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October 5, 2022

Daniel Wick
VIA Email: **dan@hvhinc.com**

Geotechnical Engineering Evaluation
Creekside Village Residential Development
7811 and 7715 – 40th Street NE, 7808 – 44th Street NE
Marysville, Washington
Parcel Numbers: 29050200100400, -300, and -200
NGA File No. 1394722

Dear Daniel:

We are pleased to submit the attached report titled ***"Geotechnical Engineering Evaluation – Creekside Village Residential Development – 7811 and 7715 – 40th Street NE, 7808 – 44th Street NE – Marysville, Washington."*** This report summarizes our observations of the existing surface and subsurface conditions within the property and provides general recommendations for stormwater infiltration for the proposed site development. Our services were completed in general accordance with the proposal signed by you on August 25, 2022.

The site consists of three adjacent parcels covering a total area of approximately of 18.69 acres with the above listed addresses. There are currently single-family residences within the northern and southern portions of the site. Most of the proposed development is forested with few designated wetland areas. The property is bordered by 40th Street NE to the south, 79th Avenue NE to the east, 44th Street NE to the north, and residential properties to the west. Topographically, the site is relatively level to gently sloping, with isolated moderate slopes in different areas of the site, particularly surrounding the north-south trending creek along the eastern portion of the site. We understand the proposed development will consist of a residential subdivision occupying the northern and southeastern parcel as well as the northern portion of the southwestern parcel.

We explored the site subsurface soil conditions with eight trackhoe-excavated test pits, one of which was utilized for infiltration testing. Our explorations indicated that the site was underlain by competent native glacial soils at relatively shallow depths, below a layer of undocumented fill and/or topsoil. We performed on-site infiltration testing in accordance with the 2019 Stormwater Management Manual for Western Washington, utilized by the City of Marysville. Our on-site testing consisted of one small pilot infiltration test (PIT) performed in accordance with City of Marysville requirements.

Subsurface soils generally consisted of silty, fine to medium sand with varying amounts of gravel and cobbles in a medium dense to dense condition, which we interpreted as native glacial till soils. The test procedures and results are summarized in the attached report. However, based on our observations, and the silty nature of the native glacial till deposits that underlie the site at depth, it is our opinion that the on-site native soils encountered at depth are not conducive for traditional stormwater infiltration systems. Due to the relatively thin recessional outwash layer in the central-northern portion of the site, shallow, low-impact systems may be feasible, provided they incorporate an overflow component directed to an approved point of discharge. This is further discussed in the attached report.

In the attached report, we have also provided general recommendations for earthwork, foundations, slab-on-grade, erosion control and drainage. We should be retained to review and comment on final drainage plans and observe installation of drainage systems. We recommend that NGA be retained to review the design and 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.

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

TABLE OF CONTENTS

LIST OF FIGURES

Geotechnical Engineering Evaluation
Creekside Village Residential Development
7811 and 7715 – 40th Street NE, 7808 – 44th Street NE
Marysville, Washington
Parcel Numbers: 29050200100400, -300, and -200

INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the infiltration feasibility within the site located at **7811 and 7715 – 40th Street NE, 7808 – 44th Street NE in Marysville, Washington,** as shown on the Vicinity Map in Figure 1. The parcel numbers for the proposed residential development include 290502-001-003-00, 290502-001-002-00, and 290502-001-004-00.

The site consists of three adjacent parcels covering a total area of approximately of 18.69 acres. There are currently single-family residences within the northern and southern portions of the site. Most of the proposed development is forested with few designated wetland areas. The property is bordered by 40th Street NE to the south, 79th Avenue NE to the east, 44th Street NE to the north, and residential properties to the west. Topographically, the site is relatively level to gently sloping. We understand the proposed development will consist of a residential subdivision occupying the northern and southeastern parcel as well as the northern portion of the southwestern parcel.

Specific grading and stormwater plans have not been developed, but we understand that stormwater may be directed to on-site infiltration systems, if feasible. The existing site layout is shown on the Site Plan in Figure 2.

We performed on-site infiltration testing in accordance with the 2019 Stormwater Management Manual for Western Washington, utilized by the City of Marysville. Our on-site testing consisted of one small pilot infiltration test (PIT) performed in accordance with City of Marysville requirements.

