DRAFT GEOTECHNICAL REPORT
Green To Cedar Rivers Trail
Maple Valley, Washington

HWA Project No. 2021-163-21

Prepared for
Parametrix, Inc.

May 26, 2023

HWA GEOSCIENCES INC.
DBE/MWBE

Geotechnical Engineering
Pavement Engineering
Geoenvironmental
Hydrogeology
Inspection & Testing

May 26, 2023
HWA Project No. 2021-163-21

Parametrix, Inc.
719 2nd Avenue, Suite 200
Seattle, Washington 98104

Attention: Jennifer Dvorak, P.E.

Subject: **DRAFT GEOTECHNICAL ENGINEERING REPORT**
Green to Cedar Rivers Trail
Maple Valley, Washington

Dear Jennifer:

As requested, HWA GeoSciences Inc. (HWA) has completed a geotechnical investigation and infiltration testing to support the Green to Cedar Rivers Trail South Interim Segment A project in Maple Valley, Washington. This report presents the results of our field explorations, infiltration testing and laboratory testing along with recommendations pertaining to the new bridge, retaining walls, luminaire and pedestrian flashing beacon foundations, stormwater infiltration feasibility, the interim gravel trail, and earthwork. The attached draft report summarizes the results of our study and presents our conclusions and recommendations.

We appreciate the opportunity to have provided geotechnical engineering services on this project. If you have any questions regarding this report or require additional information or services, please contact the undersigned at your convenience.

Sincerely,

HWA GEOSCIENCES INC.

Joe Westergreen, P.E.
Geotechnical Engineer

JoLyn Gillie, P.E.
Geotechnical Engineer, Principal

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DRAFT GEOTECHNICAL ENGINEERING REPORT
GREEN TO CEDAR RIVERS TRAIL
MAPLE VALLEY, WASHINGTON

1.0 INTRODUCTION

1.1 GENERAL

This report summarizes the results of a geotechnical engineering investigation performed by HWA GeoSciences Inc. (HWA) for the Green to Cedar Rivers Trail South Interim Segment A project in Maple Valley and unincorporated King County, Washington. The approximate location of the project site is shown on the Site and Vicinity Map, Figure 1, and on the Site and Exploration Plan, Figures 2A through 2G. Our field work included drilling two boreholes, excavating 17 test pits and conducting 8 small scale Pilot Infiltration Tests (PITs) to evaluate soil and groundwater conditions and to evaluate infiltration potential of native soils. Laboratory tests were conducted on select soil samples to determine relevant engineering properties of the subsurface soils.

1.2 PROJECT UNDERSTANDING

It is our understanding that the project will extend approximately 1.8 miles from SE Kent Kangley Road to an existing footpath in the Black Diamond Open Space area. Improvements will include a new bridge to cross the active east-west BNSF Railroad line, and safe at grade trail crossings at SE Tahoma Way, SE 276th Street, SE 280th Street and SE 288th Street.

The trail will consist of a 12-foot-wide gravel path with 2-foot-wide gravel shoulders on each side and will be graded to meet ADA requirements. Grading will generally require minor cuts, fills, and retaining walls up to about 11 feet in height. Paving of the trail will be performed at a future date, which is not included with this phase of the project.

Based on correspondence with the design team, the pedestrian bridge will be approximately 167 feet long and will be supported on drilled shafts. Fill embankments up to about 7 feet in height will be needed to construct the bridge approaches. Structural Earth Walls (SEWs) will be utilized to keep the embankment fill from extending into the BNSF right-of-way. In addition, construction of the bridge will require re-routing an existing 12-inch water line. The water line is located outside of the BNSF right-of-way, about 60 feet from the tracks where the proposed south SEW wall will be located. We understand that waterline will be relocated by others.

We understand that infiltration of stormwater is desired as part of the project. Infiltration facilities are anticipated to consist of shallow infiltration trenches in areas where infiltration is deemed to be feasible.

2.0 FIELD AND LABORATORY TESTING

2.1 FIELD INVESTIGATION

Our phase 1 field investigation included review of available geologic and geotechnical data for the project corridor, a surface reconnaissance of the alignment, and subsurface explorations consisting of a combination of test pits and geotechnical borings. Our phase 2 field investigation included excavating 2 additional test pits and conducting 8 PITs. The locations of our explorations are presented in Figures 2A through 2E (Site and Exploration Plan). Summary exploration logs are presented in Appendix A.

2.1.1 Test Pits

HWA logged the excavation of 17 test pits, designated TP-1 through TP-17, to depths of about 9 to 14.5 feet. Test pits TP-1 through TP-15 were excavated on November 14 through 16, 2022 by Kelly's Excavating using a Komatsu WB 140 Backhoe under subcontract to HWA. Test pits TP-16 and TP-17 were excavated on April 3 and 4, 2023 by Kelly's Excavating using a Hitachi Mini Excavator under subcontract with HWA. An HWA geologist or geotechnical engineer monitored the excavation of the test pits. Soil samples obtained from the test pits were classified in the field and representative portions were placed in plastic bags and taken to our Bothell, Washington laboratory for further examination and testing.

Test pit exploration logs are presented in Appendix A, Figures A-5 through A-21. It should be noted that the stratigraphic contacts shown on the individual exploration logs represent the approximate boundaries between soil types, actual transition may be more gradual. The soil and groundwater conditions depicted are only for the specific date and locations reported and, therefore, are not necessarily representative of other locations and times.

The test pits were backfilled with native cuttings and compacted using the bucket on the backhoe or excavator. During backfilling PVC piezometers with groundwater transducers were installed in six of the test pits (TP-2, TP-3, TP-5, TP-7, TP-11, and TP-12) to monitor for potential high groundwater conditions throughout the wet winter months. During our Phase 1 test pit explorations, the excavator cleared a path to the proposed bridge abutment locations to establish access for the drill rig.

2.1.2 Geotechnical Borings

HWA logged the drilling of two machine-drilled borings to assess subsurface conditions for the proposed pedestrian bridge crossing the active BNSF rail line. The borings were completed on November 21 and 22, 2022 by Holocene Drilling, Inc. using a Diedrich D-70 tracked drill rig under subcontract to HWA. Boring BH-1 was advanced to approximately 60 feet. During drilling BH-2 auger refusal was encountered at approximately 40 feet due to rock obstructions. The drill was moved approximately 8 feet to the north and a second attempt was made (boring BH-2A) to get past the obstructions, on the second attempt refusal was encountered at approximately 38 feet.

Soil samples were collected within the exploratory borings at 2.5- to 5-foot depth intervals per Standard Penetration Test (SPT) sampling methods, which consisted of using a 2-inch outside diameter, split-spoon sampler driven with a 140-pound auto-hammer. During the test, each sample was obtained by driving the sampler up to 18 inches into the soil with the hammer free-falling 30 inches per stroke. The number of blows required for each 6 inches of penetration was recorded. The standard penetration resistance (N-value) of the soil was calculated as the number of blows required for the final 12 inches of penetration. If a total of 50 blows was recorded within a single 6-inch interval, the test was terminated, and the blow count was recorded as 50 blows/number of inches of penetration. This resistance provides an indication of the relative density of granular soils and the relative consistency of cohesive soils.

Additionally, a larger 3-inch outside diameter California Modified sampler was utilized at specific depths in boring BH-2 to improve sample recovery. The samples collected with this sampler have blow counts that do not reflect standardized values as they utilized the larger California Modified sampler with the standard 140-lb hammer. These values have been adjusted using the Burmister's relationship in our analyses to reflect standard SPT N-value blow counts for the purpose of design.

At the completion of the drilling, borings BH-1 and BH-2A were backfilled with bentonite chips per Washington State Department of Ecology (DOE) requirements. A groundwater monitoring well was installed in boring BH-2 to monitor seasonal groundwater fluctuations using a water level transducer. Once the target depth was reached, a 2-inch PVC groundwater monitoring piezometer was installed per Washington State DOE requirements. A flush mount monument was installed directly over the well to allow access for continued monitoring over the course of the design process.

The explorations were completed under the full-time observation of a geologist from HWA, who collected pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence, as the borings were advanced. Soils were classified in general accordance with the classification system described in Figure A-1, which also provides a key to the exploration log symbols. The boring logs are presented on Figures A-2 through A-4.

The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types. Actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

2.1.3 Pilot Infiltration Testing

As part of Phase 2 of our field explorations, HWA conducted 8 small-scale PITs in general accordance with the King County Surface Water Design Manual (KCSWDM) and the Stormwater Management Manual for Western Washington (SWMMWW). The excavations for the PITs were performed by Kelly's Excavating using a Hitachi Mini Excavator under subcontract to HWA between April 3 and April 10, 2023.

At each test location, an approximately 19 to 24 square foot excavation was advanced to the proposed infiltration receptor depths, approximately 3 to 4.5 feet below ground surface. Water was introduced in the excavation via fire hoses into a 6-inch diameter, perforated diffuser pipe from a water truck provided by Kelly's Excavating, or directly from a fire hydrant. Using a flow meter to control the flow rate, water was added to the PITs at a rate that maintained a water depth of approximately 1 foot for the pre-soak period. Due to the high permeability of the on-site soils, in some of the tests we were unable to achieve a constant head flow rate. Each test was run until the water truck (approximately 3,000 gallons) was empty. Following the constant head test, a falling head test was performed in each pit. For the falling head test, the water was shut off and the rate at which the water infiltrated into the pit was recorded until all of the water had infiltrated. The locations of the PITs are shown on the Site and Exploration Plan, Figures 2A through 2E.

A geotechnical engineer from HWA monitored the infiltration testing. After completing infiltration testing procedures, each PIT was excavated to depths between about 8 to 10.5 feet to assess and sample receptor soils and to evaluate if groundwater was present at shallow depths. The PIT exploration logs are presented in Appendix A, Figures A-22 through A-29. Observations from each PIT are included below.

PIT-1: The PIT was about 4.4 feet wide and 4.9 feet long, and the test was completed at a depth of about 3.5 feet bgs. During the pre-soak, water rose to about 1 foot above the excavation bottom and was maintained near that level until the water truck ran out of water. Water was introduced at a rate of about 19 gallons per minute (gpm) to maintain a constant water head of about 1 foot. After the water truck was empty the PIT was allowed to drain. It took 12 minutes for a head of about 1 foot of water to drain.

PIT-2: The PIT was about 4.4 feet wide and 5 feet long, and the test was completed at a depth of about 4.5 feet bgs. During the pre-soak, water rose to about 1 foot above the excavation bottom and was maintained near that level until the water truck ran out of water. Water was introduced at a rate of about 41 gpm to maintain a constant water head of approximately 1 foot. After the water truck was empty the PIT was allowed to drain. It took 11 minutes for a head of about 1 foot of water to drain.

PIT-3: The PIT was about 4 feet wide and 5 feet long, and the test was completed at a depth of about 3 feet bgs. During the pre-soak, water rose to about 1 foot above the excavation bottom and was maintained near that level until the water truck ran out of water. Water was introduced at a rate of about 45 gpm to maintain a constant water head of approximately 1 foot. After the water truck was empty the PIT was allowed to drain. It took about 2 minutes for a head of about 1 foot of water to drain.

PIT-4: The PIT was about 4 feet wide and 5 feet long, and the test was completed at a depth of about 4 feet bgs. During the pre-soak we were unable to achieve 1 foot of water head due to the high permeability of the soil. We were able to achieve a constant water head of about 0.85 feet, by pumping at a rate of about 45 gpm until the water truck ran out of water. After the water truck was empty the PIT was allowed to drain. It took 8 minutes for a head of about 0.85 feet of water to drain.

PIT-5: The PIT was about 4 feet wide and 4.8 feet long, and the test was completed at a depth of about 4 feet bgs. During the pre-soak, water rose to about 1 foot above the excavation bottom and was maintained near that level until the water truck ran out of water. Water was introduced at a rate of about 5 gpm to maintain a constant water head of approximately 1 foot. After the water truck was empty the PIT was allowed to drain. It took 85 minutes for a head of about 1 foot of water to drain.

PIT-6: The PIT was about 4.1 feet wide and 4.6 feet long, and the test was completed at a depth of about 4 feet bgs. During the pre-soak, water rose to about 1 foot above the excavation bottom and was maintained near that level until the water truck ran out of water. Water was introduced at a rate of about 39 gpm to maintain a constant water head of approximately 1 foot. After the water truck was empty the PIT was allowed to drain. It took 4 minutes for a head of about 1 foot of water to drain.

PIT-7: The PIT was about 4.5 feet wide and 5.2 feet long, and the test was completed at a depth of about 4.5 feet bgs. During the pre-soak period we were unable to achieve 1 foot of water head due to the high permeability of the soil. We were able to achieve a constant water head of about 0.6 feet, by pumping at a rate of about 63 gpm until the until the water truck ran out of water. After the water truck was empty the PIT was allowed to drain. It took 1.5 minutes for a head of about 0.6 feet of water to drain.

PIT-8: The PIT was about 4.8 feet wide and 5 feet long, and the test was completed at a depth of about 4.3 feet bgs. During the pre-soak water rose to about 1 foot above the excavation bottom and was maintained near that level until the water truck ran out water. Water was introduced at a rate of about 87 gpm to maintain a constant water head of approximately 1 foot. After the water truck was empty the PIT was allowed to drain. It took less than 1.5 minutes for a head of about 1 foot of water to drain.

2.2 LABORATORY TESTING

Laboratory tests were conducted at HWA's laboratory in Bothell, Washington, on selected samples retrieved from the test pits and borings to determine relevant index and engineering properties of the soils encountered. The tests included natural moisture content and grain size distribution analysis. The tests were conducted in general accordance with appropriate American Society of Testing and Materials (ASTM) standards. The test results and a discussion of the laboratory test methodologies are presented in Appendix B, and/or are displayed on the exploration logs in Appendix A, as appropriate.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The Green to Cedar Rivers Trail segment to be improved is located to the west of Maple Valley-Black Diamond Road SE (SR 169) and extends between SE Kent Kangley Road and the Black Diamond Open Space area. The existing trail is constructed on an old railroad grade, and generally slopes gently up from north to south gaining approximately 16 feet of elevation over about 2 miles. The alignment is bisected by an active east-west trending BNSF railroad line approximately 0.25 miles south of SE 280th Street, and Ravensdale Creek approximate 550 feet south of SE 288th Street.

From SE Kent-Kangley Road to SE 276th Street the existing trail is about 10 feet wide, surfaced with gravel, and bordered by grass on either side. This segment of the existing trail is bordered by commercial properties to the east, and residential properties or Summit Park to the west. From SE 276th Street to the active BNSF railroad crossing the existing gravel trail extends through a forested area and is constructed on an existing fill embankment. The embankment extends up to about 10 feet above existing grades with side slopes that generally have inclinations of about 2H:1V. Surrounding properties beyond the wooded area primarily consist of residential properties to the west, and Maple Valley-Black Diamond Road SE (SR 169) to the east.

The east-west BNSF rail line lies within an approximately 20-foot-deep cut through the embankment constructed to support a now abandoned north-south BNSF rail line. The old bridge footings and abutments from the abandoned rail line are still present. Soil exposed near the abandoned abutments generally consist of sand and gravel with cobbles. The existing embankments extending down towards the rail line appear to be stable. No groundwater seepage or signs of deep-seated slope instability, such as tension cracks, were observed during our site visits in December 2022.

From the active BNSF railroad line to SE 288th Street the existing trail is narrow and appears to be less maintained. Existing trail widths range from about 3 to 5 feet consisting primarily of exposed soil with vegetated shoulders constructed on the existing fill embankment. The exception being where the trail crosses an approximate 525-foot-wide cleared high voltage transmission line corridor approximately 0.25 miles south of the BNSF railroad tracks.

From SE 288th Street to south of Ravensdale Creek there is an approximate 900-foot-long segment of the existing trail that is being improved by Washington State Department of Transportation (WSDOT) as part of a culvert replacement project. Starting south of the Ravensdale Creek improvements and extending to the end of the project alignment at the footpath to the Black Diamond Open Space area the existing trail is about 10 feet wide, surfaced with gravel, relatively level, and bordered by forest with heavy underbrush on either side.

3.2 General Geologic Conditions

The project is located within the Puget Lowland. The Puget Lowland has repeatedly been occupied by a portion of the continental glaciers that developed during the ice ages of the Quaternary period. During at least four periods, portions of the ice sheet advanced south from British Columbia into the lowlands of Western Washington. The southern extent of these glacial advances was near Olympia, Washington. Each major advance included numerous local advances and retreats, and each advance and retreat resulted in its own sequence of erosion and deposition of glacial lacustrine, outwash, till, and drift deposits. Between and following these glacial advances, sediments from the Olympic and Cascade Mountains accumulated in the Puget Lowland. As the most recent glacier retreated, it uncovered a sculpted landscape of elongated, north-south trending hills and valleys between the Cascade and Olympic Mountain ranges. This landscape is composed of a complex sequence of glacial and interglacial deposits.

General geologic information for the site was obtained from the publication *Geologic Map of the Black Diamond Quadrangle, King County, Washington* (Mullineaux, 1965). The map indicates that the surficial geology in the project area generally consists of proglacial outwash plains deposits (Gpo), which were deposited by meltwater at the terminus of, and beyond, the retreating ice front of the Puget Lobe of the Cordilleran Ice Sheet during the Vashon Stade of the Fraser Glaciation. This unit is described as unweathered pebble and cobble gravel with localized areas of sand, ranging from 10 to 50 feet thick. As a recessional deposit, this unit would not have been overridden by glacial ice and would be considered normally consolidated.

3.3 Subsurface Conditions

3.3.1 Soils

In general soils encountered during our investigation are consistent with those identified on the geologic map. Brief descriptions of the soil units observed in our explorations are presented below in order of deposition, beginning with the most recently deposited. The geotechnical logs in Appendix A (Figures A-2 through A-29) provide more detail of subsurface conditions observed at specific locations and depths. The soils encountered in the explorations are described as follows:

- **Topsoil** – An approximate 3- to 18-inch-thick layer of organic-rich topsoil was generally encountered at the surface of the test pit excavations. The exception was TP-9 where the original topsoil (about 6-inches thick) was buried below about 4 feet of existing fill, and TP-12 where fill was observed starting at the ground surface.
- **Fill** – Up to 5 feet of fill was encountered in all the test pits, except for in PIT-4 through PIT-8. In addition, up to 12.5 feet of fill was encountered in the borings. The fill material generally consisted of loose to medium dense, gravel with variable amounts of sand, silt, and organics or sand with variable amounts of silt, gravel, and organics. Coarser-grained material observed in the test pit excavations ranged from fine gravel to cobbles and small boulders. Fill was not observed in PIT-4, or PIT-6 through PIT-8. The fill material is of similar composition to the underlying recessional outwash material.
- **Weathered Recessional Outwash** – Weathered recessional outwash was encountered in PIT-1, PIT-4, and PIT-6 through PIT-8 between the topsoil and underlying undisturbed recessional outwash. The weathered recessional outwash generally consists of slightly silty to silty, sandy gravel of slightly silty to silty gravelly sand. The material of similar composition to the underlying non-weathered soils but have undergone some observable loosening due to weathering and root growth to warrant a separate unit designation. Based on excavation difficulty the weathered recessional outwash is interpreted to be generally loose to medium dense.
- **Recessional Outwash** – Recessional outwash generally consisting of sandy gravel to silty, sandy, gravel or slightly silty to silty gravelly sand with variable amounts of cobbles and boulders was encountered below the fill and weathered recessional outwash. Based on excavation difficulty the recessional outwash is interpreted to be generally loose to medium dense. Based SPT blow counts the material is classified as medium dense to dense, however, the blow counts are likely overstated due to the number of cobbles and coarse gravel present in this unit. The recessional outwash was observed to a depth of about 25 feet in the borings, and to the maximum depth explored in the test pits, except in test pit TP-14. Significant caving of the sidewalls was observed in most of the test pit excavations in this material.

- **<u>Glacial Till</u>** – Glacial till was encountered in test pit TP-14 below the recessional outwash starting at about 12 feet extending to test pit termination of 12.5 feet below ground surface (bgs). The glacial till consists of silty, sand, gravel. Based on the excavation difficulty the material is interpreted to be dense to very dense.
- **<u>Advance Outwash</u>** – Advance outwash was encountered in the borings below depths approximately 25 feet. The advance outwash generally consists of dense to very dense, sandy, gravel to silty, gravelly, sand with variable amounts of cobbles. Difficult drilling conditions were encountered in the advance outwash with auger refusal in boring BH-2 on two advancement attempts at depths between about 38 to 40 feet.

3.3.2 Groundwater

During our explorations, groundwater was observed at depths between about 35 to 40 feet in the borings. Perched groundwater was observed on top of the glacial till at a depth of about 10 feet in test pit TP-14. No groundwater was encountered in the other test pit explorations extending to depths up to about 14.5 feet bgs. Perforated PVC standpipes were installed in six of the test pits (TP-2, TP-3, TP-5, TP-7, TP-11, and TP-12) with pressure transducers to monitor groundwater conditions. The transducers were set to collect a reading every hour starting on November 16, 2022, and remained dry until the end of the test pit standpipe monitoring in April 2023. Variations in groundwater conditions should be expected to occur seasonally and with changes in precipitation.

A groundwater monitoring well was installed in boring BH-2 to monitor fluctuations in groundwater. The monitoring well was screened from approximately 30 to 40 feet bgs. Pressure transducers were installed to monitor groundwater fluctuation in the wells. Since drilling, groundwater depths in the monitoring well have ranged from about 32 to 36 feet bgs. A plot of groundwater depths from November 21, 2022, through March 23, 2023, obtained from the pressure transducers, are included on Figure 3.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

The subsurface soils along the alignment consist of variable fill, over weathered and undisturbed recessional outwash deposits, underlain by glacial till or advance outwash. The soil conditions are suitable for the proposed trail improvements, provided the recommendations in this report are incorporated into design and construction.

The pedestrian bridge can be supported by drilled shafts extending in the advance outwash below depths of about 25 feet below existing grade. We understand that 3- or 4-foot diameter drilled shafts are being considered. Groundwater has been measured at depths up to about 32 feet below the existing ground surface in boring BH-2, so wet drilling conditions should be expected. The onsite material contains significant amounts of cobbles and occasional boulders, difficult drilling conditions should be anticipated. Construction of the southern bridge abutment and southern SEW will require re-routing of the existing 12-inch water main. We understand that re-routing of the water line will be done by others.

Based on our PIT testing and grain-size testing, infiltration of stormwater appears to feasible along the alignment. The infiltration rates of the underlying recessional outwash are variable due to the variable amounts of silt, sand, gravel, cobbles and boulders in the material. Based on our explorations there appears to be adequate groundwater separation for design of shallow infiltration systems. Perched groundwater was encountered in test pit TP-14 at 10 feet on top of a layer of glacial till. Groundwater was not observed during our other test pit explorations extending to depths between about 9 to 14.5 feet bgs, and the PVC piezometers installed in 6 of the test pits remained dry through our wet weather monitoring which ended in April 2023.

Site work can be completed with conventional excavation and fill methods. The surficial fill and recessional outwash are prone to caving and will require temporary sloping of excavations. The earthwork contractor should be prepared to encounter cobbles and boulders.

The following sections present geotechnical recommendations for design of the bridge, retaining walls, luminaire and flashing beacon pole foundations, infiltration feasibility, a gravel trail section, and general earthwork.

4.2 Seismic Design Considerations

4.2.1 Design Parameters

Earthquake loading for the project was developed in accordance with the General Procedure provided in Section 3.4 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, 2nd Edition, 2011, and the Washington State Department of Transportation (WSDOT) amendments to the AASHTO Guide Specifications provided in the *Bridge Design Manual (LRFD)* (BDM) (WSDOT, 2022). For seismic analysis, the Site Class is required to be established and is determined based on the average soil properties in the upper 100 feet below the ground surface. Based on our subsurface explorations and understanding of site geology, it is our opinion that the site is underlain by soils that are consistent with Site Class D.

The design parameters for the design level event (equal to a return period of 975 years) were obtained from the United States Geologic Society (USGS) Unified Hazard Tool website using the U.S. 2014 Dynamic Conterminous edition (v4.2.0), which provides the probabilistic seismic hazard parameters from the 2014 Updates to the National Hazard Maps (Peterson, et al., 2014). Site coefficients were developed following the WSDOT BDM that adopts the site coefficients provided in American Society of Civil Engineers 7-16 (ASCE, 2017). The recommended seismic coefficients for the design event at site are provided in Table 1.

The spectral acceleration coefficient at 1-second period (S_{D1}) is between 0.3 and 0.5; therefore, Seismic Design Category C, as given by AASHTO Table 3.5-1 (AASHTO, 2011), should be used.

Table 1.
Seismic Coefficients Using AASHTO Guide Specifications
Calculated by USGS Seismic Uniform Hazard Tool
Location: Lat. 47.347245; Long. -122.019725

Site Class	Peak Horizontal Bedrock Acceleration PBA, (g)	Spectral Bedrock Acceleration at 0.2 sec S_s, (g)	Spectral Bedrock Acceleration at 1.0 sec S_1, (g)	Site Coefficients			Peak Horizontal Acceleration PGA (A_s), (g)
				F_{pga}	F_a	F_v	
D	0.374	0.865	0.228	1.226	1.154	2.144	0.459

4.2.2 Liquefaction

Liquefaction is a temporary loss of soil shear strength due to earthquake shaking. Loose, saturated cohesionless soils are highly susceptible to earthquake-induced liquefaction. However, research has shown that certain silts and low-plasticity clays are also susceptible. Primary factors controlling the development of liquefaction include the intensity and duration of strong ground motions, the characteristics of subsurface soils, in-situ stress conditions and the depth to ground water.

Based on our explorations the soil below the groundwater table is dense to very dense and soil liquefaction is not anticipated during a seismic event.

4.3 BRIDGE FOUNDATIONS

We recommend that the bridge abutments be supported on drilled shaft foundations that bear in the underlying advance outwash deposits starting at an elevation of approximately 557 feet at both abutment locations. To support preliminary design, we understand that drilled shaft capacities for 3- or 4-foot diameter shafts were previously provided by Shannon and Wilson based on historical borings completed by WSDOT in 1988 at the existing bridge crossing SR 169 approximately 150 feet to the southeast of the proposed pedestrian bridge, under subcontract with Icicle Creek Engineers, Inc. (Icicle, 2019). The previously completed report is included in Appendix D for reference.

To supplement this existing information and provide specific information at the abutment and shaft locations we completed one boring at each proposed abutment location. Based on our recent explorations at the abutment locations groundwater is slightly higher than anticipated, and very difficult drilling conditions were encountered, particularly at the south abutment where we encountered auger refusal twice at depths between 38 to 40 feet with a Diedrich D-70 track-mounted drill rig.

Based on these recent explorations, 4-foot diameter shafts may have construction benefits, over 3-foot diameter shafts. One benefit is larger diameter shafts may make it easier to remove large cobbles that are anticipated during shaft construction.

4.3.1 Drilled Shaft Axial Capacity

Axial shaft capacities were evaluated using Load and Resistance Factor Design (LRFD) methods in general conformance with the procedures referenced in the FHWA Drilled Shafts Manual (Brown, et al., 2010). Axial shaft capacities will be derived from both shaft friction and end bearing. Similar soil and groundwater conditions were observed in the borings at each abutment location. Nominal axial shaft capacities versus embedment depths for the abutments are presented in Figures 4 and 5 for 3-foot and 4-foot diameter shafts, respectively.

As indicated on these figures, a resistance factor (ϕ) of 0.55 should be applied to the nominal side, or friction, capacities. A resistance factor of 0.50 should be applied to the nominal base resistance for Strength I Limit State design. For the Extreme I and the Service I Limit States, the resistance factor (ϕ) should be 1.0 for both shaft friction and end bearing.

For the Service I Limit state, total shaft resistance (i.e., friction plus end bearing) is provided for an allowable settlement of 1 inch. If a Service I Limit State capacity for a different settlement value (e.g. 2 inches or ½ inch) is needed, we should be contacted to revise our calculations.

Additionally, we recommend that the shaft be spaced no closer than 3 shaft diameters to avoid excessive reductions in vertical capacity due to group effects. Based on the bridge design and shaft layout in the 60% plans, group reductions are not anticipated to be necessary.

4.3.2 Drilled Shaft Lateral Design Parameters

Lateral loads may be resisted by the passive earth pressure against deep foundations and foundation caps. The magnitude of lateral resistance developed by drilled shafts depends on the subsurface conditions encountered and the moment capacity at the foundation cap connection. We recommend ignoring the friction sliding resistance at the base of the foundation cap, because a deep foundation-supported cap may not transmit load directly to the soil beneath it.

We understand that the design team desire to use a conventional p-y method of lateral analysis (i.e., LPILE) to estimate shears, moments, and deflections of the shafts. Soil parameters for use in LPILE analyses are provided in Table 2. Liquefaction potential is considered low; therefore, the values provided are for both static and seismic conditions.

Table 2. LPILE Parameters for Lateral Pile Loading

Soil Layer	Soil Type (p-y model)	Depths of Layer (feet)	Effective Unit Wt, γ' (pcf)	Friction Angle (degrees)	p-y Modulus Static, k (pci)
Existing Fill	Sand (Reese)	0 to 12	125	31	40
Recessional Outwash	Sand (Reese)	12 to 25	130	35	140
Advance Outwash	Sand (Reese)	25 to 30	130	37	140
Advance Outwash	Sand (Reese)	30 to 45	65*	37	85
Very Dense Advance Outwash	Sand (Reese)	45 to 60	65*	40	125

Notes: pcf = pounds per cubic foot
psf = pounds per square foot
pci = pounds per cubic inch
*Below Groundwater Table

The p-y curves generated by the lateral parameters provided in Table 2 must be modified by the applicable p-multipliers to account for group reduction effects. The p-multipliers for a shaft spacing of 4.33 shaft diameters (3-foot diameter shafts on 13-foot spacing) are provided in Table 3. The p-multipliers for a shaft spacing of 3.25 shaft diameters (4-foot diameter shafts on 13-foot spacing) are provided in Table 4.

Table 3.
P-Multipliers for Center-to-Center Spacing of 4.33 Shaft Diameters

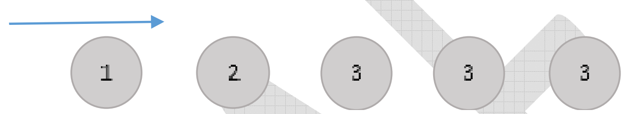
Row	P-Multiplier
1	0.93
2	0.70
3 or more	0.57

Table 4.
P-Multipliers for Center-to-Center Spacing of 3.25 Shaft Diameters

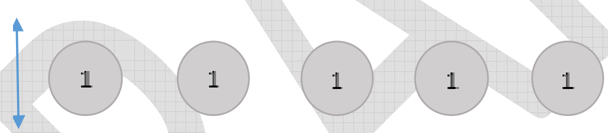
Row	P-Multiplier
1	0.83
2	0.46
3 or more	0.35

The same p-multiplier factor should be applied parallel and perpendicular to the group shaft alignment. The following diagram shows how the p-multipliers should be assigned with respect to the load direction and shaft orientation.

Parallel Direction



Perpendicular Direction



Based on our analysis, to achieve fixity of the proposed shafts we recommend minimum shaft lengths of 30 and 40 feet for 3-foot and 4-foot diameter shafts, respectively.

4.3.3 Drilled Shaft Construction Considerations

The drilled shafts will be drilled through relatively dense layers of soil containing large cobbles and boulders. The drilled shaft contractor should be prepared to encounter and handle cobbles and boulders, this may require rock coring using a core barrel. The contractor should be prepared to drill for extended periods of time to advance through particularly dense layers or obstructions.

Groundwater was recently measured at depths between about 32 to 36 feet below the ground surface in the monitoring well in boring BH-2 (south abutment). The high groundwater table is anticipated to occur in late winter/early spring.

The contractor should be prepared to construct the shafts below the groundwater level and provide appropriate methods for stabilizing the sides and bottom of the shaft excavations. We anticipate that temporary casing will likely be necessary during shaft excavation. Soils excavated from the shafts will likely be saturated and could require decanting prior to being transported off-site. The contractor should be prepared to undertake decanting of the soil excavated from the drilled shafts.

Drilled shaft bottoms should be cleaned to the extent practical using appropriate excavation methods to provide for a relatively undisturbed shaft base. After the shaft bottoms are cleaned, concrete should be placed by the tremie method into the shafts. If temporary casing is used it should be withdrawn such that the level of concrete is maintained above the bottom of the casing at all times and at such elevations to counteract any potential hydrostatic effects associated with groundwater conditions that may be present at the location of the work.

4.4 RETAINING WALLS

We understand that Structural Earth Walls (SEWs) will be utilized to keep the embankment fill from extending into the BNSF right-of-way. Based on the 60 percent bridge design plans this will require an SEW approximately 38 feet in length to the west of the northern bridge abutment, and an SEW approximately 21 feet in length to the east of the southern bridge abutment. The SEWs to support cuts into the trail embankment construction are anticipated to vary in height up to about 11 feet tall.

In addition, we anticipate that retaining walls up to 4 to 5 feet in height may be necessary to support cuts and fills along the trail alignment in order to meet ADA requirements. We anticipate walls along the trail will consist of gravity block or SEW Walls.

4.4.1 Wall Design Parameters

We assume that gravity block or SEW walls will consist of a proprietary wall system that the wall supplier will design for internal stability. The walls should be designed in accordance with the most current version of the AASHTO *LRFD Bridge Design Manual* and Section 6.13 of the WSDOT *Standard Specifications* (WSDOT, 2023). We recommend that the walls be designed using the parameters presented in Table 5. We understand that the design for these walls will be performed using LRFD. Appropriate AASHTO resistance factors should be used for design of all retaining walls.

For the Extreme Event I Limit State, the walls shall be designed for a horizontal seismic acceleration coefficient K_h of one-half the peak ground acceleration or 0.23g and vertical seismic coefficient Kv of 0.0g (assuming the wall is free to move during a seismic event). Extreme Event I Limit State is defined in the AASHTO Standard Specifications as a safety check involving an extreme load even resulting from an earthquake in combination with the dead load and a fraction of the live loads.

Table 5. Recommended Design Parameters for SEW Walls

Soil Properties	Reinforced soil	Retained Soil	Foundation Soil
Unit Weight (pcf)	135	125	130
Friction Angle (deg)	36	32	36
Cohesion (psf)	0	0	0
		Strength Limit State (EP+LL)	**Extreme Limit State (EP+EQ)**
Ultimate Bearing Resistance (ksf)		6.0	6.0
Horizontal Seismic Acceleration Coefficient (k_h) (g)		N/A	0.23

To satisfy global wall stability requirements we recommend that walls over 5 feet in height be embedded at least 2 feet below existing grades; walls 5 feet or less in height should be embedded at least 1 foot below existing grades. These minimum embedment depths assume grades in front of the walls of up to 3H:1V. For walls with grades in front of the wall of up to 2H:1V we recommend a minimum embedment of at least 2 feet below existing grades.

It is important that the walls be designed per specific toe- and back-slope geometry at each wall location. Additionally, vertical, and lateral dead loads such as pavement, guard rails, and chain-link fences, and live loads such as vehicular, pedestrian, construction equipment loading should be considered in design of each retaining wall. An unfactored coefficient of friction of 0.5 times the effective stress at the base of the wall can be used for sliding resistance.

The onsite soils are coarse grained and not expected to undergo consolidation settlement due to construction of retaining walls. If the wall subgrade is prepared as recommended in Section 4.4.2, the total wall settlement is not expected to exceed 1 inch. For the Service Limit State, the wall should be designed to accommodate differential settlement of up to ¾ inch per 100 feet of wall length. Most of the wall settlement is expected to occur during construction, as the loads are applied.

4.4.2 Subgrade Preparation

Subgrade preparation is important to limit differential settlement of walls and maintain stability. All organic material should be removed from beneath the entire footprint of walls. The exposed subgrade should be inspected by the geotechnical engineer, or their representative, and any loose or unsuitable soils should be over-excavated as directed by the engineer or inspector on site.

Once the subgrade has been approved, the walls should be founded on a leveling pad consisting of compacted Crushed Surfacing Base Course (CSBC), as described in Section 9-03.9(3) of the

WSDOT *Standard Specifications* (WSDOT, 2023), compacted to at least 95% of the laboratory maximum dry density as determined by ASTM D 1557. Leveling pads should be graded to establish proper wall batter. We recommend that the leveling pads for the SEW wing walls near the proposed bridge crossing be at least 12 inches thick. Leveling pads for SEW or gravity block walls along the trail with heights less than about 5 feet should be a minimum of 6 inches thick.

4.4.3 Wall Backfill

Wall backfill materials should consist of Gravel Backfill for Walls, as described in Section 9-03.12(2) of the WSDOT *Standard Specifications* (WSDOT, 2023) and should be compacted to at least 95% of the maximum dry density as determined by ASTM D 1557 (Modified Proctor). The wall backfill should be placed and compacted in layers as each row of blocks is placed.

The Contractor should consider the weight of construction equipment operating within the fill zone behind the wall. For compaction, materials within about 3 feet of the wall face should be compacted with lighter equipment to limit the loading on the back of the wall.

4.4.4 Wall Drainage

Drainage should be provided behind all walls to prevent buildup of hydrostatic pressures and should consist of a 4- to 6-inch diameter, perforated, rigid plastic pipe, bedded and backfilled with Gravel Backfill for Drains, as specified in Section 9-03.12(4) of the WSDOT *Standard Specifications* (WSDOT, 2023). The drain rock should surround the drainpipe by at least 6 inches. The pipes should slope to drain to a suitable outlet.

4.4.5 Back-to-back Walls

We understand that construction of back-to-back SEW walls may be necessary to achieve grade requirements as the trail approaches SE 288th Street. Section 4.3 in Chapter 15 of the WSDOT *Geotechnical Design Manual* (WSDOT, 2022) provides guidance for back-to-back SEW walls. Per Section 4.3 in Chapter 15 of the WSDOT *Geotechnical Design Manual*, preapproved propriety wall systems may be used for back-to-back SEW walls provided that they have a height to width ratio of 1.1 or greater, and provided grid overlap requirements are met. Based on the proposed trail width, we anticipate that wall faces will be approximately 16 feet apart, which will result in a width to height ratio greater than 1.1 for the wall proposed wall heights of up to 11 feet. The *Geotechnical Design Manual* indicates reinforcement may overlap, provided the reinforcement from one wall does not contact the reinforcement for the other wall.

