



**NELSON GEOTECHNICAL
ASSOCIATES. INC.**

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April 24, 2025

Jocelyn Gonan
Drift Interiors and Architecture
VIA Email: staff@drift-ia.com

**Geotechnical Engineering Evaluation – REVISED
Vishy and Madhuri Remodel Seismic Evaluation
8203 Avalon Drive
Mercer Island, Washington
NGA File No. 1579125**

Dear Jocelyn:

This report summarizes our opinions and recommendations regarding the proposed residence remodel project located at **8203 Avalon Drive on Mercer Island, Washington**. The parcel number for the property is **0321100090**. Our services were completed in general accordance with the proposal signed by you on February 21, 2025.

The site is currently occupied by a single-family residence and garage in the central portion. The property is bordered by Avalon Drive to the southeast, East Mercer Way to the northwest, and neighboring residential properties on all other sides. Topographically, a majority of the site is relatively flat to gently sloping and contains a moderate to steep slope within the northwestern portion of the site. We understand that plans for development include remodeling the existing residence and garage. The City of Mercer Island has requested a geotechnical report to address the mapped seismic hazard that covers the property.

We explored the subsurface conditions within the site with a borehole utilizing a limited-access drill. Our boring extended to a depth of 36.5 feet below the existing ground surface. Our exploration indicated that the site was underlain by loose colluvium and competent native Lawton-Clay deposits at a depth of 35.0 feet below the existing ground surface. We encountered groundwater at 7.0 feet in our borehole exploration on March 17, 2025.

It is our opinion that the planned remodel is feasible from a geotechnical standpoint, provided that our recommendations are incorporated into the design and construction of this project. Specifically, the report includes recommendations for earthwork, foundations, structural fill placement, erosion control, and drainage.

We should be retained to review and comment on final development plans, provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

It has been a pleasure to provide service to you on this project. Please contact us if you have any questions regarding this report or require further information.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.



Khaled M. Shawish, PE
Principal

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Geotechnical Engineering Evaluation – **REVISED**
Vishy and Madhuri Remodel Seismic Evaluation
8203 Avalon Drive
Mercer Island, Washington

INTRODUCTION

This report summarizes our opinions and recommendations regarding the proposed residence remodel project located at **8203 Avalon Drive on Mercer Island, Washington**. The parcel number for the property is **0321100090**.

The site is currently occupied by a single-family residence and garage in the central portion. The property is bordered by Avalon Drive to the southeast, East Mercer Way to the northwest, and neighboring residential properties on all other sides. Topographically, a majority of the site is relatively level to gently sloping and contains a moderate to steep slope within the northwestern portion of the site. We understand that plans for development include remodeling the existing residence and garage. We also understand that the majority of the existing foundations will remain. The City of Mercer Island has requested a geotechnical report to address the mapped seismic hazard that covers the property. The proposed site development is presented on the attached Site Plan in Figure 2.

We have been provided with plans for the proposed remodel titled *“Vishy and Madhuri Remodel,” dated January 27, 2025, prepared by Drift Interiors and Architecture.*

SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions within the vicinity of the planned development and provide geotechnical recommendations for site development.

Specifically, our scope of services included the following:

1. Reviewing available soil and geologic maps of the area as well as other relevant geotechnical information, as provided.
2. Exploring the subsurface soil and groundwater conditions within the proposed development areas with one, 30-foot-deep, geotechnical borehole using a limited-access drill rig. Drilling services were subcontracted by NGA.
3. Mapping the conditions on the site slopes using shallow, hand-tool explorations where necessary to construct geological cross sections and qualitatively evaluate slope stability.
4. Conducting seismic analysis for the property in accordance with the City of Mercer Island requirements.

5. Addressing the mapped geologic hazards and provide recommendations for site development to lessen the impact to the critical areas.
6. Performing laboratory grain-size sieve analysis on soil samples, as necessary.
7. Providing general recommendations for site drainage and erosion control.
8. Documenting the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The property is rectangular in shape and covers 0.42 acres in area. It is currently occupied by a single-family residence and garage in the central portion. The property is bordered by Avalon Drive to the southeast, East Mercer Way to the northwest, and neighboring residential properties on all other sides. The ground surface throughout a majority of the site is relatively level to gently sloping and contains a moderate to steep slope within the northwestern portion of the site that ascends towards East Mercer Way at gradients in the range of 19 to 34 degrees (42 to 75.6 percent slopes), as shown on Cross-Section A-A', presented as Figure 3. Vegetation throughout the site generally consists of grass yard areas, landscaping plants, ivy, and young to mature trees. Surface water was not observed within the site or emitting from the slope during our visit on March 17, 2025.

Subsurface Conditions

Geology: The geologic units for this site are shown on [Geologic Map of Mercer Island, Washington](#), by Troost, K.G. and Wisher, A.P. (GeoMapNW and the City of Mercer Island, 2006). The site is mapped as Lawton-Clay (Qvlc). Lawton-Clay is generally described as laminated to massive silt, clayey silt, and silty, clay. Our explorations generally encountered interbedded layers of silty, fine to medium sand with gravel and gravelly, fine to coarse sand with carrying amounts of silt, which we interpreted as colluvium from the northwestern hillside. Underlying the colluvium, we encountered silt with fine sand, consistent with the description of Lawton-Clay at depth.

Explorations: The subsurface conditions within the site were explored on March 17, 2025, by monitoring the drilling of a geotechnical boring that extended to a depth of 36.5 feet below the existing ground surface. The approximate location of the explorations is shown on the Site Plan in Figure 2. A geologist from Nelson Geotechnical Associates, Inc. (NGA) was present during the explorations, examined the soils and geologic conditions encountered, obtained samples of the soil, and maintained logs of the explorations.

A Standard Penetration Test (SPT) was performed on each of the samples during drilling to document soil density at depth. The SPT consists of driving a 2-inch outer-diameter, split-spoon sampler 18 inches using a 140-pound hammer with a drop of 30 inches. The number of blows required to drive the sampler the final 12 inches is referred to as the “N” value and is presented on the boring logs. The N value is used to evaluate the strength and density of the deposit. The soils were visually classified in general accordance with the Unified Soil Classification System, presented in Figure 4. The logs of our explorations are presented in Figure 5. We present a brief description of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the exploration logs should be reviewed.

At the surface of our boring, we encountered loose, dark brown, silty, fine to medium sand with gravel, roots, and organics, which we interpreted as surficial topsoil and/or undocumented fill. Underlying the surficial soils, we encountered loose, interbedded layers of silty, fine to medium sand with gravel and gravelly, fine to coarse sand with varying amounts of silt, which we interpreted as colluvium from the northwestern hillside. Underlying the loose colluvium, we encountered medium dense or better silt with fine sand, which we interpreted as competent native Lawton-Clay. Our boring was completed within the native Lawton-Clay at a depth of 36.5 feet below the existing ground surface.

Hydrogeologic Conditions

Groundwater was encountered in the boring at a depth of 7.0 feet. We interpret this water as the regional groundwater table associated with Lake Washington. We expect the groundwater level fluctuates with tidal movement.

SENSITIVE AREA EVALUATION

Seismic Hazard

We reviewed the **2021 International Building Code (IBC)** for seismic site classification for this project. Since competent Lawton-Clay deposits were encountered at depth, the site conditions best fit the IBC description for **Site Class D**.

