defined in the following equations:

$$\sigma = \begin{cases} K_a q_o & \text{for active case} \\ K_o q_o & \text{for at rest case} \end{cases}$$

A coefficient of base friction of 0.55 and 0.45 may be used between concrete and structural fill and between concrete and native fine sandy soil, respectively. However, if passive pressures are used in conjunction with frictional resistance to determine lateral resistance to sliding, only 1/2 the value of passive pressure presented above should be used since larger strains are required to mobilize passive soil resistance as compared to frictional resistance.

8.8 Seismic Design Parameters

The site is located in the seismically active Puget Low lands. Deep focus earthquakes from subduction of the Juan de Fuca plate beneath the North American plate can cause amplified shaking at the ground surface due to seismic waves of different velocities interacting. Seismic waves propagate relatively slow through soft soils and considerably faster in rock. As a result, areas with softer soils underlain by rock tend to experience greater ground shaking than areas with little variation in the underlying substratum. Local building codes and design practices now consider the possible effects of soil conditions and large subduction related earthquake in the design of structures.

8.8.1 Liquefaction

Liquefaction is a phenomenon associated primarily with near surface saturated cohesionless soils under zero effective stress. Effective stress equals the confining pressure of the soil minus pore water pressure. When saturated cohesionless soils undergo cyclic seismic loading, the induced excessive pore pressure cannot dissipate and thus grows larger. When the pore pressure becomes equal to the confining pressure from the overburden load, the effective stress of the soil becomes zero and the soil lost its strength or stiffness and becomes liquefied. Foundation settlement and lateral movement could damage structures supported by liquefiable soils and sites with conditions favorable for liquefaction are designated as Site Class F. Site classes are a simplified method for describing the amplification of ground shaking during a seismic event due to effects of underlying soil conditions and are defined by a unique range of average shear wave velocities in the upper 100´ of the site soil column.

The site and vicinity is mapped as having a low to moderate liquefaction susceptibility to seismic shaking according to the King County Liquefaction hazard map (May, 2010). According to our site specific evaluations, the site soils consist primarily of very dense till soil at shallow depth. Based on these soil, it is our opinion that liquefaction potential at the site is low because soils at the site are generally very dense till.

8.8.2 Design Parameters

Using the results of our DCP (Dynamic Cone Penetration) test holes and geologic setting as discussed in this report, we estimate the average N value, using methods provided in Section 1613 of the 2018 IBC. The results of our average N value estimated and projected for a 100´ section indicate a very dense soil N > 50. Based on the results from our subsurface exploration the soil profile at the site may be defined as Site Class D according to IBC (International Building Code) 2018, representing a stiff soil. Seismic design parameters for this site class and location, from ASCE 7 Hazard Tool with ASCE/SEI 7-22 reference document, are summarized in the following table:

Table 4: Spectral Response Acceleration (SRA)

SRA and Site Conditions	Short Period (0.2 sec)	1- Second Period
Mapped SRA	$S_S = 1.61$	$S_1 = 0.62$
SITE CLASS D		
Site Coefficients	$F_a = 1.06$	$F_v = 2.08$
Max. Considered Earthquake SRA	$S_{MS} = 1.72$	$S_{M1} = 1.29$
Design SRA	$S_{DS} = 1.14$	$S_{D1} = 0.86$

9.GEOLOGIC HAZARD MITIGATION

9.1 Landslide Hazard Mitigation (LDC 19.07.100)

Based on our subsurface exploration and literature research, we have determined that the site slope is composed of relatively shallow till soil. We have completed a slope stability analysis and determined that the steep slope is generally having a factor of safety greater than 1.5, which is considered acceptable for permanent important structures on steep slope area or adjacent area. Please note that the Puget Sound area is seismically active and there may be seismic hazards associated with the slope as discussed below.

A critical slope area and associated “No Building Buffer” area is typically designated (refer to chapter 9.4), which should not be developed in order to mitigate possible landslide and erosion hazards. This scenario is apparently not applicable to this site. The main concern with the proposed development involves loading or disturbing shallow cover soils in a manner that increases erosion or causes shallow slumps, surface creep, or subsidence. Our recommendations above are generally for cut grading to construct the proposed ADU on the top of slope of the property. It is our opinion that the subject slope may not be adversely affected since the proposed ADU will be at same location of existing detached garage with 5 feet setback from slope, and therefore, the proposed development will not increase the potential for landslide or erosion hazard if the recommendations in this report are followed.

9.2 Erosion Hazard Mitigation (LDC 19.07.100)

The following general recommendations will assist in preventing high rates of erosion that may cause site slopes to become unstable during and after construction. Vegetation removal and ground disturbance on the slope should be limited to only as necessary in work areas. During construction if warranted or desired, soil containing measures such as silt fencing, hay bails, or straw waddles may be helpful in limiting erosion and the effects of earthwork activity on adjacent areas down slope. The contractor should comply with Best Management Practices (BMPs) and temporary erosion control measures as required.

The soil cutslope recommendations above are intended to reduce the overall area of ground disturbance while allowing for re-establishment of adequate slopes after construction. Care should be taken to restore disturbed or cleared areas to a naturalized state via plantings, control drainage and limit runoff onto these areas, and provide ongoing maintenance of vulnerable slope faces as needed in the long term, to minimize the potential for erosion and local soil slumps.

Construction and construction equipment should be confined to the areas of the building construction. It should be understood that the impacts of construction equipment and vegetation removal disturbs the soil and removes stabilizing rooting, causing the slope surface to be less stable. Equipment use on the slopes should be limited to light machinery.

We recommend drainage controls and appropriate outlets be applied as possible to limit the potential for slope instability due to water inundation. The on-site soil may cause areas of temporary saturation during storm events, especially around grading alterations and improvements. Features that may collect ground or surface water, should be designed with perforated pipe for water collection and drainage. Drainage pipes should be tightlined to a natural drain course or dispersed in an appropriate area below and away from slope faces.

Provided that the recommendations above are implemented, we conclude that the erosion hazard will be adequately mitigated during and after site development.

9.3 Seismic Hazard Considerations

The seismic hazards has been addressed early in this report Section 8.8 Seismic Design Parameters.

9.4 Buffers and Setbacks (LDC 19.07.100)

Based on our evaluation, and provided that the proposed development is designed and constructed in accordance with our recommendations provided in this report, the proposed development:

- Proposed development is not located in a landslide hazard area and seismic hazard is low;
- Would not increase the hazard to adjacent properties with similar slopes over the existing condition or result in a need for increased buffers on neighboring properties; and
- Can be designed safely under anticipated conditions.

