Northern, Inc.
Consulting Engineers Environmental Scientists Geologists
Construction Materials Testing Geophysical Services

GEOTECHNICAL SITE INVESTIGATION REPORT

PROPOSED ROCK CREEK COVE DEVELOPMENT
PARCEL # 02070100130200, 02070100130300 & 02070100130400
ROCK CREEK DRIVE, STEVENSON, WASHINGTON

GNN PROJECT NO. 219-1183

JANUARY 2020

Prepared for

FDM DEVELOPMENT INC.
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Prepared by

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GN Northern, Inc.

At GN Northern our mission is to serve our clients in the most efficient, cost effective way using the best resources and tools available while maintaining professionalism on every level. Our philosophy is to satisfy our clients through hard work, dedication and extraordinary efforts from all of our valued employees working as an extension of the design and construction team.

GN Northern, Inc.

January 13, 2020

FDM Development Inc.
5101 NE 82nd Ave, Suite 200
Vancouver, WA 98662

Attn: Zachary Pyle, PE, Development Manager

CC: F. Dean Maldonado, Principal

Subject: **Geotechnical Site Investigation Report**
Proposed Rock Creek Cove Development
Parcel # 02070100130200, 02070100130300 & 02070100130400
Rock Creek Drive, Stevenson, Washington

GNN Project No. 219-1183

Gentlemen,

As requested, GN Northern (GNN) has completed a geotechnical site investigation for the proposed Rock Creek Cove vacation homes project to be constructed at the vacant site located on Rock Creek Drive, east of the intersection with Attwell Road, in the City of Stevenson, Washington.

Based on the findings of our subsurface study, we conclude that the site is suitable for the proposed construction provided that our geotechnical recommendations presented in this report are followed during the design and construction phases of the project.

This report describes in detail the results of our investigation, summarizes our findings and presents our recommendations concerning earthwork and the design and construction of foundation for the proposed project. It is important that GN Northern provide consultation during the design phase as well as field compaction testing and geotechnical monitoring services during the earthwork phase to ensure implementation of the geotechnical recommendations.

If you have any questions regarding this report, please contact us at 509-248-9798 or 541-387-3387.

Respectfully submitted,

GN Northern, Inc.

Karl A. Harmon

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Geotechnical Engineer

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1.0 PURPOSE AND SCOPE OF SERVICES

This report has been prepared for the proposed Rock Creek Cove vacation homes project to be constructed at the vacant site located on Rock Creek Drive, east of the intersection with Attwell Road, in the City of Stevenson, Washington; site location is shown on the *Vicinity Map* (Figure 1, Appendix I). Our investigation was conducted to collect information regarding subsurface conditions and present recommendations for suitability of the subsurface materials to support the proposed building structures and allowable bearing capacity for the proposed construction.

GN Northern, Inc. has prepared this report for use by the client and their design consultants in the design of the proposed development. Do not use or rely upon this report for other locations or purposes without the written consent of GN Northern, Inc.

Our study was conducted in general accordance with our *Proposal for Geotechnical Engineering Services* dated October 29, 2019. Notice to proceed was provided in the form of a signed/authorized copy of our proposal via email on November 19, 2019.

A conceptual site plan (*Concept D*, prepared by FDM Development, dated 10/28/2019), along with a topographic survey of the project site (Lots 2, 3, and 4 of Rock Creek Cove, prepared by S&F Land Services, dated 12/11/2019), were provided by Mr. Pyle via email on December 17, 2019. Field exploration, consisting of twelve (12) test-pits and one (1) infiltration test, was completed on December 23, 2019. Locations of the exploratory test-pits and infiltration test are shown on the *Site Exploration Map* (Figure 2, Appendix I), and detailed test-pit logs are presented in Appendix II.

This report has been prepared to summarize the data obtained during this study and to present our recommendations based on the proposed construction and the subsurface conditions encountered at the site. Results of the field exploration were analyzed to develop recommendations for site development, earthwork, pavements, and foundation bearing capacity. Design parameters and a discussion of the geotechnical engineering considerations related to construction are included in this report.

2.0 PROPOSED CONSTRUCTION

Based on the preliminary information presented on the conceptual site plan and communication with your office, we understand that the proposed development will likely include approximately 15 to 25 structures. The various vacation rental structures are anticipated to consist of 6 to 8 single-room studio units along with 8 to 16 multi-story 3-bedroom units. Based on the current site layout, the studio units are planned across the southern finger, while the multi-story units are planned across the northern and western portions of the site. Proposed development will also include a 3-story central building with upstairs suite, central floor reception area, and lower floor kitchen and bar. Site development will also include associated infrastructure elements consisting of underground utilities, stormwater facilities, parking areas, and drive lanes. While the current site plan calls for a proposed wedding chapel/shelter on the eastern finger, we understand that development across this portion of the site may not be permitted.

Structural loading information was not available at the time of this report. Based on our experience with similar projects, we expect maximum wall loads to be on the order of 2,500 plf and maximum column loads to be less than 80 kips. It shall be noted that assumed loading is based on limited preliminary information provided at the time of this report. If loading conditions differ from those described herein, GNN should be given an opportunity to perform re-analysis. Settlement tolerances for structures are assumed to be limited to 1 inch, with differential settlement limited to ½ inch.

3.0 FIELD EXPLORATION & LABORATORY TESTING

The field exploration was completed on December 23, 2019. A local public utility clearance was obtained prior to the field exploration. Twelve (12) exploratory test-pits were completed at various locations within the footprint of the proposed development. Test-pits were excavated by Riley Materials using a Link-Belt 145x4 excavator to depths of approximately 8 to 14.5 feet below existing ground surface (BGS) and logged by a GNN field geologist/engineer. Additionally, an infiltration test was performed on the north side of the entrance driveway. Upon completion, all excavations were loosely backfilled with excavation spoils. Test-hole locations are shown on *Site Exploration Map* (Figure 2)

The soils observed during our field exploration were classified according to the Unified Soil Classification System (USCS), utilizing the field classification procedures as outlined in ASTM D2488. A copy of the USCS Classification Chart is included in Appendix II. Photographs of the site and exploration are presented in Appendix IV. Depths referred to in this report are relative to the existing ground surface elevation at the time of our investigation. The surface and subsurface conditions described in this report are as observed at the time of our field investigation.

Representative samples of the subsurface soils obtained from the field exploration were selected for testing to determine the index properties of the soils in general accordance with ASTM procedures. The following laboratory tests were performed:

Table 1: Laboratory Tests Performed

Test	To determine
Particle Size Distribution (ASTM D6913)	Soil classification based on proportion of sand, silt, and clay-sized particles
Natural Moisture Content (ASTM D2216)	Soil moisture content indicative of in-situ condition at the time samples were taken

Results of the laboratory test are included on the test-pit logs and are also presented in graphic form in Appendix III attached to the end of the report.

4.0 SITE CONDITIONS

The project site is located east of the intersection of Rock Creek Drive and Attwell Road, approximately ½-mile north of State Highway 14, in the City of Stevenson, Washington. The 6.4-acre project site is currently comprised of three separate parcels identified by the Skamania County Assessor as Parcel Numbers: 020701001302000 (Lot 2), 020701001303000 (Lot 3), and 020701001304000 (Lot 4) located within the SW ¼ of the NW ¼ of Section 1, Township 2 North and Range 7 East, Willamette Meridian.

The subject site is generally characterized as an irregular shaped peninsula with several fingers extending east from Rock Creek Drive into Rock Cove. The majority of the upper surface of the site is relatively flat, while the irregular shaped peninsula fingers typically include steep slopes along the perimeter down to the shoreline. Surface conditions across the site include a variety of gravel covered and paved areas (asphalt and concrete), as well as areas with a dense growth of mature trees and vegetation, with selected areas across slope faces that include a veneer of angular rock (apparent rip-rap). Recently placed stockpiles of apparent landscape clippings are present across an area located south of the existing entrance driveway.

Surface topography across the subject site has been historically altered by previous grading activity related to the preexisting use. The upper historically graded portions of the site are relatively flat at elevations ranging from approximately 95' to 101' across a majority of the site. Site grades step down towards that eastern finger with surface elevations ranging from approximately 87' to 90'. The surrounding edges of the various peninsula fingers typically include relatively steep slopes, with gradients as steep as 1H:1V, from the upper flat portions descending down to the shoreline.

The history of past use and development of the property was not investigated as part of our scope of services for this geotechnical site investigation. Based on our cursory review of available historic aerial photos (Appendix V) and topographic maps, along with a previously completed phase II environmental site assessment (Maul Foster Alongi, 2017), the site is known to have been historically developed with an industrial lumber mill facility. Scattered buried remnants related to the noted previous development and operations at the site including concrete foundation and slabs, miscellaneous utilities, trash and debris should be anticipated. Additionally, the eastern finger extending into Rock Cove appears to have been created by historic filling of the area between the main portion of the site and a preexisting island toward the eastern tip. The 1935 aerial photograph taken prior to historic site development of the site shows the site vicinity at the time when the Rock Cove had not been flooded by construction of the Bonneville Dam.

5.0 SITE & REGIONAL GEOLOGY

The City of Stevenson and Skamania County are located in the South Cascades physiographic province that extends from the Columbia River to the south to Interstate 90 to the north, and is dominated by three massive stratovolcanoes. The current day volcanoes are the most recent installments of a 40-million-year-old volcanic complex called the Cascades Volcanic Arc. The bedrock geology of the western Columbia Gorge is dominated by Oligocene to early Miocene volcaniclastic rocks and minor interbedded lava flows of the ancestral Cascade Volcanic Arc. At many locations, the ancestral arc rocks are unconformably overlain by lava flows of the middle Miocene Columbia River Basalt Group, late Miocene to Pliocene fluvial deposits, or Quaternary olivine-phyric mafic lavas (Pierson et al., 2016).

The western part of the Columbia River Gorge is characterized by massive landslides on the Washington side, and the instability of these land masses is associated with abundant rainfall, high relief, composition and structure of the underlying rocks, tectonic uplift associated with the structural evolution of the Cascade Range and Yakima Fold Belt, and valley-side erosion by the incising Columbia River, which flows across the uplifting terrains (Pierson et al., 2016). The Cascade landslide complex is one such landslide feature that spans from the town of North Bonneville to the western portion of Stevenson. The Cascade landslide complex is subdivided into four individual landslides: the Carpenters Lake, Bonneville, and Red Bluffs landslides, as well as a reactivated part of the Red Bluffs landslide body known as the Crescent Lake landslide. Immediately east of the Cascade landslide complex is the newly recognized Stevenson landslide which is occupied by the City of Stevenson.

The project site is located near the eastern toe of the Red Bluffs landslide, approximately 1-mile east of the reactivated Crescent Lake landslide. The head scarp of the Red Bluffs landslide is located approximately 3½ miles northwest of the site. Surface geology at the site is mapped as Quaternary landslide deposits [Qls] of the Red Bluffs landslide (mass wasting deposits), consisting of poorly sorted blocks, boulders, gravels, and fines sediments produced by the gravitational failure and rotational-translational slide of bedrock and/or unconsolidated sediments above the bedrock (Korosec, 1987).

6.0 SUBSURFACE CONDITIONS

Based on the findings of our field exploration, subsurface soils at the project site include a variably-thick layer of artificial fill soils likely associated with historic site development, atop the native silty gravel with sand stratum (mass wasting deposits). The undocumented artificial fill soils were noted to depths of approximately 3 to 8 feet across the upper portion of the site. Test-pit TP-9 excavated on the lower eastern finger encountered fill to the full depth of exploration (~8 feet) that is believed to represent historic fill placed to create new land. Fill soils were generally classified as silty gravel with sand and variable amounts of cobbles and boulders, and with some areas also including organics, wood debris and miscellaneous trash. The fill soils at the site are likely to be related to the previous historic development at the site. The apparent native underlying soils were classified as Silty Gravel with Sand (GM) and included varying amounts of cobbles and boulders. The native soil stratum typically appeared medium dense. Due to similar soil condition between the upper fills and the underlaying native stratum, the fill/native transition was typically ambiguous and therefore not clearly discernable within the test-pits. Test-pit logs in Appendix II show detailed descriptions and stratification of the soils encountered.

