REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Marysville Corporate Center Project
15908 47th Avenue NE
Marysville, Washington

For
PacTrust
March 25, 2022

Project: PacTrust-219-02

March 25, 2022

PacTrust
15350 SW Sequoia Parkway, Suite 300
Portland, OR 97224

Attention: Matthew Oyen, P.E.

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15908 47th Avenue NE
Marysville, Washington
Project: PacTrust-219-02

NV5 is pleased to present this report of geotechnical engineering services for the Marysville Corporate Center Project in Marysville, Washington.

We appreciate the opportunity to be of continued service to you. Please call if you have questions regarding this report.

Sincerely,

NV5

Kevin J. Lamb, P.E.
Principal Engineer

cc: Adam Solomonson, Mackenzie (via email only)

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One copy submitted (via email only)
Document ID: PacTrust-219-02-032522-geor.docx

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ATB	asphalt-treated base
ATPB	asphalt-treated permeable base
BGS	below ground surface
BMP	Best Management Practice
CPT	cone penetration test
DMFZ	Devils Mount fault zone
DOE	Washington State Department of Ecology
g	gravitational acceleration (32.2 feet/second^2)
GSP	General Special Provisions
HMA	hot mix asphalt
H:V	horizontal to vertical
km	kilometers
LID	low-impact development
MCE	maximum considered earthquake
NGA	Nelson Geotechnical Associates, Inc.
PCC	portland cement concrete
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
PIT	pilot infiltration test
psf	pounds per square foot
PVC	polyvinyl chloride
ROW	right-of-way
SFZ	Seattle fault zone
SMMWW	Stormwater Management Manual for Western Washington
SPT	standard penetration test
SWIFZ	South Whidbey Island fault zone
WISHA	Washington Industrial Safety and Health Act
WSDOT	Washington State Department of Transportation
WSS	Washington Standard Specifications for Road, Bridge, and Municipal Construction (2022)

1.0 INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for the Corporate Center Project in Marysville, Washington. The site is composed of two rectangular-shaped parcels encompassing an area of approximately 57.5 acres. The parcels are undeveloped open space used for agriculture and Hayho Creek borders the west side of the site. The site location relative to surrounding physical features is shown on Figure 1.

Proposed development includes construction of ten one- to two-story office/warehouse structures, along with associated infrastructure; AC-paved access drives and parking areas; and stormwater detention ponds (Figure 2). ROW improvements are also anticipated to support the extension of 156th Avenue NE.

The current conceptual plan includes two buildings (A1 and A2) on the south parcel and eight buildings (B1 through B8) on the north parcel. The buildings are anticipated to range from approximately 56,000 to 120,000 square feet, with loading docks on one side of each building. Approximately 6 to 10 feet of fill will be placed on site to raise site grades to facilitate drainage. Finish floor elevations are anticipated to range from 117.85 to 119 feet, which will require 6 to 8 feet of fill to be placed across the site. 156th Street NE will be extended from its terminus on the west side of the side southeastward to intersect 152nd Avenue NE (Levin Road). We understand it will be generally located between the two parcels and curve to the southeast through the south parcel.

Permeable pavement and other shallow infiltration facilities stormwater ponds and bioswale utilities are being considered as part of the stormwater management plan. We understand that permeable pavement, if used, will be limited to automobile parking areas around the perimeter of the buildings.

As part of our services, we competed subsurface explorations to provide geotechnical recommendations to support the proposed site development. Explorations logs and laboratory test results are presented in Appendix A, and results of the CPT completed on site to explore subsurface conditions and evaluate liquefaction susceptibility are presented in Appendix B.

Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 BACKGROUND

GeoDesign, Inc. (now NV5) previously provided geotechnical engineering services during the due diligence period of the project prior to purchase. The results of our earlier services are presented in Appendix C (GeoDesign, Inc., 2021).

Nelson Geotechnical Associates, Inc. (NGA) also completed a preliminary geotechnical engineering evaluation for the property prior to it being offered to PacTrust (NGA, 2019). The results of the NGA investigation are presented in Appendix D. The NGA investigation included excavating 15 test pits to a depth of 6 feet BGS. Soil conditions summarized on the exploration logs include a thick organic-rich topsoil/tilled zone extending to depths between 0.8 foot and 1.5 feet BGS. The topsoil is underlain by loose to medium dense, silty sand with varying amounts of gravel and iron oxidation staining (weathered recessional outwash) that varies between 0.9 foot and 2.2 feet thick. The underlying un-weathered recessional outwash material consists of gray sand with varying amounts of gravel. The recessional outwash sands, referred to locally as the Marysville Recessional Outwash, is consistent with the mapped geology for the area. Groundwater seepage was encountered in all the test pit explorations at depths between 2.8 and 5 feet BGS.

The test pits were generally limited to 6 feet in depth.

3.0 PURPOSE AND SCOPE

The purpose of our scope was to provide geotechnical engineering services for design and construction of the proposed development. The specific scope of our services included the following:

- Reviewed available information from previous geological and geotechnical studies conducted at and in the vicinity of the site.
- Reviewed preliminary grading plans.
- Coordinated utility locates, site access, and subconsultant services for our subsurface explorations.
- Conducted a subsurface exploration program that included the following:
  - Drilled one boring to a depth of 41.5 feet BGS. Installed a monitoring well in the boring to monitor groundwater conditions.
  - Drilled six hand auger borings to depths between 2 and 5 feet BGS. Installed two shallow groundwater monitoring wells in two of the hand auger borings.
  - Excavated four test pits to depths between 1.5 and 3.2 feet BGS. Completed small-scale PITs.
  - Advanced one CPT probe to a depth of 36.4 feet BGS (refusal).
- Conducted a laboratory testing program that included the following:
  - Moisture content determinations in general accordance with ASTM D2216
  - Grain-size distribution analysis in general accordance with ASTM C117, ASTM C136, and/or ASTM D1140
- Prepared this geotechnical report that summarizes our explorations and analyses and provides geotechnical design criteria and construction recommendations, including information relating to the following:
  - Subsurface conditions
  - Site preparation and grading, including over-excavation, general and temporary excavation, temporary and permanent slopes, fill placement and compaction criteria, suitability of on-site soil for fill, and subgrade preparation for buildings and pavement
  - Foundation recommendations
  - Recommendations for wet weather construction
  - Groundwater conditions, including recommendations for dewatering during construction
  - Liquefaction and lateral spreading potential

- AC recommendations for heavy-duty and light-duty pavement
- Seismic design criteria in accordance with ASCE 7-16

4.0 SITE CONDITIONS

4.1 GEOLOGY

The site is situated in the Puget Sound lowland north of Marysville, Washington, which is generally made up of Vashon Stade recessional outwash known as the Marysville Sand Member. The recessional outwash material was deposited by meltwater flowing south from the stagnating and receding Vashon glacier and is typically at least 20 meters thick. The recessional outwash is generally composed of well-drained, stratified to massive outwash sand with variable amounts of fine gravel.

4.2 SURFACE CONDITIONS

The approximately 57.5-acre site consists of former agricultural land. Remnants of a past residence and outbuildings are present at the southeast corner of the north parcel, which includes portions of concrete building slabs, small piles of concrete and wood debris, and an abandoned well.

Site access is provided by a gravel access road (47th Avenue NE). The site is relatively level and bordered by Hayho Creek to the west, which is contained within a ditch, and by agricultural land to the north, east, and south. The site is relatively flat with a slight slope to the west toward Hayho Creek. Geologic hazard areas are not present on or adjacent to the site.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

Subsurface conditions were evaluated by reviewing the test pit logs completed by NGA as discussed in the “Background” section, reviewing geologic maps, and by completing additional explorations on site to evaluate liquefaction susceptibility and stormwater infiltration. We also observed subgrade conditions during stripping of the site in the summer of 2021 for conformance with subsurface conditions encountered in the explorations.

NGA excavated 15 test pits in a grid pattern across the two parcels. The locations of the NGA test pits are shown on Figure 2. We reviewed the summary logs of the test pits and the laboratory test data on soil samples collected from the test pits.

Our subsurface exploration program consisted of drilling one boring (B-1) to a depth of 41.5 feet BGS, advancing one CPT probe (CPT-01) to depth of 36.4 feet BGS (refusal), drilling six shallow hand auger borings (HA-1 through HAS-6) to depths between 2 and 5 feet BGS, and excavating four shallow test pits (PIT-1 through PIT-4) to depths between 1.5 and 3.2 feet BGS. The approximate locations of our explorations are shown on Figure 2. The boring and test pit logs and laboratory test results for our investigation are presented in Appendix A. The CPT data is presented in Appendix B. The NGA investigation report, which includes test pit logs, is presented as Appendix C.

4.3.2 Soil Conditions

Subsurface conditions in the explorations are consistent with the mapped geology: agricultural tilled soil overlying recessional outwash deposits. The subsurface conditions encountered in the explorations are described below.

4.3.2.1 Topsoil/Tilled Zone

Topsoil/tilled zone was encountered at the ground surface in all exploration except B-1, HA-6, and PIT-1 and extends to depths between 0.8 foot and 1.3 feet BGS. It generally consists of very loose, silty sand with abundant organics and rootlets at generally all explorations locations.

4.3.2.2 Fill

Recently placed fill was encountered at the surface of B-1, HA-6, and PIT-1 to depth between 0.2 foot and 1.5 feet BGS. The fill generally consists of silty sand with gravel. Based on SPT blow counts, the encountered fill is medium dense.

4.3.2.3 Weathered Recessional Outwash – Marysville Sand

Weathered recessional outwash is present at all explorations locations at the ground surface or beneath the recent fill or tilled/topsoil zone and extends to depths between 0.9 foot and 5 feet BGS, except for PIT-3 and PIT-4. It typically consists of orange, gray, and brown, silty sand. Based on SPT blow counts, the weathered recessional outwash is loose.

Isolated pockets of rusty brown, sandy clay are also present near/at the surface of the weathered recessional outwash material and have been encountered during recent grading operations. The clay is generally soft and easily disturbed under construction traffic loading.

4.3.2.4 Recessional Outwash

The weathered recessional outwash material grades to un-weathered outwash at depths between 0.9 foot and 5 feet BGS. It is distinguished from the weathered material from a color change to gray. Its composition is similar and consists of gray sand with silt and fine gravel. Based on SPT blow counts, the recessional outwash is medium dense to dense.

4.3.3 Groundwater

Groundwater is present at shallow depth across the site and was encountered in most of our and all of NGA's explorations. Water levels fluctuate seasonally. During site stripping activities completed during the summer of 2021, groundwater was generally at a depth of 2 to 3 feet below the stripped ground surface. During the wet season, the groundwater level is generally at or within 1.5 feet of the original ground surface, as recorded in the NGA investigation dated December 2018, our investigation of December 2020, and our recent observations during earthwork activities during the wet season of 2021/2022.

Groundwater monitoring activities included installing a 2-inch-diameter PVC monitoring well in boring B-1 to a depth of 20 feet BGS. A 0.010-inch well screen was installed between 5 and 20 feet BGS. The annular space between the casing and well screen was backfilled with 10/20 silica sand. The well was completed at the surface with a cast iron flush monument. The Washington State Department of Ecology well tag for the well is number BND 356. Groundwater was measured in B-1 the day after the well was installed at 1.1 feet BGS.

Groundwater seepage was encountered in hand auger borings HA-1 and HA-2 at 1.75 feet and 1 foot BGS, respectively. Shallow standpipe piezometers were installed within hand auger borings HA-5 and HA-6 to monitor groundwater conditions. The standpipe piezometers consist of 1.5-inch diameter PVC with hand cut slots and installed to depths of approximately 5 feet BGS. Native soil was used to backfill around the pipe.

Groundwater measurements in explorations that have been completed on site are summarized in Table 1.

Table 1. Groundwater Observations

Exploration	Depth to Groundwater Below Original Ground Surface (feet BGS)			
	12/21/18	**12/11/20**	**1/19/21**	**2/21/22**
B-1		1.1		
HA-1		1.75		
HA-2		1		
HA-5			1	0.9
HA-6			1.5	1
PIT-1				0
PIT-2				0.5
PIT-3				0.8
PIT-4				0.8
Nelson Geotechnical Associates Groundwater Observations[1]				
TP-1	2.8			
TP-2	2			
TP-3	4			
TP-4	3			
TP-5	4			
TP-6	4.5			
TP-7	5			
TP-8	3			
TP-9	3			
TP-10	3			
TP-11	3			
TP-12	4			
TP-13	4			
TP-14	4			
TP-15	3			

1. NGA reported depth of moderate to rapid seepage during excavation. Actual groundwater surface may have been shallower.

4.4 INFILTRATION TESTING

Small-scale PITs were completed in PIT-1 through PIT-4 at the locations shown on Figure 2. Small-scale PITs were completed in general accordance with the SMMWW (DOE, 2019). At each location, a 5-foot-diameter steel casing was pushed below the ground surface to depths of up to 3.2 feet BGS. The casing was then filled with water to establish a head of 12 to 24 inches above the existing groundwater level. The drop in water level was monitored over time and the tests were repeated once the initial water level had dropped sufficiently. A summary of the infiltration test results is provided in Table 2.

Table 2. Infiltration Testing Results

Exploration	Depth (feet BGS)	Unfactored Infiltration Rate (inches per hour)	Soil Type
PIT-1	1.5	1.5	Sand with silt
PIT-2	2.3	0.4	Sand with silt
PIT-3	2.3	4.4	Sand with silt
PIT-4	2.2	2.9	Sand with silt

4.5 SEISMICITY

Washington State is situated at a convergent continental margin and is susceptible to subduction zone, intraplate, and shallow crustal source earthquakes. We reviewed published geologic maps for the site vicinity (Johnson et al., 1999; Sherrod et al., 2004) to evaluate seismic hazards. The site is approximately 13 miles south of the DMFZ, 19 miles north of the SWIFZ, and 40 miles north of the SFZ, which are the result of shallow crustal faulting.

The DMFZ is a 0.5- to 5-mile-wide zone extending from Vancouver Island to the Cascade Mountain Foothills (Johnson et al., 2001). The SWIFZ is a 4- to 7-mile wide, greater than 150-km zone extending from Vancouver Island to the Cascade Range (Liberty and Pape, 2006). The SFZ represents a 2- to 4-mile-wide zone, extending from the Kitsap Peninsula near Bremerton to the Sammamish Plateau. Within the SFZ are several east to west-trending fault splays of the Seattle fault (Johnson et al., 1999). The Seattle fault is thought to be a reverse fault, with the southern side “shoved up.” The SFZ is considered an active major fault and is capable of producing earthquakes of Magnitude ~7 with associated surface rupture and ground motions, posing a significant hazard to the Puget Sound Region (Sherrod et al., 2004). Geologic evidence indicates at least three episodes of movement on the fault within the last 10,000 years, with the most recent earthquake with surface rupture approximately 1,100 years ago (Nelson et al. 2000).

5.0 DESIGN

5.1 GENERAL

Based on our review of available information and the results of our explorations and analyses, it is our opinion that the site is suitable for construction of the proposed development.

- Shallow spread footing foundations bearing on a subgrade prepared as recommended below will provide adequate support for the proposed buildings.
- The building floor slabs can be supported on proposed fill thickness of 6 to 10 feet, provided the fill and subgrade are placed/prepared as recommended below.

- Near-surface soil generally consists of silty sand with a fines content generally in excess of 15 percent, which will be susceptible to deterioration during wet weather. Subgrade stabilization measures will be required prior to placement of fill.
- Based on our explorations and proposed site grades, we understand that the bottom of proposed foundations will be approximately 3 feet above existing site grades; as such, significant groundwater seepage is not anticipated during excavation for foundations. Where utilities or other excavations extend below the existing ground surface, groundwater seepage and soil caving or instability should be expected along with subgrade stabilization measures.
- The Puget Sound area is a seismically active region. The medium dense to dense outwash underlying the site is not susceptible to amplified earthquake ground motions and is not susceptible to liquefaction or lateral spreading. We did not observe evidence of faults on the site in the explorations or on geologic maps of the area and have concluded that the probability of surface rupture is low. We have provided appropriate seismic design recommendations based on the ASCE 7-16 criteria.
- The un-weathered and weathered recessional outwash (Marysville Sand) generally has a moderate to high infiltrative capacity. The groundwater level is near the existing ground surface during the wet season and adequate separation will need to be maintained below infiltration systems. Fill being used to raise site grades generally has a high fines content and cement-amended soil has recently been constructed on site. These soils have a significantly lower permeability than the underlying recessional outwash material. At infiltration system locations, it will be necessary to excavate through the fill material to expose the underlying recessional outwash material and then backfill with a clean sand filter layer to provide adequate separation, for water quality treatment, and to maintain the infiltration rate similar to that of the recessional outwash. Overflow relief should be provided at all infiltration system locations.
- Shallow infiltration using systems that do not concentrate flows (such as permeable pavement) can be used in most areas of the site, without excavation to remove fill placed to establish site grades.

Our specific recommendations and design guidelines for development of the site are presented in the following sections. These should be incorporated into the design and implemented during construction of the proposed development.

5.2 SEISMIC DESIGN CRITERIA

Moderate to high levels of earthquake shaking should be anticipated during the design life of the buildings and they should be designed to resist earthquake loading in accordance with the appropriate code-based methodology described in ASCE 7-16. The recommended seismic design parameters are presented in Table 3.

Table 3. Seismic Design Parameters (ASCE 7-16)

Seismic Design Parameter	Short Period	1 Second Period
MCE Spectral Acceleration	S_s = 1.072 g	S_1 = 0.383 g
Site Class	D	
Site Coefficient	F_a = 1.2	F_v = 1.8
Adjusted Spectral Acceleration	S_{MS} = 1.287 g	S_{M1} = 0.689 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.858 g	S_{D1} = 0.459 g

Based on our subsurface explorations, literature review, and experience, a summary of the seismic hazards in the area and their associated impact at the site are as follows:

- **Amplification:** Areas subject to amplification are typically soft soil overlying stiff soil or bedrock. Based on our explorations and available geologic maps, the site is underlain by medium dense to dense granular deposits. In our opinion, this material has a low potential for site amplification.
- **Liquefaction/Settlement:** Based on the results of the site explorations, the site is mostly underlain by medium dense to dense outwash deposits consisting of sand with variable amounts of gravel, and groundwater was encountered at a shallow depth. Granular soil, which relies on interparticle friction for strength, is susceptible until the excess pore pressures can dissipate. Liquefaction analysis was performed using the information from boring B-1 and CPT-01, laboratory test results, and earthquake hazard mapping. Based on our analysis, we estimate the potential for liquefaction is low for the site, and liquefaction-induced settlement based on the design earthquake is negligible.
- **Lateral Spreading:** Areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediments adjacent to an open face (such as riverbanks or bay fronts). Liquefied soil adjacent to open faces may "flow" in that direction, resulting in lateral displacement and surface cracking. There is no potential for the site to be affected by lateral spreading.
- **Fault Surface Rupture:** We did not find evidence of faults through the site or on maps of the area. We conclude that the potential for fault surface rupture at the site is low over the life of the structures.

5.3 FOUNDATION SUPPORT – SHALLOW SPREAD FOOTINGS

5.3.1 General

Conventional shallow spread footings bearing on undisturbed recessional outwash or on structural fill placed over the recessional outwash will provide adequate support for the anticipated column or perimeter foundation loads.

Based on current grading plans, we anticipate approximately 6 to 10 feet of embankment fill (structural fill) will be placed and compacted to establish the planned site grades. Embankment fill should conform and be placed in accordance with the recommendations presented in the "Fill Materials" section.

5.3.2 Dimensions and Capacities

Continuous and isolated spread footings should be at least 18 and 24 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the adjacent exterior grade for frost heave protection, and interior footings should be at least 12 inches below the top of the slab.

Foundations supported on properly compacted embankment fill or recessional outwash material may be designed for an allowable bearing pressure of 2,500 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third for short-term loads, such as those resulting from wind or seismic forces.

5.3.3 Resistance to Sliding

Wind, earthquake, and unbalanced earth loads will subject the proposed structures to lateral forces. Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the footings. An allowable passive resistance may be calculated as a triangular equivalent fluid pressure distribution using an equivalent fluid density of 300 pcf, provided the footings are cast directly against properly placed and compacted structural fill and the footing is above the groundwater table.

Where footings extend below the groundwater table, a passive resistance of 175 pcf should be used.

Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. For footings in contact with granular backfill, a coefficient of friction equal to 0.35 may be used. A safety factor of 1.5 has been applied to the recommended sliding friction and passive pressure.

5.3.4 Settlement

Based on our analysis, total post-construction static (consolidation-induced) settlement for conventional and semi-rigid foundation systems should be less than 1 inch, with differential settlement of up to ½ inch.

5.4 CONCRETE SLAB-ON-GRADE

Concrete slab-on-grade floor slabs should be constructed over a 12-inch-thick layer of floor slab base rock, as defined in the "Fill Materials" section. This section is recommended based on the anticipated silty sand composition of the fill materials and its susceptibility to deterioration during wet weather. The upper 4 inches of the aggregate base rock should consist of a layer of capillary break material.

Where concrete slabs are designed as beams on an elastic foundation, the properly prepared subgrade should be assumed to have a modulus of subgrade reaction of 150 pci.

A vapor barrier product (such as Vapor Block BB-10 or VB-15) should be placed directly over the floor slab base rock. Edges of the vapor barrier, between adjoining pieces, should be properly sealed.

We recommend that exterior slabs, such as those for walkways, be structurally independent from the foundation of the structure. This will allow minor movement of the slabs to occur as a result of vehicular loading, tree root growth, seasonal soil shifting, and other factors, while reducing the potential for slab cracking around the perimeter. Interior slabs may be tied to the foundation system of the structure.

5.5 RETAINING WALLS

5.5.1 General

The following recommendations should be used for design of retaining walls for site grading and loading dock walls. Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered or gravity walls; (2) the walls are less than 8 feet in height; (3) the backfill is drained and consists of imported granular material; and (4) the backfill has a slope flatter than 4H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

Walls located in level ground areas should be founded at a depth of 18 inches below the adjacent grade. If the ground descends in front of the wall up to 2H:1V, a minimum embedment depth of 4 feet is required. A 4-foot-wide, horizontal bench should be planned in front of retaining walls to facilitate construction, develop passive soil resistance, and prevent impacts to slopes.

5.5.2 Design Parameters

Lateral earth pressures for design of retaining structures and below-grade structures, such as a detention vault, should be estimated using an equivalent fluid density of 35 pcf, provided the walls will not be restrained against rotation when backfill is placed. If the walls will be restrained from rotation (i.e., detention vault and basement walls that are internally braced by the roof or first floor slab), we recommend using an equivalent fluid density of 50 pcf. Walls are assumed to be restrained if top movement during backfilling is less than H/1,000, where H is the wall height.

Static lateral earth pressures acting on walls should also be increased to account for seismic loading. The seismic pressure should be estimated as follows:

- For yielding retaining walls and active soil conditions, a value of six times the height of the wall: 6H (psf)
- For rigid, non-yielding walls and at-rest soil conditions, a value of nine times the height of the wall: 9H (psf)

The height of the wall used in the above equations should be measured from the finished ground surface in front of the wall to the top of the wall. The seismic pressure for cantilever retaining walls should be applied as a uniform rectangular pressure from the top of the wall to the elevation of the finished ground surface in front of the wall, and the resultant should be applied at 0.6H of the exposed wall height.

The recommended lateral earth pressures do not account for surcharges. If surcharges (e.g., building foundations, vehicles, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads. An additional 2 feet of fill, representing a typical traffic surcharge, should be included in the design if vehicles are allowed to operate a horizontal distance equal to the height of the wall.

These recommendations are based on the assumption that adequate drainage will be provided behind below-grade walls and retaining structures, as discussed below. The values for soil bearing, frictional resistance, and passive resistance presented above for foundation design are applicable to retaining wall design.

5.5.3 Retaining Wall Foundations

Retaining wall bearing surfaces should be prepared as recommended above for shallow foundations. Retaining wall foundations may be designed using the allowable bearing pressure and lateral resistance values provided in the "Foundation Support – Shallow Spread Footings" section. We estimate settlement of the wall will be similar to values provided in the "Settlement" section: less than 1 inch, with differential settlement of up to ½ inch along the wall alignment.

5.5.4 Drainage

Positive drainage should be provided behind below-grade walls and retaining walls by placing a minimum 1.5-foot-wide zone of free-draining backfill directly behind the wall. The free-draining backfill should meet the criteria for WSS 9-03.12(4) – Gravel Backfill for Drains. The free-draining backfill zone should extend from the base of the wall to within 2 feet of the finished ground surface. The top 2 feet of fill should consist of relatively impermeable or native soil to prevent infiltration of surface water into the wall drainage zone.

A minimum 4-inch-diameter, perforated drainpipe should be installed within the free-draining material at the base of walls. The drainpipe should consist of smooth-walled, perforated or slotted PVC pipe. The pipes should be laid with minimum slopes of 0.5 percent and routed to a suitable discharge location. The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush-mount access boxes. We recommend against discharging roof downspouts into the perforated pipe providing wall drainage. Collected downspout water should be routed to appropriate discharge points in separate pipe systems.

For exterior walls where seepage at the face of a wall is not objectionable, the walls can be constructed with weep holes to discharge water from the free-draining wall backfill material. The weep holes should be a minimum of 3 inches in diameter and spaced approximately every 8 feet center-to-center along the base of the walls. The weep holes should be backed with galvanized heavy wire mesh to help prevent loss of the backfill material.

5.5.5 Retaining Wall Backfill

Backfill should be placed and compacted as recommended for structural fill and retaining wall select backfill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 92 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

5.5.6 Settlement

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork within a horizontal distance equal to the height of the wall be postponed at least four weeks after construction, unless survey data indicates that settlement is complete prior to that time.

5.6 PAVEMENT DESIGN – DENSE AC

5.6.1 General

We anticipate soil conditions exposed at the surface of the embankment fill will consist of silty sand with gravel that is moisture sensitive and that will deteriorate in wet weather. The exposed subgrade beneath paved areas should generally be prepared as recommended in the "Subgrade Preparation" section. If paving cannot be completed in the summer months, additional subgrade preparation and a thicker pavement section will be required, as recommended below.

We recommend using dense AC pavement to construct access/driveway roadways (heavy-duty pavement section) and parking areas (light-duty pavement section). We understand that the access roadway and loading dock areas will be subjected to daily truck traffic and parking areas will be limited to automobile traffic.

The dense AC should be Class B PG 58V-22, with ½-inch aggregate, gradation, and asphalt requirement in accordance with the specifications provided in WSS 9-03.8(6) – HMA Proportions of Materials and compacted to 91 percent of the maximum specific gravity of the mix, as determined by ASTM D2041. Minimum lift thickness for ½-inch HMA is 1.5 inches. Asphalt binder should be performance graded and conform to PG 58V-22. The aggregate base material should meet the specifications for aggregate base rock provided in the "Embankment Fill/Structural Fill" section. The subgrade should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

These recommendations are based on general assumptions regarding anticipated traffic and assume adequate subgrade and drainage conditions. Pavement material and placement should conform to the WSS (2022). We recommend the following pavement sections.

5.6.2 Heavy-Duty Pavement

We recommend a pavement section consisting of 4 inches of AC over 8 inches of 1¼-inch-minus crushed rock (base course) in accordance with WSS 9-03.9(3) – Crushed Surfacing. Alternatively, an applicable pavement section using ATB should consist of 4 inches of ATB and 4 inches of AC.

5.6.3 Light-Duty Pavement

In areas limited to automobile traffic only, we recommend a pavement section consisting of 2.5 inches of AC over 6 inches of 1¼-inch-minus crushed rock in accordance with WSS 9-03.9(3) – Crushed Surfacing. Alternatively, an applicable section using ATB would consist of 3 inches of ATB and 2 inches of AC.

5.6.4 Wet Season Paving

During the wet season, additional measures will likely be necessary to construct a stable subgrade on which to pave. This can be accomplished through increasing the aggregate section thickness in paved areas to 16 inches or through cement amendment of the subgrade and a reduced thickness of aggregate.

Applicable pavement sections using a cement-amended subgrade can consist of the following:

Heavy-Duty Pavement:

- 4.0-inch-thick, ½-inch, dense HMA
- 4.0 inches aggregate base course
- 12.0-inch-thick cement-amended subbase

Light-Duty Pavement:

- 2.0-inch-thick, ½-inch, dense HMA wearing course (one lift)
- 4.0-inch-thick aggregate base
- 12.0-inch-thick cement-amended subbase

We estimate a cement content of 4 percent based on our experience with fill soil consisting of silty sand and gravel. The amount of cement added to the soil may need to be adjusted based on field observations and performance. In addition, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

Information regarding mixing, compacting, and construction should be included in the specifications; however, we recommend the following additional considerations:

- Cement amendment should occur during a period of dry weather.
- Grading should not be attempted at greater than three hours after initial tilling of the cement-soil mixture.
- Paving within four hours of final grading or application of a curing sealant (e.g., emulsion) and a minimum curing period of four days prior to placement of the AC pavement.
- During curing, the area should be closed to construction vehicle traffic.