SCOPE

The purpose of this study was to explore and characterize the site's surface and subsurface conditions and to provide geotechnical recommendations for the planned residence development.

Specifically, our scope of services included the following:

1. Reviewing available soil and geologic maps of the area.
2. Exploring the subsurface soil and groundwater conditions within the site with trackhoe-excavated test pits. Trackhoe to be provided by the client.
3. Determining feasibility of on-site stormwater infiltration based on the soil and groundwater conditions encountered.
4. Installing piezometers within the test pit explorations throughout the site, as needed. Gravel backfill and well materials to be provided by NGA.
5. Providing recommendations for infiltration system installation.
6. Providing recommendations for site drainage and erosion control.
7. Providing recommendations for site preparation and earthwork.
8. Providing recommendations for foundation support and retaining walls.
9. Providing recommendations for temporary and permanent slopes.
10. Providing recommendations for slab and pavement subgrade preparation.
11. Providing long-term design infiltration rates with onsite testing per the 2019 DOE Stormwater Manual.
12. Documenting the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The site consists of three adjacent parcels covering a total area of approximately of 18.69 acres. There are currently single-family residences on each of the three properties that comprise the site in the northern and southern portions of the site. Most of the proposed development is forested with scattered mapped wetland areas, the largest being associated with the creek running along the eastern portion of the site. The property is bordered by 40th Street NE to the south, 79th Avenue NE to the east, 44th Street NE to the north, and residential properties to the west.

Topographically, the site is relatively level to gently sloping, with a few very small, isolated steep slopes. In general, these slopes were less than 10 feet in height and did not appear to constitute a landslide hazard. Vegetation across the site consisted of grass yard areas with a moderate density of evergreen and deciduous trees and native shrubs. No surface water was encountered anywhere during our site visit; however, we did see evidence from standing water in the winter in some of the mapped wetland areas.

Subsurface Conditions

Geology: The geologic units for this site are shown on the Geologic Map of the Marysville Quadrangle, Snohomish County, Washington, by J.P. Minard, (USGS 1985). The site is mapped as glacial till (Qvt). The till is described as a nonsorted mixture of clay, silt, sand, pebbles, and cobbles with occasional large boulders all in varying amounts. Our explorations within the site generally encountered surficial undocumented fill/topsoil underlain by silty, fine to medium sand with gravel, and iron oxide staining, which we interpreted as native glacial till deposits throughout the property.

Explorations: The subsurface conditions within the site were explored on August 26, 2022 by excavating eight test pits extending to depths in the range of 6.0 to 8.4 feet below the existing ground surface, one of which was used for infiltration testing. 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 the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the explorations. The soils were visually classified in general accordance with the Unified Soil Classification System, presented in Figure 3. The logs of our explorations are attached to this report and are presented as Figure 4 and 5. We present a brief summary 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 infiltration test pit one we encountered silty, fine to medium sand with gravel, and roots to a depth of 3.0 feet below the existing ground surface, which we interpreted to be undocumented fill. Underlying the fill, we encountered compact fine to coarse sand with gravel, cobbles, and iron-oxide staining, which we interpreted to be part of a small cap of recessional outwash that overlies the native glacial till soils in the northern section of the site to a depth of 6.0 feet. At the surface of Test Pit One, Two, Three, Five, and Seven we encountered grass underlain by silty, fine to medium sand with gravel, roots, and various amounts of organics, which we interpreted to be undocumented fill to depths in the range of 0.8 – 6.0 feet below the existing ground surface. Underlying the fill, we encountered silty, fine to medium sand with gravel and various amounts of cobbles, and iron-oxide staining, which we interpreted to be native glacial till soils to depths in the range of 6.0 – 8.4 feet. At the surface of Test Pit Four and Six we encountered grass underlain by silty, fine to medium sand with gravel, roots, and various amounts of organics, which we interpreted to be undocumented fill to depths in the range of 3.0 – 3.5 feet below the existing ground surface. Underlying the fill, we encountered fine to coarse sand with silt and gravel, which we interpreted to be part of a small cap of recessional outwash that overlies the native glacial till soils throughout the northern section of the site to depths in the range of 6.8 – 8.0 feet. Test Pit Six was completed within this layer; however, in Test Pit Four between the depths of 6.8 to 7.5 feet, we encountered silty, fine to medium sand with gravel, which we interpreted to be native glacial till soils.