6.1 NRCS Soil Survey

Although altered at the surface, the soil survey map of the site prepared by the Natural Resources Conservation Service (NRCS) identifies the site soils as *Arents* with typical profile described as *gravelly sandy loam* grading to *extremely gravelly sandy loam*. Based on the NRCS map (Appendix VII), these units generally consist of *well drained* materials.

6.2 Groundwater

Groundwater was encountered within two of the exploratory test-pits at depths ranging from approximately 12 to 14 feet BGS at the time of our exploration in late December. Approximate correlating groundwater elevations ranged from approximately 83' in TP-3 in the western portion, down to 78' in TP-8 near the eastern portion. A review of the Washington Department of Ecology’s online water well log database revealed a lack of nearby water wells in the site vicinity. Water levels within the adjacent Rock Cove portion of the Columbia River, controlled by the down-river Bonneville Dam, are typically noted at an elevation approximately 20 to 25 feet below the upper leveled-off site elevation. Therefore, we believe groundwater at the site is not directly affected by pool elevations in the Columbia River, and is likely controlled by the complex hydrogeological conditions of the up-gradient mass-wasting landslide deposits, as well as regional precipitation and snowmelt. Groundwater levels will fluctuate with irrigation, precipitation, drainage, and regional pumping from wells.

7.0 SOIL INFILTRATION TESTING

A single infiltration test was performed on the north side of the existing entrance drive at a depth of approximately 5.5 feet BGS using a small-scale Pilot Infiltration Test (PIT). To the degree possible, care was exercised during excavation to attempt to maintain relatively uniform side walls, and the resulting size and geometry of the finished test-pit was carefully recorded in the field. Water was introduced into the test-pit using a garden hose connected to a nearby fire hydrant. The water flow into the test-pit was continued until the soils with the test-pit were saturated and a constant flow rate was established. The stabilized inflow rate was measured and recorded, and the resulting un-factored infiltration rates are presented in the table below:

Table 2: Infiltration Test Results

Test ID	Approximate Location (GPS Coordinates)	Soil Tested	Field Infiltration Rate
P-1	45°41'20.69"N, 121°53'56.06"W	Silty Gravel	**4 inches/hour**

The infiltration rate presented herein represents the un-factored field soil infiltration rate. An appropriate factor of safety should be applied to the field infiltration rate to determine long-term design infiltration rate. Determination of safety factors for long-term design infiltration should consider the following: pretreatment, potential for bio-fouling, system maintainability, horizontal and vertical variability of soils, and type of infiltration testing. Typical factors of safety for these soils generally range from 2 to 3. If stormwater management facilities are selected at other locations, additional site-specific infiltration testing shall be performed.

8.0 GEOLOGIC HAZARDS

Potential geologic hazards that may affect the proposed development include: [i] landslides & slope instability, [ii] seismic hazards (ground shaking, surface fault rupture, soil liquefaction, and other secondary earthquake-related hazards), and [iii] flooding & erosion. The perimeter/shoreline edges of the subject property are generally all mapped by the City of Stevenson's Critical Areas & Geologic Hazards Map as 'Potentially Unstable Slope' which refers to an area with slopes of 25% or greater per Stevenson Municipal Code (SMC), Chapter 18.13, Section 18.13.090, Critical Area - Geologically Hazardous Areas. A discussion follows on the specific hazards to this site:

8.1 Landslides

As discussed above in Section 5.0, the project site lies within the Cascade landslide complex that is subdivided into four individual landslides (Carpenters Lake, Bonneville, Red Bluffs, & Crescent Lake landslide). The Bonneville landslide has been dated to have occurred from 1416-1452 A.D. by a combination of dating methods. The Red Bluffs landslide has crosscutting morphologic features suggesting a younger age than that of the Bonneville landslide, with an age range of 1760-1770 A.D. The Crescent Lake landslide has reactivated within the last few decades and currently is moving downslope at an average rate of 11–18 cm/year and possibly as fast as 25 cm/year (Pierson et al., 2016). Results of another recent study (Hu et al., 2015) showed that the central upper part of the Crescent Lake landslide moved a total of 700 mm downslope during a 4-year observation period from 2007 to 2011, and that the movement was seasonal and showed a strong correlation with winter precipitation. In contrast to the Crescent Lake landslide, coherent parts of Red Bluffs, Bonneville and Stevenson landslides were observed to remain stable during the observation period.

Although considered a recent landslide (< 1,000 years old), the Red Bluffs landslide is not considered an active landslide (movement in last 20 years). Based on Table 18.13.090-1, Landslide Hazard Classification, of the Stevenson Municipal Code (SMC), the landslide hazard for the site classifies as 'Moderate Hazard'.

8.2 Regional Faulting & Surface Fault Rupture

The nearest regional faulting with Quaternary displacement (< 130,000 years) consists of the Faults near The Dalles located approximately 12 miles east of the project site (Czajkowski, 2014). Published slip rates for these faults are listed at less than 0.2 mm/year. For the purposes of this report, an active fault is defined as a fault that has had displacement within the Holocene epoch or last 11,700 years. Due to the lack of any known active fault traces in the immediate site vicinity, surface fault rupture is unlikely to occur at the subject property. While future fault rupture could occur at other locations, rupture would most likely occur along previously established fault traces.

8.3 Earthquakes & Seismic Conditions

Earthquakes caused by movements along crustal faults, generally in the upper 10 to 15 miles, occur on the crust of the North America tectonic plate when built-up stresses near the surface are released. The two largest crustal earthquakes felt in the state of Washington included the 1872, M 6.8 quake near Lake Chelan and the 1936, M 6.0 Walla Walla earthquake. Noteworthy to the City of Stevenson, the Mount Saint Helens Seismic Zone is located approximately 30 miles towards the north-northwest. The following list provides information gathered from the online USGS database regarding historic earthquakes (≥4.0 M) within the past 50 years for epicenters within 100 kilometers of project site, sorted by magnitude (largest to smallest):

Table 3: Earthquakes within 100-kilometers of project site

Date(s) of Event	Magnitude(s)	Nearby Faults / Seismic Zone	Approx. Distance from Site (miles)
March to May, 1980	4.0 - 5.7	Mt. Saint Helens Seismic Zone	33 – 47
March 25, 1993	5.6	Mt. Angel Fault Zone	57
February 14, 1981	5.2	Mt. Saint Helens Seismic Zone	48

May 13, 1981	4.5	Mt. Saint Helens Seismic Zone	50
June 29, 2002	4.5	Faults near The Dalles	26
March 1, 1982	4.4	Mt. Saint Helens Seismic Zone	48
February 14, 2011	4.3	Mt. Saint Helens Seismic Zone	44
July 14, 2008	4.2	*Unknown*	60
December 13, 1974	4.1	Faults near The Dalles	33
February 2, 1981	4.0	Toppenish Ridge Fault Zone	59

Based on seismic scenarios published by the Washington State Department of Natural Resources (DNR), M 7.0 Mount Saint Helens and M 7.1 Mill Creek earthquake events would result in a shaking intensity of ‘V’ (moderate shaking) on the Modified Mercalli Intensity (MMI) scale. We further used the USGS deaggregation tool which provides the relative contributions of hazard for each seismic source based on Probabilistic Seismic Hazard Analysis (PSHA). Based on the deaggregation, it appears that about 23% of the contribution to the probabilistic hazard at the site comes from the Cascadia Subduction Zone, with the remaining contribution primarily from the shallower sources.

8.4 Soil Liquefaction

Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. Based on the published Liquefaction Susceptibility Map of of Skamania County, Washington (Palmer et al., 2004a), the site is mapped with a ‘low to moderate’ relative suceptibility for seismically-induced liquefaction to occur. A detailed assessment of the liquefaction potential at the site, including liquefaction-induced settlement and the effects of lateral spreading, is beyond the scope of this investigation.

8.5 Secondary Seismic Hazards

Additional secondary seismic hazards related to ground shaking include ground subsidence, tsunamis, and seiches. The site is far inland, so the hazard from tsunamis is non-existent. The potential hazard of seiches from a significant seismic event is relatively low for development on the upper portion of the project site that is elevated approximately 20 to 25 feet above Rock Cove.

8.6 Site Slopes

Surface topography across the subject site has been historically altered by previous grading activity related to the preexisting lumber mill facility. The upper historically graded portions of the site are relatively flat at elevations ranging from approximately 95' to 101'. The surrounding edges of the various peninsula fingers typically include relatively steep slopes, with gradients as great as 1H:1V, from the upper flat portions descending down to the shoreline. A field reconnaissance of the subject property was performed to observe site conditions and look for common geomorphic features of landslides as well as indications of possible signs demonstrating recent activity and instability of slide masses. While several areas across the site include a relatively dense cover of vegetation, no apparent indications of recent failures or significant slope instability were observed. Section 9.0 presents results of a preliminary slope stability analysis completed at the site and Section 12.0 provides recommendations for appropriate structure setbacks.

8.7 Flooding and Erosion

The subject property is mapped by Federal Emergency Management Agency (FEMA) as Zone 'C' which translates to areas of minimal flooding. Portions of the subject property are however situated in areas where sheet flow and erosion may occur. Soil erodibility is only one of several factors affecting the erosion susceptibility. Soil erosion by water also increases with the length and steepness of the site slopes due to the increased velocity of runoff and resulting greater degree of scour and sediment transport. The need for and design of erosion protection measures is within the purview of the design Civil Engineer. Appropriate erosion and sediment control plan(s) and a drainage plan shall be prepared by the project civil engineer with the final construction drawings. Erosion should be mitigated with appropriate BMPs consisting of proper drainage design including collecting and disposal (conveyance) of water to approved points of discharge in a non-erosive manner. Appropriate project design, construction, and maintenance will be necessary to mitigate the site erosion hazards.

9.0 SLOPE STABILITY ANALYSIS

A preliminary slope stability analysis was conducted for a critical slope section across the southern finger as shown on Figure 2. The analysis was conducted using a generalized geologic cross-section model developed from the existing site topography and data obtained from our subsurface exploration. An output of our slope stability analysis is attached in Appendix VI.

The slope stability analysis was conducted by a two-dimensional limit equilibrium stability analysis of selected trial failure surfaces using the computer program *SLIDE (Version 7)*. Potential circular-arc failure surfaces were evaluated using the Spencer method under static conditions. The computer program searched for critical potential failure surfaces with low computed factors of safety. The computed factor of safety (FS) against slope failure is simply the ratio of total resisting forces or moments (strength of the slope) to the total driving forces or moments for planar or circular failure surfaces respectively. A slope with a factor of safety of 1.0 is in equilibrium, indicating that the disturbing forces driving the slope down are equal to its strength to resist failure. Simply put slope-failure result when the strength of the slope is overcome by gravity.

The selection of unit weight and shear strength parameters for the various earth materials were based on judgment and data obtained during our field investigation, laboratory testing, review of previous studies, research and previous experience with similar materials in similar geotechnical and geologic settings. Engineering and geologic judgment must be applied to the estimated shear strength parameters in order to consider lateral and vertical variations in the subsurface conditions, such as degree of cementation, fracturing, planes of weakness, and gradational characteristics. The following geotechnical strength parameters were used in our stability calculations:

Table 4: Estimated Strength Parameters

Material	**Shear Strength Parameters**		**Unit Weight (pcf)**
	Friction Angle: ϕ	**Cohesion: c (psf)**	
Fill/Disturbed Soil	33	25	120
Native Silty Gravel w/ Sand	35	50	130 (moist) 138 (saturated)

GN Northern recommends that any existing or reconfigured slopes should meet or be designed and constructed to meet a minimum factor of safety of 1.5 for the static condition and 1.1 under seismic loading. Based on the results of our slope stability analysis, we conclude that the steep perimeter slopes do not meet minimum recommended safety factors. Consequently, the currently proposed layout with future structures sited at/over the edge of slopes is generally considered unfeasible, and remedial grading and/or other appropriate mitigation measures will be required to increase slope safety factors and provide adequate subgrade support for the proposed structures.