5.7 PERMEABLE PAVEMENT

We understand porous HMA or pervious PCC pavement may be incorporated into hardscape areas to address stormwater management. Recommendations for the use of permeable pavement in walkway or light-duty parking areas are provided below.

5.7.1 Recommended Pavement Section

Appropriate permeable pavement sections composed of pervious PCC or permeable HMA, based on the assumed traffic loading for parking areas, are provided in Table 4.

Table 4. Permeable Pavement Sections

Layer	Porous HMA Section (inches)	Alternate Porous HMA Section (inches)
Permeable HMA		
Porous Asphalt Wearing Layer	2[1]	3[1]
ATPB	3	--
Choker	--	2 maximum
Storage Aggregate	6 minimum	8 minimum
Pervious PCC		
Pervious Concrete Slab	7	--
Storage Aggregate	5 minimum	--

1. For driveway areas, the recommended thickness shown in the table should be increased by a minimum of 1 inch.

The use of a choker course is provided under "Alternate Porous HMA Section" in Table 4. A choker course layer will facilitate grading; without it, the exposed storage aggregate is susceptible to rutting under the dump trucks and may require hand grading during paving operations. The thickness of the storage aggregate layer is a minimum thickness required for structural support of the pavement. The thickness may need to be increased based on hydraulic storage requirements.

5.7.2 Subgrade Preparation

Subgrade below permeable pavement areas can be sloped up to approximately 2 percent but should be relatively flat, if possible, to prevent uneven ponding of water within the storage aggregate. On sloping sites, the subgrade can be stepped, and the lowest step should be flat or sloped back into the slope 1 to 2 percent to help decrease downslope seepage from the storage aggregate layer.

Prior to placing reservoir rock, the exposed subgrade should be scarified to a depth of 12 inches and compacted to a firm condition under the direction of the geotechnical engineer prior to placing the storage aggregate. We recommend compacting the exposed subgrade to between 90 and 92 percent of the maximum dry density, as determined by ASTM D1557.

If soft areas are identified during subgrade preparation or areas deflect under construction equipment traffic, the material should be excavated and replaced with storage aggregate.

Utilities within the parking area should be backfilled with storage aggregate or alternatively clean sand and gravel fill meeting WSS 9-03.12(2) – Gravel Backfill for Walls. Trench dams should be placed intermittently to reduce lateral flow within the pipe bedding. Trench dams can be constructed using native silty sand and gravel, controlled density fill, or lean-mix concrete.

A geotextile should be placed between the storage aggregate and the underlying subgrade for separation. A heavy-duty geotextile with high permittivity and flow rate should be used beneath the roadway, as specified in the "Permeable Pavement Materials" section.

Exposed subgrades will be moisture sensitive and deteriorate under construction traffic loading during wet conditions. If earthwork construction is expected to extend into the wet season, we recommend limiting the size of the work area and stabilizing the exposed surface by placing the storage aggregate to protect the subgrade. Construction traffic should be minimized or restricted from trafficking over the permeable pavement subgrade.

After subgrade preparation measures are completed, the infiltration rate of the prepared subgrade should be verified through in-situ infiltration tests using small-scale PITs in accordance with test procedures provided in Puget Sound Partnership (2012). We can provide an average short-term rate that the verification tests should meet after we complete in-situ infiltration tests to support the design of LID BMP elements.

5.7.3 Permeable Pavement Materials

5.7.3.1 Pervious PCC

Pervious PCC typically consists of a near-zero-slump concrete consisting of portland cement, coarse aggregate with little to no fines, various admixtures, and water. Design of the mix should conform to ACI 522.1-08 specification (ACI, 2013). We recommend a maximum of ½-inch aggregate for roadway applications; however, other aggregate sizes may be preferred depending on the desired surface texture.

5.7.3.2 Porous HMA

AC used for porous HMA pavement should be designed as a ½- to ¾-inch nominal, open-graded HMA. Selection of the preferred aggregate size should be based on the desired surface texture and the required layer thickness limitations. Approximate "broad band" gradations for recommended aggregate gradation for porous HMA are provided in Table 5.

Table 5. Porous HMA Gradation (3/8 inch)

Sieve Size	3/8 inch Percent Passing	½ inch Percent Passing	¾ inch Percent Passing
1 inch	--	--	99 – 100
¾ inch	--	100	85 – 96
½ inch	99 – 100	90 – 98	55 – 71
3/8 inch	90 – 100	55 – 90	--
#4	22 – 40	10 – 40	10 – 24
#8	5 – 15	0 – 13	6 – 16
#200	0 – 3	0 – 3	0 – 3
Recommended Maximum Layer Thickness (inches)	2.5	3	4

The actual mix design should be completed under the direction of a competent mix design technician familiar with the WSDOT mix design procedures. The asphalt binder to construct porous HMA pavement should be PG 70-22ER.

The preferred and recommended asphalt binder is PG 70-22ER (polymer modified); however, its availability can be limited because some of the local asphalt suppliers limit their on-hand binder to PG 64-22. PG 70-22ER is available but is typically stocked by asphalt suppliers for a specific project, which requires pre-ordering it so that it is available when needed. Suppliers prefer a project size of approximately 600 tons of asphalt in order to use a complete tanker volume of the binder. Its availability is further restricted to the warm months of the year because of its stiffness, so it is not readily available between October and May. Projects specifying PG 70-22ER should be scheduled accordingly and specifications should address supplier availability.

The binder should be between 6.0 and 6.5 percent of the pavement section by weight of total (dry aggregate) mix.

Warm-mix asphalt technology with a proper mix design and appropriate additives can be used to construct the porous HMA. Use of the warm-mix additives may require a longer curing time for the asphalt prior to allowing cars to traffic over the surface.

Compaction of the porous HMA should consist of approximately two to four complete passes by an 8-ton, dual-steel roller compactor working in static mode only. Compaction of the porous HMA should be to a target air voids content of 15 to 18 percent (82 to 85 percent of maximum theoretical [Rice] density). A nuclear density gage should be used to monitor compaction.

We recommended that porous HMA specifications are prepared in conformance with those approved by the APWA-WA Construction Materials Committee. The specifications have now been integrated into the WSDOT Local Agency GSPs and are now available at http://www.wsdot.wa.gov/partners/apwa/Division_5_Page.htm.

5.7.3.3 Choker Aggregate

Imported granular material used as choker aggregate beneath permeable pavement should be clean crushed rock that meets a No. 57 size gradation according to AASHTO M 43, as provided in Table 6.

Table 6. Permeable Pavement Choker Aggregate (AASHTO No. 57)

Sieve Size	Percent Passing
1½ inches	100
1 inch	95 – 100
½ inch	25 – 60
No. 4	0 – 10
No. 8	0 – 5

The percent fracture should be a minimum of 75 percent and a minimum of two fracture faces.

Alternatively, aggregate for bituminous surface treatment [WSS 9-03.4(2) – Grading and Quality], 5/8-inch or 3/4-inch washed crushed rock, which is available from local suppliers, will also be suitable. The aggregate should have at least two mechanically fractured faces.

5.7.3.4 Storage Aggregate

Imported granular material used as storage aggregate beneath pervious pavement should be clean crushed rock or crushed gravel and sand that meets a No. 2 or No. 3 size gradation according to AASHTO M 43 or clean crushed rock that conforms to WSS 9-03.9(2) – Permeable Ballast. Recommended gradations for acceptable storage aggregate are provided in Table 7.

Table 7. Storage Aggregate

Sieve Size	AASHTO No. 2 Percent Passing	AASHTO No. 3 Percent Passing	WSS 9-03.9(2) – Permeable Ballast Percent Passing
2 ½ inches	100	100	90 – 100
2 inches	35 – 70	90 – 100	65 – 100
1 ½ inches	0 – 15	35 – 70	--
1 inch	--	0 – 15	40 – 80
¾ inch	0 – 5	--	--
½ inch	--	0 – 5	--
No. 4	--	--	0 – 5

"Rail ballast" or "clean ballast" products available from local quarries will typically meet the AASHTO gradation criteria. The percent fracture should be greater than 75 percent to improve interlocking between fragments, and the aggregate should have a minimum WSS degradation value of 30. We anticipate that the storage aggregate gradations specified above will have between 35 and 40 percent voids compaction in the field.

The storage aggregate should be placed in one lift and compacted to a firm and unyielding condition. Over-compaction and construction traffic should be avoided.

5.7.4 Subgrade Geotextile

A layer of geotextile fabric should be placed as a barrier between the native soil subgrade and the pavement storage aggregate. Beneath drive lanes, a heavy-duty geotextile, such as Mirafi RS380i, should be used and equivalent products should conform to WSS 9-33.2(1) – Geotextile Properties, Table 4, Permanent Erosion Control, High Survivability, Woven and Table 5, Class A. Elsewhere, the geotextile should conform to the specifications for non-woven separation material provided in WSS 9-33.2(1) – Geotextile Properties, Table 3, Geotextile for Separation. The geotextile should be installed in conformance with the specifications provided in WSS 2-12 – Construction Geosynthetic.

5.8 DRAINAGE

5.8.1 Temporary

During work at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

5.8.2 Surface

The ground surface at finished pads should be sloped away from their edges at a minimum 2 percent gradient for a distance of at least 5 feet. Roof drainage from the buildings should be directed into solid, smooth-walled drainage pipes that carry the collected water to the storm drain system.

5.8.3 Subsurface

Perimeter footing drains should be installed around continuous perimeter wall foundations. Drains should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 12 inches below the lowest adjacent grade (i.e., slab subgrade elevation). A perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and filter fabric should meet specifications outlined in the “Fill Materials” and “Geosynthetics” sections. Discharge for footing drains should not be tied directly into the stormwater drainage system, unless mechanisms are installed to prevent backflow.

5.8.4 Stormwater Infiltration Systems

Infiltration testing was completed in test pits PIT-1 through PIT-4 at depths between 1.5 feet and 2.3 feet BGS. The short-term infiltration rate ranged from 0.4 inch per hour to 4.4 inches per hour as shown in Table 2. Based on the explorations and testing, on-site infiltration systems are feasible, provided a minimum of 5 feet of separation can be provided between the groundwater table and the bottom infiltration facilities, in accordance with requirements presented in the SMMWW (DOE, 2019). Separation may be decreased down to 3 feet pending the results of a groundwater mounding study.

The SMMWW (DOE, 2019), adopted by the City of Marysville, provides correction factors to be applied to measured short-term infiltration rates.

- Correction factor CF_V accounts for variability in subsurface conditions between testing locations. The SMMWW recommends a range of 0.33 to 1.0. We recommend a factor of 0.9 be applied.
- Correction factor CF_t accounts for uncertainty in the test method. A correction factor of 0.5 is recommended by the SMMWW for small-scale PITs.
- Correction factor CF_m accounts for reduction infiltration rates over the long term due to siltation and bio-buildup. The SMMWW recommends a correction factor of 0.9.

The total correction factor to be applied is obtained by multiplying the individual correction factors. We recommend a cumulative correction factor of 0.40 be applied to the measured short-term infiltration rates. The infiltration test results along with the correction factor are summarized in Table 8.

Table 8. Soil Infiltration Rate Analysis[1]

Infiltration Location	Soil Type	Measured Short-Term Infiltration Rate (inches per hour)	Long-Term Design Infiltration Rate[1] (inches per hour)
PIT-1	Sand with silt	1.5	0.6
PIT-2	Sand with silt	0.4	0.2
PIT-3	Sand with silt	4.4	1.8
PIT-4	Sand with silt	2.9	1.2

1. Based on the recommended combined correction factor of 0.40 in accordance with the 2019 SMMWW.

We understand that the base of infiltration systems will be within the embankment fill material in order to maintain adequate separation from the shallow groundwater layer. We recommend fill placed below infiltration systems have an infiltration rate similar to the underlying Marysville Sand. Materials meeting this requirement include sand and gravel or sand with less than 5 percent fines. This material will also likely provide water quality treatment and may require amendment to meet the cation exchange capacity and organic matter requirements. We recommend using a long-term design infiltration rate of 1 inch per hour.

5.9 PERMANENT SLOPES

Permanent cut and fill slopes, created in the embankment fill material, should not exceed 2H:1V. Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as *soon* as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.10 TEMPORARY DEWATERING

Groundwater should be anticipated at approximately 1 foot BGS across the site. Where excavations extend below this depth, groundwater should be anticipated.

The contractor will be responsible for selection and design of the dewatering system. The contractor's dewatering methods should be capable of maintaining groundwater levels at least 2 feet below the base of the excavation (including the depth required for trench bedding and stabilization material). Perched water may be encountered in other areas and groundwater may be encountered within the trench along the existing utilities due to preferential flow within the pipe bedding. We anticipate shallow sumps within the excavation will be enough for managing the flow at most locations.

Flow rates for dewatering are likely to vary depending on location, soil type, and the season in which the excavation occurs. Dewatering systems should be capable of adapting to variable flows. We note that these recommendations are for guidance only. Dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select these systems based on their means and methods. The contract plans and specification should address and identify a suitable dewatering discharge location and allowable quantities. If sumps within the excavation are used, the discharge water will likely have a high turbidity and require detention prior to disposal.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 GENERAL

The project area is undeveloped and consists of agricultural land. Site preparation will generally include removal of the existing topsoil or tilled zone, fill placement, and site grading to the required subgrade elevations.

6.2 SITE PREPARATION

Site preparation activities will include removing vegetation and undesirable material, site grading, excavation, and subgrade preparation and stabilization.

6.2.1 Grubbing and Stripping

Where present, topsoil or tilled zones within the footprint of structural improvements should be stripped and removed. We anticipate topsoil or tilled zone thickness will be between 1 foot and 1.5 feet with an average depth of approximately 1.25 feet, although greater stripping depths will be required to remove localized zones of organic soil. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

6.2.2 Subgrade Preparation

After grubbing and stripping, site grading should be completed to the required elevations. Based on the results of our explorations, we anticipate variable soil conditions will be exposed across the site consisting of limited amounts of loose tilled zone soil and weathered recessional outwash.

Where recessional outwash is encountered, subgrade preparation should consist of compacting any disturbed exposed material at the surface. The exposed subgrade will generally consist of silty sand and gravel with variable fines content. The subgrade will be moisture sensitive and deteriorate under construction traffic loading during wet weather.

After stripping, we recommend cement amending the exposed subgrade to stabilize it for supporting truck traffic and embankment fill placement. We estimate a cement content of 4 to 6 percent based on our experience with fill soil consisting of silty sand.

6.2.3 Site Grading

Site grading will generally consist of placement of approximately 6 to 10 feet of embankment fill to achieve site grades. The soil at the site can be excavated with conventional earthwork equipment. Excavations extending below the groundwater table (greater than approximately 1 foot below the original ground surface) will likely experience caving and sloughing.

Fill in improved areas should consist of embankment fill as defined in the "Fill Materials" section. The use of on-site excavation spoils as embankment fill will be dependent on the material composition and weather conditions. We anticipate that the recessional outwash material will be suitable for use but will be limited to use during the dry season. It will be prudent to provide a 12-inch-thick cap of crushed rock aggregate as the last lift of embankment fill where moisture-sensitive materials are used for fill.

We recommend capping the embankment fill surface within building areas and areas where pavement is planned with a 12-inch-thick gravel pad or stabilizing with cement-amended soil overlain by 4 inches of crushed rock.

6.2.4 Excavation

The soil at the site can be excavated with conventional earthwork equipment. Excavations should stand vertical to a depth of approximately 4 feet, provided groundwater is not encountered.

Open excavation techniques may be used to excavate utility trenches with depths greater than 4 feet, provided the walls of the excavation are cut at appropriate cut slopes determined by the contractor. Approved temporary shoring is recommended where sloping is not possible. If a conventional shield is used, the contractor should limit the length of open trench. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the plan of operation and the subsurface conditions. All excavations should be made in accordance with applicable WISHA regulations.

Dewatering of excavations below the groundwater table will be required and we anticipate large-diameter wells or vacuum well points will be suitable for the conditions. If a conventional shield is used, the contractor should limit the length of open trench. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the plan of operation and the subsurface conditions.

6.2.5 Subgrade Verification

Exposed subgrades should be evaluated by a representative from NV5 to verify conditions are as anticipated and will provide the required support. Where pavement or hardscaped areas will be constructed, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similar heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas. Beneath foundations and during wet weather, subgrade evaluation should be performed by probing with a foundation probe. If soft or loose zones are identified, these areas should be excavated to the extent indicated by the engineer or technician and replaced with structural fill or stabilization material.

6.3 FILL MATERIALS

Fill material will be required for site grading, backfilling over-excavations, pavement support, installation of utilities, and drainage. The recommended fill materials are discussed below.

6.3.1 On-Site Soil

The site is relatively flat, and development involves constructing an embankment fill over the existing area. We do not anticipate a significant volume of on-site soil will be generated during site grading or utility construction. On-site soil that consists of the recessional outwash material (silty sand to sand) can be used as fill, provided it can be moisture conditioned to achieve compaction requirements and is free of deleterious material (such as wood, organic material, and man-made material). The use of on-site soil as fill should be subject to review and approval by NV5

6.3.2 Off-Site Recycled Fill Material

Off-site-generated recycled material should not be used on site without approval from the geotechnical engineer and acceptance by the owner. The use of recycled material will be subject to performance criteria, gradation requirements, and hazardous material testing in conformance with WSS 9-03.21(1) – General Requirements. Provided performance, gradation, and hazardous material testing results are acceptable, recycled material may be suitable for use beneath hardscape areas outside of the building footprint.

6.3.3 Embankment Fill/Structural Fill

Fill used for site grading to raise site grades to the required elevations for building construction can consist of embankment fill or structural fill.

Embankment fill placed for general site grading in improved areas should consist of clean soil classified as GP, GW, GM, SP, SW, SM, SC, and ML under the Unified Soil Classification System. It should be free of roots, sod, or other organic material; man-made debris; rubbish; or other deleterious materials. It should have maximum dry density of at least 115 pcf, as determined by the modified Proctor compaction test (ASTM D1557). Existing site soil that meets these criteria should be acceptable for use as structural fill, although wet and dry soil may require moisture adjustment prior to use.

The above material classified as GM, SM, SC, and ML will be susceptible to moisture and where left exposed will deteriorate rapidly during wet weather. Their use will require additional measures that may require significant earthwork to remove, moisture condition, and/or stabilize these soil types.

The use of structural fill consisting of granular, free-draining material will increase the workability of the material during the wet season and the likelihood that the material can be placed and adequately compacted. Imported material classified as GP, GW, SP, and SW is less susceptible to moisture conditions and are less likely to deteriorate during wet weather. We recommend capping the embankment with structural fill or stabilization material. Structural fill should consist of naturally occurring pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in WSS 9-03.14(1) – Gravel Borrow, with the exception that the percentage passing the U.S. Standard No. 200 sieve does not exceed 5 percent by dry weight.

Embankment fill and structural fill should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.3.4 Common Fill

Fill placed outside of areas where improvements or pavement area planned, where structural support is not required (such as planters, landscaped areas, and detention ponds) is defined as "common fill." Common fill may contain a higher concentration of fines and organic material than structural fill but should be free of man-made material. Imported common fill should meet the specifications provided in WSS 9-03.14(3) – Common Borrow. Fill placed in non-structural areas should be compacted to a minimum of 90 percent of the maximum dry density, as determined by ASTM D1557.

6.3.5 Floor Slab, Hardscape and Pavement Base Course

Imported granular material used as aggregate base for pavement and beneath hardscape areas should consist of 1½-inch-minus material meeting the specifications provided in WSS 9-03.9(3) – Crushed Surfacing, with the exception that the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve and at least two mechanically fractured faces. The imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.3.6 Trench Backfill

Trench backfill for utilities should consist of and be compacted in accordance with the specifications for structural fill in improved areas and for common fill in non-structural areas. Trenches within the ROW should be bedded and backfilled with 5/8-inch-minus screened crushed rock meeting the specifications provided in WSS 9-03.9(3) – Crushed Surfacing.

Trench backfill within the zone of influence of adjacent or overlying foundations should be backfilled with controlled density fill.

Trench bedding material should also consist of 5/8-inch-minus screened crushed rock meeting the specifications provided in WSS 9-03.9(3) – Crushed Surfacing.

6.3.7 Stabilization Material
Stabilization material to cap the surface of the embankment, to backfill over-excavations, or to stabilize soft subgrade areas may consist of either of the following:

- WSS 9-03.9(2) – Permeable Ballast
- WSS 9-13.7(2) – Backfill for Rock Wall

The initial lift of stabilization material used to fill over-excavations should be 18 inches thick and compacted to a firm condition. Successive lifts should be 12 inches thick and compacted to a dense and unyielding condition.

6.3.8 Drain Rock
Drain rock used in infiltration systems, subsurface drains, or against retaining walls should consist of granular material with a maximum particle size of 1 inch and should meet the specifications provided in WSS 9-03.12(4) – Gravel Backfill for Drains. The material should be free of roots, organic material, and other unsuitable material and should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis).

6.4 GEOSYNTHETICS
We have recommended the use of geotextiles for stabilizing the base of over-excavations or soft subgrade areas below hardscape or pavement when soft, wet, or saturated soil conditions are encountered and as a separator between subsurface drainage material and native material or fill. The geotextiles should be installed in conformance with the specifications provided in WSS 2 12 – Construction Geosynthetic.

6.4.1 Stabilization Geotextile
We recommend using a woven geotextile stabilization material at the base of over-excavations and to stabilize the exposed subgrade beneath paved areas if construction is completed during the wet season. The geotextile should conform to the specifications for woven soil stabilization material provided in WSS 9-33.2(1) – Geotextile Properties, Table 3, Geotextile for Separation or Soil Stabilization.

6.4.2 Separation and Drainage Geotextile
We recommend using a non-woven geotextile drainage material around subsurface drains to separate drain rock from adjacent materials. The geotextile should conform to the specifications for non-woven separation material provided in WSS 9-33.2(1) – Geotextile Properties, Table 3, Geotextile for Separation or Soil Stabilization.

6.5 WET WEATHER CONSIDERATIONS
This section describes additional recommendations with potential budget and schedule impacts that may affect the owner and site contractor if earthwork occurs during the wet season. These recommendations are based on the site conditions and our experience on previous construction projects completed in the area.

- The near-surface soil encountered in the explorations is typically silty sand. The fines content of the material is high, and the soil will be susceptible to deterioration during wet weather. If construction is completed or extends into the wet season, we recommend stabilizing the areas of the site where construction traffic is anticipated through cement amendment or construction of a working pad constructed using stabilization material. Additional BMPs will be necessary in cement-amended areas and to monitor/manage the pH levels in stormwater discharge.
- Earthwork should be accomplished in small sections to minimize exposure to wet weather.
- Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill.
- The size of construction equipment and access to the area should be limited to prevent soil disturbance.
- The ground surface in the construction area should be sloped and sealed with a smooth-drum roller to promote rapid runoff of precipitation, to prevent surface water from flowing into excavations, and to prevent puddles from forming.
- The building pads should be surfaced with a 12-inch-thick gravel pad consisting of stabilization material as described in the "Fill Materials" section. This layer will help protect the pads from deterioration under construction traffic during wet weather. The protected area should also extend outward from the building pads a sufficient distance to provide stabilized access for construction equipment around the perimeter of the buildings.
- Additional excavation below planned foundation subgrades should be anticipated to construct a 2-inch-thick lean-mix concrete rat slab or to install a 6-inch-thick layer of crushed surfacing base course to protect the foundation subgrade from deterioration.
- Installation of sumps within excavations may be necessary to remove accumulated stormwater. The sumps should be located outside of the footing footprint and be installed to a depth sufficient to lower the water to below the excavated subgrade elevation.
- Increased handling, excavation, and disposal of wet and disturbed surface material should be expected.
- Protection of exposed soil subgrades and stockpiles will be required.
- Heavy rainfall can occur during winter months and can compromise earthwork schedules in this region.
- In general, snowfall is not dramatically high; however, frozen ground should not be proof rolled or compacted, and fill should not be placed over frozen ground.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory pavement, earthwork, and foundation performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. NV5 should be retained to observe subgrade preparation, fill placement, foundation excavations, drainage system installation, and pavement placement and to review laboratory compaction and field moisture density information.

Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

8.0 LIMITATIONS

We have prepared this report for use by PacTrust and members of the design team for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor’s methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, should be understood.

♦ ♦ ♦

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

NV5

Kevin J. Lamb, P.E.
Principal Engineer

Signed 03/25/2022

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FIGURES

NV5	PACTRUST-219-02	VICINITY MAP	
	MARCH 2022	MARYSVILLE PROJECT MARYSVILLE, WA	**FIGURE 1**

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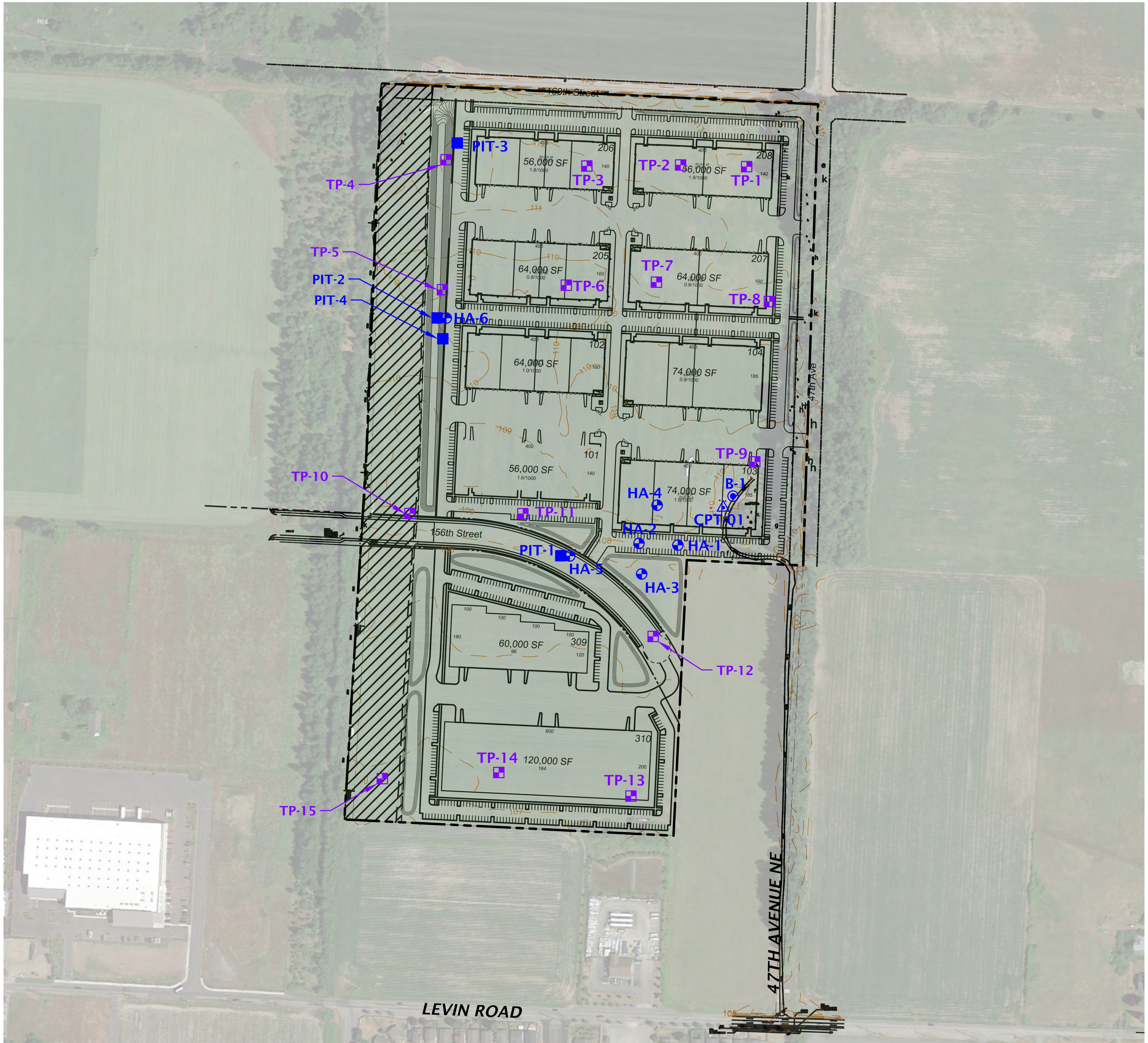
File Name: J:\M-R\PacTrust\PacTrust-219\PacTrust-219-02\Figures\CAD\PacTrust-219-02-VM01.dwg | Layout: FIGURE 1



LEGEND:

SITE BOUNDARY

B-1 BORING

HA-1 HAND AUGER BORING

PIT-1 PIT

CPT-01 CPT

TP-1 TEST PIT (NELSON GEOTECHNICAL ASSOCIATES, INC.)

N

0 300 600

(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON DRAWING OBTAINED FROM MACKENZIE ON MARCH 8, 2022.
2. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO DECEMBER 21, 2020.