Hydrogeologic Conditions

Groundwater seepage was not encountered within any of our explorations. If groundwater is encountered within the site during construction, we would interpret this water to be perched water. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of relatively low permeability materials. The more permeable soils consist of the topsoil/weathered soils. The low permeability soil consists of relatively silty glacial soils. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Perched water tends to vary spatially and is dependent upon the amount of rainfall. We would expect the amount of perched groundwater to decrease during drier times of the year and increase during wetter periods.

SENSITIVE AREA EVALUATION

Seismic Hazard

We reviewed the 2018 International Building Code (IBC) for seismic site classification for this project. Since competent glacial soils are inferred to underlie the site 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 2018 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	1.105	0.393	1.058	Null	0.779	Null

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2014 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. It is our opinion that the medium dense or better glacial deposits interpreted to underlie the site have a low potential for liquefaction or amplification of ground motion.

Steep Slope Hazard

Two isolated mapped areas with slopes steeper than 33 percent exist on the subject site. However, in the field, these areas did not appear overly steep and were less than 10 feet in vertical relief. In our opinion, these isolated mapped steep slopes do not constitute a landslide hazard.

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. We reviewed data from the Soil Survey of the Snohomish County Area, Washington, by the Natural Resources Conservation Service (NRCS). The soils on the site are listed as Tokul gravelly medial loam, 0 to 8 percent slopes, with an outcrop of Norma loam in the northeast corner of the site, and a small outcrop of Tokul gravelly medial loam, 8 to 15 percent slopes in the southeast corner of the site. The erosion hazard is listed as slight for the Tokul gravelly medial loam, 0 to 8 percent slopes, and for the Norma loam. The erosion hazard for the Tokul gravelly medial loam, 8 to 15 percent slopes is listed as moderate. Based on our experience in the area and the material encountered, we would interpret this site as having low to moderate erosion hazard where the surficial soils are exposed, and where vegetation has been removed. We also interpret that the erosion hazard for site soils should be low in areas where the site is not disturbed, and vegetation remains in place.

LABORATORY ANALYSIS

We performed one grain size analysis on a sample obtained from Test Pit 1 at a depth of 5.0 feet below the ground surface. The grain size analysis generally indicated that the native recessional outwash soils underlying the site meet the USCS classification of sand. The results of our grain size analysis are presented in Figure 6.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the planned residential development within the site is feasible from a geotechnical standpoint. Our explorations indicated that the site was underlain by medium dense or better native glacial till soils at relatively shallow depths, with a small cap of recessional outwash overlying the native glacial till soils in the northern section of the site. These glacial till soils should provide adequate support for foundation, slab, and pavement loads. We recommend that the structures be designed utilizing shallow foundations.

Footings should extend through any loose surficial soil and be keyed into the underlying competent native bearing soils. These soils should be encountered roughly 2.5 to 6.0 feet below the existing ground surface throughout the site. We should note that localized areas of deeper unsuitable soils and/or undocumented fill could be encountered at this site in unexplored areas. This condition would require additional excavations in foundation, slab, and pavement areas to remove the unsuitable soils if encountered. We should be retained to evaluate the foundation subgrades prior to placing rebar and forms.

We performed on-site infiltration testing based on the 2019 Stormwater Management Manual for Western Washington which is used by the City of Marysville. Based on our observations, and results of the onsite testing, it is our opinion that the on-site native recessional outwash soils in the northern-central section of the site, in the area surrounding Test Pit 2 and Infiltration Test Pit 1 may be able to support limited infiltration systems. However, based on our observations, results of the onsite testing, and the relatively silty and dense nature of the native glacial till deposits that are interpreted to underlie the site, it is our opinion that the on-site native glacial till soils on the site are not conducive for traditional stormwater infiltration systems. but rather limited infiltration systems. Low impact systems such as pervious pavements, bioretention, or bioswales could be incorporated into the sites' drainage plans with an overflow component included in the design. Depth to seasonal high groundwater across the site is unknown, however, we installed two piezometers within two of our test pits on our site visit, to monitor the depth to seasonal high groundwater in the wet season as needed. More detailed information on our infiltration tests and drainage recommendations can be found in the **Stormwater Infiltration** and **Site Drainage** subsections of this report.