Our site reconnaissance did not find obvious visual evidence of current or past slope stability issues within the project area. We are not aware of any historic slope failures in the site vicinity. The slope stability analysis was performed for proposed ADU structure at top of middle portion slope of subject site. The factor of safety for the critical surfaces range from

2.14 to 5.94, which is over 1.5 for representative cross-sections A-A´. This gives us that at top of slope with proposed ADU may be satisfactory.

10.GENERAL CONDITIONS

The recommendations provided herein are based on our understanding of the project at this time. We expect the on-site geologic conditions to reflect our findings, however, some variations may occur. Should soil conditions be encountered that cause concern and/or are not discussed herein, Merit Engineering, Inc. should be contacted immediately to determine if additional or alternate recommendations are required.

We recommend that we review those portions of the plans and specifications pertaining to earthwork, cutslopes, and wall design to ensure that they are consistent with the recommendations in this report.

We recommend that we verify site excavation to suitable soil stratum, verify imported fill materials, and observe and test compaction of structural fill. We recommend that we be retained to evaluate the condition of cutslopes once open, including soil condition, structures, and soil stability, and provide recommendations as needed for remediation or reinforcement of permanent cutbanks.

This report is prepared for Buping Wang for specific application to the critical areas evaluation for the proposed development located at 6450 E Mercer Way, in Mercer Island, Washington 98040 (Parcel #302405-9004). This report has been prepared in accordance with generally accepted geotechnical/geological engineering practices in this area. No other warranty, expressed or implied, is made.

This report is an instrument of our professional service, and we (Merit Engineering, Inc.) shall retain an ownership and property interest therein. We grant Buping Wang a license to use the instrument of our professional service for the purpose of constructing the above mentioned proposed improvements. We do not permit reuse or modification of this document for application to a different structure or location other than the proposed or to another property because soil and subsurface conditions are unique and site specific for different locations.

The STABLE computer model was calibrated using the observed slope height of ~14´. Should the slope height change, or soil conditions or other related parameters change, the calculations would be inaccurate and Merit Engineering, Inc. should be contacted to determine if alternate recommendations are required. We expect the onsite conditions to reflect our findings; however, some variations may occur. Should soil conditions be encountered that cause concern and/or are not discussed herein, Merit Engineering, Inc. should be contacted immediately to determine if additional or alternate recommendations are required.

Results presented in this report are based on our field tests and engineering analysis using the best available knowledge we have, which by no means gives any warranty or guarantee that slope failure after site development will not occur. We understand the owners have chosen and are ultimately willing to live an area that is a potential geohazard. Therefore, we recommend long term monitoring of the slope. Additionally, the owners and/or their representatives should understand that they are willing to take the risk to live in a geologically critical area and, therefore, agree to indemnify and hold Merit Engineering, Inc.

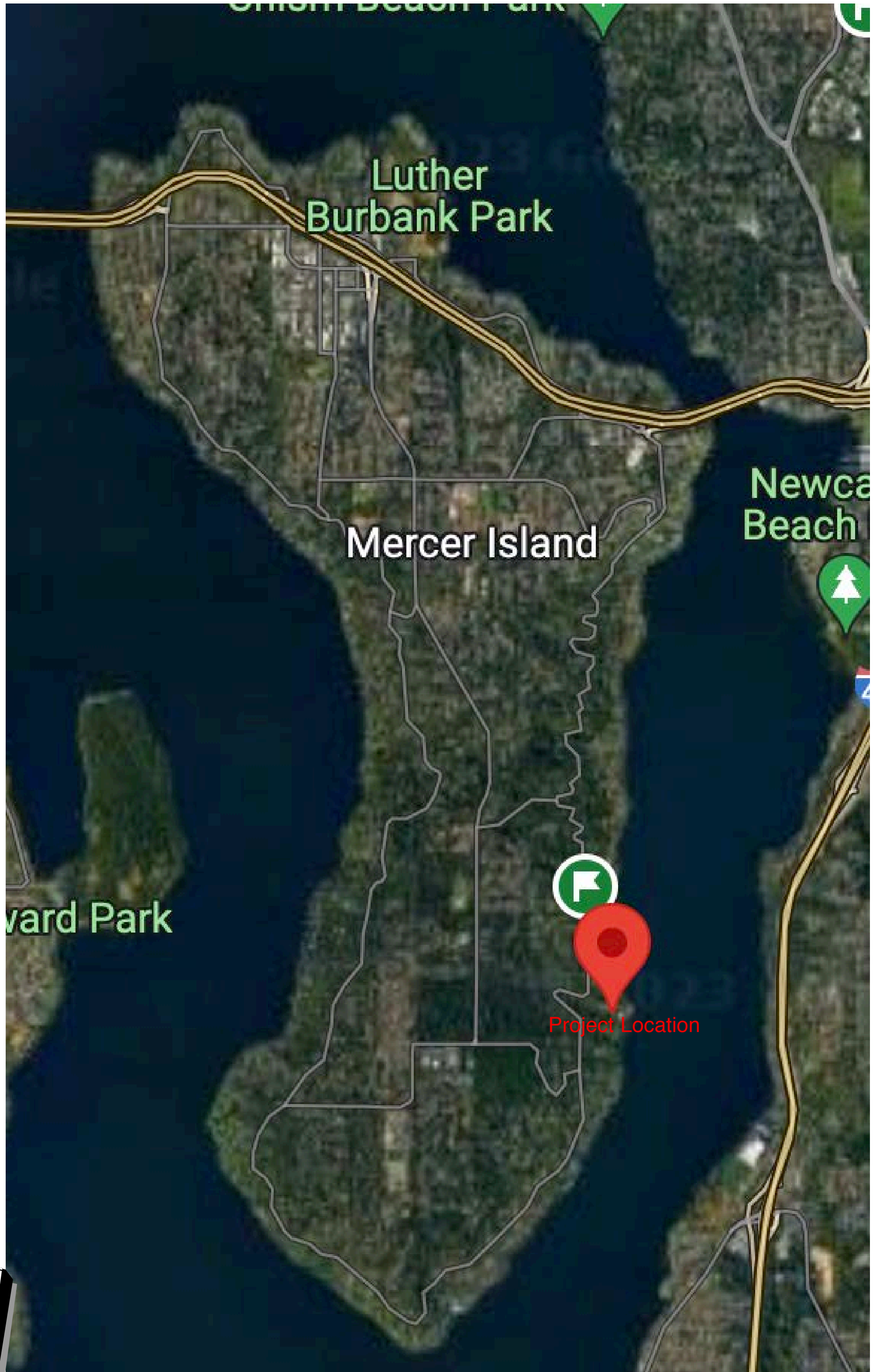
harmless, including its owners and employees, for the property owners are ultimately responsible for the potential adverse consequences of living in a geologically critical area.