FIGURE 2

SITE PLAN

MARYSVILLE PROJECT
MARYSVILLE, WA

PACTRUST-219-02

MARCH 2022

NV5

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We supplemented previous explorations at the site by drilling one boring (B-1) to a depth of 41.5 feet BGS, drilling six hand auger borings to depths between 2 and 5 feet BGS, excavating four test pits to depths between 1.5 and 3.2 feet BGS, and advancing one CPT probe (CPT-01) to depth of 36.4 feet BGS (refusal). The boring was drilled by Holt Services, Inc. of Edgewood, Washington. The hand auger borings were drilled by NV5. The test pits were excavated by Team Nelson of Woodinville, Washington, using a Komatsu 308US excavator. The CPT probe was advanced by In Situ Engineering of Snohomish, Washington. The explorations were completed under the supervision of NV5. The exploration logs are presented in this appendix. The approximate exploration locations are shown on Figure 2.

SOIL SAMPLING

We collected representative samples of the various soils encountered in the explorations for geotechnical laboratory testing. Samples were collected from the boring (B-1) using a 1½-inch-inside diameter, split-spoon sampler (SPT sampler) in general accordance with ASTM D1586. The split-spoon samplers were driven into the soil with 140-pound hammer free falling 30 inches. The samplers were driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the boring logs, unless otherwise noted. Representative samples of the soil observed in the test pits were collected from the walls or base of the test pits using the excavator bucket. Sampling intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by Holt Services, In. was 97 percent. The calibration testing results are presented at the end of this appendix.

SOIL CLASSIFICATION

The soil samples were classified in the field in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil characteristics change, although the change could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

GRAIN-SIZE ANALYSIS

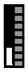
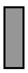
We completed grain-size analysis on select soil samples to determine the distribution of soil particle sizes. The testing was completed in general accordance with ASTM C117 and ASTM C136. The test results are presented in this appendix.

MOISTURE CONTENT

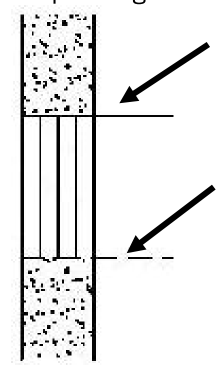
We tested the moisture content of select soil samples in general accordance with ASTM D2216. The moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

FINES CONTENT

We completed fines content testing on select soil samples to determine the soil characteristics. The testing was completed in general accordance with ASTM D1140. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test (SPT) with recovery
	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
	Location of sample collected using 3-inch-outside diameter California split-spoon sampler and 140-pound hammer with recovery
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

NV5	EXPLORATION KEY	TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL			
Relative Density	**Standard Penetration Test (SPT) Resistance**	**Dames & Moore Sampler (140-pound hammer)**	**Dames & Moore Sampler (300-pound hammer)**
Very loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL				
Consistency	**Standard Penetration Test (SPT) Resistance**	**Dames & Moore Sampler (140-pound hammer)**	**Dames & Moore Sampler (300-pound hammer)**	**Unconfined Compressive Strength (tsf)**
Very soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION	
Term	**Field Test**
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS					
Secondary granular components or other materials such as organics, man-made debris, etc.					
Percent	**Silt and Clay In:**		**Percent**	**Sand and Gravel In:**	
	Fine-Grained Soil	**Coarse-Grained Soil**		**Fine-Grained Soil**	**Coarse-Grained Soil**
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

NV5

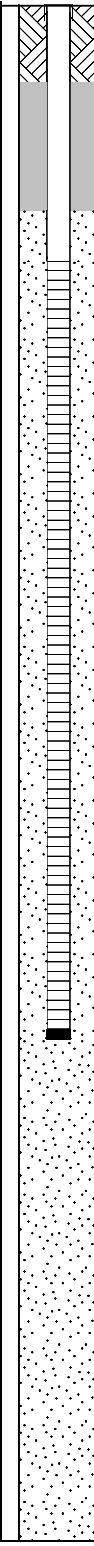
SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Medium dense, dark brown, silty SAND with gravel (SM); moist - **FILL**.			11	Flush-mount monument with 1.5 feet of concrete backfill
1.5	Loose, orange, silty SAND (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	1.5			Bentonite chips
2.5	gray-brown at 3.0 feet		SIEV	6	2-inch, Schedule 40 PVC well casing
5.0	Medium dense, gray SAND with silt and gravel (SP-SM); moist to wet, sand is fine to coarse - **RECESSIONAL OUTWASH**.	5.0	SIEV	20	
7.5				26	10/20 filter pack sand
10.0			P200	22	2-inch, Schedule 40 PVC screen, 0.010-inch slot width; P200 = 9%
12.5				30	
15.0				21	
17.5					
20.0				26	Well No. BND 356 set at 20.0 feet
22.5					
25.0	dense at 25.0 feet		P200	31	P200 = 6%
27.5					
30.0					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

0 50 100



1.1 feet on 12/11/20

DRILLED BY: Holt Services, Inc. LOGGED BY: R. Hilal COMPLETED: 12/10/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 2 3/4 inches

NV5	PACTRUST-219-01	**BORING B-1**
	MARCH 2022	MARYSVILLE PROJECT MARYSVILLE, WA — **FIGURE A-1**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0		(continued from previous page)				34	
32.5							
35.0		minor gravel at 35.0 feet				43	
37.5							
40.0						30	
		Exploration completed at a depth of 41.5 feet. Hammer efficiency factor is 97 percent.	41.5				Surface elevation was not measured at the time of exploration.
42.5							
45.0							
47.5							
50.0							
52.5							
55.0							
57.5							
60.0							

DRILLED BY: Holt Services, Inc. LOGGED BY: R. Hilal COMPLETED: 12/10/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 2 3/4 inches

NV5	PACTRUST-219-01	**BORING B-1** (continued)
	MARCH 2022	MARYSVILLE PROJECT MARYSVILLE, WA — **FIGURE A-1**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT %	COMMENTS

HA-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	SAMPLE	MOISTURE CONTENT % (0 / 50 / 100)	COMMENTS
0.0	Very loose, brown, silty SAND with organics (SM); moist - **TOPSOIL**.		☒	●	
	Loose, gray-brown with orange mottled, silty SAND with gravel (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	0.8	☒		▽ 1.75 feet, during drilling
2.5	Exploration completed at a depth of 2.5 feet.	2.5			Surface elevation was not measured at the time of exploration.
5.0					
7.5					

HA-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	SAMPLE	MOISTURE CONTENT % (0 / 50 / 100)	COMMENTS
0.0	Very loose, brown, silty SAND with organics (SM); moist - **TOPSOIL**.		☒	●	▽ 1.0 foot, during drilling
	Loose, gray-brown with orange mottled, silty SAND with gravel (SM); moist to wet - **WEATHERED RECESSIONAL OUTWASH**.	1.0	☒		
2.5	Exploration completed at a depth of 2.0 feet.	2.0			Surface elevation was not measured at the time of exploration.
5.0					
7.5					

DRILLED BY: NV5 staff | LOGGED BY: R. Hilal | COMPLETED: 12/11/20

BORING METHOD: hand auger (see document text) | BORING BIT DIAMETER: 4 inches

NV5

PACTRUST-219-01

MARCH 2022

BORING

MARYSVILLE PROJECT
MARYSVILLE, WA

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % (0 50 100)	COMMENTS

HA-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	BLOW COUNT / MOISTURE CONTENT %	COMMENTS
0.0		Very loose, dark brown, silty SAND with organics (SM); moist - **TOPSOIL**.			☒		
		Loose, gray-brown with orange mottled, silty SAND (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	1.3	SIEV	☒	●	
2.5		Exploration completed at a depth of 2.5 feet.	2.5				Surface elevation was not measured at the time of exploration.
5.0							
7.5							

HA-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	BLOW COUNT / MOISTURE CONTENT %	COMMENTS
0.0		Very loose, dark brown, silty SAND with organics (SM); moist - **TOPSOIL**.			☒		
		Medium stiff, gray-brown with orange mottled, sandy SILT (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	1.0	SIEV	☒	●	
2.5		Exploration completed at a depth of 2.0 feet.	2.0				Surface elevation was not measured at the time of exploration.
5.0							
7.5							

DRILLED BY: NV5 staff LOGGED BY: R. Hilal COMPLETED: 12/11/20

BORING METHOD: hand auger (see document text) BORING BIT DIAMETER: 4 inches

NV5	PACTRUST-219-01	**BORING** (continued)	
	MARCH 2022	MARYSVILLE PROJECT MARYSVILLE, WA	**FIGURE A-3**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % (0 50 100)	COMMENTS

HA-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT %	COMMENTS
0.0		Loose, brown, silty SAND (SM); wet, sand is fine to coarse (13-inch-thick topsoil/tilled zone) - **WEATHERED RECESSIONAL OUTWASH**.	104.0				Installed 1 1/2-inch-diameter PVC in well. Bentonite seal from 0.0 to 2.0 feet. Well screen and sand pack from 2.0 to 4.5 feet. Bottom of well at 4.5 feet.
2.5		Medium dense, gray SAND with silt (SP-SM); wet, sand is fine to coarse - **RECESSIONAL OUTWASH**.	102.1 / 1.9				
5.0		Exploration completed at a depth of 5.0 feet.	99.0 / 5.0				
7.5							

HA-6

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT %	COMMENTS
0.0			104.0				Installed 1 1/2-inch-diameter PVC in well. Bentonite seal from 0.0 to 2.0 feet. Well screen and sand pack from 2.0 to 5.0 feet. Bottom of well at 5.0 feet.
		Loose, gray, silty SAND (SM); wet - **FILL**.	103.8 / 0.2				
		Loose, brown, silty SAND (SM); wet - **WEATHERED RECESSIONAL OUTWASH**.	103.1 / 0.9				
2.5		Medium dense, gray SAND with silt (SP-SM); wet - **RECESSIONAL OUTWASH**.					
5.0		Exploration completed at a depth of 5.0 feet.	99.0 / 5.0				
7.5							

DRILLED BY: NV5 staff LOGGED BY: E. Larson COMPLETED: 01/19/22

BORING METHOD: hand auger (see document text) BORING BIT DIAMETER: 4 inches

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%) 0 50 100	COMMENTS
PIT-1							
0.0		Loose, gray, silty SAND (SM); wet - **FILL**.	104.0 / 103.8 / 0.2				
		Loose, brown, silty SAND (SM); wet - **WEATHERED RECESSIONAL OUTWASH**.	103.1 / 0.9	SIEV		●	Moderate groundwater seepage observed at 1.5 feet. Small-scale PIT at 1.5 feet.
		Medium dense, gray SAND with silt (SP-SM); wet, sand is fine to coarse - **RECESSIONAL OUTWASH**.	102.5 / 1.5				No caving observed to the depth explored.
2.5		Exploration completed at a depth of 1.5 feet.					Surface elevation was not measured at the time of exploration.
5.0							
7.5							
PIT-2							
0.0		Loose, brown, silty SAND (SM); wet, sand is fine to coarse (13-inch-thick topsoil/tilled zone) - **WEATHERED RECESSIONAL OUTWASH**.	104.0				Moderate groundwater seepage observed at 1.0 foot.
		Medium dense, gray SAND with silt (SP-SM); wet - **RECESSIONAL OUTWASH**.	102.1 / 1.9	SIEV		●	Small-scale PIT at 2.3 feet.
2.5		Exploration completed at a depth of 2.5 feet.	101.5 / 2.5				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
5.0							
7.5							

EXCAVATED BY: Team Nelson   LOGGED BY: E. Larson   COMPLETED: 01/27/22

EXCAVATION METHOD: excavator (see document text)

NV5   PACTRUST-219-01   MARCH 2022

TEST PIT

MARYSVILLE PROJECT
MARYSVILLE, WA

FIGURE A-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%)	COMMENTS

PIT-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	COMMENTS
0.0	Medium dense, gray SAND with silt (SP-SM); moist to wet, sand is fine (topsoil to 15.0 inches) - **RECESSIONAL OUTWASH**.	104.0	Slow groundwater seepage observed at 1.0 foot.
2.5	Exploration completed at a depth of 2.3 feet.	101.7 / 2.3	Small-scale PIT at 2.3 feet. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
5.0			
7.5			

Moisture content scale: 0 50 100

PIT-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	COMMENTS
0.0	Medium dense, gray SAND with silt (SP-SM); moist to wet, sand is fine (topsoil to 10.0 inches) - **RECESSIONAL OUTWASH**.	104.0	Slow groundwater seepage observed at 1.5 feet. Small-scale PIT at 2.2 feet.
2.5	Exploration completed at a depth of 3.2 feet.	100.8 / 3.2	No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
5.0			
7.5			

Moisture content scale: 0 50 100

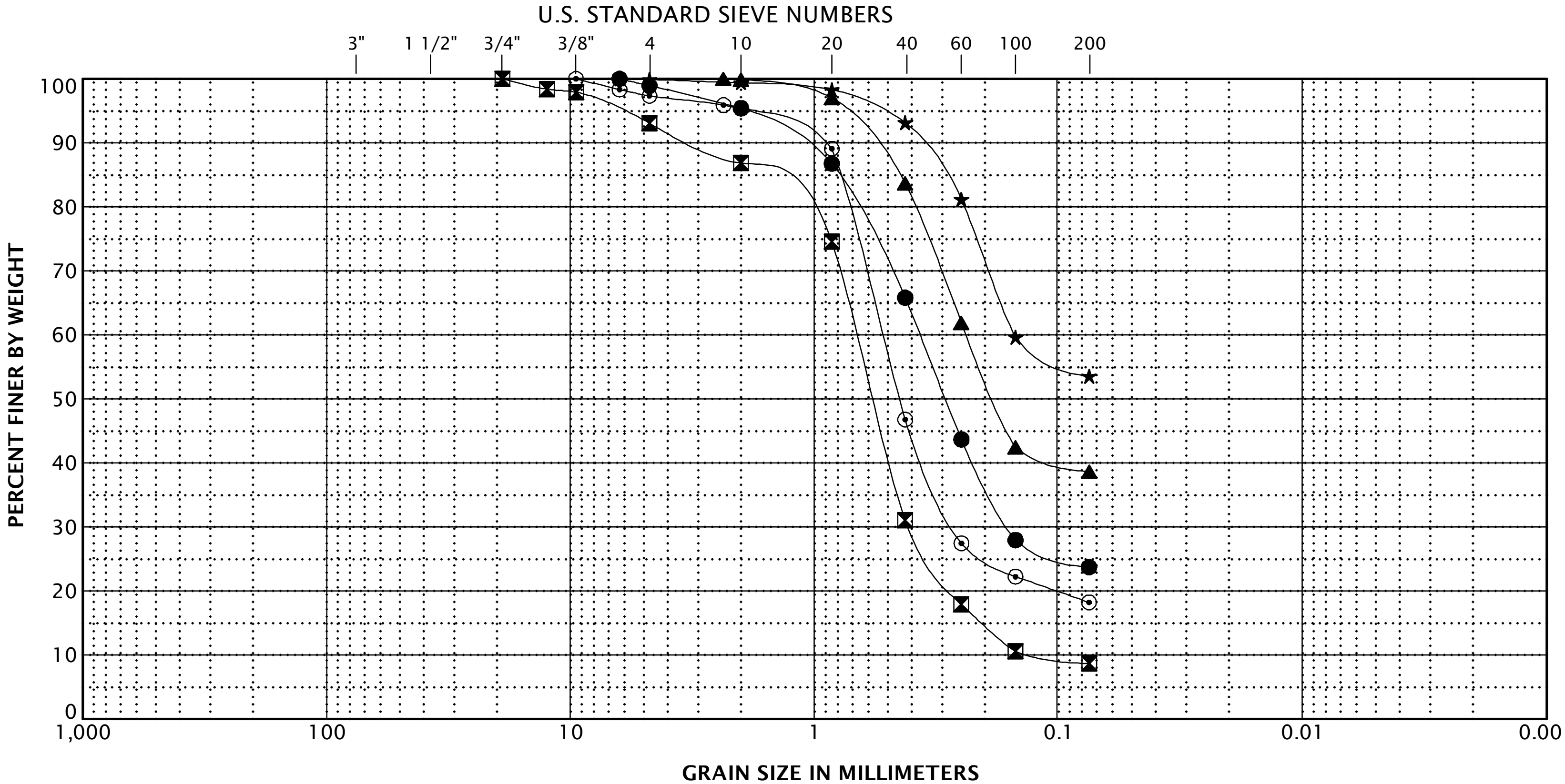
EXCAVATED BY: Team Nelson

LOGGED BY: S. Burford

COMPLETED: 02/17/22

EXCAVATION METHOD: excavator (see document text)

NV5	PACTRUST-219-01	**TEST PIT**	
	MARCH 2022	MARYSVILLE PROJECT MARYSVILLE, WA	**FIGURE A-6**



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	B-1	2.5	35	0.37	0.29	0.16			1	75	24	
⧗	B-1	5.0	19	0.67	0.57	0.41	0.12		7	84	9	
▲	HA-3	1.2	35	0.24	0.18				0	61	39	
★	HA-4	1.0	28	0.15					0	46	54	
⊙	PIT-1	1.0	21	0.53	0.45	0.27			3	79	18	

NV5

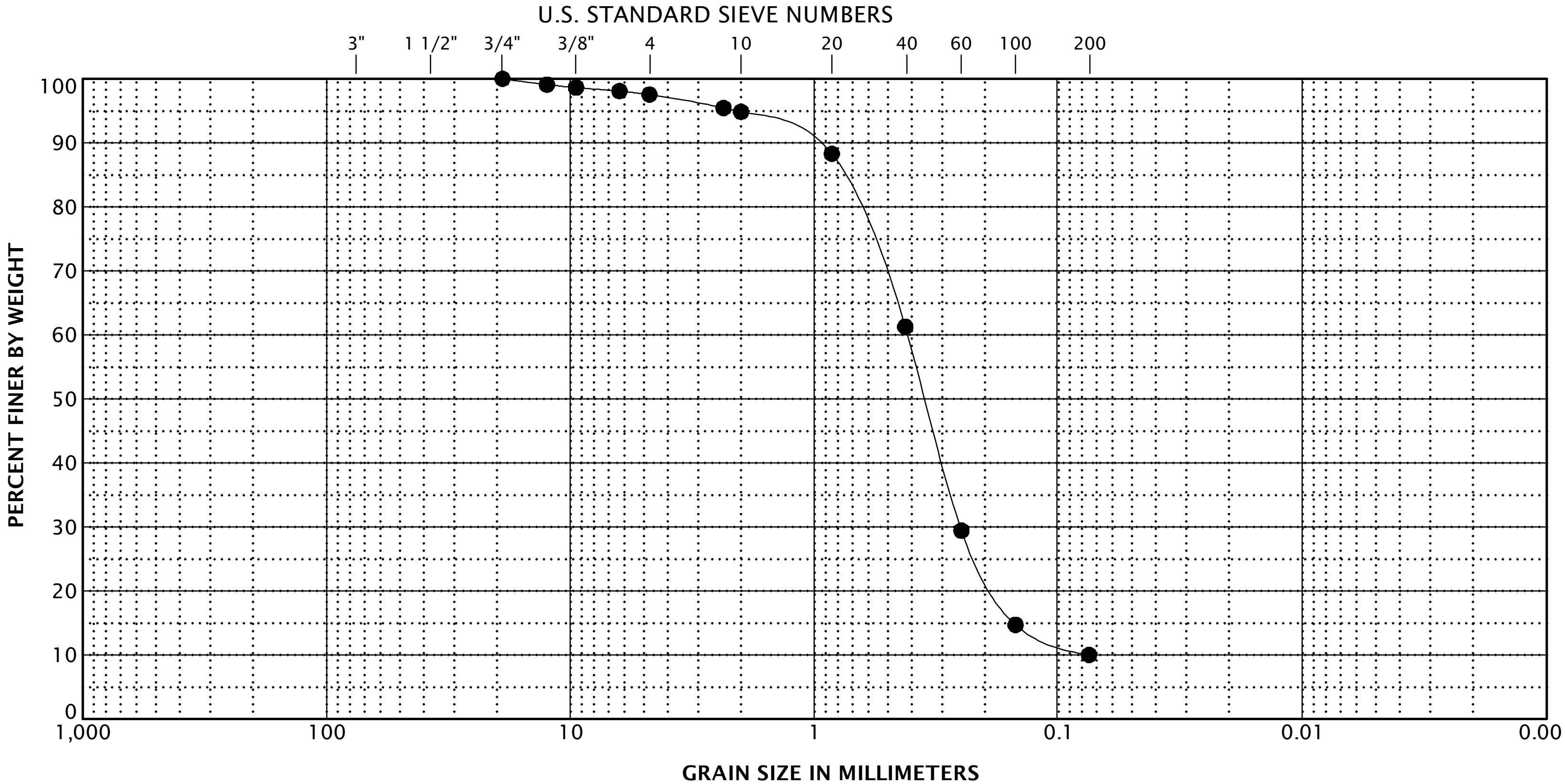
PACTRUST-219-01

MARCH 2022

GRAIN-SIZE TEST RESULTS

MARYSVILLE PROJECT
MARYSVILLE, WA

FIGURE A-7



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	PIT-2	2.0	26	0.42	0.35	0.25			2	88	10	

NV5	PACTRUST-219-01	GRAIN-SIZE TEST RESULTS (continued)	
	MARCH 2022	MARYSVILLE PROJECT MARYSVILLE, WA	FIGURE A-7

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	0.0		24							
B-1	2.5		35		1	75	24			
B-1	5.0		19		7	84	9			
B-1	10.0		15				9			
B-1	12.5		17							
B-1	25.0		18				6			
HA-1	0.0		40							
HA-2	0.0		39							
HA-3	1.2		35		0	61	39			
HA-4	1.0		28		0	46	54			
PIT-1	1.0	103.0	21		3	79	18			
PIT-2	2.0	102.0	26		2	88	10			

Robert Miner Dynamic Testing, Inc.
Dynamic Measurements and Analyses for Deep Foundations

July 24, 2018

Mr. Dale Abernathy
Holt Services, Inc.
10621 Todd Rd. East
Edgewood, WA 98372

Re: Penetration Test Energy Measurements
Mobile B-57 Rig No. 5, Mobile Auto Hammer
Bore Hole: Yard Test Hole, June 15, 2018
Holt Services Yard, Edgewood, Washington

RMDT Job No. 18F19

Dear Mr. Abernathy,

This letter presents energy transfer measurements made during Standard Penetration Tests for the drill hole and drill rig referenced above. Robert Miner Dynamic Testing, Inc. (RMDT) made dynamic measurements with a Pile Driving Analyzer® as a hammer advanced the NW rod during sampling with a split spoon sampler.

The purpose of RMDT's testing was the measurement of energy transferred to the drill rods. Measurements were made on a section of NW gauge rod at the top of the drill string. Strain gages and accelerometers on the rod were connected to a Pile Driving Analyzer® (PDA) which generally processed acceleration and strain measurements from each hammer blow and stored both the measurements and computed results. Measurements and data processing generally followed the ASTM D 4633-16 standard. Energy transfer past the gage location, EFV, was computed by the PDA using force and velocity records as follows:

$$EFV = \int_{a}^{b} F(t)\ v(t)\ dt$$

The value "a" corresponds to the start of the record which is when the energy transfer begins and *"b"* is the time at which energy transferred to the rod reaches a maximum value. Appendix A contains more information on our measurement equipment and methods of analysis. The EFV energy calculation is identical to the EMX energy result discussed in Appendix A. The EFV and EMX values apply to the sensor location near the top of the rod.

TEST DETAILS

On June 15, 2018, a single boring was advanced at the maintenance yard of Holt Services in Edgewood, Washington. The drill rig used during sampling was a truck mounted Mobile B-57 auger unit manufactured by Mobile Drill International and referred to as Rig 5 by the operator. RMDT observed a tag on the rig indicated the rig Serial No. Is 2015 -25. The B-57 unit drilled to six depth intervals ranging from 20 to 60 ft below ground surface and SPT tests were

Mailing Address: P.O. Box 340, Manchester, WA, 98353, USA **Phone:** 360-871-5480
Location: 2288 Colchester Dr. E., Ste A, Manchester, WA, 98353 **Fax:** 360-871-5483

completed through hollow-stem augers at each of 6 depths. The rod used to advance the spoon at each sample depth had a diameter matching that of NW rod. The automatic hammer in use during our testing was manufactured by Mobile Dill International and appeared to use a chain drive powered by a hydraulic motor, with the ram and chain drive enclosed within an outer casing.

RESULTS

A summary of testing and monitoring results is given in Table 1. The tabulated results include the starting sample depth, the penetration resistance, the number of hammers blows in our data set, measured energy transfer, EFV, the computed transfer efficiency, ETR, and the hammer blow rate, BPM. Appendix B contains detailed numeric results for each individual test.

Energy measurements must be divided by the theoretical free fall energy of the hammer to obtain an efficiency. A 140 lb ram raised 30 inches above an impact surface has 350 lb-ft of potential energy. Thus, the transfer energy results for sampling with the 140 lb ram may be divided by 350 lb-ft to yield the ratio of the delivered energy to the nominal potential energy. This efficiency ratio, ETR, is given for each sample interval as a percent efficiency.

Table 1. Summary of Test Details and Results for the 140-lb ram and Split Spoon Sampler

Sample Starting Depth	Penetration Resistance (Blow/Set)	Number of Blows in Data Set	Average Transfer Energy EFV (lb-ft)	Average Transfer Efficiency ETR (percent)	Average Hammer Blow Rate BPM (blow/min)
20 ft	6/1 ft	6	336	96	45
25 ft	38/1 ft	38	343	98	51
40	20/1 ft	20	344	98	48
50 ft	40/1 ft	40	337	96	48
55 ft	16/1 ft	16	341	97	48
60 ft (1)	9/ 1.3 ft	8	345	99	39
Average for Split Spoon samples:			**341**	**97**	**47**

Note (1): Because only 5 blows were required for the first 1.5 ft of sample at 60 ft starting depth, that sample was advanced a total of 1.8 ft so that our data set at this depth would include more hammer blows. Due to poor measurement quality, one of the nine blows comprising the final 1.3 ft was excluded from our data set.

Six sample returns were monitored while the 140 lb ram and standard split spoon sampler were in use. The overall average ETR and hammer blow rate was 97 percent and 47 blows per minute, respectively.

It was a pleasure to assist you and to participate on this project with the staff of Holt Services Inc. Please do not hesitate to contact us if you or other project participants have any questions about this report.

Sincerely,

ROBERT F. MINER
STATE OF WASHINGTON
29099
REGISTERED
PROFESSIONAL ENGINEER

July 24, 2018

Robert Miner, P.E.

Robert Miner Dynamic Testing, Inc.