Table 1 below provides seismic design parameters for the site that are in conformance with the 2021 IBC, which specifies a design earthquake having a two percent probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – ASCE 7-16 Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g) S_s	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
D	1.460	0.503	1.000	Null	0.974	Null

The spectral response accelerations were obtained from the **OSHPD Seismic Design Maps website (ASCE 7-2016 data)** for the project latitude and longitude.

Hazards associated with seismic activity include liquefaction potential and amplification of ground motion. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. The type of soil most susceptible to liquefaction during an earthquake is a saturated loose to medium dense sand deposit. A loose saturated sand deposit, when subjected to vibration, tends to compact and decrease in volume. If the sand deposit does not drain, the pore water pressure increases. If the pore water pressure is allowed to build up by continued vibration, a “quick” condition can be reached when the pore water pressure equals the effective overburden pressure at that depth. Under this condition, the sand temporarily loses its shear strength and acts as liquid.

We used the LiquefyPro liquefaction analysis software to evaluate the liquefaction potential and the possible liquefaction-induced settlement for the site soil and groundwater conditions based on the subsurface conditions encountered in our exploration. The **Maximum Considered Earthquake (MCE)** was selected in accordance with the **U.S. Geological Survey (USGS) Earthquake Hazards Program website**. For this analysis, a peak horizontal ground surface acceleration of 0.687g was used. Our analysis assumed a groundwater depth of 7.0 feet during the earthquake. The analysis was performed using the subsurface information from Boring B-1. The settlement resulting from this seismic event within upper loose colluvial soils is estimated to be on the order of 15 inches. A plot of the liquefaction analyses showing the factor of safety and estimated settlement is presented in Figure 10.

Erosion Hazard

The criteria used for determination of the erosion hazard for affected areas include soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types, which are related to the underlying geologic soil units. The Natural Resources Conservation Service (NRCS) of the Mercer Island area was reviewed. The soil within the site is mapped as Kitsap silt loam, 15 to 30 percent slopes. The erosion hazard for this soil is listed as severe for exposed soils.

Based on our experience in the area and the material encountered, it is our opinion that the erosion hazard for site soils should be moderate for disturbed soils and slight to moderate in areas where the site is not disturbed, and vegetation remains in place.

Landslide Hazard/Slope Stability

The criteria used for evaluation of landslide hazards include soil type, slope gradient, and groundwater conditions. We understand the remodel will utilize a majority of the existing foundations and will maintain the existing setbacks from the northwestern slope. The northwestern slope descends from East Mercer way at gradients in the range of 19 to 34 degrees (42 to 75.6 percent slopes), as shown on Cross-Section A-A', presented as Figure 3. The overall vertical relief of the northwestern slope is approximately 40 feet. We did not observe surface water within the site or emitting from the slope during our visit on March 17, 2025.

We did not observe evidence of recent significant slope instability within the property during our investigation, such as indications of recent deep-seated landsliding, erosional activity, shallow sloughing events, or groundwater seepage emitting from the face of the slopes. From our LiDAR imagery review, we observed that the northwestern has experienced surficial sloughing; however, we did not observe evidence of historical deep-seated landslides below East Mercer Way.

It should be noted that the hillside to the north of East Mercer Way contains an ancient landslide scarp has been identified. This scarp and overall steep slope area, when viewed on LiDAR imagery, depicts very steep/sharp features, generally indicating more geologically active conditions. We anticipate most of the chronic stability issues that may have occurred on this mapped scarp/slope area occurred as a reaction to post-glacial conditions some 10- to 14- thousand years ago and have since reached a more stable configuration with respect to deep-seated failures.

Based on our exploration and review of geotechnical explorations within the vicinity of the site, we interpret that the core of the site slopes consists primarily of medium dense or better native Lawton-Clay deposits. Inclinations of up to 34 degrees (75.6 percent) on the slopes within the property indicate high internal strength within the underlying soils. Relatively shallow sloughing failures as well as surficial erosion are natural processes and should be expected on these slopes during extreme weather conditions. It is our opinion that while there is potential for erosion, soil creep, and shallow failures within the loose surficial soils on the steep slopes, there is not a significant potential for deep-seated slope failure under current site conditions. Proper site grading and drainage as recommended in this report should help maintain current stability conditions. It is critical that no cuts over 3.0 feet are attempted into the toe of the slope unless such cuts are shored from the top down.

LABORATORY ANALYSIS

Grain-size Analysis: We performed grain-size distribution analyses on four samples from on samples from Boring One at depths of 7.5, 15.0, 20.0, and 35.0 feet. The results of the grain-size analysis are presented in Figures 6 through 9.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the planned residence remodel is feasible from a geotechnical standpoint and should not impact the northwestern slope. It is also our opinion that the soils that form the core of the site slopes should be stable with respect to deep-seated earth movements due to their inherent strength and slope geometry. There is, however, a significant potential for shallow sloughing and erosional events to occur on the steeper portions of the slopes, especially during extreme weather conditions or seismic activity. Our subsurface exploration found that the site was underlain by loose colluvium and medium dense or better Lawton-Clay at a depth of 35.0 feet below the existing ground surface. The upper loose colluvium has potential to experience settlement exceeding typical construction tolerances if subjected to foundation loads under static conditions and could experience up to 15 inches of liquefaction-induced settlement following a large earthquake.

To mitigate the potential adverse effects to the existing structure from the potential liquefaction-induced settlement, as well as static settlement within the upper loose colluvial deposits, we recommend that all of the residence's existing and new foundations be supported on a deep foundation system consisting of driven pin piles. Recommendations for the deep foundation design and installation are included in the **Deep Foundation Support** subsection of this report. We understand that the load on the garage foundation is not being increased and no one will be residing in the garage. Therefore, it is our opinion that the existing garage foundation does not need to be underpinned. However, the property owner should continuously monitor the garage foundation for signs of cracking and settlement.

The soils encountered on this site are considered moisture-sensitive and will disturb when wet. We recommend that construction take place during the drier summer months, if possible. If construction is to take place during wet weather, the soils may disturb, and additional expenses and delays may be expected due to the wet conditions. Additional expenses could include the need for placing a blanket of rock spalls to protect exposed subgrades and construction traffic areas. We do not recommend using the on-site soil as structural fill.

Erosion Control Measures

The erosion hazard for the on-site soils is interpreted to be moderate for exposed soils, but actual erosion potential will be dependent on how the site is graded and how water is allowed to concentrate. Best Management Practices (BMPs) should be used to control erosion. Areas disturbed during construction should be protected from erosion. Erosion control measures may include diverting surface water away from the stripped or disturbed areas. Silt fences and/or straw bales should be erected to prevent muddy water from leaving the site. Disturbed areas should be planted as soon as practical and the vegetation should be maintained until it is established. The erosion potential of areas not disturbed should be low to moderate.

Site Preparation and Grading

Plans for site grading should be devised such that cuts and fills are kept to a minimum if possible. Site preparation and grading should generally consist of excavating any new structure footprints to desired subgrade and installing the deep foundation elements for support of all foundation elements and slab-on-grade. No cuts should be attempted into the toe of the steep slope unless engineered shoring systems are planned.

Alternative site grading techniques might be necessary if groundwater or inclement weather conditions are encountered. These could include using large excavators equipped with wide tracks and a smooth bucket to complete site grading and covering exposed subgrade with a layer of crushed rock for protection. In wet conditions, it may be necessary to cover the exposed subgrade with a layer of crushed rock as soon as it is exposed to protect the extremely moisture-sensitive soils from disturbance by machine or foot traffic during construction. The prepared subgrade should be protected from construction traffic and surface water should be diverted around areas of prepared subgrade.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since they are continuously at the job site, able to observe the subsurface materials and groundwater conditions encountered, and able to monitor the nature and condition of the cut slopes. The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 2.0 Horizontal to 1.0 Vertical (2.0H:1V). Permanent cuts should have an inclination no steeper than 3H:1V. If significant groundwater seepage or surface water flow were encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. Protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations.

