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September 2, 2025

Mr. Daniel Wick  
Horizon View Homes  
VIA Email: [horizonviewhomes@gmail.com](mailto:horizonviewhomes@gmail.com)

Geotechnical Engineering Evaluation  
**Horizon View Homes 177<sup>th</sup> Ave SE Residential Development**  
**16311 - 177<sup>th</sup> Avenue SE**  
**Monroe, Washington**  
NGA File No. 1613825

Dear Mr. Wick:

We are pleased to submit the attached report titled **“Geotechnical Engineering Evaluation – Horizon View Homes 177<sup>th</sup> Ave SE Residential Development – 16311 177<sup>th</sup> Avenue SE – Monroe, Washington.”** This report summarizes our observations of the existing surface and subsurface conditions within the site and provides recommendations for the proposed site development. Our services were completed in general accordance with the proposal signed by you on August 7, 2025.

The site consists of two adjoining parcels covering approximately 2.8 acres. The current site use is recreational storage and an automotive repair shop. We were provided with a preliminary concept plan, which depicts the proposed site use. Based on this plan, the proposed site development will consist of removing the existing site structures and constructing 55 townhomes. Specific grading and stormwater handling plans were not available at the time this report was prepared, however, we understand that stormwater generated by the proposed development may be directed to onsite infiltrations systems, if feasible.

We explored the subsurface conditions within the site on August 15, 2025, by documenting the conditions in six trackhoe excavated test pits to depths in the range of 8 to 10 feet below the existing ground surface. Based on the explorations, the subsurface conditions were relatively homogenous throughout the site and consisted of thin layer of undocumented fill underlain by an approximately 3- to 5-foot-thick mantle of silt to silty sand with granular sand and gravel mixtures below this layer and to the depths explored.

It is our opinion that the proposed site development is feasible from a geotechnical standpoint, provided that our recommendations for site development are incorporated into the project plans. In general, the medium dense or better native soils encountered at relatively shallow depths throughout the site should provide adequate support for the planned structures. For bearing capacity and settlement consideration, all foundation elements should advance through any undocumented fill or other loose soils and be supported directly on competent native soil. Based on our explorations, we would expect these soils to be encountered at depths in the range of 1 to 3 feet below the existing ground surface. Deeper areas of undocumented fill or loose soils are possible in unexplored areas of the site. Where encountered, these soils should be removed and replaced with structural fill for foundation and pavement support.

Final stormwater plans have not been developed; however, we understand onsite stormwater disposal is being considered. Accordingly, design infiltration rates were estimated by correlation to grain size distributions performed on representative samples collected from the site. Determination of infiltration rates and feasibility for this site were conducted in general accordance with the Department of Ecology's 2019 Stormwater Management Manual for Western Washington (2019 SWMMWW). Based on the results, the granular sand and gravel deposits encountered 4 to 5.5 feet below the site are considered suitable for stormwater infiltration.

In the attached report, we have also provided general recommendations for foundations, site grading, slabs-on-grade, structural fill placement, retaining walls, erosion control, and drainage. We should be retained to review and comment on final development plans and observe the earthwork phase of construction. 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 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.**

A handwritten signature in black ink that reads "Alex Rinaldi". The signature is written in a cursive, flowing style.

Alex Rinaldi, PE, LG  
**Project Manager**

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## Horizon View Homes 177<sup>th</sup> Ave SE Residential Development 16311 – 177th Avenue SE Monroe, Washington

### INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the planned residential development in Monroe, Washington. The project site is located at 16311 177<sup>th</sup> Avenue Southeast in Monroe, Washington, as shown on the Vicinity Map in **Figure 1**. The parcel numbers for the subject site are 27061100100100 and 27060200408500. The purpose of this study is to explore and characterize the site's surface and subsurface conditions and to provide geotechnical recommendations for the proposed site development, specifically grading.

The site is currently occupied by various recreational vehicles and other equipment storage along the northern approximately two-thirds of the site and by an automotive repair garage along the southern portion of the property. We understand the proposed development will consist of constructing 55 townhomes throughout the site. Specific grading and development plans had not been developed at the time we prepared this report. The existing site layout is shown on the Site Plan in **Figure 2**.

### SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions, and provide general recommendations for site development. Specifically, our scope of services included the following:

1. Review available soil and geologic maps of the area, as well as other documentation pertaining to the site.
2. Explore the subsurface soil and groundwater conditions within the site with trackhoe excavated test pits. **Excavation services were subcontracted by NGA.**
3. Perform an onsite private utility locate prior to site explorations. **Private utility locator was subcontracted by NGA.**
4. Provide recommendations for site grading and earthwork, including structural fill.
5. Provide recommendations for foundation support and slab-on-grade subgrade preparation.
6. Provide recommendations for retaining walls.
7. Provide recommendations for temporary and permanent slopes.

8. Provide recommendations for site drainage and erosion control.
9. Provide long-term design infiltration rates based on grain-size analysis performed on representative soil samples collected from the site.
10. Install up to two groundwater monitoring piezometers within the excavated test holes for verification of seasonal high groundwater levels, if necessary.
11. Document the results of our findings, conclusions, and recommendations in a written geotechnical report.

## SITE CONDITIONS

### Surface Conditions

The site consists of two adjoining parcels, totaling 2.8 acres. It is currently occupied by recreational vehicles and an automotive repair garage. The site is bordered to the west by 177<sup>th</sup> Avenue SE, to the east by Park Place Middle School, to the north by apartments and a bus barn, and to the south by a low-lying wetland area associated with a historical side channel of the Skykomish River. The ground surface within the site is relatively level and is generally gravel surfaced with some surface slabs adjacent to the existing garage. The site is also located along the apex of a historical fluvial terrace associated with the Skykomish River, whose main channel is approximately 0.6 miles to the south.

### Subsurface Conditions

**Geology:** The geologic units for this area are shown in the Geologic map of the Monroe 7.5-minute quadrangle, King and Snohomish Counties, Washington, by Joe D. Dragovich, et al. (WADNR, 2011). The site is mapped as deltaic outwash and kame deltas (Qgod) with alluvium (Qa) in the near vicinity. Unit Qgod is a recessional deposit of the Vashon Stade of the Fraser glaciation and is described as cobble gravel to pebbly sand; moderately to well sorted; thin to very thickly bedded and well stratified. Unit Qa is described as sand, silt, gravel, gravelly sand, sandy pebble gravel, peat, and organic sediments. Our explorations encountered a mantle of fine-grained sediments underlain by granular sand and gravel mixtures, consistent with the geologic mapping.

**Explorations:** The subsurface conditions within the site were explored on August 15, 2025, by excavating six test pits to depths in the range of 8 to 10 feet below the existing ground surface. The approximate locations of our explorations are shown on the Site Plan in **Figure 2**. A geologist from NGA was present during the explorations, examined soils and geologic conditions encountered, obtained samples of different soil types, and maintained exploration logs.

The soils were visually classified in general accordance with the Unified Soil Classification System, presented in **Figure 3**. The logs of our test pits are attached to this report and are presented as **Figures 4 through 7**. We present a summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the exploration logs should be reviewed.