APPENDIX B

APPENDIX B

IN SITU CONE PENETROMETER TEST RESULTS

HOLE NUMBER: CPT-01

CPT CONTRACTOR: In Situ Engineering

COMMENT: Parcel #31052800400300

LOCATION: Marysville

JOB NUMBER: PacTrust-219-01

CUSTOMER:

TEST DATE: 12/11/2020 11:03:56 AM

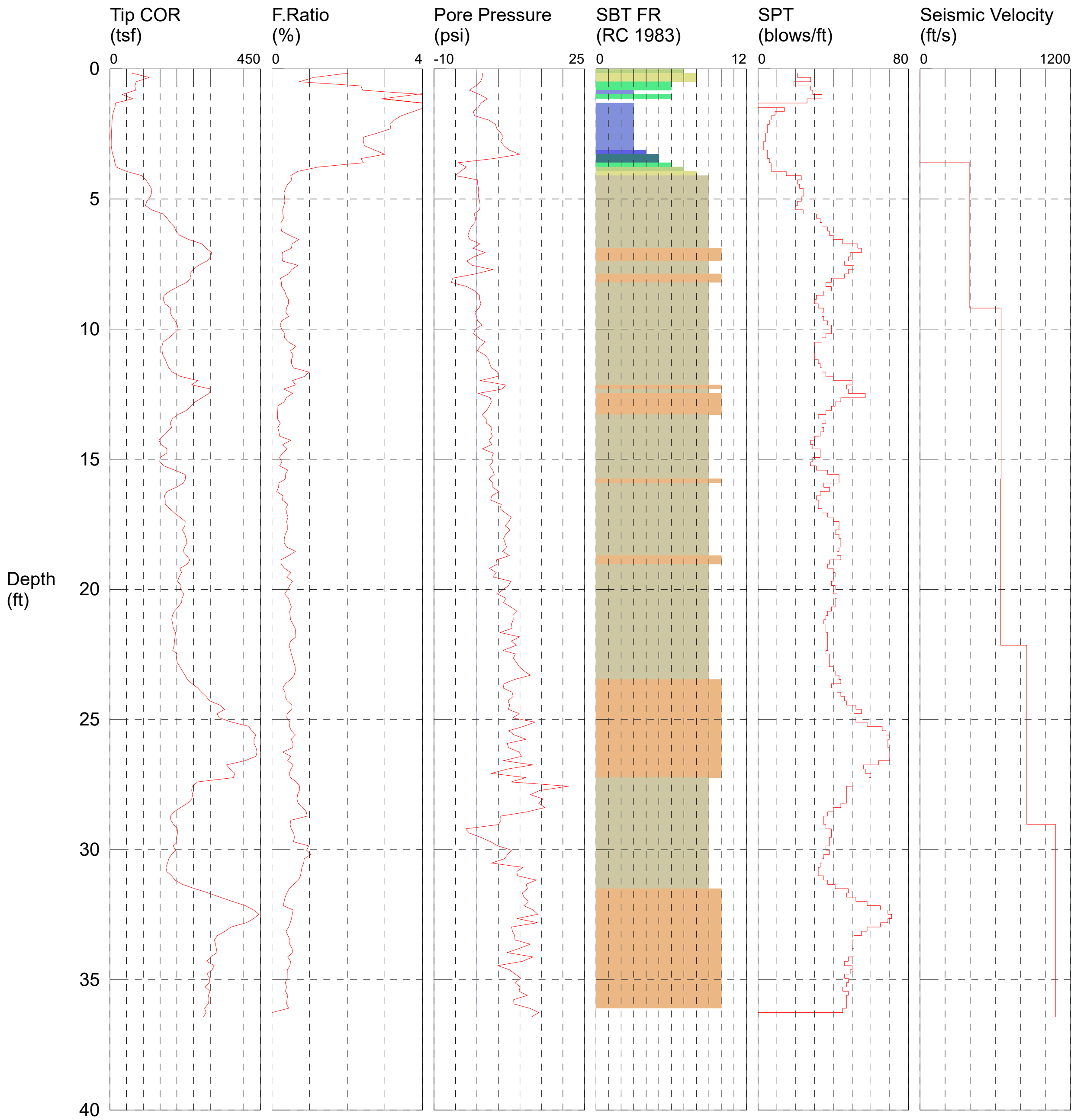
OPERATOR: Walsh

CONE ID: DDG1263

CPT-01

CPT CONTRACTOR: In Situ Engineering
CUSTOMER: Geodesign
LOCATION: Marysville
JOB NUMBER: PacTrust-219-01
COMMENT: Parcel #31052800400300
COMMENT:

OPERATOR: Walsh
CONE ID: DDG1263
TEST DATE: 12/11/2020 11:03:56 AM
PREDRILL: 0 ft
BACKFILL: 20% Grout & Bentonite Chips
SURFACE PATCH: None

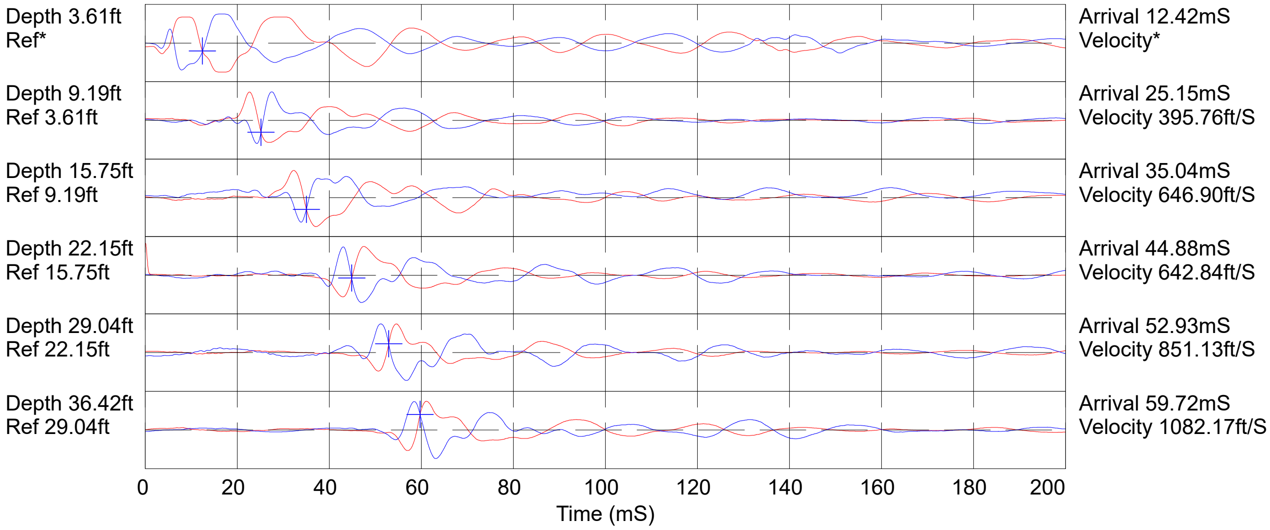


TOTAL DEPTH: 36.417 ft

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

HOLE NUMBER: CPT-01



Hammer to Rod String Distance (ft): 2.79
* = Not Determined

JOB NUMBER: PacTrust-219-01

APPENDIX C

APPENDIX C

GEODESIGN MEMORANDUM – DUE DILIGENCE GEOTECHNICAL ENGINEERING EVALUATION SERVICES

GEODESIGN INC
AN NV5 COMPANY

Memorandum

To:	Matthew Oyen, P.E.	From:	Joe Westergreen, P.E. Kevin J. Lamb, P.E. George Saunders, P.E.
Company:	PacTrust	Date:	January 6, 2021
Address:	15350 SW Sequoia Parkway, Suite 300 Portland OR 97224		

cc:	n/a

GDI Project:	PacTrust-219-01
RE:	Due Diligence Geotechnical Engineering Services Marysville Project 15908 47th Avenue NE Marysville, Washington

INTRODUCTION

This memorandum summarizes the primary geotechnical considerations as part of our due diligence geotechnical engineering services for the proposed office warehouse development project located in Marysville, Washington.

The site is composed of two rectangular-shaped parcels encompassing an area of approximately 57.5 acres. The parcels are undeveloped open space used for agriculture. The site location relative to surrounding physical features is shown on Figure 1. Figure 2 shows the existing conditions, our approximate exploration locations, and the approximate locations of explorations completed by Nelson Geotechnical Associates, Inc. (NGA) in December 2018.

The boring logs and laboratory test results, the cone penetration test (CPT) data, and the Preliminary Geotechnical Engineering Evaluation prepared by NGA are presented in Attachments A, B, and C, respectively. Preliminary conceptual grading plans provided by Mackenzie are presented in Attachment D.

PROJECT UNDERSTANDING

Plans were conceptual at the time of this due diligence memorandum. The current conceptual plan includes three buildings (A1 through A3) on the southern parcel and eight buildings (B1 through B8) on the northern parcel with buildings anticipated to range from approximately 33,000 to 100,000 square feet. Currently, finish floor elevations are anticipated to range from 117.85 to 119 feet, indicating between 6 to 10 feet of fill will be required across the building pads; however, we understand that revisions are being considered.

Loading docks, parking, and drive aisles are planned around the buildings. In addition, we understand that detention ponds, permeable pavement, and other shallow infiltration facilities are being considered as part of the stormwater management plan. We understand that permeable pavement, if used, will be limited to automobile parking areas around the perimeter of the buildings.

Building foundations loads and settlement tolerances were unknown at the time of this due diligence memorandum. We assume the buildings will consist of single-story, concrete tilt-up structures and have assumed maximum column loads will be less than 150 kips, maximum wall loads of less than 4.5 kips per linear foot, and a distributed floor slab live load of 250 pounds per square foot (psf).

BACKGROUND

The NGA investigation included excavating 15 test pits to a depth 6 feet below ground surface (BGS). Soil conditions consists of a thick organic-rich topsoil zone to depths between 0.8 foot and 1.5 feet BGS. The topsoil is underlain by an intermediate zone between 0.9 foot and 2.2 feet thick that consists of loose to medium dense, silty sand with varying amounts of gravel and iron oxidation staining (weathered recessional outwash). Underlying the weathered material NGA encountered gray sand with varying amounts of gravel interpreted to be unweathered recessional outwash sands consistent with the mapped geology for the area. Groundwater seepage was encountered in all the test pit explorations at depths between 2.8 and 4.5 feet BGS.

The test pits that were limited to approximately 6 feet in depth and did not provide data on the consistency of the material other than qualitative options.

SCOPE OF SERVICES

The purpose of our due diligence scope of services was to provide a summary of key geotechnical design and construction elements that will affect the project to assist with pre-development cost analysis. Additional site-specific explorations, infiltration testing, and a full geotechnical report will be required to support design for the project. Our specific scope included the following:

- Reviewed available information from previous geological and geotechnical studies conducted at and in the vicinity of the site.
- Reviewed preliminary grading plans.
- Coordinated utility locates, site access, and subconsultant services for our limited subsurface explorations.
- Conducted a subsurface exploration program that included the following:
  - Drilled one boring to a depth of 41.5 feet BGS. Installed a monitoring well in the boring to monitor groundwater conditions.
  - Drilled four hand auger borings to depths between 2 and 2.5 feet BGS.
  - Advanced one CPT probe to a depth of 36.4 feet BGS (refusal).
- Maintained continuous logs of the borings and collected soil samples at representative intervals.

- Conducted a laboratory testing program that included the following:
  - Ten moisture content determinations in general accordance with ASTM D2216
  - Six grain-size distribution analysis in general accordance with ASTM C117, ASTM C136, and/or ASTM D1140
- Provided preliminary recommendations on the following:
  - Subgrade preparation
  - Foundation support
  - Seismic design criteria
  - Infiltration feasibility

SITE CONDITIONS

GEOLOGIC SETTING

The site is situated in the Puget Sound lowland north of Marysville, Washington, which is generally made up of Vashon stade recessional outwash known as the Marysville Sand Member. The recessional outwash material was deposited by meltwater flowing south from the stagnating and receding Vashon glacier and is typically at least 20 meters thick. The recessional outwash is generally composed of well-drained, stratified to massive outwash sand with variable amounts of fine gravel.

SURFACE CONDITIONS

The approximately 57.5-acre site consists of agricultural land that has primarily been used for growing grass crops. Remnants of a past residence and outbuildings are present at the southeast corner of the northern parcel, which includes portions of concrete building slabs, small piles of concrete and wood debris, and an abandoned well.

Site access is provided by a gravel access road (47th Avenue NE). The site is relatively level and bordered by Hayho Creek to the north and west. Surface water was also observed in the ditch line and shallow drainage pipe between the parcels.

SUBSURFACE CONDITIONS

Subsurface conditions were evaluated by reviewing the test pit logs completed by NGA as discussed in the "Background" section, reviewing geologic maps, and by completing a limited exploration program.

Our subsurface exploration program consisted of drilling one boring (B-1) to a depth of 41.5 feet BGS, drilling four shallow hand auger borings to depths between 2 and 2.5 feet BGS, and advancing one CPT probe (CPT-01) to depth of 36.4 feet BGS (refusal). The approximate exploration locations are shown on Figure 2. The boring logs and laboratory test results are presented in Attachment A. The CPT data is presented in Attachment B. Due the soft and saturated conditions of the fields, we completed the boring and CPT in the southeast corner of the northern parcel near the location of the historical residence at the north end of the 47th Avenue NE extension into the site.

In boring B-1 we encountered approximately 1.5 feet of fill consisting of medium dense, silty sand with gravel. Weathered recessional outwash consisting of loose, silty sand with was encountered below the fill to a depth of approximately 5 feet BGS. Unweathered recessional outwash consisting of medium dense gray sand with silt and gravel was encountered below the weathered material to the maximum depth explored of 41.5 feet BGS. Based on SPT blow counts, the recessional outwash becomes dense starting at approximately 25 feet BGS. The CPT indicates similar soil conditions.

In the hand auger borings we encountered a topsoil zone between 0.8 foot and 1.3 feet thick that generally consists of very loose, silty sand with organics. The hand auger borings were completed in the weathered recessional outwash below the topsoil zone. The weathered recessional outwash varies between medium stiff, sandy silt (HA-4) and loose, silty sand (HA-1 through HA-3).

Groundwater

Due to the mud rotary drilling techniques, groundwater could not be measured directly while drilling boring B-1. To monitor groundwater conditions, a 2-inch-diameter PVC monitoring well was installed in the boring to a depth of 20 feet BGS. It was constructed in general accordance with Washington Administrative Code 173-60. A 0.010-inch well screen was installed between 5 and 20 feet BGS. The annular space between the casing and well was backfilled with 10/20 silica sand. A blank PVC section with bentonite chip backfill extends up to the monument at the surface. The well was completed at the surface with a cast iron flush monument. The Washington State Department of Ecology well tag for the well is number BND 356.

Groundwater was measured in the monitoring well the day after drilling at 1.1 feet BGS, and a data logger was installed in the well to obtain regular groundwater measurements from the monitoring well.

Groundwater seepage was encountered in hand auger borings HA-1 and HA-2 at 1.75 feet and 1 foot BGS, respectively. As discussed in the “Background” section, groundwater seepage was encountered in all the test pit explorations completed by NGA at depths varying from 2.8 to 4.5 feet BGS.

CONCLUSIONS

GENERAL

The primary geotechnical considerations are the shallow groundwater and loose/moisture-sensitive near-surface soil. The weathered material exposed after stripping will provide poor subgrade support during construction. Liquefaction-induced settlement is anticipated to be negligible; however, the near-surface site soil is somewhat prone to settlement under fill and foundation loads. Grading plans currently indicate raising grades between 6 and 10 feet across the building pads.

Based on the results of our explorations and our review of existing information, our preliminary conclusions and recommendations are provided below.

SUBGRADE PREPARATION

- If not adequately planned, construction can lead to extensive soft areas and significant repair costs. The on-site soil will provide inadequate support for construction equipment during the wet construction season and, to a lesser degree, during the dry season. Stabilization of the subgrade should be anticipated prior to fill placement, and granular haul roads and working pads or cement amendment should be anticipated.
- A thick topsoil zone is present across the site (approximately 1 foot to 1.5 feet thick) that will require removal during stripping. Construction equipment should not traffic the stripped subgrade as it will soften and require additional stabilization measures. Truck traffic will need to be supported on haul roads and working pads.
- The on-site soil is sensitive to small changes in moisture content and difficult, if not impossible, to adequately compact during wet weather or when the moisture content of the soil is more than a couple percent above the optimum required for compaction. The existing moisture content of the soil is significantly above optimum and drying will be required if used as common fill.
- Groundwater was encountered near the ground surface. Excavations that extend below groundwater levels will require significant dewatering and shoring.
- Soil stabilization of the subgrade will be required after stripping and prior to placing fill due to the shallow groundwater and soft, moisture-sensitive near-surface soil.
    - In our opinion, the best approach to stabilizing the subgrade will be to use a granular pad incorporating geotextile fabric at the base and top of the pad. We recommend that a geogrid be included at mid-height within the granular pad. We recommend that the granular pad be at least 30 inches thick, with the geogrid located 18 inches from the bottom of the granular pad. We recommend that the bottom 18 inches be placed, geogrid placed, and the top 12 inches placed prior to compacting. All fill placed for the granular pad should be placed using wide, tracked, low ground pressure equipment.
    - Cement amending the stripped subgrade may be an option. The primary concern is the shallow groundwater and is the basis for recommending that this approach only be attempted toward the end of summer or early fall. The cement amendment should be a minimum of 18 inches thick and we recommend a preliminary minimum cement ratio of 8 to 9 percent by dry weight. We recommend cement-spreading equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A static sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction of the cement-amended soil. A smooth-drum roller with a minimum applied linear force of 700 pounds per inch should be used for final compaction. The amended soil should be compacted to at least 92 percent of the achievable dry density at the moisture content of the material, as defined in ASTM D1557. Lastly, given the high moisture contents, we anticipate that multiple passes with the tiller will be required to adequately mix the cement.

FOUNDATION SUPPORT

- The underlying loose, silty sand, particularly the weathered material, will be susceptible to settlement from fill placement and foundation loads. As discussed, stabilization of the subgrade will be required prior to raising site grades.
- Settlement of the loose, silty sand is anticipated to occur relatively quickly during fill placement. Settlement tolerances, both total and differential, have not been established for the project. However, assuming total and differential post-construction settlements of footings of 1½ inches and ¾ inch, respectively, it is our opinion that conventional spread foundations are possible. We recommend that up to five settlement plates be established prior to filling to monitor the amount and duration of the fill-induced settlement.
- Based on the preliminary grading plans, we anticipate that foundations will be established on structural fill. We anticipate an allowable bearing pressure of 2,000 to 2,500 psf for foundations established on a minimum 3 feet of structural fill.
- The recommended allowable bearing pressures applies to the total of dead plus long-term live loads and may be doubled for short-term loads, such as those resulting from wind or seismic forces.

SEISMIC DESIGN CRITERIA

Moderate to high levels of earthquake shaking should be anticipated during the design life of the structures, and they should be designed to resist earthquake loading in accordance with the methodology described in the 2018 International Building Code (IBC). Seismic design parameters for the 2018 IBC are based on ASCE 7-16. The recommended seismic design parameters are presented in Table 1.

Table 1. IBC Seismic Design Parameters

Seismic Design Parameter	Short Period	1-Second Period
Maximum Considered Earthquake Spectral Acceleration	S_s = 1.072 g	S_1 = 0.383 g
Site Class	D	
Site Coefficient	F_a = 1.2	F_v = 1.8
Adjusted Spectral Acceleration	S_{MS} = 1.287 g	S_{M1} = 0.689 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.858 g	S_{D1} = 0.459 g

Based on our subsurface exploration, literature review, and experience, a summary of the seismic hazards in the area and their associated impact at the site are as follows:

- **Amplification:** Areas subject to amplification are typically soft soil overlying stiff soil or bedrock. Based on our explorations and available geologic maps, the site is underlain by medium to dense recessional deposits over glacially consolidated deposits. In our opinion, this material has a low potential for site amplification.
- **Liquefaction/Settlement:** Based on the results of the site explorations, the site is mostly underlain by medium dense to dense outwash deposits consisting of sand with variable amounts of gravel, and groundwater was encountered at a shallow depth. Granular soil, which relies on interparticle friction for strength, is susceptible until the excess pore pressures can dissipate. Liquefaction analysis was performed using the information from the boring and CPT, laboratory test results, and earthquake hazard mapping. Based on our analysis, we estimate the potential for liquefaction is low for the site.
- **Lateral Spreading:** Areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediments adjacent to an open face (such as riverbanks or bay fronts). Liquefied soil adjacent to open faces may "flow" in that direction, resulting in lateral displacement and surface cracking. There is no potential for the site to be affected by lateral spreading.
- **Fault Surface Rupture:** We did not find evidence of faults through the site or on maps of the area. We conclude that the potential for surface rupture at the site is low over the life of the structure.

INFILTRATION FEASIBILITY

- Stormwater infiltration will be limited to shallow systems, such as permeable pavement and bioretention, due to the shallow depth of groundwater.
- NGA provided preliminary infiltration rates of the on-site soil based on grain-size analysis. Infiltration rates of 4.33 and 8.79 inches per hour were provided for the unweathered and weathered material, respectively. Based on the shallow groundwater and our experience with similar soils, these rates seem optimistic. We recommend completing pilot infiltration tests at the locations and proposed depths of infiltration facilities to determine in situ infiltration rates, if infiltration is relied upon as part of the stormwater plan.
- Due to the shallow groundwater, mounding analysis may be required as part of the development design.
- Infiltration capacity can be improved by using imported granular material in infiltration areas.

♦ ♦ ♦

We appreciate the opportunity to provide our services on this project, please call if you have any questions.

JTW:GPS:KJL:kt

Attachments

One copy submitted (via email only)

Document ID: PacTrust-219-01-010621-geom.docx

Signed 01/06/2021

FIGURES

GEODESIGN INC AN NV5 COMPANY	PACTRUST-219-01	VICINITY MAP	
	JANUARY 2021	MARYSVILLE PROJECT MARYSVILLE, WA	FIGURE 1

Printed By: mmiller | Print Date: 1/6/2021 1:56:20 PM
File Name: J:\M-R\PacTrust\PacTrust-219\PacTrust-219-01\Figures\CAD\PacTrust-219-01-VM01.dwg | Layout: FIGURE 1

LEGEND:

— - —	SITE BOUNDARY
B-1	BORING (GEODESIGN)
HA-1	HAND AUGER BORING (GEODESIGN)
CPT-01	CPT (GEODESIGN)
TP-1	TEST PIT (NELSON GEOTECHNICAL ASSOCIATES, INC.)

N

0 300 600

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, DECEMBER 21, 2020

SITE PLAN

FIGURE 2

MARYSVILLE PROJECT
MARYSVILLE, WA

PACTRUST-219-01

JANUARY 2021

GEODESIGN™ AN NV5 COMPANY

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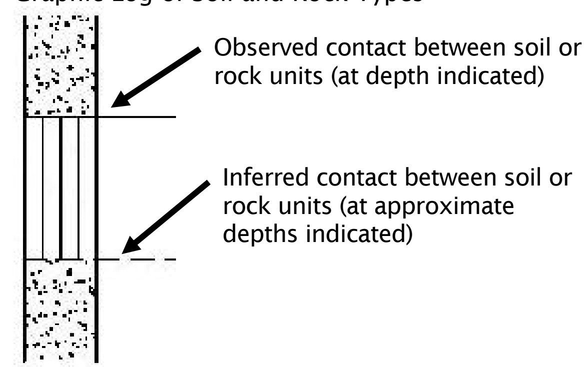
File Name: J:\M-R\PacTrust\PacTrust-219\PacTrust-219-01\Figures\CAD\PacTrust-219-01-SP01.dwg | Layout: FIGURE 2

ATTACHMENT A

SYMBOL	SAMPLING DESCRIPTION
	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium Dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium Stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very Stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In: Fine-Grained Soil	Silt and Clay In: Coarse-Grained Soil	Percent	Sand and Gravel In: Fine-Grained Soil	Sand and Gravel In: Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Medium dense, dark brown, silty SAND with gravel (SM); moist - **FILL**.			11	Flush-mount monument with 1.5 feet of concrete backfill
1.5	Loose, orange, silty SAND (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	1.5			Bentonite chips
2.5	gray-brown at 3.0 feet		SIEV	6	
5.0	Medium dense, gray SAND with silt and gravel (SP-SM); moist to wet, sand is fine to coarse - **RECESSIONAL OUTWASH**.	5.0	SIEV	20	2-inch, Schedule 40 PVC well casing
7.5				26	10/20 filter pack sand
10.0			P200	22	2-inch, Schedule 40 PVC screen, 0.010-inch slot width P200 = 9%
12.5				30	
15.0				21	
17.5					
20.0				26	Well No. BND 356 set at 20.0 feet
22.5					
25.0	dense at 25.0 feet		P200	31	P200 = 6%
27.5					
30.0					

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

1.1 feet on 12/11/20

DRILLED BY: Holt Services, Inc. LOGGED BY: R. Hilal COMPLETED: 12/10/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 2 3/4 inches

GEODESIGN INC AN NV5 COMPANY	PACTRUST-219-01	**BORING B-1**
	JANUARY 2021	MARYSVILLE PROJECT MARYSVILLE, WA — **FIGURE A-1**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
30.0		(continued from previous page)				34	
32.5							
35.0		minor gravel at 35.0 feet				43	
37.5							
40.0						30	
		Exploration completed at a depth of 41.5 feet.	41.5				Surface elevation was not measured at the time of exploration.
42.5		Hammer efficiency factor is 97 percent.					
45.0							
47.5							
50.0							
52.5							
55.0							
57.5							
60.0							

DRILLED BY: Holt Services, Inc. LOGGED BY: R. Hilal COMPLETED: 12/10/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 2 3/4 inches

GEODESIGN INC AN NV5 COMPANY

PACTRUST-219-01

JANUARY 2021

BORING B-1
(continued)

MARYSVILLE PROJECT
MARYSVILLE, WA

FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % (0 50 100)	COMMENTS

HA-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	COMMENTS
0.0	Very loose, brown, silty SAND with organics (SM); moist - **TOPSOIL**.		
	Loose, gray-brown with orange mottled, silty SAND with gravel (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	0.8	1.75 feet, during drilling
2.5	Exploration completed at a depth of 2.5 feet.	2.5	Surface elevation was not measured at the time of exploration.
5.0			
7.5			

HA-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	COMMENTS
0.0	Very loose, brown, silty SAND with organics (SM); moist - **TOPSOIL**.		
	Loose, gray-brown with orange mottled, silty SAND with gravel (SM); moist to wet - **WEATHERED RECESSIONAL OUTWASH**.	1.0	1.0 foot, during drilling
2.5	Exploration completed at a depth of 2.0 feet.	2.0	Surface elevation was not measured at the time of exploration.
5.0			
7.5			

DRILLED BY: GeoDesign, Inc. staff LOGGED BY: R. Hilal COMPLETED: 12/11/20

BORING METHOD: hand auger (see document text) BORING BIT DIAMETER: 4 inches

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PACTRUST-219-01

JANUARY 2021

BORING

MARYSVILLE PROJECT
MARYSVILLE, WA

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % (0 – 50 – 100)	COMMENTS
HA-3							
0.0		Very loose, dark brown, silty SAND with organics (SM); moist - **TOPSOIL**.			☒		
		Loose, gray-brown with orange mottled, silty SAND (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	1.3	SIEV	☒	●	
2.5		Exploration completed at a depth of 2.5 feet.	2.5				Surface elevation was not measured at the time of exploration.
5.0							
7.5							
HA-4							
0.0		Very loose, dark brown, silty SAND with organics (SM); moist - **TOPSOIL**.			☒		
		Medium stiff, gray-brown with orange mottled, sandy SILT (SM); moist - **WEATHERED RECESSIONAL OUTWASH**.	1.0	SIEV	☒	●	
2.5		Exploration completed at a depth of 2.0 feet.	2.0				Surface elevation was not measured at the time of exploration.
5.0							
7.5							

DRILLED BY: GeoDesign, Inc. staff LOGGED BY: R. Hilal COMPLETED: 12/11/20

BORING METHOD: hand auger (see document text) BORING BIT DIAMETER: 4 inches

GEODESIGN INC
AN NV5 COMPANY

PACTRUST-219-01

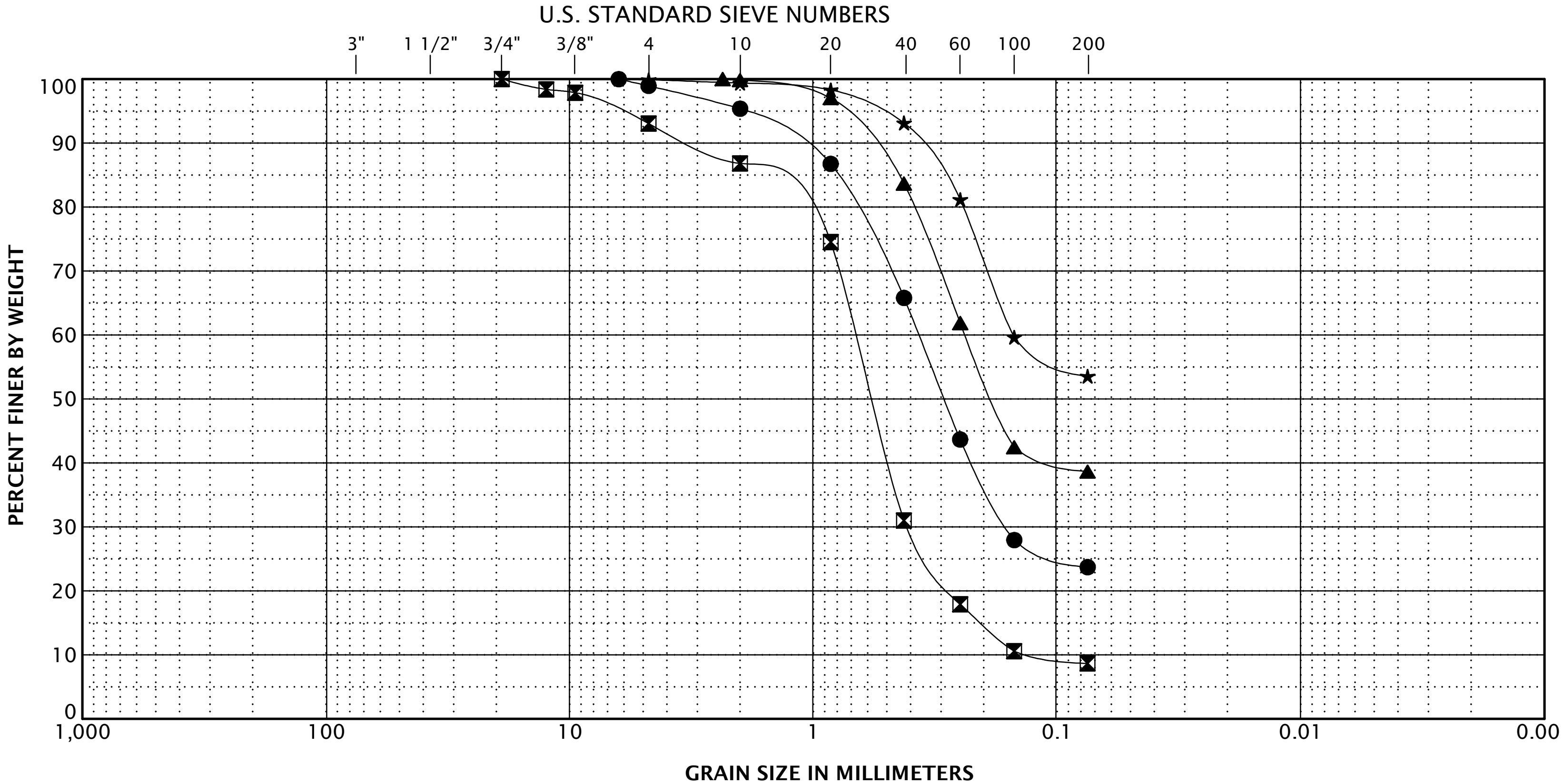
JANUARY 2021

BORING
(continued)

MARYSVILLE PROJECT
MARYSVILLE, WA

FIGURE A-3

BORING LOG - GDI-NV5 - 2 PER PAGE PACTRUST-219-01-B1-HA1_4.GPJ GDI_NV5.GDT PRINT DATE: 1/6/21:KT



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	B-1	2.5	35	0.37	0.29	0.16			1	75	24	
⊠	B-1	5.0	19	0.67	0.57	0.41	0.12		7	84	9	
▲	HA-3	1.2	35	0.24	0.18				0	61	39	
★	HA-4	1.0	28	0.15					0	46	54	

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PACTRUST-219-01

JANUARY 2021

GRAIN-SIZE TEST RESULTS

MARYSVILLE PROJECT
MARYSVILLE, WA

FIGURE A-4

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	0.0		24							
B-1	2.5		35		1	75	24			
B-1	5.0		19		7	84	9			
B-1	10.0		15				9			
B-1	12.5		17							
B-1	25.0		18				6			
HA-1	0.0		40							
HA-2	0.0		39							
HA-3	1.2		35		0	61	39			
HA-4	1.0		28		0	46	54			

Robert Miner Dynamic Testing, Inc.