4.5 EMBANKMENT SLOPES

We recommend that compacted fill slopes or bank slopes that may be needed for the project be constructed/restored no steeper than 2H:1V (Horizontal:Vertical). For fill slopes constructed at

2H:1V or flatter, and comprised of fill soils placed and compacted as structural fill as described in Section 4.8.3 of this report, we anticipate that adequate factors of safety against global failure will be maintained. Measures should be taken to prevent surficial instability and/or erosion of embankment material. This can be accomplished by conscientious compaction of the embankment fills in level lifts, benched cuts into the slope face, maintaining adequate drainage, and planting the disturbed slope face with vegetation as soon as possible after construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to finish grade. In our experience, compaction of slope faces by "track-walking" is generally ineffective and is, therefore, not recommended.

4.6 EVALUATION OF INFILTRATION POTENTIAL OF SITE SOILS

4.6.1 Feasibility of Using Infiltration

We understand there is a desire to infiltrate stormwater as part of the project improvements, if feasible. We understand that stormwater Best Management Practices (BMPs) for the project will be evaluated in accordance with the King County Surface Water Design Manual (KCSWDM).

Groundwater was encountered in our borings at depths between 35 and 40 feet during drilling and have subsequently been measured in the monitoring well at depths up to about 32 feet bgs. Perched groundwater was encountered at about 10 feet bgs in test pit TP-14 on a restrictive layer of glacial till starting at about 12 feet bgs. A restrictive fine grained silt layer was encountered in TP-6 between approximately 8.5 to 9 feet below existing grade. In PIT-5 some red mottling and organic material was encountered within the gravelly outwash deposits between about 7 to 7.5 feet bgs, which will have lower permeability than the cleaner outwash deposits above and below. In TP-9 a layer of buried topsoil was encountered between 4.5 to 5 feet bgs that is less permeable than the fill above and recessional outwash deposits below. The rest of the test pits generally encountered permeable sand and gravel with variable amounts of cobbles and boulders to the maximum depths explored of between 9 and 14.5 feet bgs.

Per the KCSWDM, infiltration facilities require at least 5 feet of permeable soil below the bottom of the infiltration facility and at least 5 feet between the bottom of the facility and the maximum wet-season water table. This may be decreased to 3 feet of permeable soil below the bottom of the infiltration facility and at least 3 feet between the bottom of the facility and the maximum wet-season water table with additional testing and performance of mounding analyses.

Based on correspondence with the stormwater design engineers, we understand that shallow infiltration trenches and sheet flow dispersion will be used to manage stormwater for the project. Based on our explorations and infiltration testing, infiltration trenches or other shallow infiltration facilities are feasible. The KCSWDM indicates that soil infiltration rates should be determined by Pilot Infiltration Tests (PITs).

4.6.2 Infiltration Rates

HWA performed eight small-scale PITs in general accordance with the KCSWDM and the SWMMWW, as described in Section 2.3.1 of this report. As discussed, due to the high permeability of the soil, the water truck ran out of water (about 3,000 gallons) before a full 6-hour pre-soak could be completed, and at some of the test locations we were unable to establish a constant head of 1 foot of water. For each test, the falling head rates were less than the constant head rates. Based on the test results, the measure infiltration rates from the falling head tests are shown in Table 6.

The KCSWDM recommends correction factors be applied to the measured infiltration rate to obtain the design infiltration rate. The KCSWD recommends the following correction factors be applied to obtain the design infiltration rate:

$$I_{design} = I_{measured} \text{ X } F_{testing} \text{ X } F_{geometry} \text{ X } F_{plugging}$$

For small-scale PITs, the KCSWDM recommends a correction factor $F_{testing}$ of 0.5 to account for uncertainties in the testing method.

A shallow water table or impervious layer will also reduce the effective short term infiltration rate. To account for the influence of facility geometry and depth of the water table or impervious strata on the actual infiltration capacity a correction factor, $F_{geometry,}$ must be between 0.25 and 1.0 as determined by the following equation:

$$F_{geometry} = 4 \text{ D/W} + 0.05$$

Where, D is the depth from the bottom of the proposed facility to the maximum wet-season water table or nearest impervious layer (whichever is less), and W is the width of the facility. Based on our explorations and infiltration testing, we anticipate that infiltration facilities will generally have a depth to wide ratio greater than 0.25, resulting a $F_{geometry}$ correction factor of near 1. For preliminary design calculations we used a $F_{geometry}$ factor of 1.0. This will need to be confirmed once the dimensions and depths of the proposed facilities are determined.

A correction factor to account for reduction in infiltration rates over the long term due to soil plugging is also required in the KCSWDM. We conservatively recommend a $F_{plugging}$ correction factor of 0.9 for medium sands based on our explorations.

After applying the correction factors, the calculated design infiltration rates ranged from 4 to 103 inches per hour, per the KCSWDM the recommended maximum design rate cannot exceed 20 inches per hour. Therefore, we recommend using 20 inches per hour, except near PIT-5, where a maximum rate of about 4 inches per hour was calculated. Calculated and recommended design rate are shown in Table 6.

Table 6. PIT Infiltration Rates

	Measured Infiltration Rate (in/hr)	Calculated Design Infiltration Rate (in/hr)	Recommended Design Infiltration Rate (in/hr)
PIT-1	54	24	20
PIT-2	57	25	20
PIT-3	176	79	20
PIT-4	69	31	20
PIT-5	10	4	4
PIT-6	144	64	20
PIT-7	192	86	20
PIT-8	230	103	20

4.6.3 Subgrade Preparation for Infiltration Facilities

Prior to installation of infiltration facilities, the subgrade should be cut to the base of the infiltration facility. Once the soil is cut to the base of the facility, the exposed soils should be verified by the geotechnical engineer, or their representative, to confirm that they are similar to materials tested for the infiltration analyses. Given the variability of site soils, the depth of the receptor soil may differ across the site. The existing subgrade under areas used for infiltration **should not** be compacted or subjected to excessive construction equipment traffic prior to installation. Where erosion of subgrade occurs during construction and has caused accumulation of fine materials and/or surface ponding, this material shall be removed with light equipment and the underlying soils scarified to a minimum depth of 8 inches. Once prepared, the geotechnical engineer should inspect the subgrade to verify that it is suitable to provide the recommended infiltration rates.

4.6.4 Soil Suitability for Water Quality Treatment

Laboratory testing to evaluate Cation Exchange Capacity (CEC) and organic content of site soils was originally planned to be conducted on samples obtained from the PIT testing. Based on correspondence with the design team, we understand that CEC and organic content testing will not be completed, because water quality treatment is not required for this project.

4.7 Luminaire and Flashing Beacon Foundations

We anticipate that the improvements for the safe at grade trail crossings at SE Tahoma Way, SE 276th Street, SE 280th Street and SE 288th Street will include new lighting and flashing beacons for pedestrian crossings.

Based on our test explorations near the crossing streets (TP-2, TP-4, TP-6, and TP-13) soil conditions are anticipated to consist of some surficial fill over recessional outwash. The recessional outwash contains significant cobbles and boulders, and the material is susceptible to caving. The large material and low fines content will likely result in larger excavations than is typical for these types of pole foundation elements.

4.7.1 Pole Foundation Recommendations

Based on our explorations, luminaire or flashing beacon foundations can be designed based on an average allowable lateral bearing pressure for the upper 8 feet of 2,000 psf based on the Table 17-2 of the WSDOT Geotechnical Design Manual (WSDOT, 2022).

Pole foundations can also be designed using Brom's method recommended in the *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (AASHTO, 2013). Table 7 provides recommended design parameters to be used for non-standard foundation design.

Table 7: Design Parameters for Pole Foundations

Φ (deg)	Kp	Moist Unit Weight (pcf)	Buoyant Unit Weight (pcf)
30	3.0	120	45.6

The above soil parameters are ultimate values, we recommend that a factor of safety of at least 1.5 be applied to the values.

4.7.2 Pole Foundation Construction Considerations

The gravel with variable amounts of sand, cobbles, and boulders encountered in our explorations will be prone to caving. Groundwater was not encountered in our test pit explorations conducted near the intersection improvements to the maximum depth explored of 14.5 feet.

If pole foundations are constructed using drilled shafts, the contractor should be prepared to handle cobbles and boulders, this may require rock coring using a core barrel. Drilled shaft bottoms should be cleaned to the extent practical using appropriate methods. If more than 12 inches of water are present in the shaft, concrete should be placed by the tremie method into the shafts. Temporary casing will likely be necessary during shaft excavation. Temporary casing should be withdrawn such that the level of concrete is maintained above the bottom of the casing at all times and at such elevations to counteract any potential hydrostatic effects associated with ground water conditions that may be present at the location of the work.

All luminaire or flashing beacon foundation locations should also be evaluated to confirm that the proposed excavations do not conflict with existing utilities.

4.8 TRAIL SECTION

We understand that as part of this project the trail will be graded to meet ADA requirements and surfaced with crushed ledge rock. Paving of the trail is not planned as part of this project, however, there is a desire to pave this section of trail in the future. For this project, we understand there is a desire to install an economical, yet robust gravel base section, that can be paved in the future with minimal maintenance and base reconstruction.

Based on correspondence with the design team we understand that anticipated vehicle traffic on the trail will be minimal. Traffic will likely consist of a pick-up truck about once a week, a dump truck with a trailer for trail maintenance about once a month, and occasional emergency vehicles as necessary.

Since the trail surface will be left as gravel as part of this project, we understand that the crushed surfacing top course will consist of ledge rock. Ledge rock should meet the requirements in Section 9-03.9(3) for crushed surfacing top course (CSTC) of the WSDOT *Standard Specifications* (WSDOT, 2023), with the exception that the material is 100 percent fractured.

Based on our explorations, and anticipated traffic loading, we recommend a minimum gravel section of 3 inches of crushed ledge rock over 5 inches of crushed surfacing base course (CSBC). CSBC should meet requirements described in Section 9-03.9(3) of the WSDOT *Standard Specifications* (WSDOT, 2023). The trail section should be compacted to at least 95% of the maximum dry density (MDD) as determined by ASTM D1557 (Modified Proctor).

4.9 GENERAL EARTHWORK

We understand that the trail will be graded to meet ADA requirements and surfaced with crushed rock. Based on the gravelly and cobbly nature of the materials observed at the site, the Contractor should be prepared to encounter cobbles and boulders within the existing fill, weathered material, and recessional outwash.

4.9.1 Temporary Slopes and Excavations

Temporary slopes and excavations will be needed to install the bridge foundations, retaining walls, pole foundations, and for installation of infiltration facilities. The onsite soils will be prone to sloughing and raveling.

Maintenance of safe working conditions, including temporary excavation stability is the responsibility of the contractor. In accordance with Part N of Washington Administrative Code (WAC) 296-155, all temporary cuts more than 4 feet in height must be either sloped or shored prior to entry by personnel. The existing fill and recessional outwash are generally classified as Type C soils per WAC 296-155. Where shoring is not used, temporary cuts in Type C soils should be sloped no steeper than 1.5H:1V (horizontal: vertical); however, if ground water seepage is observed on cut slopes, shallower inclinations may be needed to maintain safe working conditions.

4.9.2 Trail Subgrade Preparation

In areas being cleared or widened to accommodate the trail section, subgrade preparation should begin with the removal of all topsoil, deleterious material, and vegetation. Using a smooth bucket, the soils should be excavated to the proposed subgrade elevation. The exposed subgrade should be inspected by the Geotechnical Engineer, or their representative, and any loose or unsuitable soils should be over-excavated and replaced with properly compacted structural fill.

4.9.3 Structural Fill Materials and Compaction

Based on our subsurface explorations, we anticipate that some of the onsite fill and recessional outwash soils will be suitable for reuse as structural fill to raise grades provided certain requirements are met. Oversize material (greater than 4 inches) should be removed from materials to be used as structural fill as well as any organic or other deleterious materials. Use of onsite materials should be evaluated by the geotechnical engineer. Moisture conditioning may be required to ensure that soils to be used as structural fill are not too wet or too dry for proper compaction.

Structural fill soils should be moisture conditioned and compacted to the requirements specified in Section 2-03.3(14)C, Method C, of the WSDOT *Standard Specifications* (WSDOT, 2023); except the standard of compaction achieved shall not be less than 95% of the maximum dry density (MDD) determined for the fill material by test method ASTM D1557 (Modified Proctor). Subgrade compaction of the trail should conform to the requirements of Section 2 06.3(1) of the WSDOT *Standard Specifications* (WSDOT 2023).

Imported structural fill for trail base should consist of Crushed Surfacing Base Course (CSBC) and Crushed Surfacing Top Course Ledge Rock (CSTC), as described in Section 9-03.9(3) of the WSDOT *Standard Specifications* (WSDOT, 2023). Note that where the subgrade will be part of an infiltration facility, the materials should NOT be compacted, as this will reduce the infiltration rates of the receptor soils.

Structural fill should be placed in loose, horizontal, lifts of not more than 8 inches in thickness and at the time of placement, the moisture content of structural fill should be at or near optimum. The procedure required to achieve the specified minimum relative compaction depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and the soil moisture-density properties.

When the first fill is placed in a given area, and/or anytime the fill material or compaction equipment changes, the area should be considered a test section. The test section should be used to establish fill placement and compaction procedures required to achieve proper compaction. The geotechnical consultant should observe placement and compaction of the test section to assist in establishing an appropriate compaction procedure. Once a placement and compaction procedure are established, the contractor's operations should be monitored, and periodic density tests performed to document that proper compaction is being achieved.

Generally, loosely compacted soils result from poor construction technique or improper moisture content. Soils with a high percentage of silt or clay content are particularly susceptible to becoming too wet, and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried, as necessary, or moisture conditioned by mixing with drier materials, or other treatment methods. For coarse-grained structural fill soils, moisture conditioning by sprinkling before and during compaction is sometimes required to achieve the required relative compaction.

4.9.4 Wet Weather Earthwork

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. These recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance that may be caused by equipment traffic.

- For wet weather conditions, material used as structural fill should consist of clean granular soil with less than 5 percent passing the U.S. Standard No. 200 sieve, based on wet sieving the fraction passing the ¾-inch sieve. The fine-grained portion of the structural fill soils should be non-plastic. It should be noted that this is an additional restriction on the structural fill materials specified.

- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- The ground surface within the construction area should be sealed on completion of each shift by a smooth drum vibratory roller, or equivalent, and under no circumstances should soil be left uncompacted and exposed to moisture.
- Bales of straw and/or geotextile silt fences should be strategically located to control erosion and the movement of soil.

5.0 CONDITIONS AND LIMITATIONS

We have prepared this report for Parametrix and King County for use in design of portions of this project. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as our warranty of the subsurface conditions. Experience has shown that soil and ground water conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations and may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified for review of the recommendations of this report, and revision of such if necessary.

We recommend HWA be retained to review the plans and specifications to verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing, and consultation should be provided during construction to confirm the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in the area at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or ground water at this site.

HWA does not practice or consult in the field of safety engineering. We do not direct the contractor's operations and cannot be responsible for the safety of personnel other than our own on the site. As such, the safety of others is the responsibility of the contractor(s). The contractor(s) should notify the owner if it is considered that any of the recommended actions presented herein are unsafe.

We appreciate this opportunity to be of service.

Sincerely,

HWA GEOSCIENCES, INC.

Joe Westergreen, P.E.
Geotechnical Engineer

JoLyn Gillie, P.E.
Geotechnical Engineer, Principal

6.0 REFERENCES

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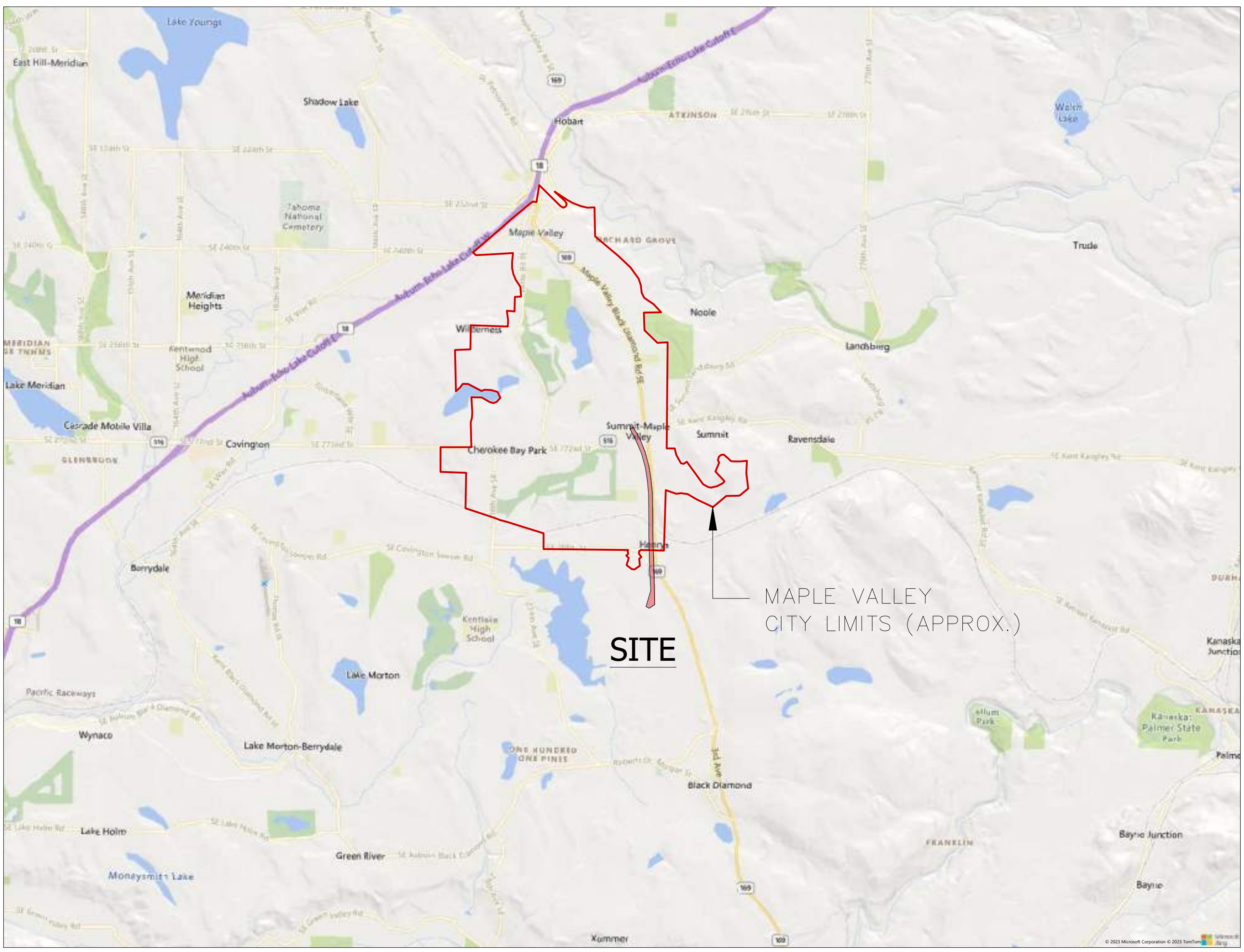
SITE MAP

0 1000' 2000' 3000' 4000'

SCALE: 1" = 2000'

N



VICINITY MAP

0 5000' 10000' 15000' 20000'

SCALE: 1" = 10000'

N

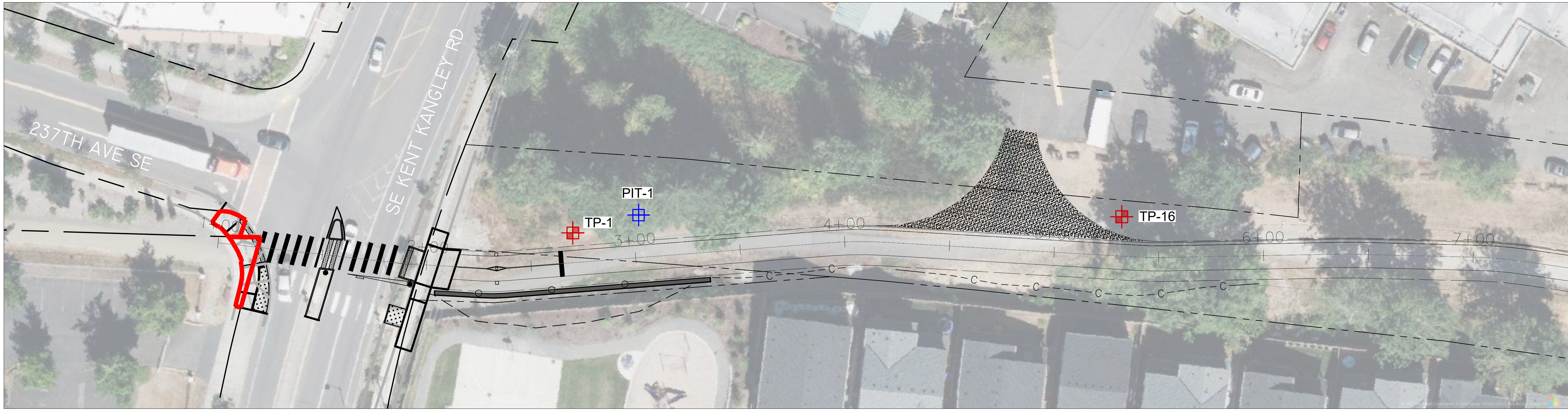
HWA GEOSCIENCES INC.
DBE/MWBE

SITE AND VICINITY MAP

GREEN TO CEDAR RIVERS REGIONAL TRAIL
SOUTH SEGMENT
MAPLE VALLEY, WASHINGTON

FIGURE NO.: 1	
DRAWN BY: CF	CHECK BY: MAB/JTW
PROJECT # 2021-163-21	

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <1> Plotted: 5/25/2023 5:26 PM



SE KENT KANGLEY RD
Scale: 1" = 50'-0"

SE 271ST PL
Scale: 1" = 50'-0"

0 25 50 75 100
SCALE: 1" = 50'

EXPLORATION LEGEND

PIT-1 SMALL SCALE PIT DESIGNATION AND APPROXIMATE LOCATION

TP-1 TEST PIT DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY: BING

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <2A> Plotted: 5/5/2023 3:37 PM

HWA GEOSCIENCES INC. DBE/MWBE	GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT MAPLE VALLEY, WASHINGTON	SITE & EXPLORATION PLAN	DRAWN BY: CF	FIGURE NO.: 2A
			CHECK BY: MAB/JTW	PROJECT NO.: 2021-163-21

TRAIL
Scale: 1" = 50'-0"

SE 276TH ST
Scale: 1" = 50'-0"

EXPLORATION LEGEND

PIT-3 SMALL SCALE PIT DESIGNATION AND APPROXIMATE LOCATION

TP-3 TEST PIT DESIGNATION AND APPROXIMATE LOCATION

0 25 50 75 100
SCALE: 1" = 50'

BASE MAP PROVIDED BY: BING

HWA GEOSCIENCES INC. DBE/MWBE

GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT MAPLE VALLEY, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY: CF	FIGURE NO.: 2B
CHECK BY: MAB/JTW	PROJECT NO.: 2021-163-21

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <2B> Plotted: 5/5/2023 3:38 PM

TRAIL
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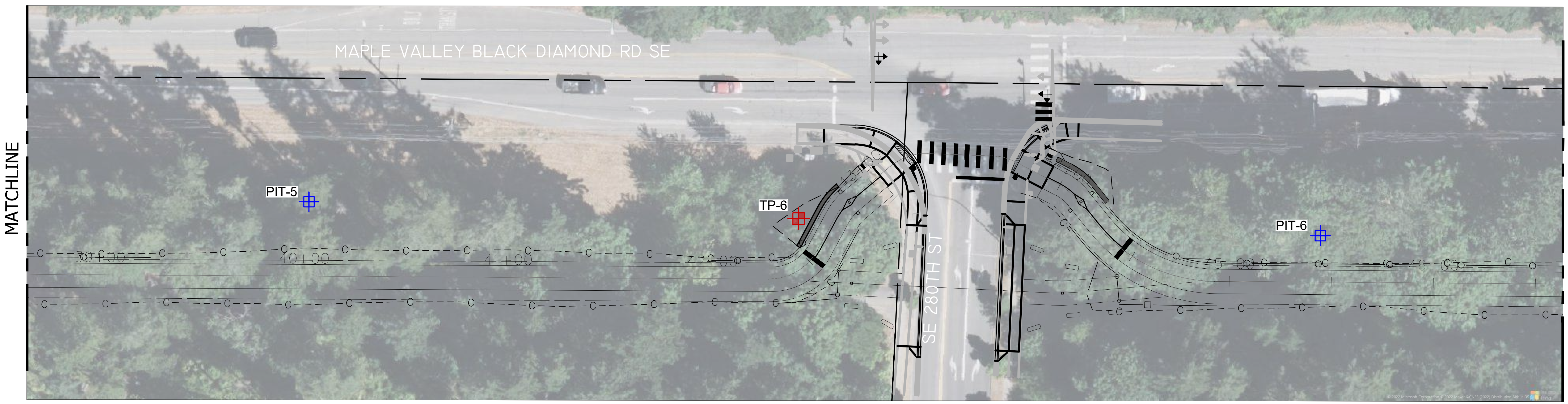


SE 276TH ST
Scale: 1" = 50'-0"

0 25 50 75 100
SCALE: 1" = 50'

EXPLORATION LEGEND

PIT-4 SMALL SCALE PIT DESIGNATION AND APPROXIMATE LOCATION

TP-5 TEST PIT DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY: BING

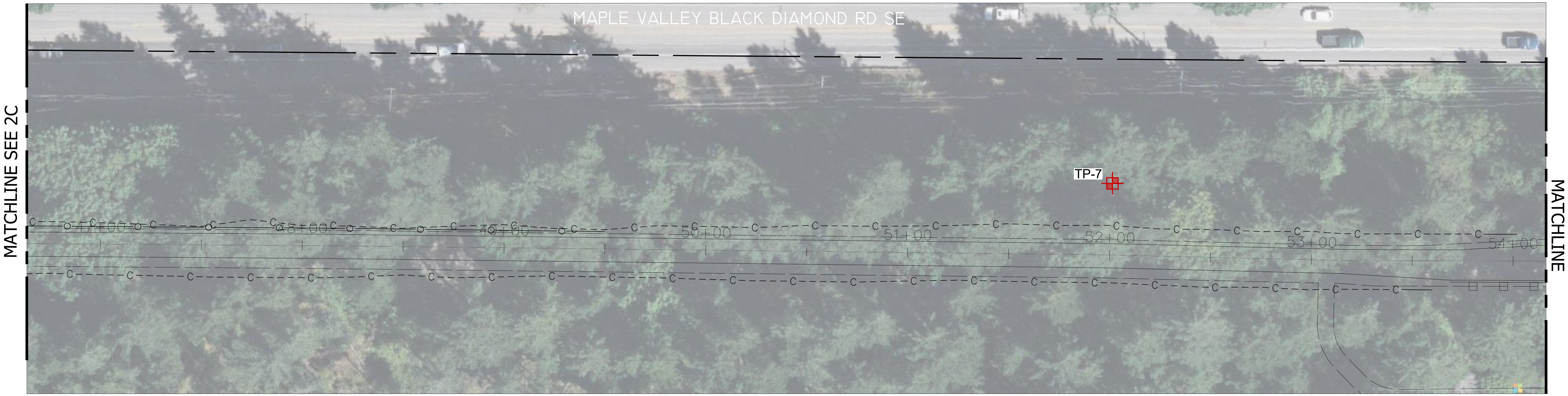
HWA GEOSCIENCES INC. DBE/MWBE

GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT
MAPLE VALLEY, WASHINGTON

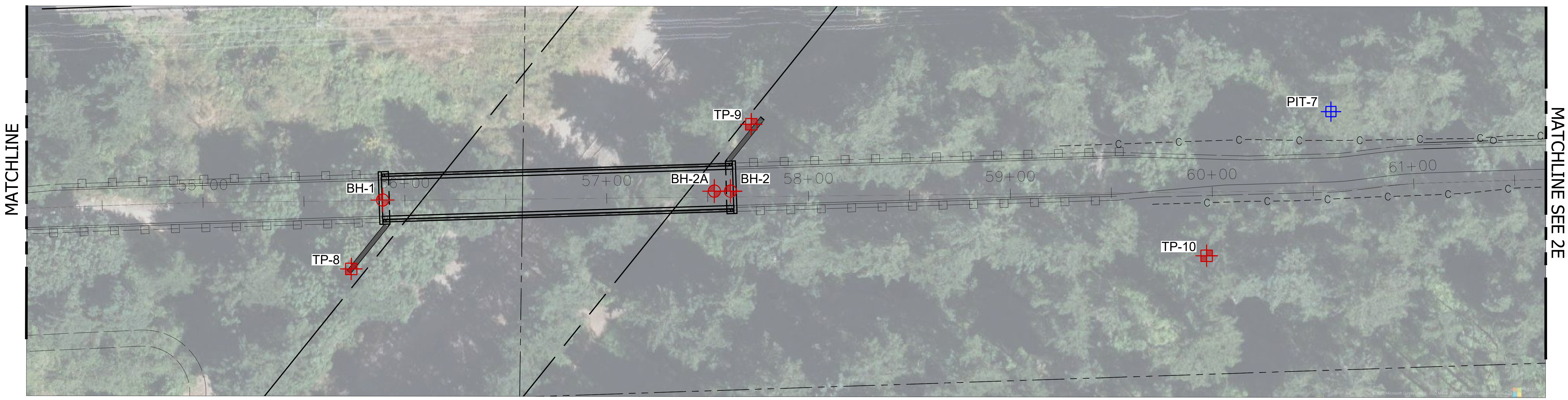
SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2C
CHECK BY:	PROJECT NO.:
MAB/JTW	2021-163-21

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <2C> Plotted: 5/5/2023 3:39 PM



MAPLE VALLEY BLACK DIAMOND RD SE
Scale: 1" = 50'-0"



TRAIL
Scale: 1" = 50'-0"

0 25 50 75 100
SCALE: 1" = 50'

EXPLORATION LEGEND

- PIT-7 SMALL SCALE PIT DESIGNATION AND APPROXIMATE LOCATION
- TP-7 TEST PIT DESIGNATION AND APPROXIMATE LOCATION
- BH-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY: BING

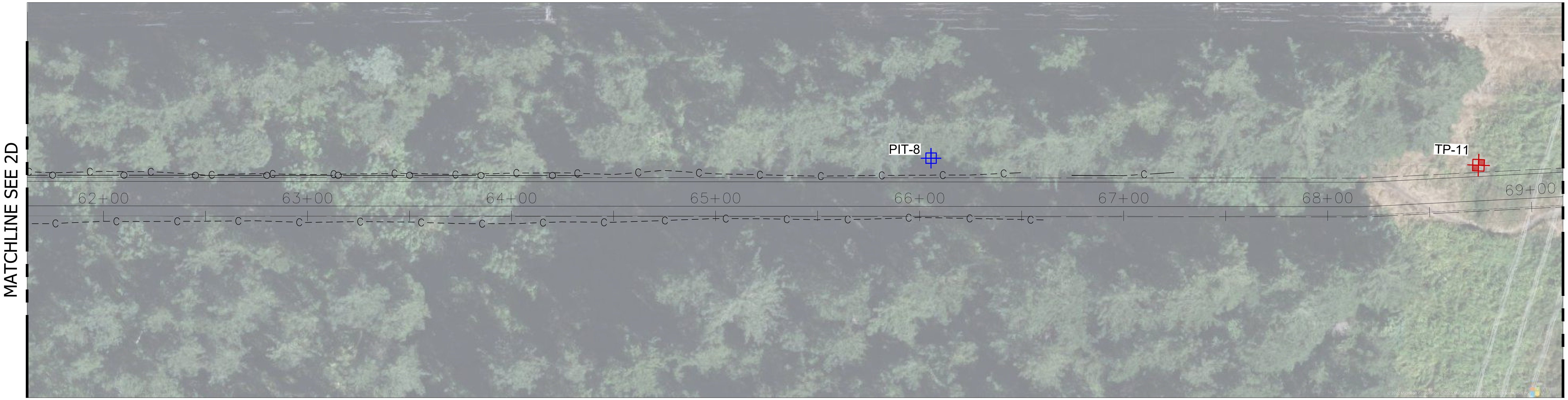
HWA GEOSCIENCES INC.
DBE/MWBE

GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT
MAPLE VALLEY, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2D
CHECK BY:	PROJECT NO.:
MAB/JTW	2021-163-21

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <2D> Plotted: 5/5/2023 3:39 PM



TRAIL
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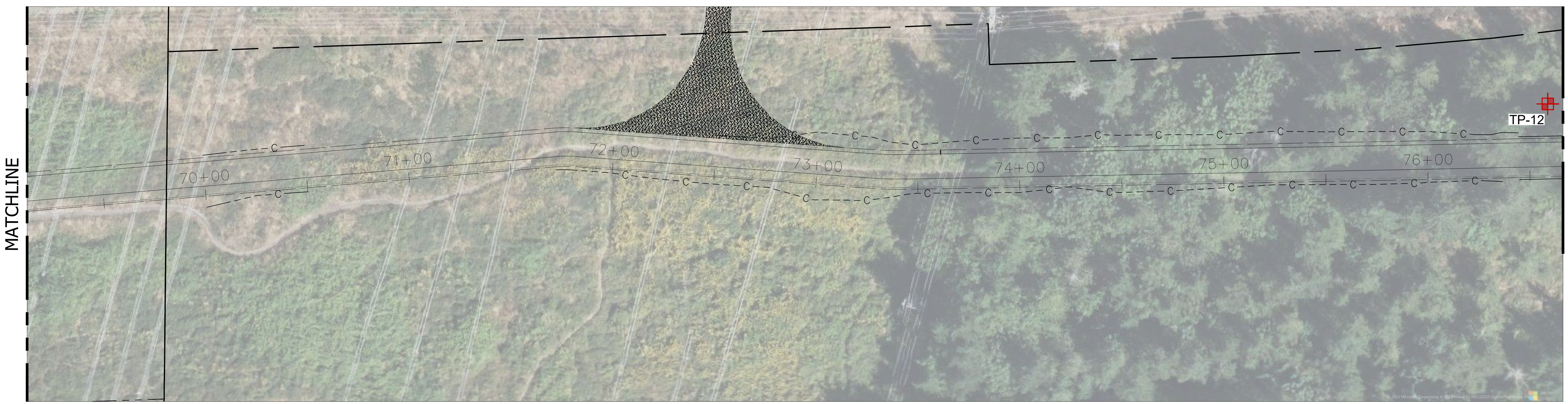

TRAIL
Scale: 1" = 50'-0"

EXPLORATION LEGEND

PIT-8 PIT DESIGNATION AND APPROXIMATE LOCATION

TP-11 TEST PIT DESIGNATION AND APPROXIMATE LOCATION

0 25 50 75 100
SCALE: 1" = 50'

BASE MAP PROVIDED BY: BING

HWA GEOSCIENCES INC. DBE/MWBE

GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT
MAPLE VALLEY, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2E
CHECK BY:	PROJECT NO.:
MAB/JTW	2021-163-21

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <2E> Plotted: 5/5/2023 3:40 PM

SE 288TH ST
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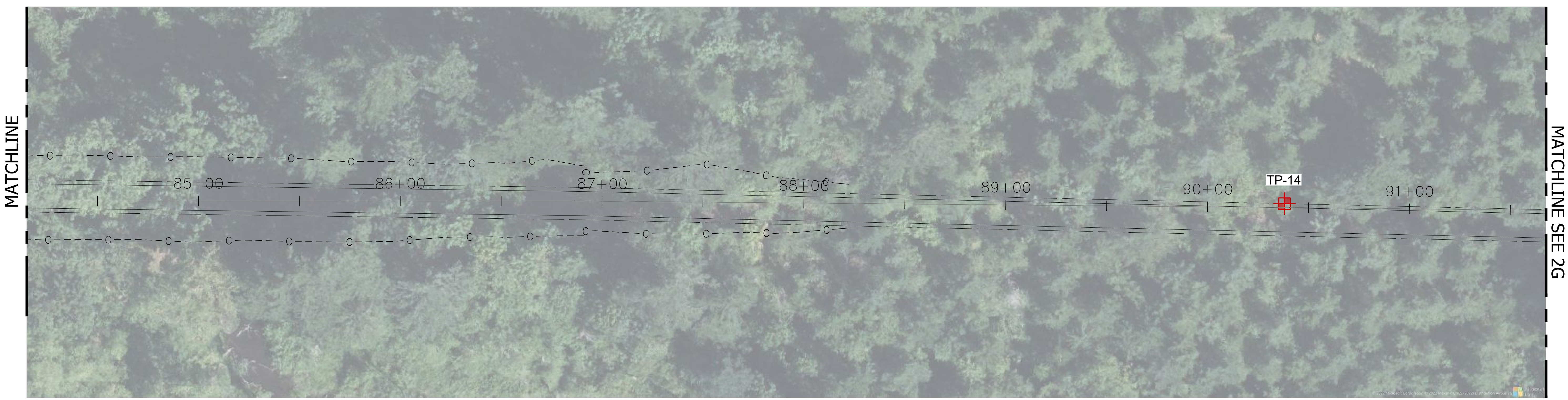


TRAIL
Scale: 1" = 50'-0"

EXPLORATION LEGEND

TP-13 TEST PIT DESIGNATION AND APPROXIMATE LOCATION

0 25 50 75 100
SCALE: 1" = 50'