Deep Foundation Support

From our liquefaction analysis of the on-site native soils, we expect settlement within the upper loose colluvial soils to be in the order of 15 inches during a major seismic event. A plot of the liquefaction analyses showing the factor of safety and estimated settlement is presented in Figure 10.

The proposed structures, including any existing or new foundations and any new slab-on-grade, should be supported on either 2-, 3-, or 4-inch driven pin piles to transfer foundation loads through the upper loose colluvial soils down to the underlying native competent bearing materials, interpreted to underlie the site at depth. Due to the nature of the colluvium, debris may be encountered during pile installation. There should be contingencies in the budget and design for additional/relocated piles to replace piles that may be obstructed by debris. We also recommend that excavation equipment be available on-site during pile installation so that shallow obstructions can be removed from the planned pile locations. Supporting the existing foundation on piles larger than 2-inches in diameter may prove challenging.

For **2-inch diameter pipe piles** driven to refusal using a hand-held, 140-pound jackhammer, we recommend a design axial compression capacity of three tons for each pile. Refusal criterion for 2-inch steel piles driven using a 140-pound hammer is defined as less than 1-inch of movement per 60 seconds of driving.

For **3-inch diameter pipe piles** driven to refusal using a track-hoe mounted, 550-pound jackhammer, we recommend a design axial compression capacity of six tons for each pile. Refusal criterion for 3-inch steel piles driven using a 550-pound hammer is defined as less than 1-inch of movement per 15 seconds of driving.

For **4-inch diameter pipe piles** driven to refusal using a track-hoe mounted, 850-pound jackhammer, we recommend a design axial compression capacity of ten tons for each pile. Refusal criterion for 4-inch steel piles driven using a 850-pound hammer is defined as less than 1-inch of movement per 15 seconds of driving.

The piles should advance a minimum of 35 feet below current grade to extend through the upper loose soils and into the more competent Lawton-Clay deposits at depth. Piles installed at the basement foundation elevation should be embedded a minimum of 30 feet. Piles that do not meet this minimum embedment criterion should be rejected, and replacement piles should be driven after consulting with the structural engineer on the new pile locations. The piles could consist of 2-, 3-, or 4-inch diameter galvanized extra strong (**Schedule 80**) steel pipe sections.

Due to the varying nature of the colluvium soils encountered within the exploration, we recommend that two or more “test” piles be installed to verify design parameters and estimate an approximate pile refusal depth for budgeting purposes. The piles should be spaced a minimum of three feet apart to avoid a grouping effect on the piles. Due to the relatively small slenderness ratio of pin piles, maintaining pin pile confinement and lateral support is essential in preventing pile buckling. Pin piles should be suitably embedded into the reinforced concrete. The structural engineer should design the connections of the piles to the foundations.

Vertically driven pin piles do not provide meaningful lateral capacity. Due to the rigid pile support, friction between the foundation and subgrade soil should not be considered as resisting lateral pressures on this structure. We recommend that all lateral loads be resisted on battered pin piles and/or passive resistance on the below-grade portions of the foundations. The upper foot of soil should be neglected when calculating the passive resistance. We recommend using an equivalent fluid density of 150 pcf for calculating the passive resistance.

Debris Protection

To reduce the risk of the existing garage structure, it may be prudent to design and install a 4-foot-tall debris fence along the toe of the steep slope to protect the garage from potential debris flow on the slope during severe weather or seismic activity. The debris fence should be designed for 100 pcf triangular pressure distribution to model the potential impact from debris flows. There should be a mechanism built into the system to allow for the removal of any accumulated debris. No excavations whatsoever should be attempted into the toe of the slope.

Slab-on-Grade

It is our opinion that the existing slab-on-grade could remain; however, it should be noted that the existing slab on grade is expected to experience some settlement and cracking over the lifetime of the residence due to the loose soils underlying the slab. The property owner should monitor the existing slab on an ongoing basis to look for signs of increased cracking or settlement. Depending on the total allowance for cracking, the slab on grade could be removed and replaced with a new slab on grade or by a structural slab. For a new slab on grade, we recommend over excavating the loose subgrade below the slab by 18-inches and then replacing the loose material with 1.25-inch clean crushed rock. The slab-on-grade should be reinforced with additional rebar and concrete to further reduce the risk of cracking. To further mitigate the risk of slab cracking, the new slab-on-grade could be designed as a structural slab and be fully supported on pin piles, as recommended for the foundation lines. In any case, all new slab-on-grade should be underlain by a minimum of six inches of free-draining gravel to act as a capillary break layer. The capillary break layer should be overlain by a vapor barrier consisting of a minimum 6-mil plastic sheeting.

Site Drainage

Surface Drainage: The finished ground surface should be graded such that runoff is directed to an approved stormwater collection system. Water should not be allowed to collect in any areas where footings, slabs, or pavements are to be constructed. Final site grades should allow for drainage away from the structures. We suggest that the finished ground be sloped at a minimum gradient of three percent, for a distance of at least 10 feet away from the structures. Surface water should be collected by permanent catch basins and drain lines and be routed into an appropriate discharge system.

Subsurface Drainage: If groundwater is encountered during construction, we recommend that the contractor slope the bottom of the excavation and collect the water into ditches and small sump pits where the water can be pumped out of the excavation and routed into an approved outlet. We recommend that the residence down spouts and footing drains be tightlined to a discharge system that routes the water to the approved drainfield located off site. We recommend the use of footing drains around structures. Footing drains should be installed at least one foot below planned finished floor elevation. The drains should consist of a minimum 4-inch-diameter, rigid, slotted or perforated, PVC pipe surrounded by free-draining material wrapped in a filter fabric. We recommend that the free-draining material consist of an 18-inch-wide zone of clean (less than three-percent fines), granular material placed along the back of walls.

Washed rock is an acceptable drain material, or drainage composite may be used instead. The free-draining material should extend up the wall to one foot below the finished surface. The top foot of soil should consist of low permeability soil placed over plastic sheeting or building paper to minimize the migration of surface water or silt into the footing drain. Footing drains should discharge into tightlines leading to an approved collection and discharge point with convenient cleanouts to prolong the useful life of the drains. Roof drains should not be connected to wall or footing drains.

USE OF THIS REPORT

NGA has prepared this report for **Jocelyn Gonan**, and associated agents, for use in the planning and design of the remodel on this site only. The scope of our work does not include services related to construction safety precautions and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explorations and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule.

We recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

Within the limitations of scope, schedule and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

O-O-O

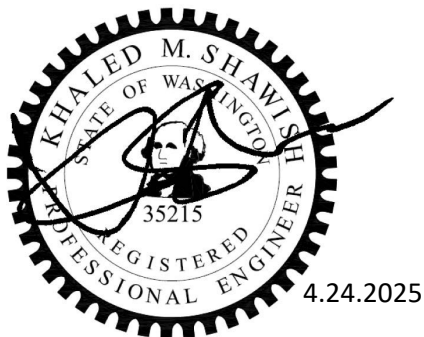
We appreciate the opportunity to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.