Dynamic Measurements and Analyses for Deep Foundations

July 24, 2018

Mr. Dale Abernathy
Holt Services, Inc.
10621 Todd Rd. East
Edgewood, WA 98372

Re: Penetration Test Energy Measurements
Mobile B-57 Rig No. 5, Mobile Auto Hammer
Bore Hole: Yard Test Hole, June 15, 2018
Holt Services Yard, Edgewood, Washington

RMDT Job No. 18F19

Dear Mr. Abernathy,

This letter presents energy transfer measurements made during Standard Penetration Tests for the drill hole and drill rig referenced above. Robert Miner Dynamic Testing, Inc. (RMDT) made dynamic measurements with a Pile Driving Analyzer® as a hammer advanced the NW rod during sampling with a split spoon sampler.

The purpose of RMDT's testing was the measurement of energy transferred to the drill rods. Measurements were made on a section of NW gauge rod at the top of the drill string. Strain gages and accelerometers on the rod were connected to a Pile Driving Analyzer® (PDA) which generally processed acceleration and strain measurements from each hammer blow and stored both the measurements and computed results. Measurements and data processing generally followed the ASTM D 4633-16 standard. Energy transfer past the gage location, EFV, was computed by the PDA using force and velocity records as follows:

$$EFV \quad {}_{a}^{b}\ F\,(t)\ v\,(t)\ dt$$

The value "a" corresponds to the start of the record which is when the energy transfer begins and *"b"* is the time at which energy transferred to the rod reaches a maximum value. Appendix A contains more information on our measurement equipment and methods of analysis. The EFV energy calculation is identical to the EMX energy result discussed in Appendix A. The EFV and EMX values apply to the sensor location near the top of the rod.

TEST DETAILS

On June 15, 2018, a single boring was advanced at the maintenance yard of Holt Services in Edgewood, Washington. The drill rig used during sampling was a truck mounted Mobile B-57 auger unit manufactured by Mobile Drill International and referred to as Rig 5 by the operator. RMDT observed a tag on the rig indicated the rig Serial No. Is 2015 -25. The B-57 unit drilled to six depth intervals ranging from 20 to 60 ft below ground surface and SPT tests were

Mailing Address: P.O. Box 340, Manchester, WA, 98353, USA **Phone:** 360-871-5480
Location: 2288 Colchester Dr. E., Ste A, Manchester, WA, 98353 **Fax:** 360-871-5483

completed through hollow-stem augers at each of 6 depths. The rod used to advance the spoon at each sample depth had a diameter matching that of NW rod. The automatic hammer in use during our testing was manufactured by Mobile Dill International and appeared to use a chain drive powered by a hydraulic motor, with the ram and chain drive enclosed within an outer casing.

RESULTS

A summary of testing and monitoring results is given in Table 1. The tabulated results include the starting sample depth, the penetration resistance, the number of hammers blows in our data set, measured energy transfer, EFV, the computed transfer efficiency, ETR, and the hammer blow rate, BPM. Appendix B contains detailed numeric results for each individual test.

Energy measurements must be divided by the theoretical free fall energy of the hammer to obtain an efficiency. A 140 lb ram raised 30 inches above an impact surface has 350 lb-ft of potential energy. Thus, the transfer energy results for sampling with the 140 lb ram may be divided by 350 lb-ft to yield the ratio of the delivered energy to the nominal potential energy. This efficiency ratio, ETR, is given for each sample interval as a percent efficiency.

Table 1. Summary of Test Details and Results for the 140-lb ram and Split Spoon Sampler

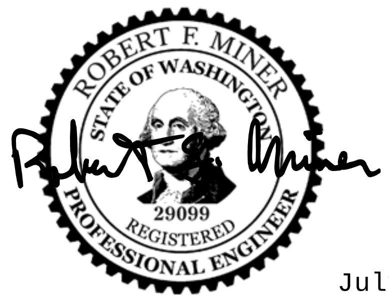
Sample Starting Depth	Penetration Resistance (Blow/Set)	Number of Blows in Data Set	Average Transfer Energy EFV (lb-ft)	Average Transfer Efficiency ETR (percent)	Average Hammer Blow Rate BPM (blow/min)
20 ft	6/1 ft	6	336	96	45
25 ft	38/1 ft	38	343	98	51
40	20/1 ft	20	344	98	48
50 ft	40/1 ft	40	337	96	48
55 ft	16/1 ft	16	341	97	48
60 ft (1)	9/ 1.3 ft	8	345	99	39
Average for Split Spoon samples:			**341**	**97**	**47**

Note (1): Because only 5 blows were required for the first 1.5 ft of sample at 60 ft starting depth, that sample was advanced a total of 1.8 ft so that our data set at this depth would include more hammer blows. Due to poor measurement quality, one of the nine blows comprising the final 1.3 ft was excluded from our data set.

Six sample returns were monitored while the 140 lb ram and standard split spoon sampler were in use. The overall average ETR and hammer blow rate was 97 percent and 47 blows per minute, respectively.

It was a pleasure to assist you and to participate on this project with the staff of Holt Services Inc. Please do not hesitate to contact us if you or other project participants have any questions about this report.

Sincerely,



July 24, 2018

Robert Miner, P.E.

Robert Miner Dynamic Testing, Inc.

ATTACHMENT B

HOLE NUMBER: CPT-01

CPT CONTRACTOR: In Situ Engineering

COMMENT: Parcel #31052800400300

LOCATION: Marysville

JOB NUMBER: PacTrust-219-01

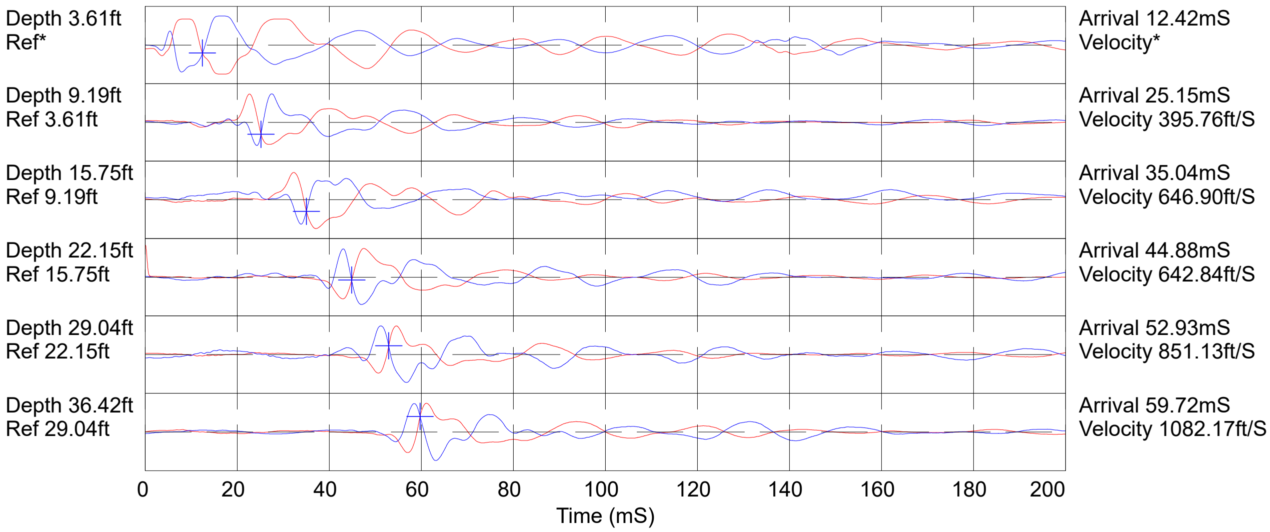
CUSTOMER:

TEST DATE: 12/11/2020 11:03:56 AM

OPERATOR: Walsh

CONE ID: DDG1263

HOLE NUMBER: CPT-01



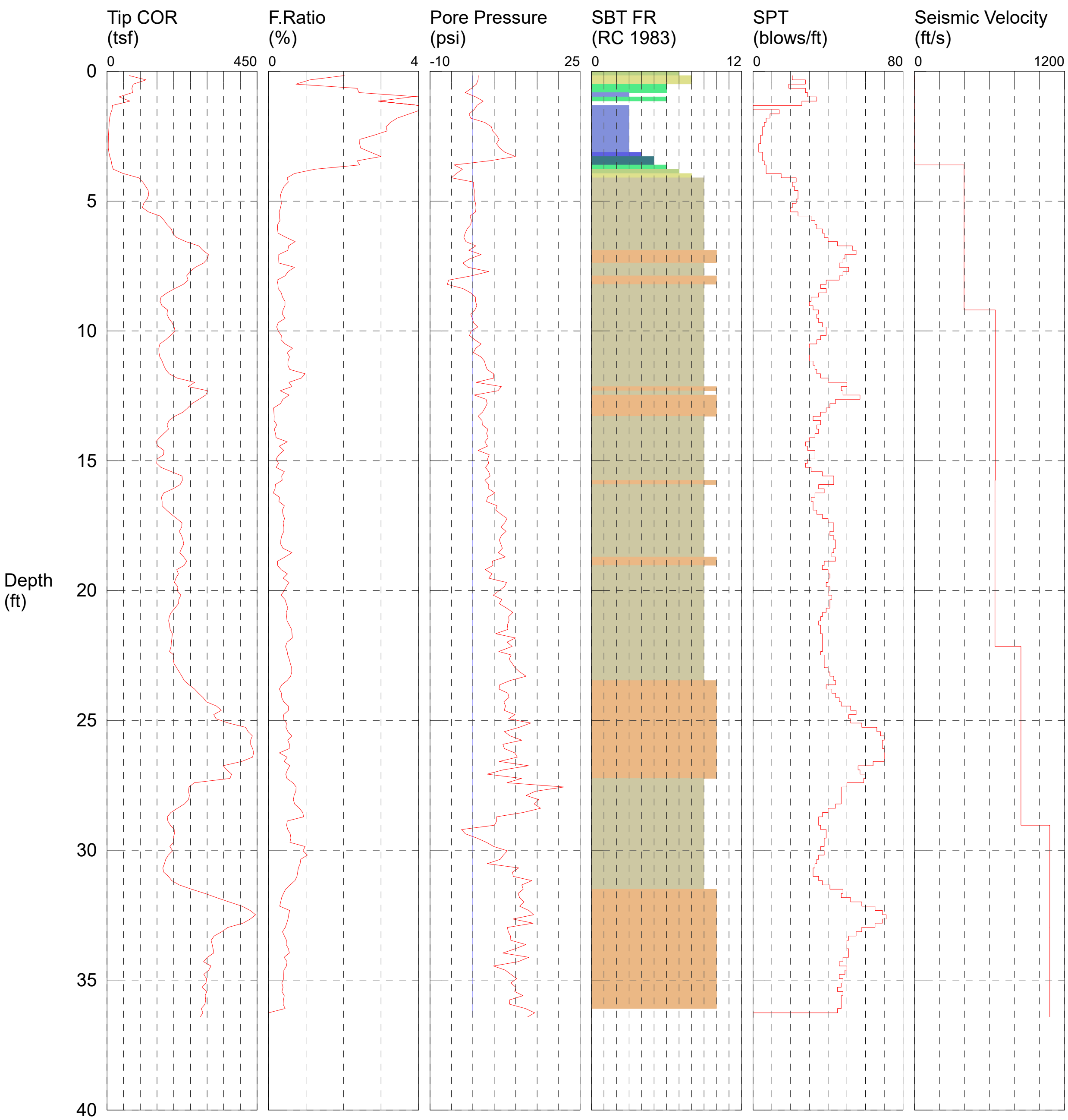
Hammer to Rod String Distance (ft): 2.79
* = Not Determined

JOB NUMBER: PacTrust-219-01

CPT-01

CPT CONTRACTOR: In Situ Engineering
CUSTOMER: Geodesign
LOCATION: Marysville
JOB NUMBER: PacTrust-219-01
COMMENT: Parcel #31052800400300
COMMENT:

OPERATOR: Walsh
CONE ID: DDG1263
TEST DATE: 12/11/2020 11:03:56 AM
PREDRILL: 0 ft
BACKFILL: 20% Grout & Bentonite Chips
SURFACE PATCH: None



TOTAL DEPTH: 36.417 ft

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

ATTACHMENT C

NELSON GEOTECHNICAL ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS

Main Office
17311 – 135th Ave NE, A-500
Woodinville, WA 98072
(425) 486-1669 · FAX (425) 481-2510

Engineering-Geology Branch
5526 Industry Lane, #2
East Wenatchee, WA 98802
(509) 665-7696 · FAX (509) 665-7692

February 19, 2019

Mr. Michael Neagle
Smokey Point Business Center, LLC
19305 Olympic View Drive
Edmonds, WA 98020

Preliminary Geotechnical Engineering Evaluation – Rev3
Industrial Property Development – Filling and Grading
15908 – 47th Avenue NE
Marysville, Washington
NGA File No. 1063718

Dear Mr. Neagle:

We are pleased to submit the attached report titled **"Preliminary Geotechnical Engineering Evaluation – Industrial Property Development - Filling and Grading – 15908 - 47th Avenue NE – Marysville, Washington."** This report summarizes our observations of the existing surface and subsurface conditions within the site, and provides general recommendations for the proposed site filling and grading. Our services were completed in general accordance with the proposal signed by you on December 11, 2018.

We visited the site on December 21, 2018 to observe the current site conditions and complete explorations of the subsurface conditions. The proposed development area consists of two rectangular-shaped parcels covering an approximate combined area of 57.48 acres. The proposed development areas of the site are generally relatively level. We understand that you intend to develop the site from its existing use as open space agriculture into several warehouse/light industrial structures throughout the property, with associated underground utilities and asphalt parking and access at a future date. Specific grading or stormwater handling plans were not available at the time this report was prepared; however, we understand that the entire site grades will be raised by five feet as part of the proposed fill and grade plan.

We monitored the excavation of fifteen test pit explorations within the site. Our explorations indicated that the site was generally underlain by loose, glacial recessional sands beneath a layer of topsoil across the entire site. Shallow groundwater was also encountered throughout the site. We have concluded that the site planned development is feasible from a geotechnical standpoint, provided our recommendations are followed during site development. We have recommended that geosynthetic reinforcement be placed prior to raising site grades with structural fill, for settlement and bearing capacity considerations.

In the attached report we have provided design infiltration rates based off grain-size analyses performed in accordance with the Department of Ecology's <u>Stormwater Management Manual for Western Washington, as amended in 2014</u>. We also include recommendations for erosion control, site preparation and grading, structural fill, foundations, retaining walls and site drainage.

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

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal

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Preliminary Geotechnical Engineering Evaluation
Industrial Property Development – Filling and Grading
15908 – 47th Avenue NE
Marysville, Washington

INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the proposed Industrial Property Development – Filling and Grading project in Marysville, Washington. The project site is located at 15908 – 47th Avenue NE in Marysville, Washington, as shown on the Vicinity Map in Figure 1. The tax parcel numbers for this property are 31052800400300 and 31053300100700. 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 filling and grading.

The site consists of two rectangular-shaped parcels covering an approximate combined area of 57.48 acres. The site is relatively level and currently vacant of existing development. However, the site is currently and has historically been used for agricultural purposes, primarily for growing grass crops. The property is bordered to the north, east and west by other vacant pasture properties and to the west by Hayho Creek. We understand that the proposed development will consist of constructing warehouse/light industrial structures throughout the property along with associated underground utilities and asphalt parking and drive areas. Specific stormwater handling plans were not available at the time this report was prepared. However, we understand that stormwater generated within the property may be directed to onsite infiltrations systems, if feasible. We also understand that the entire site will be raised by five feet as part of the proposed grading plan. The existing site layout is shown on the Site Plan in Figure 2.

For our use in preparing this report, we were provided with a copy of a preliminary site plan titled "Smokey Point Business Center," dated August 22, 2017 and produced by Lance Mueller & Associates.

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. A review of 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 was provided by NGA.
3. Installing piezometers within the test pit explorations throughout the site, as needed. Six groundwater monitoring wells were provided and installed by NGA. Monitoring wells to be remeasured up to three times.

4. Conducting laboratory analyses on selected soil samples, as needed.
5. Providing recommendations for earthwork, foundation support, and slab on grade subgrades.
6. Providing recommendations for pavement subgrade preparation.
7. Providing recommendations for temporary and permanent slopes.
8. Determining feasibility of on-site stormwater infiltration.
9. Providing long-term design infiltration rates based on grain-size analysis per the 2014 DOE Stormwater Manual.
10. Providing recommendations for infiltration system installation.
11. Providing recommendations for site drainage and erosion control.
12. Documenting the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The site consists of two rectangular-shaped parcels covering an approximate combined area of 57.48 acres. The site is relatively level and currently vacant of existing development. However, the site is currently and has historically been used for agricultural purposes, primarily for growing grass crops. The property is bordered to the north, east and west by other vacant pasture properties and to the west by Hayho Creek. The alignment of Hayho Creek has historically been modified for local agriculture practices. Very shallow, ponded surface water was observed within topographic low points throughout the property during our site visit on December 21, 2018, which have been indicated in a blue-shaded overlay on the Site Plan in Figure 2. Stagnant surface water was observed at the base of the ditch line separating the two subject parcels. Reconnaissance of LiDAR imagery provided by Snohomish County PDS Map portal suggests surface water expressions may be related to historical drainage modifications such as drainage tile installations within the central and southwestern portions of the northernmost parcel, although no subsurface drainage installations were found in explorations.

Subsurface Conditions

Geology: The geologic units for this area are shown in the Geologic map of the Arlington West 7.5-minute Quadrangle, Snohomish County, Washington, by James P. Minard (USGS 1985). The site is mapped as surficial deposits of the Vashon Stade, consisting of glacial recessional outwash of the Marysville Member (Qvrm). The Marysville Sand member of the recessional outwash is described as well-drained stratified sand and gravel. Our explorations generally encountered a mantling, surficial layer of topsoil underlain by a layer of loose to medium dense, fine to coarse sand with gravel consistent with the description of the Marysville Sand at depth.

Explorations: The subsurface conditions within the site were explored on December 21, 2018 by excavating fifteen test pit explorations to depths of 6.0 feet below the existing ground surface using a track-mounted backhoe. 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 Figures 4 through 7. We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the logs should be reviewed.

All explorations exposed a surficial layer of dark brown, organic-rich topsoil extending to depths between 0.8 and 1.5 feet below the existing ground surface. Underlying the topsoil in every exploration, we encountered an intermediate layer between 0.9 and 2.2 feet in thickness of brown to tan-gray, silty, fine to medium sand with varying amounts of gravel and iron oxidation staining. The intermediate layer was encountered in a loose to medium dense condition, and we interpreted the material to be weathered, native, glacial recessional outwash. Underlying the intermediate layer to depths of 6.0 feet, all test pits exposed gray, fine to coarse sand with varying amounts of gravel in a loose condition. We interpreted these sediments to be the unweathered glacial recessional sands mapped in the area.

Hydrogeologic Conditions

Surface water was present in gentle depressions throughout the site, as documented on the Site Plan in Figure 2. Hayho Creek borders the site to the west. Groundwater seepage was observed at depths between 2.8 and 4.5 feet below existing ground surface in every exploration on the site. We interpret this groundwater to be associated with the regional groundwater table in the area. We would expect the levels of groundwater to slightly decrease during drier times of the year and increase during wetter periods. We also installed six piezometers within Test Pits 1, 2, 3, 4, 5, and 6. Due to our explorations being performed near the start of the winter wet weather season, we anticipate groundwater levels to likely rise during wetter periods. The seasonal high groundwater elevation would need to be determined in the wetter winter months by monitoring the groundwater piezometers during that time.

SENSITIVE AREA EVALUATION

Seismic Hazard

We reviewed the 2018 International Building Code (IBC) for seismic site classification for this project. Since dense 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 2018 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – 2018 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	1.089	0.424	1.064	1.576	0.773	0.446

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 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 glacial sand deposits interpreted to underlie the site have a low to moderate potential for liquefaction or amplification of ground motion.

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 native soils on the northernmost portion of the site as Custer fine sandy loam, and areas in the central and southern portions of the site as Norma loam. The native soils on site are listed as having a slight erosion hazard. It is our opinion that the erosion hazard for site soils should be low in areas where vegetation is not disturbed.

LABORATORY ANALYSES

We performed five grain-size analyses with moisture contents on selected soil samples obtained from the site. The laboratory tests were performed on samples taken from Test Pit One at 5.0 feet, Three at 2.0 and 5.0 feet, Six at 2.0 feet, and Nine at 4.0 feet below the ground surface. The results of the sieve analyses are attached as Figures 8 through 12.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion from a geotechnical standpoint that the site is compatible with the proposed development, provided the geotechnical recommendations presented in this report are incorporated into project plans and followed during construction. Our explorations indicated that the site is underlain by a 0.9- to 2.2-foot-thick mantle of topsoil, underlain by variably-loose, native, outwash soils. Due to the variably-loose condition of the native subsurface soils, we recommend that provisions be made to stabilize the subgrade to avoid potential post-construction total and differential settlements which could cause distress to proposed structures. Placement of structural fill directly on top of present materials on the site could result in bearing capacity failures and significant long-term settlement. Stabilization provisions discussed in this report, generally consisting of geosynthetic reinforcement overlain by several feet of structural fill, should be implemented to reduce this risk.

We should stress that even with the placement of geogrid and structural fill, some post-construction settlement should be anticipated and planned for. It may be prudent to allow a period of six months or more between fill placement and the start of actual site development to allow for some of the settlement to take place, thus reducing potential advance impacts to the future structures and hard surfaces. In any case, individual building subgrades, along with pavement and other hard surface subgrades, should be specifically evaluated by us on a case-by-case basis during final design, and specific design and construction recommendations and risk analysis provided at that time.

We also evaluated the native soils for infiltration feasibility based on the 2014 WSDOE Stormwater Management Manual for Western Washington. We completed five grain-size distribution analyses on the recessional outwash materials to establish a design infiltration rate. Feasibility for infiltration is based on permeability among a number of other factors, including groundwater separation. Based exclusively on the grain-size analyses, it is our opinion that on-site stormwater infiltration is feasible within this site. However, it is unlikely that sufficient separation can be maintained between the groundwater table and an infiltration basin within the native soils. This will ultimately depend on the final site grades and types of infiltration systems.

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. We understand that a source for imported structural fill has been identified. NGA should be retained at the time of construction to determine if the soils exported from the source are appropriate for use as structural fill material at the site during export operations.

Erosion Control

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

If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed and the exposed subgrade maintained in a semi-dry condition. After the water has been controlled and erosion control measures are implemented, the site should be grubbed using large excavators equipped with wide tracks and smooth buckets. The exposed subgrade should not be compacted if wet, as compaction of a wet subgrade may result in further disturbance of the native soils.

After site mowing and grubbing has been completed and the surficial plant material has been exported from the site, geosynthetic reinforcement should be placed across all areas to receive structural fill. The prepared subgrade should be protected from construction traffic, and surface water should be diverted around areas of prepared subgrade. A blanket of rock spalls should be used in construction access areas if wet conditions are prevalent. The thickness of this rock spall layer should be based on subgrade performance at the time of construction. We recommend NAUE Combigrid 30/30 Q1, or equivalent, for use as geosynthetic reinforcement to provide reinforcement, filtration, and separation between the imported fill and underlying, native soils and sediments on the site. Geosynthetic layers should be placed so that overlap of at least 2.0 feet is maintained between geogrid sections. We should observe placement of geosynthetic reinforcement to ensure proper installation. After installation of any composite geogrid buried by structural fill, excavation through the reinforcement layer should be avoided to prevent the surrounding reinforcement from becoming compromised. Careful consideration should be made during planning the construction sequence for utility or structure installation.

Temporary and Permanent Slopes

We do not anticipate major cuts as a result of grading associated with this project, however, 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 water or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since they are continuously at the job site, able to observe the soil and groundwater conditions encountered, and able to monitor the nature and condition of the cut slopes.

The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 2 Horizontal to 1 Vertical (2H:1V) or shored. This should be further evaluated at the time of construction. If significant groundwater seepage or surface water flow were encountered, we would expect that shoring or trench boxes be considered for the planned cuts. We are available to provide recommendations for shoring, if needed, as the project plans are developed. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut heights and inclinations conform to appropriate OSHA/WISHA regulations.

Foundations

We understand that the planned structure foundations will be placed atop the structural fill planned for this site. We recommend that the bottom of the foundations be at least 3.0 feet above the geogrid layer and be supported on structural fill compacted to at least 95 percent of the ASTM D1557 dry density. Conventional 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 placed on at least 3.0 feet of competent structural fill that is underlain by the recommended geogrid. 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 3.0 inches total and one inch differential between adjacent footings or across a distance of about 20 feet, based on our experience.

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 medium dense soils or compacted fill should be used as backfill against the front of the footing. We recommend the upper one foot of soil be neglected when calculating passive resistance.

Retaining Walls

We do not anticipate the need for retaining walls on this site; however, should any walls be utilized, they should be designed and constructed as outlined above and hereon. 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 assume a horizontal ground surface behind the wall for a distance of at least the subsurface height of the wall, not accounting 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 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 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). We understand that an export site likely consisting of glacial till and advance outwash soils has been selected as a source for structural fill import. We should be retained to evaluate all proposed structural fill material prior to placement, ideally at the location of export to prevent deleterious material from being transported to the site indiscriminately. In any case, the fill should be imported to the site and placed during extended periods of dry weather.

Fill Placement: Following subgrade preparation and placement of the recommended geogrid, 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 12 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 permanently 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 four grain-size analyses on selected soil samples obtained within the site in accordance with the 2014 Washington State Department of Ecology (WSDOE) Stormwater Management Manual for Western Washington. Grain-size analyses were performed on selected samples from Test Pit One at 5.0 feet, Three at 2.0 and 5.0 feet, Six at 2.0 feet, and Nine at 4.0 feet below the ground surface. The results of the sieve analyses are presented as Figures 8 through 12. Based on the laboratory analysis, the soils encountered in our explorations within the proposed infiltration area meet the classification of sand in the USDA Textural Triangle.

An equation provided in Section 3.3.6.3 of the 2014 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 in the range of 13.35 to 25.74 inches per hour were calculated for the near-surface weathered glacial soils. Grain-size analyses on deeper, unweathered recessional outwash soils returned infiltration rates in the range of 27.14 to 93.27 inches per hour. We also referenced Table 3.3.1 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 0.90, 0.40, and 0.90 were utilized in this equation for CF_v, CF_t, CF_m, respectively. Using these correction factors, we applied it to the most conservative rate obtained from the grain-size analysis calculations for each soil, which are 13.35 inches per hour and 27.14 inches per hour for weathered and unweathered outwash soils, respectively. We calculated a long-term design infiltration rate of 4.33 inches per hour and 8.79 inches per hour for the weathered and unweathered native soils, respectively. We recommend that any infiltration systems be extended down through any unsuitable topsoils and founded within the native, granular outwash soils. Based on our explorations, the native, weathered outwash soils should be encountered between one and three feet below the existing ground surface. Unweathered outwash soils should be encountered at depths three feet and greater below the existing ground surface. We should be retained during construction to evaluate the soils exposed in the infiltration systems to verify that the soils are appropriate for infiltration.

The stormwater manual recommends a minimum three-foot separation between the base of an infiltration system and any underlying bedrock, impermeable horizon, or groundwater. Groundwater was encountered in each of our explorations at depths as shallow as 2.8 feet below the existing surface. It is likely that groundwater will greatly impact the design and performance of infiltration systems on this site. If infiltration systems are proposed, mounding analyses should be completed to verify appropriate sizing.