HWA GEOSCIENCES INC.
DBE/MWBE

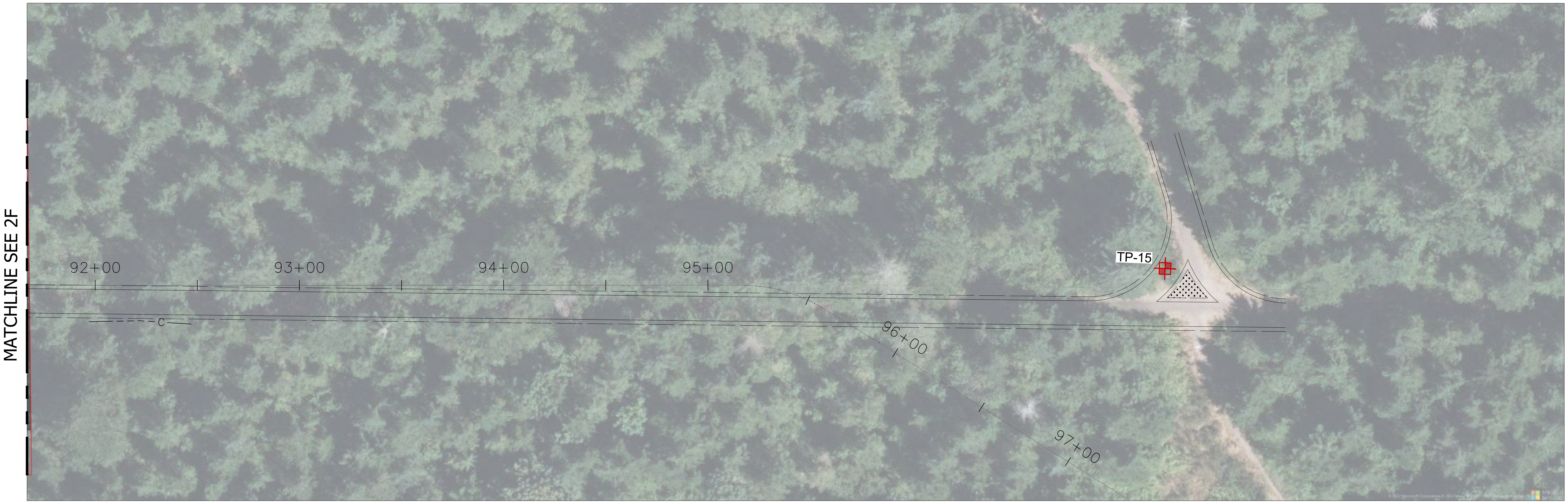
GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT
MAPLE VALLEY, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2F
CHECK BY: MAB/JTW	PROJECT NO.: 2021-163-21

BASE MAP PROVIDED BY: BING

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <2F> Plotted: 1/26/2023 11:05 AM



SE 288TH ST

Scale: 1" = 50'-0"

EXPLORATION LEGEND

TP-16 TEST PIT DESIGNATION AND APPROXIMATE LOCATION

0 25 50 75 100

SCALE: 1" = 50'

BASE MAP PROVIDED BY: BING

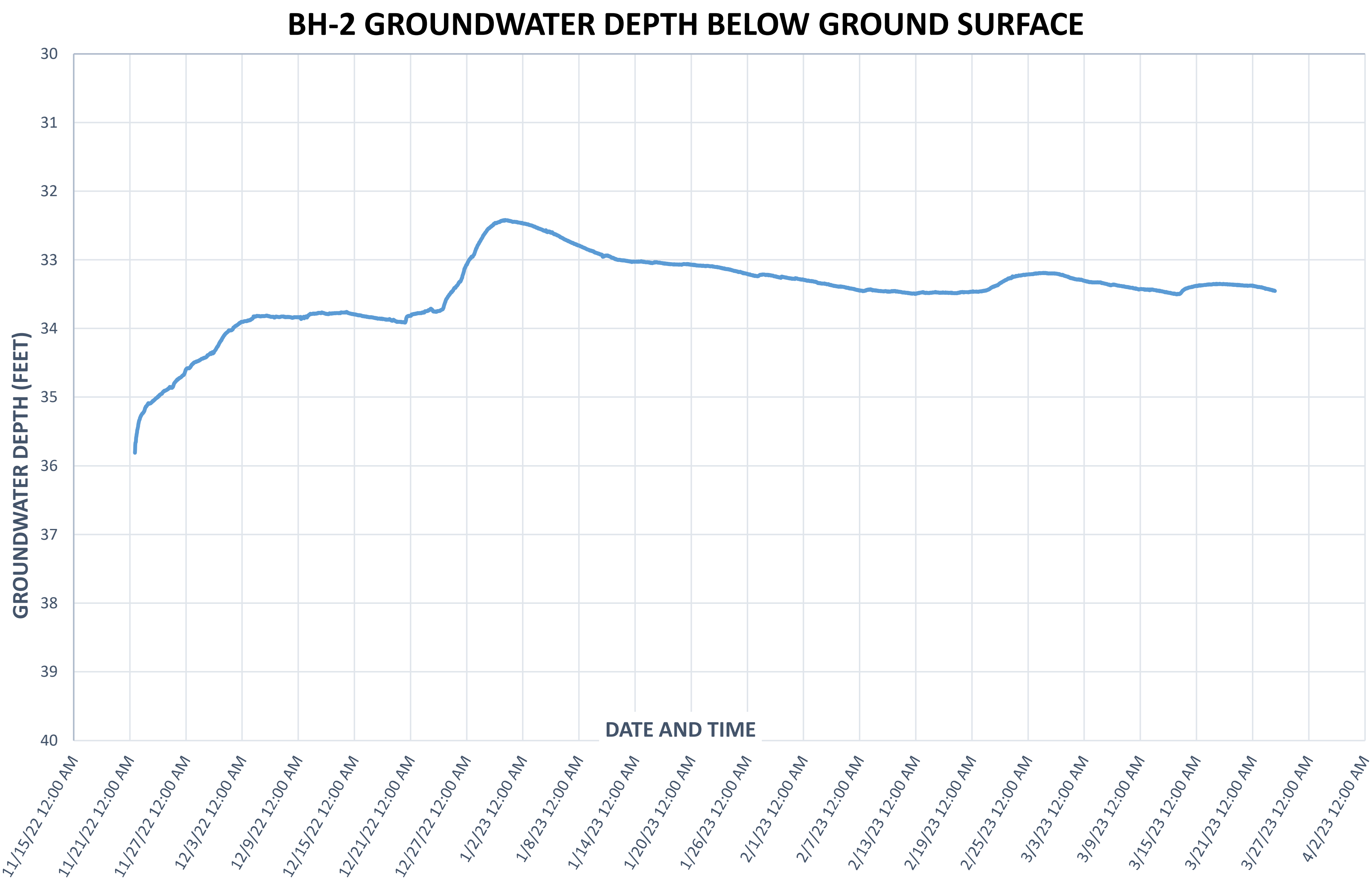
HWA GEOSCIENCES INC. DBE/MWBE

GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT
MAPLE VALLEY, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2G
CHECK BY:	PROJECT NO.:
MAB/JTW	2021-163-21

C:\USERS\CFRY\DESKTOP\2021-163-21 GREEN TO CEDAR RIVER TRAIL\2021-163-21 GREEN TO CEDAR RIVER TRAIL.DWG <2G> Plotted: 1/26/2023 11:06 AM



HWA GEOSCIENCES INC.

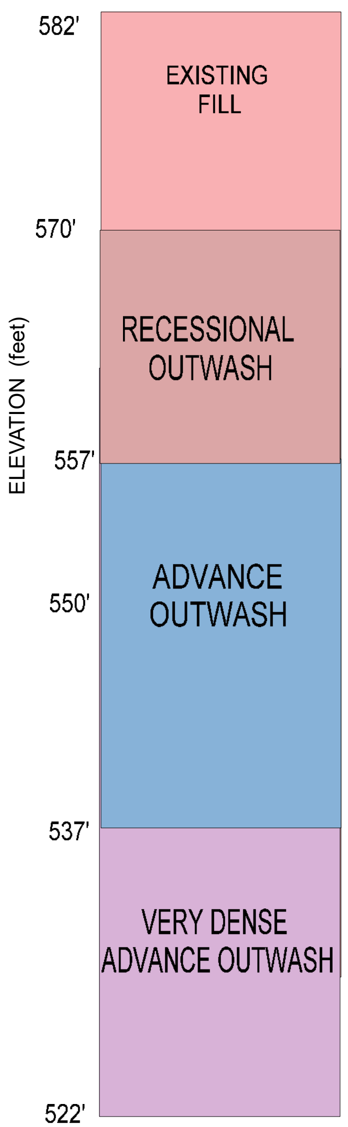
BH-2 WATER LEVEL DATA

GREEN TO CEDAR RIVERS REGIONAL TRAIL - SOUTH SEGMENT
MAPLE VALLEY, WASHINGTON

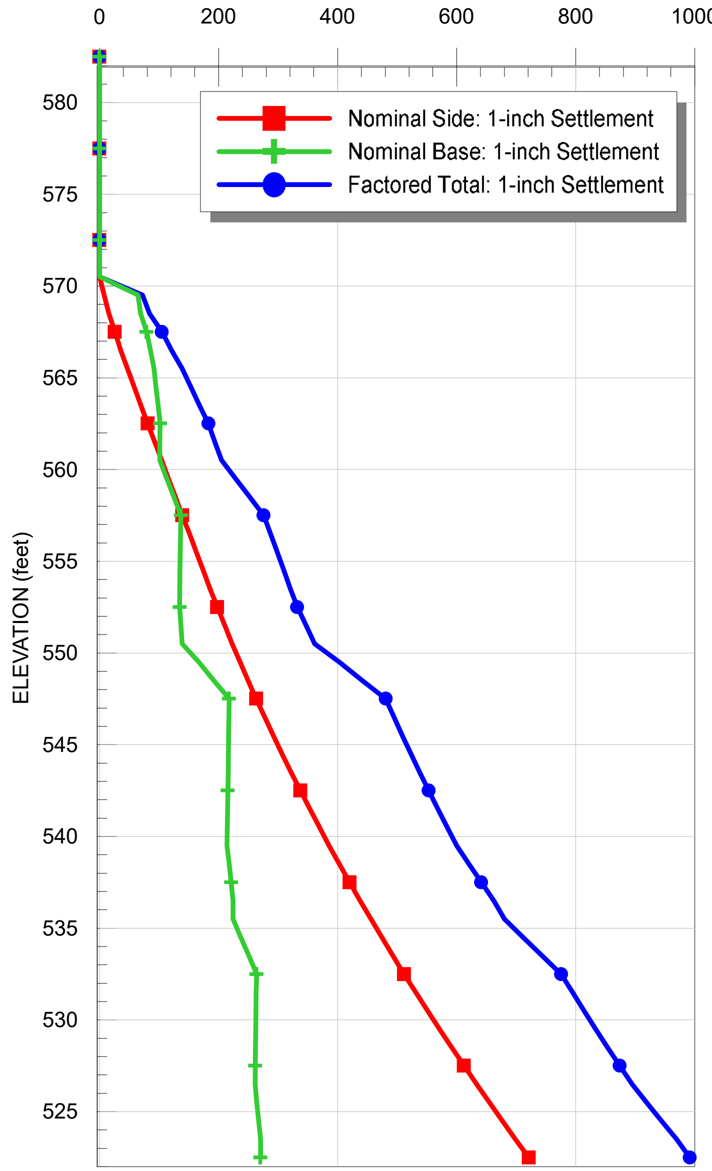
FIGURE NO. **3**

PROJECT NO. 2021-163-21

ASSUMED SUBSURFACE PROFILE



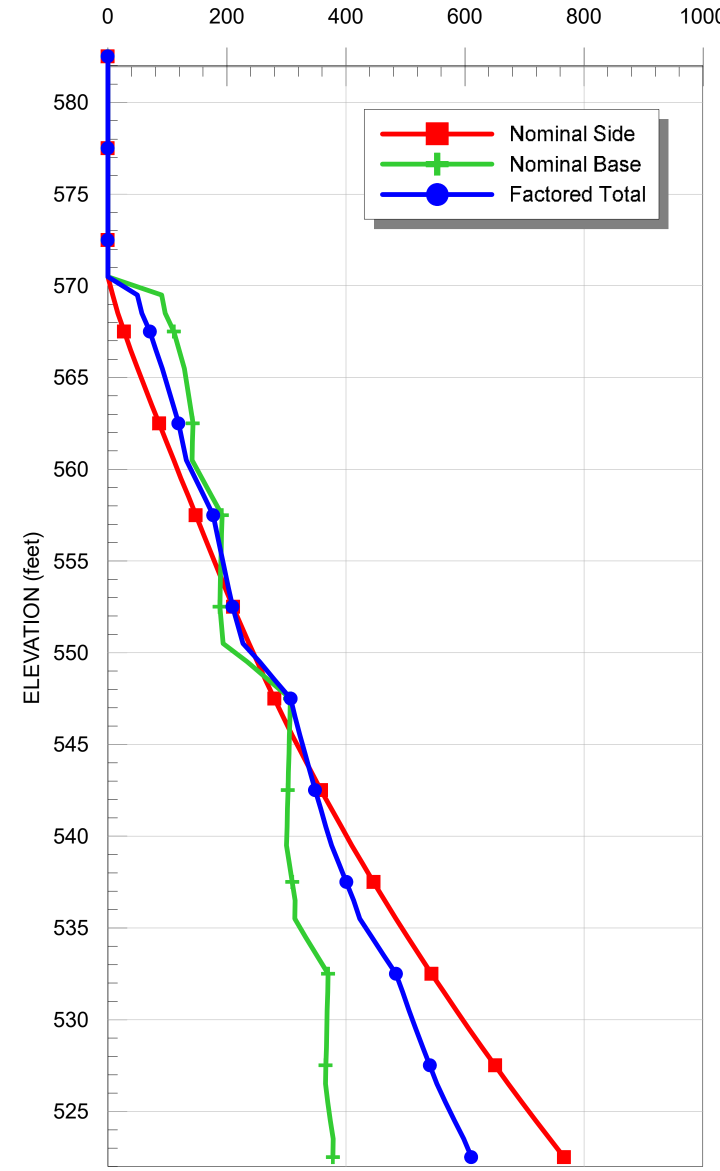
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.
2. Settlements is based on a single shaft. No group action is considered.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

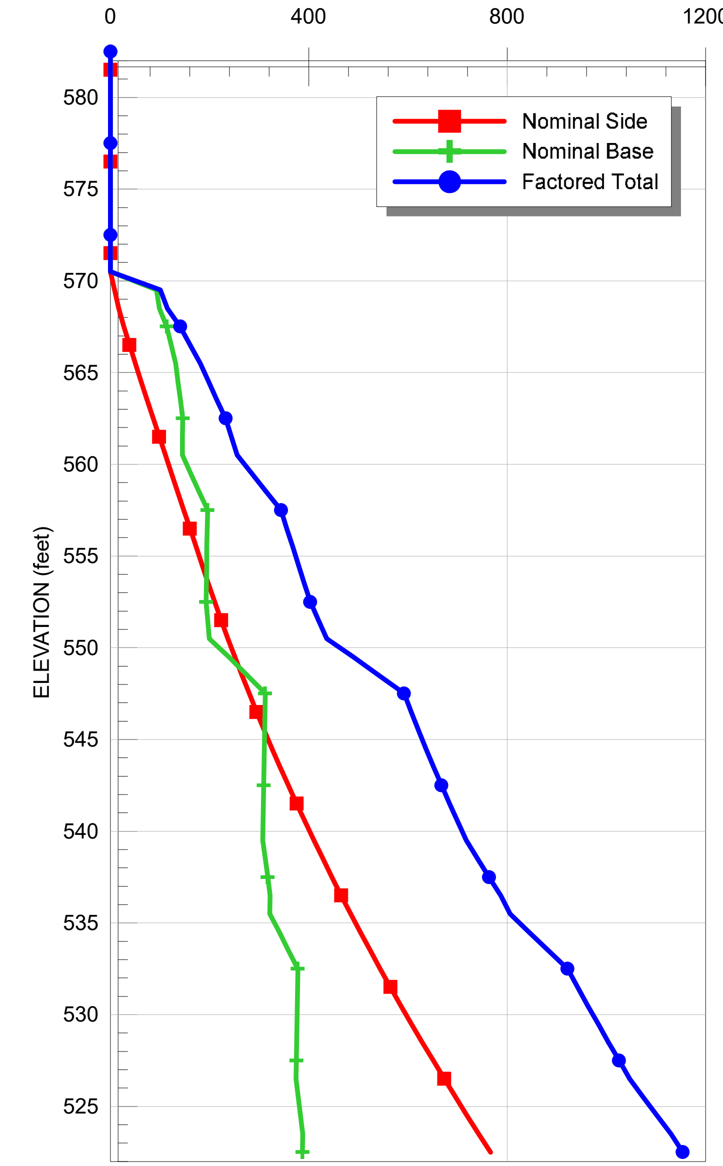
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).
3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

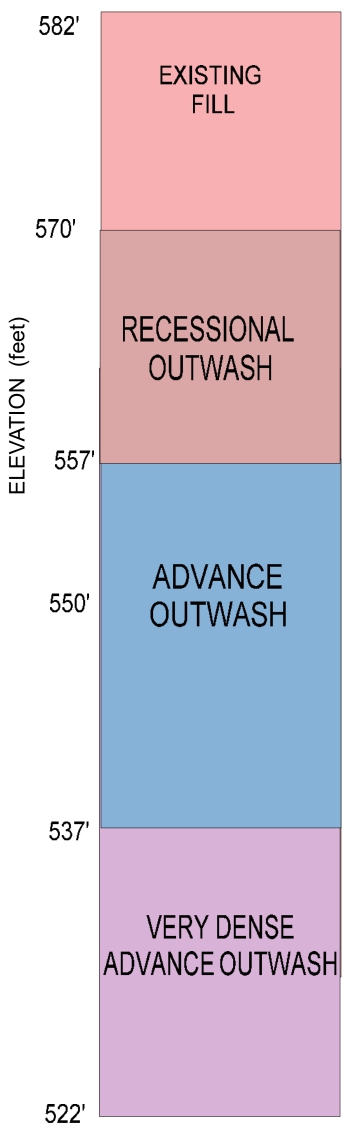
1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

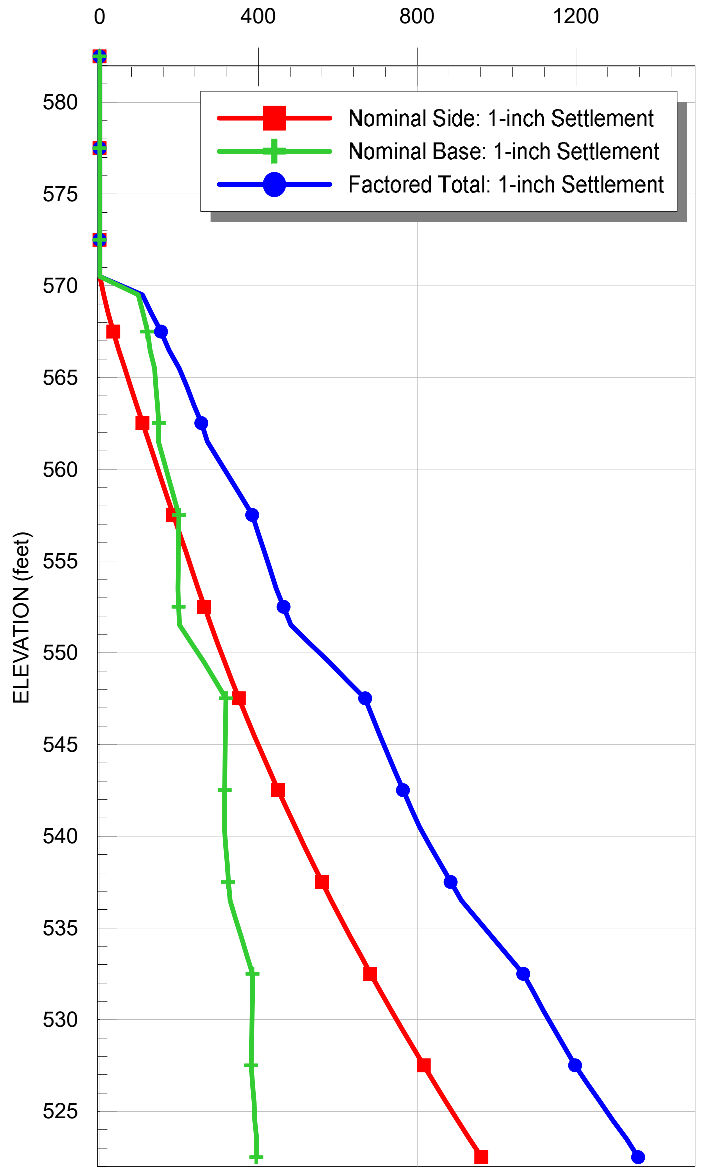
1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.
2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. The nominal side and base resistance values presented do not include the resistance factors.
4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.
5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GEOSCIENCES INC.	GREEN TO CEDAR RIVERS REGIONAL TRAIL -SOUTH SEGEMENT MAPLE VALLEY, WASHINGTON	BRIDGE ABUTMENTS AXIAL SHAFT CAPACITIES 3-FOOT DIAMETER SHAFT	DRAWN BY JTW CHECKED BY JLG DATE 1.25.2203	FIGURE NO. 4 PROJECT NO. 2021-163-21

ASSUMED SUBSURFACE PROFILE



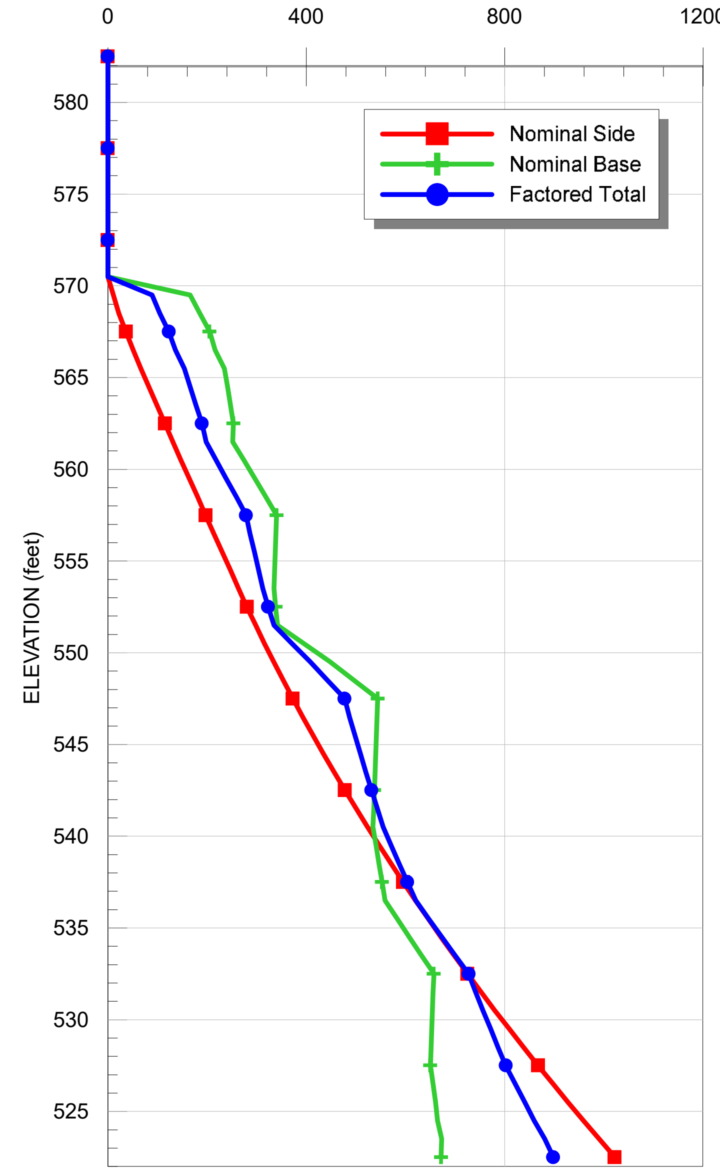
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.
2. Settlements is based on a single shaft. No group action is considered.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

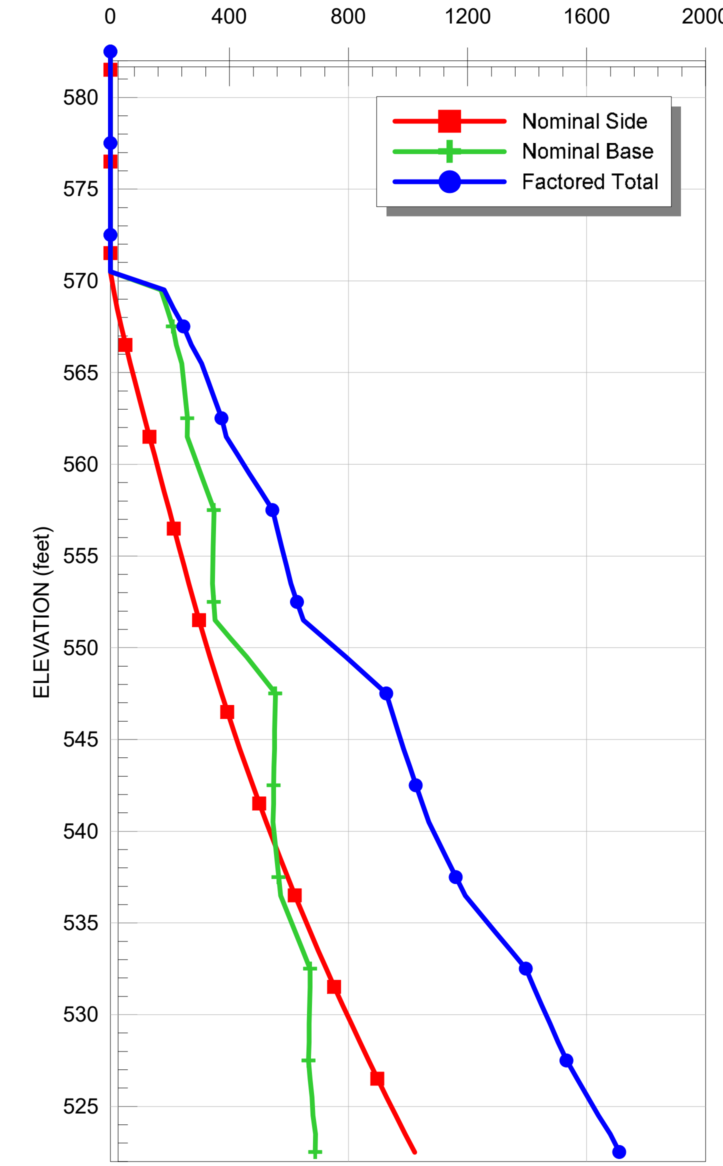
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).
3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.
2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. The nominal side and base resistance values presented do not include the resistance factors.
4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.
5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GEOSCIENCES INC.

GREEN TO CEDAR RIVERS REGIONAL TRAIL -SOUTH SEGEMENT
MAPLE VALLEY, WASHINGTON

BRIDGE ABUTMENTS
AXIAL SHAFT CAPACITIES
4-FOOT DIAMETER SHAFT

DRAWN BY JTW	FIGURE NO. 5
CHECKED BY JLG	
DATE 1.25.2203	PROJECT NO. 2021-163-21

APPENDIX A

FIELD EXPLORATIONS

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils	Gravel and Gravelly Soils	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
	More than 50% of Coarse Fraction Retained on No. 4 Sieve	Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
More than 50% Retained on No. 200 Sieve Size	Sand and Sandy Soils	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
	50% or More of Coarse Fraction Passing No. 4 Sieve	Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILT/Organic CLAY
Highly Organic Soils			PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit, LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
OC	Organic Content
pH	pH of Soils
PID	Photoionization Device Reading
PP	Pocket Penetrometer (Approx. Comp. Strength, tsf)
Res.	Resistivity
SG	Specific Gravity
CD	Consolidated Drained Triaxial
CU	Consolidated Undrained Triaxial
UU	Unconsolidated Undrained Triaxial
TV	Torvane (Approx. Shear Strength, tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- Non-standard Penetration Test (3.0" OD Split Spoon with Brass Rings)
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- 3-1/4" OD Split Spoon

GROUNDWATER SYMBOLS

- ∇ Groundwater Level (measured at time of drilling)
- ▼ Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)

Components are arranged in order of increasing quantities.

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation. Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

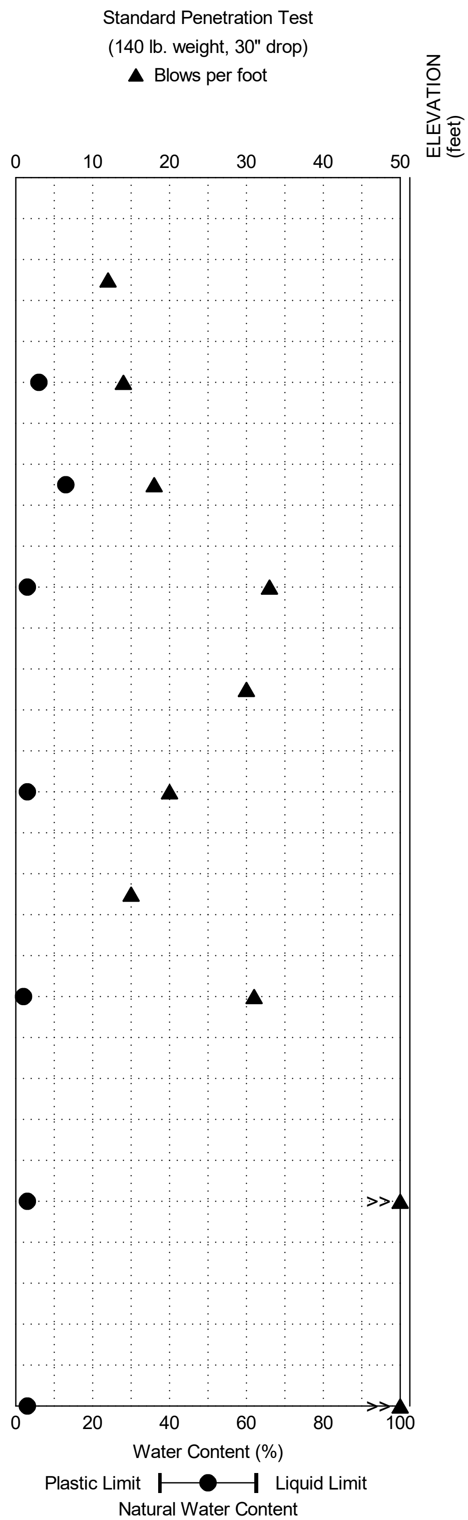
LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2021-163-21 FIGURE: A-1

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-70 Tracked Rig, 4.25" ID
SAMPLING METHOD: SPT w/Auto Hammer, Cal Mod
LOCATION: See Figure 2

DATE STARTED: 11/22/2022
DATE COMPLETED: 11/22/2022
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	SM	Medium dense, no recovery. Likely silty, gravelly SAND or silty, sandy GRAVEL, based on test pits. **(FILL)** Gravelly drill action.	S-1	3-7-5		
5		Medium dense, dark grayish-brown, gravelly, silty SAND, moist.	S-2	4-6-8		
		Medium dense, brown, gravelly, silty SAND, moist.	S-3	4-7-11		
10	SP	Dense, grayish-brown, gravelly SAND, moist. **(RECESSIONAL OUTWASH)**	S-4	10-16-17		
	GM	Dense, light brown, silty, sandy Gravel, moist. Blow counts collected with SPT, sample collected with Cal.Mod.	S-5A S-5B	14-15-15		
15	SP	Medium dense, grayish-brown, gravelly SAND, moist.	S-6	6-10-10	GS	
	SM	Medium dense, olive brown, silty, fine to course SAND, moist. Scattered gravel.	S-7	7-7-8		
20	GM	Dense. olive brown, silty, sandy GRAVEL, moist.	S-8	6-9-22		
25	GM	Very dense, grayish-brown, silty, sandy GRAVEL, moist. **(ADVANCE OUTWASH)**	S-9	18-22-33		
30						



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

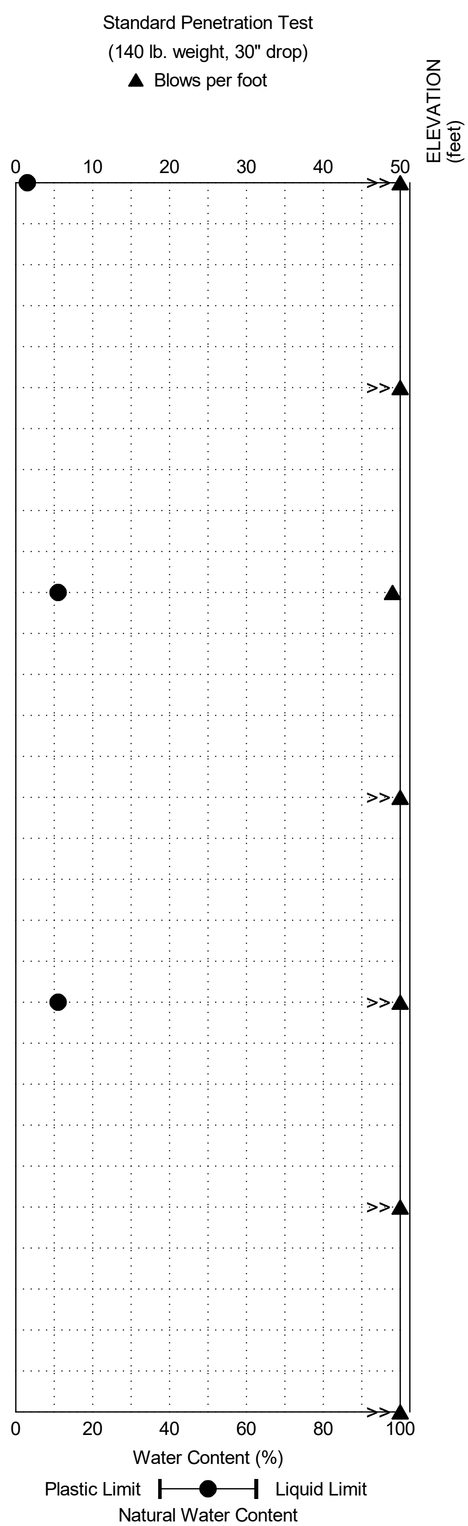
BORING:
BH-1

PROJECT NO.: 2021-163-21 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-70 Tracked Rig, 4.25" ID
SAMPLING METHOD: SPT w/Auto Hammer, Cal Mod
LOCATION: See Figure 2

DATE STARTED: 11/22/2022
DATE COMPLETED: 11/22/2022
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30	GP GM	Very dense, olive brown, slightly silty, sandy, GRAVEL, moist.	S-10	17-28-32	GS	
		Gravelly drill action.				
		Drill bit grinding on something hard.				
35	GM	Very dense, olive brown, silty, sandy GRAVEL, moist.	S-11	50/1"		
40		Dense, olive brown, silty, sandy GRAVEL, wet.	S-12	21-22-27		▽
		Gravelly drill action, drill rig stalling on obstructions.				
45		Very dense, olive brown, silty, sandy GRAVEL, wet.	S-13	20-50/5"		
50	SW SM	Very dense, gray, slightly silty, gravelly, fine to course SAND, wet.	S-14	15-27-46		
		Gravelly drill action, drill rig stalling on obstructions.				
55		Very dense dark gray, slightly silty, gravelly, fine to course SAND, wet.	S-15	50-50/5"		
60						



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

BORING:
BH-1

PAGE: 2 of 3

PROJECT NO.: 2021-163-21 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-70 Tracked Rig, 4.25" ID
SAMPLING METHOD: SPT w/Auto Hammer, Cal Mod
LOCATION: See Figure 2

DATE STARTED: 11/22/2022
DATE COMPLETED: 11/22/2022
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
60		SM	Very dense, dark gray, silty, gravelly, fine to course SAND, wet.		S-16	14-50/5"		
			BH-1 terminated at 60.9 feet below ground surface (bgs). Groundwater encountered at about 40 feet bgs during the exploration. Boring abandoned with 3/8" bentonite chips.					
65								
70								
75								
80								
85								
90								

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

0 10 20 30 40 50

ELEVATION (feet)

0 20 40 60 80 100
Water Content (%)
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

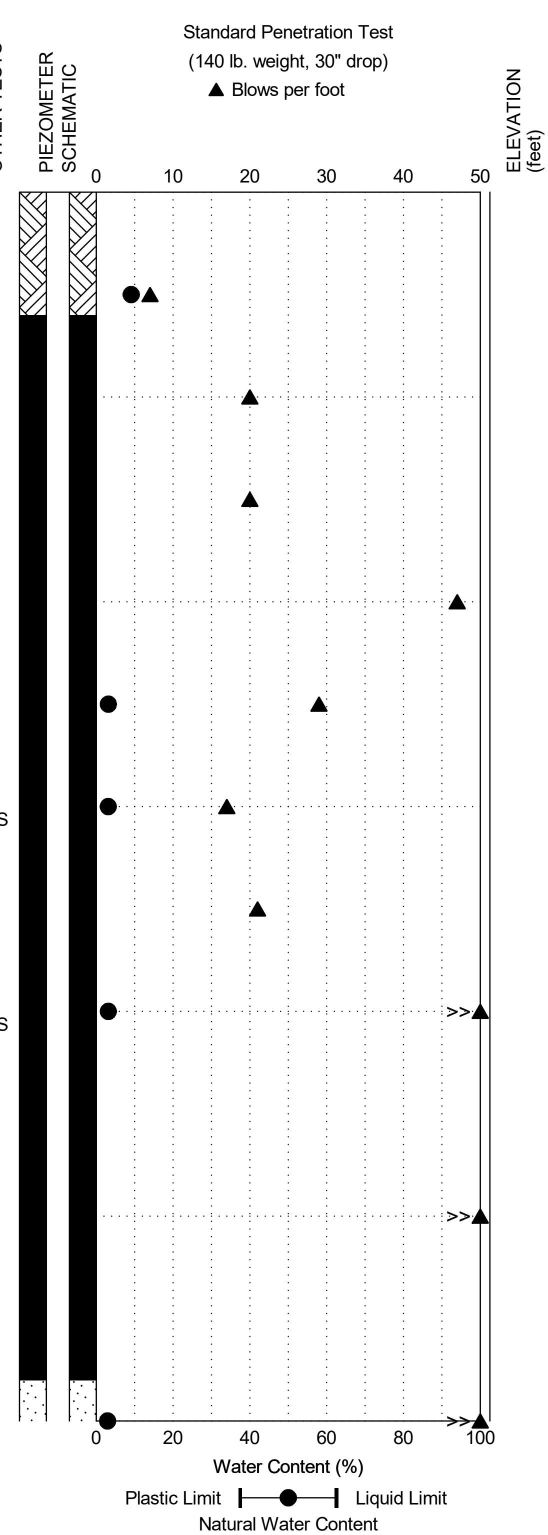
BORING:
BH-1

PROJECT NO.: 2021-163-21 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-70 Tracked Rig, 4.25" ID
SAMPLING METHOD: SPT w/Auto Hammer, Cal Mod
LOCATION: See Figure 2

DATE STARTED: 11/21/2022
DATE COMPLETED: 11/21/2022
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	SM	Loose, dark brown, silty, gravelly SAND, moist. Abundant organics and roots. **(FILL)**	S-1	6-3-4	
5		Medium dense, no recovery. Cuttings are very gravelly.	S-2	10-11-9	
		Medium dense, no recovery. Cuttings same as previous.	S-3	9-9-11	
		Gravelly drill action.			
10	GW	Medium dense, gray, GRAVEL, moist. Recovered broken gravel only.	S-4	22-21-26	
	SM	Medium dense, olive brown, silty, gravelly SAND, moist. **(RECESSIONAL OUTWASH)**	S-5	6-12-17	
15	GW	Loose, slightly rust-mottled dark grayish-brown, very sandy GRAVEL, moist to wet.	S-6	9-8-9	GS
	GW GM	Medium dense, olive brown, slightly silty, sandy GRAVEL, moist. Drove rock, poor recovery.	S-7	13-13-8	
20		Dense, dark olive brown, slightly silty, sandy GRAVEL, moist.	S-8	41-30-30	GS
		Very gravelly drill action.			
25	GW GM	Very dense, no recovery. Sampler appeared to be bouncing on a rock. **(ADVANCE OUTWASH)**	S-9	50/5"	
		Very gravelly drill action, cuttings gravel.			
30					



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

BORING:
BH-2

PAGE: 1 of 2

PROJECT NO.: 2021-163-21 FIGURE: A-3

PZO-DSM 2021-163.GPJ 1/23/23

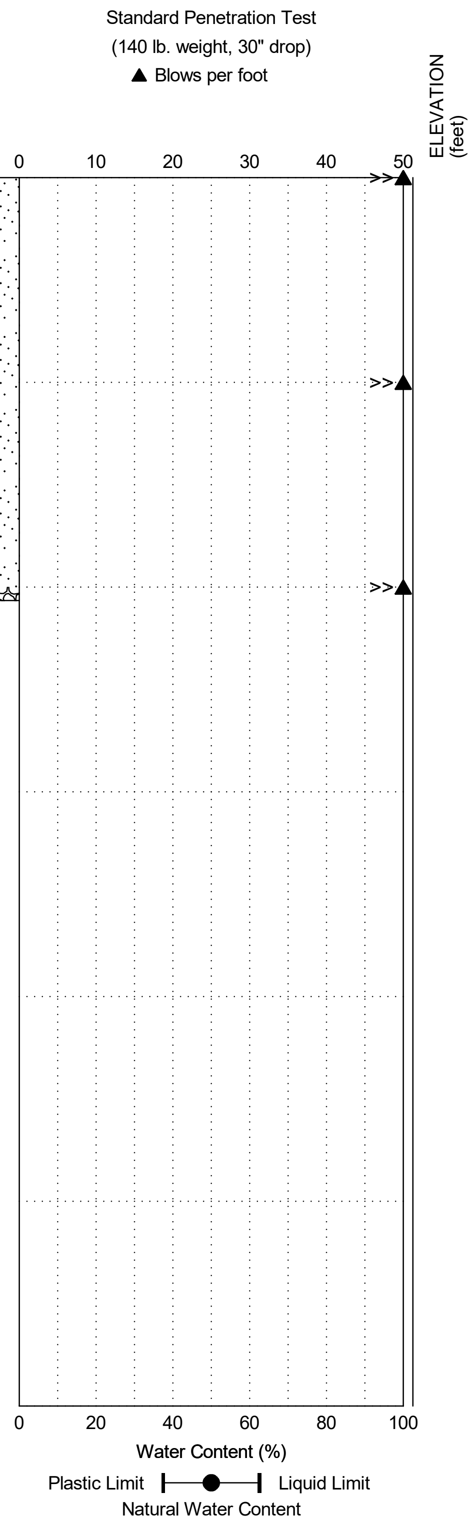
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-70 Tracked Rig, 4.25" ID
SAMPLING METHOD: SPT w/Auto Hammer, Cal Mod
LOCATION: See Figure 2

DATE STARTED: 11/21/2022
DATE COMPLETED: 11/21/2022
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30		Very dense, dark olive brown, slightly silty, sandy GRAVEL, moist.	S-10	30-40-50	GS
35	GW	Very dense, dark gray, GRAVEL, moist. Drove rock, poor recovery of broken gravel.	S-11	50/5"	
		Gravelly drill action, slow advancement at about 39 feet. Drill rig stalling out on hard obstruction.			
40	SM	Very dense, olive brown, silty SAND, wet.	S-12	50/4"	

BH-2 terminated at 40.3 feet below ground surface (bgs) due to refusal on rock obstructions.
Groundwater encountered at about 40 feet bgs during the exploration.
Groundwater measured to 35.81 feet on 11-21-2022 after the exploration.
2 inch PVC piezometer installed to 40 feet bgs.
DOE well tag #BPP 647.