Faith K. Stelter
Staff Geologist II



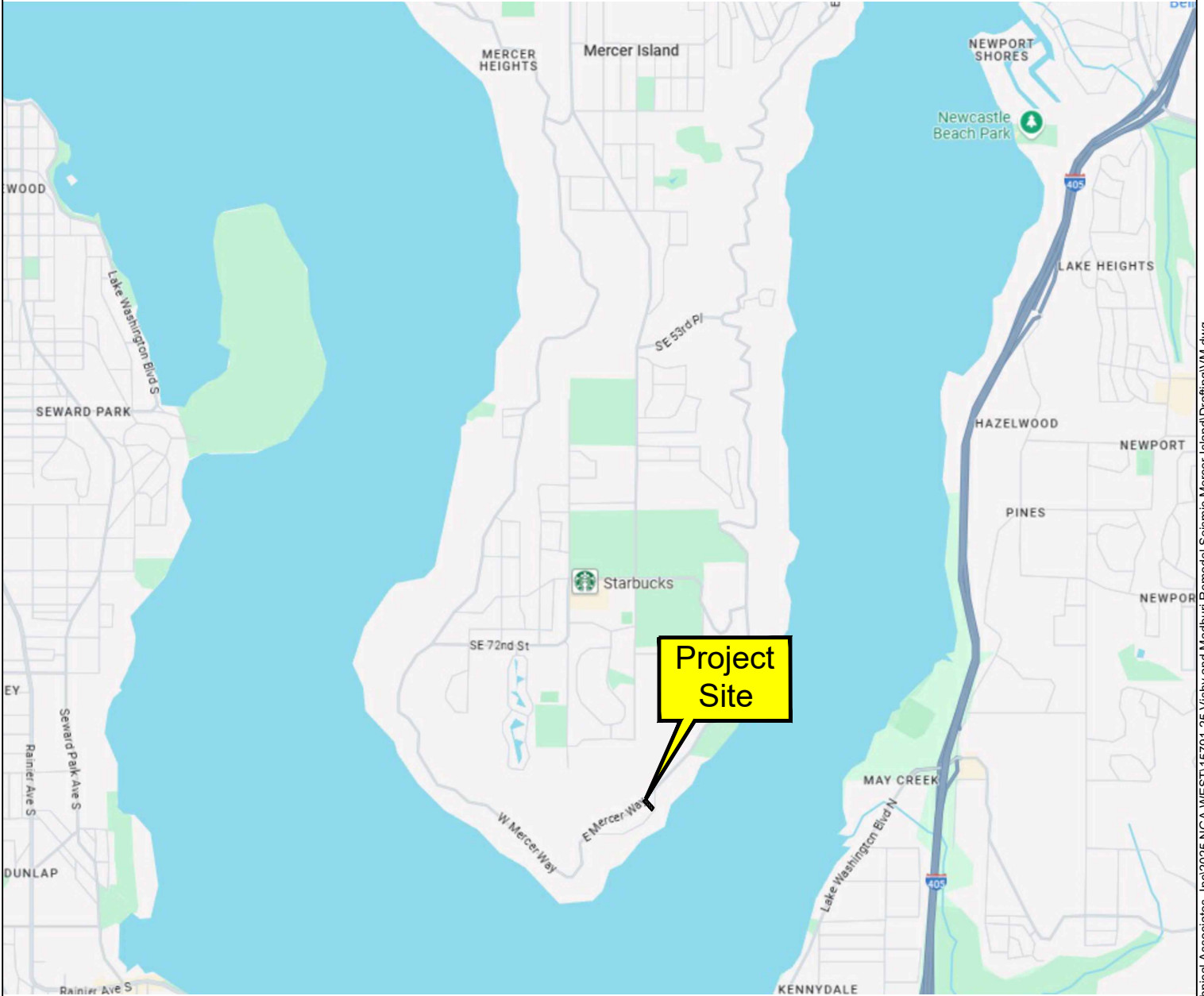
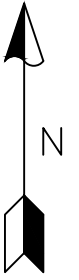
Khaled M. Shawish, PE
Principal

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Ten Figures Attached

VICINITY MAP

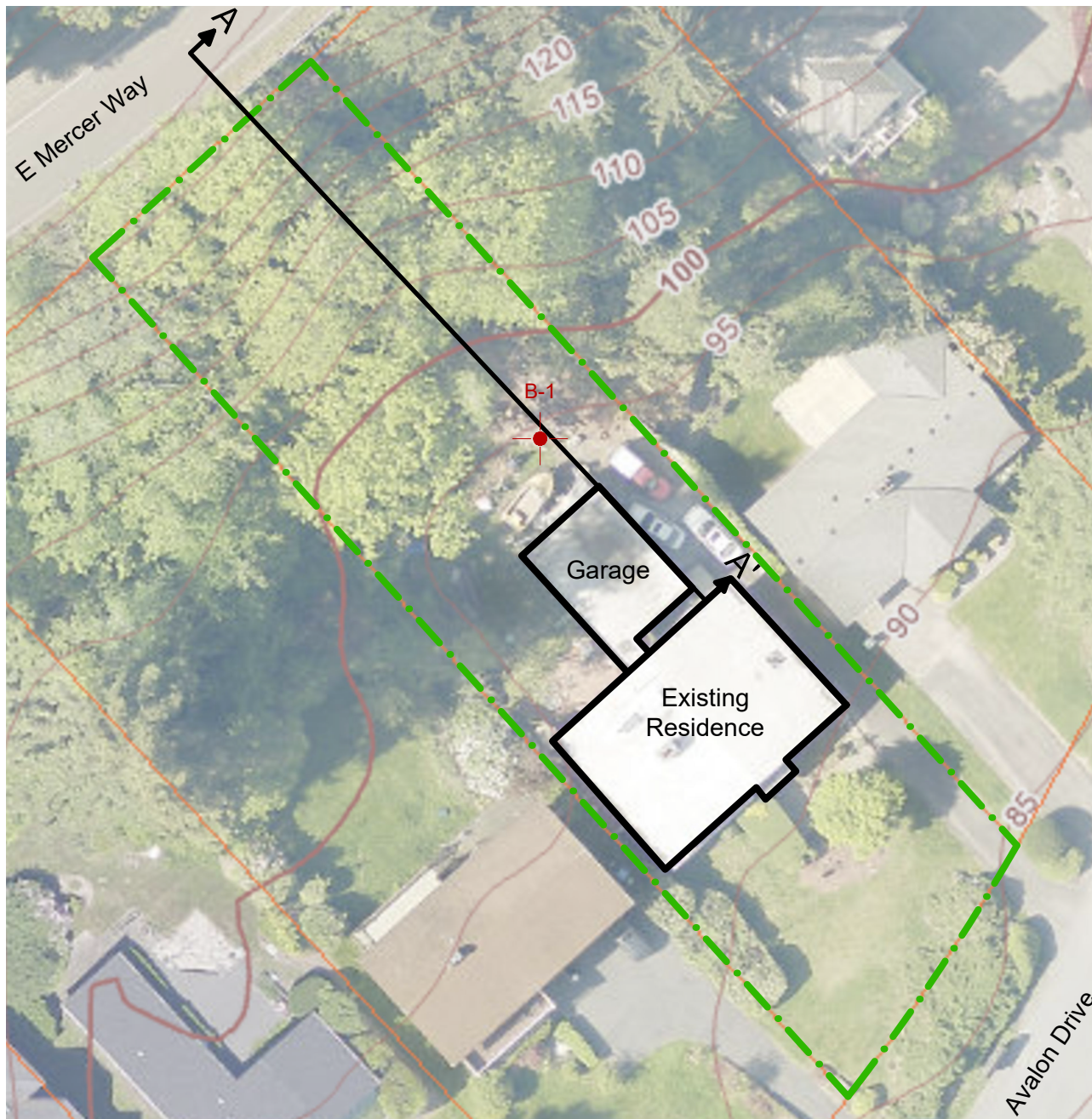
Not to Scale






Mercer Island, WA

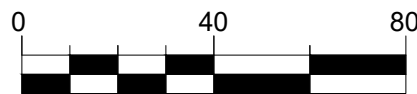
Project Number 1579125	Visy and Madhuri Remodel Seismic Evaluation Vicinity Map	 <p>NELSON GEOTECHNICAL ASSOCIATES, INC</p> <p>Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510</p> <p>Wenatchee Office 105 Palouse St Wenatchee, WA 98801 (509) 665-7696 /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