This study has used limited methods employed for evaluating slope conditions within the site and its vicinity, with limitations noted herein, as allowed within the reasonable scope of the project. This study does not attempt to perform an in-depth subsurface investigation of deep soils or bedrock, which is beyond the feasible scope and cost of the project at this time. The on-site soil condition should be reevaluated, if different soils are encountered during construction or if the deep excavation below current ground surface occur. We recommend that we be retained consult and review the soil conditions when subsurface open up or during the construction phase of the project to confirm they are in accordance with the soils encountered in this report. The owners should therefore understand that the soil conditions described herein are the result of surficial interpretations provided within the scope of this study, and thus are not intended to represent or substitute for a comprehensive study of subsurface conditions.

APPENDIX

Subsurface conditions at the site were investigated by conducting three (3) DCP tests on September 7, 2023. DCP test locations were determined by a representative of Merit Engineering Inc. as shown approximately on the Site Plan (Figure 2) presented in the Appendix of this report. Tests were conducted near the adjacent to the building footprint to generalize subsurface soil conditions. Depths referred to in this report are relative to the existing ground surface at the time of this field investigation.

Descriptions of subsurface conditions are based on observations made at the site at the time of the field investigation. DCP test logs are presented in Figures 4 through 6. The soils observed at the site were classified using the USCS (Unified Soils Classification System) in accordance with ASTM D-2488-69 and ASTM D 2487. This classification system is also presented in the Appendix (Figure 3).



N

Project No: 2EH03221024	**PROJECT LOCATION & VICINITY MAP**	Date: 09/07/2023	*Figure 1*

Proposed Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, WA

Prepare For: Buping Wang

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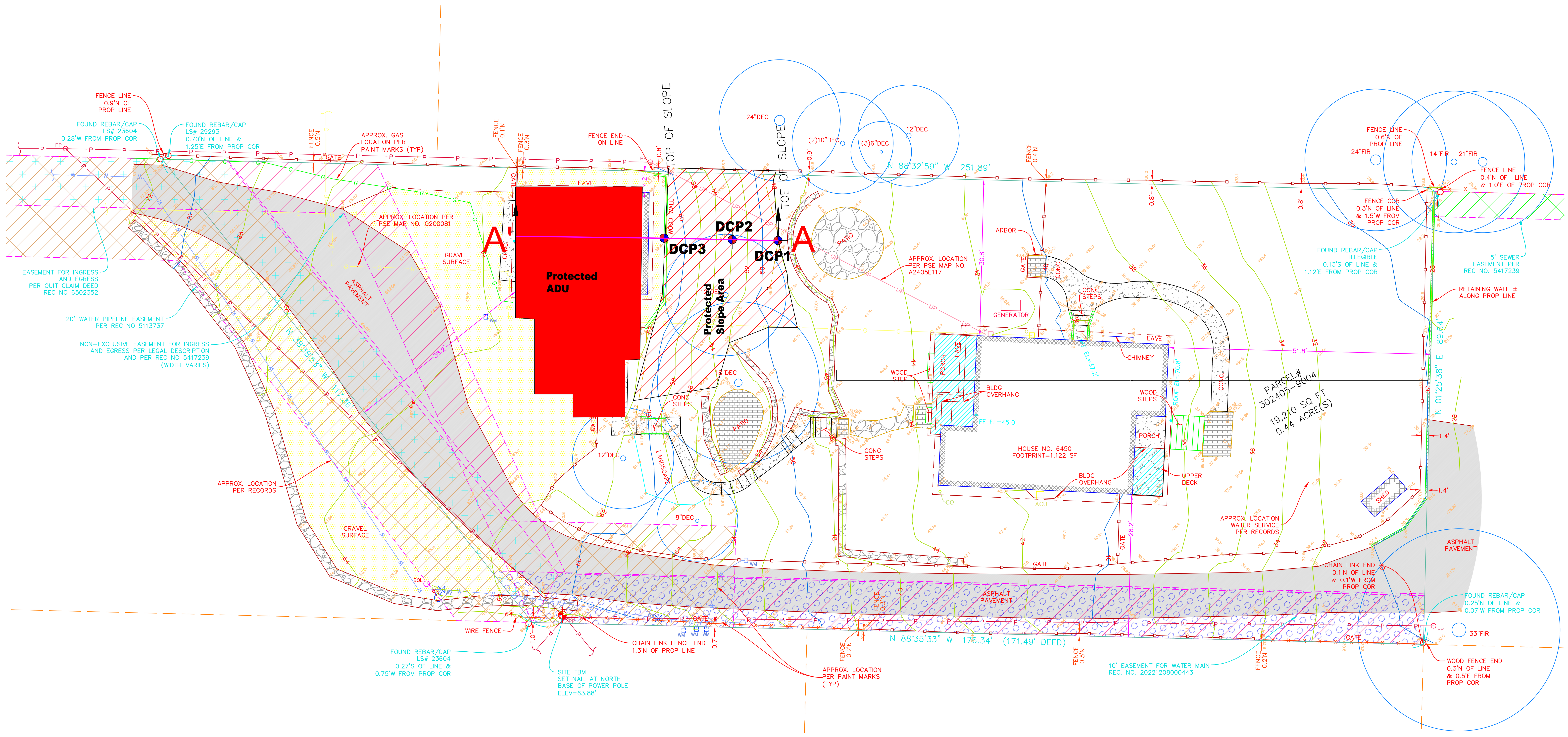

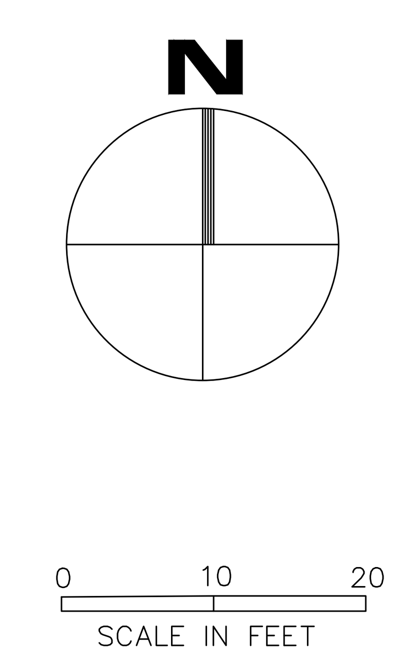

LEGEND

- DYNAMIC CONE PENETRATION TEST HOLE
- **PROTECTED** SLOPE OVER 40% (22°)
- PRELIMINARY BUILDING FOOTPRINT

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SITE PLAN

PROJECT NO.	DATE	APPROVED BY
2EH03221024	10/04/2023	AXH

New Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, WA
Parcel Number: 302405-9004

