

---

November 18, 2025

JN 25340

Wishwas Mohan  
2441 – 66<sup>th</sup> Avenue S.E.  
Mercer Island, Washington 98040  
via email: [gmwishwas@gmail.com](mailto:gmwishwas@gmail.com)

Subject: **Updated Geotechnical Engineering Study**  
Proposed Property Redevelopment  
8203 Avalon Drive  
Mercer Island, Washington

Reference: *Geotechnical Engineering Evaluation, Vishy and Madhuri Remodel Seismic Evaluation, 8203 Avalon Drive, Mercer Island, Washington, NGA File No. 1579125*; Nelson Geotechnical Associates; March 28, 2025.

Dear Mr. Mohan:

This *Geotechnical Engineering Study* is intended to supersede the above-referenced report completed by Nelson Geotechnical Associates. You have retained our firm as the geotechnical engineer of record for your project.

In order to prepare this report, we have completed the following tasks:

1. Visited the site on two occasions to observe the existing conditions on the property and inside the current residence.
2. Reviewed the findings of the above-referenced report.
3. Researched the Mercer Island GIS for critical area mapping and results of previous nearby explorations.
4. Reviewed our project files encompassing the past approximately 40 years for geotechnical studies and explorations that we have completed on nearby properties.

From our discussions with you, and Paul Monsef of Atera Homes, we understand that the existing home will be replaced by a new residence that will have an additional story on it. The new house will be located in the same footprint. The existing basement walls will remain to provide temporary shoring of the surrounding soil while the new home is constructed. The existing detached garage will not be structurally changed. No development of the northern portion of the property, between the garage and East Mercer Way, is planned.

## **SITE CONDITIONS**

### **SURFACE**

The rectangular-shaped site extends between Avalon Drive and East Mercer Way. For reference, Project North is assumed to be perpendicular to these two streets, parallel to the long axis of the property. The residence consists of one floor overlying a basement that is full-depth on the north side and which is embedded only 4 to 5 feet below grade on the southern side. Between the house

and Avalon Drive is flat to gently-sloped yard and landscaping. A paved driveway extends along the east side of the residence to the detached garage located behind, north of, the house.

To the north of the garage is a relatively flat to gently-sloped rear yard. At a distance of 45 feet from the garage (85 feet from the house), the ground begins to rise toward East Mercer Way over an approximate 40-foot slope. The lower one-third of this slope is moderately-inclined (15 to 30 percent). The remainder of the slope is inclined at 50 to 70 percent. The steep portion of the slope is covered with a heavy growth of mature laurels and contains numerous small to medium-sized trees. There are two large big leaf maple trees located at the bottom of the steep portion of the slope. Between the top of the steep slope and the paved edge of East Mercer Way is a permanent soldier pile wall, which is discussed further below. A storm drain pipe serving catch basins located in East Mercer Way apparently crosses this steep slope.

Our review of Mercer Island's GIS confirms that the site and surrounding neighborhood are mapped as: 1) Potential Landslide Hazard, 2) Erosion Hazard, and 3) Seismic Hazard. Additionally, the north slope is mapped as a Steep Slope Hazard.

### **Landslide History of Site and Vicinity**

The landslide history of this area of Mercer Island is well documented by information on Mercer Island's GIS and Washington Department of Natural Resources' (DNR) *Geologic Information Portal*. A large ancient landslide affected the southeastern portion of Mercer Island sometime after the recession of the last glaciers. It has been hypothesized that this ancient landslide occurred during a massive Cascadia Subduction Zone earthquake. Radiocarbon dating of sunken trees in Lake Washington indicate that this could have occurred as recently as 1,000 to 1,300 years ago. The headscarp of this ancient landslide is the tall bluff located along the north side of the lots that front on the northern edge of East Mercer Way. The Avalon Park neighborhood, in which the site is located, is within the center of this old slide mass. The soil conditions between the old headscarp and shore of Lake Washington through this area consist of landslide deposits (colluvium) comprised of disturbed sands and silt of varying composition overlying glacially-compressed silt. The undisturbed glacially-compressed silt is the old slide plane. Borings our firm has conducted on lots between Avalon Drive and Lake Washington show that the thickness of the colluvium decreases to the south, to depths of approximately 15 feet.

To our knowledge, no recorded mass movement of the ancient landslide mass has occurred in recent times. According to King County Assessor's records, the house on the site was constructed in approximately 1957, with the neighboring houses to the east and west being completed in 1958 to 1961. Since that time, the Seattle area has been subjected to numerous earthquakes of magnitude 5.0, or less, and at least two larger earthquakes having magnitudes of 6.5 and 6.8. During the 2001 Nisqually earthquake (Magnitude 6.8) there were episodes of liquefaction and slope instability recorded in the Puget Sound. To our knowledge, no liquefaction or ground failures were reported within the Avalon Park neighborhood.

Numerous shallow slides affecting both fill and disturbed native soils have occurred on steep slopes along East Mercer Way. These landslides typically affect oversteepened cut or fill slopes, and result from wet conditions related to natural precipitation or leakage from buried utilities. The Mercer Island *Landslide Hazard Assessment* indicates that at least one episode of documented slope movement has occurred on the site. In the course of our research on the Mercer Island GIS we found a portion of a geotechnical report completed by

Hart Crowser in March 1997 following slope movement that affected East Mercer Way and the top of the steep slope immediately north of the site. A copy of this 1997 document is attached. Based on the Hart Crowser report, tension cracks formed in the southern lanes of East Mercer Way, and the south shoulder slumped downward. This soil movement occurred following extended wet weather, and similar landslides occurred throughout the Puget Sound at that time. The soil movement appears to have resulted from excessive subsurface water, potentially exacerbated by leaking of the storm sewer, which caused failure of the old road fill underlying the southern portion of the street.

During our visits to the property, we climbed the steep slope, and found no indications of more recent slope movement. Fractured silt is exposed at several locations on the slope where the ground is bare of vegetation and duff. At the top of the slope, along the south edge of East Mercer Way, is a permanent soldier pile wall that was apparently installed after the 1997 slope movement. This soldier pile wall consists of painted wide-flange beams, with the upper approximately 3 feet of the wall having been lagged with treated timber. The wall extends the full width of the site, and beyond the east and west property lines. A metal guardrail is attached to the top of the soldier pile wall. There are no indications of tilting or movement of the guardrail or soldier pile walls.

## ***SUBSURFACE***

The subsurface conditions on the site were explored by Nelson Geotechnical Associates drilling one test boring to the northeast of the detached garage. The log of this boring is attached. Additionally, borings have been drilled on the lower (southern) portion of the adjacent eastern lot (#8211) by our firm, and at the top of the slope by Hart Crowser as a part of the 1997 landslide study. The logs of these explorations are attached.

### **Soil Conditions**

The site and surrounding lots are underlain by varying thicknesses of loose disturbed deposits remaining from the large ancient landslide. These landslide deposits (colluvium) consist of silty, fine-grained soils that are interspersed with organics. The boring log prepared by Nelson Geotechnical Associates describes the colluvium to consist of sand having a low silt content and varying amounts of gravel. Our exploration to the east, as well as Hart Crowser's boring, generally found the colluvium to consist of disturbed silt soils.

At a depth of 25 to 35 feet lies glacially-compressed silt. This lacustrine silt was not disturbed by the ancient landslide and represents the failure surface.

### **Groundwater Conditions**

Groundwater is typically found through the entire thickness of the colluvium, perched on top of the glacially-compressed silt. The borings found groundwater within 7 to 12 feet of the ground surface.

## ***SEISMIC CONSIDERATIONS***

In accordance with the International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type F (Failure-Prone Site Class), because the loose,

saturated soils are theoretically susceptible to liquefaction during a large earthquake. However, the code allows for an exception from the F classification if the building period is less than 0.5 seconds. Due to its relatively short height, we anticipate the proposed house will have a fundamental period of less than 0.5 seconds, and therefore a Site Class Type E can be used for the project. As noted in the *ASCE7 Hazard Tool* website, the mapped spectral acceleration value for a 0.2 second ( $S_s$ ) and 1.0 second period ( $S_1$ ) equals 1.46g and 0.50g, respectively.

The near-surface soils beneath the site consist of loose, saturated silty sand, sand, and silt. These soils have been demonstrated to have a moderate to high potential for liquefaction during a large earthquake. The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period).