Site Plan



LEGEND

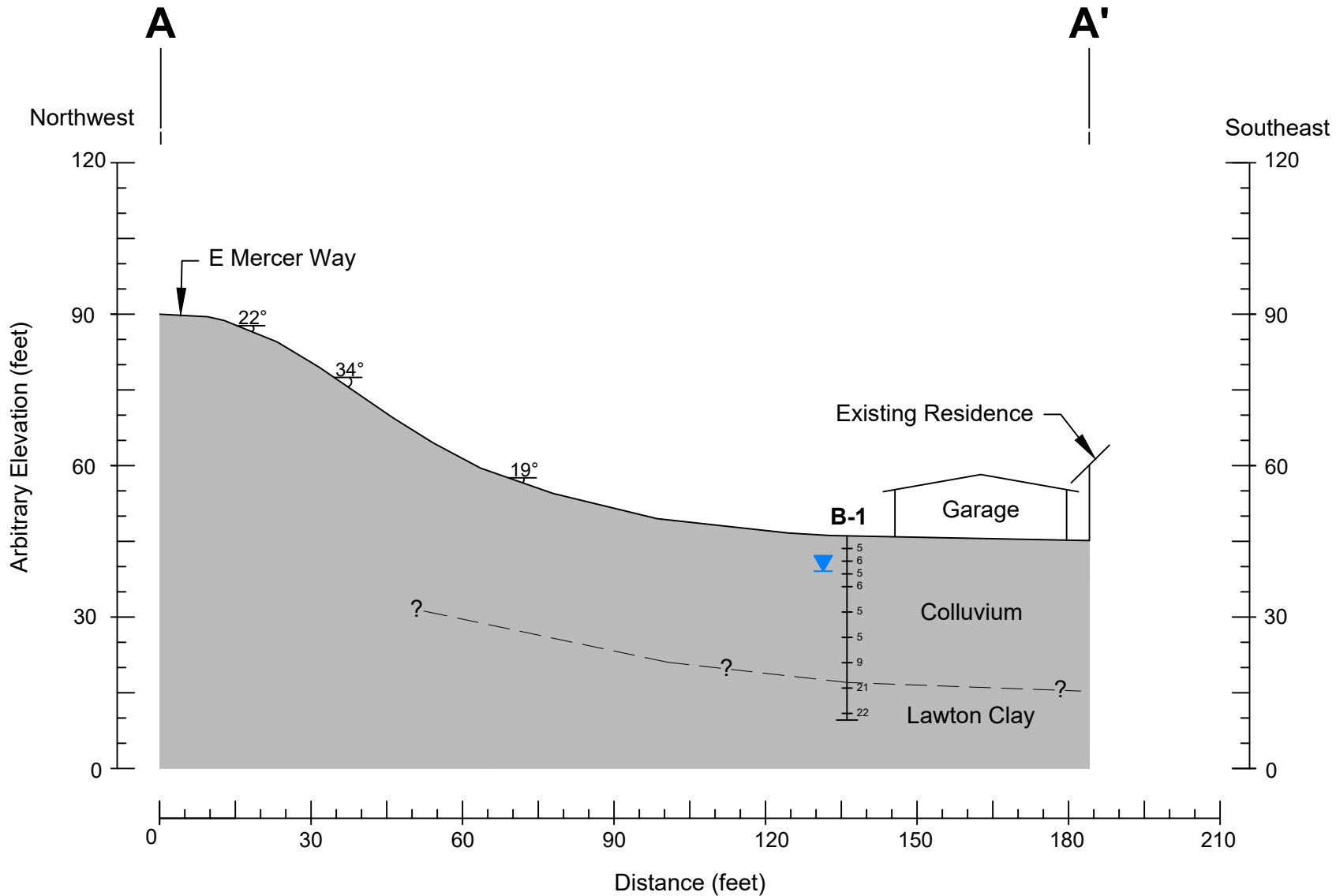
-  Property line
-  B-1
Number and approximate location of boring
-  A A'
Approximate location of cross-section



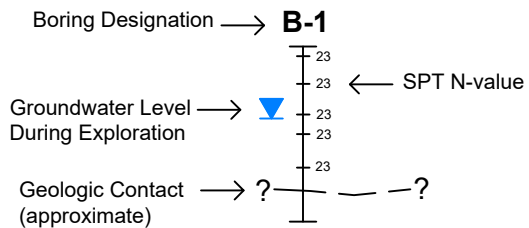
Scale: 1 inch = 40 feet

Reference: Site Plan based on field measurements, observations, and aerial parcel map review.

Project Number 1579125	Vishy and Madhuri Remodel Seismic Evaluation Site Plan	 www.nelsongeotech.com	NELSON GEOTECHNICAL ASSOCIATES, INC Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510	No. 1	Date 3/18/25	Revision Original	By FKS	CK KMS
Figure 2								



Exploration



- NOTES:
- 1) Stratigraphic conditions are interpolated between the explorations. Actual conditions may vary.
 - 2) Elevations are arbitrary.

Reference: Cross Section is based on field measurements using a hand-held clinometer and 100-ft tape measure.

Project Number 1579125	Figure 3	Vishy and Madhuri Remodel Seismic Evaluation Cross-Section A-A'			 NELSON GEOTECHNICAL ASSOCIATES, INC Woodville Office 17311-135th Ave NW, Suite 2500 Woodbury, MN 55129 (425) 486-1669 / Fax: 481-2510 www.nelsongeotech.com Wenatchee Office 106 Faber Ave SE Wenatchee, WA 98801 (509) 665-7686 / Fax: 665-7692	
		No.	1	Date		3/18/25
		Revision	Original	By		FKS
		CK	KMS			

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
		GRAVEL WITH FINES	GP	POORLY-GRADED GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	GRAVEL WITH FINES	GM	SILTY GRAVEL
		SAND WITH FINES	GC	CLAYEY GRAVEL
FINE - GRAINED SOILS	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
		SAND WITH FINES	SP	POORLY GRADED SAND
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	SAND WITH FINES	SM	SILTY SAND
		SAND WITH FINES	SC	CLAYEY SAND
SILT AND CLAY LIQUID LIMIT 50 % OR MORE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	CL	CLAY
SILT AND CLAY LIQUID LIMIT 50 % OR MORE	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
SILT AND CLAY LIQUID LIMIT 50 % OR MORE	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	CH	CLAY OF HIGH PLASTICITY, FAT CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

- 1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.
- 2) Soil classification using laboratory tests is based on ASTM D 2488-93.
- 3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