Explorations were consistent across the site. In general, the explorations uncovered 0.5 to 1.0 feet of surficial fill, which consisted of gravel surfacing and dark brown to brown, silty sand with organics. Underlying the undocumented fill we encountered a layer ranging from 3 to 5 feet thick, consisting of light brown to brown, silty fine sand and silt in a loose to medium dense condition, which we interpreted as fine-grained alluvial overbank deposits. Below depths in the range of 4 to 5.5 feet below the ground surface, we encountered sandy gravel and sand with gravel to the depths explored, which we interpreted as recessional outwash deposits.

### **Hydrogeologic Conditions**

Groundwater seepage was not encountered in the test pits, to the depths explored. Two groundwater monitoring piezometers, denoted as MW-1 and MW-2, were installed within test pits, TP-1 and TP-6, respectively. In review of the Snohomish County PDS web map, a seasonal non-fish bearing stream and associated wetland areas are mapped southeast of the site.

As previously mentioned, the site is positioned on a level plateau above a historical stream terrace slope. We reviewed publicly available digital elevation model (DEM) data for the project area. This data would suggest that the site is at an elevation of roughly 55 to 56 feet above mean sea level, while the toe of the fluvial terrace southeast of the site is at an elevation of roughly 42 feet above mean sea level. We anticipate the groundwater levels may correlate to the surface water level within the side channel below the property. Additionally, a series of nearby geotechnical borings were reviewed through the Washington State DNR geologic information portal. The borings were performed along the Park Place Middle School property, immediately east of the site. The closest boring was approximately 100 feet east of the subject site's east property line within the current play field of the middle school. The exploration log for this boring noted groundwater at a depth of 19 feet below the surface.

## **SENSITIVE AREA EVALUATION**

### **Seismic Hazard**

We reviewed the 2021 International Building Code (IBC) for seismic site classification for this project. Since medium dense or better glacial and alluvial soils are interpreted to underlie the site at depth, the site best fits 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 2 percent probability of occurrence in 50 years (return interval of 2,475 years), and the 2014 USGS seismic hazard maps.

**Table 1 – 2021 IBC 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	1.186	0.417	1.026	N/A	0.811	N/A

The spectral response accelerations were obtained from the ASCE 7 Hazard Tool, derived from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion (2014 data).

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. Based on the soil composition and relative density of the native alluvial and recessional outwash deposits encountered below the site, the liquefaction susceptibility is expected to be relatively low.

**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 Soil Survey of Snohomish County Area, Washington by the Natural Resources Conservation Service (NRCS) classifies the site as Sultan silt loam. The erosion hazard listed for the soils on the property is slight. It is our opinion that the erosion hazard for site soils should be low in areas where vegetation is not disturbed and where proper erosion control materials are utilized to protect excavations.

**LABORATORY ANALYSIS**

We performed five grain size sieve analyses on selected soil samples collected from the site. A summary of the samples tested is provided in Table 2 below and the results are provided in **Figures 8 through 12**.

Table 2: Summary of Grain Size Analyses

Exploration Identification	Depth, feet	USCS Classification
TP-1	2.5	ML
TP-1	7.0	GP
TP-5	3.75	ML
TP-5	8.25	SP-SM
TP-7	8.25	GP

## CONCLUSIONS AND RECOMMENDATIONS

### General

It is our opinion from a geotechnical standpoint that the planned residential development is feasible. Our explorations indicated that the site was underlain surficial undocumented fill materials with native medium dense or better alluvial and recessional deposits at depth. The native soils are expected to provide adequate support for foundation, slab, and pavement loads. We recommend that the new structures be designed utilizing shallow foundations. Footings should extend through any loose soil, and be founded on the underlying medium dense or better native bearing soil, or structural fill extending to these soils. The competent soil should typically be encountered approximately one to three feet below the existing surface throughout the site, based on our explorations. Deeper, localized areas of undocumented fill may also exist in unexplored areas of the site. This condition, if encountered, would require deeper excavations in foundation, slab, and pavement areas to remove the unsuitable soils.

The grain size method was used to correlate an infiltration rate for the site in general accordance with the 2019 SWMMWW. We completed three grain-size distribution analyses on the granular recessional deposits at depth to establish a design infiltration rate. The upper fine-grained materials are not considered feasible for onsite stormwater infiltration. Feasibility for infiltration is based on permeability among a number of other factors, including groundwater separation. Based on the grain-size analyses, it is our opinion that on-site stormwater infiltration is feasible within this site. This is further discussed in the **Stormwater Infiltration** subsection of this report.

The surficial soils encountered on this site are considered moisture-sensitive and may disturb easily 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.

### **Erosion Control Measures**

The erosion hazard for the on-site soils is interpreted to be slight 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. Erosion potential of areas not stripped of vegetation should be low.

### **Site Preparation and Grading**

After erosion control measures are implemented, site preparation should consist of removing loose soils, topsoil, and any undocumented fill from foundations, slab, and pavement areas, to expose medium dense or better native soils at depth. The stripped soil should be removed from the site or stockpiled for later use as a landscaping fill. Based on our observations, we anticipate native, medium dense or better soil to be encountered at approximately one to three feet throughout explored areas of the site. We should note that additional deeper areas of unsuitable soils and/or undocumented fill could be encountered in unexplored areas of the site. This condition, if encountered, would require deeper excavations in foundation, slab, and pavement areas to remove the unsuitable soils.

After site preparation, if the exposed subgrade is deemed loose, it should be compacted to a non-yielding condition and then proof-rolled with a heavy, rubber-tired piece of equipment. Areas observed to pump or weave during the proof-roll test should be reworked to structural fill specifications or over-excavated and replaced with properly compacted structural fill or rock spalls. If loose soils are encountered in the foundation areas, the loose soils should be removed and replaced with rock spalls. If significant surface water flow is encountered during construction, this flow should be diverted around the work areas, and the exposed subgrades should be maintained in a semi-dry condition.

If wet conditions are encountered, alternative site grading techniques might be necessary. 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. If construction is attempted in wet weather, the subgrade should not be compacted, as this could cause further subgrade disturbance. 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 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 at all times as indicated in OSHA guidelines for 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 upper soils should be no steeper than 1.5 Horizontal to 1 Vertical (1.5H: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. The slope 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. Permanent cut and fill slopes should be no steeper than 2H:1V. Permanent slopes should be vegetated and the vegetative cover maintained until established.

## Foundations

Conventional shallow spread foundations should be placed on medium dense or better native bearing soils, as discussed in the **Site Preparation and Grading** subsection of this report. Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection and bearing capacity considerations. Foundations should be designed in accordance with the 2021 IBC. Footing widths should be based on the anticipated loads and allowable soil bearing pressure. Water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend an allowable bearing pressure of not more than 2,000 pounds per square foot (psf) be used for the design of footings founded on the medium dense or better native bearing soils or rock spalls extending to the competent native bearing material. The foundation bearing soil should be evaluated by a representative of NGA. We should be consulted if higher bearing pressures are needed. Current IBC guidelines should be used when considering increased allowable bearing pressure for short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between adjacent footings or across a distance of about 20 feet, based on our experience with similar projects.