The recommendations presented in this report to 1) utilize deep foundations embedded into the dense to very dense, non-liquefiable soils, and to 2) interconnect the piles using crisscrossing, reinforced grade beams are intended to prevent catastrophic foundation collapse in the event of lateral spreading caused by liquefaction in a large earthquake.

### **CRITICAL AREAS STUDY (MICC 19.07)**

**Potential Landslide Hazard and Potential Seismic Hazard Areas:** The entire subject site is located within mapped Potential Landslide and Potential Seismic Hazard areas. Both of these geologic hazard areas cover the entire surrounding Avalon Park neighborhood. The proposed location for the new house is essentially flat and is set back over 85 feet from the steep northern slope.

Due to its steep inclination, and the presence of loose, disturbed soils, the steep northern slope may experience soil movement in the future. This is most likely to occur following extended wet weather, as with the landslide that affected the top of the slope in 1997. The loose fill soils at the top of the steep slope now have been stabilized by the permanent soldier pile wall installed by the City of Mercer Island following the 1997 slope movement. This substantially reduces the potential for deeper landslides. Based on the previous slides that have occurred up and down the East Mercer Way corridor, a flow-type slide affecting the near-surface soils is the most likely type of slope movement that could occur on the steep ground in front of the soldier pile wall. This type of landslide would most likely follow extended heavy precipitation. However, the steep slope is relatively short in height, there are large, deep-rooted trees at the base of the slope to slow and/or deflect slide debris, and there is a wide level rear yard area for run-out of any slide material. Considering all of these factors, it is our professional opinion that a slide catchment wall is not needed between the steep slope and the garage. Soil and debris that may move down the steep slope would accumulate in the rear yard, but would have to be cleaned up.

The site and surrounding area did not experience liquefaction during the earthquakes that have occurred since the neighborhood was developed. However, the current International Building Code (IBC) requires that the potential for soil strength loss (liquefaction and/or lateral spreading) be evaluated for a larger, lower-probability earthquake than what has occurred in more recent time. In the event of the code-required Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years, the loose, saturated soils theoretically could liquefy (experience soil strength loss) and potentially be affected by lateral spreading. The recommendations presented in

this report are intended to mitigate the hazards to the new residence from potential liquefaction and/or lateral spreading.

**Buffers and Mitigation:** As noted above, the entire site lies within a mapped Potential Landslide Hazard Area. We recognize that the planned development will occur within the designated critical areas. The recommendations presented in this geotechnical report are intended to allow the project to be constructed in the proposed configuration without adverse impacts to critical areas on the site or the neighboring properties. The geotechnical recommendations associated with foundations and erosion control will mitigate any potential hazards to critical areas on the site.

**Erosion Hazard:** Appropriate temporary erosion control measures will need to be implemented to prevent silty runoff from leaving the property. The temporary erosion control measures needed during the site development will depend heavily on the weather and groundwater conditions that are encountered during the site work. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil areas to prevent accumulated water or runoff from the work area from becoming silty in the first place. Silty water cannot be discharged to the storm drainage system, so a temporary holding tank should be planned for wet weather earthwork. A wire-backed silt fence bedded in compost, not native soil or sand, should be erected as close as possible to the planned work area, and the existing vegetation between the silt fence and the southern property boundary left in place until final landscaping will occur. Rocked construction access and staging areas should be established wherever trucks will have to drive off of existing pavement, in order reduce the amount of soil or mud carried off the property by trucks and equipment. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. Wet weather construction (October 1 through March 31) on this site should be possible without adverse impacts to the surrounding properties. As with all construction projects undertaken during potentially wet conditions, it is important that the contractor's on-site personnel are familiar with erosion control measures and that they monitor their performance on a regular basis. It is also appropriate for them to take immediate action to correct any erosion control problems that may develop, without waiting for input from the geotechnical engineer or representatives of the City.

**Statement of Risk:** In order to satisfy the City of Mercer Island's requirements, a statement of risk is needed. The verbatim statement of risk required by Mercer Island can only be made after final review of the plans. As such, we make the following statement:

*Provided the recommendations in this report are followed, it is our professional opinion that the recommendations presented in this report for the planned alterations will render the development as safe as if it were not located in a geologically hazardous area, and will not adversely impact critical areas on adjacent properties.*

## **CONCLUSIONS AND RECOMMENDATIONS**

### **GENERAL**

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

The test borings conducted on the site and neighboring properties encountered loose colluvium consisting predominantly of sand and silt beneath the ground surface. Competent glacially-compressed silt was revealed below a depth of 30 to 35 feet. Conventional shallow foundations constructed on the loose, moderately-compressible, colluvial soils beneath the ground surface would experience significant post-construction settlement as the loose soils consolidate over time. Considering this, we recommend the proposed house, including the basement slab, be supported on small-diameter pipe piles driven into the glacially-compressed soil.

The existing basement walls will remain in place to avoid the need for shoring or tall temporary cut slopes to construct the new home. If possible, the new piles and floor slab should be constructed on top of the existing slab, which would avoid the need to brace the base of the existing foundation walls. This would also provide a solid working surface for pile installation and foundation construction. The potential need for bracing of the top of the existing walls when the existing house is demolished will have to be evaluated by the structural engineer. If necessary, the lateral soil forces acting on the existing walls could be temporarily reduced by removing several feet of the existing wall backfill.

**Lateral Spreading:** While the deep foundations will protect the house from catastrophic foundation settlement in the event of seismic liquefaction, the loose, saturated soils may experience lateral spreading (lateral ground movement) if widespread liquefaction occurs. The potential for lateral spreading in the event of the MCE is poorly understood. Existing methods to estimate the potential amount of lateral ground movement that could occur are empirically based on soil properties, amount of ground shaking, and ground slope conditions where lateral spreading has been documented. These methods are very approximate, and the maximum ground slope where lateral spreading has been studied is a maximum of only approximately 3 percent. The building area and the flat, southern portion of the site are sloped at approximately this inclination, but the ground south of Avalon Drive is inclined at 15 to 20 percent. The amount of lateral spreading could be limited by a few factors. Additionally, while the onsite boring indicated the colluvium to be more granular, the surrounding borings found the colluvium to be more fine-grained. The fine-grained soils likely will not experience the same degree of liquefaction and/or lateral spreading. The reduction in the thickness of the colluvium, as well as the thickness of the perched groundwater on the properties south of Avalon Drive may reduce the potential for liquefaction and/or lateral spreading downgradient of the site.

We used NovoLiq, a computer program that utilizes several different empirical methods to estimate the amount of movement that could occur if lateral spreading occurs. The results of our lateral spreading analyses, which are attached, indicate that for a 3-percent ground slope, lateral ground movement of between 10 and 30 feet could theoretically occur in the MCE. The calculated range of theoretical lateral movements is large, and the depth of colluvium that could be affected by the lateral spreading cannot be calculated. Even so, the smallest theoretical movement is large enough that no pile system, whether drilled or driven, or localized ground improvement can prevent lateral spreading from occurring. Piles or ground improvement could not withstand large lateral movements without shearing off.

From a geotechnical standpoint, preventing lateral spreading to any significant depth on a residential lot is not possible or practical. In our professional opinion, the only feasible mitigation of the life/safety hazard of foundation collapse in the event of lateral spreading would be achieved by interconnecting the piles with reinforced grade beams or a reinforced mat slab. The grade beams should be interconnected; no isolated pile caps should be used for support of the house. In the event that the ground moves sideways a sufficient distance

to bend or break the piles, the grade beams/mat slab combined with the basement walls would serve to hold the structure in one piece, essentially acting as a raft, even if it tilts a significant amount when it moves.

There are no indications of water problems in the basement of the existing house. Even so, it will be important to install waterproofing and drainage measures between the new and existing foundations and slabs. Drainage composites and waterproofing can be placed between the new and existing basement walls, and vapor protection should be installed under the new slab. An interior drain would have to be installed to collect any water that accumulates in the drainage composites.

The site is underlain by low permeability soils and a relatively shallow groundwater table. Additionally, the ground slopes down toward Avalon Drive and developed properties to the south. Considering this, it is our opinion that onsite stormwater infiltration or dispersion is infeasible on the subject site from a geotechnical standpoint. Attempting to infiltrate or disperse runoff from impervious surfaces will increase the potential for flooding on the downgradient properties.

It is likely that some settlement of the ground surrounding pile-supported buildings will occur over time.