In lieu of appropriate remediation of the slope stability concerns, in order to provide sufficient vertical and lateral support for the proposed foundations without significant risk of detrimental settlement, appropriate increased setbacks/embedment for the new building foundations should be maintained. It should be understood however that while the proposed structures may not be at significant risk from slope instability, the existing slopes will remain at risk for some future failure if not appropriately remediated.

10.0 SEISMIC DESIGN PARAMETERS

Based on subsurface data obtained during or field exploration, along with our review of the published NEHRP Site Class Map of Skamania County, Washington (Palmer et al., 2004b), a site class 'D' as defined by 2015 International Building Code (IBC) is applicable. According to Mapped Spectral Acceleration obtained from the USGS Seismic Design Maps using the 2015 IBC, the following site-specific design values may be used:

Table 5: IBC Design Response Spectra Parameters

Seismic Design Parameter	Value (unit)
S_s	0.657 (g)
S_1	0.292 (g)
F_a	1.274 (unitless)
F_v	1.816 (unitless)
SM_s	0.837 (g)
SM_1	0.530 (g)
SD_s	0.558 (g)
SD_1	0.354 (g)

S_S = MCE spectral response acceleration at short periods
S_1 = MCE spectral response acceleration at 1-second period
F_a = Site coefficient for short periods
F_v = Site coefficient for 1-second period
SM_S = MCE spectral response acceleration at short periods as adjusted for site effects
SM_1 = MCE spectral response acceleration at 1-second period as adjusted for site effects
SD_S = Design spectral response acceleration at short periods
SD_1 = Design spectral response acceleration at 1-second period

It shall be noted that determination of an appropriate site class requires shear wave velocity, soil undrained shear strength, or standard penetration resistance (N-value) data in the upper 100 feet of the subsurface profile, which was beyond the scope of this investigation.

11.0 SUMMARY OF FINDINGS & CONCLUSIONS

Conditions imposed by the proposed development have been evaluated on the basis of assumed elevations and engineering characteristics of the subsurface materials encountered in the exploratory test-pits, and their anticipated behavior both during and after construction. The following is a summary of our findings, conclusions and professional opinions based on the data obtained from a review of selected technical literature and the site evaluation.

- Based on the findings of this geotechnical evaluation and our understanding of the proposed development, from a geotechnical perspective, it is our opinion that the site is suitable for the proposed development, provided the soil design parameters and site-specific recommendations in this report are followed in the design and construction of the project.
- Final design plans for the proposed development, including grading, drainage and finished elevations, were not provided at the time of this report**.** Once the plans are finalized, GNN **must** be provided an opportunity to review final design plans to provide revised recommendations if/as necessary.
- Site soils include a variably-thick layer of artificial fill soils believed to be related to historic site development, atop the native silty gravels with sand. The undocumented artificial fill soils, largely made-up of similar soils that were apparently derived from onsite and/or near sources, extend to depths ranging from 3 to 8 feet and include some areas with miscellaneous trash and debris. Our estimation of the depth of fill materials is based on selected, localized points of exploration, and cannot quantify the full extent of the onsite fill. Additional undocumented fill soils with trash/debris, buried within the subsurface profile, may extend to greater depths at isolated locations across the site.
- Groundwater was encountered within the two of our test-pits at depths ranging from approximately 12 to 14 feet BGS at the time of our exploration in late December. Approximate correlating groundwater elevations ranged from approximately 83' in TP-3 in the western portion, down to 78' in TP-8 near the eastern portion. We believe groundwater at the site is not directly affected by pool elevations in the Columbia River, and is likely controlled by the complex hydrogeological conditions of the up-gradient mass-wasting landslide deposits, as well as regional precipitation and snowmelt.

- The onsite silty gravel soils, screened and processed to be free of oversize rocks (>5 inches) and any deleterious materials including trash and debris, are generally suitable for reuse as engineered fill and utility trench backfill.
- The proposed building structures may be supported on conventional shallow foundations bearing on a layer of crushed rock atop the recompacted native subgrade in accordance with the recommendations of this report. However, due to presence of artificial fill soils across future building footprints, over-excavation of the existing fill soils to a competent native stratum and replacement with engineered fill will be required.
- Due to ecological constraints, it appears that remedial grading of the onsite slopes to improve long-term stability is not considered feasible. Therefore, deeper embedment of the building foundations will be required in order to meet the minimum setback requirements while ignoring the stability of the onsite slopes.
- Appropriate slope setbacks for future structures should be incorporated in the final planning and design of the project. Slopes setbacks shall adhere to IBC 2015 Section 1808.7 *Foundations on or Adjacent to Slopes*, as well as the recommendations of this report.
- Site grading shall incorporate the requirements of IBC 2015, Appendix J *Grading*.
- Upon completion, all test-pit excavations were loosely backfilled with excavation spoils. The contractor is responsible to locate the test-pits to re-excavate the loose soils and re-place as compacted engineered fill.
- The underlying geologic condition for seismic design is site class 'D'. The *minimum* seismic design should comply with the 2015 International Building Code (IBC) and ASCE 07-10, Minimum Design Loads for Buildings and Other Structures.
- The near-surface site soils are susceptible to wind and water erosion when exposed during grading operations. Preventative measures and appropriate BMPs to control runoff and reduce erosion should be incorporated into site grading plans.
- Based on our evaluation, the risk for liquefaction at the project site is considered low to moderate. A site-specific liquefaction analysis to assess the risk of soil liquefaction and liquefaction-induced settlement was beyond the scope of this geotechnical evaluation and would require additional exploration including a 50-foot deep boring with continuous penetration testing.

12.0 GEOTECHNICAL RECOMMENDATIONS

The following geotechnical recommendations are based on our current understanding of the proposed project as shown on the conceptual site plan (Concept D, prepared by FDM Development, dated 10/28/2019), and as described in Section 2.0 of this report. The report is prepared to comply with the 2015 International Building Code Section 1803, Geotechnical Investigations, and as required by Subsection 1803.2, Investigations Required. Please note that Soil Design Parameters and Recommendations presented in this report are predicated upon appropriate geotechnical monitoring and testing of the site preparation and foundation and building pad construction by a representative of GNN's Geotechnical-Engineer-of-Record (GER). Any deviation and nonconformity from this requirement may invalidate, partially or in whole, the following recommendations. We recommend that we be engaged to review grading and foundation plans in order to provide revised, augmented, and/or additional geotechnical recommendations as required.

12.1 Site Development – Grading

Site grading shall incorporate the requirements of IBC 2015 Appendix J. The project GER or a representative of the GER should observe site clearing, grading, and the bottoms of excavations before placing fills. Local variations in soil conditions may warrant increasing the depth of over-excavation and recompaction. Seasonal weather conditions may adversely affect grading operations. To improve compaction efforts and prevent potential pumping and unstable ground conditions, we suggest performing site grading during dryer periods of the year.

Soil conditions shall be evaluated by in-place density testing, visual evaluation, probing, and proof-rolling of the imported fill and re-compacted on-site soil as it is prepared to check for compliance with recommendations of this report. A moisture-density curve shall be established in accordance with the ASTM D1557 method for all onsite soils and imported fill materials used as structural fill.

12.2 Clearing and Grubbing

At the start of site grading, any vegetation, large roots, non-engineered/artificial fill, including trash and debris, and any abandoned underground utilities shall be removed from the proposed building and structural areas. The surface shall be stripped of all topsoil and/or organic growth (vegetation) that may exist within the proposed structural areas. The topsoil and organic rich soils shall either be stockpiled on-site separately for future use or be removed from the construction area. Depth of stripping can be minimized with real-time onsite observation of sufficient removals. Areas disturbed during clearing shall be properly backfilled and compacted as described below.

12.3 Suitability of the Onsite Soils as Engineered Fill

The onsite silty gravel with sand soils, screened and processed to be free of oversize rocks (>5 inches) and deleterious materials including trash and debris, are generally suitable for reuse as engineered fill and utility trench backfill. Suitable onsite soils shall be placed in maximum 8-inch lifts (loose) and compacted to at least 95% relative compaction (ASTM D1557) near its optimum moisture content. Compaction of these soils shall be performed within a range of ±2% of optimum moisture to achieve the proper degree of compaction.

12.4 Temporary Excavations

It shall be the responsibility of the contractor to maintain safe temporary slope configurations since the contractor is at the job site, able to observe the nature and conditions of the slopes and be able to monitor the subsurface conditions encountered. Unsupported vertical cuts deeper than 4 feet are not recommended if worker access is necessary. The cuts shall be adequately sloped, shored or supported to prevent injury to personnel from caving and sloughing. The contractor and subcontractors shall be aware of and familiar with applicable local, state and federal safety regulation including the current OSHA Excavation and Trench Safety Standards, and OSHA Health and Safety Standards for Excavations, 29 CFR Part 1929, or successor regulations.

According to chapter 296-155 of the Washington Administrative Code (WAC), it is our opinion that the soil encountered at the site is classified as Type C soils. We recommend that temporary, unsupported, open cut slopes shall be no steeper than 1.5 feet horizontal to 1.0 feet vertical (1.5H:1V) in Type C soils. No heavy equipment should be allowed near the top of temporary cut slopes unless the cut slopes are adequately braced. Final (permanent) fill slopes should be graded to an angle of 2H:1V or flatter. Where unstable soils are encountered, flatter slopes may be required.

12.5 Utility Excavation, Pipe Bedding and Trench Backfill

To provide suitable support and bedding for the pipe, we recommend the utilities be founded on suitable bedding material consisting of clean sand and/or sand & gravel mixture. To minimize trench subgrade disturbance during excavation, the excavator should use a smooth-edged bucket rather than a toothed bucket.

Pipe bedding and pipe zone materials shall conform to Section 9-03.12(3) of the *WSDOT Standard Specifications.* Pipe bedding should provide a firm uniform cradle for support of the pipes. A minimum 4-inch thickness of bedding material beneath the pipe should be provided. Prior to installation of the pipe, the pipe bedding should be shaped to fit the lower part of the pipe exterior with reasonable closeness to provide uniform support along the pipe. Pipe bedding material should be used as pipe zone backfill and placed in layers and tamped around the pipes to obtain complete contact. To protect the pipe, bedding material should extend at least 6 inches above the top of the pipe.

Placement of bedding material is particularly critical where maintenance of precise grades is essential. Backfill placed within the first 12 inches above utility lines should be compacted to at least 90% of the maximum dry density (ASTM D1557), such that the utility lines are not damaged during backfill placement and compaction. In addition, rock fragments greater than 1 inch in maximum dimension should be excluded from this first lift. The remainder of the utility excavations should be backfilled and compacted to 95% of the maximum dry density as determined by ASTM D1557.

Onsite soils are considered suitable for utility trench backfill provided they are free of oversize material and trash/debris and can be adequately compacted. All excavations should be wide enough to allow for compaction around the haunches of pipes and underground tanks. We recommend that utility trenching, installation, and backfilling conform to all applicable federal, state, and local regulations such as OSHA and WISHA for open excavations.

Compaction of backfill material should be accomplished with soils within ±2% of their optimum moisture content in order to achieve the minimum specified compaction levels recommended in this report. However, initial lift thickness could be increased to levels recommended by the

12.6 Imported Crushed Rock Structural Fill

Imported structural fill shall consist of well-graded, crushed aggregate material meeting the grading requirements of Washington State Department of Transportation (WSDOT) Standard Specification 9-03.9(3) (1-1/4 inch minus Base Course Material) presented here:

Table 6: WSDOT Standard Spec. 9-03.9(3)

Sieve Size	Percent Passing (by Weight)
1¼ Inch Square	99 - 100
1 Inch Square	80 - 100
5/8 Inch Square	50 – 80
U.S. No. 4	25 - 45
U.S. No. 40	3 – 18
U.S. No. 200	Less than 7.5

A fifty (50) pound sample of each imported fill material shall be collected by GNN personnel prior to placement to ensure proper gradation and establish the moisture-density relationship (proctor curve).