The soils encountered on this site are considered moisture-sensitive and will disturb easily when wet. To reduce cost overruns and delays, we recommend that any construction or installation of drainage systems take place during the drier summer months. If construction takes place during the rainy months, additional expenses and delays should be expected. Additional expenses could include the need for placing erosion control and temporary drainage measures, the need for placing a blanket of rock spalls on exposed subgrades and construction traffic areas prior to placing structural fill, and the need for importing all-weather material for structural fill.

Erosion Control

The erosion hazard for the on-site soils is considered to be low but the actual hazard 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 stripped of vegetation should be low.

Site Preparation and Grading

After erosion control measures are implemented, site preparation should consist of stripping the topsoil, undocumented fill and/or loose soils from foundation, slab, pavement areas, and other structural areas, to expose medium dense or better native bearing soils. The stripped soil should be removed from the site or stockpiled for later use as a landscaping fill. Based on our observations, we anticipate stripping depths of 2.5 to 6.0 feet, depending on the specific locations. However, additional stripping may be required if areas of deeper undocumented fill and/or loose soil are encountered in unexplored areas of the site.

After site stripping, 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 pavement areas, the loose soils should be removed and replaced with rock spalls or granular structural fill. If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed, and the exposed subgrades should be maintained in a semi-dry condition.

If wet conditions are encountered, alternative site stripping and 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 wet conditions are encountered or 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.

The site soils are considered to be moisture-sensitive and will disturb easily when wet. We recommend that construction take place during the drier summer months if possible. However, if construction takes place during the wet season, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades, construction traffic areas, and paved areas prior to placing structural fill. Wet weather grading will also require additional erosion control and site drainage measures. Some of the on-site soils may be suitable for use as structural fill, depending on the moisture content of the soil at the time of construction. NGA should be retained to evaluate the suitability of all on-site and imported structural fill material during construction.

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 always maintain safe slope configurations 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 surficial and/or undocumented fill soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). Temporary cuts in the competent native glacial soils at depth should be no steeper than 1H: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. However, flatter inclinations may be required in areas where loose soils are encountered. Permanent slopes should be vegetated, and the vegetative cover maintained until established.

Foundations

Conventional shallow spread foundations should be placed on undisturbed medium dense or better native soils. Medium dense or better native glacial till bearing soils should be encountered roughly 2.5 to 6.0 feet below the ground surface based on our explorations; however, deeper areas of loose soil may be encountered in unexplored areas of the site. Where undocumented fill or less dense soils are encountered at footing bearing elevation, the subgrade should be over-excavated to expose suitable bearing native soil. The over-excavation may be filled with structural fill, or the footing may be extended down to the competent native soils. If footings are supported on structural fill, the fill zone should extend outside the edges of the footing a distance equal to one half of the depth of the over-excavation below the bottom of the footing. 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 2018 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 design 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 soils or structural fill 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 have not been developed however, retaining walls may be incorporated into the final plans. The lateral pressure acting on subsurface 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. These recommended lateral earth pressures are for a drained granular backfill and are based on the assumption of a horizontal ground surface behind the wall for a distance of at least the subsurface height of the wall, and do not account for surcharge loads. Additional lateral earth pressures should be considered for surcharge loads acting adjacent to subsurface walls and within a distance equal to the subsurface height of the wall. This would include 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 final 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 all 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.

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, but this will be highly dependent on the moisture content of these soils at the time of construction. 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. 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 may 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 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 percent 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 percent of the maximum dry density. Trench backfill compaction should be tested.