In order to reduce the potential problems associated with this, we recommend the following:

- Fill to the desired site grades several months prior to constructing on-grade slabs, walkways, and pavements around the buildings. This allows the underlying soils to undergo some consolidation under the new soil loads before final grading is accomplished.
- Connect all in-ground utilities beneath the floor slabs to the pile-supported floors or grade beams. This is intended to prevent utilities, such as sewers, from being pulled out of the floor as the underlying soils settle away from the slab. Hangers or straps can be poured into the floors and grade beams to carry the piping. The spacing of these supporting elements will depend on the distance that the pipe material can span unsupported.
- Construct all entrance walkways as reinforced slabs that are doweled into the grade beam at the door thresholds. This will allow the walkways to ramp down and away from the building as they settle, without causing a downset at the threshold.
- Isolate on-grade elements, such as walkways or pavements, from pile-supported foundations and columns to allow differential movement.

The adjacent older houses are likely supported on conventional foundations that bear on compressible soils. As a result, it is likely that they have undergone excessive settlement already. There is always some risk associated with demolition and foundation construction near structures such as this. Contractors working on the demolition and construction of your home must be cautioned to avoid strong ground vibrations, which could cause additional settlement in the neighboring foundations. During demolition, strong pounding on the ground with the excavator, which is often used to break up debris and concrete, should not occur. Large equipment should not be driven quickly near the property lines and vibratory compactors, such as hoe-packs or vibratory rollers, should not be used on the site. While driving of small-diameter pipe piles is a noisy process and can cause faint ground vibrations, the ground vibrations during their installation are only minor,

due to their relatively small end areas. These weak ground vibrations are not strong enough to cause ground settlement, unlike what occurs on commercial sites where larger piles are driven with very heavy impact or vibratory hammers. However, in order to protect yourselves from unsubstantiated damage claims from the adjacent owners, 1) the existing condition of the foundation should be documented before starting demolition, and 2) the footings should be monitored for vertical movement during the demolition, excavation, and construction process. These are common recommendations for projects located close to existing structures that may bear on loose soil and have already experienced excessive settlement. We can provide additional recommendations for documentation and monitoring of the adjacent structures, if desired.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking, cleaning, and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

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

**PIPE PILES**

Four-, or 6-inch-diameter pipe piles driven with 1,100-, 2,000-pound, or 3,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (1,100-pound hammer)	FINAL DRIVING RATE (2,000-pound hammer)	FINAL DRIVING RATE (3,000-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
4 inches	10 sec/inch	4 sec/inch	n/a	10 tons
6 inches	20 sec/inch	10 sec/inch	6 sec/inch	15 tons

**Note:** The refusal criteria indicated in the above table are valid only for pipe piles that are installed using a hydraulic impact hammer carried on leads that allow the hammer to sit on the top of the pile during driving. If the piles are installed by alternative methods, such as a vibratory hammer or a hammer that is hard-mounted to the installation machine, numerous load tests to 200 percent of the design capacity would be necessary to substantiate the allowable pile load. The appropriate number of load tests would need to be determined at the time the contractor and installation method are chosen.

As a minimum, Schedule 40 pipe should be used. Organics are often interspersed within the colluvial soils. Therefore, due to an elevated corrosion potential, it is our opinion that corrosion protection, such as galvanizing, be used on the pipe piles.

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using an ultimate (no safety factor included) passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. Compacted fill placed against the foundations can consist of imported soil that is tamped into place using the backhoe or is compacted using a jumping jack compactor. It is necessary for the fill to be compacted to a firm condition, but it does not need to reach even 90 percent relative compaction to develop the passive resistance recommended above.

### **FOUNDATION AND RETAINING WALLS**

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

<b>PARAMETER</b>	<b>VALUE</b>
Active Earth Pressure *	45 pcf
Passive Earth Pressure	300 pcf
Soil Unit Weight	130 pcf

Where: pcf is Pounds per Cubic Foot, and Active and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

\* For a restrained wall that cannot deflect at least 0.002 times its height, an at-rest earth pressure calculated using an equivalent fluid pressure of 65 pcf should be used. This applies only to walls with level backfill.

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. The existing site retaining wall north of the proposed residence and covered walkway will likely place a surcharge onto the proposed structures' northern foundation walls. We can provide appropriate surcharge loads once more detailed plans have been developed. It may be possible for the excavation shoring to be designed to withstand this surcharge. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired.

The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

### **Wall Pressures Due to Seismic Forces**

Per IBC Section 1803.5.12, a seismic surcharge load need only be considered in the design of walls over 6 feet in height. The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is  $9H$  pounds per square foot (psf), where  $H$  is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

### **Retaining Wall Backfill and Waterproofing**

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The on-site soils are not free-draining, and should not be reused as wall backfill.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls at one to 2 percent to reduce the potential for surface water to percolate into the backfill.

Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. Foundation drainage and waterproofing systems are not intended to handle large volumes of infiltrated water. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The recommended wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-

operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled **General Earthwork and Structural Fill** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a buildup of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

## **FLOOR SLABS**

As discussed in the **General** section, we recommend the basement slab be constructed as a structural slab spanning between the pipe pile supported foundations.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI recommends a minimum 10-mil thickness vapor retarder for better durability and long term performance than is provided by 6-mil plastic sheeting that has historically been used. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection.

If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when

tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

### **DRAINAGE CONSIDERATIONS**

Footing drains should be used where: 1) crawl spaces or basements will be below a structure, 2) a slab is below the outside grade, or 3) the outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock that is encircled with non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space. The discharge pipe for subsurface drains should be sloped for flow to the outlet point. Roof and surface water drains must not discharge into the foundation drain system. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

As a minimum, a vapor retarder, as defined in the **Floor Slabs** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing a few inches of free draining gravel underneath the vapor retarder is also prudent to limit the potential for seepage to build up on top of the vapor retarder.

Final site grading in areas adjacent to a building should slope away at least one to 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the **Foundation and Retaining Walls** section.

### **GENERAL EARTHWORK AND STRUCTURAL FILL**

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process. As discussed in the **General** section, the on-site soils are not suitable for reuse as structural fill, due to their high fines content and moisture sensitivity.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches, but should be thinner if small, hand-operated compactors are used. We recommend testing structural fill as it is placed. If the fill is not sufficiently compacted, it should be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the

required compaction. The following table presents recommended levels of relative compaction for compacted fill:

<b>LOCATION OF FILL PLACEMENT</b>	<b>MINIMUM RELATIVE COMPACTION</b>
Beneath slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

**Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).**

Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

### **LIMITATIONS**

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

### **ADDITIONAL SERVICES**

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

The following attachments complete this report:

Log of Previous Boring (8203 Avalon Drive)

Log of Previous Boring (8211 Avalon Drive)

Log of Previous Boring (1997 Hart Crowser Study)

NovoLiq Output

Please contact us if you have any questions regarding this report, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.



11/18/2025

Marc R. McGinnis, P.E.  
Principal

cc: **Atera Homes** – Paul Monsef  
via email: [paul@aterahomes.com](mailto:paul@aterahomes.com)

MRM:kg

# **Log of Previous Boring**

**8203 Avalon Drive**

**Mercer Island, Washington**

# BORING LOG

## B-1

Approximate Ground Surface Elevation:

Soil Profile		Sample Data		Penetration Resistance (Blows/foot - ●)					Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)	
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)	Moisture Content (Percent - ■)						
					10	20	30	40	50	50+	
Dark brown, silty, fine to medium sand with gravel, roots, and organics (loose, moist) ( <b>Topsoil / Fill</b> )			5	5							
Gray, fine to medium sand with silt (loose, moist)		SP-SM	6	5							5
Gray, gravelly, fine to coarse sand with silt (loose, wet)		GP-GM	5	5							
			6	10							
Gray, silty, fine to medium sand with gravel and iron-oxide staining (loose, moist to wet)		SM	5	15							
Gray, fine to medium sand with gravel and silt (loose, moist to wet)		SP-SM	5	20							
Gray-brown, silty, fine to medium sand with gravel (loose, moist to wet)		SM	9	25							

### LEGEND

Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler	Solid PVC Pipe	Concrete	M	Moisture Content
Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler	Slotted PVC Pipe	Bentonite	A	Atterberg Limits
	Monument/ Cap to Piezometer	Native Soil	G	Grain-size Analysis
	Liquid Limit	Silica Sand	DS	Direct Shear
	Plastic Limit	Water Level	PP	Pocket Penetrometer Readings, tons/ft
			P	Sample Pushed
			T	Triaxial