Shallow Infiltration System Design

Due to the potential for relatively shallow seasonal high groundwater within the site, we recommend that be considered shallow infiltration systems such as rain gardens or bio-retention swales to maintain adequate separation from the seasonal high groundwater. We recommend that the rain gardens and bio-retention swales be designed and sized in accordance with the 2014 WSDOE Stormwater Management Manual for Western Washington utilizing the above recommended long-term design infiltration rate. The base of the shallow infiltration systems should be extended through the upper silty soils and down to expose the granular outwash soils. Based on our explorations, these granular outwash soils should typically be encountered from 1.0 to 3.0 feet below the existing ground surface. Due to the overall depth of the granular outwash soils, the base of the shallow infiltration system may be higher than the granular outwash soils at depth. If this is the case, we recommend that the base of the shallow infiltration system be overexcavated down to expose the granular outwash soils and the overexcavation then backfilled with pea gravel up to the base of the infiltration system. Amended soils could then be placed over the pea gravel. We also recommend that an appropriate overflow system be incorporated into the design of the shallow infiltration systems if feasible. The inside and outside slopes of the shallow infiltration systems should be no steeper than 3H:1V. We should review final infiltration system design and monitor the system installation.

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 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 appropriate discharge system. The overflow water should be directed to discharge into an appropriate location.

Subsurface Drainage: If groundwater is encountered during construction, we recommend that the contractor slope the bottom of the excavation and collect the water into ditches and small sump pits where the water can be pumped out and routed into a permanent storm drain. Excavations below the groundwater table may require more elaborate dewatering systems. This should be determined during final design.

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.

CONSTRUCTION MONITORING

We should be retained to provide construction monitoring services during the earthwork phase of the project to evaluate subgrade conditions, geosynthetic reinforcement installation, temporary cut conditions, fill compaction, and drainage system installation.

USE OF THIS REPORT

NGA has prepared this preliminary report for Mr. Michael Neagle 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.

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.

Carston Curd

Carston T. Curd, GIT
Staff Geologist II

Khaled M. Shawish, PE
Principal

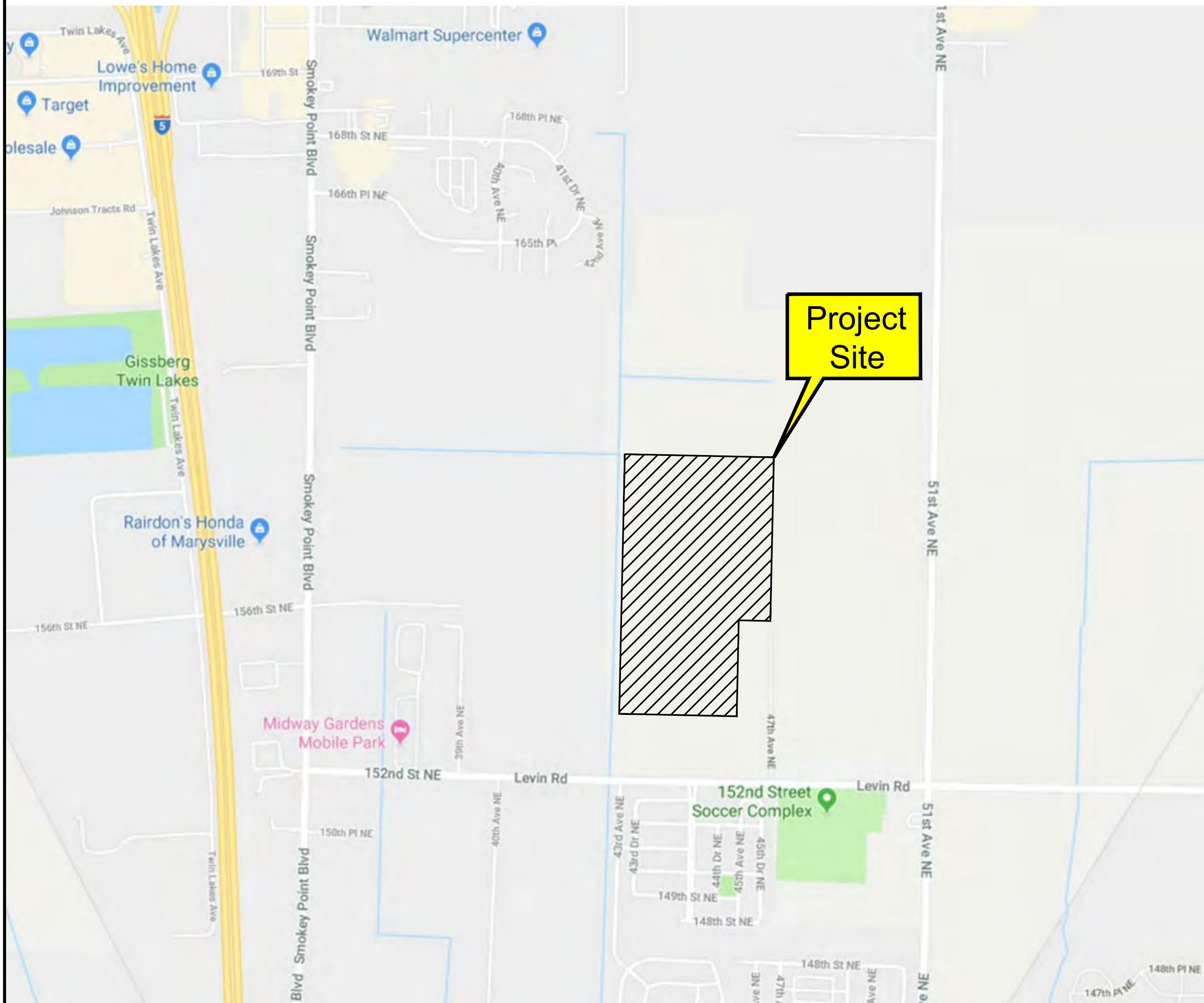
CTC:KMS:dy

Twelve Figures Attached

VICINITY MAP

Not to Scale

N



Marysville, WA

Project Number	Marysville Industrial Property Development Vicinity Map	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
1063718		Geotechnical Engineers & Geologists	1	1/7/19	Original	DPN	CTC
Figure 1							

NGA
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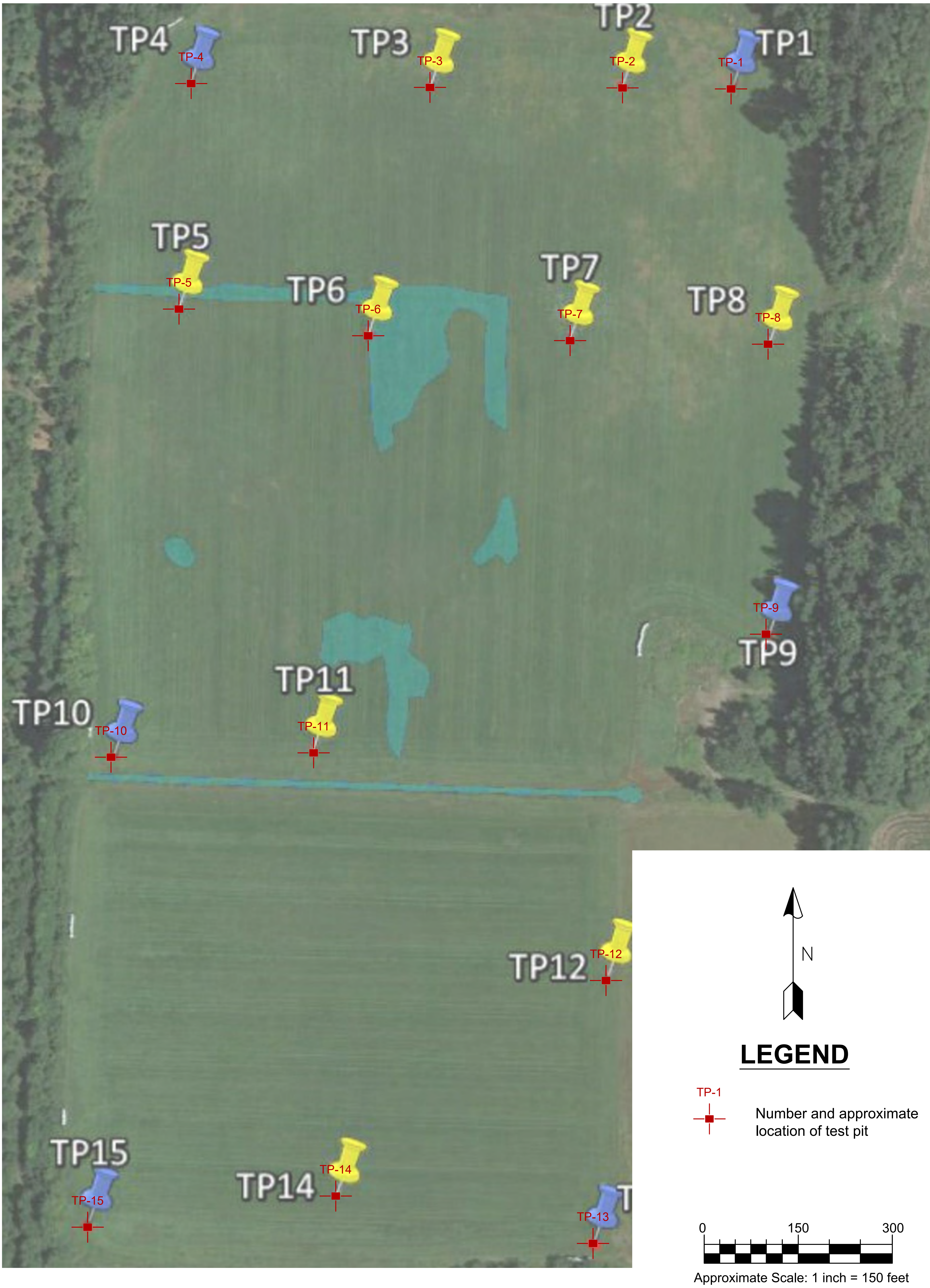

Reference: Site Plan based on field measurements, observations, and aerial parcel map review.

Project Number	Marysville Industrial Property Development Site Plan	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
1063718		Geotechnical Engineers & Geologists	1	1/7/19	Original	DPN	CTC
Figure 2							

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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	Marysville Industrial Property Development Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC.
1063718		GEOTECHNICAL ENGINEERS & GEOLOGISTS
Figure 3		

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No.	Date	Revision	By	CK
1	1/7/19	Original	DPN	CTC

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LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 1.4		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.4 – 2.5	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH TRACE ORGANIC DEBRIS AND IRON OXIDATION STAINING (LOOSE-MEDIUM DENSE, WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY FINE TO COARSE SAND WITH SILT (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 3.0 FEET MODERATE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 2.8 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 2.5 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT TWO		
0.0 – 1.0		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.0 – 2.8	SM	BROWN SILTY FINE TO MEDIUM SAND (LOOSE-MEDIUM DENSE, WET) (**WEATHERED OUTWASH**)
2.8 – 6.0	SP-SM	GRAY FINE TO COARSE SAND WITH GRAVEL AND SILT (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 3.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 2.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT THREE		
0.0 – 0.8		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
0.8 – 3.0	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
3.0 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH TRACE FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT FOUR		
0.0 – 0.9		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
0.9 – 2.1	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.1 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH TRACE FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018

LOG OF EXPLORATION

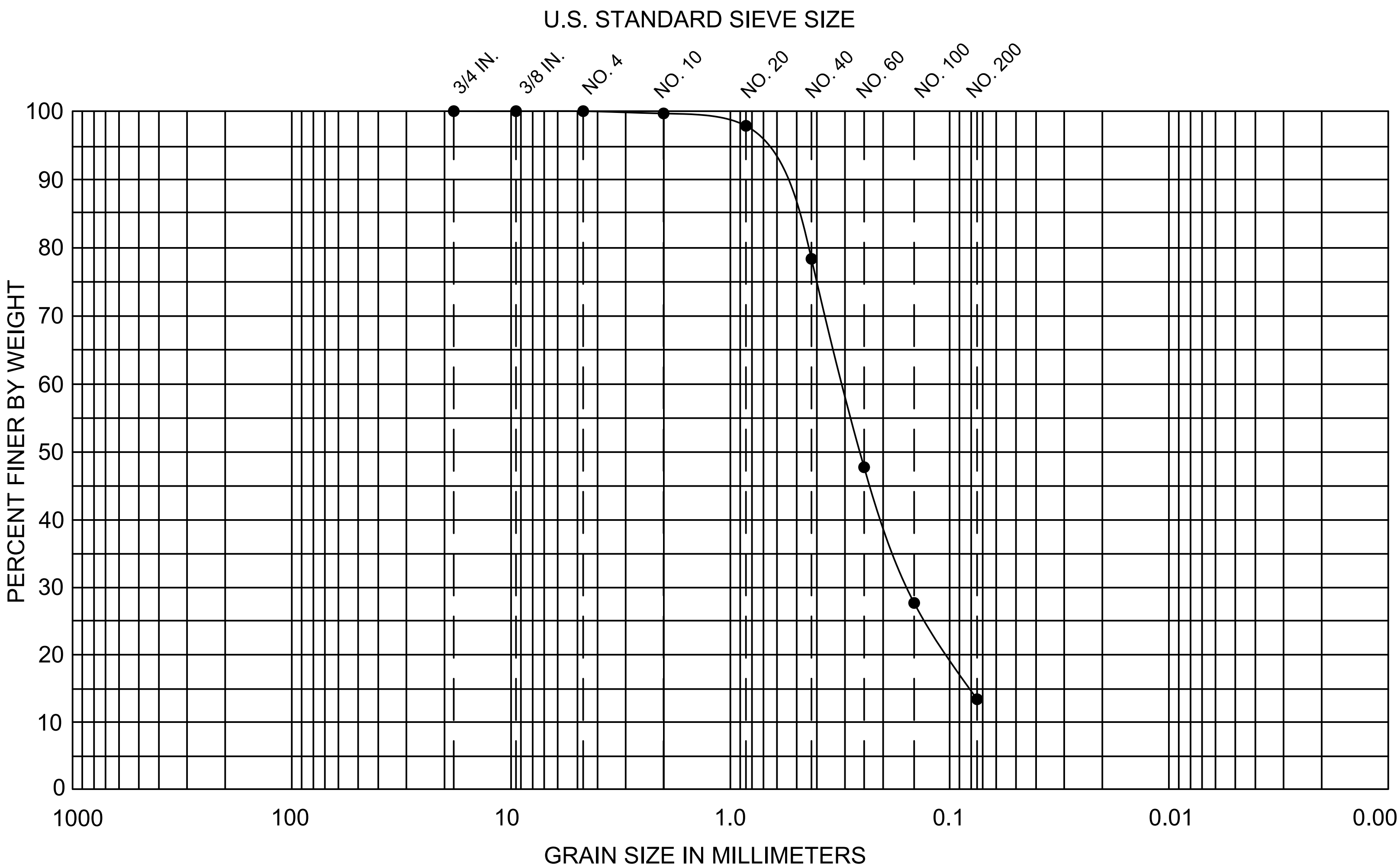
DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 1.2		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.2 – 4.0	SM	TAN-GRAY TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, WET) (**WEATHERED OUTWASH**)
4.0 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH GRAVEL (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 4.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MINOR TEST PIT CAVING WAS ENCOUNTERED BELOW 4.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT SIX		
0.0 – 1.1		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, WET) **(TOPSOIL)**
1.1 – 2.0	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.0 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH TRACE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.5 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 4.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT SEVEN		
0.0 – 1.5		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.5 – 3.5	SM	BROWN SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
3.5 – 6.0	SP	GRAY FINE TO COARSE SAND WITH FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 5.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 4.5 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT EIGHT		
0.0 – 1.4		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.4 – 2.5	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE-MEDIUM DENSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY-BROWN SILTY FINE TO COARSE SAND WITH TRACE FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 3.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT NINE		
0.0 – 1.2		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(<u>TOPSOIL</u>)**
1.2 – 2.5	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**<u>WEATHERED OUTWASH</u>**)
2.5 – 6.0	SP-SM	GRAY FINE TO COARSE SAND WITH SILT AND GRAVEL (LOOSE, WET) (**<u>RECESSIONAL OUTWASH</u>**)
		SAMPLES WERE COLLECTED AT 4.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT TEN		
0.0 – 0.8		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(<u>TOPSOIL</u>)**
0.8 – 2.8	SM	TAN-GRAY TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**<u>WEATHERED OUTWASH</u>**)
2.8 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH GRAVEL (LOOSE, WET) (**<u>RECESSIONAL OUTWASH</u>**)
		NO SAMPLES WERE COLLECTED SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MINOR TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT ELEVEN		
0.0 – 1.0		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(<u>TOPSOIL</u>)**
1.0 – 2.6	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**<u>WEATHERED OUTWASH</u>**)
2.6 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE-MEDIUM DENSE, WET) (**<u>RECESSIONAL OUTWASH</u>**)
		SAMPLES WERE COLLECTED AT 2.6 AND 4.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT TWELVE		
0.0 – 1.0		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(<u>TOPSOIL</u>)**
1.0 – 2.5	SM	TAN-GRAY TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**<u>WEATHERED OUTWASH</u>**)
2.5 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (MEDIUM DENSE, WET) (**<u>RECESSIONAL OUTWASH</u>**)
		NO SAMPLES WERE COLLECTED SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 4.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT THIRTEEN		
0.0 – 1.3		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.3 – 2.3	SM	TAN TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.3 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 5.0 FEET MODERATE TO SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.5 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT FOURTEEN		
0.0 – 1.1		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.1 – 2.0	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.0 – 3.3	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE, WET) (**RECESSIONAL OUTWASH**)
3.3 – 6.0	SP-SM	BROWN SILTY COARSE SAND WITH GRAVEL (LOOSE-MEDIUM DENSE, WET)
		NO SAMPLES WERE COLLECTED MODERATE TO SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT FIFTEEN		
0.0 – 1.1		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.1 – 2.5	SM	BROWN TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		NO SAMPLES WERE COLLECTED MODERATE TO SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018



<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP-SM	TP-1	5.0 feet	Fine to medium sand with silt	Gravel = 0% Sand = 87% Silt/Clay = 13%

Project Number
1063718

Figure 8

Marysville Industrial
Property Development
Sieve Analysis

NGA
NELSON GEOTECHNICAL
ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS

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(509) 665-7696 / Fax: 665-7692

No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC

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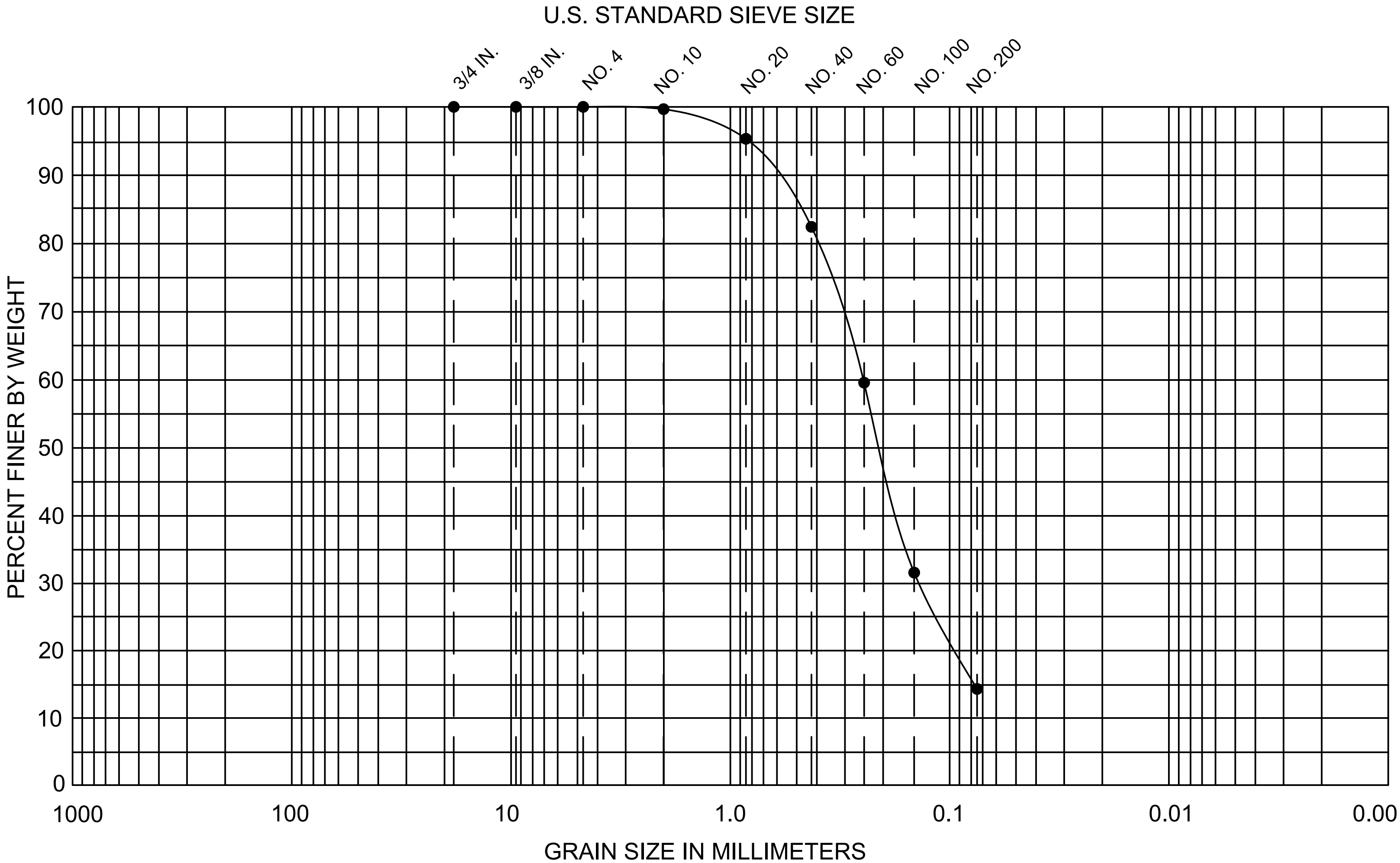

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP-SM	TP-3	2.0 feet	Fine to medium sand with silt	Gravel = 0% Sand = 86% Silt/Clay = 14%

Project Number
1063718
Figure 9

Marysville Industrial
Property Development
Sieve Analysis

NGA
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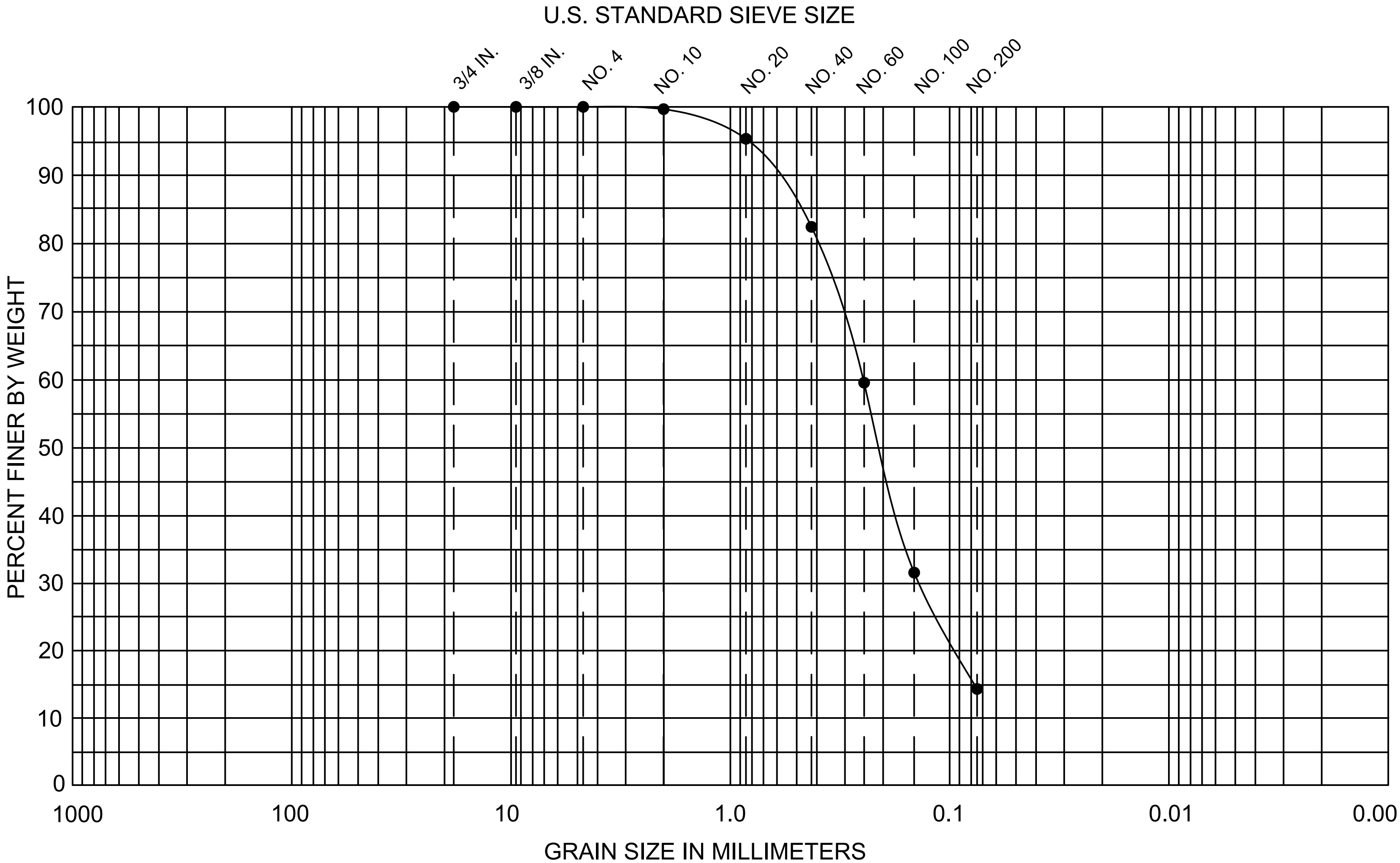
No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP	TP-3	5.0 feet	Fine to medium sand with trace gravel and silt	Gravel = 6% Sand = 90% Silt/Clay = 4%

Project Number
1063718
Figure 10

Marysville Industrial
Property Development
Sieve Analysis

NGA
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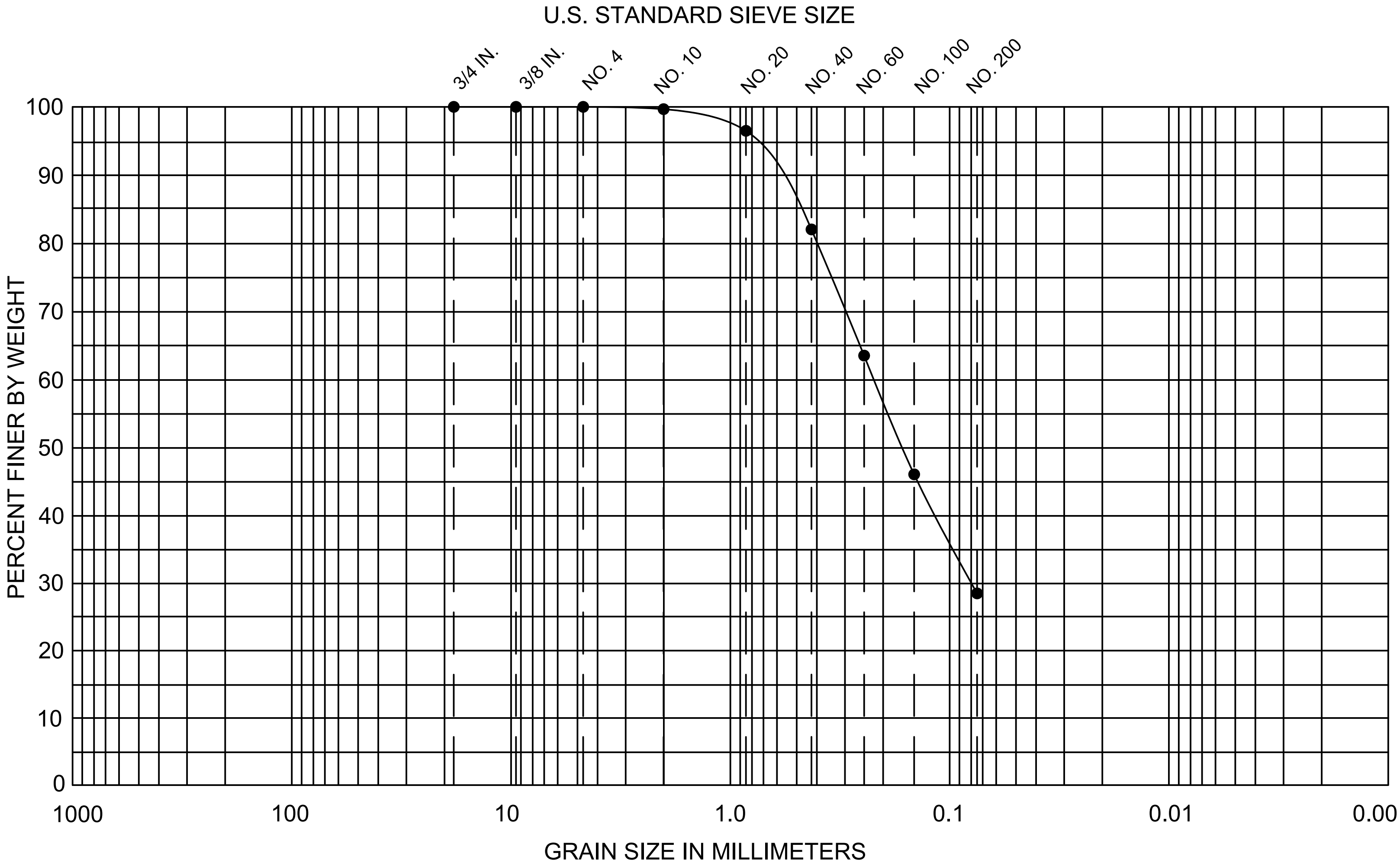
No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SM	TP-6	2.0 feet	Silty, fine to medium sand	Gravel = 0% Sand = 72% Silt/Clay = 28%