Lateral loads may be resisted by friction on the base of the footing and passive resistance against the subsurface portions of the foundation. A coefficient of friction of 0.35 may be used to calculate the base friction and should be applied to the vertical dead load only. Passive resistance may be calculated as a triangular equivalent fluid pressure distribution. An equivalent fluid density of 200 pounds per cubic foot (pcf) should be used for passive resistance design for a level ground surface adjacent to the footing. This level surface should extend a distance equal to at least three times the footing depth. These recommended values incorporate safety factors of 1.5 and 2.0 applied to the estimated ultimate values for frictional and passive resistance, respectively. To achieve this value of passive resistance, the foundations should be poured “neat” against the native medium dense soils or compacted fill should be used as backfill against the front of the footing. We recommend that the upper one foot of soil be neglected when calculating the passive resistance.

## Retaining Walls

Specific plans were not prepared yet, however, retaining walls may be incorporated into the plans. The lateral pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement which can occur as backfill is placed, wall drainage conditions, and the inclination of the backfill. For walls that are free to yield at the top at least one thousandth of the height of the wall (active condition), soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing (at-rest condition). We recommend that walls supporting horizontal backfill and not subjected to hydrostatic forces, be designed using a triangular earth pressure distribution equivalent to that exerted by a fluid with a density of 40 pcf for yielding (active condition) walls, and 60 pcf for non-yielding (at-rest condition) walls. In addition, we recommend a uniform seismic design loading of  $8H$  be used, where “H” is the total height of the wall.

These recommended lateral earth pressures are for a drained granular backfill and assume a horizontal ground surface behind the wall for a distance of at least the height of the wall, not accounting for surcharge loads. Additional lateral earth pressures should be considered for surcharge loads acting adjacent to walls and within a distance equal to the height of the wall. This includes the effects of surcharges such as traffic loads, floor slab loads, slopes, or other surface loads. We could consult with the structural engineer regarding additional loads on retaining walls during design, if needed.

The lateral pressures on walls may be resisted by friction between the foundation and subgrade soil, and by passive resistance acting on the below-grade portion of the foundation. Recommendations for frictional and passive resistance to lateral loads are presented in the **Foundations** subsection of this report.

All wall backfill should be well compacted as outlined in the **Structural Fill** subsection of this report. Care should be taken to prevent the buildup of excess lateral soil pressures due to over-compaction of the wall backfill. This can be accomplished by placing wall backfill in 8-inch loose lifts and compacting the backfill with small, hand-operated compactors within a distance behind the wall equal to at least one-half the height of the wall. The thickness of the loose lifts should be reduced to accommodate the lower compactive energy of the hand-operated equipment. The recommended level of compaction should still be maintained.

Permanent drainage systems should be installed for retaining walls. Recommendations for these systems are found in the **Subsurface Drainage** subsection of this report. We recommend that we be retained to evaluate the proposed wall drain backfill material and observe installation of the drainage systems.

## **Structural Fill**

**General:** Fill placed beneath foundations, pavement, or other settlement-sensitive structures should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The area to receive the fill should be suitably prepared as described in the **Site Preparation and Grading** subsection prior to beginning fill placement. Sloping areas to receive fill should be benched using a minimum 8-foot wide horizontal benches keyed into competent soils.

**Materials:** Structural fill should consist of a good quality, granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about three inches. All-weather fill should contain no more than five-percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). Some of the more granular on-site soils may be suitable for use as structural fill; however, this will be highly dependent on the moisture content of the soil during construction. The use of the on-site soils as structural fill during wet weather will be very difficult, if not impossible. We should be retained to evaluate all proposed structural fill material prior to placement.

**Fill Placement:** Following subgrade preparation, placement of structural fill may proceed. All filling should be accomplished in uniform lifts up to eight inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas and pavement subgrade should be compacted to a minimum of 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D-1557 Compaction Test procedure. The moisture content of the soils to be compacted should be within about two percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction and should be tested.

## **Slab-on-Grade**

Slabs-on-grade should be supported on subgrade soils prepared as described in the **Site Preparation and Grading** subsection of this report. We recommend that all floor slabs be underlain by at least six inches of free-draining gravel with less than three percent by weight of the material passing Sieve #200 for use as a capillary break. We recommend that the capillary break be hydraulically connected to the footing drain system to allow free drainage from under the slab.

A suitable vapor barrier, such as heavy plastic sheeting (6-mil minimum), should be placed over the capillary break material. An additional 2-inch-thick moist sand layer may be used to cover the vapor barrier. This sand layer is optional, and is intended to be used to protect the vapor barrier membrane and to aid in curing the concrete.

### **Pavements**

Pavement subgrade preparation and structural filling where required, should be completed as recommended in the **Site Preparation and Grading** and **Structural Fill** subsections of this report. The pavement subgrade should be proof-rolled with a heavy, rubber-tired piece of equipment, to identify soft or yielding areas that require repair. The pavement section should be underlain by a minimum of six inches of clean granular pit run or crushed rock. We should be retained to observe the proof-rolling and recommend subgrade repairs prior to placement of the asphalt or hard surfaces.

### **Utilities**

We recommend that underground utilities be bedded with a minimum six inches of pea gravel prior to backfilling the trench with on-site or imported material. Trenches within settlement sensitive areas should be compacted to 95% of the modified proctor as described in the **Structural Fill** subsection of this report. Trenches located in non-structural areas should be compacted to a minimum 90% of the maximum dry density. Trench backfill compaction should be tested.

### **Stormwater Infiltration**

**General:** We performed three representative grain-size analyses on selected soil samples obtained within the site in accordance with the 2019 SWMMWW. The grain size analyses that were utilized for infiltration rate correlation were Test Pit 1 at 7.0 feet, Test Pit 5 at 8.25 feet, and Test Pit 7 at 8.25 feet. We should note that the other two grain size analyses (**Figures 8 & 10**) classified the upper material as silt and are not considered suitable for stormwater infiltration purposes.

**Long-Term Infiltration Rate:** An equation provided in Section V-5.4 of the 2019 SWMMWW, was used to determine the infiltration capabilities of the site soil utilizing data from the grain-size analyses. Based on this equation and information obtained from the grain-size analyses, calculated initial short-term infiltration rates were 25.3 to 80.9 inches per hour for the native recessional outwash soils at depth. We also referenced Table V-5.1 of the manual to provide an adequate correction factor to infiltration rates obtained from the above equation to calculate a long-term design rate. Correction factors of 0.80, 0.40, and 0.9 were utilized in this equation for  $CF_v$ ,  $CF_t$ ,  $CF_m$ , respectively. A total correction factor of 0.288 was

applied to the most conservative sieve analysis calculated rate to determine the long-term design infiltration rate.

Using the above correction factor, we calculated a long-term design infiltration rate of 7.3 inches per hour for the native gravel and sand material encountered at the site. We recommend that the base of any on-site infiltration systems be terminated within the native, granular soils for the above rate to apply. We anticipate that the infiltration systems should encounter these soils at depths in the range of 4 to 5.5 feet below the existing ground surface.