12.7 Compaction Requirements for Engineered Fill

All fill or backfill shall be approved by a representative of the GER, placed in uniform lifts, and compacted to a minimum 95% of the maximum dry density as determined by ASTM D1557. The compaction effort must be verified by a representative of the GER in the field using a nuclear density gauge in accordance with ASTM D6938. The thickness of the loose, non-compacted, lift of structural fill shall not exceed 8 inches for heavy-duty compactors or 4 inches for hand operated compactors.

12.8 Building Pad & Foundation Subgrade Preparation

Building structures may be supported on conventional shallow foundations bearing on subgrade prepared in accordance with the recommendations of this report. We recommend that all building foundations, including all exterior footings, interior footings and isolated column footings for any over-hang patio roof/decks, be supported on uniform improved native subgrade support conditions. The minimum footing depth shall be 24 inches below adjacent grades for frost protection and bearing capacity considerations. Interior footings may be supported at nominal depths below the floor. All footings shall be protected against weather and water damage during/after construction.

Following completion of site clearing and grubbing operations, all foundation areas shall be over-excavated to expose the native silty gravels. We anticipate the native soils in the vicinity of the currently proposed building footprints will range from depths of approximately 3 to 8 feet BGS. In order to reduce the risk of differential settlement, we recommend the differential in depth of foundation over-excavation (thickness of fill) be limited to 50%; i.e. if the deepest required foundation over-ex is 6 feet, then no portion of the foundation excavation shall be less than 3 feet below footing elevation. The exposed native gravelly stratum shall be moisture-conditioned (as necessary) and proof-compacted to a dense and non-yielding surface. Any soft spots encountered during compaction shall be over-excavated an additional 12 inches and replaced as compacted fill. Although not anticipated, deeper foundation over-excavations may extend into groundwater; consequently, employment of appropriate means of dewatering by the contractor may be required.

Foundation backfill shall consist of suitable screened/processed onsite soils (see *Suitability of Onsite Soils as Engineered Fill*) and/or imported 2-inch minus Gravel Borrow material (meeting the grading and quality requirements of WSDOT Standard Spec. Sec. 9-03.14(1)). The upper 12 inches of backfill directly below the foundations shall consist of imported 1¼”-minus crushed rock structural fill placed as engineered fill, moisture-conditioned and compacted to at least 95% of the maximum dry density as determined by the ASTM D1557. Crushed rock structural fill shall extend minimum 12 inches beyond the edges of the footings.

Where future buildings are proposed near or on the existing slopes, building foundations will be required to be constructed with appropriate setbacks in accordance with IBC 2015 Section 1808.7 (see *Slope Setbacks* section below). In general, if buildings are constructed with the current proposed layout, deeper embedment of the foundations will be required in order to meet the minimum setback, such that a minimum distance of 10 feet from the exterior face of the footings to a projected 2H:1V slope face from the toe of the existing slope is maintained. These recommendations may require the need for stepped foundations across the building structure, or deeper foundations such as taller stem-walls or columns.

Footings constructed in accordance with the above recommendations may be designed for an allowable bearing capacity of **2,500 pounds per square foot (psf)**. The allowable bearing pressure may be increased by 1/3 for short-term transient loading conditions. The estimated total settlement for footings is approximately 1-inch with differential settlement less than half that magnitude. The weight of the foundation concrete below grade may be neglected in dead load computations.

Lateral forces on foundations from short term wind and seismic loading would be resisted by friction at the base of foundations and passive earth pressure against the buried portions. We recommend an allowable passive earth pressure for the compacted onsite soil of **220 pcf**. This lateral foundation resistance value includes a factor of safety of 1.5. We recommend a coefficient of friction of **0.45** be used between cast-in-place concrete and imported crushed rock fill. An appropriate factor of safety should be used to calculate sliding resistance at the base of footings.

12.9 Slab-on-Grade Floors

We recommend placing a minimum 6-inch layer of crushed aggregate fill beneath all slabs. The material shall meet the WSDOT Specification 9-03.9 (3), “Crushed Surfacing Top Course”. The crushed rock material shall be compacted to at least 95% of the maximum dry density as determined by the ASTM D1557 method. Prior to placement of crushed aggregate fill, the building pad shall be prepared as described above in the *Building Pad & Foundation Subgrade Preparation* section. We recommend a modulus of subgrade reaction equal to 120 pounds per cubic inch (pci) based on a value for gravel presented in the Portland Cement Association publication No. EB075.01D. Slab thickness, reinforcement and joint spacing shall be determined by a licensed engineer based on the intended use and loading.

An appropriate vapor retarder (15-mil polyethylene liner) shall be used (ASTM E1745/E1643) beneath areas receiving moisture sensitive resilient flooring/VCT where prevention of moisture migration through slab is essential. The slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder. The architect shall determine the need and use of a vapor retarder.

12.10 Retaining Walls

The following table presents recommendations for lateral earth pressures for use in retaining wall design. The values are given in terms of equivalent fluid pressures without surcharge loads and are based on the assumption that proper drainage is provided behind the wall, the backfill is horizontal and that no-buildup of hydrostatic pressure occurs.

Table 7: Lateral Earth Pressures

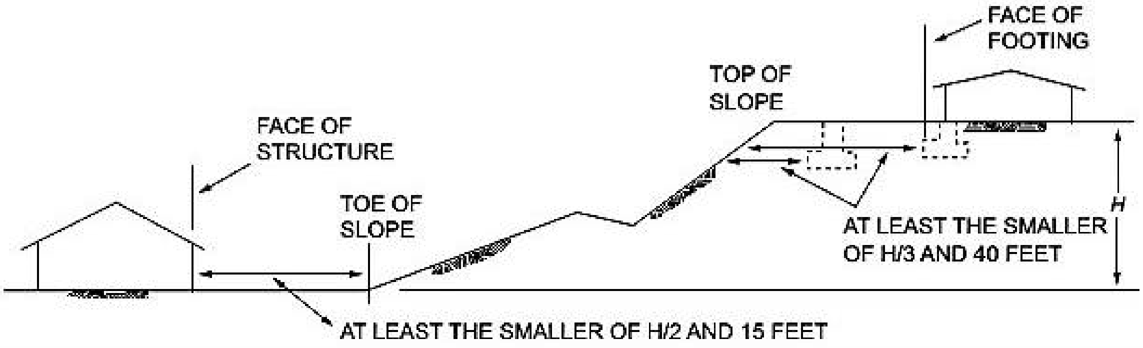
Lateral Pressures	Suitable Onsite Soils
Active Pressure Use when wall is permitted to rotate 0.1 to 0.2% of wall height for granular backfill	38 pcf - level ground
At-Rest Pressure	56 pcf - level ground

Drainage: Retaining structures should include adequate back drainage to avoid build-up of hydrostatic pressures. Positive drainage for retaining walls should consist of a vertical layer of permeable material (chimney drain), such as a pea gravel or crushed rock (typically ¼- to ¾-inch crushed), at least 18 inches thick, positioned between the retaining wall and the backfill. We recommend installing a non-woven filter fabric such as Mirafi 140N between the drainage material and the general backfill to prevent fines from migrating into the drainage material. A 4-inch diameter perforated or slotted drain-pipe, wrapped or socked in filter fabric, shall be installed at the bottom of the chimney drain.

Backfill and Subgrade Compaction: Compaction on the retained side of the wall within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment. Retaining wall foundations and subgrade improvements shall be constructed in accordance with the recommendations of this report.

12.11 Slope Setbacks

In accordance with IBC 2015 Section 1808.7 *Foundations on or Adjacent to Slopes*: "foundations on or adjacent to slope surfaces shall be founded in firm material with an embedment and setback from the slope surface sufficient to provide vertical and lateral support for the foundation without detrimental settlement." IBC Figure 1808.7.1 (presented below) defines the appropriate minimum setbacks from ascending and descending slope surfaces:



Appropriate setbacks can be accommodated by lateral offset and/or increased embedment. The long-term performance of the structure near slopes is dependent on the protection of slopes from erosion or over steepening from subsequent slope grading. Slopes should be maintained to prevent erosion or undermining of the toe.

12.12 Flexible Pavement

Due to the presence of undocumented fills throughout the project site, remedial grading will be required to minimize the risk of pavement distress. We recommend that the new pavement section be constructed on an improved subgrade. Due to the presence of artificial fills soils that include some miscellaneous trash and debris, the pavement subgrade over-excavation be completed in accordance with one of the following two options:

(1) Pavement areas shall be fully over-excavated to remove the artificial fill soils. Based on our site exploration, we anticipate that the maximum depth of excavation could be as great as approximately 8 feet.

(2) Excavate the proposed pavement areas to a minimum depth of 12 inches BGS. We recommend installing a Mirafi 600X geotextile fabric at the bottom of the over-ex. It must be understood that if this option is selected, the owner must accept some risks related to future distresses to the pavements including the potential for settlement and cracking.

After appropriate over-excavation is complete and confirmed by a representative of the GER, the exposed native subgrade shall be moisture-conditioned and compacted to a dense and non-yielding surface. After a suitable subgrade is confirmed by a representative of the GER, the over-excavation shall be backfilled with engineered structural fill soil consisting of suitable/screened onsite soil (see Section 12.3) and/or imported 2-inch minus Gravel Borrow material (meeting the grading and quality requirements of WSDOT Standard Spec. Sec. 9-03.14(1)). Engineered structural fill soils shall be placed in max. 8-inch thick loose lifts and each lift compacted to 95% of ASTM D1557. The following table presents recommended light duty and heavy-duty asphalt pavement sections for proposed project to constructed atop the prepared subgrade:

Table 8: Recommended Asphalt Concrete Paving Sections

Traffic	Asphalt Thickness (inches)	Crushed Aggregate Base Course (inches)
Heavy Duty†	4.0	10*
Standard Duty ††	3.0	6

†Heavy duty applies to pavements subjected to truck traffic and drive lanes
††Standard duty applies to general parking areas
*The upper 2" of crushed rock should be top course rock placed over the base course layer

Pavement section recommendations assume proper drainage and construction monitoring. Pavement shall be constructed on a dense and non-yielding surface. All fills used to raise low areas must be compacted structural fills and shall be placed under engineering control conditions.

Soils containing roots or organic materials shall be completely removed from the proposed paved areas prior to subgrade construction. The upper 12 inches of subgrade soils beneath the pavement section shall be moisture conditioned and proof-compacted to a dense and non-yielding condition. All fills used to raise low areas must be compacted onsite soils or structural gravel fill and shall be placed under engineering control conditions. The finished surface shall be smooth, uniform and free of localized weak/soft spots. All subgrade deficiency corrections and drainage provisions shall be made prior to placing the aggregate base course. All underground utilities shall be protected prior to grading.

The HMAC utilized for the project should be designed and produced in accordance with Section 5-04 Hot Mix Asphalt of the *Washington Department of Transportation 2014 Standard Specifications for Road and Bridge Construction* (WSDOT Specifications). Aggregate Base material shall comply with Section 9-03.9(3) Crushed Surfacing of the *WSDOT Specifications*. Aggregate base or pavement materials should not be placed when the surface is wet.

12.13 Subgrade Protection

The degree to which construction grading problems develop is expected to be dependent, in part, on the time of year that construction proceeds and the precautions which are taken by the contractor to protect the subgrade. The fine-grained soils currently present on site are considered to be moisture and disturbance sensitive due to their fines content and may become unstable (pumping) if allowed to increase in moisture content and are disturbed (rutted) by construction traffic if wet. If necessary, the construction access road should be covered with a layer of gravel or quarry spalls course. The soils are also susceptible to erosion in the presence of moving water. The soils shall be stabilized to minimize the potential of erosion into the foundation excavation. The site shall be graded to prevent water from ponding within construction areas and/or flowing into excavations. Accumulated water must be removed immediately along with any unstable soil. Foundation concrete shall be placed and excavations backfilled as soon as possible to protect the bearing grade. We further recommend that soils that become unstable are to be either:

- Removed and replaced with structural compacted gravel fill, or
- Mechanically stabilized with a coarse crushed aggregate (possibly underlain with a geotextile) and compacted into the subgrade.