Figure 2 SCALE; 1":10'

Note: Property Lines and Topography based on the Survey From Terrane

For: Buping Wang

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				DESCRIPTION
COARSE GRAINED SOILS more than 50% retained on #200 sieve	**GRAVELS** more than 50% coarse fraction is larger than No. 4 sieve size	Gravels with less than 5% fines	GW	Well graded gravels, gravel-sand mixtures
			GP	Poorly graded gravels, gravel-sand mixtures
		Gravels with more than 12% fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS more than 50% coarse fraction is smaller than No. 4 sieve size	Sands with less than 5% fines	SW	Well graded sands, gravelly sands
			SP	Poorly graded sands, gravelly sands
		Sands with more than 12% fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS more than 50% passing #200 sieve	**SILTS AND CLAYS** Liquid Limit less than 50		ML	Inorganic silts & very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, or lean clays
			OL	Organic clays and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid Limits greater than 50		MH	Inorganic silts, micaceous or diatomacious fine, sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils
UNCONTROLLED FILL				Uncontrolled, with highly variable constituents

LEGEND

SAMPLE	SYMBOL
SPLIT SPOON SAMPLER	∇ GROUNDWATER TABLE
SHELBY TUBE SAMPLER	q_u PENETROMETER READING TSF (*tons per square foot*)

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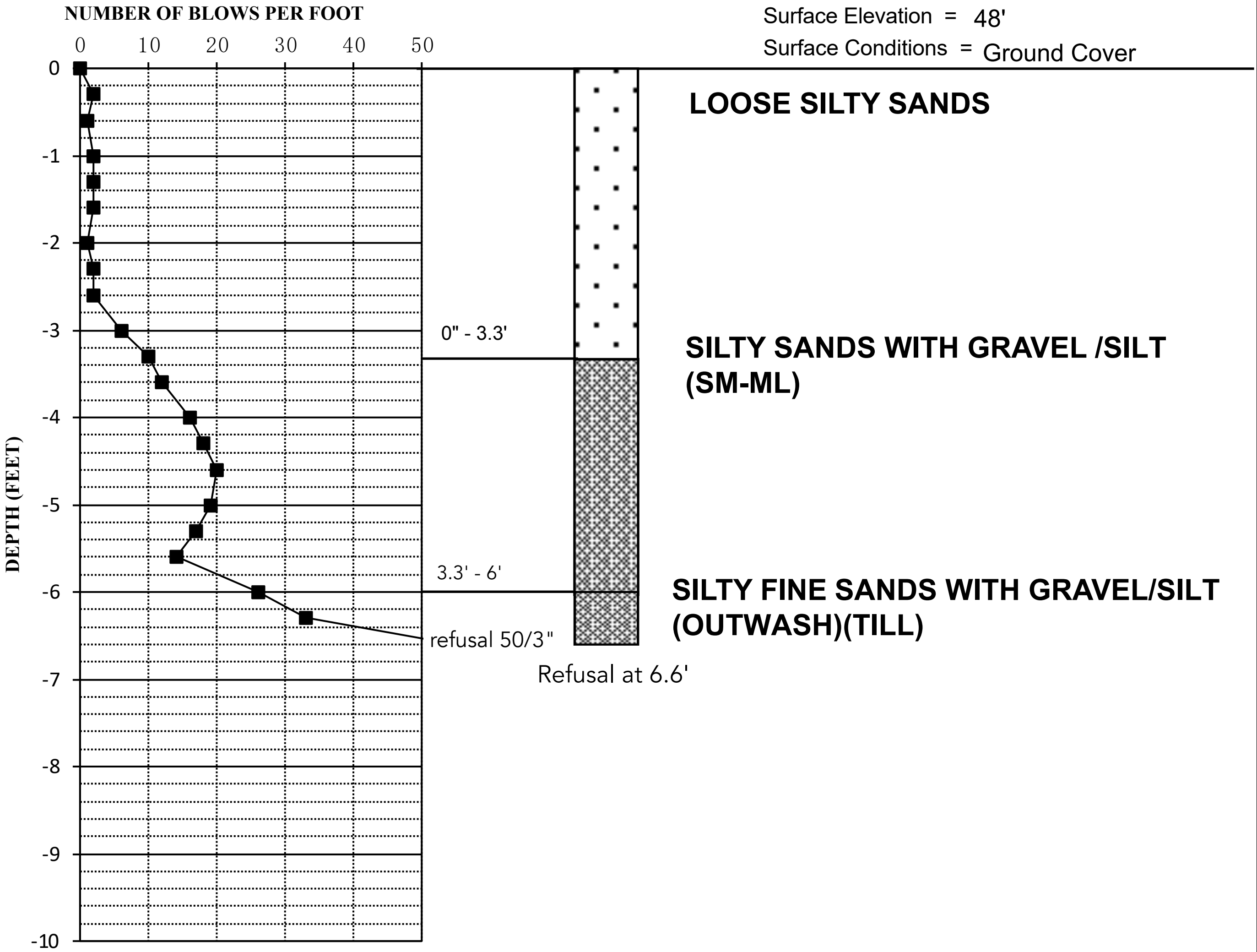
SOIL CLASSIFICATION & LEGEND

Figure 3

DCP-1 DYNAMIC CONE PENETRATION TEST | SOIL DESCRIPTION AND CLASSIFICATION

X,Y =
Surface Elevation = 48'
Surface Conditions = Ground Cover



Note: Soil profile is inferred from CPT results. Subsrface conditions were not observed directly during DCP testing.