The stormwater manual recommends a minimum 5 feet of separation between the base of an infiltration system and any underlying bedrock, impermeable horizon, or groundwater. We did not encounter groundwater within the test pits to a maximum of 10-feet below the surface. Piezometers were installed into two of the test pits, should further verification of groundwater levels be necessary.

We recommend that any proposed infiltration systems be placed as to not negatively impact any proposed or existing nearby structures and also meet all required setbacks from existing property lines, structures, and sensitive areas as discussed in the drainage manual. In general, infiltration systems should not be located within proposed fill areas within the site associated with site grading or retaining wall backfill as such condition could lead to failures of the placed fills and/or retaining structures. We should be retained to evaluate the infiltration system design and installation during construction.

### **Site Drainage**

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

**Subsurface Drainage:** If groundwater seepage 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 and routed into a permanent storm drain.

We recommend the use of footing drains around the 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. Pea gravel is an acceptable drain material. The free-draining material should extend up the wall to one foot below the finished surface. The top foot of backfill should consist of impermeable soil placed over plastic sheeting or building paper to minimize surface water or fines migration 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 Mr. Daniel Wick and his agents, for use in the planning and design of the development 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.

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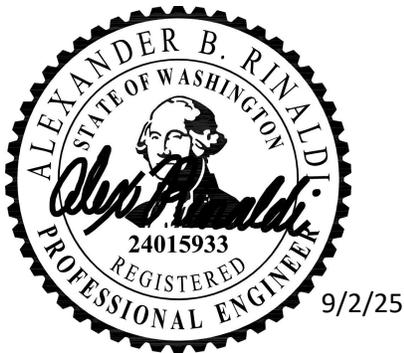
It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

**NELSON GEOTECHNICAL ASSOCIATES, INC.**



Max Gabbert, GIT  
Staff Geologist I



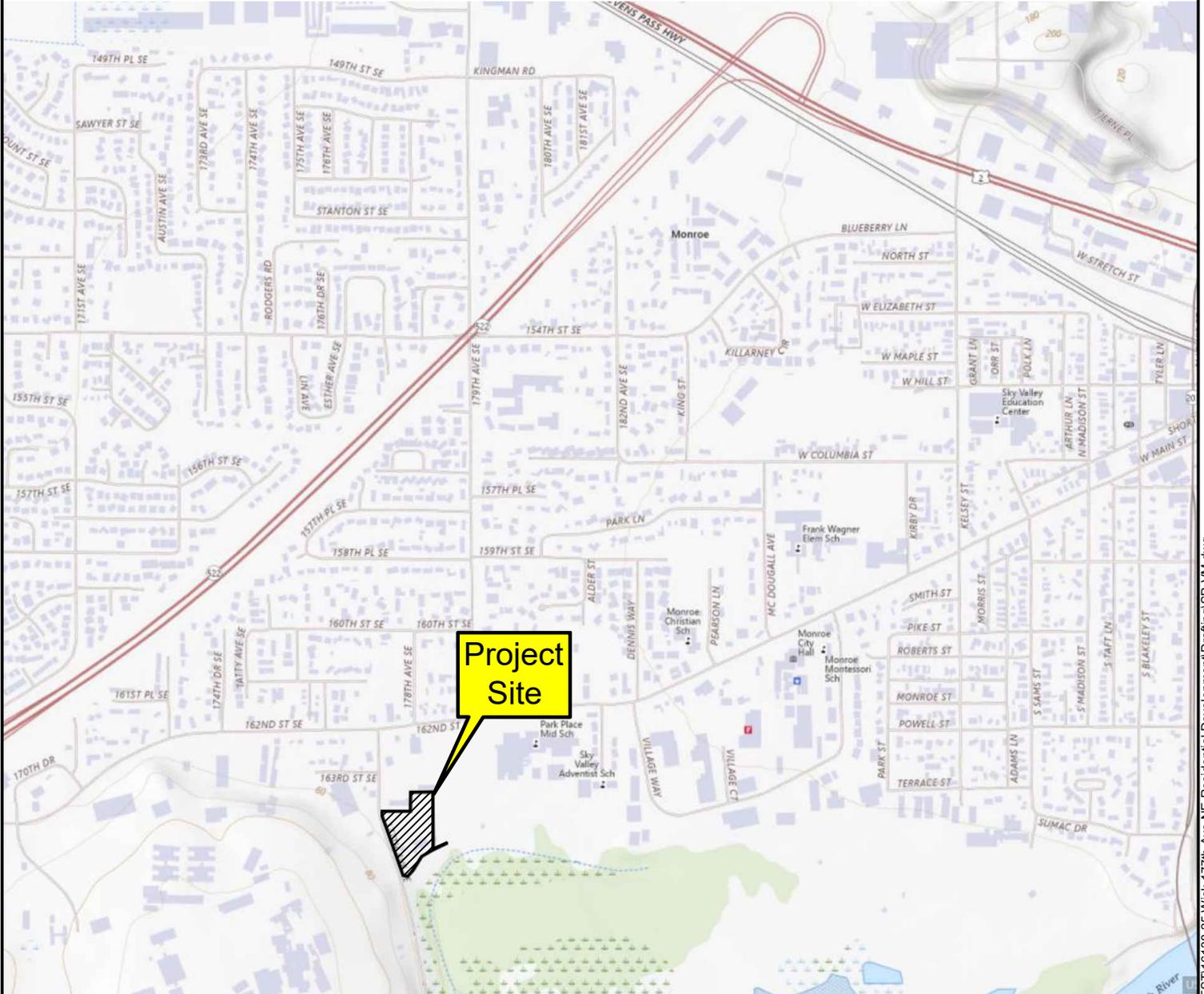
Alex Rinaldi, PE, LG  
Project Manager

ABR:mm

Twelve Figures Attached

# VICINITY MAP

Not to Scale



Snohomish County, WA

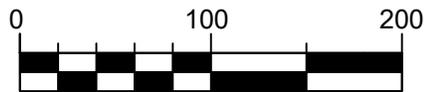
Project Number 1613825	Wick Residence Development Vicinity Map	 <b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> Woodinville Office 19900 144th Ave. NE, Ste 200 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 www.nelsongeotech.com	Wenatchee Office 105 Palouse St. Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692	No.	Date	Revision	By	CK
Figure 1				1	8/25/25	Original	AMS	ABR

# Site Plan



## LEGEND

- Property line
- TP-1  
 Number and approximate location of test pit
- MW-1  
 Number and approximate location of installed piezometer



Approximate Scale: 1 inch = 100 feet

Reference: Site Plan based on an undated, unnamed site plan.