Stormwater Infiltration

We performed one abbreviated PIT within Infiltration Test Pit One due to the limited access to water on-site. The infiltration pit measured approximately 4.0-feet long by 3.0-feet wide by 6.0-feet deep. The soils within the pit consisted of gray, compact fine to coarse sand with gravel, cobbles, and silt that we interpreted to be native recessional outwash soils. Before we performed the test, we were informed of a limitation in the water source and could not use more than 100 gallons, and the pressure of the source was limited. Therefore, we performed a limited infiltration test with a one-hour soaking period to estimate the infiltration rate. Infiltration Test Pit One was filled with approximately 6 inches of water and this level was maintained at a steady state for 1 hour. At this time, the water flow rate into the pit was monitored with a Great Plains Industries (GPI) TM 050 water flow meter and rates were recorded at every 5-minute interval. The flow rate for Infiltration Pit One stabilized at 0.629 gallons per minute (37.7 gallons per hour), which equates to an approximate average infiltration rate of 15.13 inches per hour. After the 1-hour steady state period was completed, the water was shut off and the water level within the infiltration pit was monitored every 5 minutes. After 35 minutes all the water had drained out of the pit resulting in an infiltration rate of 10.28 inches per hour, which is an estimate of the field infiltration rate.

We also performed one laboratory grain size analysis on a sample taken from Infiltration Pit One at a depth of 5.0-feet. An equation provided in Section V-5.4 of the 2019 WSDOE Stormwater Management Manual for Western Washington 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, initial short-term infiltration rates yielded 45.7 inches per hour.

In accordance with the Table V 5.1 of the 2019 WSDOE Stormwater Management Manual for Western Washington, correction factors of 0.8, 0.5, and 0.8 for CFv, CFt, and the long-term conductivity loss factor, respectively were applied to the field measured infiltration rate. A total correction factor of 0.32 was applied to the most conservative measured short term infiltration rate obtained from the falling head portion of the onsite test which was 10.28 inches per hour, to determine the long-term design infiltration rate. Using the above correction factor, we calculated a long-term design infiltration rate of approximately 3.29 inches per hour.

Based on the results of infiltration testing and of our sieve analysis, it is our opinion that the stormwater infiltration is feasible within the area surrounding Test Pit Two and Infiltration Test Pit One, which are located in a large cap of recessional outwash. However, infiltration capacity within the overall site may still be limited due to seasonal high groundwater perched on top of glacial till soils found across the site at depth. We installed two groundwater monitoring piezometers to monitor seasonal high groundwater levels, if it becomes necessary.

Based on our explorations within the remainder of the site, due to the relatively dense and silty nature of the native glacial till soils at depth, it is our opinion that traditional stormwater infiltration is not feasible within the remainder of the site. Due to this, we anticipate all runoff generated within the southern portion of the site may need to be routed and directed to an approved point of discharge, most likely located within the adjacent streets.

It is our opinion that low impact systems such as pervious pavements, bioretention systems, and/or bioswales could be utilized throughout the site and especially near Test Pit Two and Infiltration Test Pit One with an overflow component incorporated into the design.

The feasibility of any low-impact, design systems should be determined by the project civil engineer. All other runoff from impervious surfaces within the site should be directed to flow into onsite detention systems within the silty glacial till soils and ultimately tied into the existing storm drain system located near the site.

We recommend that any low-impact systems if determined feasible should extend through the upper fill soils and terminate within the native, glacial soils. Infiltration systems should be sized and designed in accordance with the 2019 Stormwater Management Manual for Western Washington and also incorporate an overflow component directed to an approved point of discharge.

We recommend that any low-impact design 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 low-impact infiltration system design and installation during construction, if necessary. We should be retained to review plans for site drainage.

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 discharged into an approved discharge system.

Subsurface Drainage: If groundwater is encountered, 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 generally recommend the use of footing drains around the structures. Footing drains may 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 **Daniel Wick** and associated agents, for use in the planning and design of the development on these sites 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 we be retained to review the project plans after they have been developed to determine that recommendations in the report were incorporated into project plans.

We recommend that NGA be retained to review final plans prior to construction. We also 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

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.