12.14 Surface Drainage

With respect to surface water drainage, we recommend that the ground surface be sloped to drain away from the structure. Final exterior site grades shall promote free and positive drainage from the building areas. Water shall not be allowed to pond or to collect adjacent to foundations or within the immediate building area. We recommend that a gradient of at least 5% for a minimum distance of 10 feet from the building perimeter be provided, except in paved locations. In paved areas, a minimum gradient of 1% should be provided unless provisions are included for collection/disposal of surface water adjacent to the structure. Catch basins, drainage swales, or other drainage facilities should be aptly located. All surface water such as that coming from roof downspouts and catch basins be collected in tight drain lines and carried to a suitable discharge point, such as a storm drain system. Surface water and downspout water should not discharge into a perforated or slotted subdrain, nor should such water discharge onto the ground surface adjacent to the building. Cleanouts should be provided at convenient locations along all drain lines.

12.15 Wet Weather Conditions

The project site soils are fine-grained and sensitive to moisture during handling and compaction. Proceeding with site earthwork operations using these soils during wet weather could add project costs and/or delays. The stability of exposed soils may rapidly deteriorate due to a change in moisture content. Therefore, if possible, complete site clearing, preparation, and earthwork during periods of warm, dry weather when soil moisture can be controlled by aeration. During/subsequent to wet weather, drying or compacting the on-site soils will be difficult. It may be necessary to amend the on-site soils or import granular materials for use as structural fill. If earthwork takes place in wet weather/conditions, the following recommendations should be followed:

- Fill material should consist of clean, granular soil, and not more than 3% fines (by weight) should pass the No. 200 sieve. Fines should be non-plastic. These soils would have to be imported to the site.
- Earthwork should be accomplished in small sections and carried through to completion to reduce exposure to wet weather. Soils that becomes too wet for compaction should be removed and replaced with clean, granular material.
- The construction area ground surface should be sloped and sealed to reduce water infiltration, to promote rapid runoff, and to prevent water ponding.
- To prevent soil disturbance, the size or type of equipment may have to be limited.
- Work areas and stockpiles should be covered with plastic. Straw bales, straw wattles, geotextile silt fences, and other measures should be used as appropriate to control soil erosion.
- Excavation and fill placement should be observed on a full-time basis by a representative of GER to determine that unsuitable materials are removed and that suitable compaction and site drainage is achieved.

13.0 REFERENCES

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U.S. Geologic Survey (USGS), Earthquake Hazards Program, U.S. Seismic Design Maps, online tool, website: earthquake.usgs.gov/designmaps/us/application.php

Washington State Department of Natural Resources (DNR), Washington Division of Geology and Earth Resources, on-line mapping tool, https://fortress.wa.gov/dnr/protectiongis/geology/

14.0 CONTINUING GEOTECHNICAL SERVICES

GNN recommends that the Client should maintain an adequate program of geotechnical consultation, construction monitoring, and soils testing during the final design and construction phases to monitor compliance with GNN's geotechnical recommendations. Maintaining GNN as the geotechnical consultant from beginning to end of the project will provide continuity of services. If GN Northern, Inc. is not retained by the owner/developer and/or the contractor to provide the recommended geotechnical inspections/observations and testing services, the geotechnical engineering firm or testing/inspection firm providing tests and observations shall assume the role and responsibilities of Geotechnical Engineer-of-Record.

GNN can provide construction monitoring and testing as additional services. The costs of these services are not included in our present fee arrangement, but can be obtained from our office. The recommended construction monitoring and testing includes, but is not necessarily limited to, the following:

- Consultation during the design stages of the project.
- Review of the grading and drainage plans to monitor compliance and proper implementation of the recommendations in GNN's Report.
- Observation and quality control testing during site preparation, grading, and placement of engineered fill as required by the local building ordinances.
- Geotechnical engineering consultation as needed during construction

15.0 LIMITATIONS OF THE GEOTECHNICAL SITE INVESTIGATION REPORT

This GEOTECHNICAL SITE INVESTIGATION REPORT ("Report") was prepared for the exclusive use of the Client. GN Northern, Inc.'s (GNN) findings, conclusions and recommendations in this Report are based on selected points of field exploration, and GNN's understanding of the proposed project at the time the Report is prepared. Furthermore, GNN's findings and recommendations are based on the assumption that soil, rock and/or groundwater conditions do not vary significantly from those found at specific exploratory locations at the project site. Variations in soil, bedrock and/or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until during or after construction. Variations in soil, bedrock and groundwater may require additional studies, consultation, and revisions to GNN's recommendations in the Report.

In many cases the scope of geotechnical exploration and the test locations are selected by others without consultation from the geotechnical engineer/consultant. GNN assumes no responsibility and, by preparing this Report, does not impliedly or expressly validate the scope of exploration and the test locations selected by others.

This Report's findings are valid as of the issued date of this Report. However, changes in conditions of the subject property or adjoining properties can occur due to passage of time, natural processes, or works of man. In addition, applicable building standards/codes may change over time. Accordingly, findings, conclusions, and recommendations of this Report may be invalidated, wholly or partially, by changes outside of GNN's control. Therefore, this Report is subject to review and shall not be relied upon after a period of **one (1) year** from the issued date of the Report.

In the event that any changes in the nature, design, or location of structures are planned, the findings, conclusions and recommendations contained in this Report shall not be considered valid unless the changes are reviewed by GNN and the findings, conclusions, and recommendations of this Report are modified or verified in writing.

This Report is issued with the understanding that the owner or the owner's representative has the responsibility to bring the findings, conclusions, and recommendations contained herein to the attention of the architect and design professional(s) for the project so that they are incorporated into the plans and construction specifications, and any follow-up addendum for the project. The owner or the owner's representative also has the responsibility to verify that the general contractor and all subcontractors follow such recommendations during construction. It is further understood that the owner or the owner's representative is responsible for submittal of this Report to the appropriate governing agencies. The foregoing notwithstanding, no party other than the Client shall have any right to rely on this Report and GNN shall have no liability to any third party who claims injury due to reliance upon this Report, which is prepared exclusively for Client's use and reliance.

GNN has provided geotechnical services in accordance with generally accepted geotechnical engineering practices in this locality at this time. GNN expressly disclaims all warranties and guarantees, express or implied.

Client shall provide GNN an opportunity to review the final design and specifications so that earthwork, drainage and foundation recommendations may be properly interpreted and implemented in the design and specifications. If GNN is not accorded the review opportunity, GNN shall have no responsibility for misinterpretation of GNN's recommendations.

Although GNN can provide environmental assessment and investigation services for an additional cost, the current scope of GNN's services does not include an environmental assessment or an investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

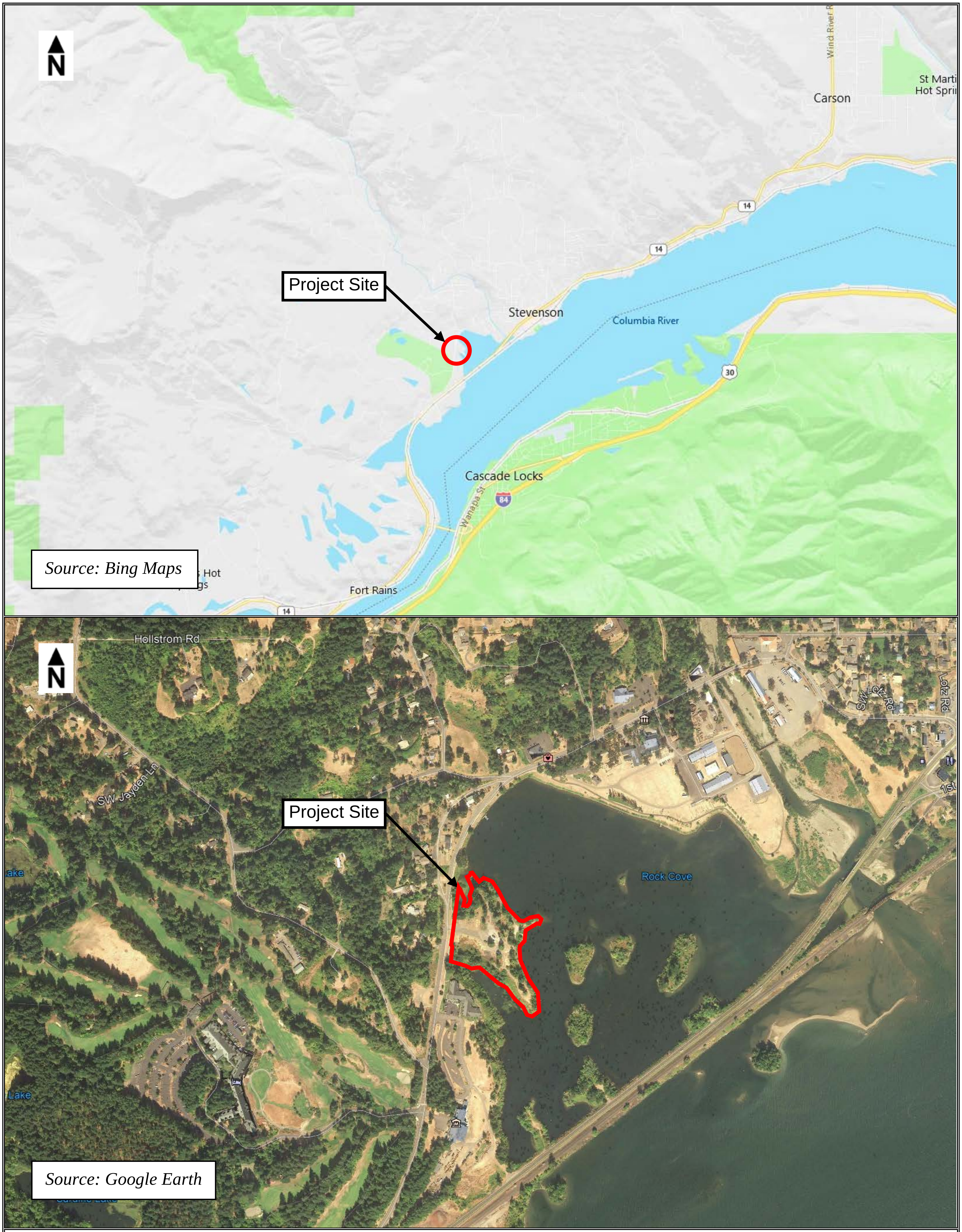
APPENDICES

Appendix I

Vicinity Map (Figure 1)

Site Exploration Map (Figure 2)

Critical Areas Map (Figure 3)