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

Project Number 1579125	Vishy and Madhuri Remodel Seismic Evaluation Boring Log		<b>NELSON GEOTECHNICAL ASSOCIATES, INC</b>	No.	Date	Revision	By	CK
Figure 5				1	3/18/25	Original	FKS	KMS
Page 1 of 2								





Woodinville Office  
17311-135th Ave. NE, A-500  
Woodinville, WA 98072  
(425) 486-1669 / Fax: 481-2510

Wenatchee Office  
105 Palouse St.  
Wenatchee, WA 98801  
(509) 665-7696 / Fax: 665-7692













# BORING LOG

## B-1 (cont.)

Logged by: CTC on 1/14/13

Soil Profile			Sample Data		Penetration Resistance (Blows/foot - ●)					Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)	
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)	Moisture Content (Percent - ■)							
					10	20	30	40	50	50+		
Gray, gravelly, fine to coarse sand with silt (medium dense, moist to wet)		GP-GM	21	21		●						
Gray, silt with fine sand (medium dense, moist)		ML	22	35							GM	35
Boring terminated at 36.5 feet below existing grade on 3/17/2025. Groundwater seepage was encountered at 7.0 feet during drilling.												
				35								
				40								
				45								
				50								
				55								

**LEGEND**

- |   |  |  |
|---|--|--|
| <ul style="list-style-type: none"> <li> Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler</li> <li> Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler</li> </ul> | <ul style="list-style-type: none"> <li> Solid PVC Pipe</li> <li> Slotted PVC Pipe</li> <li> Monument/ Cap to Piezometer</li> <li> Liquid Limit</li> <li> Plastic Limit</li> </ul> | <ul style="list-style-type: none"> <li> Concrete</li> <li> Bentonite</li> <li> Native Soil</li> <li> Silica Sand</li> <li> Water Level</li> </ul> |
|   |  | <ul style="list-style-type: none"> <li>M Moisture Content</li> <li>A Atterberg Limits</li> <li>G Grain-size Analysis</li> <li>DS Direct Shear</li> <li>PP Pocket Penetrometer Readings, tons/ft</li> <li>P Sample Pushed</li> <li>T Triaxial</li> </ul>  |

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

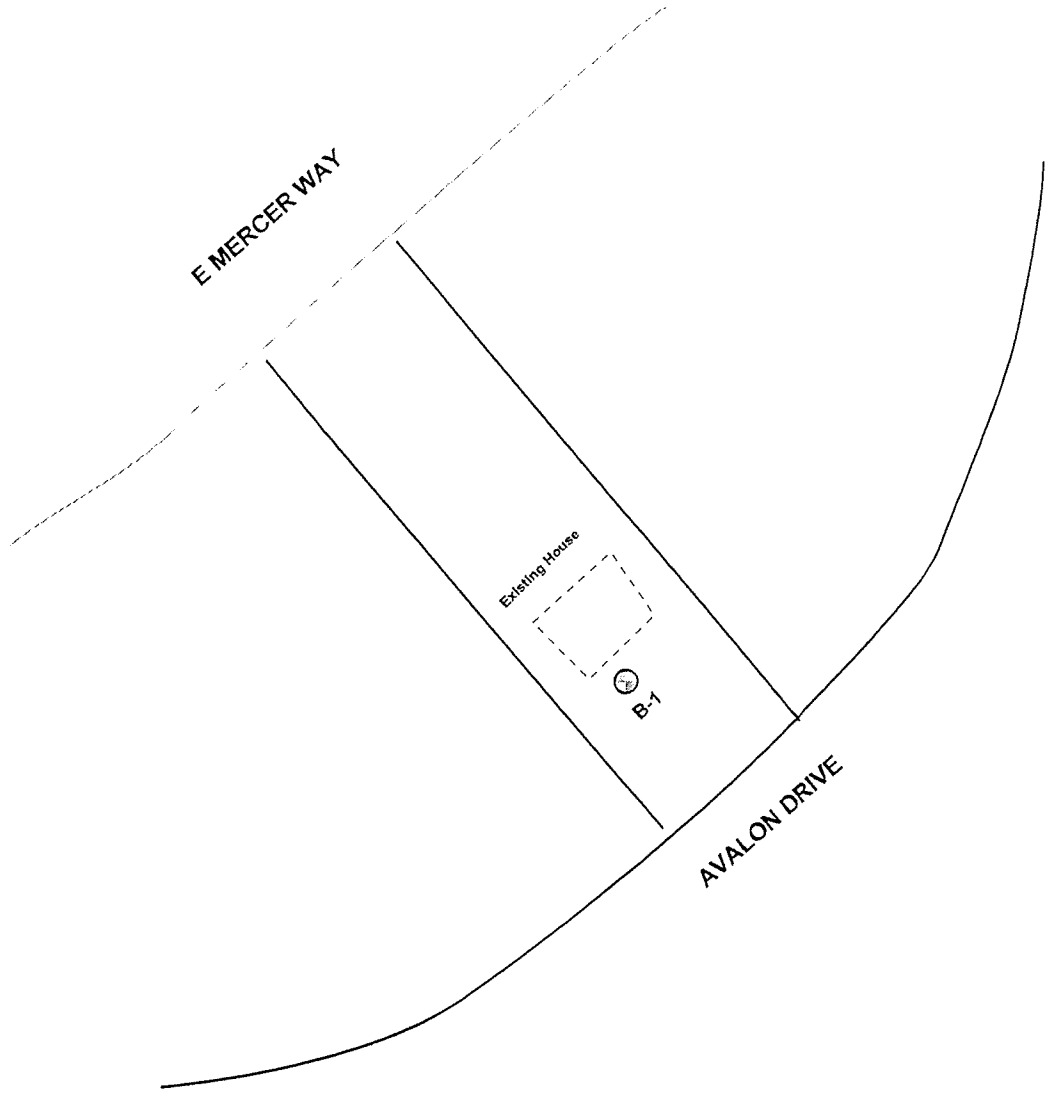
Project Number 1579125	Vishy and Madhuri Remodel Seismic Evaluation Boring Log		<b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510	Wenatchee Office 105 Palouse St Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692	No.	Date	Revision	By	CK
Figure 5					1	3/18/25	Original	FKS	KMS
Page 2 of 2									


C:\Users\Alexander\OneDrive - Nelson Geotechnical Associates, Inc\2025 NGA WEST\15791-25 Vishy and Madhuri Remodel Seismic Mercer Island\Drafting\BLOG -60FT.dwg