Project Number
1063718
Figure 11

Marysville Industrial
Property Development
Sieve Analysis

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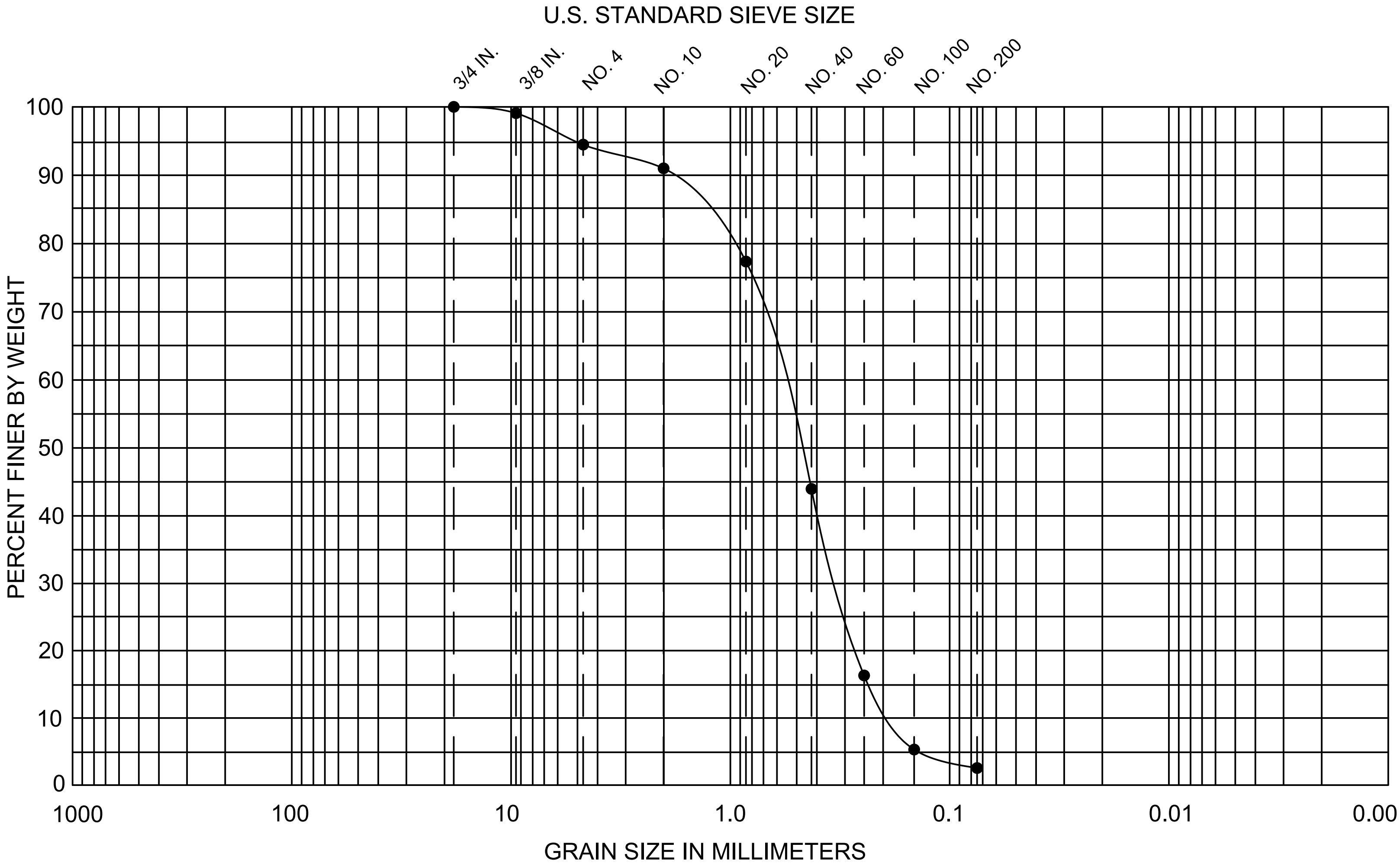
No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP	TP-9	4.0 feet	Fine to medium sand with trace gravel and silt	Gravel = 5% Sand = 92% Silt/Clay = 3%

Project Number
1063718

Figure 12

Marysville Industrial
Property Development
Sieve Analysis

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No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC

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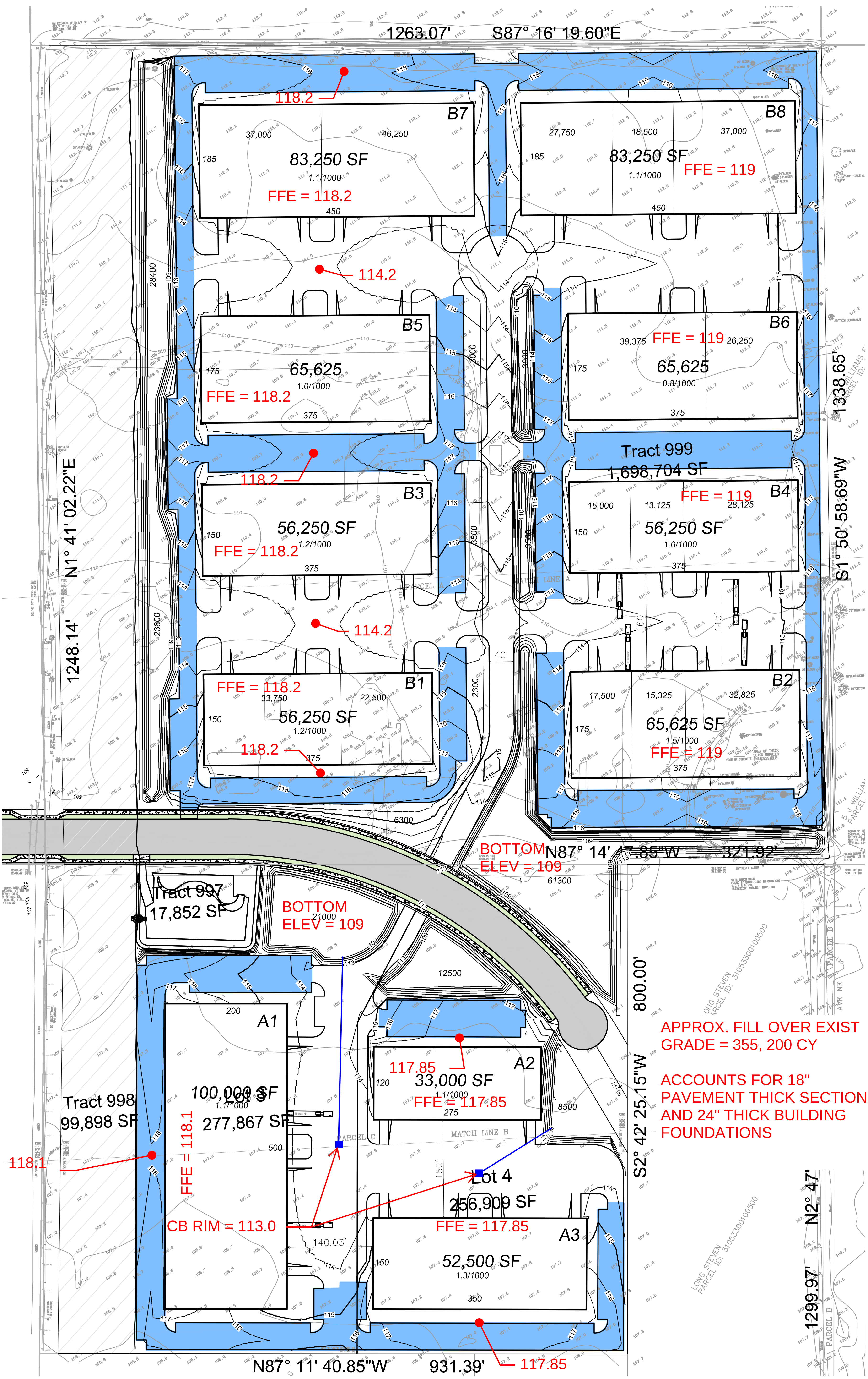
ATTACHMENT D



MARYSVILLE - OPTION 1A

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11/5/2020

APPENDIX D

APPENDIX D

NELSON GEOTECHNICAL ASSOCIATES, INC. REPORT – PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION

NELSON GEOTECHNICAL ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS

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February 19, 2019

Mr. Michael Neagle
Smokey Point Business Center, LLC
19305 Olympic View Drive
Edmonds, WA 98020

Preliminary Geotechnical Engineering Evaluation – Rev3
Industrial Property Development – Filling and Grading
15908 – 47th Avenue NE
Marysville, Washington
NGA File No. 1063718

Dear Mr. Neagle:

We are pleased to submit the attached report titled **"Preliminary Geotechnical Engineering Evaluation – Industrial Property Development - Filling and Grading – 15908 - 47th Avenue NE – Marysville, Washington."** This report summarizes our observations of the existing surface and subsurface conditions within the site, and provides general recommendations for the proposed site filling and grading. Our services were completed in general accordance with the proposal signed by you on December 11, 2018.

We visited the site on December 21, 2018 to observe the current site conditions and complete explorations of the subsurface conditions. The proposed development area consists of two rectangular-shaped parcels covering an approximate combined area of 57.48 acres. The proposed development areas of the site are generally relatively level. We understand that you intend to develop the site from its existing use as open space agriculture into several warehouse/light industrial structures throughout the property, with associated underground utilities and asphalt parking and access at a future date. Specific grading or stormwater handling plans were not available at the time this report was prepared; however, we understand that the entire site grades will be raised by five feet as part of the proposed fill and grade plan.

We monitored the excavation of fifteen test pit explorations within the site. Our explorations indicated that the site was generally underlain by loose, glacial recessional sands beneath a layer of topsoil across the entire site. Shallow groundwater was also encountered throughout the site. We have concluded that the site planned development is feasible from a geotechnical standpoint, provided our recommendations are followed during site development. We have recommended that geosynthetic reinforcement be placed prior to raising site grades with structural fill, for settlement and bearing capacity considerations.

In the attached report we have provided design infiltration rates based off grain-size analyses performed in accordance with the Department of Ecology's Stormwater Management Manual for Western Washington, as amended in 2014. We also include recommendations for erosion control, site preparation and grading, structural fill, foundations, retaining walls and site drainage.

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

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal

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Preliminary Geotechnical Engineering Evaluation
Industrial Property Development – Filling and Grading
15908 – 47th Avenue NE
Marysville, Washington

INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the proposed Industrial Property Development – Filling and Grading project in Marysville, Washington. The project site is located at 15908 – 47th Avenue NE in Marysville, Washington, as shown on the Vicinity Map in Figure 1. The tax parcel numbers for this property are 31052800400300 and 31053300100700. 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 filling and grading.

The site consists of two rectangular-shaped parcels covering an approximate combined area of 57.48 acres. The site is relatively level and currently vacant of existing development. However, the site is currently and has historically been used for agricultural purposes, primarily for growing grass crops. The property is bordered to the north, east and west by other vacant pasture properties and to the west by Hayho Creek. We understand that the proposed development will consist of constructing warehouse/light industrial structures throughout the property along with associated underground utilities and asphalt parking and drive areas. Specific stormwater handling plans were not available at the time this report was prepared. However, we understand that stormwater generated within the property may be directed to onsite infiltrations systems, if feasible. We also understand that the entire site will be raised by five feet as part of the proposed grading plan. The existing site layout is shown on the Site Plan in Figure 2.

For our use in preparing this report, we were provided with a copy of a preliminary site plan titled "Smokey Point Business Center," dated August 22, 2017 and produced by Lance Mueller & Associates.

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. A review of 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 was provided by NGA.
3. Installing piezometers within the test pit explorations throughout the site, as needed. Six groundwater monitoring wells were provided and installed by NGA. Monitoring wells to be remeasured up to three times.

4. Conducting laboratory analyses on selected soil samples, as needed.
5. Providing recommendations for earthwork, foundation support, and slab on grade subgrades.
6. Providing recommendations for pavement subgrade preparation.
7. Providing recommendations for temporary and permanent slopes.
8. Determining feasibility of on-site stormwater infiltration.
9. Providing long-term design infiltration rates based on grain-size analysis per the 2014 DOE Stormwater Manual.
10. Providing recommendations for infiltration system installation.
11. Providing recommendations for site drainage and erosion control.
12. Documenting the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The site consists of two rectangular-shaped parcels covering an approximate combined area of 57.48 acres. The site is relatively level and currently vacant of existing development. However, the site is currently and has historically been used for agricultural purposes, primarily for growing grass crops. The property is bordered to the north, east and west by other vacant pasture properties and to the west by Hayho Creek. The alignment of Hayho Creek has historically been modified for local agriculture practices. Very shallow, ponded surface water was observed within topographic low points throughout the property during our site visit on December 21, 2018, which have been indicated in a blue-shaded overlay on the Site Plan in Figure 2. Stagnant surface water was observed at the base of the ditch line separating the two subject parcels. Reconnaissance of LiDAR imagery provided by Snohomish County PDS Map portal suggests surface water expressions may be related to historical drainage modifications such as drainage tile installations within the central and southwestern portions of the northernmost parcel, although no subsurface drainage installations were found in explorations.

Subsurface Conditions

Geology: The geologic units for this area are shown in the Geologic map of the Arlington West 7.5-minute Quadrangle, Snohomish County, Washington, by James P. Minard (USGS 1985). The site is mapped as surficial deposits of the Vashon Stade, consisting of glacial recessional outwash of the Marysville Member (Qvrm). The Marysville Sand member of the recessional outwash is described as well-drained stratified sand and gravel. Our explorations generally encountered a mantling, surficial layer of topsoil underlain by a layer of loose to medium dense, fine to coarse sand with gravel consistent with the description of the Marysville Sand at depth.

Explorations: The subsurface conditions within the site were explored on December 21, 2018 by excavating fifteen test pit explorations to depths of 6.0 feet below the existing ground surface using a track-mounted backhoe. 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 Figures 4 through 7. We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the logs should be reviewed.

All explorations exposed a surficial layer of dark brown, organic-rich topsoil extending to depths between 0.8 and 1.5 feet below the existing ground surface. Underlying the topsoil in every exploration, we encountered an intermediate layer between 0.9 and 2.2 feet in thickness of brown to tan-gray, silty, fine to medium sand with varying amounts of gravel and iron oxidation staining. The intermediate layer was encountered in a loose to medium dense condition, and we interpreted the material to be weathered, native, glacial recessional outwash. Underlying the intermediate layer to depths of 6.0 feet, all test pits exposed gray, fine to coarse sand with varying amounts of gravel in a loose condition. We interpreted these sediments to be the unweathered glacial recessional sands mapped in the area.

Hydrogeologic Conditions

Surface water was present in gentle depressions throughout the site, as documented on the Site Plan in Figure 2. Hayho Creek borders the site to the west. Groundwater seepage was observed at depths between 2.8 and 4.5 feet below existing ground surface in every exploration on the site. We interpret this groundwater to be associated with the regional groundwater table in the area. We would expect the levels of groundwater to slightly decrease during drier times of the year and increase during wetter periods. We also installed six piezometers within Test Pits 1, 2, 3, 4, 5, and 6. Due to our explorations being performed near the start of the winter wet weather season, we anticipate groundwater levels to likely rise during wetter periods. The seasonal high groundwater elevation would need to be determined in the wetter winter months by monitoring the groundwater piezometers during that time.

SENSITIVE AREA EVALUATION

Seismic Hazard

We reviewed the 2018 International Building Code (IBC) for seismic site classification for this project. Since dense 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 2018 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – 2018 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	1.089	0.424	1.064	1.576	0.773	0.446

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 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 glacial sand deposits interpreted to underlie the site have a low to moderate potential for liquefaction or amplification of ground motion.

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 native soils on the northernmost portion of the site as Custer fine sandy loam, and areas in the central and southern portions of the site as Norma loam. The native soils on site are listed as having a slight erosion hazard. It is our opinion that the erosion hazard for site soils should be low in areas where vegetation is not disturbed.

LABORATORY ANALYSES

We performed five grain-size analyses with moisture contents on selected soil samples obtained from the site. The laboratory tests were performed on samples taken from Test Pit One at 5.0 feet, Three at 2.0 and 5.0 feet, Six at 2.0 feet, and Nine at 4.0 feet below the ground surface. The results of the sieve analyses are attached as Figures 8 through 12.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion from a geotechnical standpoint that the site is compatible with the proposed development, provided the geotechnical recommendations presented in this report are incorporated into project plans and followed during construction. Our explorations indicated that the site is underlain by a 0.9- to 2.2-foot-thick mantle of topsoil, underlain by variably-loose, native, outwash soils. Due to the variably-loose condition of the native subsurface soils, we recommend that provisions be made to stabilize the subgrade to avoid potential post-construction total and differential settlements which could cause distress to proposed structures. Placement of structural fill directly on top of present materials on the site could result in bearing capacity failures and significant long-term settlement. Stabilization provisions discussed in this report, generally consisting of geosynthetic reinforcement overlain by several feet of structural fill, should be implemented to reduce this risk.

We should stress that even with the placement of geogrid and structural fill, some post-construction settlement should be anticipated and planned for. It may be prudent to allow a period of six months or more between fill placement and the start of actual site development to allow for some of the settlement to take place, thus reducing potential advance impacts to the future structures and hard surfaces. In any case, individual building subgrades, along with pavement and other hard surface subgrades, should be specifically evaluated by us on a case-by-case basis during final design, and specific design and construction recommendations and risk analysis provided at that time.

We also evaluated the native soils for infiltration feasibility based on the 2014 WSDOE Stormwater Management Manual for Western Washington. We completed five grain-size distribution analyses on the recessional outwash materials to establish a design infiltration rate. Feasibility for infiltration is based on permeability among a number of other factors, including groundwater separation. Based exclusively on the grain-size analyses, it is our opinion that on-site stormwater infiltration is feasible within this site. However, it is unlikely that sufficient separation can be maintained between the groundwater table and an infiltration basin within the native soils. This will ultimately depend on the final site grades and types of infiltration systems.

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. We understand that a source for imported structural fill has been identified. NGA should be retained at the time of construction to determine if the soils exported from the source are appropriate for use as structural fill material at the site during export operations.

Erosion Control

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

If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed and the exposed subgrade maintained in a semi-dry condition. After the water has been controlled and erosion control measures are implemented, the site should be grubbed using large excavators equipped with wide tracks and smooth buckets. The exposed subgrade should not be compacted if wet, as compaction of a wet subgrade may result in further disturbance of the native soils.

After site mowing and grubbing has been completed and the surficial plant material has been exported from the site, geosynthetic reinforcement should be placed across all areas to receive structural fill. The prepared subgrade should be protected from construction traffic, and surface water should be diverted around areas of prepared subgrade. A blanket of rock spalls should be used in construction access areas if wet conditions are prevalent. The thickness of this rock spall layer should be based on subgrade performance at the time of construction. We recommend NAUE Combigrid 30/30 Q1, or equivalent, for use as geosynthetic reinforcement to provide reinforcement, filtration, and separation between the imported fill and underlying, native soils and sediments on the site. Geosynthetic layers should be placed so that overlap of at least 2.0 feet is maintained between geogrid sections. We should observe placement of geosynthetic reinforcement to ensure proper installation. After installation of any composite geogrid buried by structural fill, excavation through the reinforcement layer should be avoided to prevent the surrounding reinforcement from becoming compromised. Careful consideration should be made during planning the construction sequence for utility or structure installation.

Temporary and Permanent Slopes

We do not anticipate major cuts as a result of grading associated with this project, however, 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 water or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since they are continuously at the job site, able to observe the soil and groundwater conditions encountered, and able to monitor the nature and condition of the cut slopes.

The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 2 Horizontal to 1 Vertical (2H:1V) or shored. This should be further evaluated at the time of construction. If significant groundwater seepage or surface water flow were encountered, we would expect that shoring or trench boxes be considered for the planned cuts. We are available to provide recommendations for shoring, if needed, as the project plans are developed. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut heights and inclinations conform to appropriate OSHA/WISHA regulations.

Foundations

We understand that the planned structure foundations will be placed atop the structural fill planned for this site. We recommend that the bottom of the foundations be at least 3.0 feet above the geogrid layer and be supported on structural fill compacted to at least 95 percent of the ASTM D1557 dry density. Conventional 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 placed on at least 3.0 feet of competent structural fill that is underlain by the recommended geogrid. 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 3.0 inches total and one inch differential between adjacent footings or across a distance of about 20 feet, based on our experience.

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 medium dense soils or compacted fill should be used as backfill against the front of the footing. We recommend the upper one foot of soil be neglected when calculating passive resistance.

Retaining Walls

We do not anticipate the need for retaining walls on this site; however, should any walls be utilized, they should be designed and constructed as outlined above and hereon. 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 assume a horizontal ground surface behind the wall for a distance of at least the subsurface height of the wall, not accounting 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 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 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). We understand that an export site likely consisting of glacial till and advance outwash soils has been selected as a source for structural fill import. We should be retained to evaluate all proposed structural fill material prior to placement, ideally at the location of export to prevent deleterious material from being transported to the site indiscriminately. In any case, the fill should be imported to the site and placed during extended periods of dry weather.

Fill Placement: Following subgrade preparation and placement of the recommended geogrid, 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 12 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 permanently 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 four grain-size analyses on selected soil samples obtained within the site in accordance with the 2014 Washington State Department of Ecology (WSDOE) Stormwater Management Manual for Western Washington. Grain-size analyses were performed on selected samples from Test Pit One at 5.0 feet, Three at 2.0 and 5.0 feet, Six at 2.0 feet, and Nine at 4.0 feet below the ground surface. The results of the sieve analyses are presented as Figures 8 through 12. Based on the laboratory analysis, the soils encountered in our explorations within the proposed infiltration area meet the classification of sand in the USDA Textural Triangle.

An equation provided in Section 3.3.6.3 of the 2014 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 in the range of 13.35 to 25.74 inches per hour were calculated for the near-surface weathered glacial soils. Grain-size analyses on deeper, unweathered recessional outwash soils returned infiltration rates in the range of 27.14 to 93.27 inches per hour. We also referenced Table 3.3.1 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 0.90, 0.40, and 0.90 were utilized in this equation for CF_v, CF_t, CF_m, respectively. Using these correction factors, we applied it to the most conservative rate obtained from the grain-size analysis calculations for each soil, which are 13.35 inches per hour and 27.14 inches per hour for weathered and unweathered outwash soils, respectively. We calculated a long-term design infiltration rate of 4.33 inches per hour and 8.79 inches per hour for the weathered and unweathered native soils, respectively. We recommend that any infiltration systems be extended down through any unsuitable topsoils and founded within the native, granular outwash soils. Based on our explorations, the native, weathered outwash soils should be encountered between one and three feet below the existing ground surface. Unweathered outwash soils should be encountered at depths three feet and greater below the existing ground surface. We should be retained during construction to evaluate the soils exposed in the infiltration systems to verify that the soils are appropriate for infiltration.

The stormwater manual recommends a minimum three-foot separation between the base of an infiltration system and any underlying bedrock, impermeable horizon, or groundwater. Groundwater was encountered in each of our explorations at depths as shallow as 2.8 feet below the existing surface. It is likely that groundwater will greatly impact the design and performance of infiltration systems on this site. If infiltration systems are proposed, mounding analyses should be completed to verify appropriate sizing.

Shallow Infiltration System Design

Due to the potential for relatively shallow seasonal high groundwater within the site, we recommend that be considered shallow infiltration systems such as rain gardens or bio-retention swales to maintain adequate separation from the seasonal high groundwater. We recommend that the rain gardens and bio-retention swales be designed and sized in accordance with the 2014 WSDOE Stormwater Management Manual for Western Washington utilizing the above recommended long-term design infiltration rate. The base of the shallow infiltration systems should be extended through the upper silty soils and down to expose the granular outwash soils. Based on our explorations, these granular outwash soils should typically be encountered from 1.0 to 3.0 feet below the existing ground surface. Due to the overall depth of the granular outwash soils, the base of the shallow infiltration system may be higher than the granular outwash soils at depth. If this is the case, we recommend that the base of the shallow infiltration system be overexcavated down to expose the granular outwash soils and the overexcavation then backfilled with pea gravel up to the base of the infiltration system. Amended soils could then be placed over the pea gravel. We also recommend that an appropriate overflow system be incorporated into the design of the shallow infiltration systems if feasible. The inside and outside slopes of the shallow infiltration systems should be no steeper than 3H:1V. We should review final infiltration system design and monitor the system installation.

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 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 appropriate discharge system. The overflow water should be directed to discharge into an appropriate location.

Subsurface Drainage: If groundwater is encountered during construction, we recommend that the contractor slope the bottom of the excavation and collect the water into ditches and small sump pits where the water can be pumped out and routed into a permanent storm drain. Excavations below the groundwater table may require more elaborate dewatering systems. This should be determined during final design.

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.

CONSTRUCTION MONITORING

We should be retained to provide construction monitoring services during the earthwork phase of the project to evaluate subgrade conditions, geosynthetic reinforcement installation, temporary cut conditions, fill compaction, and drainage system installation.

USE OF THIS REPORT

NGA has prepared this preliminary report for Mr. Michael Neagle 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.

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.