- Dry - Absence of moisture, dusty, dry to the touch
- Moist - Damp, but no visible water.
- Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number 1579125	Vishy and Madhuri Remodel Seismic Evaluation Soil Classification Chart	 NELSON GEOTECHNICAL ASSOCIATES, INC <small>Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510</small>	<small>Wenatchee Office 105 Palouse St Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692</small>	No.	Date	Revision	By	CK
Figure 4				1	3/18/25	Original	FKS	KMS

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) (Topsoil / Fill)			5	5	●						
Gray, fine to medium sand with silt (loose, moist)		SP-SM	6	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	Monument/ Cap to Piezometer	Native Soil	G	Grain-size Analysis
Liquid Limit	Liquid Limit	Silica Sand	DS	Direct Shear
Plastic Limit	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		NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 5				1	3/18/25	Original	FKS	KMS
Page 1 of 2								



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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)	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.												
				40								40
				45								45
				50								50
				55								55

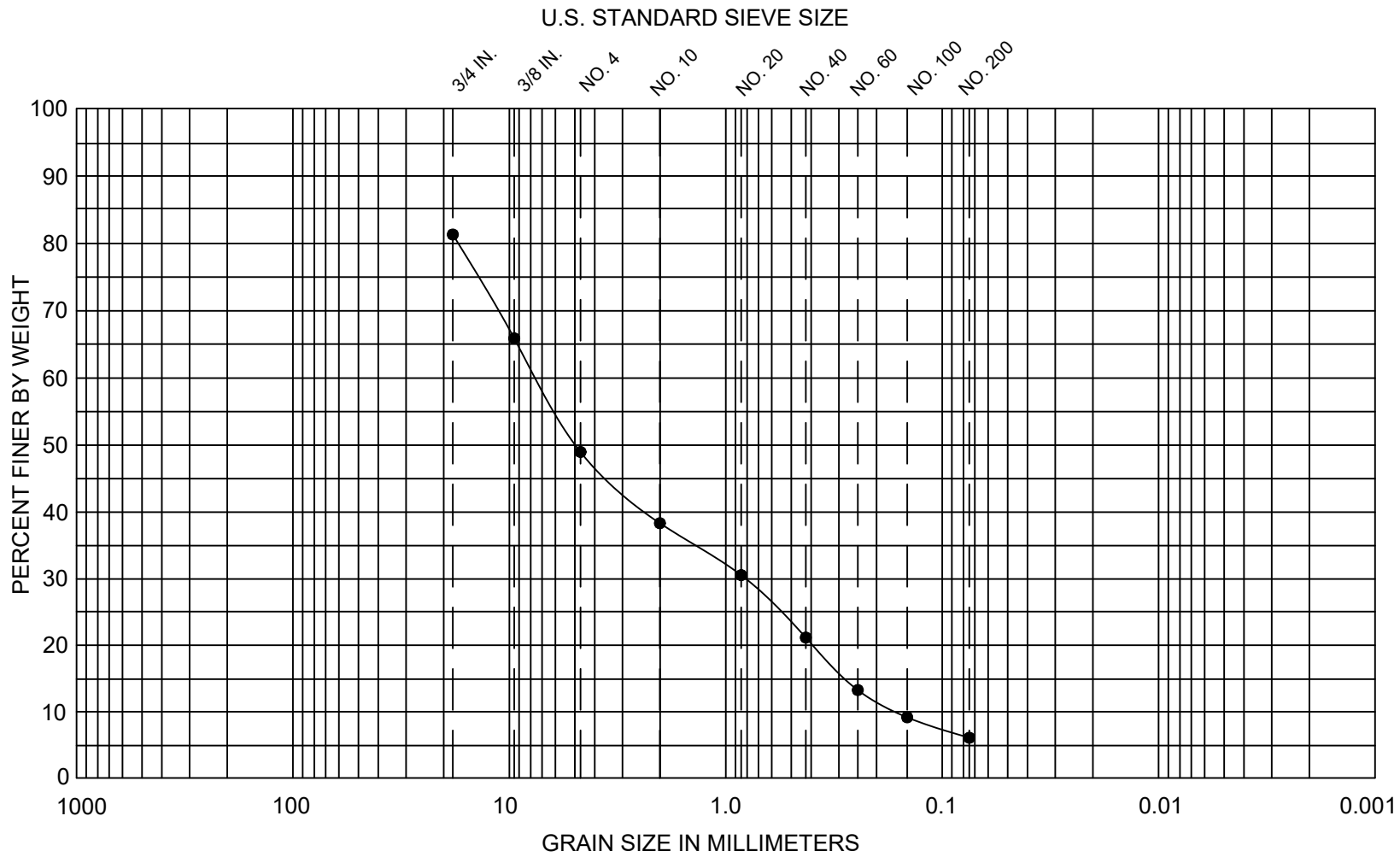
LEGEND

- | | | |
|---|---|---|
| <ul style="list-style-type: none"> Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler | <ul style="list-style-type: none"> Solid PVC Pipe Slotted PVC Pipe Monument/ Cap to Piezometer Liquid Limit Plastic Limit | <ul style="list-style-type: none"> Concrete Bentonite Native Soil Silica Sand Water Level |
| | | <ul style="list-style-type: none"> M Moisture Content A Atterberg Limits G Grain-size Analysis DS Direct Shear 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	Vishy and Madhuri Remodel Seismic Evaluation Boring Log		NELSON GEOTECHNICAL ASSOCIATES, INC 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
1579125					1	3/18/25	Original	FKS	KMS
Figure 5									
Page 2 of 2									

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COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	


U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●GP-GM	B-1	7.5 feet	Gray, gravelly, fine to coarse sand with silt	Gravel = 51% Sand = 43% Silt/Clay = 6%