Project No: 2EH03221024	Date: 09/07/2023	**LOG OF DCP TEST**	Approved by AXH	Figure 4

Proposed Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, WA

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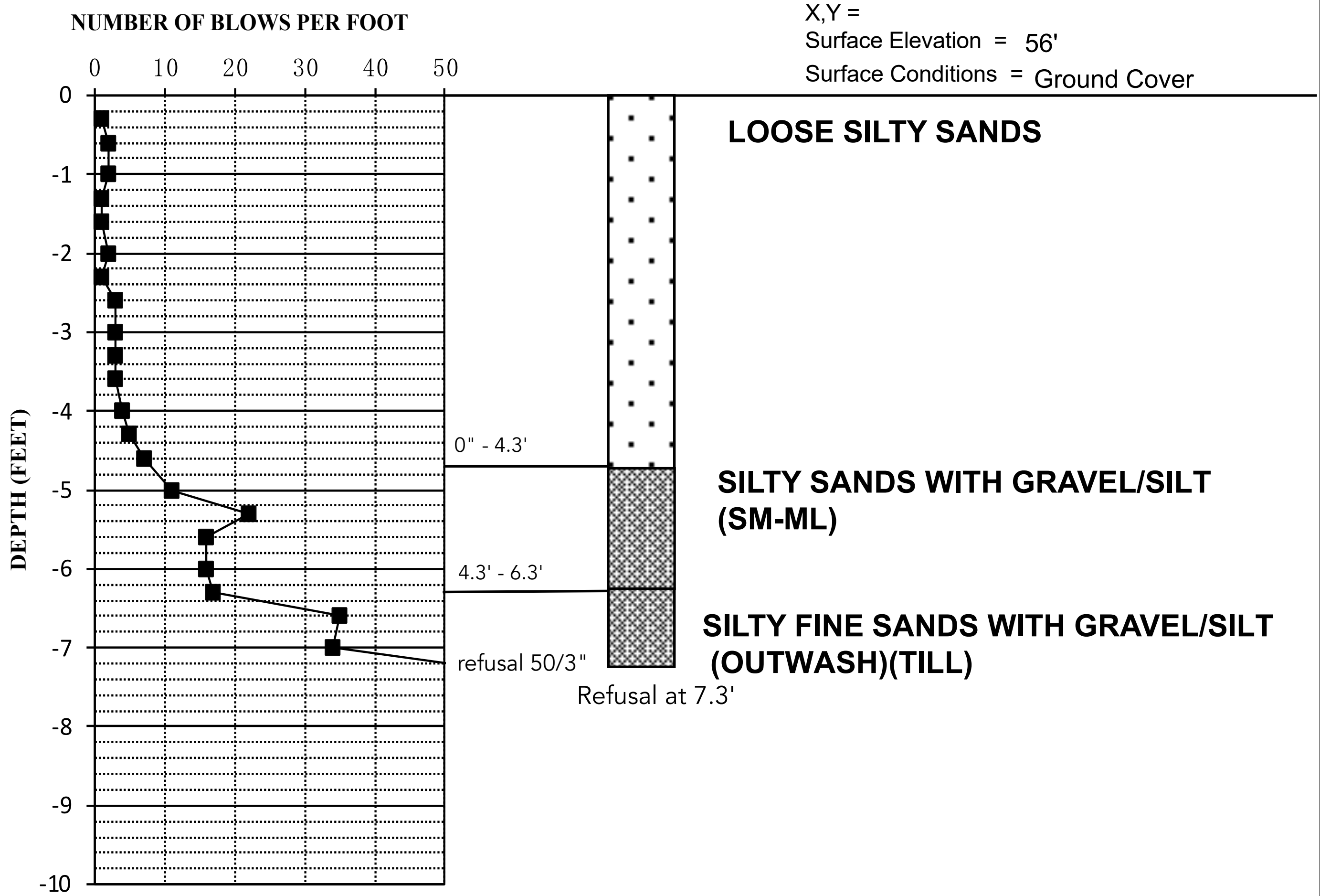
DCP-2 DYNAMIC CONE PENETRATION TEST | SOIL DESCRIPTION AND CLASSIFICATION

X,Y =
Surface Elevation = 56'
Surface Conditions = Ground Cover



Note: Soil profile is inferred from CPT results. Subsrface conditions were not observed directly during DCP testing.

Project No: 2EH03221024	Date: 09/07/2023	**LOG OF DCP TEST**	Approved by AXH	Figure 5

Proposed Accessory Dwelling Unit
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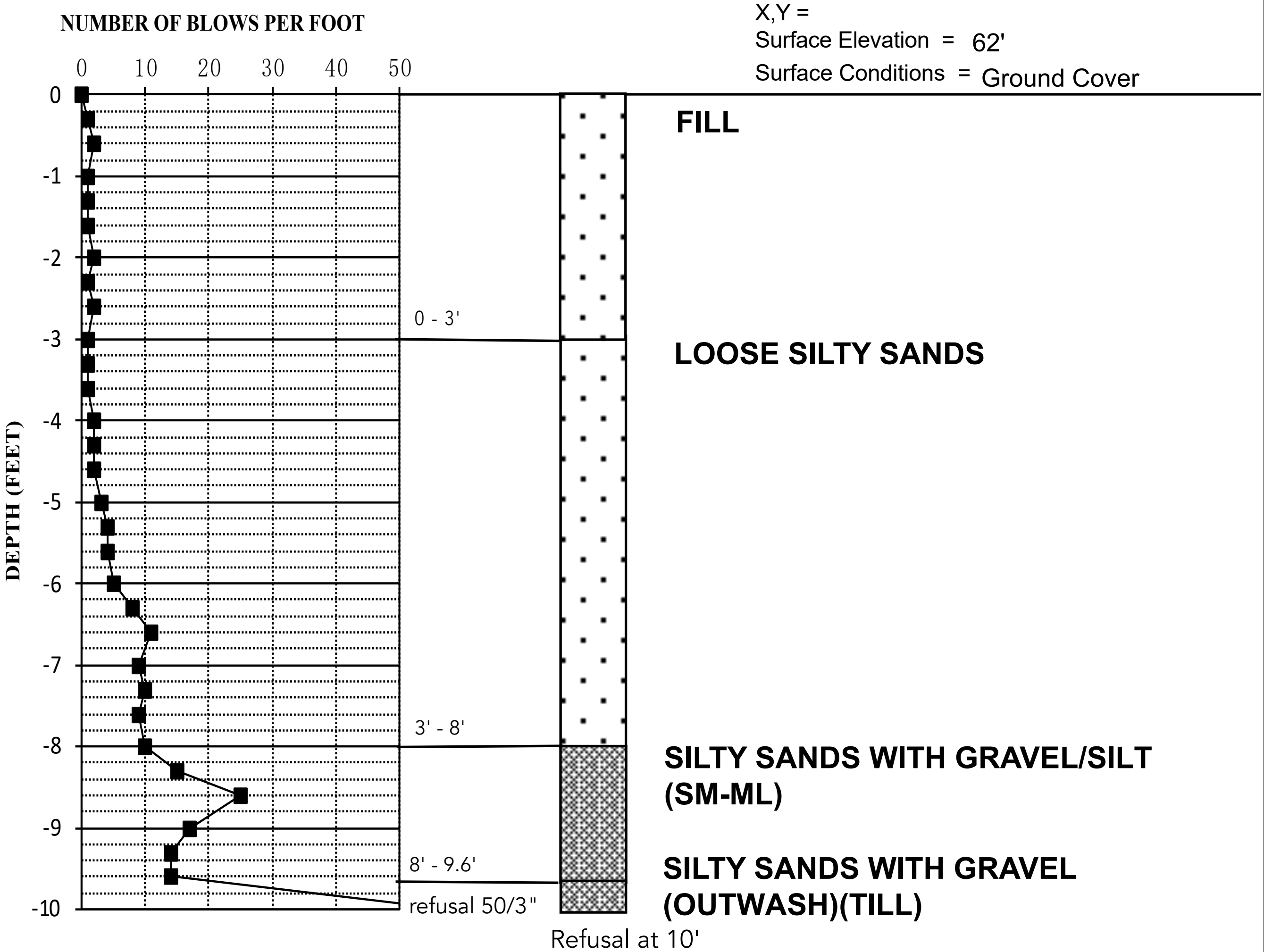
DCP-3 DYNAMIC CONE PENETRATION TEST	SOIL DESCRIPTION AND CLASSIFICATION