Carston Curd

Carston T. Curd, GIT
Staff Geologist II

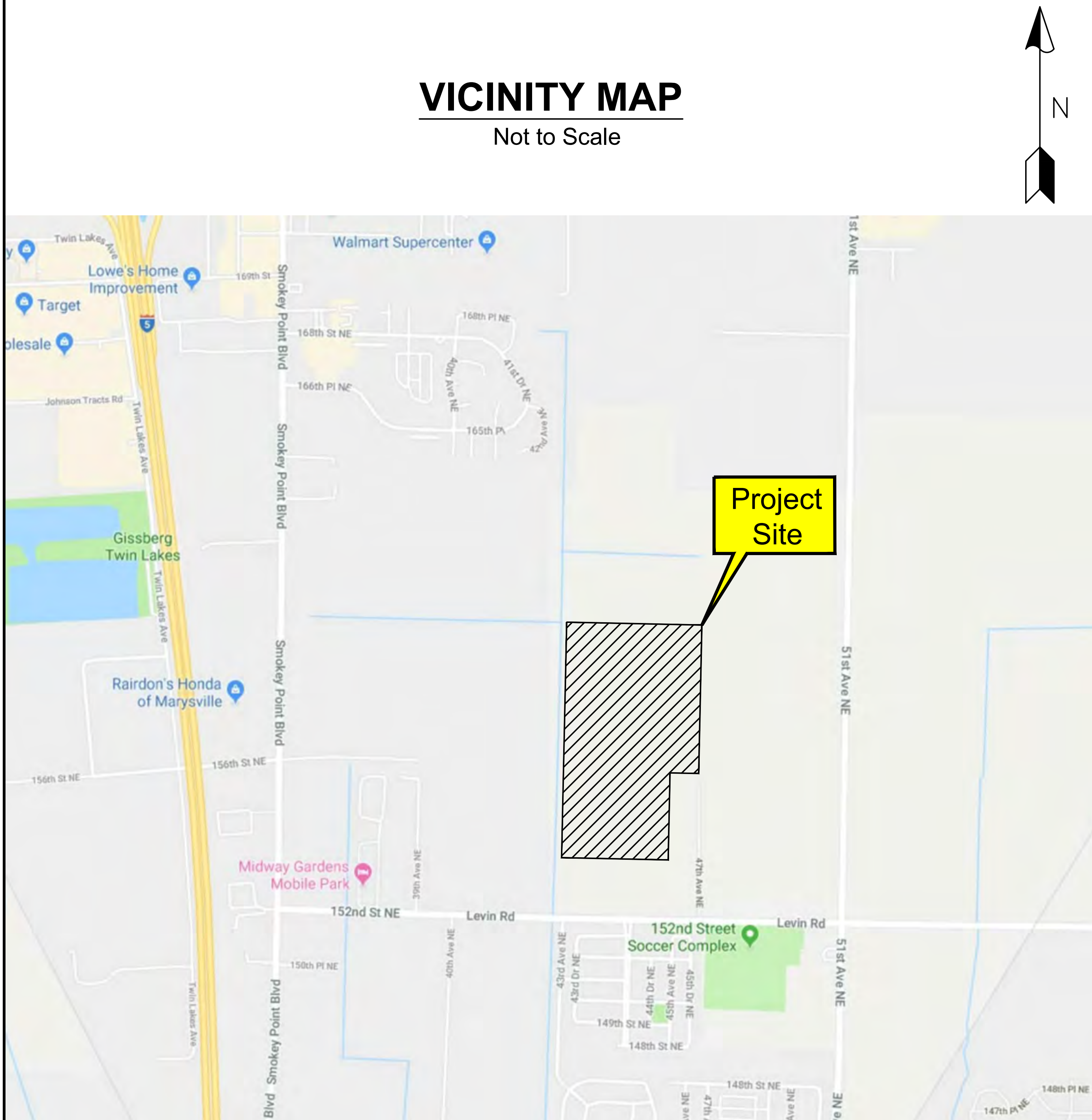
Khaled M. Shawish, PE
Principal

CTC:KMS:dy

Twelve Figures Attached

VICINITY MAP

Not to Scale



Marysville, WA

Project Number	Marysville Industrial Property Development Vicinity Map	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
1063718		Geotechnical Engineers & Geologists	1	1/7/19	Original	DPN	CTC
Figure 1		Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 · www.nelsongeotech.com · East Wenatchee Office 5526 Industry Lane, #2 East Wenatchee, WA 98802 (509) 665-7696 / Fax: 665-7692					

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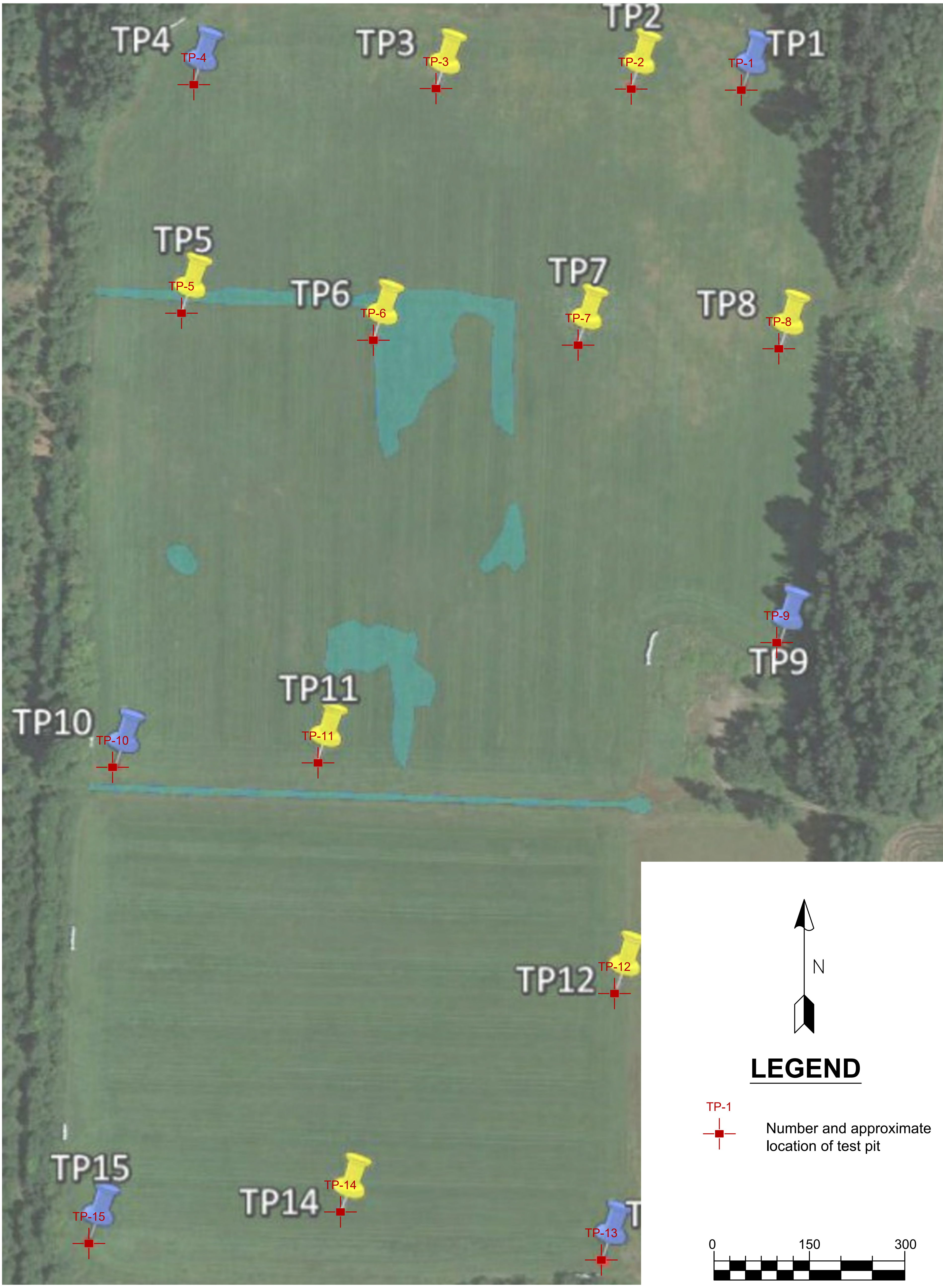

Reference: Site Plan based on field measurements, observations, and aerial parcel map review.

Project Number	Marysville Industrial Property Development Site Plan	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
1063718		Geotechnical Engineers & Geologists	1	1/7/19	Original	DPN	CTC
Figure 2							

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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 1063718	Marysville Industrial Property Development Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 3			1	1/7/19	Original	DPN	CTC

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LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 1.4		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.4 – 2.5	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH TRACE ORGANIC DEBRIS AND IRON OXIDATION STAINING (LOOSE-MEDIUM DENSE, WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY FINE TO COARSE SAND WITH SILT (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 3.0 FEET MODERATE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 2.8 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 2.5 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT TWO		
0.0 – 1.0		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.0 – 2.8	SM	BROWN SILTY FINE TO MEDIUM SAND (LOOSE-MEDIUM DENSE, WET) (**WEATHERED OUTWASH**)
2.8 – 6.0	SP-SM	GRAY FINE TO COARSE SAND WITH GRAVEL AND SILT (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 3.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 2.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT THREE		
0.0 – 0.8		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
0.8 – 3.0	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
3.0 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH TRACE FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT FOUR		
0.0 – 0.9		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
0.9 – 2.1	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.1 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH TRACE FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018

LOG OF EXPLORATION

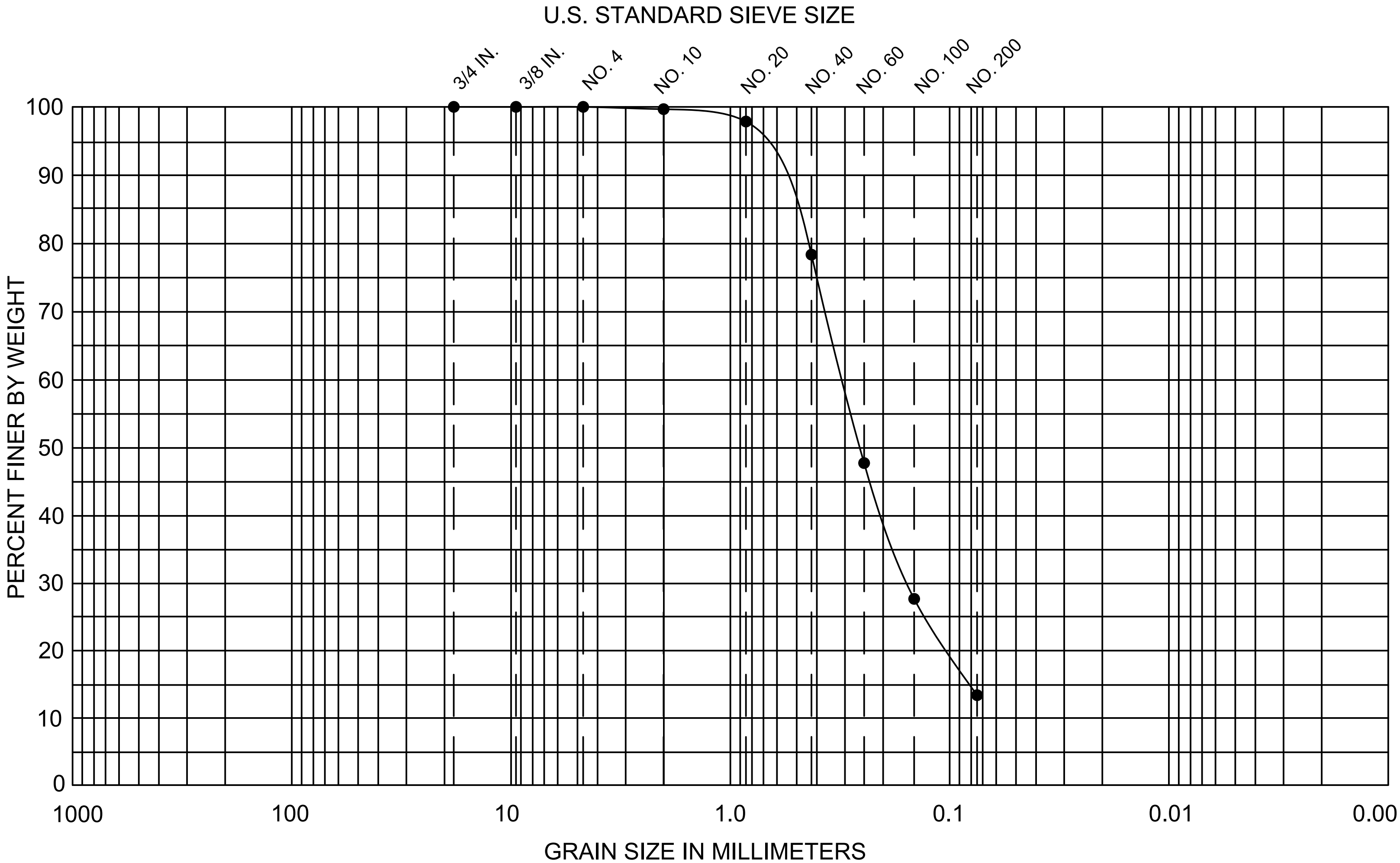
DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 1.2		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.2 – 4.0	SM	TAN-GRAY TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, WET) (**WEATHERED OUTWASH**)
4.0 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH GRAVEL (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 4.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MINOR TEST PIT CAVING WAS ENCOUNTERED BELOW 4.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT SIX		
0.0 – 1.1		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, WET) **(TOPSOIL)**
1.1 – 2.0	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.0 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH TRACE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.5 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 4.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT SEVEN		
0.0 – 1.5		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.5 – 3.5	SM	BROWN SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
3.5 – 6.0	SP	GRAY FINE TO COARSE SAND WITH FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 5.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 4.5 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT EIGHT		
0.0 – 1.4		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.4 – 2.5	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE-MEDIUM DENSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY-BROWN SILTY FINE TO COARSE SAND WITH TRACE FINE GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 3.0 AND 5.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT NINE		
0.0 – 1.2		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.2 – 2.5	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY FINE TO COARSE SAND WITH SILT AND GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 4.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT TEN		
0.0 – 0.8		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
0.8 – 2.8	SM	TAN-GRAY TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.8 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND WITH GRAVEL (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		NO SAMPLES WERE COLLECTED SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MINOR TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT ELEVEN		
0.0 – 1.0		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.0 – 2.6	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.6 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 2.6 AND 4.0 FEET SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT TWELVE		
0.0 – 1.0		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.0 – 2.5	SM	TAN-GRAY TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		NO SAMPLES WERE COLLECTED SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TO SEVERE TEST PIT CAVING WAS ENCOUNTERED BELOW 4.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT THIRTEEN		
0.0 – 1.3		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.3 – 2.3	SM	TAN TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.3 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE-MEDIUM DENSE, WET) (**RECESSIONAL OUTWASH**)
		SAMPLES WERE COLLECTED AT 5.0 FEET MODERATE TO SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.5 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT FOURTEEN		
0.0 – 1.1		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.1 – 2.0	SM	TAN-GRAY SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.0 – 3.3	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE, WET) (**RECESSIONAL OUTWASH**)
3.3 – 6.0	SP-SM	BROWN SILTY COARSE SAND WITH GRAVEL (LOOSE-MEDIUM DENSE, WET)
		NO SAMPLES WERE COLLECTED MODERATE TO SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018
TEST PIT FIFTEEN		
0.0 – 1.1		DARK BROWN ORGANIC PARTICULATE AND SILTY FINE TO COARSE SAND (VERY LOOSE, MOIST) **(TOPSOIL)**
1.1 – 2.5	SM	BROWN TO ORANGE SILTY FINE TO MEDIUM SAND WITH IRON OXIDATION STAINING (LOOSE, MOIST-WET) (**WEATHERED OUTWASH**)
2.5 – 6.0	SP-SM	GRAY SILTY FINE TO COARSE SAND (LOOSE, WET) (**RECESSIONAL OUTWASH**)
		NO SAMPLES WERE COLLECTED MODERATE TO SEVERE GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 3.0 FEET MODERATE TEST PIT CAVING WAS ENCOUNTERED BELOW 3.0 FEET TEST PIT WAS COMPLETED AT 6.0 FEET ON 12/21/2018



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	TP-1	5.0 feet	Fine to medium sand with silt	Gravel = 0% Sand = 87% Silt/Clay = 13%

Project Number 1063718

Figure 8

Marysville Industrial Property Development
Sieve Analysis

NGA
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No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC

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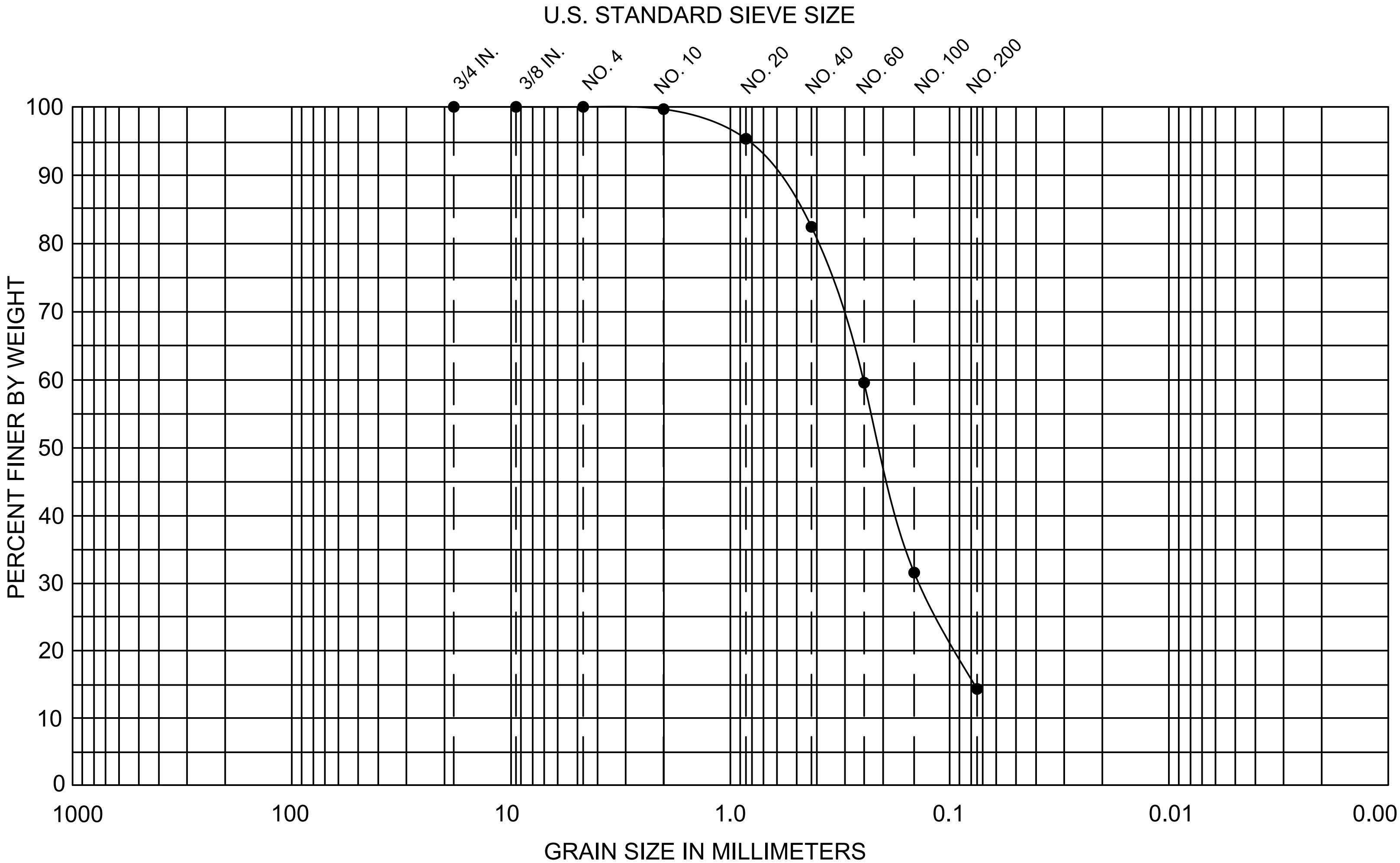
Project Number 1063718

Figure 9

Marysville Industrial Property Development
Sieve Analysis



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	TP-3	2.0 feet	Fine to medium sand with silt	Gravel = 0% Sand = 86% Silt/Clay = 14%

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No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC

Project Number 1063718

Figure 10

Marysville Industrial Property Development
Sieve Analysis

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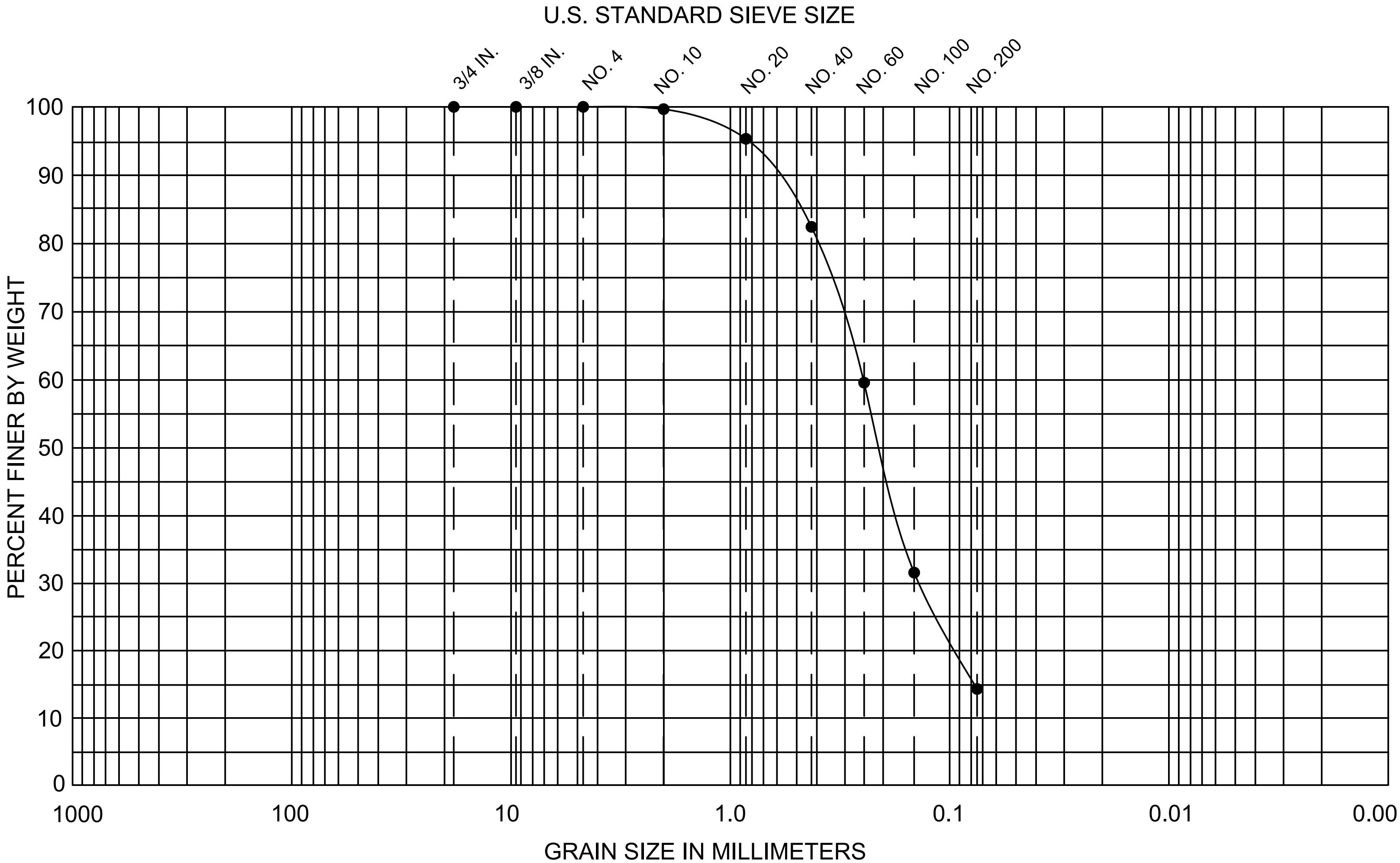
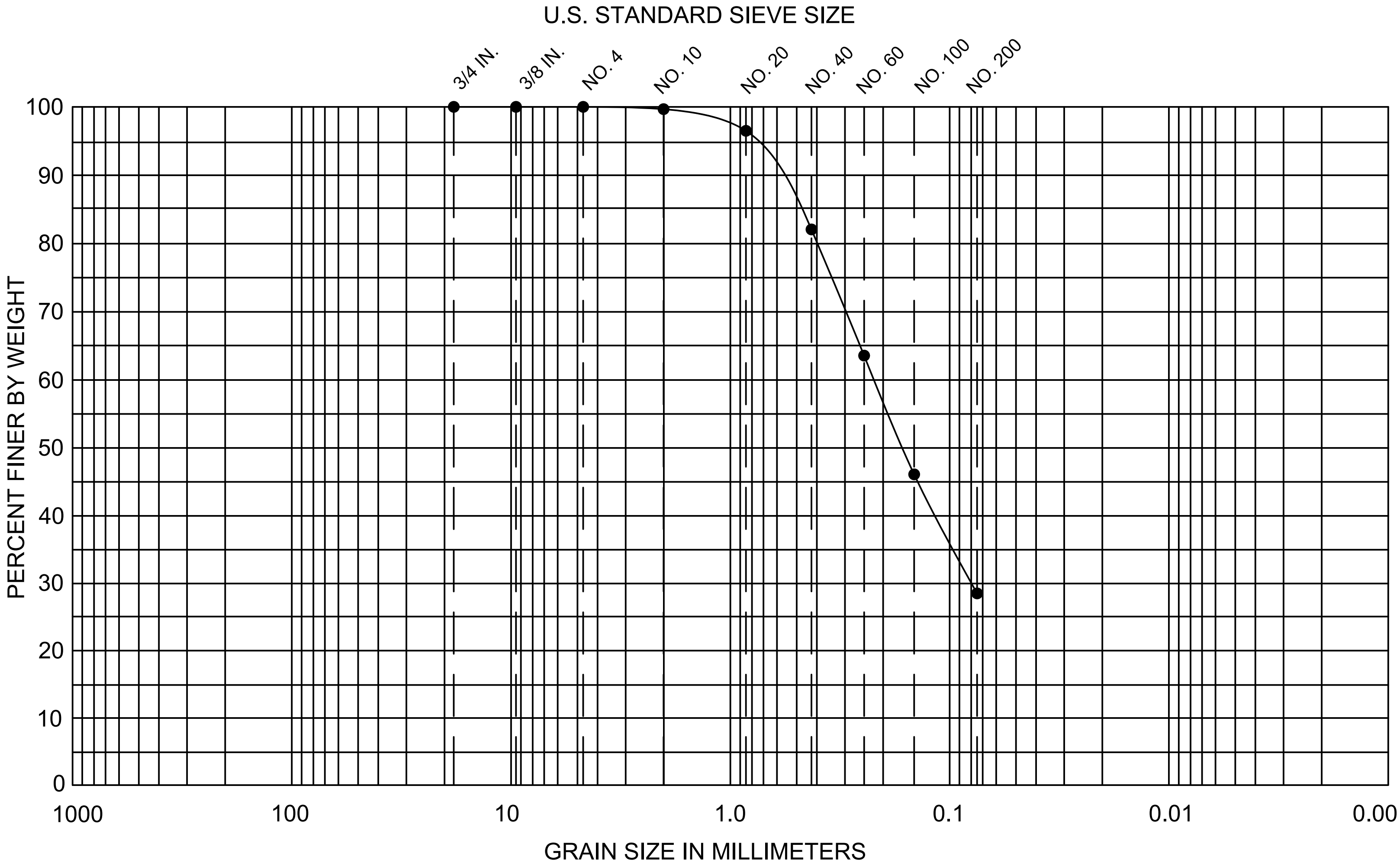
No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP	TP-3	5.0 feet	Fine to medium sand with trace gravel and silt	Gravel = 6% Sand = 90% Silt/Clay = 4%



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SM	TP-6	2.0 feet	Silty, fine to medium sand	Gravel = 0% Sand = 72% Silt/Clay = 28%

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Figure 11

Marysville Industrial
Property Development
Sieve Analysis

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No.	Date	Revision	By	CK
1	1/24/19	Original	DPN	CTC

Project Number
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Figure 12

Marysville Industrial Property Development
Sieve Analysis

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No.	Date	Revision	By	CK
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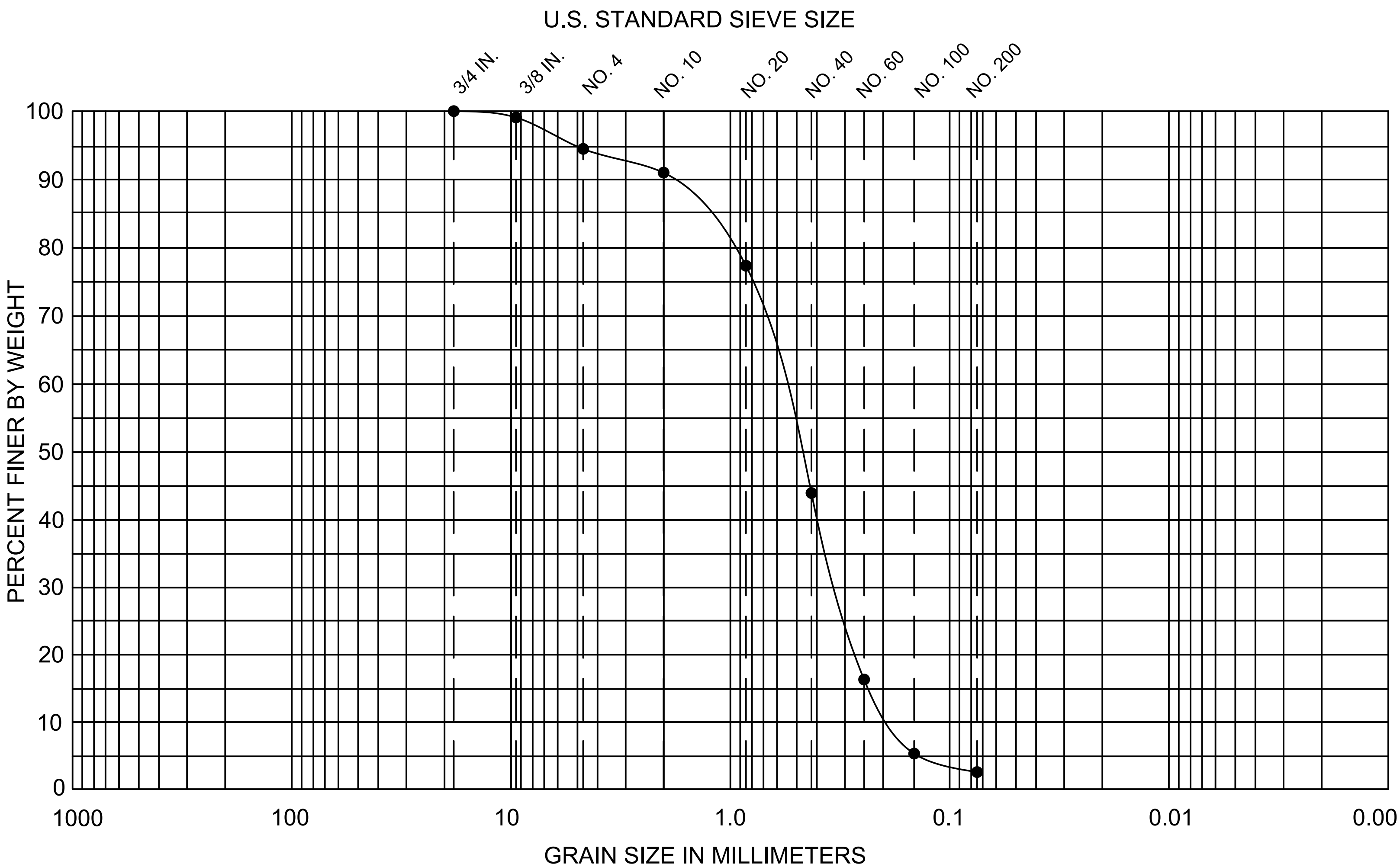

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP	TP-9	4.0 feet	Fine to medium sand with trace gravel and silt	Gravel = 5% Sand = 92% Silt/Clay = 3%

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