Katelyn S. Brower, GIT
Project Geologist

10.05.2022

Khaled M. Shawish, PE
Principal

FKS:KSB:KMS:dy

Attachment: Six Figures

VICINITY MAP

Not to Scale

N

Marysville, WA

Project Number 1394722	Creekside Village Vicinity Map	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 1		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 — www.nelsongeotech.com	1	9/12/22	Original	FKS	KSB

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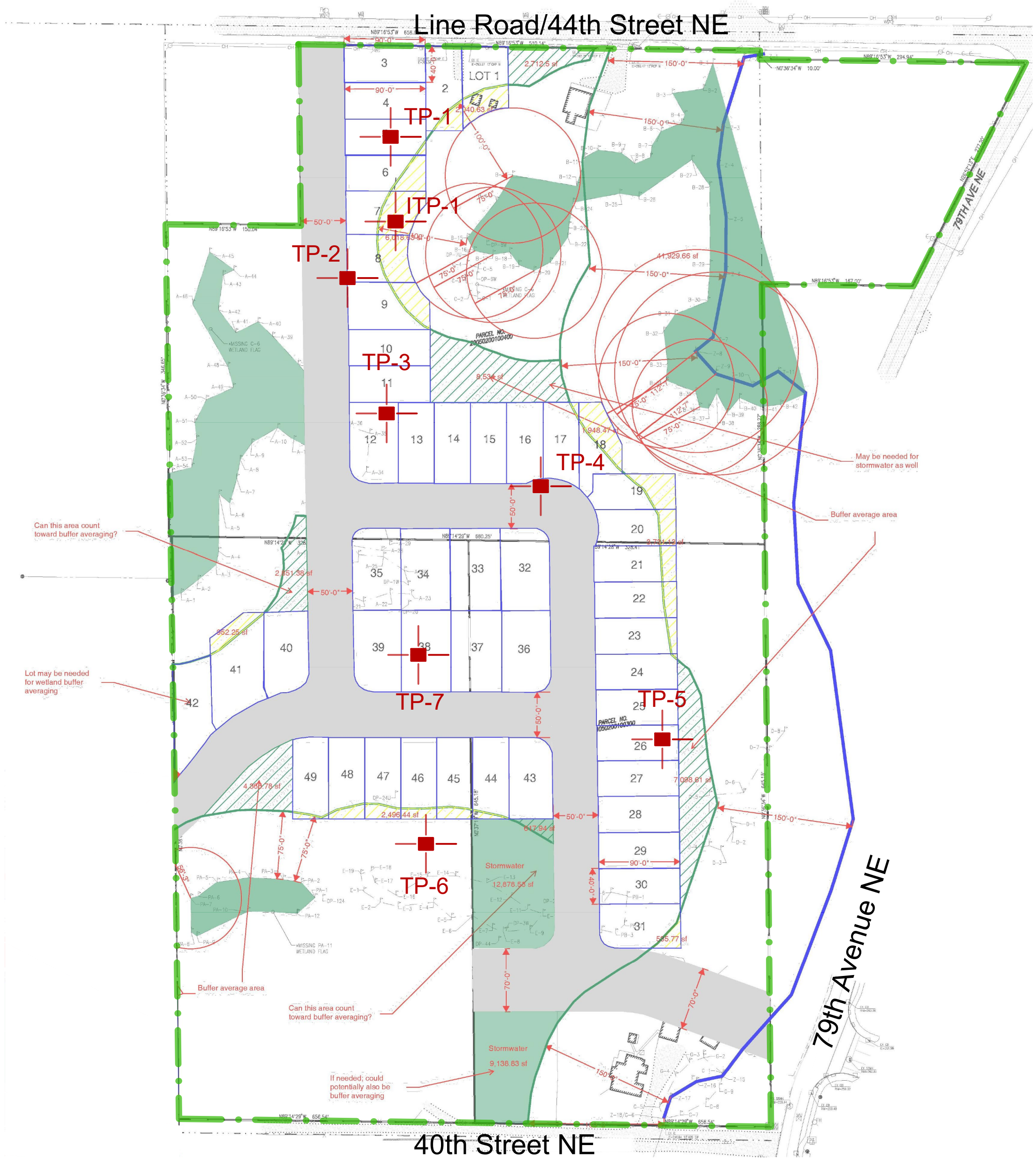
Site Plan

N



0 200 400

Scale: 1 inch = 200 feet

LEGEND

Property line

ITP-1

Number and approximate location of infiltration test pit

TP-1

Number and approximate location of test pit

Reference: Preliminary Site Plan.