Project Number 1613825	Wick Residence Development Site Plan	 <b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> Woodinville Office 19900 144th Ave. NE, Ste 200 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 www.nelsongeotech.com	Wenatchee Office 105 Palouse St Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692	No.	Date	Revision	By	CK
Figure 2				1	8/25/25	Original	AMS	ABR

# UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
<b>COARSE - GRAINED SOILS</b>  <small>MORE THAN 50 % RETAINED ON NO. 200 SIEVE</small>	<b>GRAVEL</b>  <small>MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE</small>	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	<b>SAND</b>  <small>MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE</small>	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
<b>FINE - GRAINED SOILS</b>  <small>MORE THAN 50 % PASSES NO. 200 SIEVE</small>	<b>SILT AND CLAY</b>  <small>LIQUID LIMIT LESS THAN 50 %</small>	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	<b>SILT AND CLAY</b>  <small>LIQUID LIMIT 50 % OR MORE</small>	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
<b>HIGHLY ORGANIC SOILS</b>			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

<b>Project Number</b> 1613825	<b>Wick Residence Development Soil Classification Chart</b>	 <b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> <small>Woodinville Office 19900 144th Ave. NE, Ste 200 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 <a href="http://www.nelsongeotech.com">www.nelsongeotech.com</a></small> <small>Wenatchee Office 105 Palouse St Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692</small>	<b>No.</b> 1	<b>Date</b> 8/25/25	<b>Revision</b> Original	<b>By</b> AMS	<b>CK</b> ABR
<b>Figure 3</b>							

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## LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
<b>TEST PIT TP-1/ MW-1</b>		
0.0 – 0.5		Gravel surfacing underlain by brown, silty fine to medium SAND with roots (loose to medium dense, moist) ( <b>UNDOCUMENTED FILL</b> )
0.5 – 4.0	<b>SM</b>	Orange-brown, silt with fine SAND (medium stiff to stiff, moist)
4.0 – 10.0	<b>SP-SM</b>	Gray-brown, sandy GRAVEL with cobbles and silt (medium dense, moist)
		Samples were collected at 2.5, 7.0, and 9.0 feet Groundwater seepage was not encountered Slight test pit caving was encountered between 4.0 to 10.0 feet Test Pit TP-1 was completed at 10.0 feet on 8/15/2025
<b>TEST PIT TP-2</b>		
0.0 – 0.5		Gravel surfacing underlain by brown, silty fine to medium SAND with roots (loose to medium dense, moist) ( <b>UNDOCUMENTED FILL</b> )
0.5 – 5.5	<b>SM</b>	Orange-brown, silty fine SAND (loose to medium dense, moist)
5.5 – 8.0	<b>SP-SM</b>	Gray-brown, fine to coarse SAND with gravel, cobbles, and silt (medium dense, moist)
		Samples were collected at 2.75, 6.0, and 8.0 feet Groundwater seepage was not encountered Slight test pit caving was encountered between 5.0 to 8.0 feet Test Pit TP-2 was completed at 8.0 feet on 8/15/2025

## LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
<b>TEST PIT TP-3</b>		
0.0 – 1.0		Gravel surfacing underlain by brown, silty fine to medium SAND with roots (loose to medium dense, moist) ( <b>UNDOCUMENTED FILL</b> )
1.0 – 4.0	<b>SM</b>	Orange-brown, silty fine SAND (loose to medium dense, moist)
4.0 – 5.0	<b>SP-SM</b>	Gray-brown, fine to coarse SAND with gravel, cobbles, and silt (medium dense, moist)
5.0 – 8.0	<b>SP-SM</b>	Gray-brown, fine to medium SAND with silt, gravel, and cobbles (medium dense, moist)
		Samples were collected at 3.0, 5.0, 7.0, and 8.0 feet Groundwater seepage was not encountered Slight test pit caving was encountered between 4.0 to 8.0 feet Test Pit TP-3 was completed at 8.0 feet on 8/15/2025
<b>TEST PIT TP-4</b>		
0.0 – 0.5		Gravel surfacing underlain by dark brown, silty fine to medium SAND (loose, moist) ( <b>UNDOCUMENTED FILL</b> )
0.5 – 4.0	<b>SM</b>	Gray-brown, silty fine SAND (loose to medium dense, moist)
4.0 – 8.25	<b>SP-SM</b>	Gray-brown, fine to coarse SAND with gravel, cobbles, and silt (medium dense, moist)
		Samples were collected at 3.25, 6.5, and 8.25 feet Groundwater seepage was not encountered Slight test pit caving was encountered between 4.0 to 8.25 feet Test Pit TP-4 was completed at 8.25 feet on 8/15/2025

## LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
<b>TEST PIT TP-5</b>		
0.0 – 0.75		Gravel surfacing underlain by dark brown, silty fine to medium SAND (loose, moist) ( <b>UNDOCUMENTED FILL</b> )
0.75 – 5.0	<b>SM</b>	Orange-brown, SILT with fine sand (medium stiff to stiff, moist)
5.0 – 9.25	<b>SP-SM</b>	Orange-brown to gray-brown, gravelly SAND with silt and cobbles (medium dense, moist)
		Samples were collected at 3.75, 7.5, and 8.25 feet Groundwater seepage was not encountered Slight test pit caving was encountered between 5.0 to 9.25 feet Test Pit TP-5 was completed at 9.25 feet on 8/15/2025
<b>TEST PIT TP-6/ MW-2</b>		
0.0 – 0.25		Gravel surfacing underlain by dark brown, silty fine to medium SAND (loose, moist) ( <b>UNDOCUMENTED FILL</b> )
0.25 – 4.25	<b>SM</b>	Orange-brown, silty fine SAND (loose to medium dense, moist)
4.25 – 5.0	<b>SP-SM</b>	Gray-brown, fine to coarse SAND with silt, gravel, and cobbles (medium dense, moist)
5.0 – 10.0	<b>SP-SM</b>	Orange-brown, silty fine to coarse SAND with silt, gravel, and cobbles; 2-inch iron-oxide weathering layer at 8.0 feet (medium dense, moist)
		Samples were collected at 2.5, 4.75, and 8.0 feet Groundwater seepage was not encountered Slight test pit caving was encountered between 4.25 to 9.25 feet Test Pit TP-6 was completed at 10.0 feet on 8/15/2025

## LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
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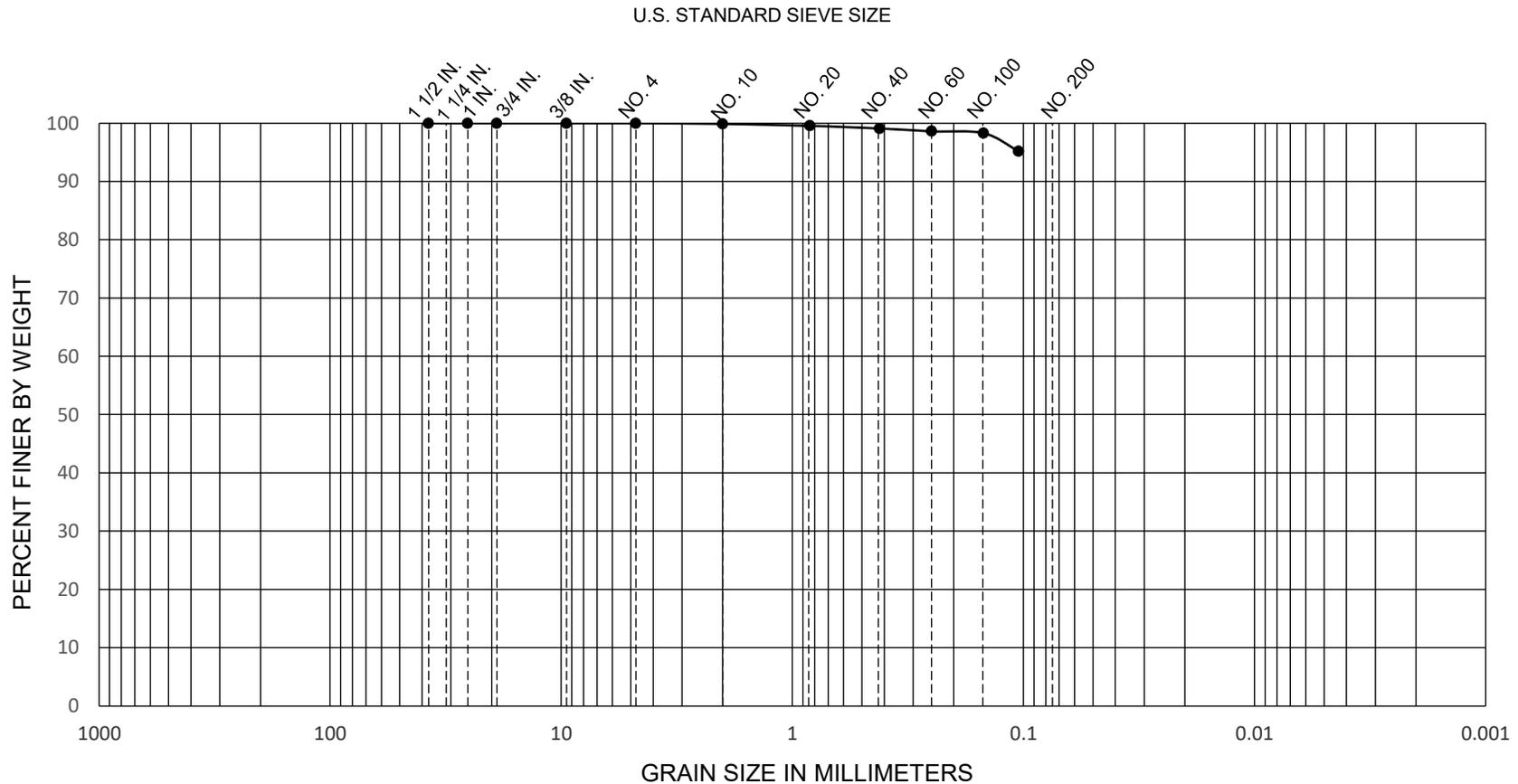
### TEST PIT TP-7

0.0 – 1.0		Gravel surfacing underlain by dark brown, silty fine to medium SAND (loose, moist) ( <b>UNDOCUMENTED FILL</b> )
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1.0 – 4.25	<b>SM</b>	Orange-brown, silty fine SAND (medium dense, moist)
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4.25 – 8.25	<b>SP-SM</b>	Gray-brown, sandy GRAVEL with silt and cobbles (medium dense, moist)
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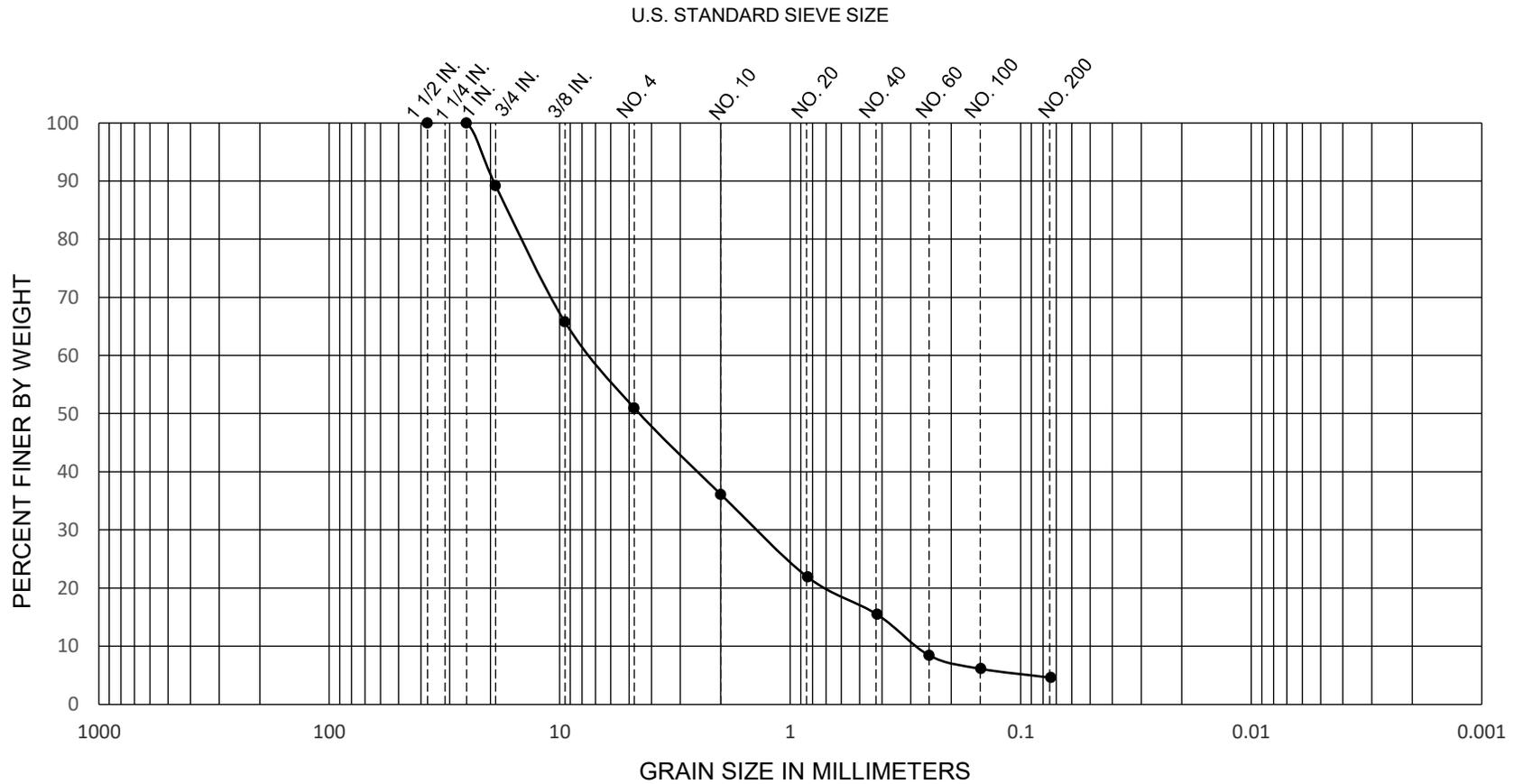
Samples were collected at 5.0 and 8.3 feet  
Groundwater seepage was not encountered  
Slight test pit caving was encountered between 4.25 to 8.25 feet  
Test Pit TP-7 was completed at 8.25 feet on 8/15/2025