Depth scale (feet): 30, 35, 40, 45, 50, 55, 60



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

BORING:
BH-2

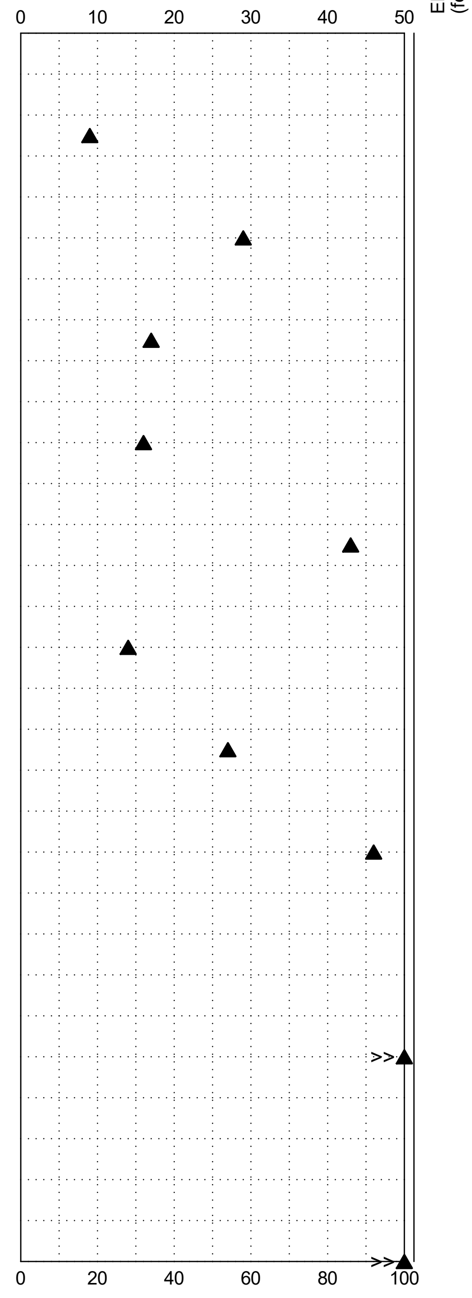
PROJECT NO.: 2021-163-21 FIGURE: A-3

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-70 Tracked Rig, 4.25" ID
SAMPLING METHOD: SPT w Auto Hammer
LOCATION: See Figure 2

DATE STARTED: 11/21/2022
DATE COMPLETED: 11/21/2022
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	SM					
		Loose, no recovery. **(FILL)**	S-1	7-4-5		
5		Medium dense, dark brown, silty, gravelly SAND, moist. Poor recovery.	S-2	16-17-12		
		Gravelly drill action.				
		Medium dense, brown, silty, gravelly SAND, moist. Poor recovery.	S-3	9-8-9		
10		Medium dense, no recovery.	S-4	9-7-9		
	GM	Dense, olive brown, silty, sandy GRAVEL, moist. Poor recovery. **(RECESSIONAL OUTWASH)**	S-5	10-20-23		
15	GW GM	Medium dense, dark grayish-brown, slightly silty, sandy GRAVEL, moist. Poor recovery.	S-6	3-5-9		
	SW	Medium dense, dark grayish-brown, gravelly, fine to course SAND, moist.	S-7	8-10-17		
20		Dense, slightly rust-mottled dark grayish-brown, gravelly SAND, moist to wet.	S-8	12-23-23		
25	GW GM	Very dense, dark grayish-brown, slightly silty, sandy GRAVEL. moist. **(ADVANCE OUTWASH)**	S-9	8-25-45		
		Gravelly drill action.				
30						

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot



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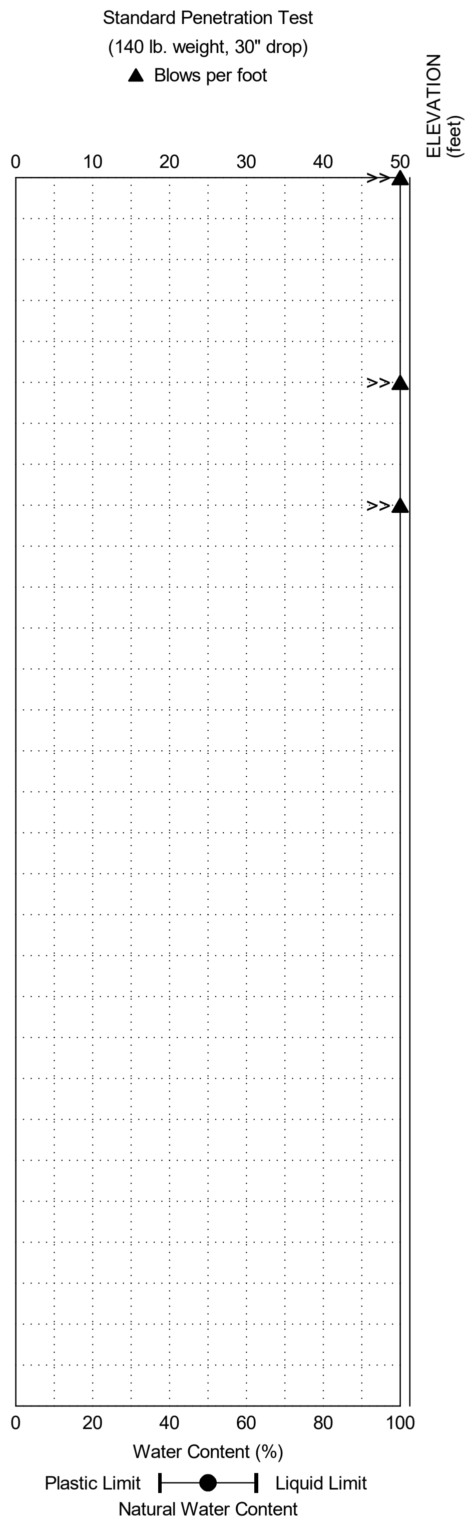
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-70 Tracked Rig, 4.25" ID
SAMPLING METHOD: SPT w Auto Hammer
LOCATION: See Figure 2

DATE STARTED: 11/21/2022
DATE COMPLETED: 11/21/2022
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30	GM	Very dense, dark olive brown, silty, sandy GRAVEL. moist.	S-10	45-35-45		
35	GW	Very dense, dark gray GRAVEL, moist. Drove rock, only recovered broken gravel.	S-11	50/2"		
		Very gravelly drill actoin. Drill stalling on hard obstruction.				
		Very dense, no recovery.	S-12	50/1"		∇

BH-2A terminated at 38.1 feet below ground surface (bgs) due to refusal on rock obstructions.
Groundwater encountered at about 38 feet bgs during the exploration.
Boreing abandoned with 3/8" bentonite chips.



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HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

BORING:
BH-2A

PROJECT NO.: 2021-163-21 FIGURE: A-4

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/15/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**	○	S-1	3	GS
		GP	Loose to medium dense, grayish-brown, sandy GRAVEL, moist. Gravel subangular. **(FILL)**	○	S-2	3	GS
5		GW GM	Loose to medium dense, slightly rust-mottled dark grayish-brown, sandy GRAVEL, moist. Scattered cobbles, boulders. **(RECESSIONAL OUTWASH)** East and west side walls caving in towards existing path and adjacent sewer trench. Soil becomes dark grayish-brown.				
10				○	S-3		
15			TP-1 terminated at about 12 feet below ground surface due to refusal as walls caved in. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.				
20							

TEST PIT PHOTO

E

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 1

PROJECT NO.: 2021-163-21 FIGURE: A-5

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/15/22
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**				
	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Scattered to adundant roots. **(FILL)**	○	S-1	8	GS
5	GP	Loose to medium dense, dark olive brown, very sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)** Minor side wall caving.				
10		Gravel becomes coarser with depth, sand fraction decreases.	○	S-2	2	GS
15	GW	Loose to medium dense, dark olive brown, very sandy GRAVEL, moist. Scattered cobbles.				
20						

TP-2 terminated at about 14.5 feet below ground surface (bgs).
No groundwater encountered during the exploration.
1.25 inch PVC well installed to 9.5 feet bgs.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

E

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 2

PROJECT NO.: 2021-163-21 FIGURE: A-6

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/15/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**				
		GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Scattered to abundant roots. **(FILL)**	○	S-1	3	GS
5		GW	Loose to medium dense, dark grayish-brown, sandy GRAVEL, moist. Abundant cobbles and boulders. **(RECESSIONAL OUTWASH)**				
			Gravel fraction increases.	○	S-2	2	GS
			East side wall caving.				
10							
				○	S-3		
15			TP-3 terminated at about 13 feet below ground surface (bgs). No groundwater encountered during the exploration. 1.25 inch PVC well installed to 9.5 feet bgs. Test pit abandoned with native cuttings.				
20							

TEST PIT PHOTO

W

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HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 3

PROJECT NO.: 2021-163-21 FIGURE: A-7

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/15/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty, very gravelly SAND, moist. Abundant roots. **(TOPSOIL)**				
		SM	Loose to medium dense, dark brown, silty, very gravelly SAND, moist. Scattered to abundant roots. **(FILL)**	○	S-1	1	GS
5		GW	Loose to medium dense, grayish-brown, sandy GRAVEL, moist. Abundant cobbles, boulders. **(RECESSIONAL OUTWASH)**				
10		GP	Loose to medium dense, dark grayish-brown, sandy GRAVEL, moist. Abundant cobbles, boulders. **(RECESSIONAL OUTWASH)**	○	S-2	3	GS
15			TP-4 terminated at about 14 feet below ground surface. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.	○	S-3		
20							

TEST PIT PHOTO

W

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 4

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-8

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/15/22
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**			
	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Scattered to abundant roots, scattered cobbles. **(FILL)**			
5	GP	Loose to medium dense, dark grayish-brown, sandy GRAVEL, moist. Scattered cobbles, boulders. **(RECESSIONAL OUTWASH)** South wall caving.	S-1	3	GS
10		Gravel coursens with depth, cobble fraction increases.			
15	GW	Loose to medium dense, olive brown, sandy GRAVEL, moist. Scattered cobbles, boulders.	S-2	3	GS

TP-5 terminated at about 14.5 feet below ground surface (bgs).
No groundwater encountered during the exploration.
1.25 inch PVC well installed to 9.25 feet bgs.
Test pit abandoned with native cuttings.

Depth scale: 0, 5, 10, 15, 20

TEST PIT PHOTO

S

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 5

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-9

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/15/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty, gravelly SAND, moist. Abundant roots. **(TOPSOIL)**				
		GP GM	Loose to medium dense, olive brown, slightly silty, sandy GRAVEL, moist. Scattered cobbles, boulders. **RECESSIONAL OUTWASH)**	○	S-1	8	GS
5		GP	Loose to medium dense, olive brown, sandy GRAVEL, moist. Scattered cobbles, boulders.	○	S-2	2	GS
		SP	Loose to medium dense, gray, gravelly SAND, moist. Laminated.	○	S-3	7	GS
		ML	Stiff to very stiff, gray, sandy SILT, moist.				
10		GW GM	Loose to medium dense, olive brown, slightly silty, sandy GRAVEL, moist. Scattered cobbles, boulders.	○	S-4		
15			TP-6 terminated at about 14 feet below ground surface. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.				
20							

TEST PIT PHOTO

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N

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 6

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-10

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/14/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty, gravelly SAND, moist. Abundant roots. **(TOPSOIL)**				
		GP GM	Loose to medium dense, reddish brown, slightly silty, sandy GRAVEL , moist. scattered roots. **(FILL)**	○	S-1	7	GS
5		GW	Loose to medium dense, grayish-brown, sandy GRAVEL, moist. scattered cobbles. **(RECESSIONAL OUTWASH)**	○	S-2	3	GS
10		GP	Loose to medium dense, dark grayish brown, sandy GRAVEL, moist to wet.	○	S-3	4	GS
15			TP-7 terminated at about 14 feet below ground surface (bgs). No groundwater encountered during the exploration. 1.25 inch PVC well installed to 9 feet bgs. Test pit abandoned with native cuttings.				
20							

TEST PIT PHOTO

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S

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 7

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-11

SMART TP 2021-163.GPJ 5/15/23

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/14/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty, gravelly SAND, moist. Abundant roots. **(TOPSOIL)**				
		SM	Loose to medium dense, dark brown, silty, gravelly SAND, moist. Scattered to abundant roots. **(FILL)**	○	S-1	2	GS
5		GP	Loose to medium dense, brown, sandy GRAVEL, moist. Scattered rootlets, cobbles. **(RECESSIONAL OUTWASH)** Grades to grayish-brown, no organics.				
10							
				○	S-2		
15			TP-8 terminated at about 14 feet below ground surface. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.				
20							

TEST PIT PHOTO

N

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HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 8

PROJECT NO.: 2021-163-21 FIGURE: A-12

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/14/22
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	GW	Loose to medium dense, brown, sandy GRAVEL, moist. Abundant roots. **(FILL)**	○	S-1	4	GS
5	SM	Loose to medium dense, dark brown, silty SAND, moist. Scattered gravel. **(BURIED TOPSOIL)**	○	S-2	28	GS
			○	S-3		
	GM	Loose to medium dense, yellowish-brown, silty, sandy GRAVEL, moist. Abundant cobbles, scattered boulders. **(RECESSIONAL OUTWASH)**				
10	GW GM	Loose to medium dense, grayish-brown, slightly silty, sandy GRAVEL, moist. Side walls caving.	○	S-4		
15		TP-9 terminated at about 12 feet below ground surface due to refusal as walls caved in. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.				
20						

TEST PIT PHOTO

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N

HWA
HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP- 9

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-13

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/14/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, brown, silty, gravelly SAND, moist. Abundant roots. **(TOPSOIL)**				
		GP	Loose to medium dense, dark grayish-brown, sandy GRAVEL, moist. Abundant cobbles, scattered boulders. **(RECESSIONAL OUTWASH)** Side walls caving.	○	S-1	3	GS
5							
10							
		GW	Loose to medium dense, dak grayish brown, sandy GRAVEL, moist.	○	S-2	3	GS
15							
20							

TP-10 terminated at about 13.5 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

E

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HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-10

PROJECT NO.: 2021-163-21 FIGURE: A-14

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/14/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, gravelly, silty SAND, moist. Abundant roots. **(TOPSOIL)**	○	S-1	16	GS
		GP GM	Loose to medium dense, reddish brown, sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)**	○	S-2	6	GS
5		GW GM	Loose to medium dense, dark brown, slightly silty, sandy GRAVEL, moist. Scattered cobbles.	○	S-3		
10		GW	Loose to medium dense, dark grayish-brown,sandy GRAVEL, moist. Scattered cobbles.				
15							
20							

TP-11 terminated at about 14 feet below ground surface (bgs).
No groundwater encountered during the exploration.
1.25 inch PVC well installed to 9.5 feet bgs.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

E

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HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-11

PROJECT NO.: 2021-163-21 FIGURE: A-15

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/14/22
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	SM	Loose to medium dense, dark brown, gravelly, silty SAND, moist. Abundant roots. **(FILL)**	S-1	4	GS
	GP	Loose to medium dense, reddish-brown, sandy GRAVEL, moist. Scattered cobbles, boulders. **(RECESSIONAL OUTWASH)**			
5		Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Scattered cobbles, boulders.			
10	GW	Loose to medium dense, dark grayish-brown, sandy GRAVEL, moist. Scattered cobbles.	S-2	3	GS
15		TP-12 terminated at about 14 feet below ground surface (bgs). No groundwater encountered during the exploration. 1.25 inch PVC well installed to 9 feet bgs. Test pit abandoned with native cuttings.			
20					

TEST PIT PHOTO

E

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HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-12

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-16

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/16/22
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		GM	Loose to medium dense, silty, sandy GRAVEL, moist. Abundant Roots. **(TOPSOIL)**				
		GP	Loose to medium dense, brown, silty, sandy GRAVEL, moist. Abundant cobbles, boulders. **(RECESSIONAL OUTWASH)**				
					S-1	4	GS
5							
10			Loose to medium dense, dark grayish-brown, slightly silty, sandy GRAVEL, moist. Abundant cobbles, boulders.		S-2	2	GS
					S-3		
15			TP-13 terminated at about 13 feet below ground surface. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.				
20							

TEST PIT PHOTO

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HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-13

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-17

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/16/22
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**			
	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Scattered to abundant roots. **(FILL)**			
5	GP	Loose to medium dense, reddish-brown, sandy GRAVEL, moist. Abundant cobbles, boulders. **(RECESSIONAL OUTWASH)**	S-1	2	GS
		Becomes dark grayish-brown. Side walls caving.			
		Becomes moist to wet.	S-2		
10	GM	Loose to medium dense, olive brown, silty, sandy GRAVEL, wet. Abundant cobbles, boulders.	S-3		
	GM	Dense to very dense olive brown, silty, sandy GRAVEL, moist. **(TILL)**	S-4	8	GS
15					
20					

TP-14 terminated at about 12 feet below ground surface (bgs) due to refusal as walls caved in.
Perched groundwater encountered at 10 feet bgs during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-14

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-18

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu WB 140 Backhoe

LOCATION: See Figure 2
DATE COMPLETED: 11/16/22
LOGGED BY: M.A. Benson

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**			
	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Scattered to abundant roots. **(FILL)**			
5	GW	Loose to medium dense, reddish-brown, sandy GRAVEL, moist. Abundant cobbles, boulders. **(RECESSIONAL OUTWASH)**	S-1	4	GS
	GP	Side walls caving. Loose to medium dense, dark grayish-brown, slightly sandy GRAVEL, moist. Abundant cobbles, boulders.	S-2	2	GS
10			S-3		

TP-15 terminated at about 12 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

15

20

TEST PIT PHOTO

N

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-15

PROJECT NO.: 2021-163-21 FIGURE: A-19

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/4/23
LOGGED BY: L. Cressler

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**			
	GM	Loose to medium dense, reddish-brown, silty, sandy GRAVEL, moist. Scattered cobbles and black charcoal. **(FILL)**	S-1	14	GS
5	GW	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered cobbles and sparse boulders. **(RECESSIONAL OUTWASH)**	S-2		
10		TP-16 terminated at about 9.1 feet below ground surface due to refusal as walls caved in. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.			
15					
20					

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-16

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-20

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/3/23
LOGGED BY: L. Cressler

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty. gravelly SAND, moist. Scattered cobbles. **(TOPSOIL)**				
		SM	Loose to medium dense, gray, silty, very gravelly SAND, moist. Minor roots. **(FILL)**				
		GM	Loose to medium dense, dark brown, silty. sandy GRAVEL, moist. Scattered cobbles.				
5		GP	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)**		S-1	3	GS
10					S-2		
15							
20							

TP-17 terminated at about 10 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
TP-17

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/4/23
LOGGED BY: L. Cressler

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		GM	Loose to medium dense, dark brown, silty, sandy GRAVEL, moist. Abundant roots. **(TOPSOIL)**				
		SW SM	Loose to medium dense, reddish-brown, slightly silty, gravelly SAND, moist. Scattered cobbles and roots. **(WEATHERED RECESSIONAL OUTWASH)**		S-1		
					S-2	5	GS Infiltration test - 3.5' bgs
5		GP	Loose to medium dense, grayish-brown, sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)**				
					S-3		
10							
15							

PIT-1 terminated at about 8 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-1

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-22

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/5/23
LOGGED BY: L. Cressler

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty, gravelly SAND, moist. Scattered cobbles, abundant roots and charcoal. **(TOPSOIL)**				
		SM	Loose to medium dense, reddish-brown, silty, gravelly SAND, moist. Scattered cobbles and red mottling visible. **(FILL)**				
5		GP	Loose to medium dense, gray to dark gray, sandy GRAVEL, moist. Scattered cobbles and red mottling visible. **(RECESSIONAL OUTWASH)**		S-1	5	GS Infiltration test - 4.5' bgs
					S-2		
10							
15							

PIT-2 terminated at about 10.5 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-2

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-23

SMART TP 2021-163.GPJ 5/15/23

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/3/23
LOGGED BY: L. Cressler

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty, gravelly SAND, moist. Abundant roots. **(TOPSOIL)**				
		SM	Loose to medium dense, reddish-brown, silty, gravelly SAND, moist. Scattered roots. **(FILL)**	■	S-1		
		GP	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)**	■	S-2	3	GS Infiltration test - 3' bgs
5				■	S-3		
10			PIT-3 terminated at about 9 feet below ground surface due to refusal as walls caved in. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.				
15							

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-3

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-24

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/6/23
LOGGED BY: L. Cressler

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	SM	Loose to medium dense, dark brown, silty SAND, moist. Abundant roots. **(TOPSOIL)**			
	GM	Loose to medium dense, reddish-brown, silty, sandy GRAVEL, moist. Scattered cobbles and roots. **(WEATHERED RECESSIONAL OUTWASH)**	S-1		
	GP	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)**	S-2		Infiltration test - 4.0' bgs
5	SP	gray, poorly-graded SAND, moist. Minor roots.	S-3	19	GS

10

PIT-4 terminated at about 10 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

15

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

TEST PIT PHOTO

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-4

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-25

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/6/23
LOGGED BY: L. Cressler

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0	SM	Loose to medium dense, dark brown, silty SAND, moist. Abundant roots. **(TOPSOIL)**			
	GP GM		S-1		
	GP GM	Loose to medium dense, grayish-brown, slightly silty, sandy GRAVEL, moist. Scattered cobbles. **(FILL)**			
	GW	Loose to medium dense, reddish-brown, slightly silty, sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)**	S-2	5	Infiltration test - 4.0' bgs
5	GW	Loose to medium dense, brown, sandy GRAVEL, moist. Scattered cobbles.			
	GW GM	Loose to medium dense, reddish-brown, slightly silty, very sandy, GRAVEL with organics, moist. Red mottling visible.	S-3	11	GS
	GP	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered cobbles.			
10		PIT-5 terminated at about 10.2 feet below ground surface due to refusal as walls caved in. No groundwater encountered during the exploration. Test pit abandoned with native cuttings.			
15					

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-5

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-26

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/7/23
LOGGED BY: L. Cressler

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty SAND, moist. Abundant roots. **(TOPSOIL)**				
		SM	Loose to medium dense, dark reddish-brown, silty, gravelly SAND, moist. Abundant roots. **(WEATHERED RECESSIONAL OUTWASH)**				
		GM	Loose to medium dense, grayish-brown, slightly silty, sandy GRAVEL, moist. Visible red mottling.		S-1		
5		GW	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered Cobbles. **(RECESSIONAL OUTWASH)**		S-2	4	Infiltration test - 4' bgs GS
					S-3		
10							
15							

PIT-6 terminated at about 10.5 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-6

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-27

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/10/23
LOGGED BY: L. Cressler

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty SAND, moist. Abundant roots. **(TOPSOIL)**				
		GP GM	Loose to medium dense, reddish-brown, slightly silty, sandy GRAVEL. Roots and scattered cobbles. **(WEATHERED RECESSIONAL OUTWASH)**				
5		GP	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered cobbles. **(RECESSIONAL OUTWASH)**		S-1	5	Infiltration test - 4.5' bgs GS
10							
15							

PIT-7 terminated at about 9.5 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-7

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-28

EXCAVATION COMPANY: Kelly's Excavation
EXCAVATING EQUIPMENT: Hitachi Mini Excavator

LOCATION: See Figure 2
DATE COMPLETED: 4/10/23
LOGGED BY: L. Cressler

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose to medium dense, dark brown, silty SAND, moist. Abundant roots. **(TOPSOIL)**				
		GP GM	Loose to medium dense, reddish-brown, slightly silty, sandy GRAVEL, moist. Scattered cobbles and roots. Red mottling visible. **(WEATHERED RECESSIONAL OUTWASH)**		S-1		
5		GW	Loose to medium dense, gray, sandy GRAVEL, moist. Scattered cobbles, boulders. **(RECESSIONAL OUTWASH)**		S-2	7	Infiltration test - 3.3' bgs GS
					S-3		
10							
15							

PIT-8 terminated at about 9.3 feet below ground surface due to refusal as walls caved in.
No groundwater encountered during the exploration.
Test pit abandoned with native cuttings.

TEST PIT PHOTO

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

LOG OF TEST PIT
PIT-8

PAGE: 1 of 1

PROJECT NO.: 2021-163-21 FIGURE: A-29

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

HWA personnel performed laboratory tests in general accordance with appropriate ASTM test methods. We tested selected soil samples to determine moisture content, grain-size distribution, and Atterberg Limits. The test procedures and results are briefly discussed below.

MOISTURE CONTENT OF SOIL: Laboratory tests were conducted to determine the natural moisture content of selected soil samples, in general accordance with ASTM D-2216. Test results are indicated at the sampled intervals on the appropriate exploration logs in Appendix A and on the Summary of Materials Properties report, Figures B-1 through B-3.

PARTICLE SIZE ANALYSIS OF SOILS: Selected samples were tested to determine the particle size distribution of material in general accordance with ASTM 6913. The results are summarized on the attached Grain Size Distribution reports, Figures B-4 through B-19, and provide information regarding the classification of the sample.

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	ATTERBERG LIMITS (%)			% COBBLES	% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
					LL	PL	PI						
BH-1,S-2	5.0	6.5	6.1									SM	Dark brown, silty SAND
BH-1,S-3	7.5	9.0	12.5									SM	Very dark grayish-brown, silty SAND
BH-1,S-4	10.0	11.5	3.5									SP	Olive-gray, poorly graded SAND
BH-1,S-6	15.0	16.5	2.5						37.6	60.3	2.1	SP	Olive-gray, poorly graded SAND with gravel
BH-1,S-8	20.0	21.5	1.9									GM	Olive-gray, silty GRAVEL
BH-1,S-9	25.0	26.5	3.1									GM	Light olive-gray, silty GRAVEL
BH-1,S-10	30.0	31.5	2.8						55.2	38.2	6.6	GP-GM	Light olive-gray, poorly graded GRAVEL with silt and sand
BH-1,S-12	40.0	41.5	10.7									GM	Olive, silty GRAVEL
BH-1,S-14	50.0	51.5	11.4									GM	Dark gray, silty GRAVEL
BH-2,S-1	2.5	4.0	9.1									SM	Brown, silty SAND
BH-2,S-5	12.5	14.0	3.2									SM	Olive, silty SAND with gravel
BH-2,S-6	15.0	16.5	3.1						51.8	46.1	2.1	GW	Dark olive-gray, well-graded GRAVEL with sand
BH-2,S-8	20.0	21.5	3.1						58.5	36.5	5.0	GW-GM	Olive-gray, well-graded GRAVEL with silt and sand
BH-2,S-10	30.0	31.5	2.9						72.7	22.3	5.0	GW-GM	Olive-gray, well-graded GRAVEL with silt and sand
PIT-1,S-2	3.0	3.5	5.1						55.9	42.3	1.8	GP	Olive-brown, poorly graded GRAVEL with sand
PIT-2,S-1	4.0	4.5	5.2						73.8	22.4	3.8	GP	Dark olive-brown, poorly graded GRAVEL with sand
PIT-3,S-2	2.7	3.2	3.3						56.9	42.0	1.1	GP	Grayish-brown, poorly graded GRAVEL with sand
PIT-4,S-3	4.3	4.8	18.8						7.7	89.7	2.7	SP	Dark grayish-brown, poorly graded SAND
PIT-5,S-2	4.0	4.5	4.9						70.3	26.8	2.9	GW	Olive-brown, well-graded GRAVEL with sand
PIT-5,S-3	7.0	7.5	10.6						58.7	34.8	6.5	GW-GM	Brown, well-graded GRAVEL with silt and sand

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	ATTERBERG LIMITS (%)			% COBBLES	% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
					LL	PL	PI						
PIT-6,S-2	4.0	4.5	3.6						67.5	31.3	1.2	GW	Brown, well-graded GRAVEL with sand
PIT-7,S-1	4.5	5.0	5.4						62.0	36.5	1.6	GP	Olive-gray, poorly graded GRAVEL with sand
PIT-8,S-2	3.3	3.8	7.4						51.1	46.0	2.9	GW	Olive-brown, well-graded GRAVEL with sand
TP- 1,S-1	1.0	1.5	2.6						85.1	13.6	1.3	GP	Dark brown, poorly graded GRAVEL
TP- 1,S-2	3.0	3.5	3.5						78.9	19.5	1.5	GW	Dark brown, well-graded GRAVEL with sand
TP- 2,S-1	3.0	3.5	7.5						74.9	24.1	1.0	GP	Dark brown, poorly graded GRAVEL with sand
TP- 2,S-2	14.0	14.5	2.3						76.3	23.1	0.7	GW	Dark brown, well-graded GRAVEL with sand
TP- 3,S-1	3.0	3.5	3.1						69.9	29.6	0.5	GW	Dark brown, well-graded GRAVEL with sand
TP- 3,S-2	7.0	7.5	1.7						89.3	9.9	0.7	GW	Dark brown, well-graded GRAVEL
TP- 4,S-1	3.5	4.0	1.3						76.2	22.9	0.9	GW	Yellowish-brown, well-graded GRAVEL with sand
TP- 4,S-2	7.0	7.5	3.2						67.5	31.0	1.5	GP	Dark brown, poorly graded GRAVEL with sand
TP- 5,S-1	4.5	5.0	3.0					9.7	68.4	20.7	1.2	GP	Dark olive-brown, poorly graded GRAVEL with sand and cobbles
TP- 5,S-2	14.0	14.5	2.6						89.6	9.2	1.1	GW	Dark brown, well-graded GRAVEL
TP- 6,S-1	3.0	3.5	8.2						69.1	23.1	7.9	GP-GM	Yellowish-brown, poorly graded GRAVEL with silt and sand
TP- 6,S-2	6.0	6.5	1.9						87.2	11.1	1.6	GP	Yellowish-brown, poorly graded GRAVEL
TP- 6,S-3	8.0	8.3	7.0						34.8	64.3	0.9	SP	Dark gray, poorly graded SAND with gravel
TP- 7,S-1	2.0	2.5	7.4						76.5	17.9	5.6	GP-GM	Dark yellowish-brown, poorly graded GRAVEL with silt and sand
TP- 7,S-2	4.0	4.5	2.5						65.3	32.3	2.4	GW	Light olive-brown, well-graded GRAVEL with sand
TP- 7,S-3	11.0	11.5	4.3						60.5	38.0	1.5	GP	Dark olive-brown, poorly graded GRAVEL with sand
TP- 8,S-1	3.0	3.5	1.8					70.4	17.5	11.5	0.7	GP	Dark brown, poorly graded GRAVEL with cobbles

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

SUMMARY OF
MATERIAL PROPERTIES

PROJECT NO.: 2021-163-21 FIGURE: B-2

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	ATTERBERG LIMITS (%)			% COBBLES	% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
					LL	PL	PI						
TP- 9,S-1	1.0	1.5	3.7						73.4	25.0	1.6	GW	Very dark grayish-brown, well-graded GRAVEL with sand
TP- 9,S-2	4.5	5.0	28.0						31.5	52.1	16.4	SM	Very dark brown, silty SAND with gravel
TP-10,S-1	1.5	2.0	3.1						53.0	46.2	0.8	GP	Olive-brown, poorly graded GRAVEL with sand
TP-10,S-2	13.0	13.5	2.5					57.3	33.4	8.4	0.9	GW	Olive-brown, well-graded GRAVEL with cobbles
TP-11,S-1	1.0	1.5	16.4						56.9	32.8	10.2	GP-GM	Very dark brown, poorly graded GRAVEL with silt and sand
TP-11,S-2	3.0	3.5	6.0						83.6	10.9	5.5	GP-GM	Dark yellowish-brown, poorly graded GRAVEL with silt
TP-12,S-1	1.0	1.5	3.5						80.5	19.0	0.5	GP	Very dark gray, poorly graded GRAVEL with sand
TP-12,S-2	9.0	9.5	3.5						80.2	18.3	1.5	GW	Olive-brown, well-graded GRAVEL with sand
TP-13,S-1	4.0	4.5	3.9					14.7	51.4	30.5	3.4	GP	Dark yellowish-brown, poorly graded GRAVEL with sand and cobbles
TP-13,S-2	10.0	10.5	1.6						80.4	18.9	0.7	GP	Olive-brown, poorly graded GRAVEL with sand
TP-14,S-1	5.0	5.5	1.7					60.9	32.0	6.6	0.6	GP	Dark yellowish-brown, poorly graded GRAVEL with cobbles
TP-14,S-4	12.0	12.5	7.6						40.2	37.6	22.3	GM	Dark yellowish-brown, silty GRAVEL with sand
TP-15,S-1	4.0	4.5	3.8					13.0	67.8	17.9	1.4	GW	Dark olive-brown, well-graded GRAVEL with sand and cobbles
TP-15,S-2	6.0	6.5	2.2						93.6			GP	Light olive-brown, poorly graded GRAVEL
TP-16,S-1	4.0	4.5	14.2						52.0	34.4	13.6	GM	Dark yellowish-brown, silty GRAVEL with sand
TP-17,S-1	6.0	6.5	2.9						58.0	41.5	0.5	GP	Dark olive-gray, poorly graded GRAVEL with sand