Project Number 1394722	Creekside Village Site Plan	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 2			1	9/7/22	Original	FKS	KSB

NGA

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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		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 1394722	Creekside Village Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 3			1	9/7/22	Original	FKS	KSB

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LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
INFILTRATION TEST PIT ONE		
0.0 – 3.0		BROWN-ORANGE, SILTY, FINE TO MEDIUM SAND WITH GRAVEL AND ROOTS (LOOSE, MOIST) (**FILL**)
3.0 – 6.0	SP	GRAY, COMPACT FINE TO COARSE SAND WITH GRAVEL, COBBLES, AND IRON-OXIDE STAINING (MEDIUM DENSE, MOIST)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.0 FEET ON 8/26/2022
TEST PIT ONE		
0.0 – 3.5		GRASS UNDERLAIN BY CRUSHED DRIVEWAY GRAVEL, CONCRETE MIXED WITH DARK BROWN-BLUE GRAY, SILTY, FINE TO MEDIUM SAND AND IRON-OXIDE STAINING AT 3.5 FEET (LOOSE, MOIST) (**FILL**)
3.5 – 6.0		BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (LOOSE TO MEDIUM DENSE, MOIST) (**FILL**)
6.0 – 6.8	SM	BROWN-GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL AND IRON-OXIDE STAINING (MEDIUM DENSE, MOIST) (**TILL**)
		SAMPLES WERE COLLECTED AT 6.8 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.8 FEET ON 8/26/2022
TEST PIT TWO		
0.0 – 2.5		FOREST DUFF UNDERLAIN BY BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (LOOSE, MOIST) (**FILL**)
2.5 – 7.0	SM	GRAY, SILTY, FINE TO COARSE SAND WITH GRAVEL, SAND LENSES, AND IRON-OXIDE STAINING (MEDIUM DENSE, MOIST) (**TILL**)
		SAMPLES WERE COLLECTED AT 5.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 8/26/2022
TEST PIT THREE		
0.0 – 0.8		BROWN, SILTY, TOPSOIL WITH ROOTS (LOOSE, DRY TO MOIST) (**FILL**)
0.8 – 6.0	SM	BROWN-GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE TO VERY DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 3.0 AND 6.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.0 FEET ON 8/26/2022
TEST PIT FOUR		
0.0 – 3.0		BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL AND ROOTS (LOOSE, MOIST) (**FILL**)
3.0 – 6.8	SP-SM	GRAY, FINE TO COARSE SAND WITH SILT AND GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
6.8 – 7.5	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 4.5 AND 7.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.5 FEET ON 8/26/2022

LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 3.0		BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL, ROOTS, AND ORGANICS (LOOSE, DRY TO MOIST) (**FILL**)
3.0 – 8.4	SM	BROWN-GRAY, SILTY, FINE TO COARSE SAND WELL CEMENTED WITH GRAVEL (MEDIUM DENSE, DRY TO MOIST)
		SAMPLES WERE COLLECTED AT 5.5 AND 8.4 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 8.4 FEET ON 8/26/2022
TEST PIT SIX		
0.0 – 3.5		FOREST DUFF UNDERLAIN BY BROWN, SILTY, FINE TO MEDIUM SAND WITH ROOTS AND ORGANICS (LOOSE, MOIST) (**FILL**)
3.5 – 8.0	SP-SM	GRAY, PARTIALLY CEMENTED FINE TO COARSE SAND WITH GRAVEL AND SILT (MEDIUM DENSE TO VERY DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 6.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 8.0 FEET ON 8/26/2022
TEST PIT SEVEN		
0.0 – 2.5		GRASS UNDERLAIN BY BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL AND ROOTS (LOOSE, MOIST) (**FILL**)
2.5 – 7.5	SM	GRAY, WELL CEMENTED SILTY, FINE TO MEDIUM SAND WITH GRAVEL AND COBBLES (MEDIUM DENSE TO DENSE, MOIST TO WET)
		SAMPLES WERE COLLECTED AT 5.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.5 FEET ON 8/26/2022