No.	1	Date	3/19/25	Revision	Original	By	FKS	CK	KMS
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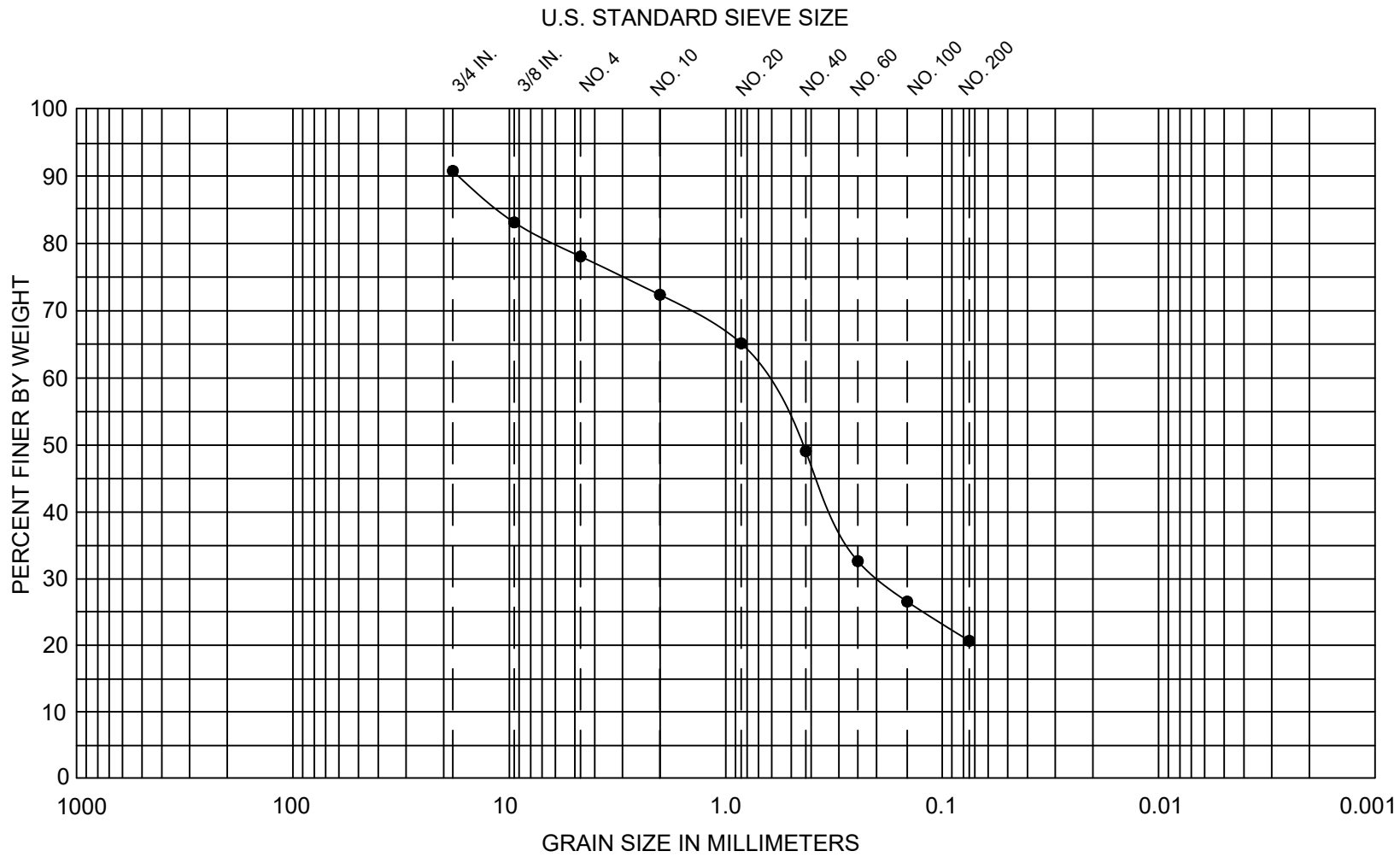
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Vishy and Madhuri Remodel
Seismic Evaluation
Sieve Analysis

Project Number
1579125

Figure 6



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	


U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SM	B-1	15.0 feet	Gray, silty, fine to medium sand with gravel and iron-oxide staining	Gravel = 22% Sand = 57% Silt/Clay = 21%

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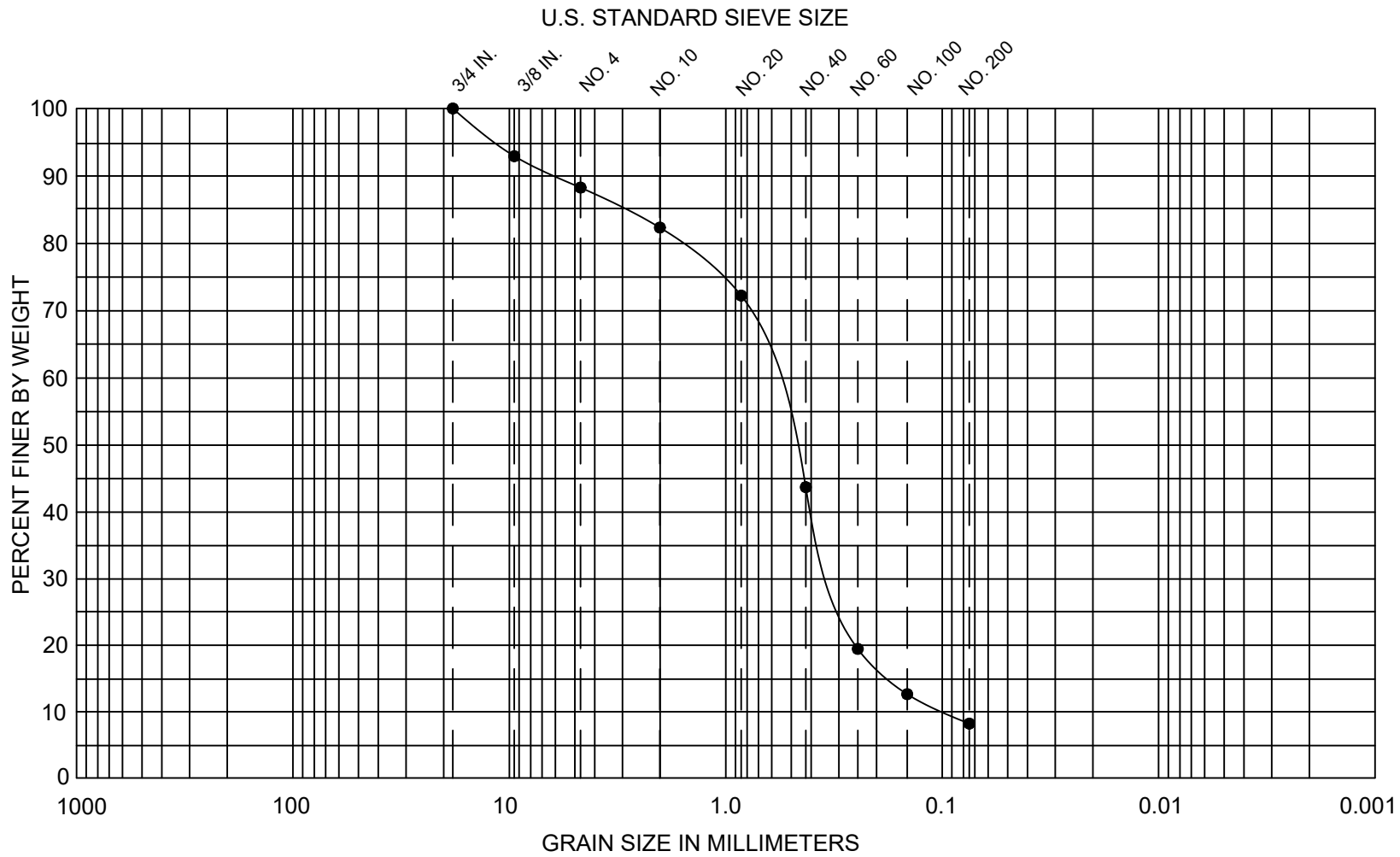
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Vishy and Madhuri Remodel
Seismic Evaluation
Sieve Analysis

Project Number	1579125
	Figure 7



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP-SM	B-1	20.0 feet	Gray, fine to medium sand with gravel and silt	Gravel = 12% Sand = 80% Silt/Clay = 8%