# **Log of Previous Boring**

**8211 Avalon Drive**

**Mercer Island, Washington**



LEGEND:  
 B-1



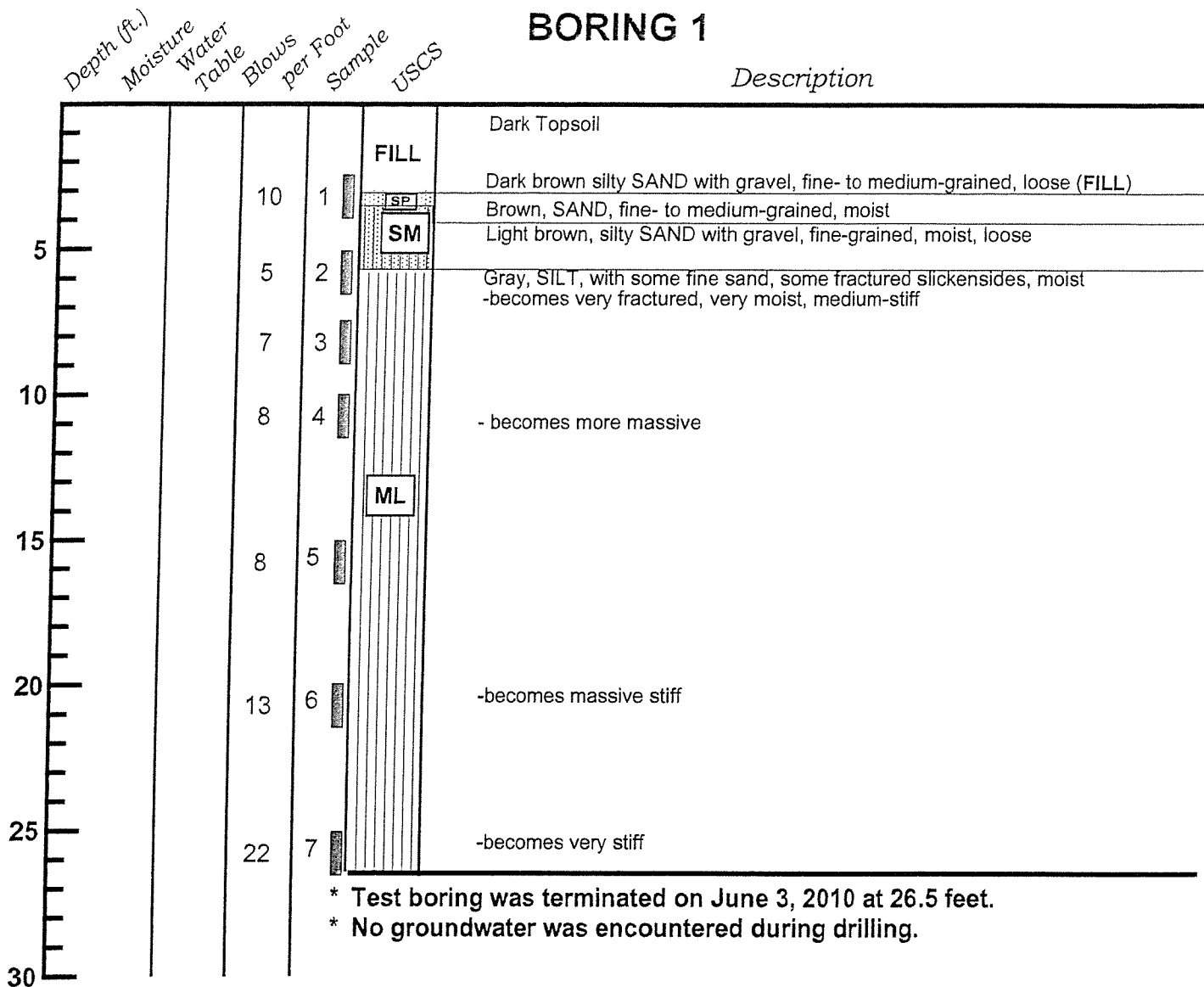

**GEOTECH**  
 CONSULTANTS, INC.

**SITE EXPLORATION PLAN**  
 8211 Avalon Drive  
 Seattle, Washington

Job No: 10141	Date: June 2010	No Scale	Plate: 2
------------------	--------------------	----------	-------------

# BORING 1

Description



\* Test boring was terminated on June 3, 2010 at 26.5 feet.  
 \* No groundwater was encountered during drilling.



**TEST BORING LOG**  
 8211 Avalon Dr  
 Seattle, Washington

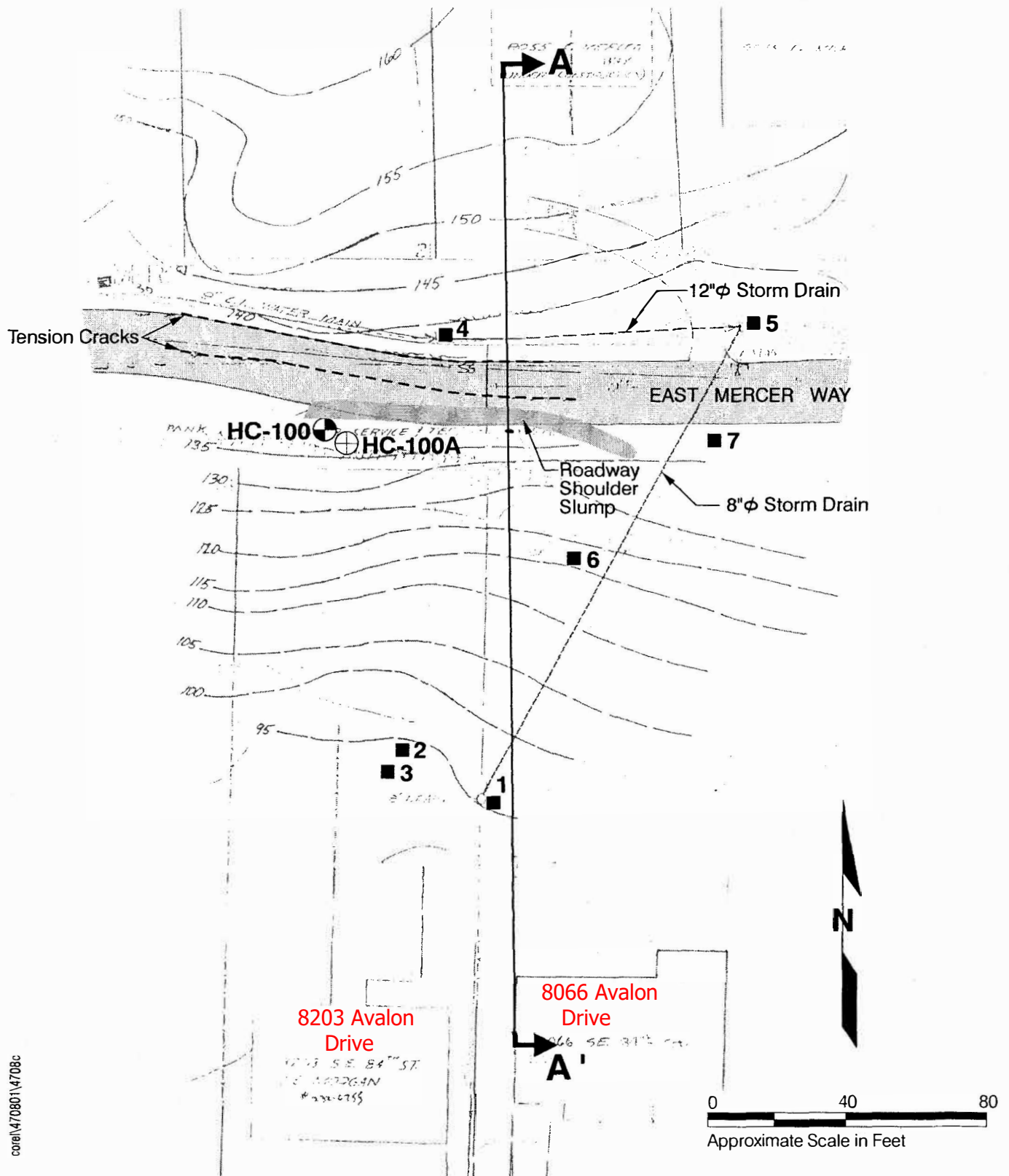
Job	Date:	Logged by:	Plate:
10141	June 2010	TAJ	3