X,Y =
Surface Elevation = 62'
Surface Conditions = Ground Cover



Note: Soil profile is inferred from CPT results. Subsrface conditions were not observed directly during DCP testing.

Project No: 2EH03221024	Date: 09/07/2023	**LOG OF DCP TEST**	Approved by AXH	Figure 7

Proposed Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, WA

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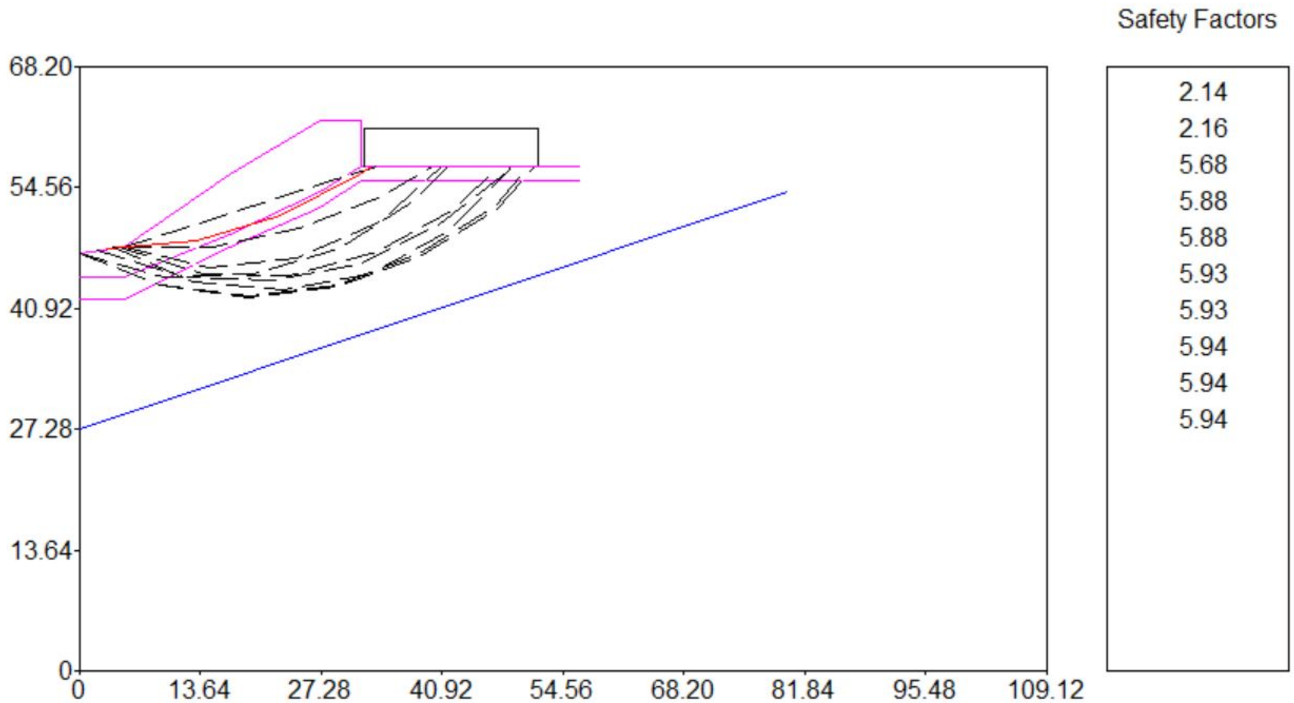
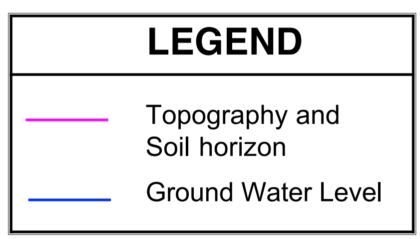
STATIC CONDITION	*Figure 7*





Project No. 2EH03221024	Date: 09/07/2023	**CROSS SECTION A-A'**	Approved by A.X.H.	*No Scale*