No.	1	Date	3/19/25	Revision	Original	By	FKS	CK	KMS
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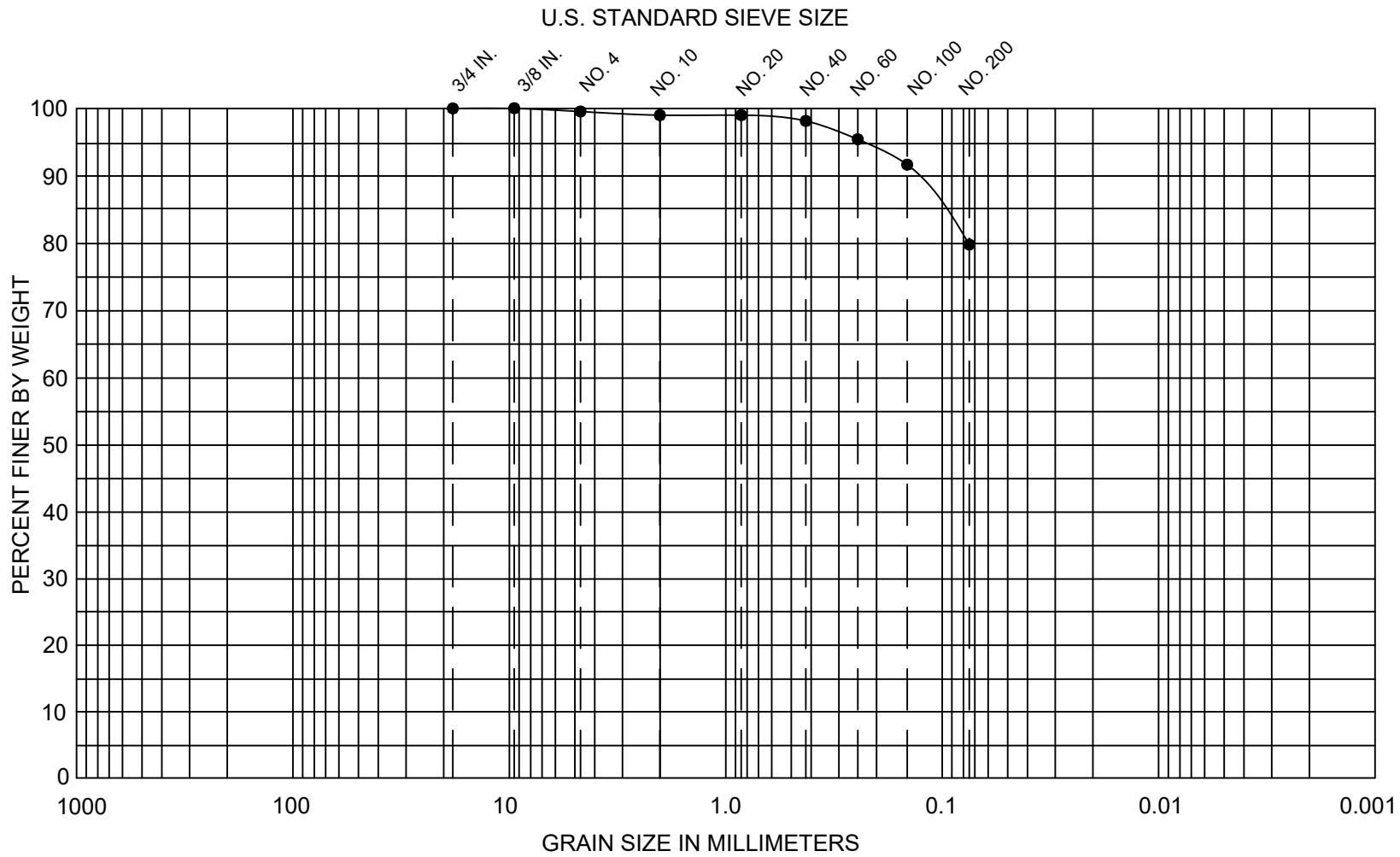
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Vishy and Madhuri Remodel
Seismic Evaluation
Sieve Analysis

Project Number	1579125
Figure	8



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	


U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●ML	B-1	35.0 feet	Gray, silt with fine sand	Gravel = 0% Sand = 20% Silt/Clay = 80%

No.	1
Date	3/19/25
Revision	Original
By	FKS
CK	KMS

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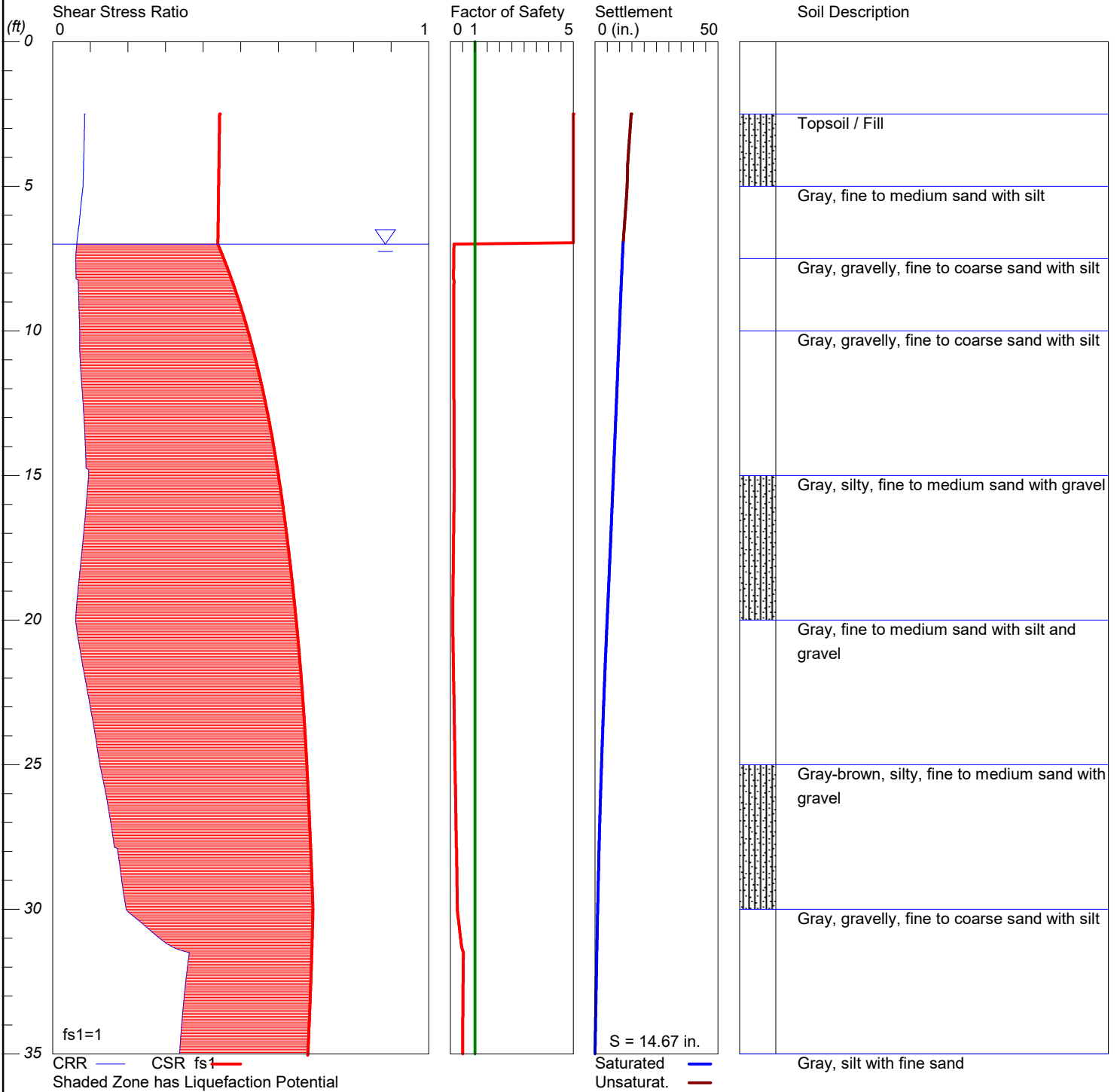
Project Number	1579125
Figure	9

LIQUEFACTION ANALYSIS

Vishy and Madhuri Remodel

Hole No.=1 Water Depth=7 ft

Magnitude=8
Acceleration=0.687g



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