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

USCS SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
<b>ML</b>	TP-1	2.5 ft	Orange-brown, SILT with trace sand	Gravel = 0.0% Sand = 4.8% Fines = 95.2%

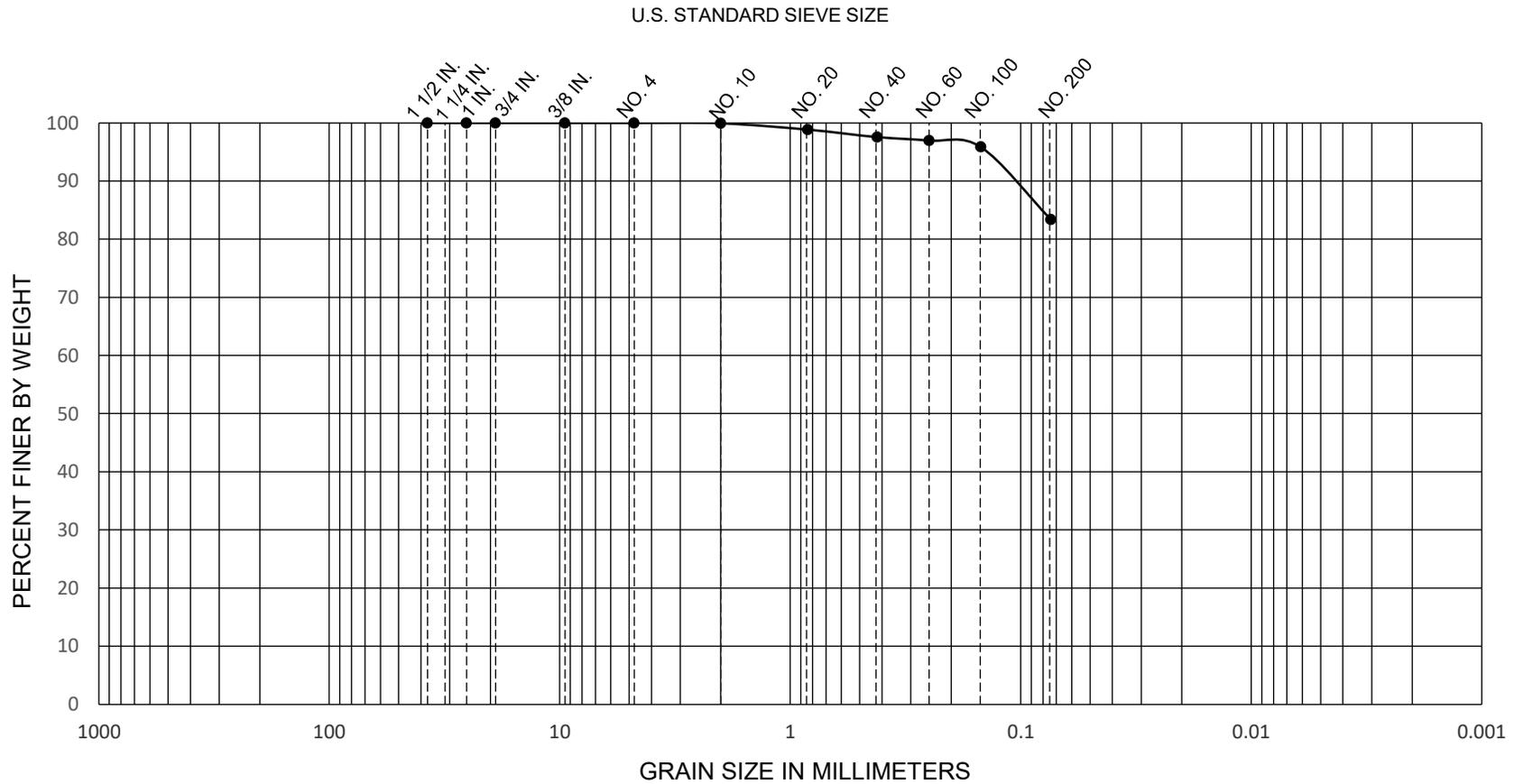
Wick Residence Development		<b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> <small>Woodinville Office 19900 144th Ave NE - Ste 200 Woodinville WA 98072 (25) 486-1669 / Fax: 481-2510</small>	<small>Wenatchee Office 105 Palouse St. Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692</small>	Project Number 1613825
Sieve Analysis		Figure 8		



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

USCS SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
<b>GP</b>	TP-1	7.0 ft	Gray-brown, sandy GRAVEL with trace silt	Gravel = 49.0% Sand = 46.4% Fines = 4.6%

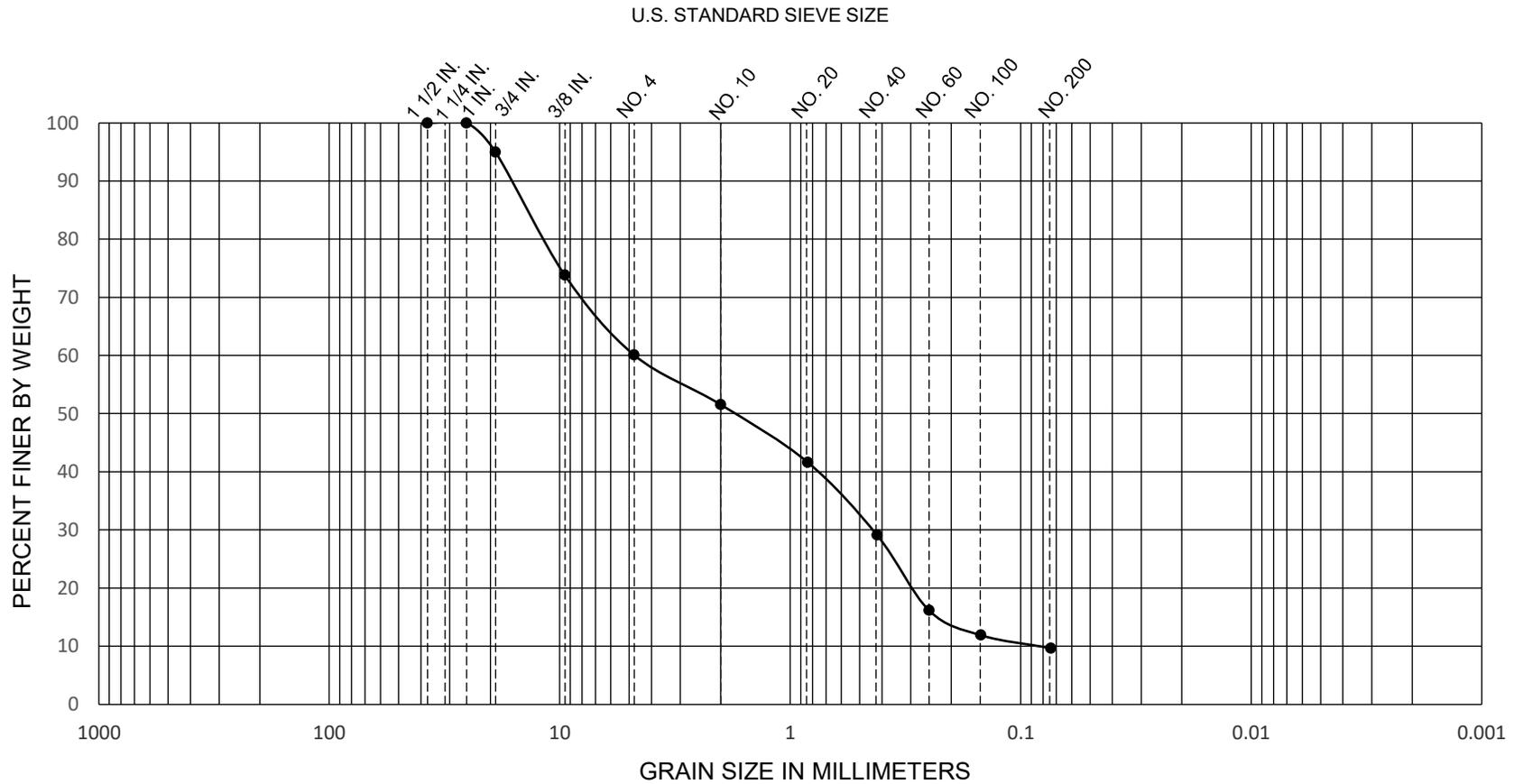
Wick Residence Development		<b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> <small>Woodinville Office 19900 144th Ave NE - Ste 200 Woodinville WA 98072 (25) 486-1669 / Fax: 481-2510</small>	<small>Wenatchee Office 105 Palouse St. Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692</small>		Project Number 1613825
Sieve Analysis		Figure 9			