Source: Bing Maps

Source: Google Earth

FIGURE 1: VICINITY MAP **PROJECT NO. 219-1183**

LEGEND

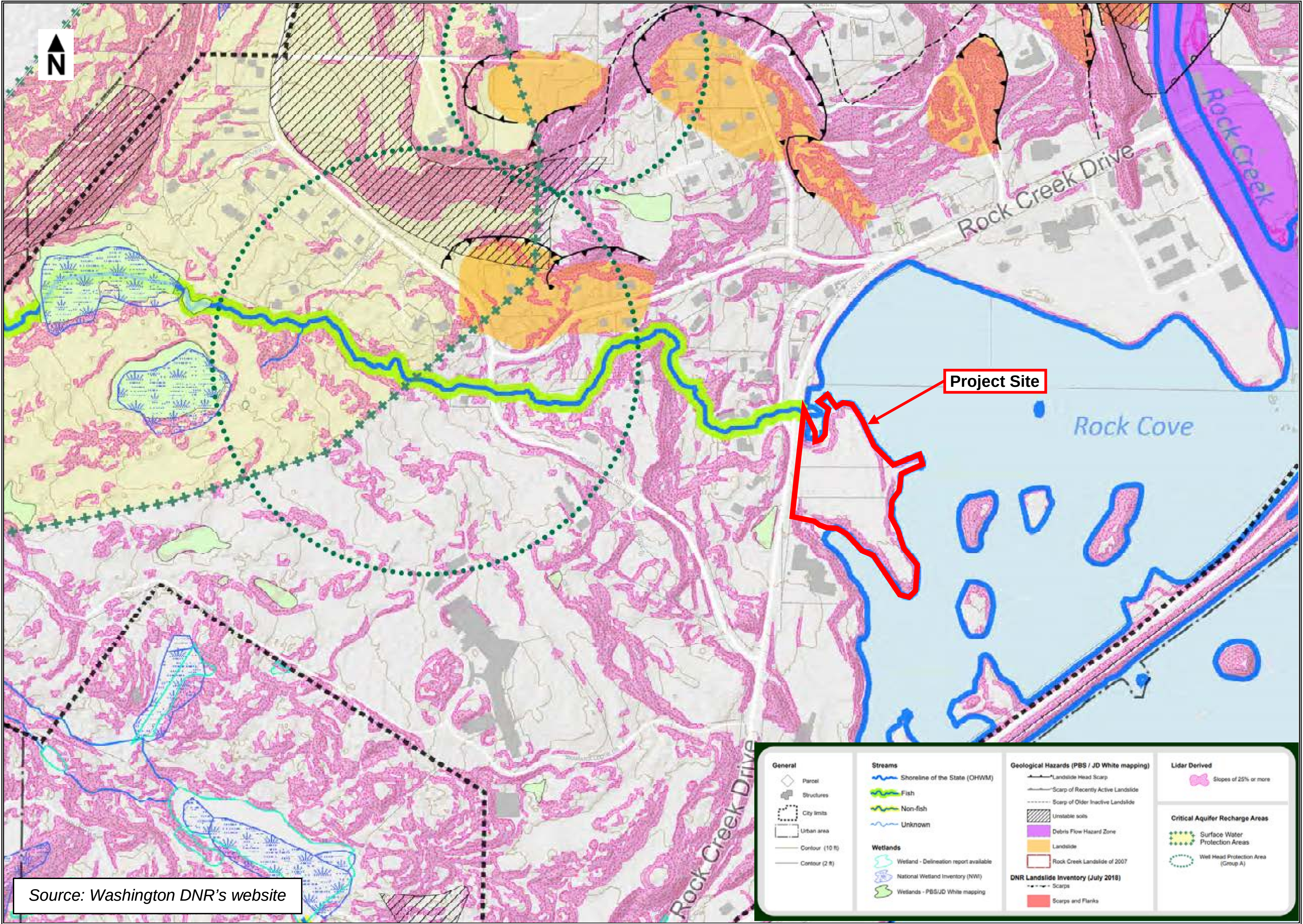
🟥 = Exploratory test pit

🟦 = Infiltration test

NOTE

Base aerial image from *Google Earth*; overlayed *Concept D* dated October 28, 2019 prepared by FDM Development, Inc.

FIGURE 2: SITE EXPLORATION MAP **PROJECT NO. 219-1183**



Source: Washington DNR's website

FIGURE 3: CRITICAL AREAS MAP

PROJECT NO. 219-1183

Appendix II

Exploratory Test-Pit Logs

Key Chart (for Soil Classification)

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TEST PIT NUMBER TP-1

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CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 98 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

AT TIME OF EXCAVATION ---

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'19.59"N, 121°53'55.44"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					~6" to 18" LANDSCAPE CUTTINGS / ORGANIC DEBRIS	
				1.5		96.5
2.5			GP-GM		FILL: POORLY GRADED GRAVEL WITH SILT AND SAND, (GP-GM) gray, moist to wet, appears loose to medium dense, with cobbles, with wood and organic debris - pipe at ~3' BGS	
				4.5		93.5
5.0	GB	MC = 22% Fines = 18%	GC		CLAYEY GRAVEL WITH SAND, (GC) brown, wet, appears loose to medium dense, with organics and roots (APPARENT NATIVE)	
7.5					- becomes blueish gray, moist, appears medium dense (NATIVE)	
10.0					- with boulders from 10' to 11'	
12.5						
				14.5		83.5

- Significant amount of surface water flowing into test-pit excavation
- Referenced elevations are approximate and based on Survey Topography for Lots 2, 3, and 4 of Rock Creek Cove dated December 11, 2019 prepared by S&F Land Services

Bottom of test pit at 14.5 feet.

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TEST PIT NUMBER TP-2

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CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 98 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

AT TIME OF EXCAVATION ---

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'18.75"N, 121°53'55.09"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0			GM		FILL: SILTY GRAVEL WITH SAND, (GM) brown, angular, moist, appears loose, with cobbles, with roots	
2.5						
					- becomes orange brown, appears loose to medium dense, some cobbles	
	GB	MC = 28% Fines = 47%				
5.0						
					- with a significant amount of woody debris, organics, roots	
7.5						
				8.0		90.0
			GM		SILTY GRAVEL WITH SAND, (GM) brown, subrounded, moist, appears medium dense to dense, with cobbles and boulders (APPARENT NATIVE)	
10.0						
12.5						
				14.0		84.0

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 14.0 feet.

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TEST PIT NUMBER TP-3

PAGE 1 OF 1

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 97 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

∇ **AT TIME OF EXCAVATION** 14.00 ft / Elev 83.00 ft

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'20.75"N, 121°53'55.36"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			GP-GM	▩	FILL: POORLY GRADED GRAVEL WITH SILT AND SAND, (GP-GM) gray brown, angular, wet, appears loose to medium dense, with cobbles
2.5				▩	
				3.0	94.0
			GM	○	SILTY GRAVEL WITH SAND, (GM) brown, subrounded, moist, appears medium dense, trace cobbles (APPARENT NATIVE)
5.0				○	
7.5				○	
				○	- becomes blueish gray
10.0				○	
	✋ GB	MC = 29% Fines = 28%		○	
12.5				○	
				○	∇
				14.5	82.5

- Groundwater encountered at ~14' BGS at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 14.5 feet.

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TEST PIT NUMBER TP-4

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 94.9 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

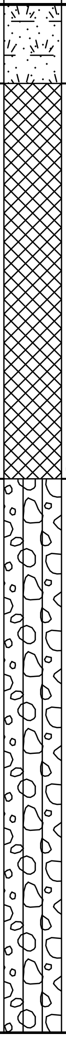
AT TIME OF EXCAVATION ---

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'23.09"N, 121°53'53.97"W

AFTER EXCAVATION ---



DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0				TOPSOIL/DUFF	
			1.0		93.9
2.5		GM		FILL: SILTY GRAVEL WITH SAND, (GM) brown, moist, appears loose to medium dense, some cobbles, trace boulders	
5.0			6.0	- chainsaw blade	88.9
7.5 10.0 12.5		GM		SILTY GRAVEL WITH SAND, (GM) light brown, damp, appears medium dense, some cobbles (APPARENT NATIVE)	
			13.0		81.9

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 13.0 feet.

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TEST PIT NUMBER TP-5

PAGE 1 OF 1

CLIENT FDM Development
PROJECT NAME Proposed Rock Creek Cove Development
PROJECT NUMBER 219-1183
PROJECT LOCATION Rock Creek Drive, Stevenson, WA
DATE STARTED 12/23/19 **COMPLETED** 12/23/19
GROUND ELEVATION 96.9 ft **TEST PIT SIZE** 36 x 96 inches
EXCAVATION CONTRACTOR Riley Materials
GROUND WATER LEVELS:
EXCAVATION METHOD Link-Belt 145x4 Excavator
AT TIME OF EXCAVATION ---
LOGGED BY KAH **CHECKED BY** MYM
AT END OF EXCAVATION ---
NOTES Approx. GPS Coords.: 45°41'22.14"N, 121°53'53.51"W
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION		
0.0				TOPSOIL/SLASH/DUFF	1.0	95.9
2.5		GM		APPARENT FILL: SILTY GRAVEL WITH SAND, (GM) brown, moist, appears loose to medium dense, some cobbles, trace boulders	5.0	91.9
5.0 7.5 10.0		GM		SILTY GRAVEL WITH SAND, (GM) light brown, damp to moist, appears medium dense, some cobbles (APPARENT NATIVE)	12.0	84.9

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 12.0 feet.

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TEST PIT NUMBER TP-6

PAGE 1 OF 1

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 98 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

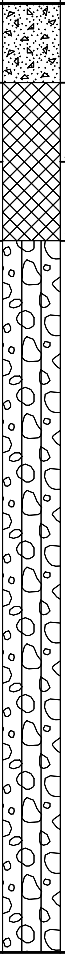
AT TIME OF EXCAVATION ---

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'21.16"N, 121°53'53.95"W

AFTER EXCAVATION ---



DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0				~12" CONCRETE SLAB	
1.0				FILL: BASALTIC GRAVEL/COBBLES, angular, some silty/sandy soil matrix	97.0
2.0					96.0
2.5		SM		FILL: SILTY SAND, (SM) gray, fine grained, damp to moist, appears medium dense	
3.0					95.0
				SILTY GRAVEL WITH SAND, (GM) brown, rounded to subrounded, damp to moist, appears medium dense to dense, with cobbles and boulders (APPARENT NATIVE)	
5.0					
7.5		GM			
10.0					
12.0					86.0

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 12.0 feet.

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TEST PIT NUMBER TP-7

PAGE 1 OF 1

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 97.6 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

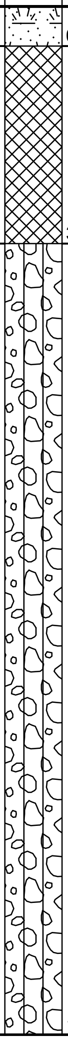
AT TIME OF EXCAVATION ---

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'19.86"N, 121°53'52.14"W

AFTER EXCAVATION ---



DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0			0.5	~6" TOPSOIL	97.1
2.5		GM	3.0	FILL: SILTY GRAVEL WITH SAND, (GM) brown, moist, appears loose to medium dense, some cobbles, trace boulders	94.6
5.0 7.5 10.0 12.5		GM	13.0	SILTY GRAVEL WITH SAND, (GM) light brown, damp to moist, appears medium dense, some cobbles (APPARENT NATIVE)	84.6

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 13.0 feet.

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TEST PIT NUMBER TP-8

PAGE 1 OF 1

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 89.5 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

∇ **AT TIME OF EXCAVATION** 12.00 ft / Elev 77.50 ft

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'20.44"N, 121°53'51.63"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0		GM	▨	FILL: SILTY GRAVEL WITH SAND, (GM) brown, moist, appears loose, some cobbles
2.0				87.5
2.5		GM	○	SILTY GRAVEL WITH SAND, (GM) brown, damp to moist, appears medium dense, some cobbles (APPARENT NATIVE)
5.0				
7.5				
10.0				- becomes moist to wet
				∇
12.5				
14.5				75.0

- Groundwater encountered at ~12' BGS at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 14.5 feet.

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TEST PIT NUMBER TP-9

PAGE 1 OF 1

CLIENT FDM Development
PROJECT NAME Proposed Rock Creek Cove Development
PROJECT NUMBER 219-1183
PROJECT LOCATION Rock Creek Drive, Stevenson, WA
DATE STARTED 12/23/19 **COMPLETED** 12/23/19
GROUND ELEVATION 87 ft **TEST PIT SIZE** 36 x 96 inches
EXCAVATION CONTRACTOR Riley Materials
GROUND WATER LEVELS:
EXCAVATION METHOD Link-Belt 145x4 Excavator
AT TIME OF EXCAVATION ---
LOGGED BY KAH **CHECKED BY** MYM
AT END OF EXCAVATION ---
NOTES Approx. GPS Coords.: 45°41'20.74"N, 121°53'49.97"W
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0			░	~6" to 12" TOPSOIL/ORGANICS	
1.0					86.0
		GM	▨	FILL: SILTY GRAVEL WITH SAND, (GM) brown, moist, appears loose to medium dense, some cobbles, trace boulders (APPARENT FILL)	
8.0					79.0

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 8.0 feet.

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TEST PIT NUMBER TP-10

PAGE 1 OF 1

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 100.3 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

AT TIME OF EXCAVATION ---

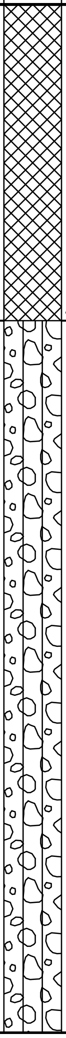
LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'15.46"N, 121°53'49.93"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0		GM		APPARENT FILL: SILTY GRAVEL WITH SAND, (GM) brown, moist, appears loose to medium dense, some cobbles
2.5				
			4.0	96.3
		GM		SILTY GRAVEL WITH SAND, (GM) light brown, damp to moist, appears medium dense, some roots in upper ~6", some cobbles (APPARENT NATIVE)
5.0				
7.5				
				- becomes orange brown, damp to moist (NATIVE)
10.0				
12.5				
			13.0	87.3



- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 13.0 feet.