Project Number 1394722

Figure 6

Creekside Village Sieve Analysis

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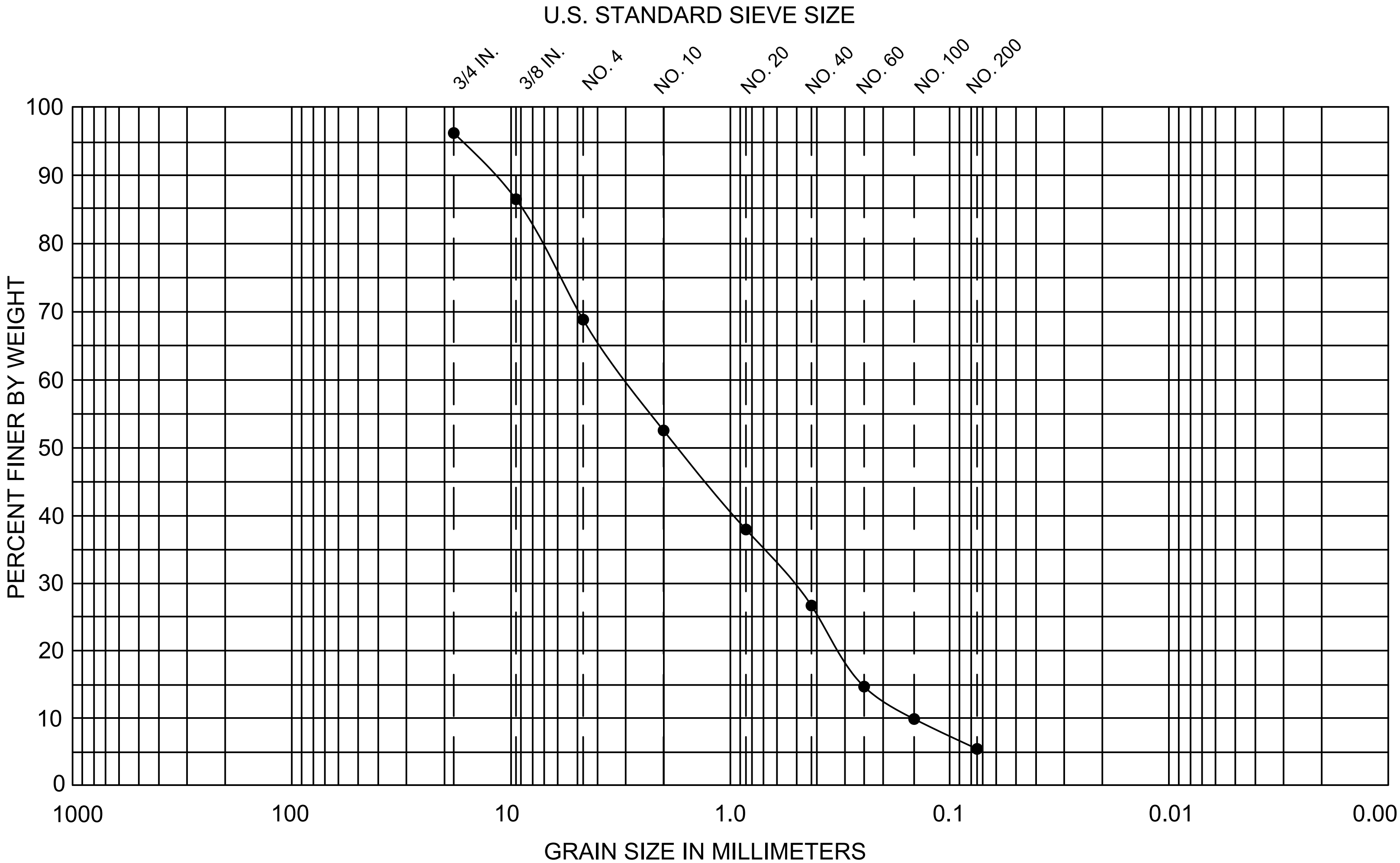
No.	Date	Revision	By	CK
1	9/12/22	Original	FKS	KSB



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	ITP-1	5.0 feet	Gray-brown, fine to coarse sand with gravel and silt	Gravel = 31% Sand = 63% Silt/Clay = 6%

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