Proposed Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, WA

Prepared For: Buping Wang

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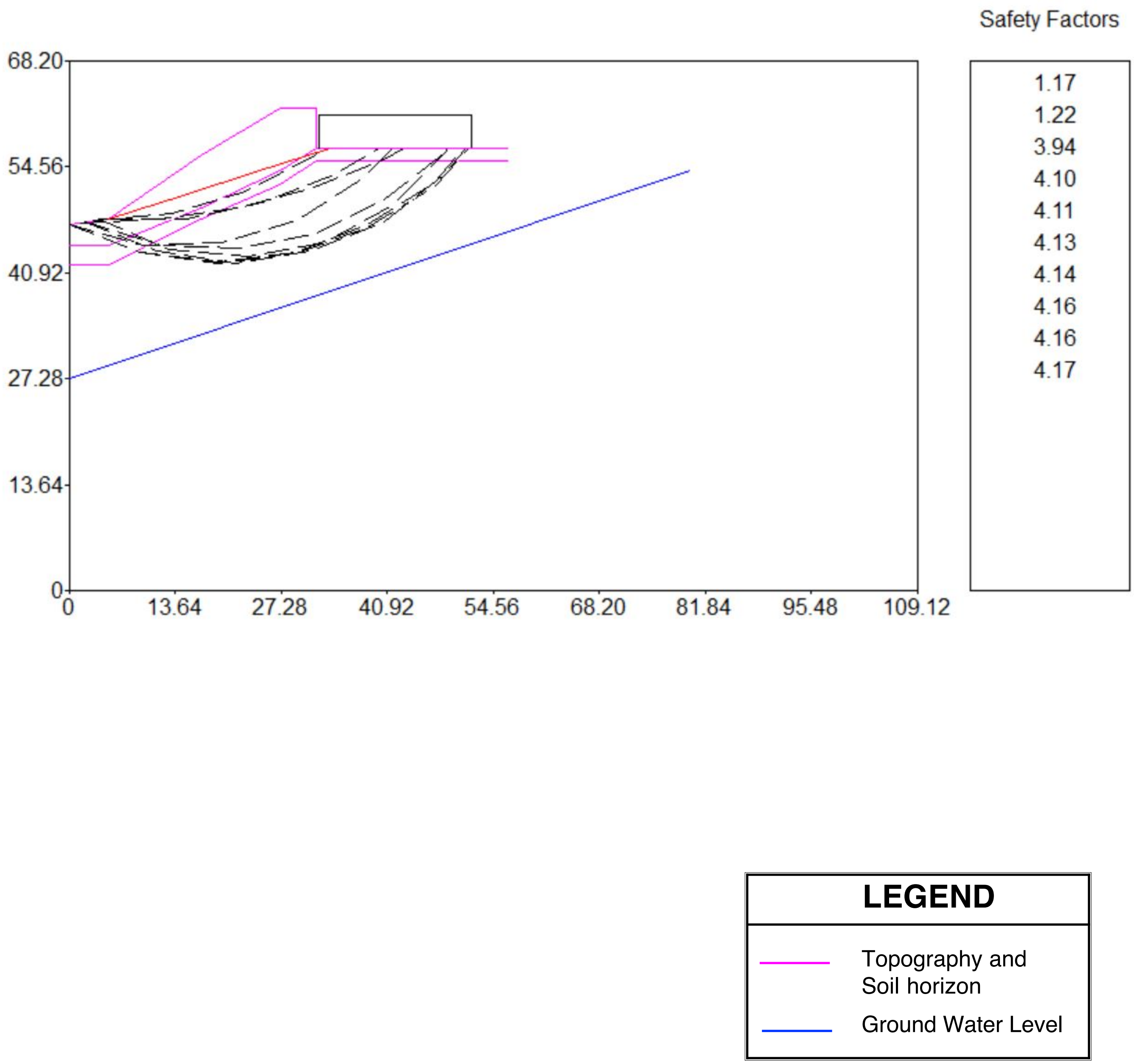
SEISMIC CONDITION g=0.31	*Figure 8*



Project No: 2EH03221024	Date: 09/07/2023	**CROSS SECTION A-A'**	Approved by A.X.H.	*No Scale*

Proposed Accessory Dwelling Unit
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Mercer Island, WA

Prepare For: Buping Wang

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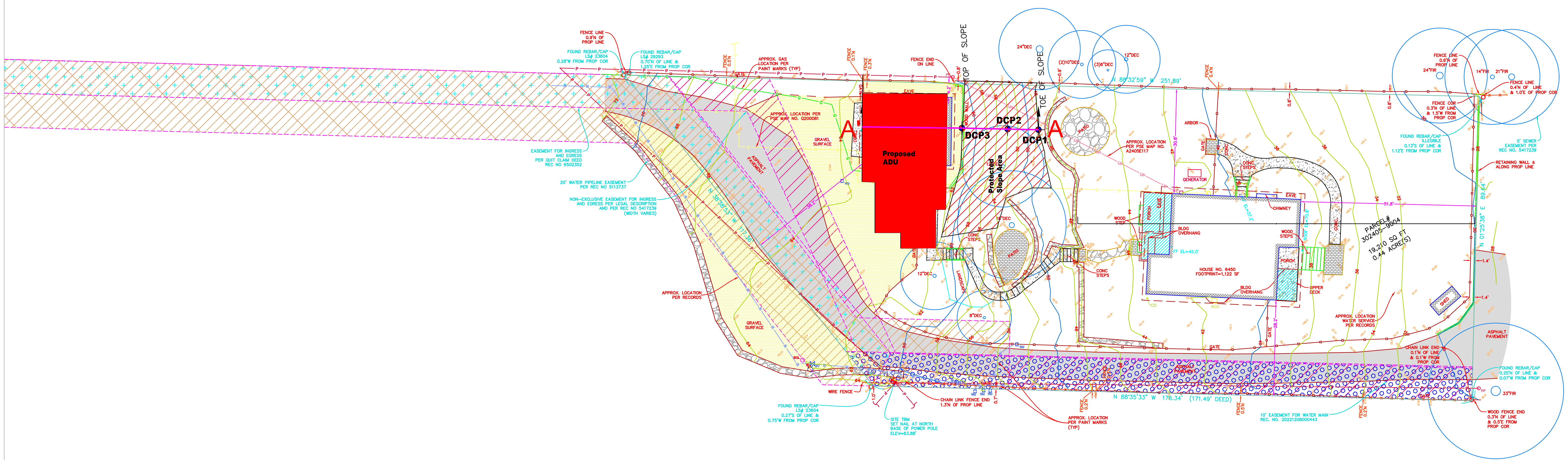