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TEST PIT NUMBER TP-11

PAGE 1 OF 1

CLIENT FDM Development
PROJECT NAME Proposed Rock Creek Cove Development
PROJECT NUMBER 219-1183
PROJECT LOCATION Rock Creek Drive, Stevenson, WA
DATE STARTED 12/23/19 **COMPLETED** 12/23/19
GROUND ELEVATION 104 ft **TEST PIT SIZE** 36 x 96 inches
EXCAVATION CONTRACTOR Riley Materials
GROUND WATER LEVELS:
EXCAVATION METHOD Link-Belt 145x4 Excavator
AT TIME OF EXCAVATION ---
LOGGED BY KAH **CHECKED BY** MYM
AT END OF EXCAVATION ---
NOTES Approx. GPS Coords.: 45°41'16.39"N, 121°53'50.59"W
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0			SM		FILL: SILTY SAND WITH GRAVEL, (SM) dark gray brown, fine grained, damp to moist, appears loose to medium dense, with misc. trash/metal debris - becomes cemented, very dense	
2.5						
3.5						100.5
	✋ GB	MC = 17% Fines = 12%	SM		APPARENT FILL: SILTY SAND, (SM) reddish brown, fine grained, damp, appears medium dense, some gravel	
5.0						
6.0						98.0
			GM		SILTY GRAVEL WITH SAND, (GM) light brown, damp to moist, appears medium dense, with cobbles, with boulders (APPARENT NATIVE)	
7.5						
10.0						94.0

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 10.0 feet.

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TEST PIT NUMBER TP-12

PAGE 1 OF 1

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA

DATE STARTED 12/23/19 **COMPLETED** 12/23/19

GROUND ELEVATION 99.5 ft **TEST PIT SIZE** 36 x 96 inches

EXCAVATION CONTRACTOR Riley Materials

GROUND WATER LEVELS:

EXCAVATION METHOD Link-Belt 145x4 Excavator

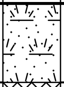
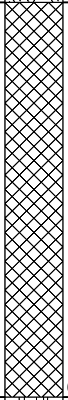
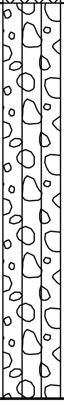
AT TIME OF EXCAVATION ---

LOGGED BY KAH **CHECKED BY** MYM

AT END OF EXCAVATION ---

NOTES Approx. GPS Coords.: 45°41'17.30"N, 121°53'51.73"W

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0				~12" TOPSOIL/DUFF	
1.0		GM		APPARENT FILL: SILTY GRAVEL WITH SAND, (GM) brown, damp, appears medium dense, some roots	98.5
6.0		GM		SILTY GRAVEL WITH SAND, (GM) light brown, damp, appears medium dense, some cobbles (APPARENT NATIVE)	93.5
11.0					88.5

- Groundwater not encountered at time of excavation
- Referenced elevations are approximate and based on Google Earth topography

Bottom of test pit at 11.0 feet.

KEY CHART

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COARSE-GRAINED SOILS			FINE-GRAINED SOILS		
DENSITY	**N (BLOWS/FT)**	**FIELD TEST**	**CONSISTENCY**	**N (BLOWS/FT)**	**FIELD TEST**
Very Loose	0 – 4	Easily penetrated with ½-inch reinforcing rod pushed by hand	Very Soft	0 – 2	Easily penetrated several inches by thumb
Loose	4 – 10	Difficult to penetrate with ½-inch reinforcing rod pushed by hand	Soft	2 – 4	Easily penetrated one inch by thumb
Medium -Dense	10 – 30	Easily penetrated with ½-inch rod driven with a 5-lb hammer	Medium-Stiff	4 – 8	Penetrated over ½-inch by thumb with moderate effort
Dense	30 – 50	Difficult to penetrate with ½-inch rod driven with a 5-lb hammer	Stiff	8 – 15	Indented about ½-inch by thumb but penetrated with great effort
Very Dense	> 50	penetrated only a few inches with ½-inch rod driven with a 5-lb hammer	Very Stiff	15 – 30	Readily indented by thumb
			Hard	> 30	Indented with difficulty by thumbnail

USCS SOIL CLASSIFICATION

MAJOR DIVISIONS			GROUP DESCRIPTION		
Coarse-Grained Soils <50% passes #200 sieve	**Gravel** and **Gravelly Soils** <50% coarse fraction passes #4 sieve	**Gravel** (with little or no fines)	○	GW	Well-graded **Gravel**
			●	GP	Poorly Graded **Gravel**
		Gravel (with >12% fines)	▮	GM	Silty **Gravel**
			▦	GC	Clayey **Gravel**
	Sand and **Sandy Soils** >50% coarse fraction passes #4 sieve	**Sand** (with little or no fines)	⁘	SW	Well-graded **Sand**
			⁖	SP	Poorly graded **Sand**
		Sand (with >12% fines)	⁞	SM	Silty **Sand**
			▨	SC	Clayey **Sand**
Fine-Grained Soils >50% passes #200 sieve	**Silt** and **Clay** Liquid Limit < 50		▥	ML	**Silt**
			▨	CL	Lean **Clay**
			┅	OL	Organic **Silt** and **Clay** (low plasticity)
	Silt and **Clay** Liquid Limit > 50		▮	MH	Inorganic **Silt**
			▨	CH	Inorganic **Clay**
			▨	OH	Organic **Clay** and **Silt** (med. to high plasticity)
Highly Organic Soils			▨	PT	**Peat** ≋ **Top Soil**

LOG SYMBOLS

⧗	2S	2" OD Split Spoon (SPT)
■	3S	3" OD Split Spoon
⧄	NS	Non-Standard Split Spoon
◈	ST	Shelby Tube
◫	CR	Core Run
⊠	BG	Bag Sample
⧓	TV	Torvane Reading
⌶	PP	Penetrometer Reading
☐	NR	No Recovery
▽ ▼	GW	Groundwater Table

MODIFIERS

DESCRIPTION	RANGE
Trace	<5%
Little	5% – 12%
Some	>12%

MOISTURE CONTENT

DESCRIPTION	FIELD OBSERVATION
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but not visible water
Wet	Visible free water

SOIL CLASSIFICATION INCLUDES

1. Group Name
2. Group Symbol
3. Color
4. Moisture content
5. Density / consistency
6. Cementation
7. Particle size (if applicable)
8. Odor (if present)
9. Comments

MAJOR DIVISIONS WITH GRAIN SIZE

SIEVE SIZE	12"	3"	3/4"	4	10	40	200
GRAIN SIZE (INCHES)	12	3	0.75	0.19	0.079	0.0171	0.0029

Boulders	Cobbles	Gravel		Sand			Silt and Clay
		Coarse	Fine	Coarse	Medium	Fine	

Conditions shown on boring and testpit logs represent our observations at the time and location of the fieldwork, modifications based on lab test, analysis, and geological and engineering judgment. These conditions may not exist at other times and locations, even in close proximity thereof. This information was gathered as part of our investigation, and we are not responsible for any use or interpretation of the information by others.

Appendix III

Laboratory Testing Results

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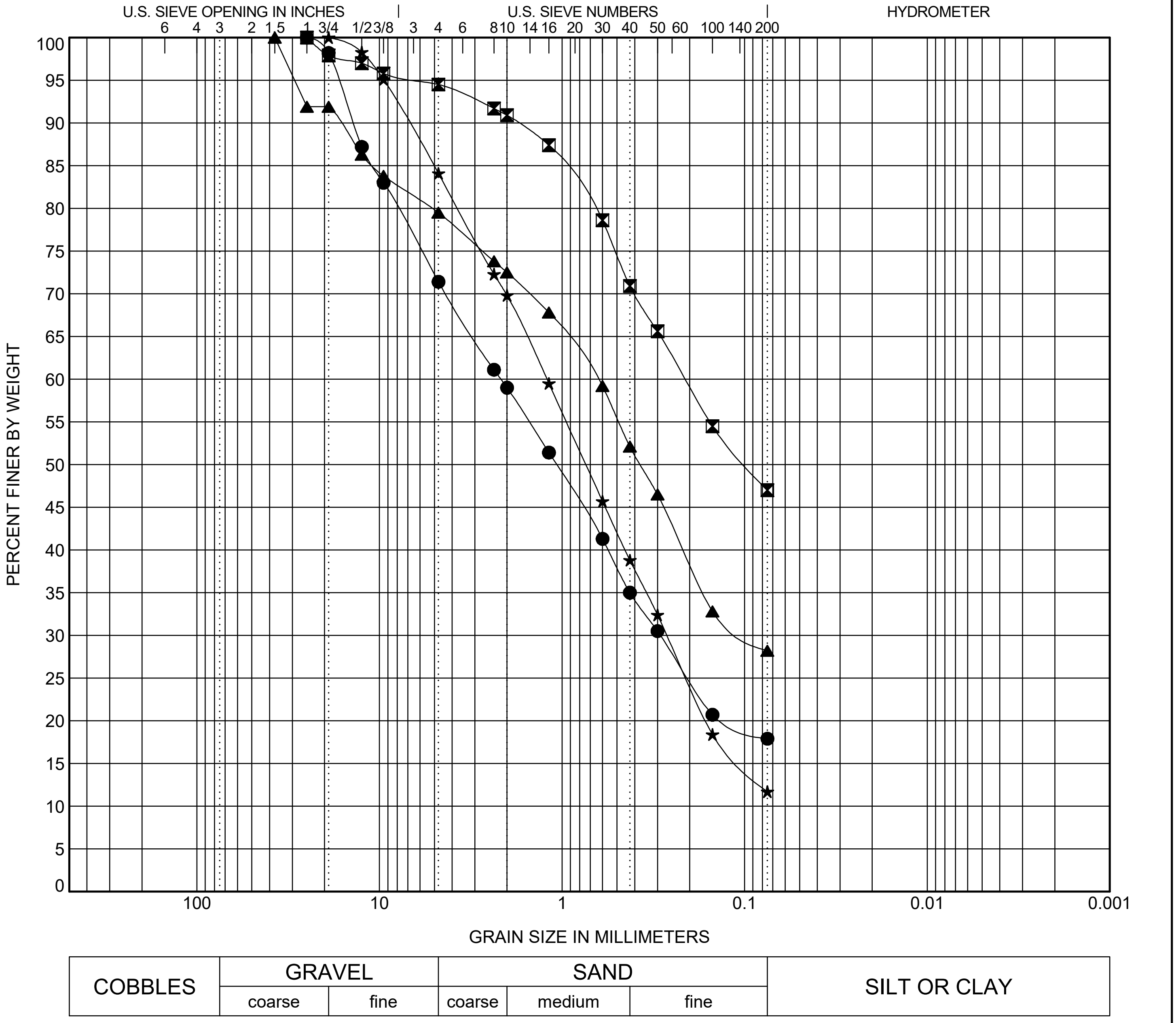
GRAIN SIZE DISTRIBUTION

CLIENT FDM Development

PROJECT NAME Proposed Rock Creek Cove Development

PROJECT NUMBER 219-1183

PROJECT LOCATION Rock Creek Drive, Stevenson, WA



BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
● **TP-1**	**5.0**	**SILTY SAND WITH GRAVEL (SM)**					
☒ **TP-2**	**3.5**	**SILTY SAND (SM)**					
▲ **TP-3**	**11.0**	**SILTY SAND WITH GRAVEL (SM)**					
★ **TP-11**	**4.0**	**SILTY SAND WITH GRAVEL (SM)**				**0.93**	**19.25**

BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **TP-1**	**5.0**	**25**	**2.164**	**0.29**		**28.6**	**53.5**	**17.9**	
☒ **TP-2**	**3.5**	**25**	**0.211**			**5.5**	**47.5**	**47.0**	
▲ **TP-3**	**11.0**	**37.5**	**0.639**	**0.098**		**20.5**	**51.3**	**28.2**	
★ **TP-11**	**4.0**	**19**	**1.211**	**0.266**		**15.9**	**72.4**	**11.7**	