# **Log of Previous Boring**



**1997 Hart Crowser Study**


**Mercer Island, Washington**

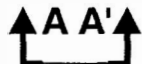
# Site and Exploration Plan



core\470801\4708c

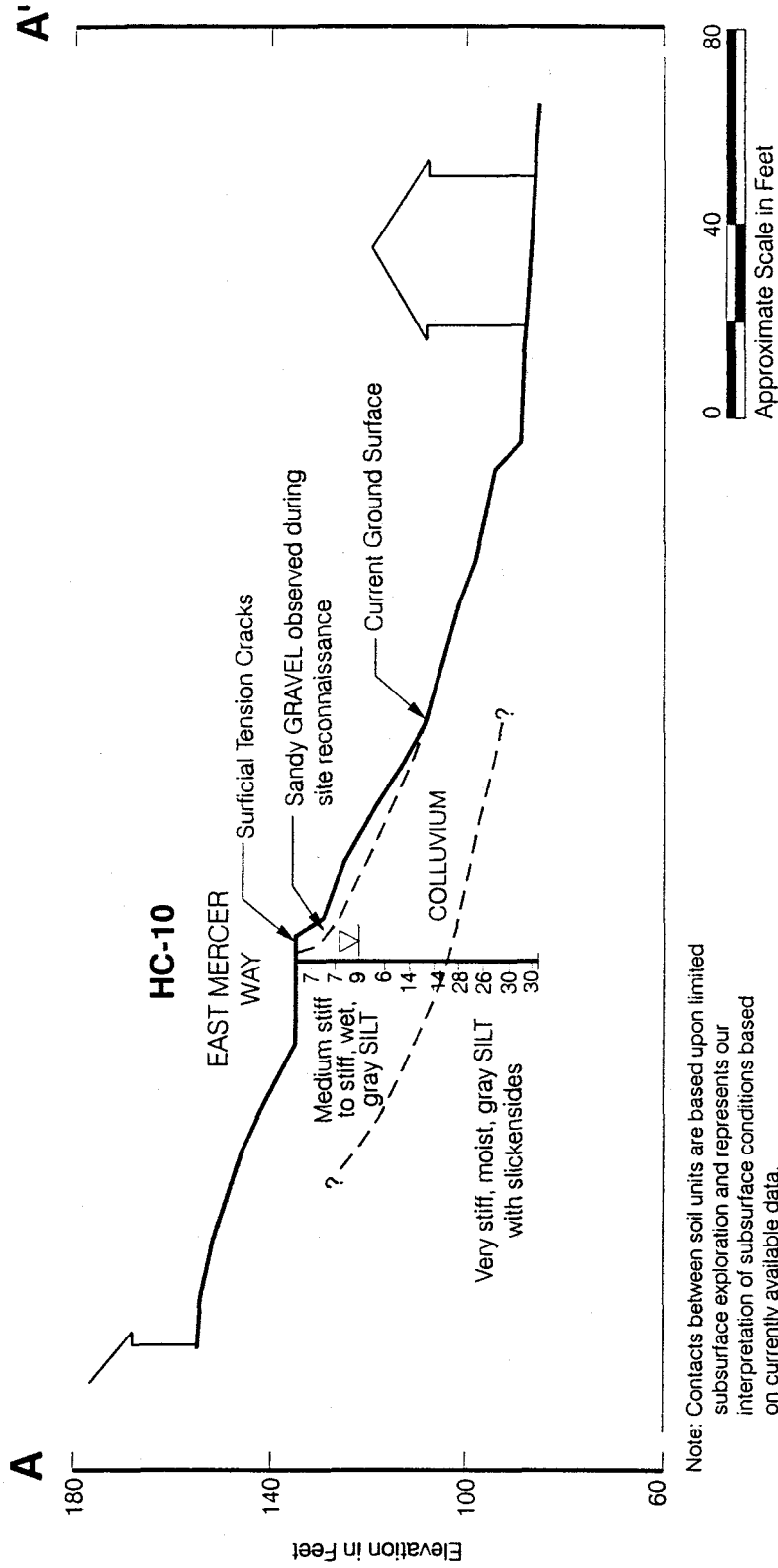
-  **HC-100** Boring Location and Number
-  **HC-100A** Monitoring Well Location and Number (See Field Report in Appendix A)

-  **1** Field Observation Location and Number

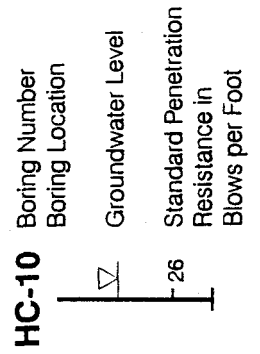
-  **A A'** Cross Section Location and Designation

# Generalized Subsurface Cross Section A-A'

## 8250 East Mercer Way

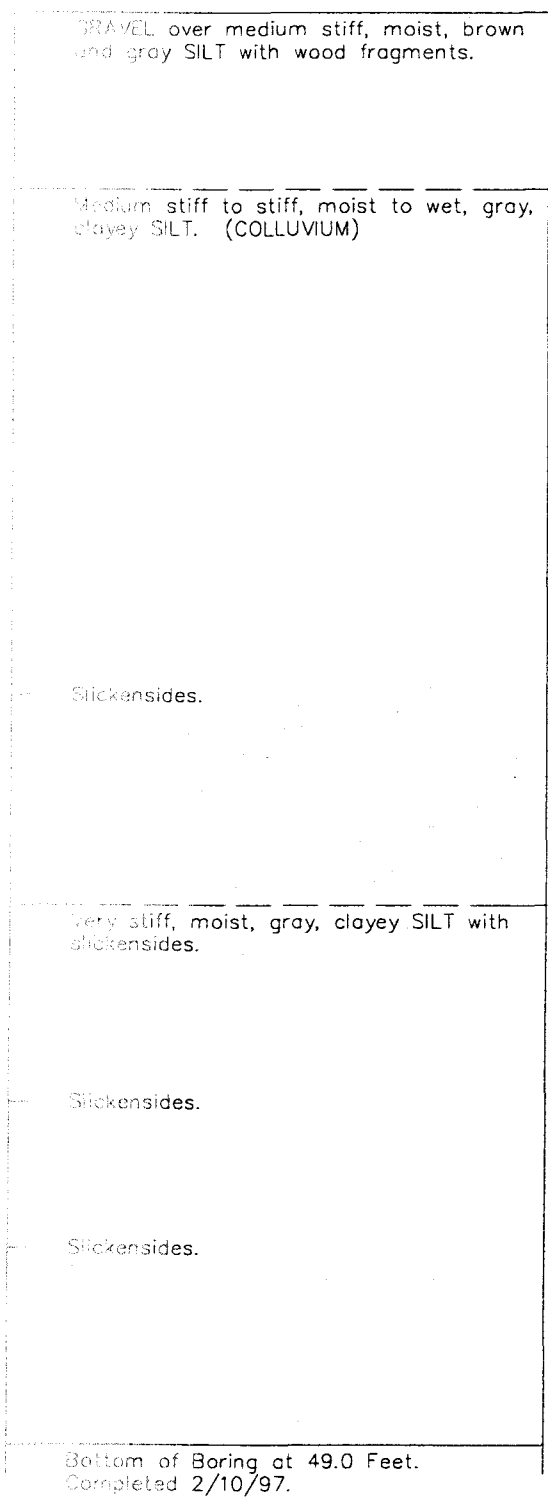


Note: Contacts between soil units are based upon limited subsurface exploration and represents our interpretation of subsurface conditions based on currently available data.



# Boring Log HC-100

## Soil Descriptions



Depth in Feet

HC-100A

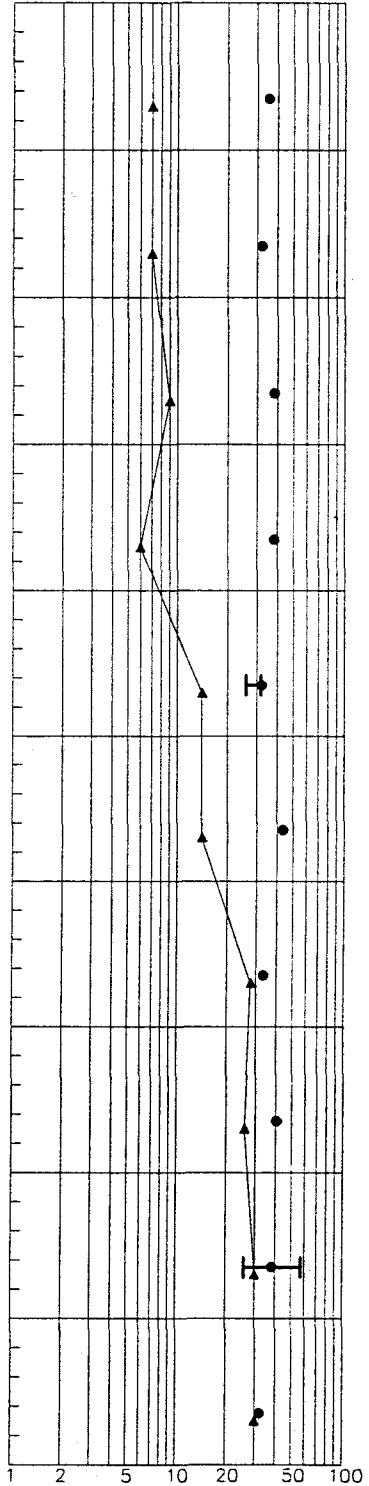
Sample

S-1  
S-2  
S-3  
S-4  
S-5  
S-6  
S-7  
S-8  
S-9  
S-10

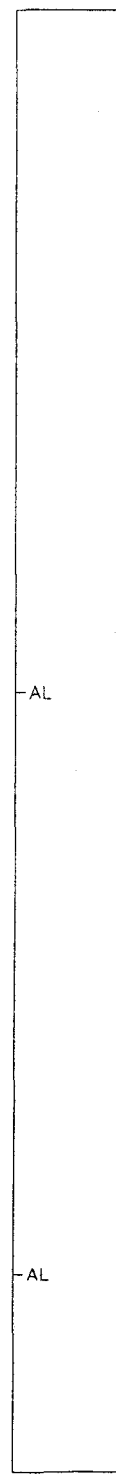
## STANDARD PENETRATION RESISTANCE

▲ Blows per Foot

1 2 5 10 20 50 100



## LAB TESTS



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

ACAD log

**NovoLiq Output**  
**8203 Avalon Drive**  
**Mercer Island, Washington**

# Soil Liquefaction Analysis Report

Project : Mohan Project  
 Project No. : 25340  
 Client : Whshwas Mohan  
 Site Address : 8203 Avalon Drive, Mercer Island, Wa 98040

Borehole : BH-1  
 Total Depth : 36 ft  
 Water Level : 7 ft  
 Calculated By : MKM  
 Reviewed By :

Geotech Consultants, Inc.