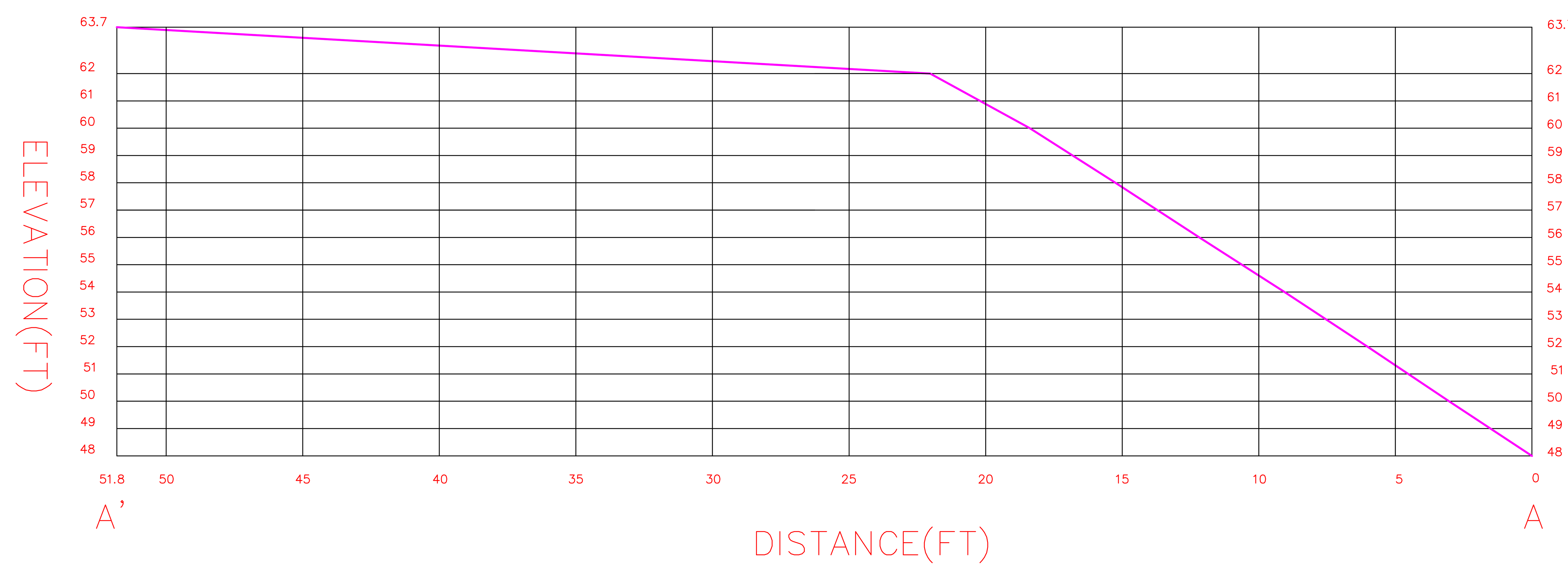

Note: Property Lines and Topogrphy based on the Survey From Terrane	New Accessory Dwelling Unit 6450 E Mercer Way Mercer Island, WA Parcel Number: 302405-9004	SLOPE PROFILE	
Figure 9		PROJECT NO. / DATE / APPROVED BY	
For: Buping Wang	SCALE: NOT ON SCALE	2EH03221024 / 10/04/2023 / AXH	

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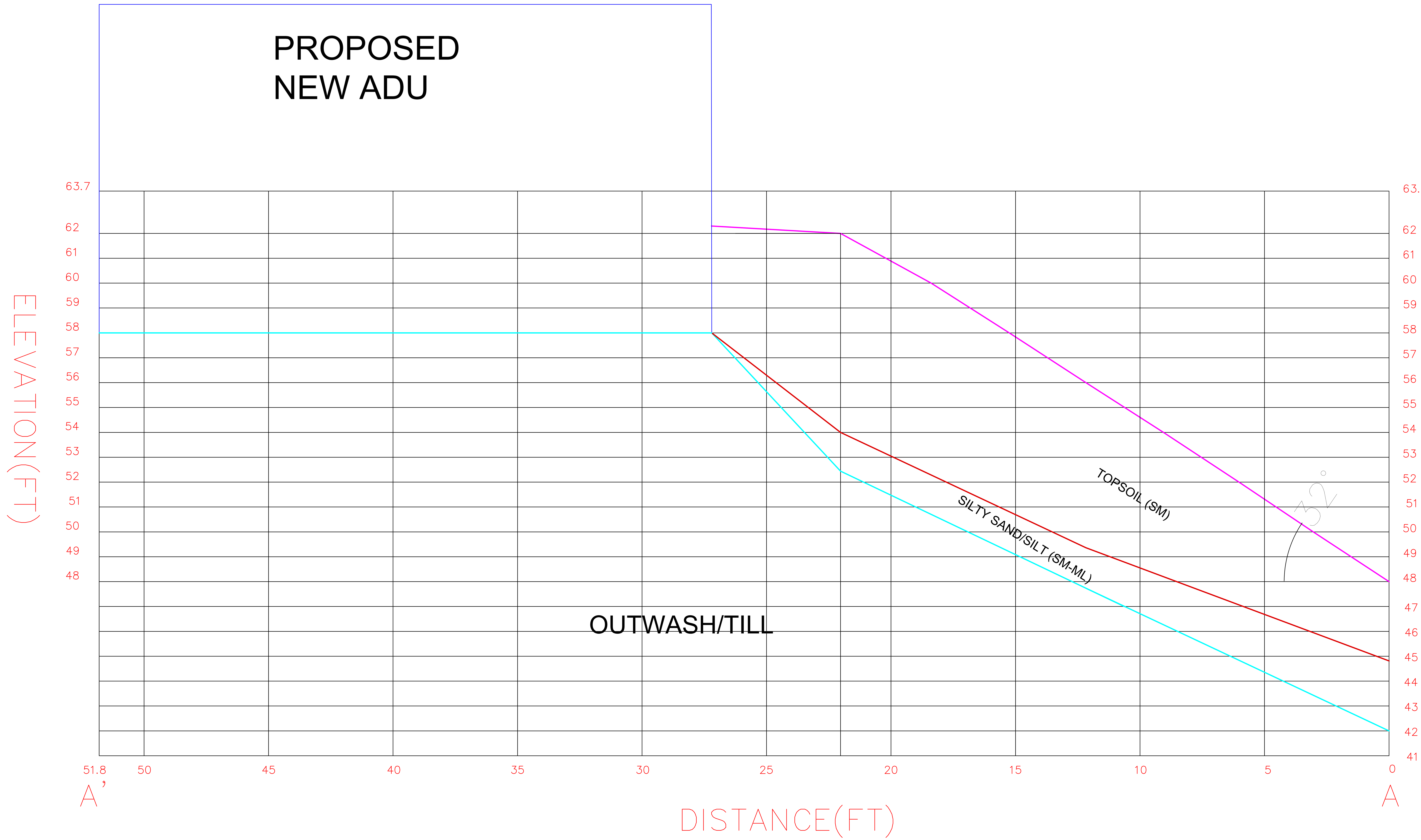

Note: Property Lines and Topogrphy based on the Survey From Terrane

Figure 10

For: Buping Wang

New Accessory Dwelling Unit
6450 E Mercer Way
Mercer Island, WA
Parcel Number: 302405-9004

SCALE: NOT ON SCALE

CROSS - SECTION A-A'

PROJECT NO.	DATE	APPROVED BY
2EH03221024	10/04/2023	AXH

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