Appendix IV

Site & Exploration Photographs

Excavation of test-pit TP-1, looking west

Exposed subsurface soil profile within test-pit TP-1

Excavation of test-pit TP-2, looking southwest

Exposed subsurface soil profile within test-pit TP-2

Excavation of test-pit TP-3, looking west

Exposed subsurface soil profile within test-pit TP-3

PLATE 1: SITE & EXPLORATION PHOTOGRAPHS **PROJECT NO. 219-1183**

View of site conditions near test-pit TP-4

Exposed subsurface soil profile within test-pit TP-4

Excavation of test-pit TP-5, looking east

Exposed subsurface soil profile within test-pit TP-5

Excavation of test-pit TP-6, looking north

Exposed subsurface soil profile within test-pit TP-6

PLATE 2: SITE & EXPLORATION PHOTOGRAPHS **PROJECT NO. 219-1183**

View of site conditions near test-pit TP-7, looking north

View of site conditions

View of site conditions near test-pit TP-8, looking west

Exposed subsurface soil profile within test-pit TP-8

Exposed subsurface soil profile within test-pit TP-9

Exposed subsurface soil profile within test-pit TP-10

PLATE 3: SITE & EXPLORATION PHOTOGRAPHS **PROJECT NO. 219-1183**

Exposed subsurface soil profile within test-pit TP-11

Exposed subsurface soil profile within test-pit TP-11

Excavation of test-pit TP-12, looking southwest

Exposed subsurface soil profile within test-pit TP-12

View of site conditions near test-pit TP-12, looking northwest

Infiltration test setup at test-pit P-1

PLATE 4: SITE & EXPLORATION PHOTOGRAPHS **PROJECT NO. 219-1183**

Appendix V

Historic Aerial Photographs

1935 Historic USGS Aerial

1952 Historic USGS Aerial

PLATE 1: HISTORIC AERIAL PHOTOGRAPHS **PROJECT NO. 119-1183**

1955 Historic USGS Aerial

1973 Historic USGS Aerial

PLATE 2: HISTORIC AERIAL PHOTOGRAPHS **PROJECT NO. 119-1183**

1975 Historic USGS Aerial

1979 Historic USGS Aerial

PLATE 3: HISTORIC AERIAL PHOTOGRAPHS **PROJECT NO. 119-1183**

1981 Historic USGS Aerial

1984 Historic USGS Aerial

PLATE 4: HISTORIC AERIAL PHOTOGRAPHS **PROJECT NO. 119-1183**

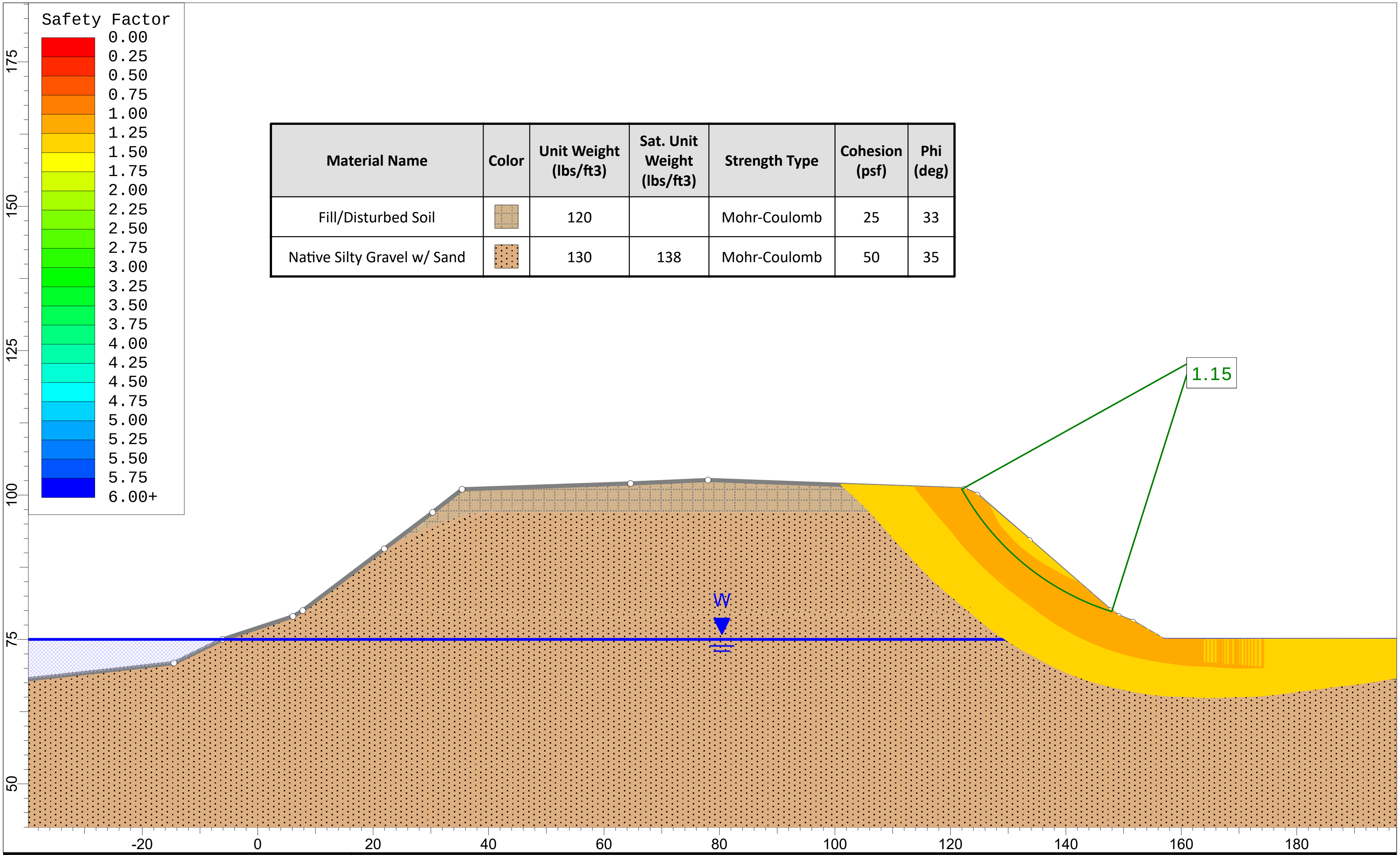
1993 Historic USGS Aerial

2007 Historic USGS Aerial

PLATE 5: HISTORIC AERIAL PHOTOGRAPHS **PROJECT NO. 119-1183**

Appendix VI

Slope Stability Analysis



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill/Disturbed Soil	▨	120		Mohr-Coulomb	25	33
Native Silty Gravel w/ Sand	░	130	138	Mohr-Coulomb	50	35

Northern, Inc.
Consulting Engineers Environmental Scientists Geologists
Construction Materials Testing Geophysical Services

Project
Rock Creek Cove Vacation Homes Project

Analysis Description
Existing Slopes Across Southern Peninsula - Spencer Method - Static

Drawn By MYM

Scale 1:276

Company GN Northern, Inc,

Date

File Name Slide1.slim

SLIDEINTERPRET 7.037

Appendix VII

NRCS Soil Survey

United States
Department of
Agriculture

NRCS

Natural
Resources
Conservation
Service

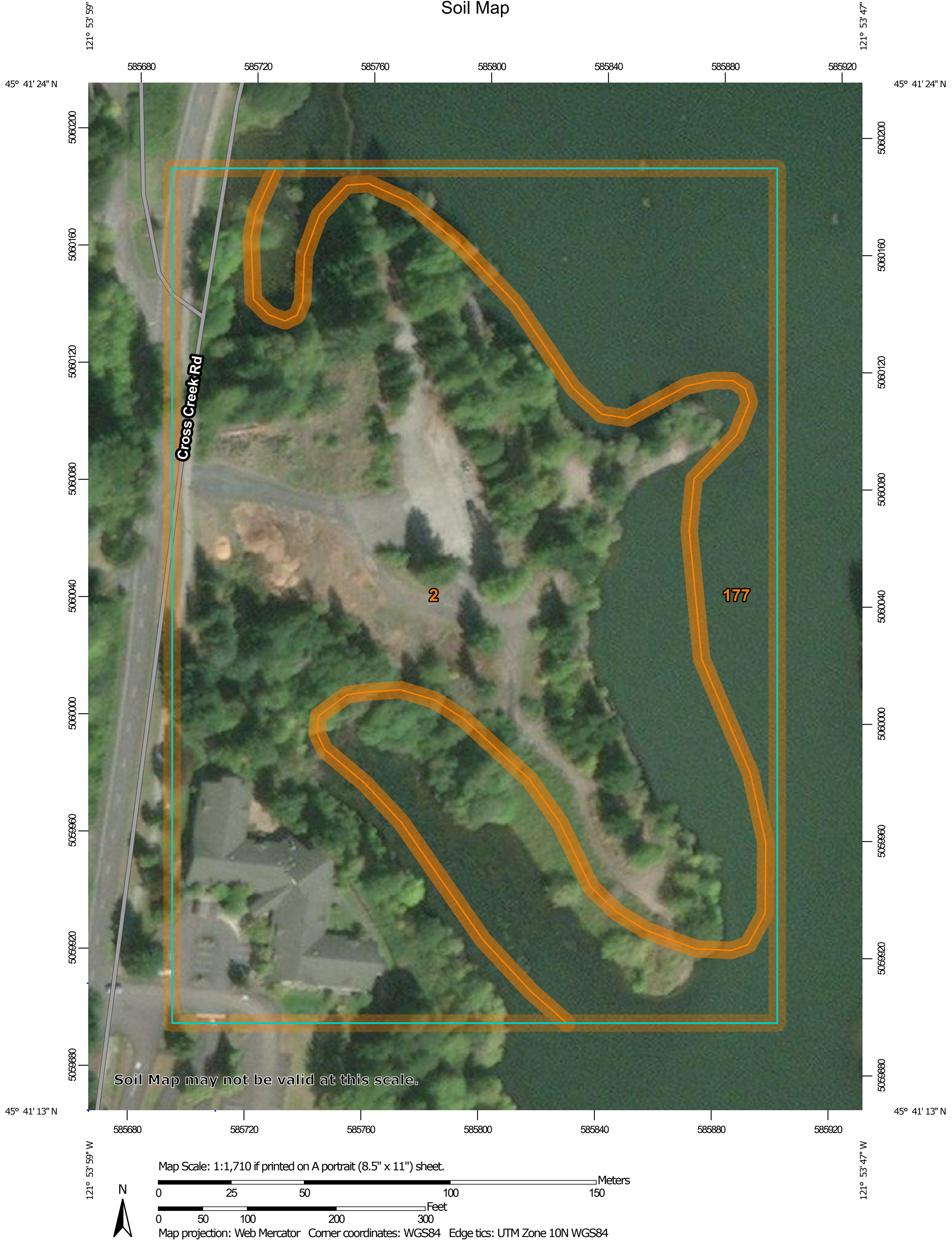
A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for
Skamania County Area, Washington

Rock Creek Cove Vacation Homes Project

January 9, 2020

Soil Map



Map Scale: 1:1,710 if printed on A portrait (8.5" x 11") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

Skamania County Area, Washington

2—Arents, 0 to 5 percent slopes

Map Unit Setting

National map unit symbol: 1hhrw
Elevation: 0 to 200 feet
Mean annual precipitation: 40 to 80 inches
Mean annual air temperature: 45 to 52 degrees F
Frost-free period: 90 to 200 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Arents and similar soils: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Arents

Setting

Landform: Terraces

Typical profile

H1 - 0 to 24 inches: gravelly sandy loam
H2 - 24 to 60 inches: extremely gravelly sandy loam

Properties and qualities

Slope: 0 to 5 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 5.95 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Moderate (about 6.3 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3s
Hydrologic Soil Group: A
Hydric soil rating: No

177—Water

Map Unit Composition

Water: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.