Table i : Input Data and Assumptions

Input Assumption	Setting
Field Test Type :	Standard Penetration Test (SPT)
Apply All Corrections to SPT?	True
Groundwater Level (ft) =	7
Earthquake Magnitude M =	7.1
Magnitude Scaling Factor (MSF) :	1.15 (Idriss, 1997 -NCEER)
Fines Content Correction :	(according to user settings)
Depth Reduction Factor (Rd) :	Idriss 1999, Golesorkhi 1989
Relative Density (Dr) Estimation :	Idriss & Boulanger, 2003
Site Topography :	Gently Sloped : 3 %
Ground Improvement Feature :	None
Peak Ground Acceleration PGA (g) =	0.687

Table ii : CRR Calculation Methods

CRR Formula	Selected?
NCEER Workshop (1997)	True
Boulanger & Idriss (2014)	True
Vancouver Task Force (2007)	False
Cetin et al. (2004)	False
Chinese Code	False
Seed et al. (1983)	False
Japanese Highway Bridge Code	False
Tokimatsu and Yoshimi (1983)	False
Shibata (1981)	False
Kokusho et al. (1983)	False

Table iv : Field Tests

Depth (ft)	SPT Blow Counts(N)
2.5	5
5	6
7.5	5
10	6
15	5
20	5
25	9
30	21
35	22

Table iii : Subsurface Soil Layers

Layer Thickness (ft)	Soil Type	Unit Weight (lb/ft <sup>3</sup> )	Fines Content (%)	D50 (mm)	Check Liquefaction	Su (ksf)
7	Sand	115	10	0.5	True	0
6	Sand	125	5	2	True	0
17	Sand	115	10	0.5	True	0
3	Sand	125	5	2	True	0
2	Silt	125	80	0.02	False	0

Table v : Post-Liquefaction Displacements

Type	Method	Movement (incht)
Lateral Spreading	Zhang & Robertson, 2004	387
Lateral Spreading	Faris, 2006	392
Lateral Spreading	Youd et al., 2002	127
Lateral Spreading	Barlett & Youd, 1992	147
Lateral Spreading	Hamada et al., 1986	129
Lateral Spreading	Youd & Perkins, 1987	LSI > 80 see details for LSI=90
Vertical Settlement	Ishihara & Yoshimine, 1992	11

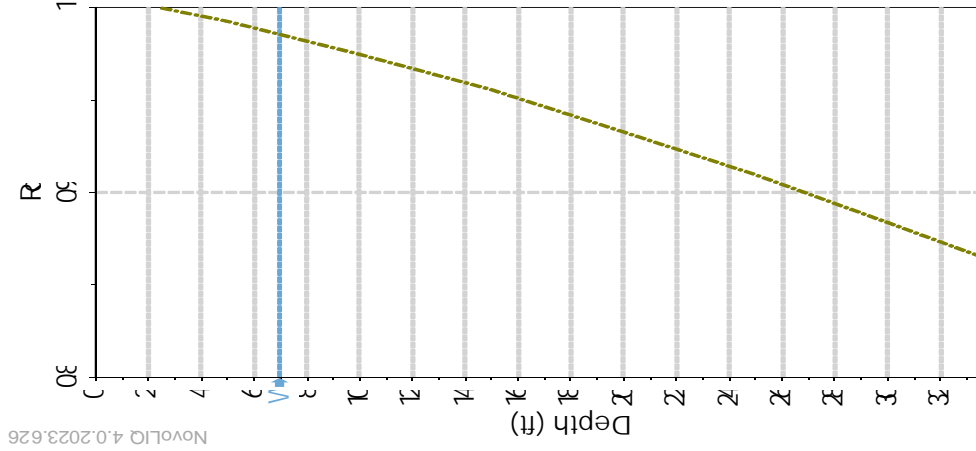
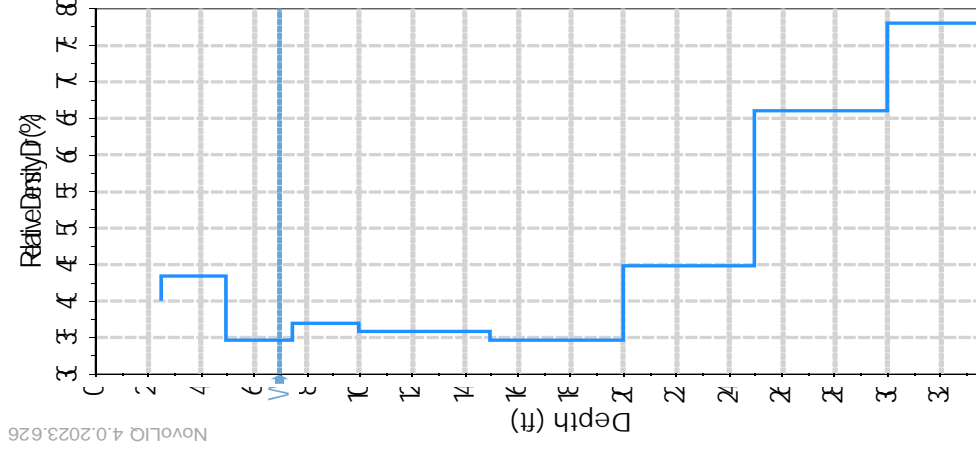
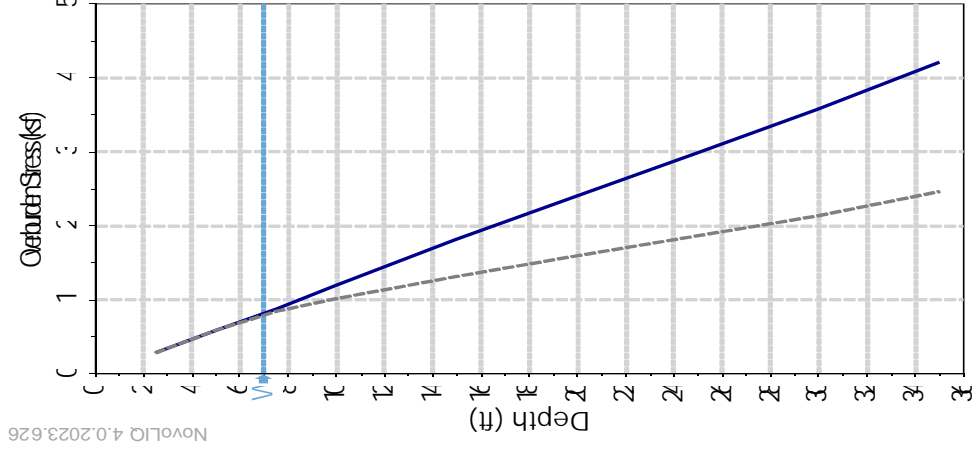
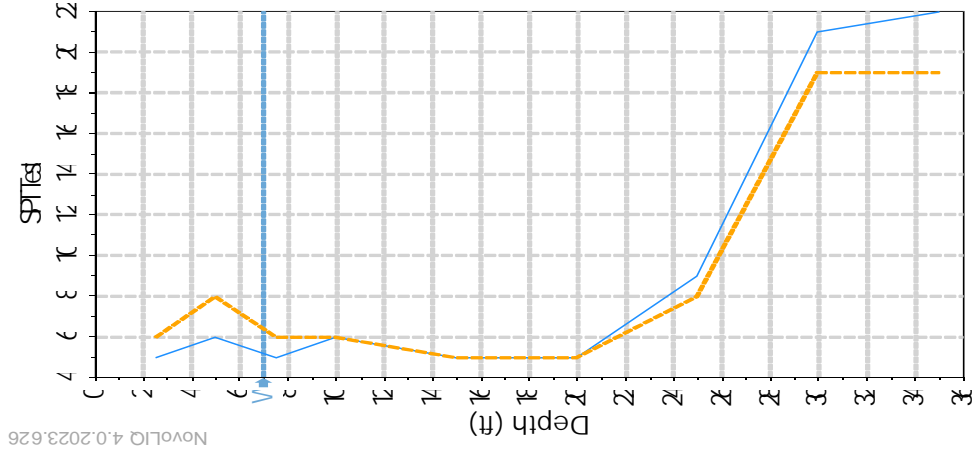
# Soil Liquefaction Analysis Report

Project : Mohan Project  
 Project No. : 25340  
 Client : Whshwas Mohan  
 Site Address : 8203 Avalon Drive, Mercer Island, Wa 98040

Borehole : BH-1  
 Total Depth : 36 ft  
 Water Level : 7 ft  
 Calculated By : MKM

Geotech Consultants, Inc.