Notes:
1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

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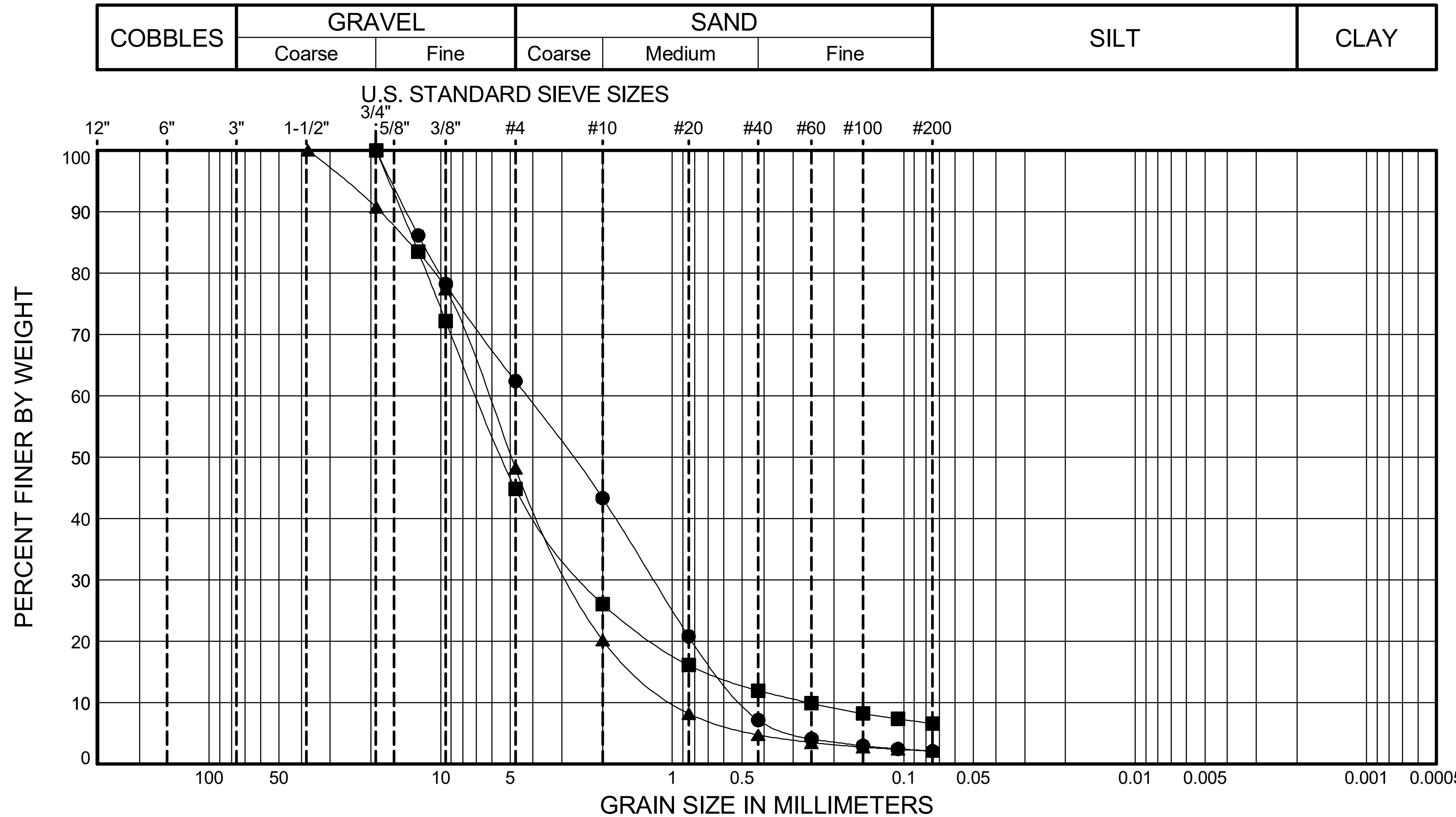
Green to Cedar River Trail
Maple Valley, Washington

SUMMARY OF
MATERIAL PROPERTIES

PROJECT NO.: 2021-163-21 FIGURE: B-3



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	BH-1	S-6	15.0 - 16.5	(SP) Olive-gray, poorly graded SAND with gravel	3		37.6	60.3	2.1
■	BH-1	S-10	30.0 - 31.5	(GP-GM) Light olive-gray, poorly graded GRAVEL with silt and sand	3		55.2	38.2	6.6
▲	BH-2	S-6	15.0 - 16.5	(GW) Dark olive-gray, well-graded GRAVEL with sand	3		51.8	46.1	2.1

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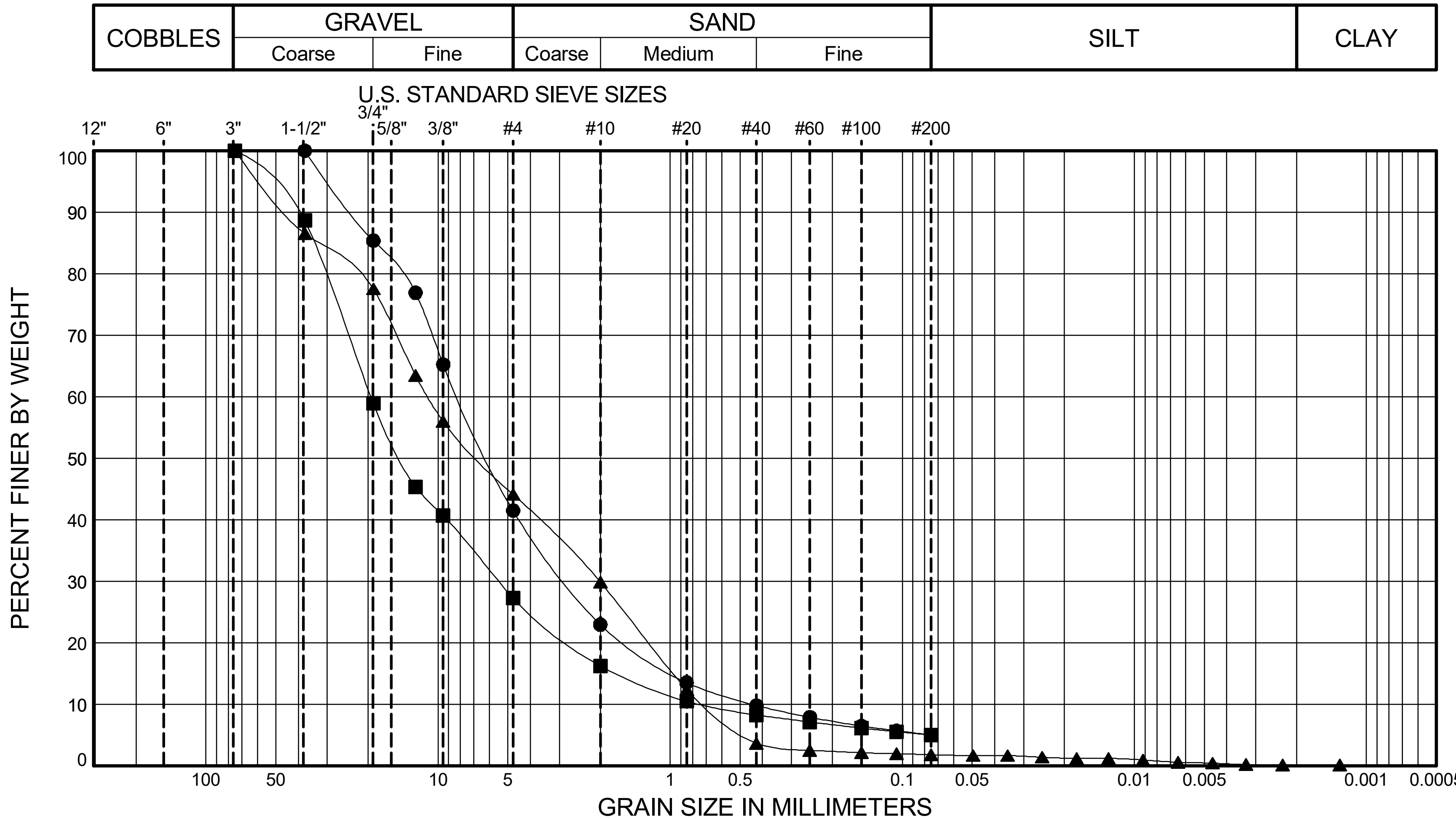
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS OF SOILS METHODS ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-4



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	BH-2	S-8	20.0 - 21.5	(GW-GM) Olive-gray, well-graded GRAVEL with silt and sand	3		58.5	36.5	5.0
■	BH-2	S-10	30.0 - 31.5	(GW-GM) Olive-gray, well-graded GRAVEL with silt and sand	3	0.0	72.7	22.3	5.0
▲	PIT-1	S-2	3.0 - 3.5	(GP) Olive-brown, poorly graded GRAVEL with sand	5	0.0	55.9	42.3	1.8

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Green to Cedar River Trail
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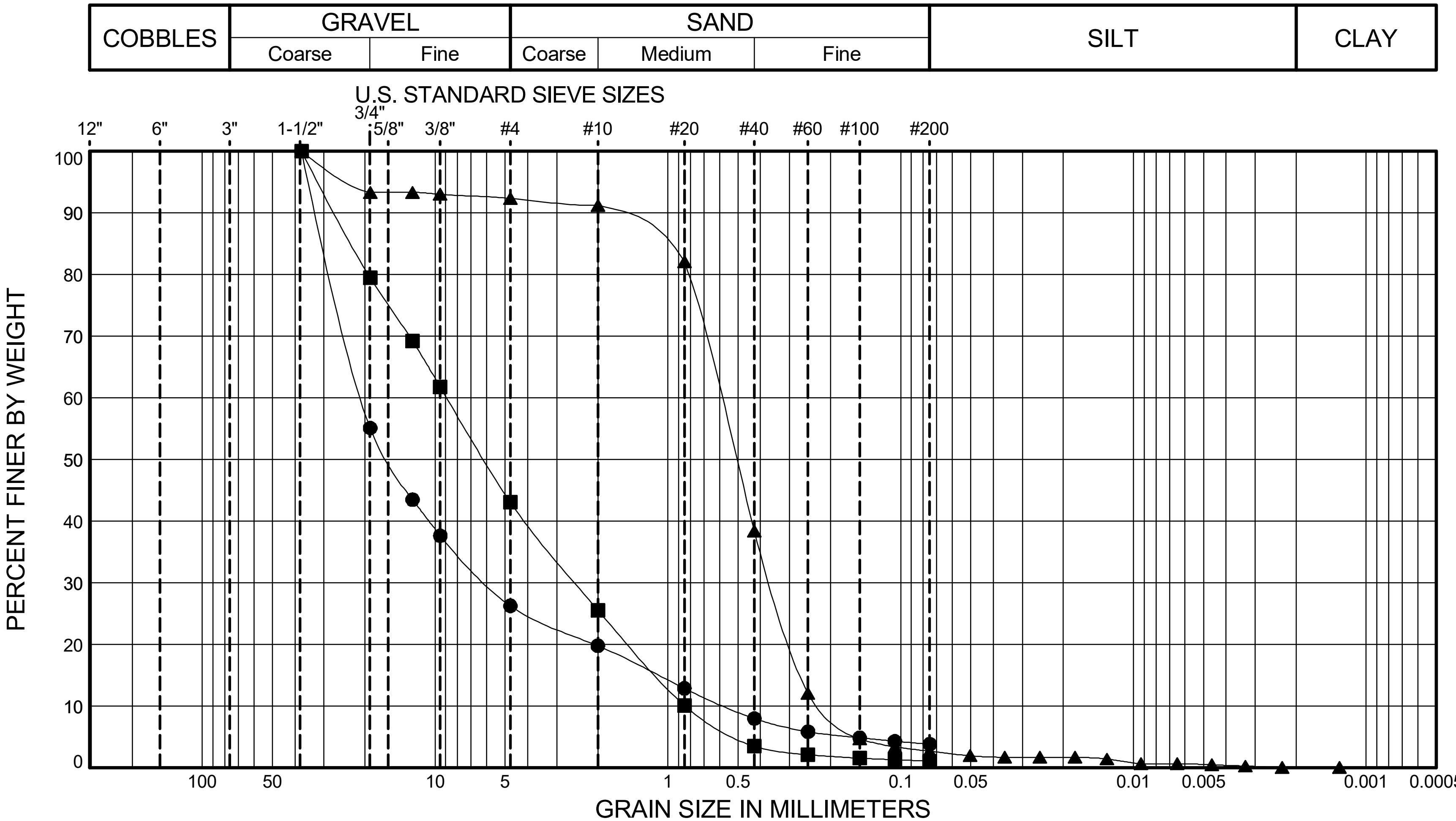
PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-5

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	PIT-2	S-1	4.0 - 4.5	(GP) Dark olive-brown, poorly graded GRAVEL with sand	5		73.8	22.4	3.8
■	PIT-3	S-2	2.7 - 3.2	(GP) Grayish-brown, poorly graded GRAVEL with sand	3		56.9	42.0	1.1
▲	PIT-4	S-3	4.3 - 4.8	(SP) Dark grayish-brown, poorly graded SAND	19		7.7	89.7	2.7

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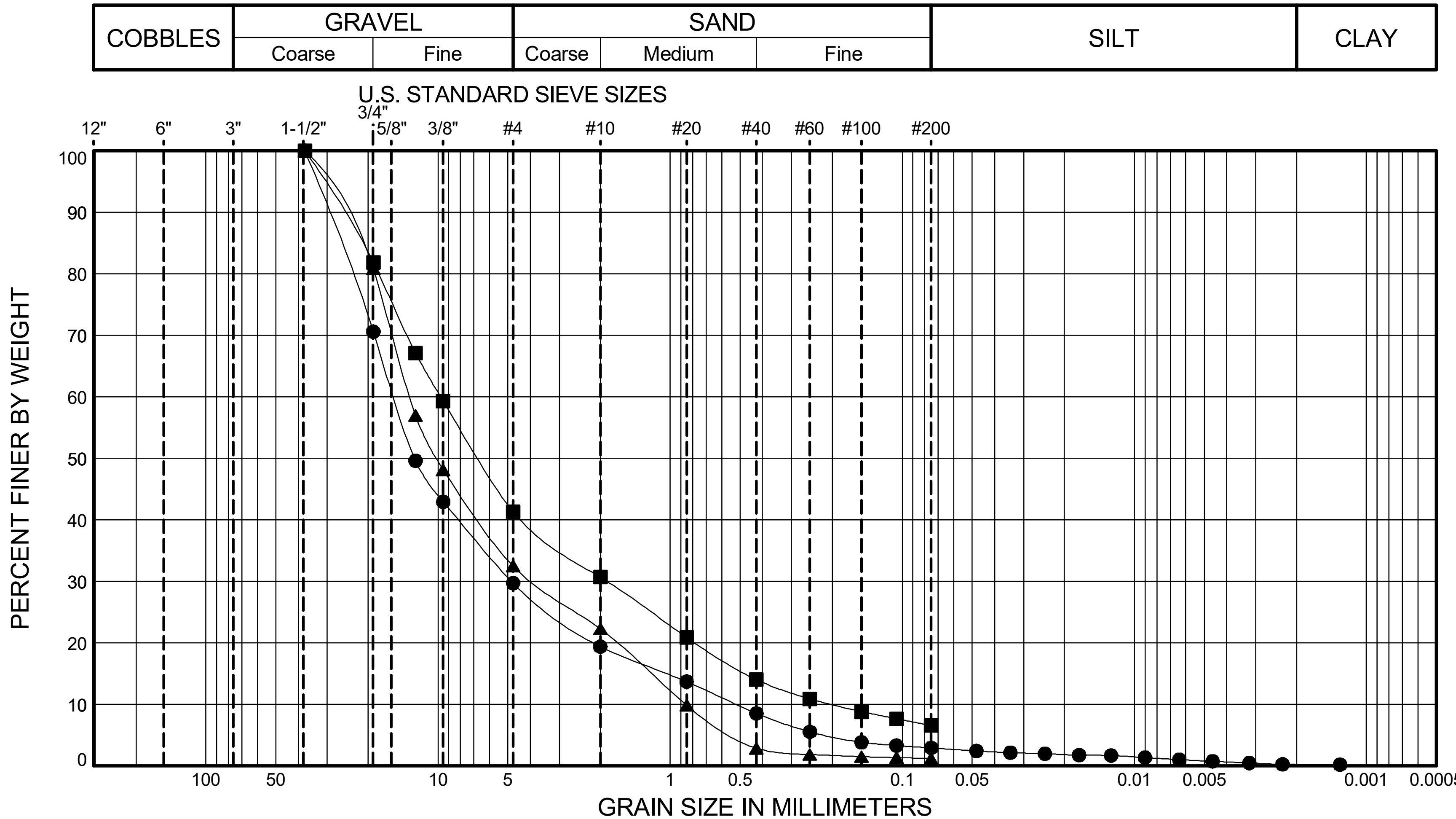
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-6

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	PIT-5	S-2	4.0 - 4.5	(GW) Olive-brown, well-graded GRAVEL with sand	5		70.3	26.8	2.9
■	PIT-5	S-3	7.0 - 7.5	(GW-GM) Brown, well-graded GRAVEL with silt and sand	11		58.7	34.8	6.5
▲	PIT-6	S-2	4.0 - 4.5	(GW) Brown, well-graded GRAVEL with sand	4		67.5	31.3	1.2

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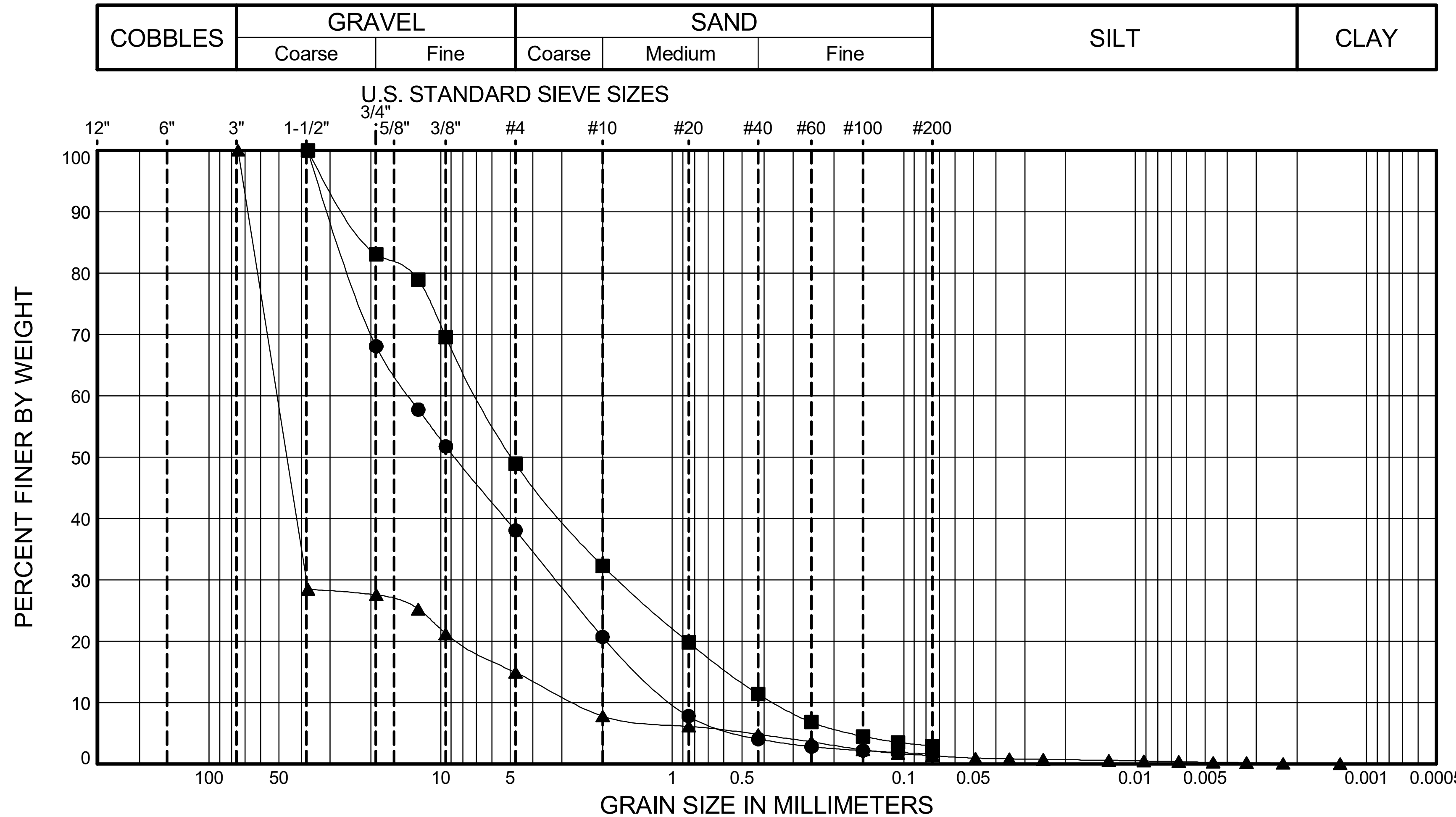
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-7

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	PIT-7	S-1	4.5 - 5.0	(GP) Olive-gray, poorly graded GRAVEL with sand	5		62.0	36.5	1.6
■	PIT-8	S-2	3.3 - 3.8	(GW) Olive-brown, well-graded GRAVEL with sand	7		51.1	46.0	2.9
▲	TP- 1	S-1	1.0 - 1.5	(GP) Dark brown, poorly graded GRAVEL	3	0.0	85.1	13.6	1.3

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Green to Cedar River Trail
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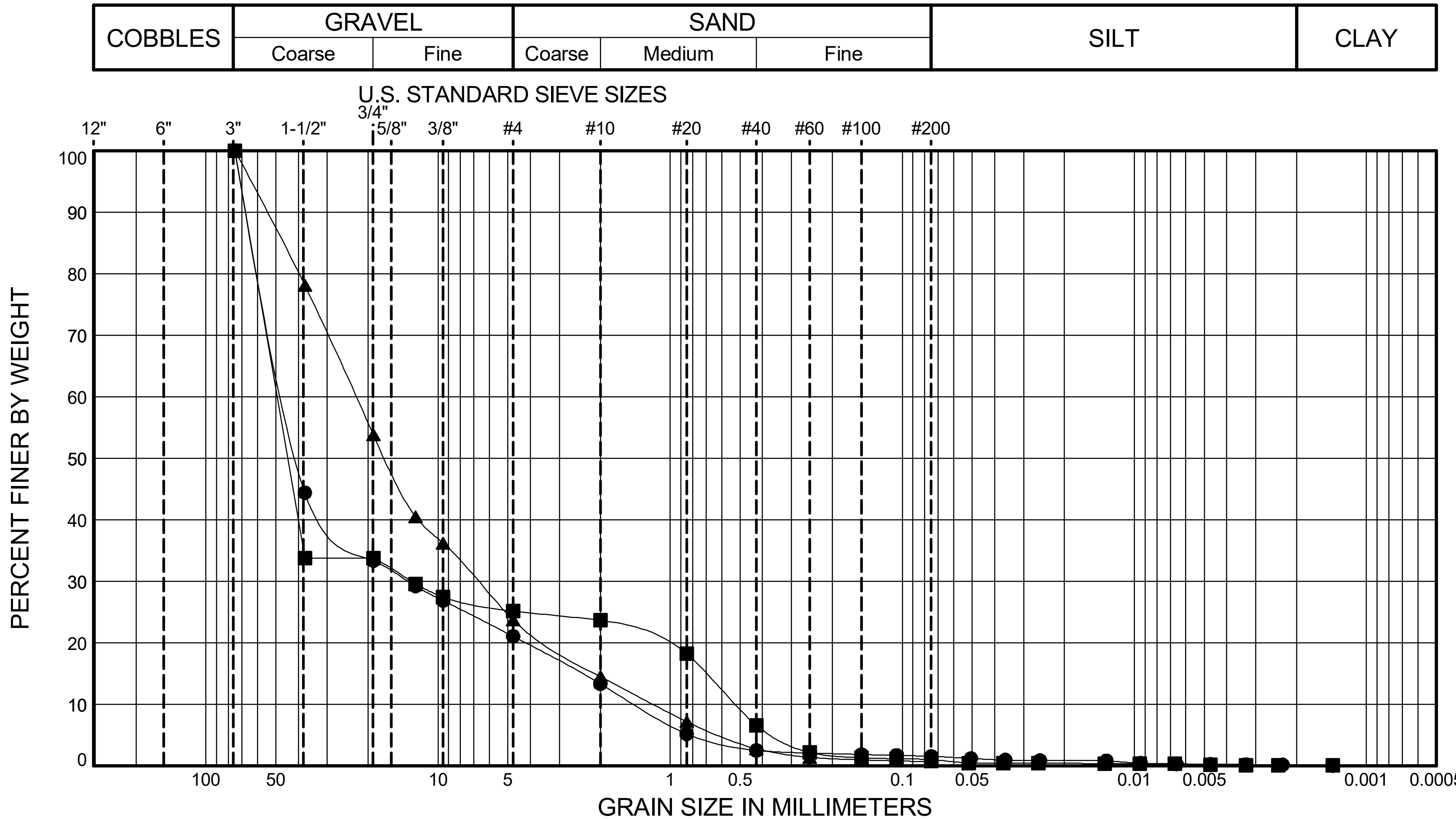
PARTICLE-SIZE ANALYSIS OF SOILS METHODS ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-8



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP- 1	S-2	3.0 - 3.5	(GW) Dark brown, well-graded GRAVEL with sand	3	0.0	78.9	19.5	1.5
■	TP- 2	S-1	3.0 - 3.5	(GP) Dark brown, poorly graded GRAVEL with sand	8	0.0	74.9	24.1	1.0
▲	TP- 2	S-2	14.0 - 14.5	(GW) Dark brown, well-graded GRAVEL with sand	2	0.0	76.3	23.1	0.7

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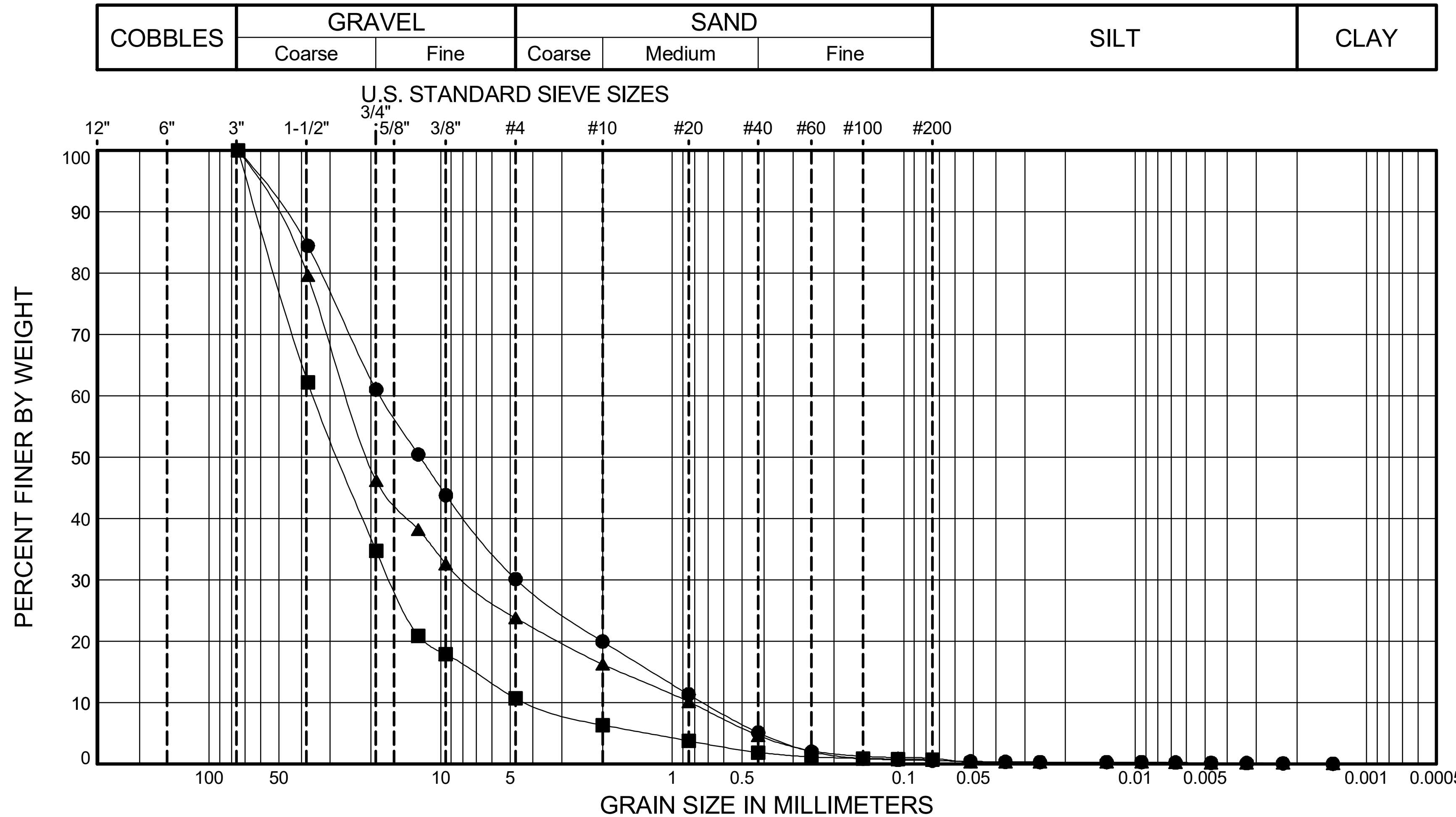
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-9



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP- 3	S-1	3.0 - 3.5	(GW) Dark brown, well-graded GRAVEL with sand	3	0.0	69.9	29.6	0.5
■	TP- 3	S-2	7.0 - 7.5	(GW) Dark brown, well-graded GRAVEL	2	0.0	89.3	9.9	0.7
▲	TP- 4	S-1	3.5 - 4.0	(GW) Yellowish-brown, well-graded GRAVEL with sand	1	0.0	76.2	22.9	0.9

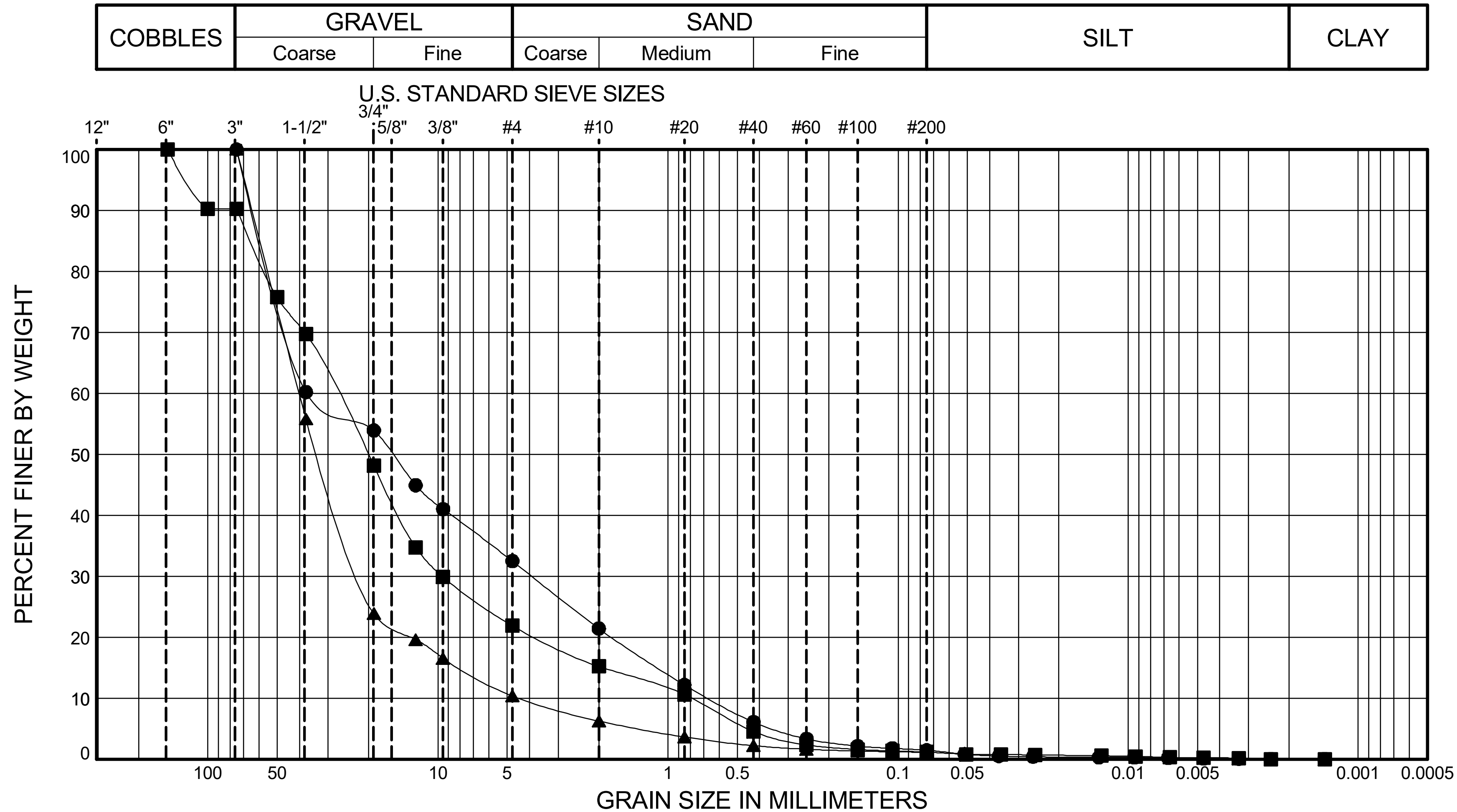
HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-10

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP- 4	S-2	7.0 - 7.5	(GP) Dark brown, poorly graded GRAVEL with sand	3	0.0	67.5	31.0	1.5
■	TP- 5	S-1	4.5 - 5.0	(GP) Dark olive-brown, poorly graded GRAVEL with sand and cobbles	3	9.7	68.4	20.7	1.2
▲	TP- 5	S-2	14.0 - 14.5	(GW) Dark brown, well-graded GRAVEL	3	0.0	89.6	9.2	1.1

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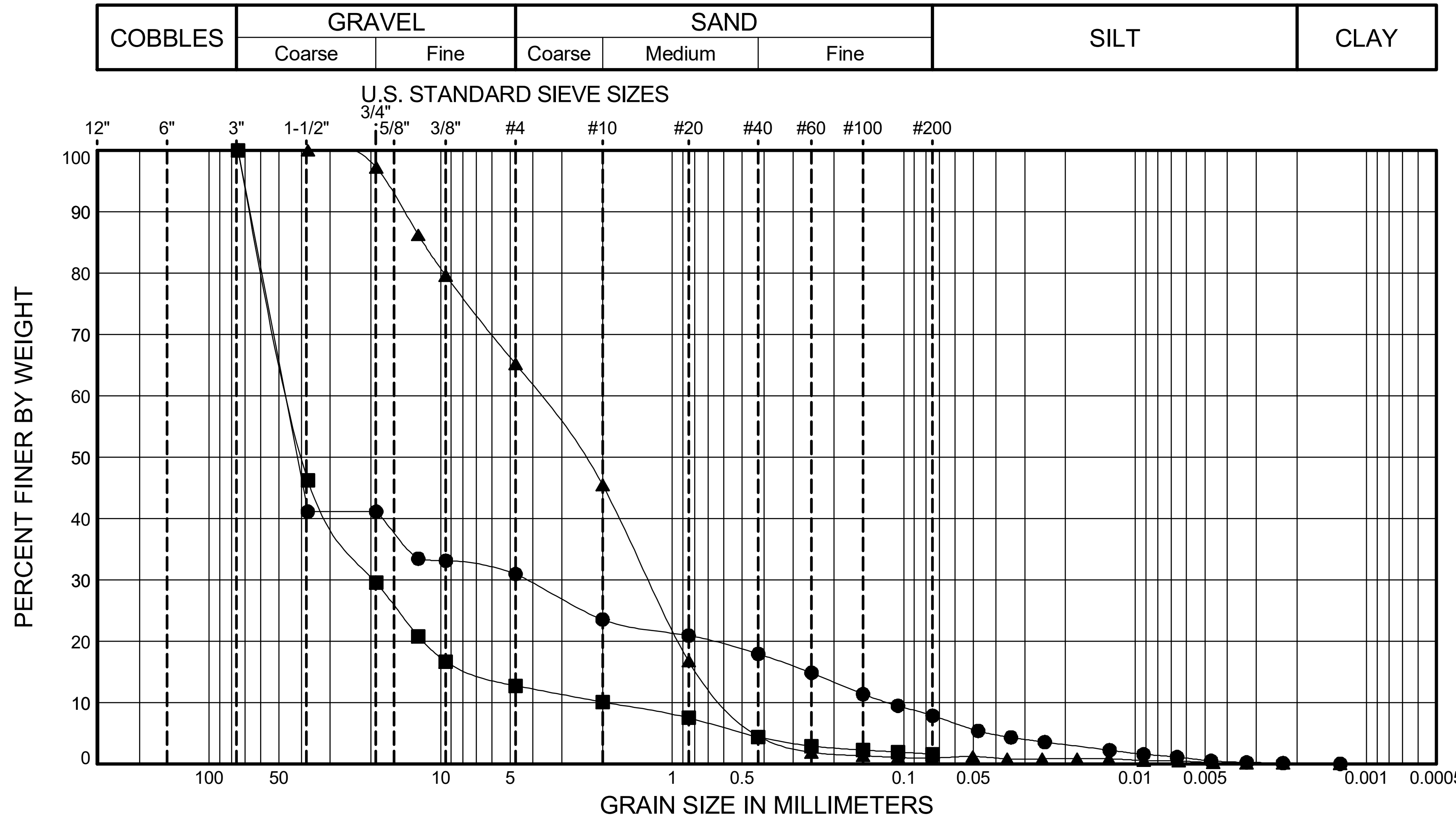
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-11



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP- 6	S-1	3.0 - 3.5	(GP-GM) Yellowish-brown, poorly graded GRAVEL with silt and sand	8	0.0	69.1	23.1	7.9
■	TP- 6	S-2	6.0 - 6.5	(GP) Yellowish-brown, poorly graded GRAVEL	2	0.0	87.2	11.1	1.6
▲	TP- 6	S-3	8.0 - 8.3	(SP) Dark gray, poorly graded SAND with gravel	7		34.8	64.3	0.9