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

USCS SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
<b>ML</b>	TP-5	3.75 ft	Orange-brown, SILT with some fine sand	Gravel = 0.0% Sand = 16.6% Fines = 83.4%

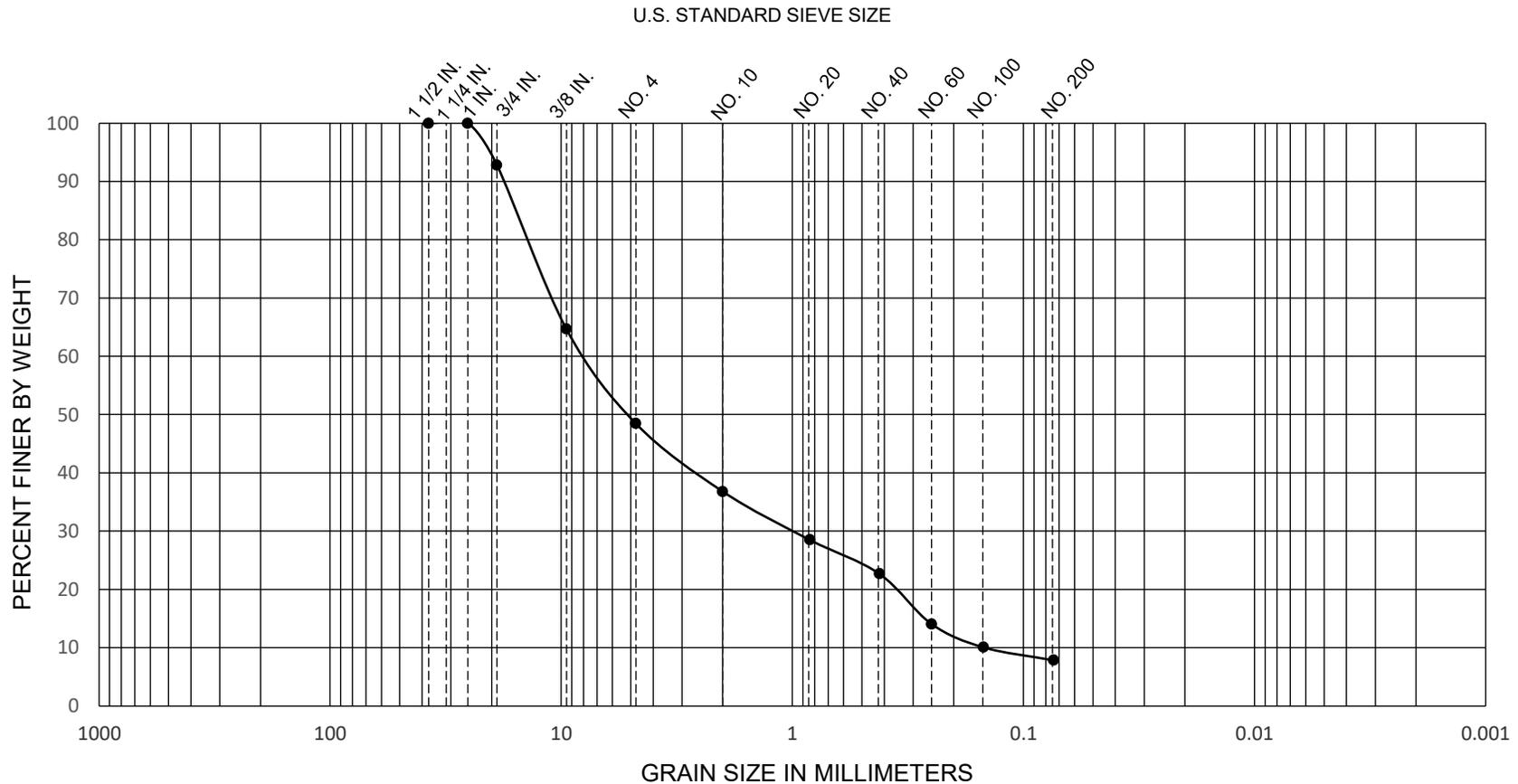
Wick Residence Development		<b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> <small>Woodinville Office 19900 144th Ave NE - Ste 200 Woodinville WA 98072 (25) 486-1669 / Fax: 481-2510</small>	<small>Wenatchee Office 105 Palouse St. Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692</small>	Project Number 1613825
Sieve Analysis		Figure 10		



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

USCS SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
<b>SP-SM</b>	TP-5	8.25 ft	Orange-brown to gray-brown, gravelly SAND with silt	Gravel = 39.9% Sand = 50.4% Fines = 9.7%

Wick Residence Development		<b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> <small>Woodinville Office 19900 144th Ave NE - Ste 200 Woodinville WA 98072 (25) 486-1669 / Fax: 481-2510</small>	<small>Wenatchee Office 105 Palouse St. Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692</small>	Project Number 1613825
Sieve Analysis		Figure 11		



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

USCS SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
<b>GP</b>	TP-7	8.25 ft	Gray-brown, sandy GRAVEL with trace silt	Gravel = 51.5% Sand = 40.6% Fines = 7.9%

Wick Residence Development		<b>NELSON GEOTECHNICAL ASSOCIATES, INC</b> <small>Woodinville Office 19900 144th Ave NE - Ste 200 Woodinville WA 98072 (25) 486-1669 / Fax: 481-2510</small>	<small>Wenatchee Office 105 Palouse St. Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692</small>	Project Number <b>1613825</b>
Sieve Analysis		<b>Figure 12</b>		