Reviewed By :



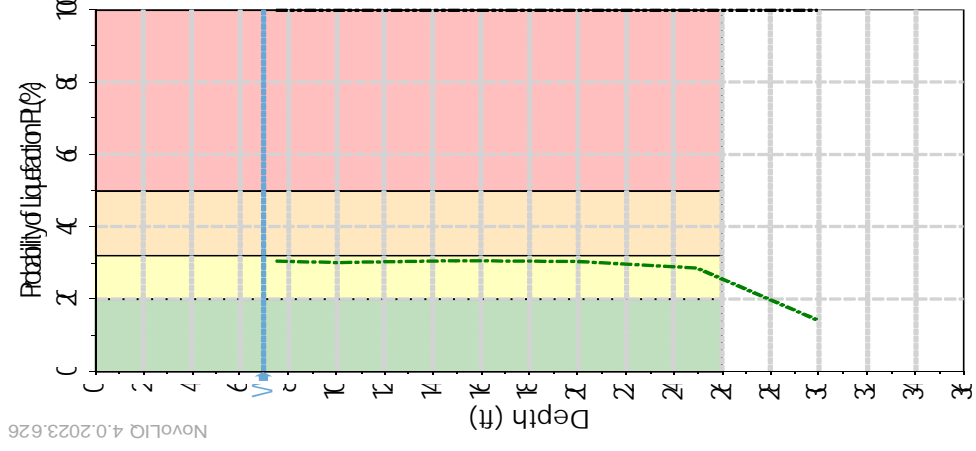
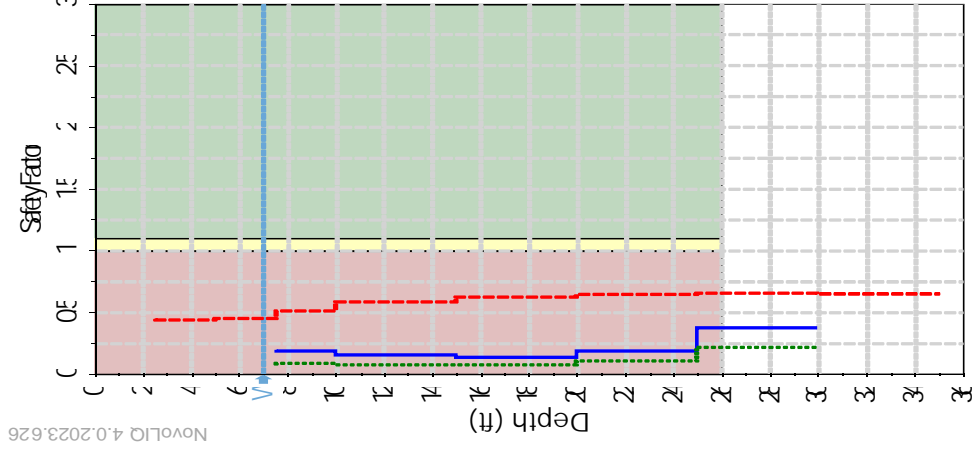
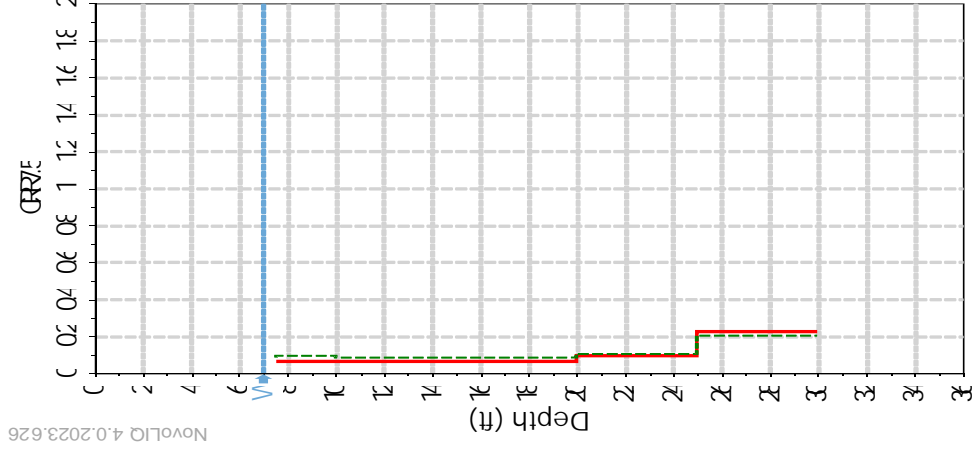
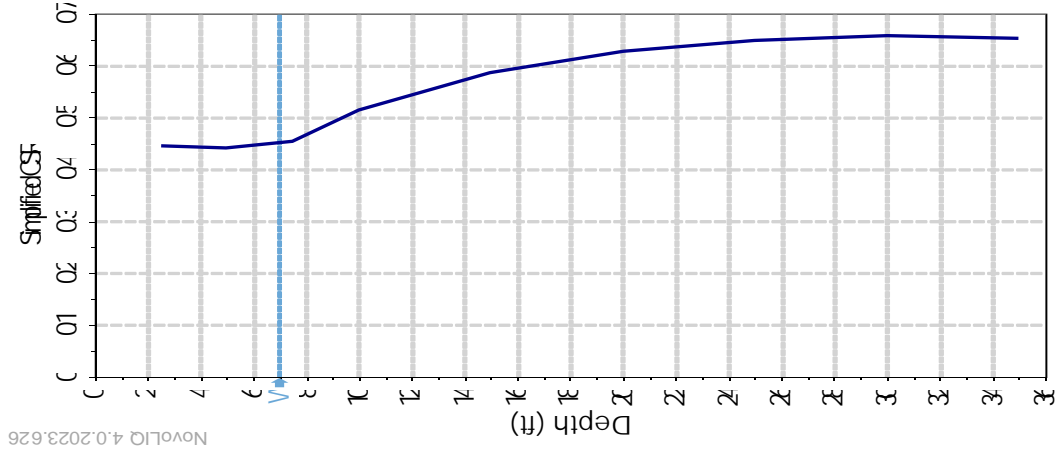
# Soil Liquefaction Analysis Report

Project : Mohan Project  
 Project No. : 25340  
 Client : Whshwas Mohan  
 Site Address : 8203 Avalon Drive, Mercer Island, Wa 98040

Borehole : BH-1  
 Total Depth : 36 ft  
 Water Level : 7 ft  
 Calculated By : MKM

Geotech Consultants, Inc.

Reviewed By :



— NEER Whitlock (1997)   
 - - - Boulanger & Idiss (2014)   
 — Safety Factor   
 - - - Simplified CSF   
 - - - CRR7E (ae)   
 - - - Youd & Ndtje   
 - - - Gained (2014)

# Soil Liquefaction Analysis Report

Project : Mohan Project  
 Project No. : 25340  
 Client : Whshwas Mohan  
 Site Address : 8203 Avalon Drive, Mercer Island, Wa 98040

Borehole : BH-1  
 Total Depth : 36 ft  
 Water Level : 7 ft  
 Calculated By : MKM

Geotech Consultants, Inc.

Reviewed By :