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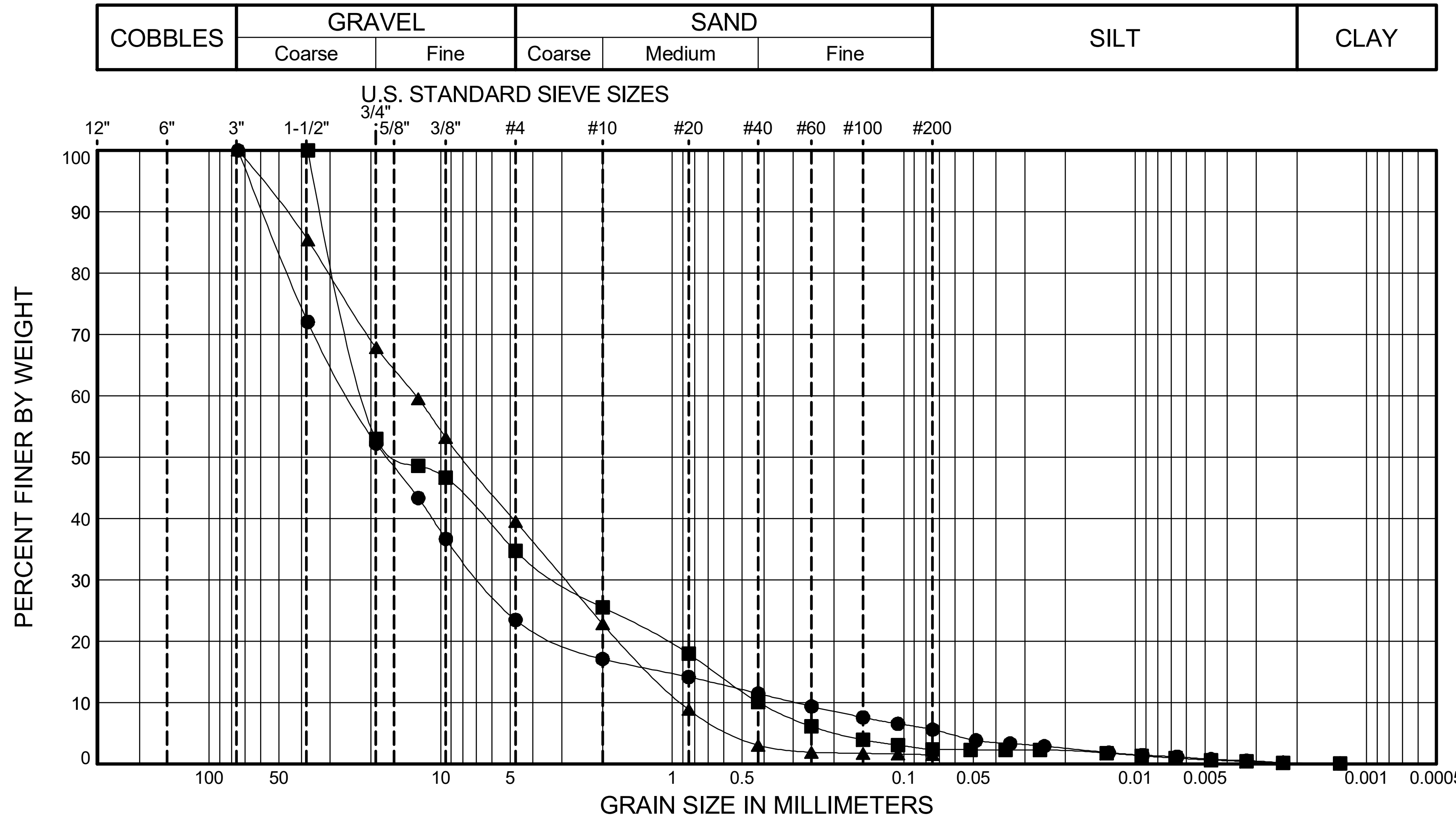
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS OF SOILS METHODS ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-12



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP- 7	S-1	2.0 - 2.5	(GP-GM) Dark yellowish-brown, poorly graded GRAVEL with silt and sand	7	0.0	76.5	17.9	5.6
■	TP- 7	S-2	4.0 - 4.5	(GW) Light olive-brown, well-graded GRAVEL with sand	3		65.3	32.3	2.4
▲	TP- 7	S-3	11.0 - 11.5	(GP) Dark olive-brown, poorly graded GRAVEL with sand	4	0.0	60.5	38.0	1.5

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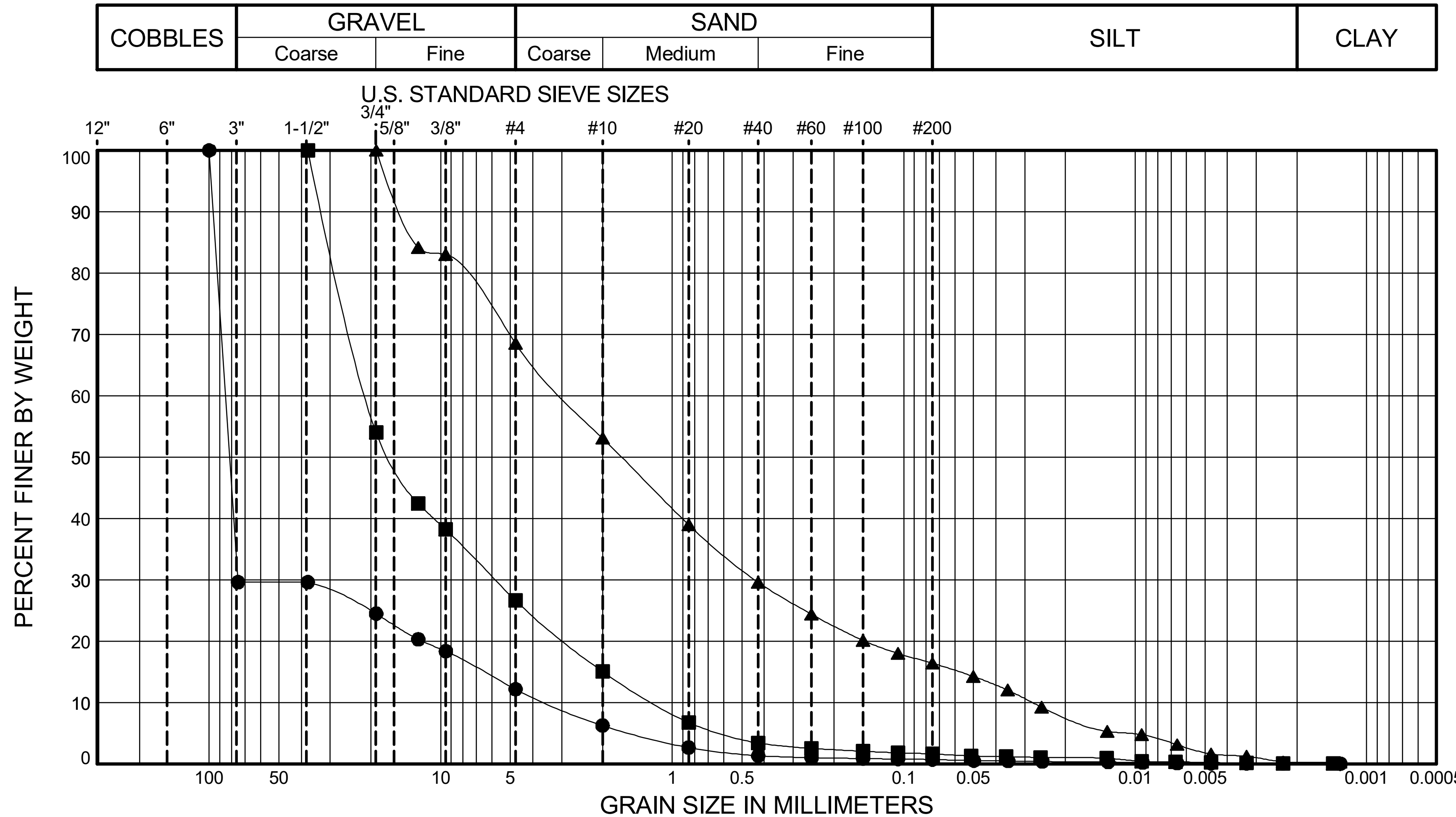
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-13

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP- 8	S-1	3.0 - 3.5	(GP) Dark brown, poorly graded GRAVEL with cobbles	2	70.4	17.5	11.5	0.7
■	TP- 9	S-1	1.0 - 1.5	(GW) Very dark grayish-brown, well-graded GRAVEL with sand	4		73.4	25.0	1.6
▲	TP- 9	S-2	4.5 - 5.0	(SM) Very dark brown, silty SAND with gravel	28		31.5	52.1	16.4

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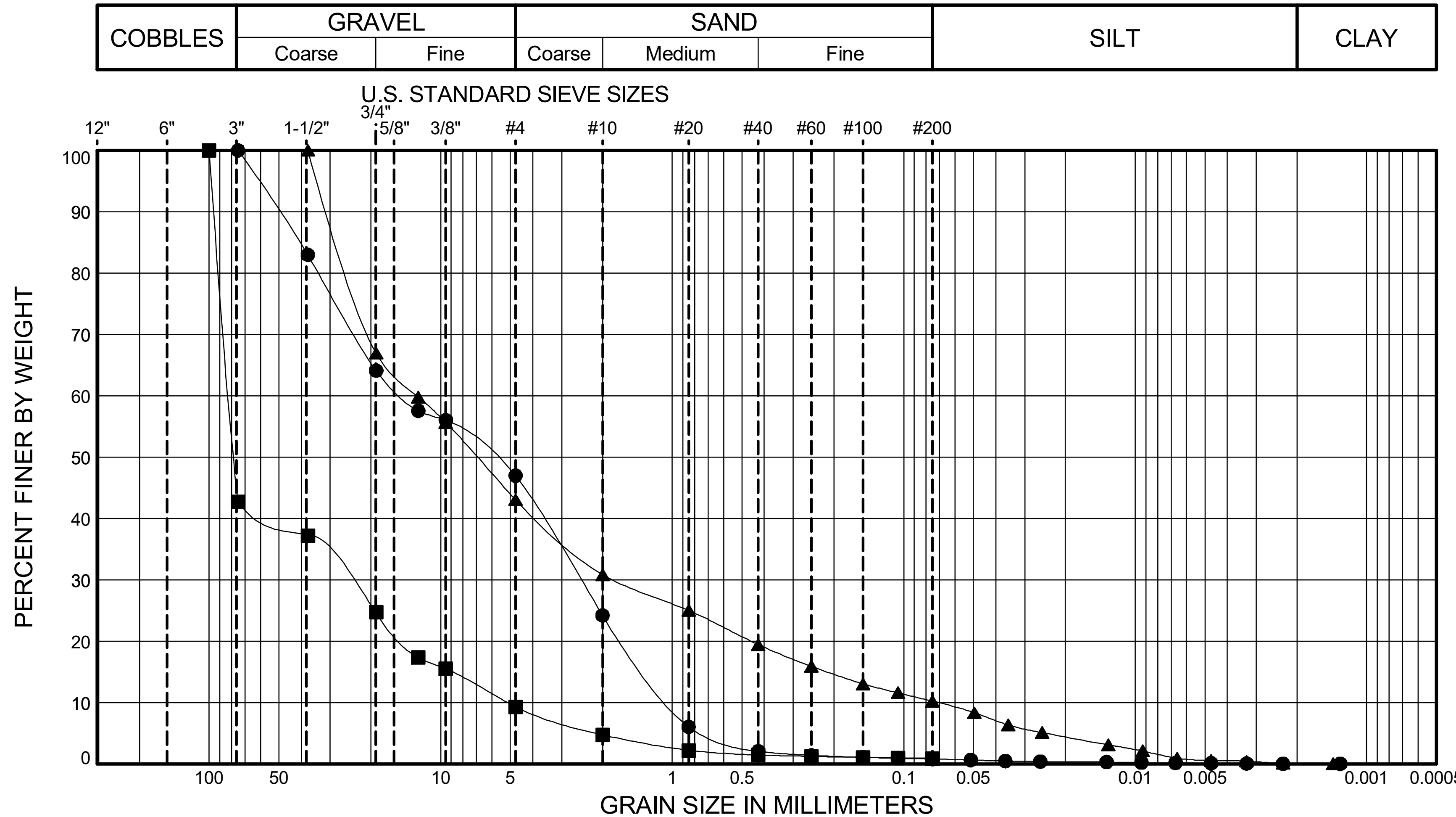
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-14

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



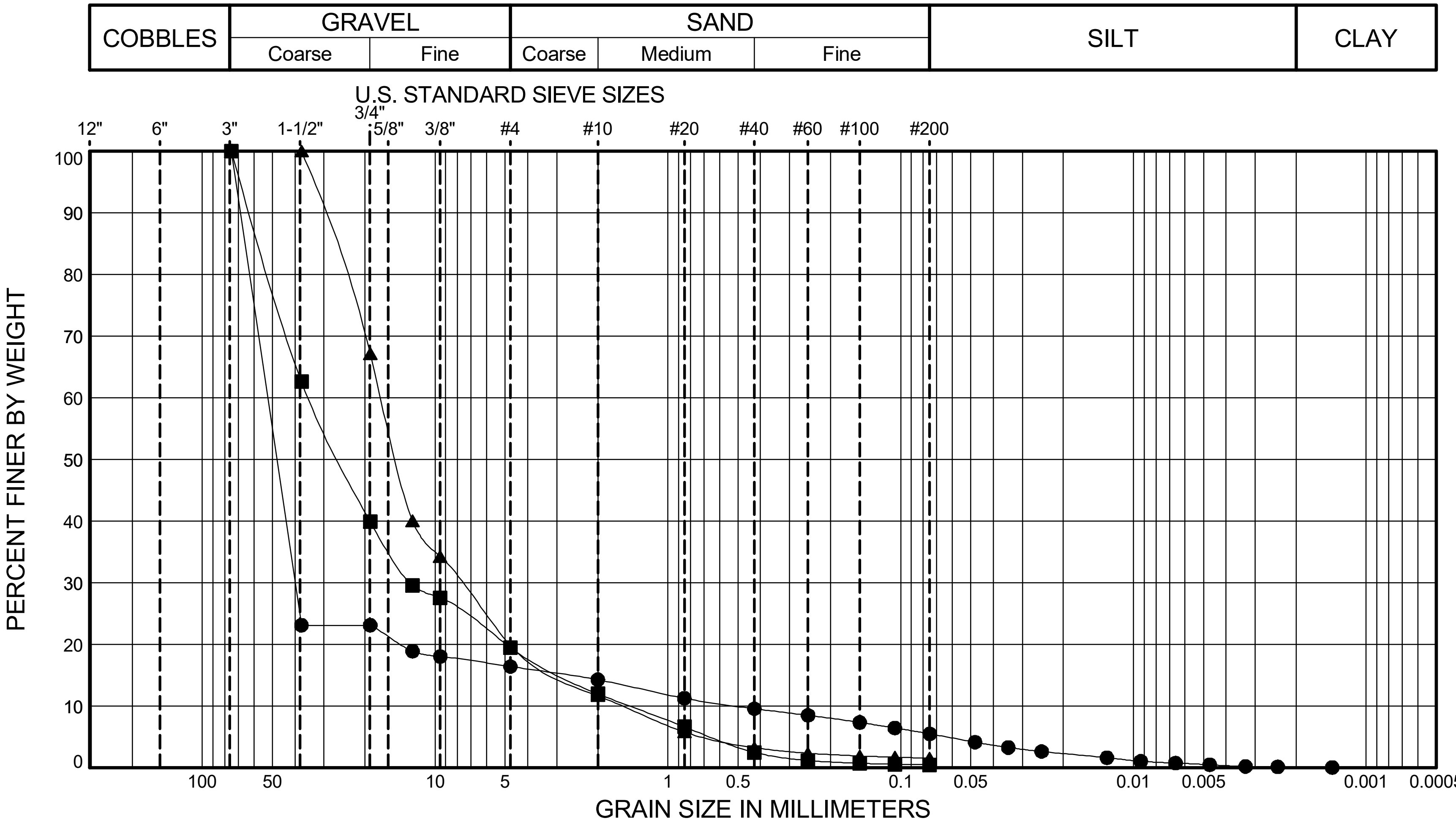
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP-10	S-1	1.5 - 2.0	(GP) Olive-brown, poorly graded GRAVEL with sand	3	0.0	53.0	46.2	0.8
■	TP-10	S-2	13.0 - 13.5	(GW) Olive-brown, well-graded GRAVEL with cobbles	3	57.3	33.4	8.4	0.9
▲	TP-11	S-1	1.0 - 1.5	(GP-GM) Very dark brown, poorly graded GRAVEL with silt and sand	16		56.9	32.8	10.2

HWA GEOSCIENCES INC.

Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-15



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP-11	S-2	3.0 - 3.5	(GP-GM) Dark yellowish-brown, poorly graded GRAVEL with silt	6	0.0	83.6	10.9	5.5
■	TP-12	S-1	1.0 - 1.5	(GP) Very dark gray, poorly graded GRAVEL with sand	4	0.0	80.5	19.0	0.5
▲	TP-12	S-2	9.0 - 9.5	(GW) Olive-brown, well-graded GRAVEL with sand	3		80.2	18.3	1.5

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Green to Cedar River Trail
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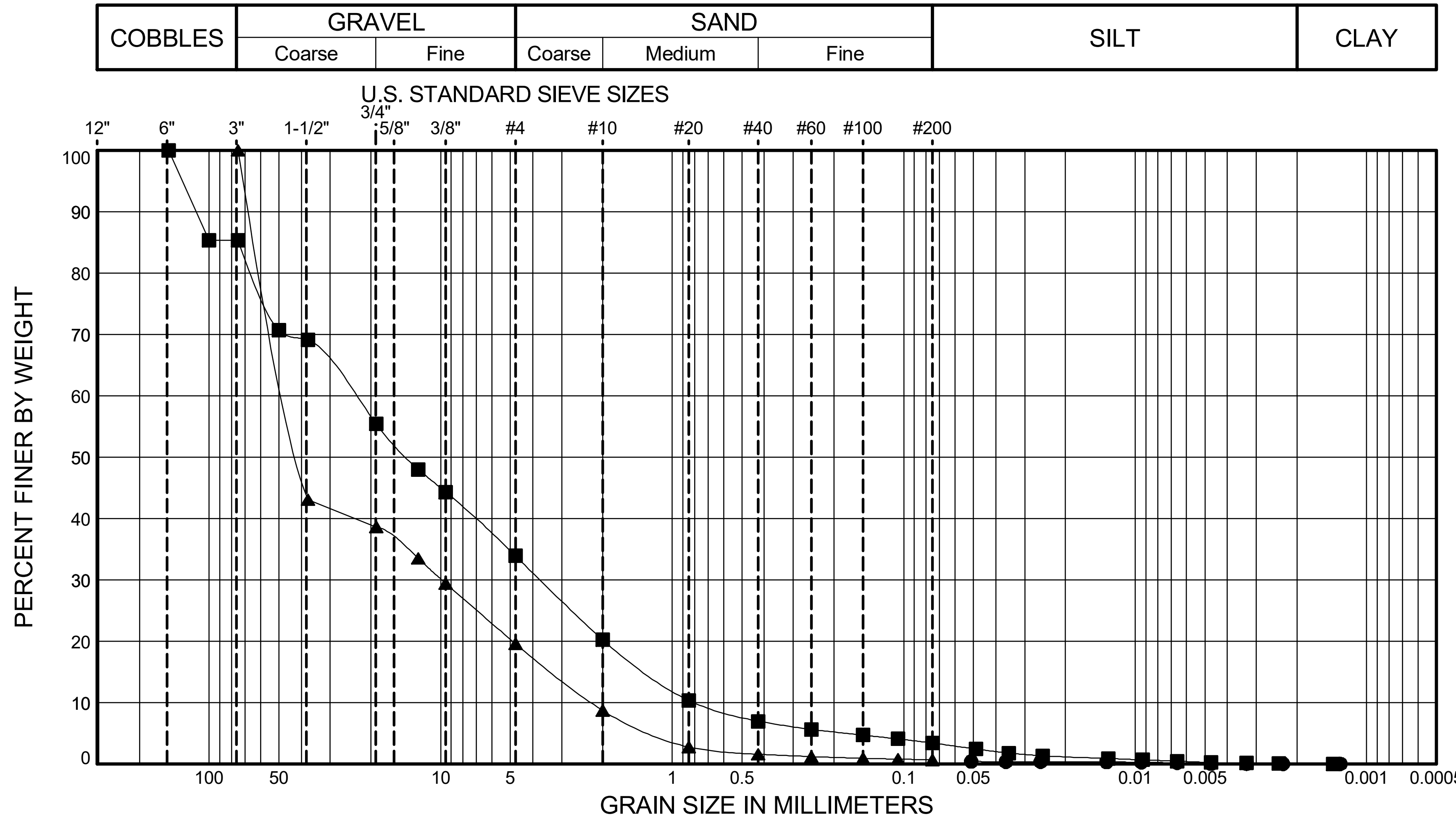
PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-16

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP-12			() ,					
■	TP-13	S-1	4.0 - 4.5	(GP) Dark yellowish-brown, poorly graded GRAVEL with sand and cobbles	4	14.7	51.4	30.5	3.4
▲	TP-13	S-2	10.0 - 10.5	(GP) Olive-brown, poorly graded GRAVEL with sand	2	0.0	80.4	18.9	0.7

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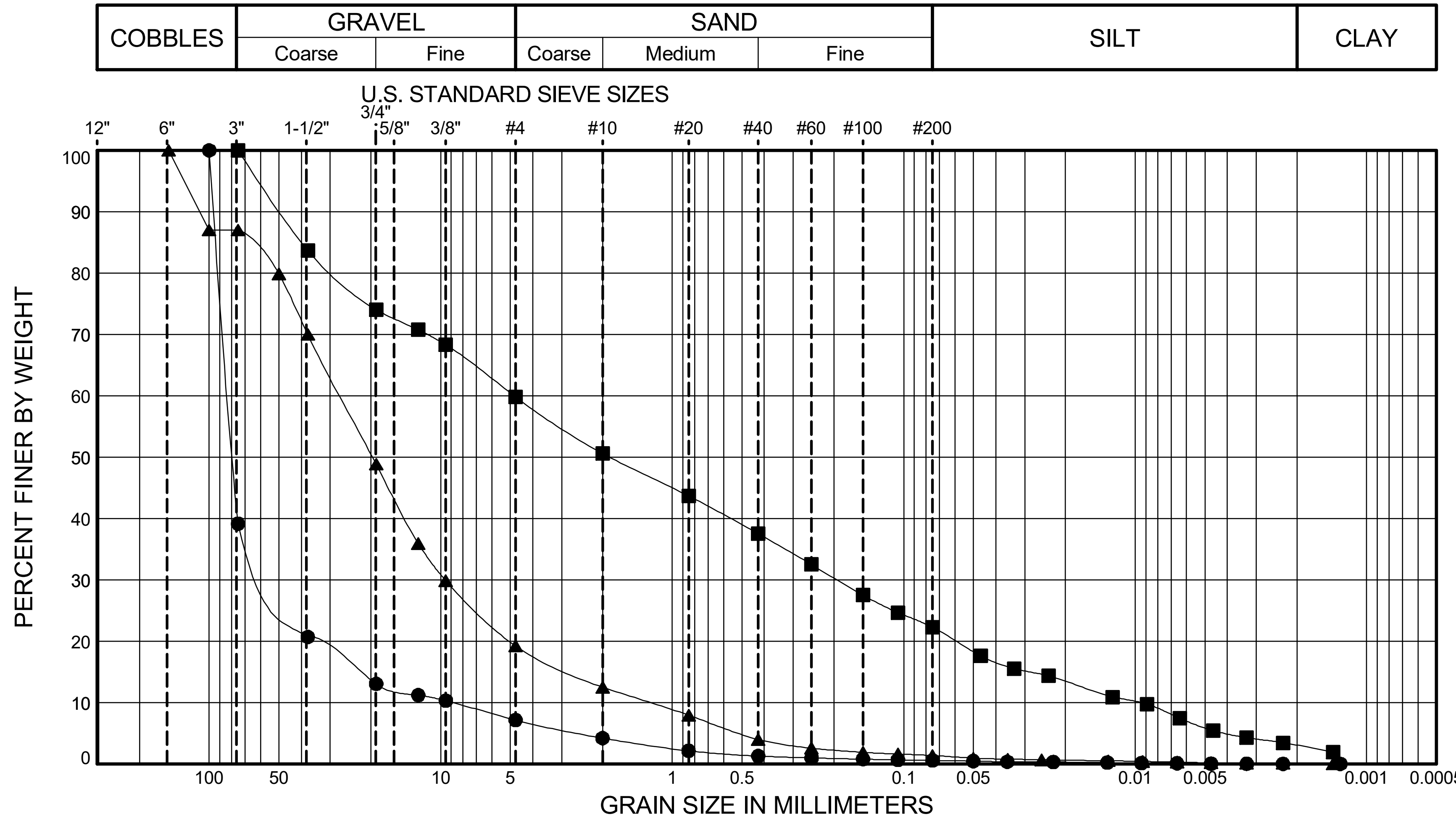
Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-17



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP-14	S-1	5.0 - 5.5	(GP) Dark yellowish-brown, poorly graded GRAVEL with cobbles	2	60.9	32.0	6.6	0.6
■	TP-14	S-4	12.0 - 12.5	(GM) Dark yellowish-brown, silty GRAVEL with sand	8	0.0	40.2	37.6	22.3
▲	TP-15	S-1	4.0 - 4.5	(GW) Dark olive-brown, well-graded GRAVEL with sand and cobbles	4	13.0	67.8	17.9	1.4

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Green to Cedar River Trail
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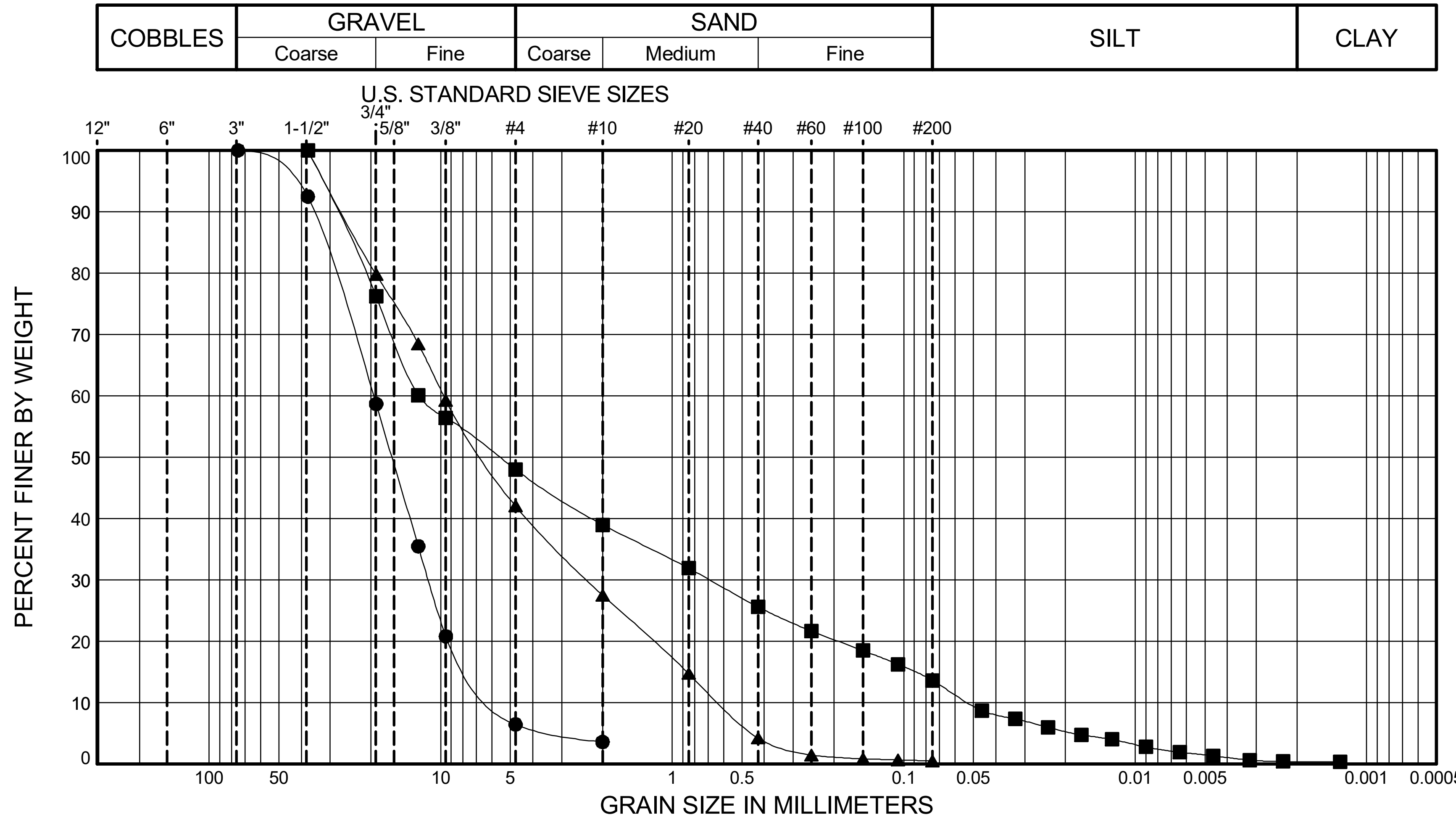
PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-18

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	% Cobble	% Gravel	% Sand	% Fines
●	TP-15	S-2	6.0 - 6.5	(GP) Light olive-brown, poorly graded GRAVEL	2	0.0	93.6		
■	TP-16	S-1	4.0 - 4.5	(GM) Dark yellowish-brown, silty GRAVEL with sand	14		52.0	34.4	13.6
▲	TP-17	S-1	6.0 - 6.5	(GP) Dark olive-gray, poorly graded GRAVEL with sand	3		58.0	41.5	0.5

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Green to Cedar River Trail
Maple Valley, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS METHODS
ASTM D6913/D7928

PROJECT NO.: 2021-163-21 FIGURE: B-19

HWAGRSZ3_COMBINED 2021-163.GPJ 5/15/23

APPENDIX C

PREVIOUS GEOTECHNICAL ENGINEERING REPORT BY OTHERS TO SUPPORT PRELIMINARY PEDESTRIAN BRIDGE

Report of Geotechnical Engineering Services
King County Parks
South Segment Green to Cedar Rivers Regional Trail Project
Task 200, Subtask 200.01B
Proposed BNSF Pedestrian Bridge
Black Diamond Area, King County, Washington

January 11, 2019
ICE File No. 0105-022

Prepared For:
Parametrix, Inc.

Prepared By:
Icicle Creek Engineers, Inc.

January 11, 2019

Jennifer Dvorak, PE, Senior Engineer
Parametrix, Inc.
1019 39th Avenue SE, Suite 100
Puyallup, Washington 98374

Report
Geotechnical Engineering Services
King County Parks
South Segment Green to Cedar Rivers
Regional Trail Project
Task 200, Subtask 200.01B
Proposed BNSF Pedestrian Bridge
Black Diamond Area, King County, Washington
ICE File No. 0105-022

1.0 INTRODUCTION

This report presents the results of Icicle Creek Engineers' (ICE's) geotechnical evaluation of the proposed BNSF Pedestrian Bridge which is a component of the King County Parks' South Segment Green to Cedar Rivers Regional Trail project near Black Diamond, in King County, Washington. The location of the proposed BNSF Pedestrian Bridge is shown on the Vicinity Map, Figure 1.

Our services were provided in general accordance with Task 200, Subtask 200.01B – BNSF Pedestrian Bridge as described in Exhibit A of Parametrix Subconsultant Agreement for Professional Services Parametrix, and with Amendment No. 02 dated July 5, 2018. This geotechnical evaluation is intended to provide 100 percent geotechnical recommendations for the proposed BNSF Pedestrian Bridge.

2.0 BACKGROUND INFORMATION

Jennifer Dvorak, PE and Mallory Miller, PE with Parametrix provided ICE with the following preliminary project plans and geotechnical information regarding the proposed BNSF Pedestrian Bridge:

- Parametrix, April 12, 2018, *BNSF ROW Skewed Crossing, GTCR Trail*, scale 1 inch = 50 feet.
- Parametrix, April 12, 2018, *BNSF Skewed Crossing Profile, GTCR Trail*, horizontal scale 1 inch = 50 feet, vertical scale 1 inch = 10 feet.
- GeoEngineers, Inc. (GEI), July 24, 2015, *Geotechnical, Geologic and Environmental Services, Green to Cedar Rivers Regional Trail Project, King County, Washington*, prepared for King County Parks, 422 pages, includes four test pits that were excavated in the abutment areas.
- Trantech Engineering LLC (Trantech), undated, *BNSF Crossing, Bridge Plan & Elevation*, one sheet.
- Trantech, undated, *BNSF Crossing, Typical Section & Guardrail Details*, one sheet.

ICE obtained the following document with subsurface information on an adjacent project.

- Washington State Department of Transportation (WSDOT), September 21, 1988, CS 1734, SR-169, L-8872, BN RR Overcrossing Bridge, Bridge 169/12 Widening, Foundation Recommendations, obtained from the Washington State Department of Natural Resources (DNR), Washington Geologic Information Portal (https://www.dnr.wa.gov/geologyportal).

3.0 PROJECT DESCRIPTION

We understand that King County Parks plans a pedestrian bridge overcrossing of the BNSF (active) rail line. This location of the proposed trail and pedestrian bridge is the site of a former bridge for an abandoned north-south trending rail line. The former rail line bridge has been removed, with the abutment and intercrossing support column remaining. The proposed BNSF Pedestrian Bridge will not utilize the historic abutment or column foundations.

The location of the proposed BNSF Pedestrian Bridge is shown on the Plan and Profile, Figure 2.

Based on our review of preliminary design plans by Parametrix and TranTech, the single-span steel truss bridge is about 177-feet long (end to end) with an 18-foot wide solid-surface concrete deck. The width will also include an approximately 1.7-foot-wide curb and rail on either side; the truss structure to support the deck, curb and rail will be about 24-feet wide. Swarna Raju of Trantech indicated by email on August 22, 2018 that each abutment will be supported on two, 3- or 4-foot diameter drilled shafts. Unfactored loads assuming two drilled shafts per abutment include the following:

Axial Loads per Drilled Shaft
Dead Load (DL) – 180K pounds
Live Load (LL) – 125K pounds
Lateral Loads per Drilled Shaft
Wind (WL) – 19K pounds (transverse)
Earthquake (EQ) – 44K pounds
Other
AASHTO Pedestrian Load – 90 pounds per square foot (psf) or HS20 Truck

At this time, the use of a Soldier Pile Wall wingwall (one at each abutment) will be used to restrict new fill from encroaching into the BNSF right-of-way (ROW) as shown on Figure 2. It is possible that a Structural Earth Wall (SEW) could be used for this purpose depending on the progression of the design requirements for grading and ROW restrictions.

We understand that the approaches to the new bridge will be fill. Some fill already exists for the rail line, but up to 8 feet of new fill will be required to achieve the high surface elevation and to achieve the proposed trail width. SEWs or an open slope may be used to provide the trail width. The SEW or open slope will depend on if there is sufficient King County ROW to accommodate an open fill slope.

Construction of the proposed BNSF Pedestrian Bridge, as currently planned, will require rerouting an existing 12-inch-diameter water line as shown on Figure 2. We understand that the rerouting of the water line will be done by others.

4.0 GEOLOGIC SETTING

Based on regional geologic mapping by the US Geological Survey (USGS, 1965, Geologic Map of the Black Diamond Quadrangle, King County, Washington, Geologic Quadrangle Map, GQ-407), the proposed BNSF Pedestrian Bridge site is underlain by Proglacial Stratified Drift consisting of sand, gravel, cobbles and occasional boulders with variable amounts of silt.

5.0 SITE CONDITIONS

5.1 SURFACE CONDITIONS

On December 21, 2017 and August 17, 2018, Brian Beaman of ICE completed site visits to observe the surface conditions of the proposed BNSF Pedestrian Bridge overcrossing of the active BNSF rail line. At the time of our December (early morning) site visit the weather was cold (upper 20s) and cloudy; the weather during our August site visit (mid-morning) was warm (70s) and clear. During our August 2018 site visit we met with Martin Page, PE, LEG and Justin Cook, PE, of Shannon & Wilson, ICE's geotechnical subconsultant.

Based on our review of historical topographic maps (US Geological Survey – USGS, https://ngmdb.usgs.gov/topoview/, 1897, 1940 and 1949) the abandoned north-south trending rail line was constructed prior to 1897. The active east-west trending BNSF rail line was constructed sometime between 1940 and 1949. State Route (SR) 169 (also referred to as the "Maple Valley – Black Diamond Road SE") was constructed sometime before 1936 based on our review of the historical aerial photographs.

The proposed BNSF Pedestrian Bridge site is bordered by forested areas to the north, west and south. The area to the east is occupied by SR 169 with partially-cleared shoulder and ROW areas. SR 169 crosses the active BNSF rail line as a concrete bridge overcrossing.

The proposed trail and BNSF Pedestrian Bridge at this location follows the previously described north-south trending abandoned rail line. The steel tracks and wood ties have been removed with the surface covered with crushed rock for interim trail use by hikers, runners and bicycles. The abandoned north abutment is blocked by a high mound of fill; the interim trail informally diverts off the rail line down to the west, crosses the active rail line, then ascends the south abutment to rejoin the abandoned rail line. The end of the abandoned south abutment is also blocked by a smaller mound of fill soil.

The abandoned concrete columns and abutment foundations for the former rail line bridge are present within the "cut" area for the active rail line. No other structures are visible related to the former bridge.

Based on our site observations, it appears that the active rail line is within an existing cut that is about 10 feet below the nearly-level natural ground surface that surrounds this area. In order to obtain clearance to cross the active rail line, it appears that the former rail line was constructed on a prism of fill (about 10-feet thick) to further elevate the grade of the former bridge crossing abutment. Soils exposed in the oversteepened abutment areas consist of sand and gravel with occasional cobbles. It appears that the native soil and fill soil are of similar consistency (sand, gravel and cobbles).

No groundwater seepage or areas of standing water were observed in the proposed BNSF Pedestrian Bridge overcrossing area or adjacent areas at the time of our December 2017 site visit.

Signage in the field and the current project plans show an existing 12-inch-diameter water line paralleling the active rail line (at an oblique angle to the former rail line) as shown on Figure 2. The water line will be relocated by others to provide space for the bridge abutments.

5.2 SUBSURFACE CONDITIONS

No explorations were completed by ICE for this geotechnical evaluation. Four test pit explorations were completed by GEI (2015) in the abutment areas as part of a separate study. In addition, three relatively deep (about 50-feet deep) test borings were completed by WSDOT in 1988 when the SR 169 bridge was widened/replaced (WSDOT, 1988). The approximate locations of the GEI (2015) test pits (Test Pits TP-1 through TP-4) and the WSDOT test borings (Boring H-1-88, H-2-88 and H-3-88) are shown on Figure 2 and are included in Attachment A (GEI 2015 Test Pits) and Attachment B (WSDOT September 21, 1988 Letter with Test Boring Logs and Cross-Section).

During our December 2017 site visit, we attempted to find the surface location of the four test pits that were completed by GEI (2015). We observed surface evidence (bare soil) of Test Pit TP-1 adjacent to the south abutment and possible surface evidence of Test Pit TP-2 (about 15 feet northwest of Test Pit TP-1). No surface evidence of Test Pits TP-3 and TP-4 (adjacent to the northeast side of the north abutment) was observed.

Based on the GEI (2015) mapped locations of the test pits and our field observation, it appears that the test pits were excavated in the lower, undisturbed ground areas rather than in the abandoned rail line fill embankment.

As previously described, WSDOT (1988) completed three relatively deep test borings for the widening/replacement of the SR 169 bridge. SR 169 parallels the abandoned rail line and is similar in configuration to the proposed BNSF Pedestrian Bridge, with filled approaches, overcrossing of the active rail line, and similar geologic conditions; SR 169 is located about 160 feet southeast of the proposed BNSF Pedestrian Bridge.

The test pits completed by GEI (2015) are useful in describing the native, undisturbed soil conditions (Proglacial Stratified Drift) that underlie the fill embankment for the former rail line/trail. In summary, the test pits encountered medium dense to very dense sand and gravel with variable amounts of silt and cobbles to the completion depth of the test pits at 7 to 7½ feet. No groundwater was observed in the GEI (2015) test pits. Slight to moderate caving of the test pits walls was observed in Test Pits TP-1 and TP-4.

The WSDOT (1988) test borings described "cleaner" (less fines) sand and gravel. As a generality, WSDOT (1988) describes the soil conditions at the SR 169 abutment sites as "*15 feet of medium dense to loose sands and gravels with cobbles and boulders underlain by 25 to 35 feet of dense slightly silty to silty sandy gravel*." Though not specifically defined by WSDOT (1988) we suspect that the upper 15 feet of soil at the SR 169 abutment sites consist of fill.

In summary, based on our review of available information (GEI, 2015 and WSDOT, 1998) and our experience in this general area, we expect the abutments to be underlain by about 10 feet of existing fill (placed over 100 years ago) consisting of loose sand and gravel with silt, cobbles and boulders underlain by native soils (ProGlacial Stratified Drift) consisting of medium dense to dense sand and gravel with variable amounts of silt, cobbles and boulders.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Based on our site observations, review of the GEI (2015) test pit logs and the WSDOT (1988) boring logs, and our general experience in this area of east King County, we expect that the surficial +/- 10 feet of fill at the north and south proposed BNSF Pedestrian Bridge abutments is underlain by loose to medium dense fill consisting of sand and gravel with variable amounts of silt and cobbles. We expect that the character of the fill will vary.

The underlying native undisturbed soil (Proglacial Stratified Drift) consists of a similar soil, but in a medium dense to very dense condition. Our conclusions regarding the character of the fill and native soils are primarily based on the results of the GEI (2015) test pit explorations and the WSDOT (1988) test borings. Because the character of the existing abutment fill is unknown, foundation support for the new bridge abutments should extend through the fill and into the underlying Proglacial Stratified Drift.

6.2 BRIDGE FOUNDATION (DRILLED SHAFTS)

As previously described, each bridge abutment will be supported on two, 3- or 4-foot diameter drilled shafts. Design details for the drilled shafts are included in Attachment C. Attachment C also contains information regarding seismic design and drilling considerations.

6.3 BRIDGE ABUTMENT WING WALLS

As previously described, the bridge abutments will have a soldier pile wingwall at each abutment (provided that an SEW is not feasible) at the locations shown on Figure 2. Design details for a soldier pile wall are included in Attachment C.

6.4 STRUCTURAL EARTH WALLS

6.4.1 General

SEWs are typically used in fill applications where sufficient space is available for fill placement within the Reinforced Fill Zone. The SEW system consists of a Reinforced Fill Zone, often reinforced with layers of geotextile fabric depending on the wall height, and a CBU facing which is usually connected (pinned) with the Reinforced Fill Zone geogrid reinforcement layers. The CBUs are typically supported on a Leveling Course Pad of crushed rock to provide uniform support and to allow for easier installation (leveling).

6.4.2 SEW Design Parameters

SEW internal design (geogrid type, length and spacing, Reinforced Fill Zone soil material and compaction specification, drainage) should be completed by the SEW material supplier. To assist in this design, we recommend the following soil parameters.

Parameter	Reinforced Fill Zone	Retained Soil	Foundation Soil
Unit Weight (pcf)	125	120	130
Phi (degrees)	32	32	36
Cohesion (psf)	0	0	0

pcf = pounds per cubic foot; psf = pounds per square foot

We strongly recommend that the Reinforced Fill Zone consist of free-draining soil such as Gravel Backfill for Walls as described in the 2018 WSDOT Standard Specification Section 9-03.12(2). The on-site soils contain a relatively high percentage of fines and may not be suitable for use in the Reinforced Fill Zone.

We recommend using an allowable soil bearing capacity of 2,500 psf.

The design heights of SEWs should include the aboveground wall heights as well as the full embedment depths of the walls down to the Leveling Course Pad. The minimum embedment depth is as follows:

Slope in Front of Wall	Minimum Embedment Depth (feet)
Horizontal	H/20 or 1 foot, whichever is greater
3H:1V	H/10 or 1 foot, whichever is greater
2H:1V	H/7 or 1 foot, whichever is greater

H:V = horizontal to vertical H = Wall Height

The minimum embedment depth assumes use of a 6-inch thick, free-draining crushed rock leveling pad. The wall embedment could be further reduced to 0.5 feet if the leveling pad thickness is increased to 1 foot, or if non-frost susceptible soils are observed at wall subgrade at the time of construction.

Depending on the SEW type and height, geogrid reinforcement of the backfill may not be required and should be discussed with the SEW material supplier. For any height of SEW, we recommend the use of free-draining soil for backfill to provide adequate drainage.

SEWs should be designed with minimum factors of safety of 1.5 for sliding and pullout of reinforcing elements and 2.0 for overturning. If proprietary wall systems are used, the wall manufacturer is responsible for evaluating these items. However, we recommend that proprietary wall system designs be reviewed by a qualified geotechnical engineer to evaluate if valid assumptions were used relative to material properties and other factors such as site-specific topography and soil/groundwater conditions.

If SEWs are subject to the influence of traffic loading or nearby retaining walls within a horizontal distance equal to the height of the SEW, the walls should be designed for the additional horizontal pressure using appropriate design methods. A common practice is to assume a surcharge loading equivalent to 2 feet of additional fill to simulate traffic loads.

6.4.3 SEW Subgrade Preparation

SEW subgrade preparation typically consists of first excavating the Leveling Course Pad for the SEW, followed by additional excavation for the Reinforced Fill Zone. We recommend that the subgrade be evaluated by probing by a representative of our firm. Acceptable Leveling Course Pad and Reinforced Fill Zone subgrade is generally defined by probe penetration of less than 6 inches.

6.5 CONSTRUCTION CONSIDERATIONS

6.5.1 General

Some of the Structural Fill placed for the trail approaches for increasing the grade height and width is underlain by existing fill for the rail bed that is of unknown quality. The existing fill has been in-place for over 100 years during which the existing fill supported train traffic for several decades. During site preparation for new fill we expect that the existing fill in the bridge approach areas will be evaluated for suitability to remain in place.

We recommend that the subgrade for new fill or SEWs be evaluated by proofrolling and/or probing by a representative of our firm. Where subgrade soils cannot be adequately compacted, or where soft or disturbed soil is present, these areas should be excavated to expose competent material and replaced with Structural Fill.

The onsite native soils have a relatively low silt content. It is reasonable to schedule earthwork in this area during the winter and early spring months with less delays as compared to sites that have soils that are more sensitive to moisture.

6.5.2 Structural Fill

6.5.2.1 General

All new Fill for the proposed BNSF Pedestrian Bridge should be placed as Structural Fill. Structural Fill material should be free of debris, organic material and rock fragments larger than 6 inches. The suitability of material for use as Structural Fill will depend on the gradation and moisture content of the soil. As the amount of fines (portion of 3/4-inch-minus soil particles passing the US Standard No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve.

6.5.2.2 Unclassified Fill

We recommend that unclassified imported fill consist primarily of granular material with less than 30 percent passing the US Standard No. 200 sieve. Unclassified fill will be sensitive to changes in moisture content and compaction will be difficult or impossible to achieve during wet weather. We recommend that unclassified fill be used as Structural Fill only during dry weather conditions when proper moisture conditioning can be achieved.

6.5.2.3 Gravel Borrow Backfill for Walls

We recommend that Structural Fill consist of Gravel Backfill for Walls for the Reinforced Fill Zone for SEWs. Gravel Backfill for Walls should conform with Section 9-03.12(2) of the 2018 WSDOT Standard Specifications.

6.5.2.4 Reuse of On-Site Materials

The site soils (Fill and Proglacial Stratified Drift) may be reused for Structural Fill during periods of extended dry weather, though may be of limited use within the Reinforced Fill Zone (for SEWs) depending on the fines content (see Section 6.5.2.3 for material specifications). The Proglacial Stratified Drift is often considered an "all-weather" Fill depending on the silt content.

6.5.2.5 Placement and Compaction

All Structural Fill placed in trail and shoulder areas should be compacted to at least 95 percent of the MDD (ASTM Test Method D 1557). Waste fill in landscaping areas need only be compacted to the extent required for trafficability of construction equipment and erosion control.

As a guideline, we recommend that Structural Fill be placed in horizontal lifts which are 10 inches or less in loose thickness. The actual lift thickness will be a function of the fill quality and size of the compaction equipment used. Each lift should be compacted to the required specification before placing subsequent layers.

For placement during wet weather or on wet subgrades, Structural Fill should contain no more than five percent fines. Structural Fill placement over wet ground should commence with an initial lift of about 12 to 18 inches Permeable Ballast (2018 WSDOT Standard Specification section 9-03.9(1) or Quarry Spalls (2018 Standard Specification section 9-13.1(5)). During dry weather, the fines content may be up to about 30 percent, provided that the fill can be moisture-conditioned and compacted to the degree specified below.

We recommend that a representative from our firm observe the preparation for, placement, and compaction of Structural Fill. An adequate number of in-place density tests should be completed in the Structural Fill to evaluate if the desired degree of compaction is being achieved.

Nonstructural Fill placed in landscape and waste-fill areas where the existing surface slope is no steeper than 4H:1V needs to be compacted only to the degree required for trafficability of construction equipment and effective surface drainage/erosion control. All Nonstructural Fills should be sloped no steeper than 4H:1V. Nonstructural Fill is very susceptible to erosion. Therefore, we recommend that all Nonstructural Fill areas be immediately seeded, planted, or otherwise protected from erosion.

6.5.2.6 Construction Dewatering

Based on our knowledge of the regional groundwater level, we do not expect that groundwater will be encountered. Groundwater was encountered at about Elevation 539 to 542 feet (WSDOT, 1988; measured on August 16, 1988; 40.5 to 42.5 below the ground surface at the proposed abutments). The drilled shafts for the abutments are the only “deep” excavations planned. ICE should review the drilled shaft design when available.

Well points or pumped wells will be necessary if large amounts of groundwater seepage are encountered. We recommend that the contractor be required to submit a proposed dewatering system design and plan layout to the project engineer for review and comment prior to beginning construction.

6.5.3 Fill and Temporary Cut Slopes

6.5.3.1 Fill Slopes

Permanent Structural Fill slopes may be sloped at 2H:1V or flatter. All surfaces which will receive Structural Fill should be properly stripped of vegetation and organic matter prior to placing Structural Fill. Structural Fill placed on existing slopes which are steeper than 4H:1V should be properly keyed into the native slope surface. This can be accomplished by constructing the Structural Fill in a series of 4- to 8-foot-wide horizontal benches cut into the slope. The Structural Fill should be placed in horizontal lifts.

Steeper (1.5H:1V) Structural Fill slopes are possible provided that these slopes are covered with quarry spalls or an appropriate permanent erosion control mat or blanket.

6.5.3.2 Temporary Cut Slopes

Temporary cut slopes may be required for soldier pile or SEW installation. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. All excavations more than 4 feet in depth should be sloped in accordance with Part N of WAC 296-155 or be shored.

Loose to medium dense existing fill classifies as a Type C soil (OSHA 1926 Subpart P, Appendix A and B; OSHA Technical Manual, Section V, Chapter 2, sections V and VIII, dated January 20, 1999) and may be inclined (temporary slope) as steep as 1.5H:1V. The medium dense to dense Proglacial Stratified Drift classifies as a Type B soil and may be inclined (temporary slope) as steep as 1H:1V (OSHA, as described above).

6.5.4 Shored Excavations

It may be necessary to support the temporary excavations to maintain the integrity of the surrounding undisturbed soils and to reduce disruption of adjacent areas, as well as to protect the personnel working within the excavation. Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. We recommend that the shoring be designed by a licensed Professional Engineer in Washington, and that the PE-stamped shoring plans and calculations be submitted to the Project Engineer for review and comment prior to construction.

7.0 USE OF THIS REPORT

We have prepared this Report for use by Parametrix and King County Parks in support of design for the proposed BNSF Pedestrian Bridge. The Report should be provided to the design team and/or contractors for their use; this Report is not applicable to other locations or for other purposes. Our interpretations, conclusions and recommendations should not be construed as a warranty of the site conditions.

If there are significant changes in the grades, configurations or types of facilities to be constructed, the conclusions and recommendations presented in this report may not be fully applicable. When the design has been finalized, we recommend that we be retained to review those portions of the specifications and drawings which relate to geotechnical considerations to see that our recommendations have been interpreted and implemented as intended.

Variations in subsurface conditions are possible between the locations of the widely-spaced explorations (completed by others during previous studies). Variations may also occur with time. Some contingency for unanticipated conditions should be included in the project budget and schedule. Sufficient observation, testing and consultation should be provided by our firm during construction to evaluate that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with the contract plans and specifications.

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

****

We trust this Report meets your present needs. Please contact us if you have any questions or need additional information.

Yours very truly,
Icicle Creek Engineers, Inc.

Kathy S. Killman, LEG
Principal Engineering Geologist

Brian R. Beaman, PE, LEG, LHG
Principal Engineer/Geologist Hydrogeologist

State of Washington Licensed Geologist Engineering Geologist 671 BRIAN R. BEAMAN

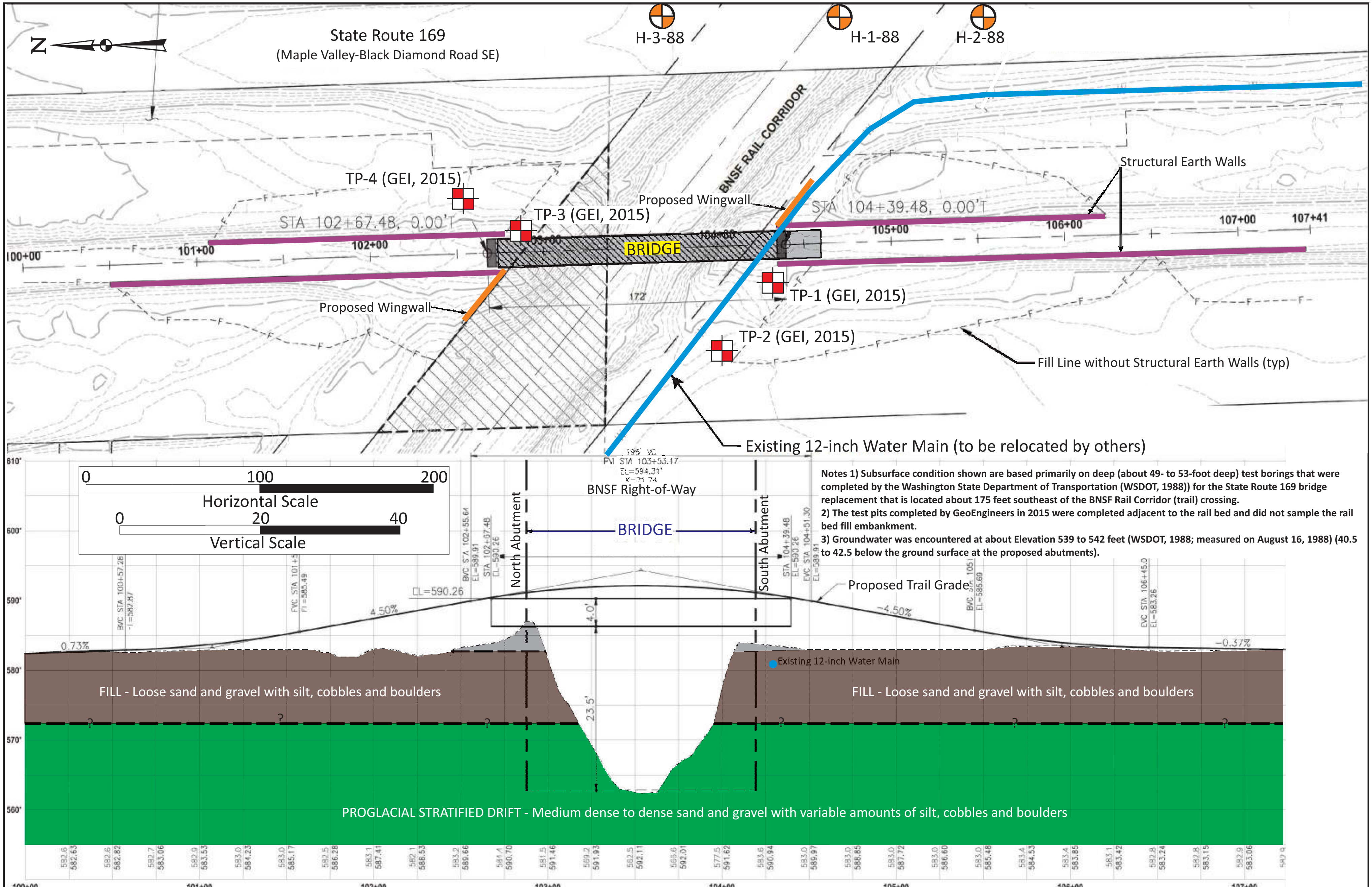
BRIAN R. BEAMAN State of Washington 24010 Registered Professional Engineer

Document ID: 0105022.BNSF Rep

Attachments: Figure 1 – Vicinity Map
Figure 2 – Plan and Profile
Attachment A – GEI (2015) Test Pit Logs
Attachment B – WSDOT (September 21, 1988) Letter with Test Boring Logs and Cross-Section
Attachment C – Shannon & Wilson Letter dated January 11, 2019

Submitted via email (pdf) and surface mail (one original copy)

FIGURES



Notes 1) Subsurface condition shown are based primarily on deep (about 49- to 53-foot deep) test borings that were completed by the Washington State Department of Transportation (WSDOT, 1988)) for the State Route 169 bridge replacement that is located about 175 feet southeast of the BNSF Rail Corridor (trail) crossing.
2) The test pits completed by GeoEngineers in 2015 were completed adjacent to the rail bed and did not sample the rail bed fill embankment.
3) Groundwater was encountered at about Elevation 539 to 542 feet (WSDOT, 1988; measured on August 16, 1988) (40.5 to 42.5 below the ground surface at the proposed abutments).

Base Map Reference: Parametrix, April 12, 2018, BNSF ROW, Skewed Crossing, GRCR Trail, scale 1 inch = 50 feet; Parametrix, April 12, 2017, BNSF Skewed Crossing Profile, GTCR Trail, horizontal scale 1 inch = 50 feet, vertical scale 1 inch = 10 feet.

EXPLANATION

TP-2 (GEI, 2005) Test Pit Location (GeoEngineers, 2015)

H-2-88 Test Boring (WSDOT, 1988)

King County

Parametrix

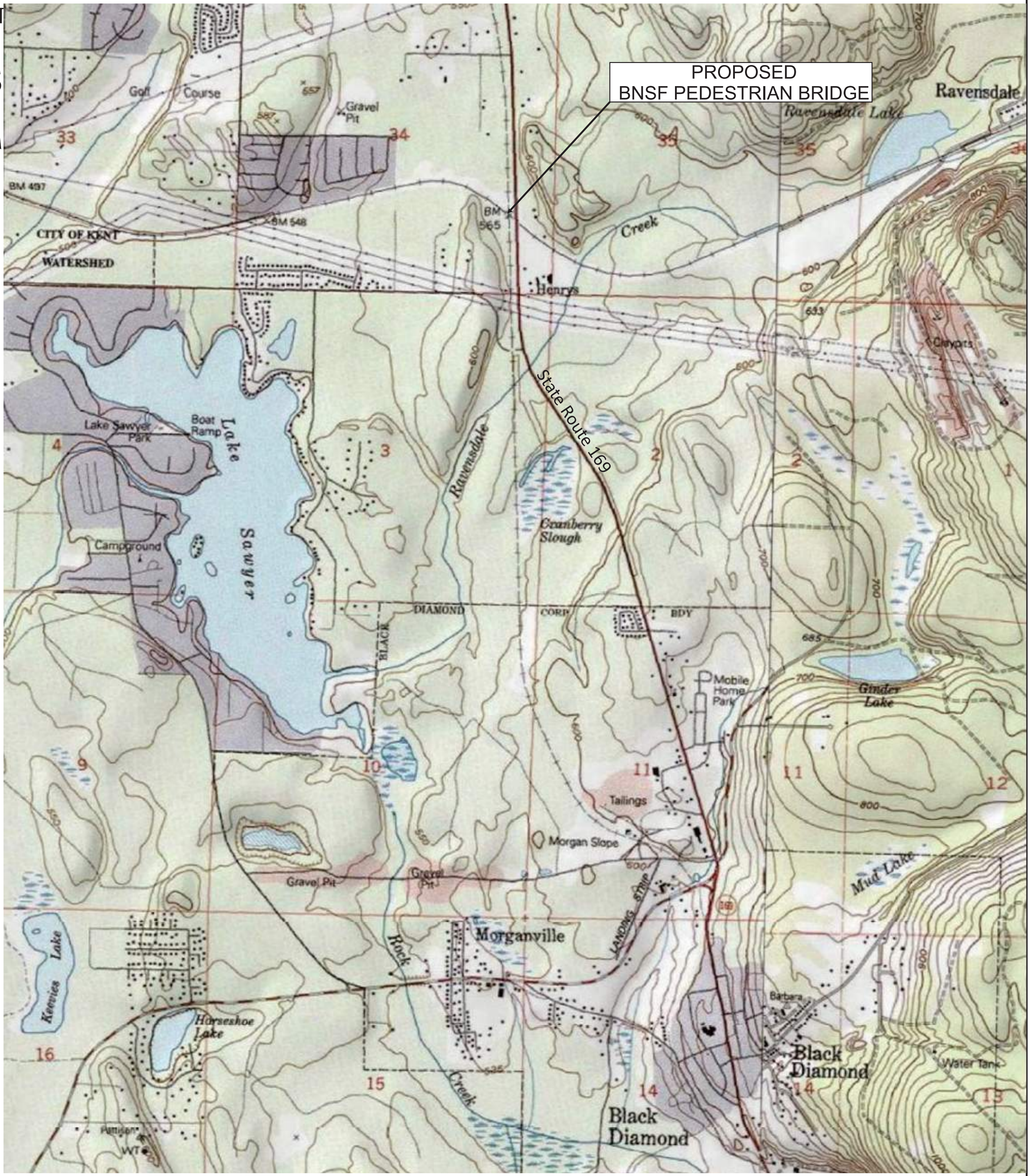
PLAN AND PROFILE
PROPOSED BNSF PEDESTRIAN BRIDGE
PRELIMINARY GEOTECHNICAL EVALUATION
SOUTH SEGMENT GREEN TO CEDAR RIVERS REGIONAL TRAIL PROJECT

ICICLECREEK ENGINEERS
29335 NE 20th Street
Carnation, Washington 98014
(425) 333-0093

SCALE: As Shown
DESIGNED: ---
DRAWN: BRB
CHECKED: KSK
DATE: 01/11/19

ICE FILE NO. 0105-022

Figure 2



Base map from Washington State Department of Natural Resources Washington Geologic Information Portal

0 ½ 1

Approximate Scale in Miles

King County / Parametrix	VICINITY MAP PROPOSED BNSF PEDESTRIAN BRIDGE PRELIMINARY GEOTECHNICAL EVALUATION SOUTH SEGMENT GREEN TO CEDAR RIVERS REGIONAL TRAIL PROJECT	ICICLECREEK ENGINEERS 29335 NE 20th Street Carnation, Washington 98014 (425) 333-0093	SCALE: As Shown DESIGNED: --- DRAWN: BRB CHECKED: KSK DATE: 01/11/19	ICE FILE NO. 0105-022 Figure 1

ATTACHMENT A

GEOENGINEERS 2015 TEST PIT LOGS

(see Report for full reference)

Date Excavated: 7/20/2015

Equipment: TB235 Mini trackhoe

Logged By: BHC

Total Depth (ft) 7.5

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				GM		Brownish-gray silty gravel with sand (dense, dry)		
	1		S-1						Probe = 3 to 4 inches at 1 foot
	2								
	3				GM		Brown silty gravel with sand and cobbles (very dense, dry to moist)		
	3.5		S-2						Oxidation staining Probe < 1 inch at 3.5 feet
	4								
	5				GP		Brown gravel with sand (very dense, moist)		
	5.5		S-3						
	6								
	7								

Test pit completed at 7.5 feet
No groundwater seepage observed
Moderate caving observed at 3 to 5 feet

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-01

GeoEngineers

Project: Green to Cedar Rivers Regional Trail Project
Project Location: King County, Washington
Project Number: 1813-017-00

Date Excavated: 7/20/2015

Equipment: TB235 Mini trackhoe

Logged By: BHC

Total Depth (ft) 7.0

Elevation (feet)	Depth (feet)	Testing Sample	Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Brown-gray silty fine to coarse sand with gravel and cobbles (very dense, dry)		
	1	X	S-1		GP-GM		Brown gravel with silt, sand and cobbles (very dense, dry)		Probe = 1 to 2 inches at 1 foot
	2								
	3	X	S-2						Probe = 0 to 1 inch at 3 feet
	4								
	5	X	S-3						
	6								
	7						Test pit completed at 7 feet No groundwater seepage observed No caving observed		

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-02

GEOENGINEERS

Project: Green to Cedar Rivers Regional Trail Project
Project Location: King County, Washington
Project Number: 1813-017-00

Date Excavated: 7/20/2015

Equipment: TB235 Mini trackhoe

Logged By: BHC

Total Depth (ft) 7.5

Elevation (feet)	Depth (feet)	Testing Sample	Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				GM		Brown silty gravel with sand and cobbles (medium dense, dry)		Probe = 1 foot depth at surface Abundant roots in top 2 feet
	1	X	S-1						
	2						Becomes very dense		
	3	X	S-2						Probe = 1 inch at 3 feet
	4								
	5				SM-GM		Orange-brown silty fine to coarse sand with gravel and cobbles (very dense, dry)		
	6								
	6.5	X	S-3						
	7								

Test pit completed at 7.5 feet
No groundwater seepage observed
No caving observed

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-03

GEOENGINEERS

Project: Green to Cedar Rivers Regional Trail Project
Project Location: King County, Washington
Project Number: 1813-017-00

Date Excavated: 7/20/2015 Logged By: BHC
Equipment: TB235 Mini trackhoe Total Depth (ft) 7.0

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Dark brownish-gray silty fine to coarse sand with gravel and cobbles (medium dense, dry)		Abundant roots
	1	X	S-1						
	1.5				GM		Light brown silty gravel with sand and cobbles (medium dense, dry)		
	2	X	S-2						Probe = 1 inch at 2 feet Harder digging
	3						Becomes very dense		
	4								
	5								
	6	X	S-3						
	7						Test pit completed at 7 feet No groundwater seepage observed Slight caving observed at 3 feet		

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-04

GeoEngineers

Project: Green to Cedar Rivers Regional Trail Project
Project Location: King County, Washington
Project Number: 1813-017-00

ATTACHMENT B

WSDOT SEPTEMBER 21, 1988 LETTER WITH TEST BORING LOGS AND CROSS-SECTION

(see Report for full reference)

DocID 9374

Source: WSDOT

Highway (ID#1) SR 169

Milepost (ID#2) 10.41 - 10.44

Site Address RR O'xing, Bridge widening

Date Copied 6/18/01 By KMD

- ☒ **Title page with the following information:**
  - *Company (Author) name*
  - *Report date*
  - *Project Name*
  - *Company's job number*
  - *Site address*
- ☒ Executive Summary / Introduction of the report
- ☐ Table of contents
- ☒ Project Location Map / Vicinity Map
- ☒ **Site / Exploration Plans, Boring Location Plans**
- ☒ Cross-sections / Subsurface profiles
- ☒ **Exploration Logs**
- ☐ Monitoring Well Logs
- ☐ Cone Penetrometer Logs
- ☐ Groundwater Elevation Tables / Data

- ☐ Includes data from Previous Reports
- ☐ No new data / data review
- ☐ Missing Data / Illegible Data
  Explanation

Comments: Site Map ⇒ Hole 1 was moved from original location to other side of the highway, and no new site map was made so follow how I marked the map - Its right, I promise!

Points - EF Layers - AF

√'ed - SW √'ed - JT

(9374)

Washington State Department of Transportation
Transportation Building KF-01
Olympia, Washington 98504-5201
206 753-6005

Duane Berentson
Secretary of Transportation

September 21, 1988

Mr. C. S. Gloyd
Bridge Engineer
Transportation Building
Olympia, WA 98504

RE: CS 1734, SR-169, L-8872
BN RR Overcrossing Bridge
Bridge 169/12 Widening
Foundation Recommendations

Dear Mr. Gloyd:

This letter presents the foundation recommendations for the widening of the Burlington Northern Railroad Bridge No. 169/12. This bridge carries SR-169 traffic over Burlington Northern Railroad tracks 2.7 miles North of Black Diamond. The three span Haunched Tee Beam structure will be widened 8 ft on each side of the existing structure. When completed, the new structure will provide two 10 ft lanes with 10 ft shoulders. The approaches will require minor sliver fills up to 7 ft in height. The northern (Pier 4) approach fill will be contained by a 12 ft long retaining wall on the right and a 20 ft long retaining wall on the left.

The analyses, conclusions, and recommendations contained in this report are based on the project description, and site conditions which existed at the time of the field explorations, and further assume that the exploratory borings are representative of the subsurface conditions throughout the project area. If, during construction, subsurface conditions different from those found by the explorations are encountered, or appear to be present beneath or beyond the excavations, we should be advised so that we can assist you and reevaluate our recommendations.

SITE GEOLOGY

The project site is located on the Covington Drift Plain formed during the last glacial advance and retreat, known as the Vashon Period of the Fraser Glaciation. As the glacier melted, approximately 13,500 years ago, large quantities of meltwater sediment were released. Large streams from receding ice margins formed meltwater channels and deposited recessional outwash material. The recessional outwash deposits are comprised of well sorted, silty, gravelly sand varying to clean sandy gravel with cobbles and boulder size material. The thickness ranges from 10 ft to more than 50 ft in places.

Advance outwash deposits and glacial till soils comprise the very dense soils underlying the recessional deposits. The 10 ft to 50 ft thick mantle of till consists of a non-sorted mixture of clay to boulder size material that has been compacted by the overriding ice.

FIELD INVESTIGATION

The field investigation consisted of drilling three test holes to determine the type of foundation support required. Standard penetrometer tests, in general, were taken at five-foot intervals. Disturbed soil samples from the standard penetrometer were visually identified in the field and than submitted to the Materials Laboratory for a more detailed classification. A total of 31 standard penetrometer tests were performed. Copies of the three test hole logs are presented in Appendix B, detailing specific site conditions.

In general, the foundation material consists of 15 ft of medium dense to loose sands and gravel with cobbles and boulders underlain by 25 ft to 35 ft of dense slightly silty sand and gravel, which is underlain by very dense slightly silty to silty sandy gravel. The cobbles and boulders appear to comprise roughly 15 percent of the upper stratum. "Nested" boulders should be expected in some areas. A soil profile depicting generalized soil conditions at the site is presented in Appendix A.

LABORATORY TESTING

The laboratory program for this project consisted of the identification and classification of nine disturbed samples obtained from the standard penetrometer tests conducted in the field. The unified soil classification system was used as the basis to describe all soil samples. Visual classification included density or consistency, color, moisture content, major soil type, and the modifying fractions of the samples. Grain-size analyses and moisture-content determinations were performed on the disturbed samples. Grain-size analyses were performed in accordance with procedures detailed in AASHTO T88, and water content tests were performed in accordance with the procedures detailed in ASTM D-2216. The results of all laboratory tests are presented in Appendix C.

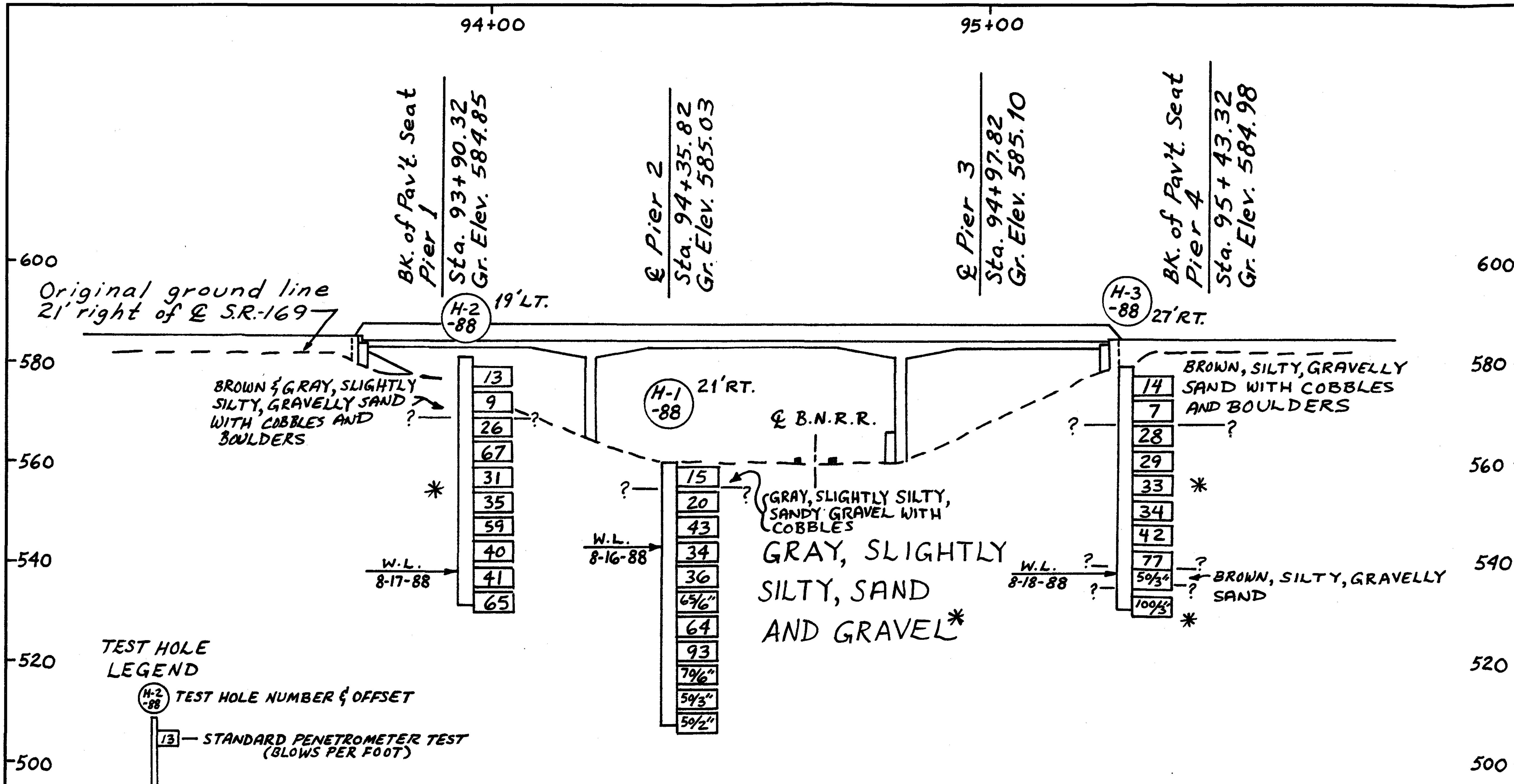
EMBANKMENT DESIGN

The approach embankments will require minor sliver fills which will vary up to 7 feet in height. The new sliver fills will be stable with 1.75:1 or flatter slopes, provided the existing embankments are terraced prior to placement of the new fill as required by Section 2-03.3(14) of the Standard Specifications. The sliver fills will settle as much as 0.5 inches during construction. Post-construction settlement will be negligible.

PIER FOUNDATION SUPPORT

We recommend that the end piers for the proposed widening be supported by piling. A less desirable spread footing option is provided. Steel "H" piles are the recommended pile option. Steel "H" piles can be designed for allowable loads up to 70 tons. For design purposes, it may be assumed that steel "H" piles will drive through the dense sand and gravel stratum and achieve bearing at approximate elevation 523 ft at Pier 1 and elevation 533 ft at Pier 4.

Spread footings designed for allowable loads up to 3 tsf are also feasible at Piers 1 and 4. The footings should be located on the basis of the criteria in the Bridge Design Manual for footings on slopes, but not above the elevation of the existing footings. This option appears to be more costly as shoring will be required between the new and existing footings during construction. The loose soil directly beneath the proposed footings will need to



SOIL PROFILE

HOLE NO	SAMPLE NUMBERS
H-1-88	E-8320-1 TO E-8320-11
H-2-88	E-8318-1 TO E-8318-10
H-3-88	E-8319-1 TO E-8319-10

JOB L-8872 S.R. 169 C.S. 1734

B.N.R.R. O-XING BRIDGE NO. 169/12 WIDENING

WASHINGTON STATE TRANSPORTATION COMMISSION
DEPARTMENT OF TRANSPORTATION
MATERIALS BRANCH
J.R. Strada Materials Engineer

DATE: SEPT. '88
SCALE: 1"=20' VERT.
1"=20' HORIZ.
SHEET 1 OF 1
DRAWN BY P.A.A.

DOTC-1135

LOG OF TEST BORING — **WASHINGTON STATE DEPARTMENT OF TRANSPORTATION**

S.H. ____ S.R. 169 SECTION BN RR O/C Bridge No. 169/12 Job No. L-8872

Hole No. H-1-88 Sub Section ____ Cont. Sec. 1734

Station 94+36 Offset 21' Rt. ℄ Ground El. 560.0' Contour on Layout

Type of Boring Augers Casing 4.5 x 52' W.T. El. 543.0'

Inspector ____ Date August 16, 1988 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	15		6 / 6 / 9 — STD PEN 1	GP, M.C. = 0.9% Medium dense, gray, dry, slightly silty, fine to coarse sandy, sub-angular GRAVEL with cobbles. Retained 8".
5				
	20		8 / 12 / 8 — STD PEN 2	Medium dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 10".
10				
	43		10 / 21 / 22 — STD PEN 3	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 11".
15				
	34		10 / 15 / 19 — STD PEN 4	Dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 8".
20				

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

Copy to ____

Hole No. H-1-88 Sub Section BN RR O/xing Bridge No. 169/12 Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	36		12 16 20 STD PEN 5	Dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 12".
25				
	65/6		35 65 STD PEN 6	GW Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 12".
30				
	64		21 28 36 STD PEN 7	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 18".
35				
	93		25 35 58 STD PEN 8	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 12".
40				
	70/6"		70 STD PEN 9	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 6".
45				

Hole No. H-1-88 Sub Section BN RR O'xing Bridge No. 169/12 Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	50/3"		43 50/ 3" STD PEN 10	Very dense, gray, wet, slightly sitly, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 9".
50				
	50/2"		41 50/ 2" STD PEN 11	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 8".
55				
				Stopped Test Boring 52' 8" below ground elevation.
				Bensealed Hole
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

LOG OF TEST BORING **WASHINGTON STATE DEPARTMENT OF TRANSPORTATION**

S.H. ______ S.R. 169 SECTION BN RR O/C Bridge No. 169/12 Job No. L-8872

Hole No. H-2-88 Sub Section ______ Cont. Sec. 1734

Station 93+95 Offset 19' Lt. ℄ Ground El. 581.0' Contour on Layout

Type of Boring Augers Casing 4" x 10" x 48' W.T. El. 539.0'

Inspector ______ Date August 17, 1988 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				1' to 2' 6" cobbles and boulders
5	13		5 STD 6 PEN 7 1	SP, M.C. = 4.6% Medium dense, brown, dry, slightly silty, gravelly, fine to coarse SAND. Retained 8".
10	9		4 STD 4 PEN 5 2	Loose, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 2".
15	26		10 STD 12 PEN 14 3	SP Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
20	67		14 STD 30 PEN 37 4	Very dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 6".

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

Copy to ______

Hole No. H-2-88 Sub Section BN RR O'xing Bridge No. 169/12 Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
25	31		10 14 17 STD PEN 5	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 10".
30	35		11 16 19 STD PEN 6	SW/SM, M.C. = 4.2% Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
35	59		20 24 35 STD PEN 7	Very dense, gray,moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
40	40		12 20 20 STD PEN 8	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
45	41		18 20 21 STD PEN 9	GP/GM Dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 12".

Hole No. H-2-88 Sub Section BNRR O'xing Bridge No. 169/12 Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			18 STD	
	65		32 PEN	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub angular
50			33 10	GRAVEL. Retained 6".
				Stopped Test Boring 49' 6" below ground elevation.
				Ben Seal and Hole Plug.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

DOT FORM 351-003A (X) REVISED 4/80

LOG OF TEST BORING

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 169 SECTION BN RR O/C Bridge No. 169/12 Job No. L-8872

Hole No. H-3-88 Sub Section ______ Cont. Sec. 1734

Station 95+27 Offset 27' Rt. ℄ Ground El. 579.5' Contour on Layout

Type of Boring Augers Casing 4" x 48' W.T. El. 537.5'

Inspector ______ Date August 18, 1988 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				0' to 3' Cobbles and boulders
			6 STD	SW/SM, M.C. = 6.2%
	14		7 PEN	Medium dense, brown, dry, silty, gravelly, fine to coarse SAND.
5			7 1	Retained 8".
			3 STD	
	7		3 PEN	Loose, brown, dry, silty, gravelly, fine to coarse SAND.
10			4 2	Retained 3".
			11 STD	
	28		14 PEN	Dense, gray, dry, slightly silty, gravelly, fine to coarse SAND.
15			14 3	Retained 9".
			9 STD	
	29		11 PEN	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND.
20			18 4	Retained 10".

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

Copy to ______

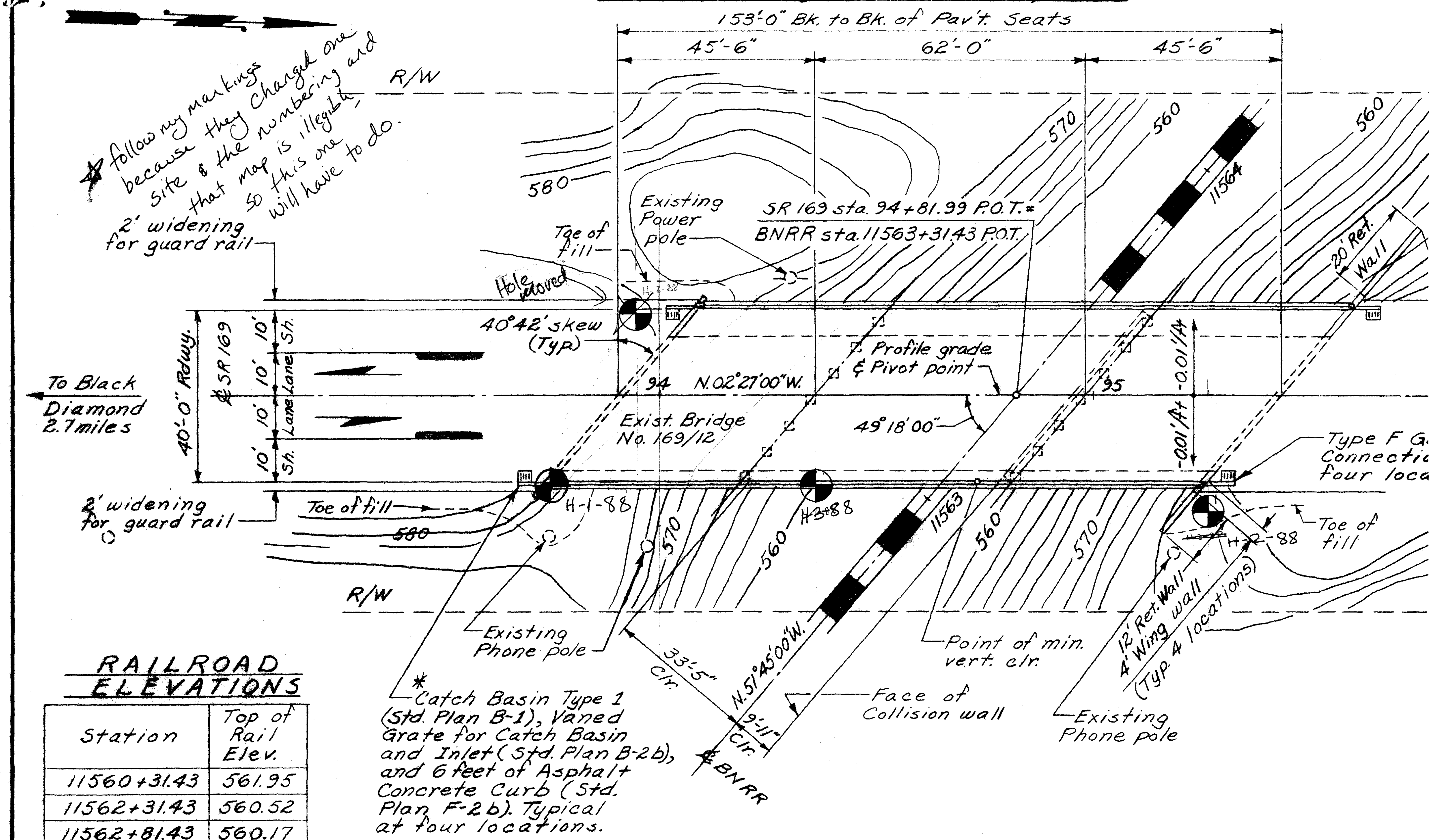
Hole No. H-3-88 Sub Section BN RR O'xing Bridge No. 169/12 Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
25	33		11 16 17 STD PEN 5	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
30	34		12 16 18 STD PEN 6	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 10".
35	42		14 20 22 STD PEN 7	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
40	77		14 27 50 STD PEN 8	Very dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
45	50/3"		50/3" STD PEN 9	SW/SM Very dense, brown, wet, silty, gravelly, fine to coarse SAND. Retained 3".

Hole No. H-3-88 Sub Section BN RR O'xing Bridge No. 169/12

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	100/3"		50 100/ 3" STD PEN 10	GW/GM Very dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 9".
50				
				Benseal and hole plug.
				Stopped Test Boring 48' 9" below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

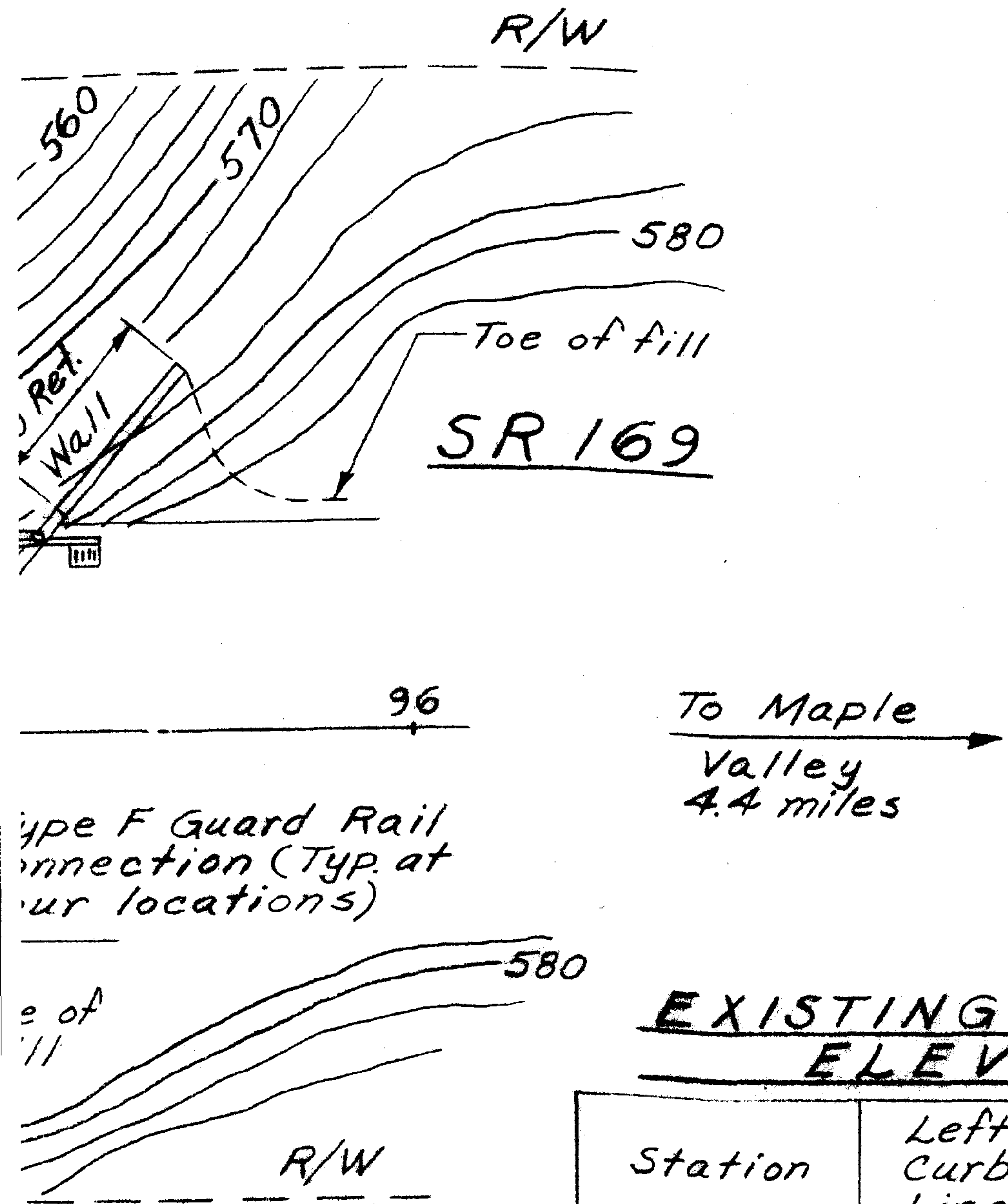
SECTION 34, T. 22 N., R. 6 E., W.M.

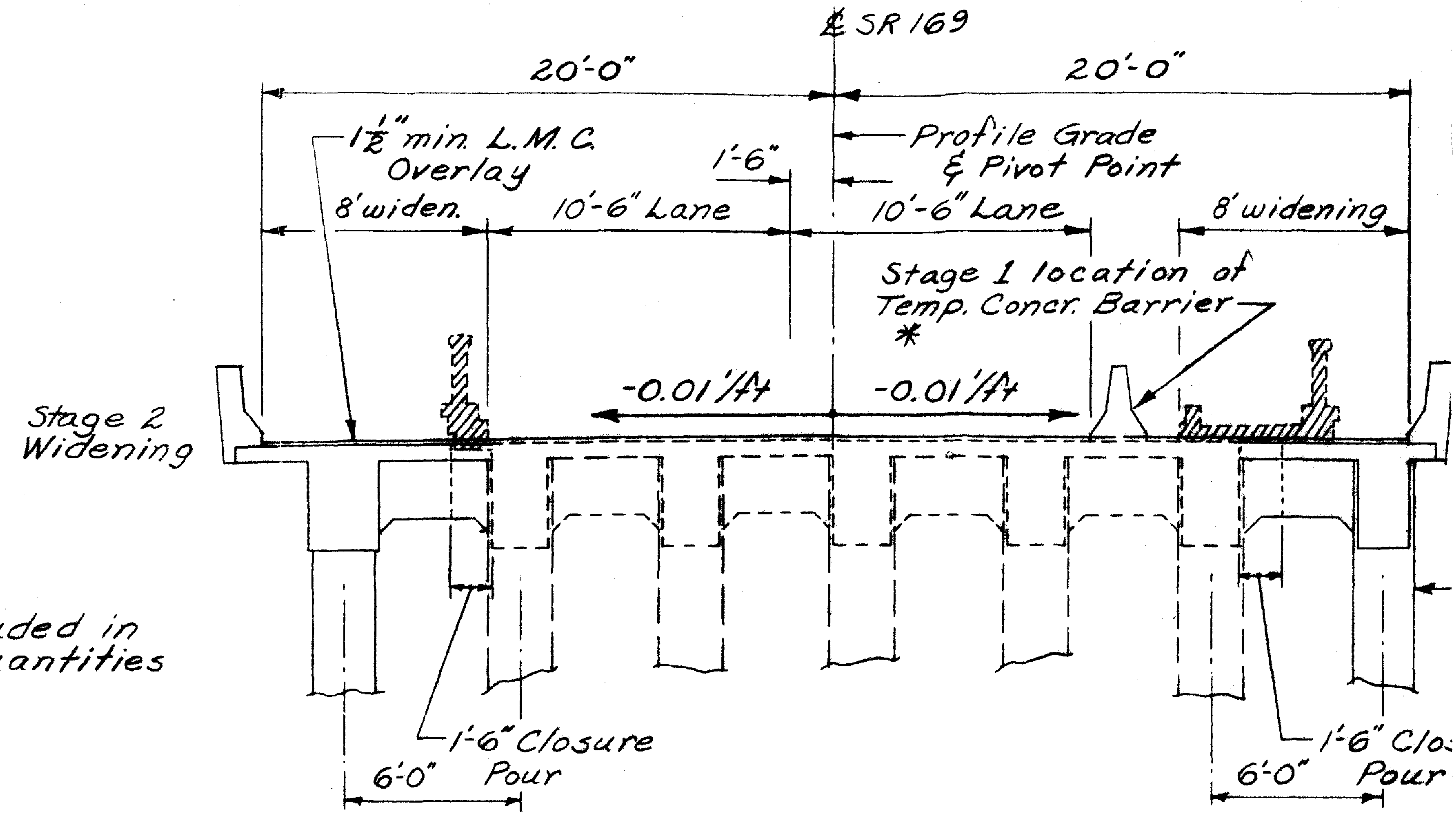


PLAN
Scale: 1" = 20'

RAILROAD ELEVATIONS

Station	Top of Rail Elev.
11560+31.43	561.95
11562+31.43	560.52
11562+81.43	560.17
11563+31.43	559.93
11563+81.43	559.69
[illegible]	[illegible]





TYPICAL ROADWAY SECTION

Scale: $\frac{3}{16}$" = 1'

* Not included in bridge quantities

EXISTING BRIDGE ELEVATIONS

Station	Left Curb Line	℄ SR169	Right Curb Line
93+81.64	—	—	584.60
93+91.64	—	584.71	584.62
94+01.64	584.67	584.72	584.65
94+11.64	584.72	584.77	584.74
94+21.64	584.75	584.84	584.78
94+31.64	584.82	584.88	584.80
94+41.64	584.86	584.91	584.80
94+51.64	584.86	584.92	584.80
94+61.64	584.86	584.90	584.80
94+71.64	584.86	584.91	584.85
94+81.64	584.87	584.94	584.88

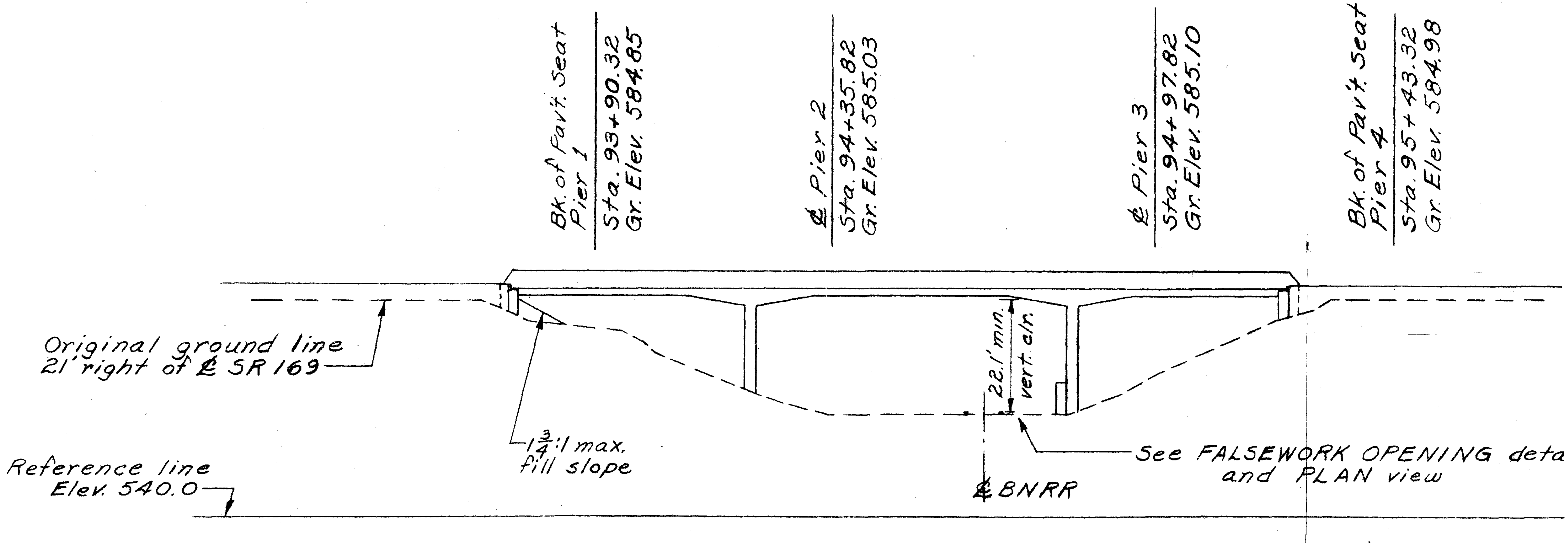
SR 169 PROFILE

Station	℄ SR 169
93+81.64	—
93+91.64	584.85
94+01.64	584.88
94+11.64	584.95
94+21.64	584.99
94+31.64	585.03
94+41.64	585.05
94+51.64	585.05
94+61.64	585.05
94+71.64	585.07
94+81.64	585.10

NOTES TO DISTRICT

VERIFY THAT THE HORIZONTAL AND VERTICAL CLEARANCES FOR BO CONSTRUCTION AND ULTIMATE SHOWN FOR THE RAILROAD A ACCEPTABLE.

WILL THERE BE ANY UTILITIES, LIGHTING, OR SIGNS ON T STRUCTURE? IF SO, PLEASE INDICATE TYPE, SIZE, AND LOCATIO



DATUM
Nat'l. Geod. Vert. Datum of 1929

ELEVATION

Grade elevations shown are finish grades at the top of the 1½" min. L.M.C. overlay on ℄ SR 169 and are equal to profile grade.

LAYOUT APPROVED BY:

C. E. Mays
Bridge and Structures Engineer

District Administrator

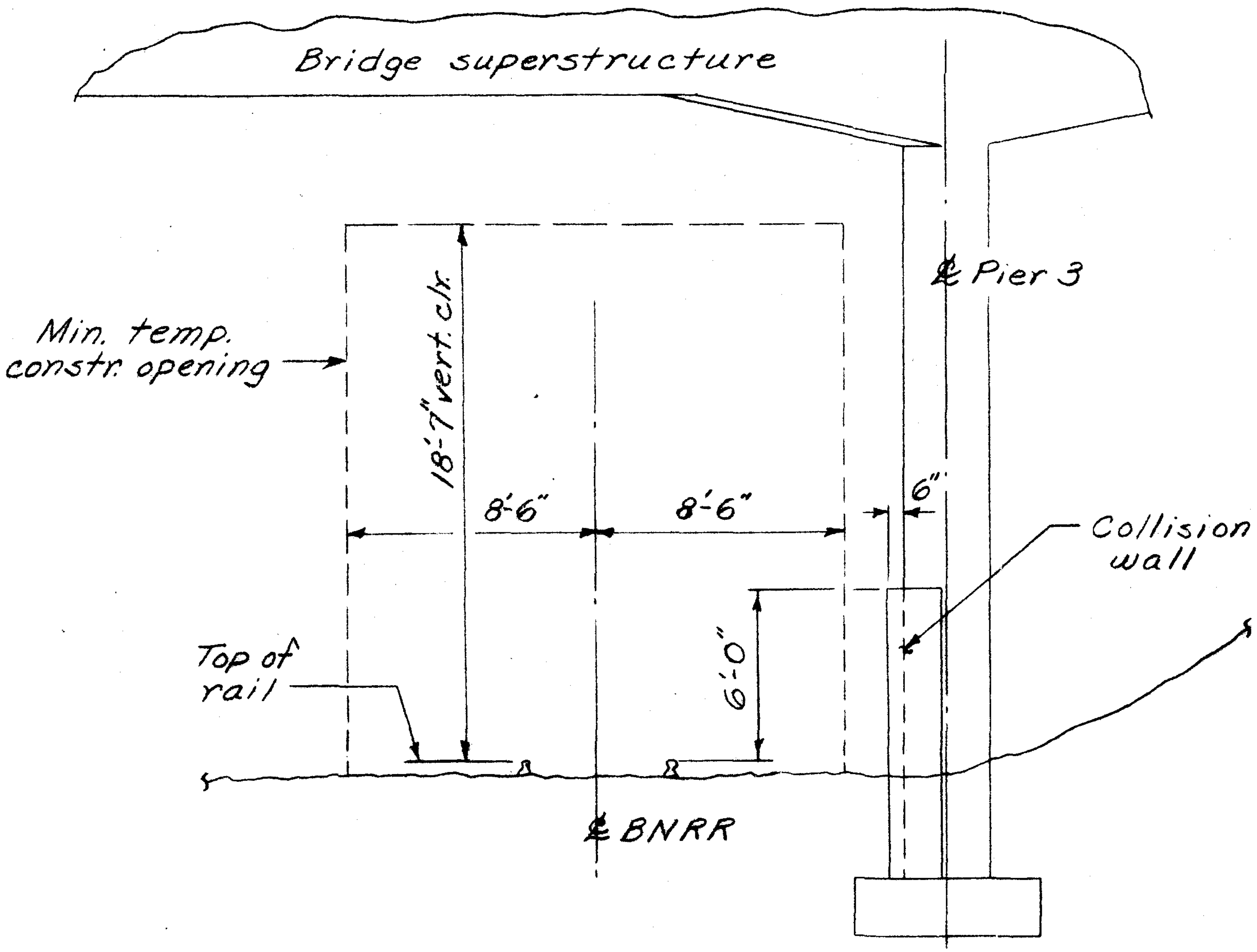
Bridge Design Engineer	G. Machuih 7-88		
Supervisor	Karl Kirker 7-88		
Reviewed By			
Designed By			
Checked By	A. C. Diedrich 6/88		
Detailed By	R. D. Braxmeyer 6/88		
Architect	R. MAYS 6/88		

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH			
JOB NUMBER				
CONTRACT NO.				

BRIDGES AN

169 JOB NO. 1505 SHEET 1

95+01.64	584.91	584.97	584.85	95+01.64	585.10
95+11.64	584.87	584.97	584.80	95+11.64	585.06
95+21.64	584.82	584.89	584.79	95+21.64	585.03
95+31.64	584.75	584.86	584.80	95+31.64	585.00
95+41.64	584.72	584.86	———	95+41.64	584.98
95+51.64	584.75	———	———	95+51.64	———



FALSEWORK OPENING

Looking ahead on stationing.

R.C. TEE BEAM WIDENING

LOADING: HS-20 OR TWO 24K AXLES @ 4' CTRS.

STRUCTURES

Washington State Department of Transportation

BRIDGE SHEET NO.

SHEET

OF

ATTACHMENT C

SHANNON & WILSON LETTER
JANUARY 11, 2019

GEOTECHNICAL EVALUATION
GREEN TO CEDAR RIVER TRAIL SOUTH SEGMENT – BNSF PEDESTRIAN BRIDGE
BLACK DIAMOND, WASHINGTON

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

January 11, 2019

Icicle Creek Engineers, Inc.
Attn: Brian R. Beaman, PE, LEG, LHG
29335 NE 20th Street
Carnation, WA 98014

Attn: Mr. Brian R. Beaman, PE, LEG, LHG

RE: GEOTECHNICAL EVALUATION, GREEN TO CEDAR RIVER TRAIL SOUTH SEGMENT – BNSF PEDESTRIAN BRIDGE, BLACK DIAMOND, WASHINGTON

This letter presents the observations and recommendations regarding our evaluation of the proposed Green to Cedar River Trail South Segment – BNSF Pedestrian Bridge, located in Black Diamond, Washington. This letter addresses our geotechnical evaluation of available subsurface information and provides recommendations for design of drilled shafts to support the bridge. Our services were performed in accordance with our Proposal Letter dated August 9, 2018, and Icicle Creek Engineers Task Order No. 007. We based the conclusions in this letter on a recent site visit and on previous subsurface data gathered by the Washington State Department of Transportation (WSDOT) and GeoEngineers, Inc. (GEI).

PROJECT DESCRIPTION

We understand that King County Parks plans to construct an approximately 172-foot-long pedestrian bridge over an east-west BNSF rail line. The bridge will be constructed at the location of an abandoned north-south BNSF rail line adjacent to Highway 169. Four 3- to 4-foot-diameter reinforced concrete drilled shafts will support the bridge at concrete abutments on either end of the bridge (two shafts per abutment). Structural fill will be used to construct the approaches, and soldier pile wingwalls will separate and retain fill from the BNSF right-of-way.

SITE VISIT AND SUBSURFACE CONDITIONS

Shannon & Wilson representatives visited the project site on August 17, 2018, to observe surface features and correlate them with subsurface information collected by WSDOT and GEI. We did not perform any additional subsurface explorations.

400 NORTH 34TH STREET, SUITE 100
P.O. BOX 300303
SEATTLE, WASHINGTON 98103-8636
206-632-8020 FAX: 206-695-6777
www.shannonwilson.com

At the location of the proposed pedestrian bridge, we observed that the east-west BNSF rail line lies within an approximately 20-foot-deep cut, shown in Photo 1. Bridge footings and abutments for the abandoned north-south BNSF rail line are still present at the site, but we understand that no existing bridge structures will be used in the construction of the proposed pedestrian bridge. We observed gravel and cobbles on the surface near the east-west tracks; the slopes on either side or the east-west rail line primarily consisted of brown sand with silt and gravel.

Photo 1: Active east-west BNSF rail line extending through the cut. Existing north-south railway bridge footings and abutment visible in background. View facing east toward the Highway 169 bridge.

Based on our review of borings H-1-88, H-2-88, and H-3-88, performed by WSDOT and test pits TP-01, TP-02, TP-03, and TP-04, performed by GEI, the ground surface near the top of the slopes at the location of the proposed abutments is likely underlain by approximately 10 feet of fill consisting of loose to dense, silty sand with gravel and cobbles. This layer is underlain by native advance outwash soils consisting of dense to very dense sand and gravel to the bottom of explorations (maximum depth of approximately 53 feet below surface near the east-west track, or elevation 507 feet). Groundwater level was observed at WSDOT borings from approximate elevation 543 to 538 feet; it is likely variable depending on season and may be encountered during drilling depending on drilled shaft depth and time of year. Perched water may also be present and may affect drilled shaft construction.

GEOTECHNICAL RECOMMENDATIONS

Based on our observations during our site visit and our review of previous subsurface data, we have prepared recommendations for the following geotechnical items:

- Axial resistance estimates for drilled shafts,
- LPILE soil parameters for drilled shafts,
- Lateral earth pressure estimates for design of wingwalls,
- Seismic design parameters, and
- Drilling Considerations.

Drilled Shaft Axial Resistance

Drilled shaft axial resistance will vary with shaft penetrations, shaft sizes, and installation techniques. We performed axial resistance analyses to evaluate the axial resistance of the drilled shafts upon which the proposed BNSF pedestrian bridge will be founded.

Estimated axial resistances were determined based on subsurface conditions as indicated by soil types, descriptions, and Standard Penetration Test (SPT) N-values for WSDOT borings and soil types, description, and relative densities for GEI test pits. The analyses were performed using an in-house computer program that determines nominal axial compressive resistance. For the calculations, we assumed that vertical loads from the proposed bridge structure can be supported by side resistance, or skin friction, and tip resistance on the bottom of the shafts. Side resistance is assumed to act along the majority of the outside surface of the shafts, extending from the pier cap to the base of the shafts.

Figures 1 (3-foot-diameter) and 2 (4-foot-diameter) provide our estimated nominal and factored resistances, and recommended resistance factors for the Service, Strength, and Extreme Event

Limits States. Service Limit settlement should be reviewed based on final shaft dimensions and loads.

Drilled Shaft LPILE Estimates

Lateral loads may be resisted by the passive earth pressure against deep foundations and foundation caps. The magnitude of lateral resistance developed by drilled shafts depends on the subsurface conditions encountered and the moment capacity at the foundation cap connection. We recommend ignoring the frictional sliding resistance at the base of the foundation cap, because a deep foundation-supported cap may not transmit load directly to the soil beneath it.

The computer program LPILE may be used to generate load-deflection (P-Y) curves for the lateral resistance analysis of deep foundations, and to calculate the magnitude of deflection, shear, and moment along the shafts. The following Table 1 presents our recommended geotechnical parameters for lateral resistance analysis for the proposed pedestrian bridge. Liquefaction potential is considered low; therefore, the values provided below are for both static and seismic conditions.

TABLE 1
RECOMMENDED PARAMETERS FOR
LATERAL RESISTANCE ANALYSIS USING LPILE

Depth* (feet)		Elevation* (feet)		LPILE Soil Type	Effective Unit Weight (pcf)	Friction Angle (degrees)	Subgrade Modulus (pci)
From	To	From	To				
0	10	580	570	Sand (Reese)	120	31	40
10	25	570	555	Sand (Reese)	130	35	120
25	37	555	543	Sand (Reese)	130	37	140
37	45	543	535	Sand (Reese)	65	37	85
45	60	535	520	Sand (Reese)	65	40	125

Notes:
* Approximate
pcf = pounds per cubic foot
pci = pounds per cubic inch

Lateral Earth Pressures on Wingwalls

We understand that wingwalls at both abutments will retain existing and new site fill. Active earth pressure against the walls will be the primary controlling parameter for design. For walls constructed against existing and new site soils, we recommend using the active earth pressure parameters presented in Table 2:

TABLE 2
RECOMMENDED SOIL PARAMETERS FOR WINGWALL DESIGN

Soil Type	Approximate Layer Depth (feet bgs)*	Active Earth Pressure (psf/ft)	Active Earth Pressure Coefficient, Ka	Soil Friction Angle (degrees), φ**
Existing and New Fill	0 – 10	40	0.31	32

Notes:
* From approximate elevation 580 feet.
** Based on our knowledge of and previous work with structural fill.
bgs = below ground surface
psf/ft = pounds per square foot per foot

The estimates in Table 2 are based on a soil unit weight of 120 pounds per cubic foot and on the assumption that the walls include proper drainage, so hydrostatic pressures will not build up.

The static active pressures act on the walls in a triangular distribution. We recommend increasing the active lateral pressures by a factor of 10H in a uniform pressure distribution (where H is the height of the wall) for seismic conditions. This value is based on the inclusion of a peak ground acceleration (PGA) coefficient determined using seismic mapping data for the Project site provided by the U.S. Geological Survey (USGS, 2009) and estimated using the Mononobe-Okabe trial wedge analysis. Structural fill placed behind walls should be compacted to a dense and unyielding condition.

Seismic Design Parameters

Our seismic design recommendations are intended for design in accordance with the 2017 Load and Resistance Factor Design Bridge Design Specifications, eighth edition, as outlined by the American Association of State Highway and Transportation Officials (AASHTO, 2017). Characterization of soil profile type is required in the AASHTO specifications to determine the site class definition. Based on the SPT N-values and soil classifications derived from available explorations completed at the project site, it is our opinion that the project site could be adequately classified as Site Class D.

AASHTO criteria specify that bridge design and evaluations be based on earthquake ground motions with a 7 percent chance of exceedance in 75 years (approximately 1,000-year return period). Based on ground motion hazard studies conducted by the USGS, the PGA for the site is 0.39g for a recurrence interval of 1,000 years (site-factored PGA of 0.43 for Site Class D – with F_{PGA} of 1.11). The recommended site-factored horizontal response spectrum is presented in Table 3:

TABLE 3
2018 AASHTO PARAMETERS FOR SEISMIC DESIGN OF STRUCTURES

Spectral Response Acceleration (SRA) and Site Coefficients	Short Period	1-Second Period
Mapped SRA[1, 2]	$S_S = 0.86$	$S_1 = 0.28$
Site Coefficients	$F_a = 1.16$	$F_v = 1.83$
Design SRA[1,2]	$SD_s = 1.00$	$SD_1 = 0.52$

Notes:
(1) Mapped SRA and Design SRA values are in units of gravity.
(2) The SRA values are based on regional probabilistic ground motion studies conducted by USGS and determined using the USGS Java ground motion parameter calculator. USGS maps corresponding to AASHTO specifications were provided using 2002 hazard data.

DRILLING CONSIDERATIONS

The drilling Contractor should be prepared to drill through relatively dense layers of soil containing large cobbles and boulders. This may require rock coring using a core barrel or relocating to a different drilling location if an obstruction is encountered through which the drill cannot advance. The Contractor should be prepared to drill for extended periods of time to advance through particularly dense layers or obstructions.

We recommend that the Contractor use a drill rig that has the appropriate specifications to drill in material such as that present at the project site. Due to the possibility of encountering groundwater or perched water during drilling and the granular nature of the alluvial outwash soils, drilled shafts should be cased to prevent caving within the holes. The driller should also be prepared to add water to the holes ("waterhead") to prevent heave of sand layers if groundwater is encountered.

CLOSURE AND LIMITATIONS

This letter was prepared for the exclusive use of Icicle Creek Engineers for the design of the Green to Cedar River Trail South Segment – BNSF Pedestrian Bridge. This letter should be relied on for factual data only, and not as a warranty of subsurface conditions, such as those interpreted from our observations and review of previous subsurface explorations. The conclusions and recommendations contained in this letter are based on site conditions observed during our site visit and review of the subsurface information referenced in this letter.

Within the limitations of the scope, schedule, and budget, the conclusions and recommendations presented in this letter were prepared in accordance with generally accepted professional geotechnical engineering principles and practices in the area at the time this letter was prepared. We make no other warranty, either express or implied.

Unanticipated soil conditions are commonly encountered and cannot be fully determined with data from test borings or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

The scope of our present work did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Shannon & Wilson, Inc. has prepared the enclosed "Important Information About Your Geotechnical/Environmental Report" to assist you and others in understanding the use and limitations of this report.

Sincerely,

SHANNON & WILSON, INC.

Justin P.B. Cook, PE
Geotechnical Engineer

MARTIN W. PAGE
STATE OF WASHINGTON
30308
REGISTERED
PROFESSIONAL ENGINEER

1/11/19

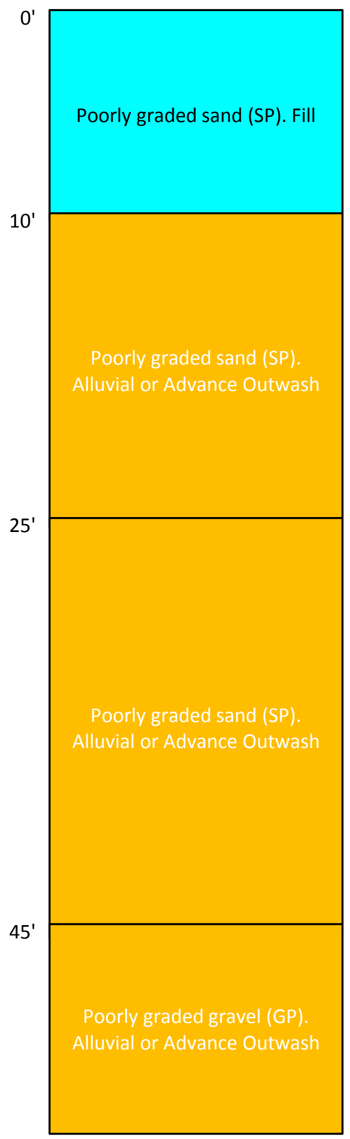
Martin W. Page, PE, LEG
Vice President

JPC:MWP/jpc

Enc: Figure 1 – Estimated Axial Shaft Resistance, 3-Foot-Diameter Shaft
Figure 2 – Estimated Axial Shaft Resistance, 4-Foot-Diameter Shaft
WSDOT H-1-88
WSDOT H-2-88
WSDOT H-3-88
GEI TP-01
GEI TP-02
GEI TP-03
GEI TP-04
Important Information About Your Geotechnical/Environmental Report

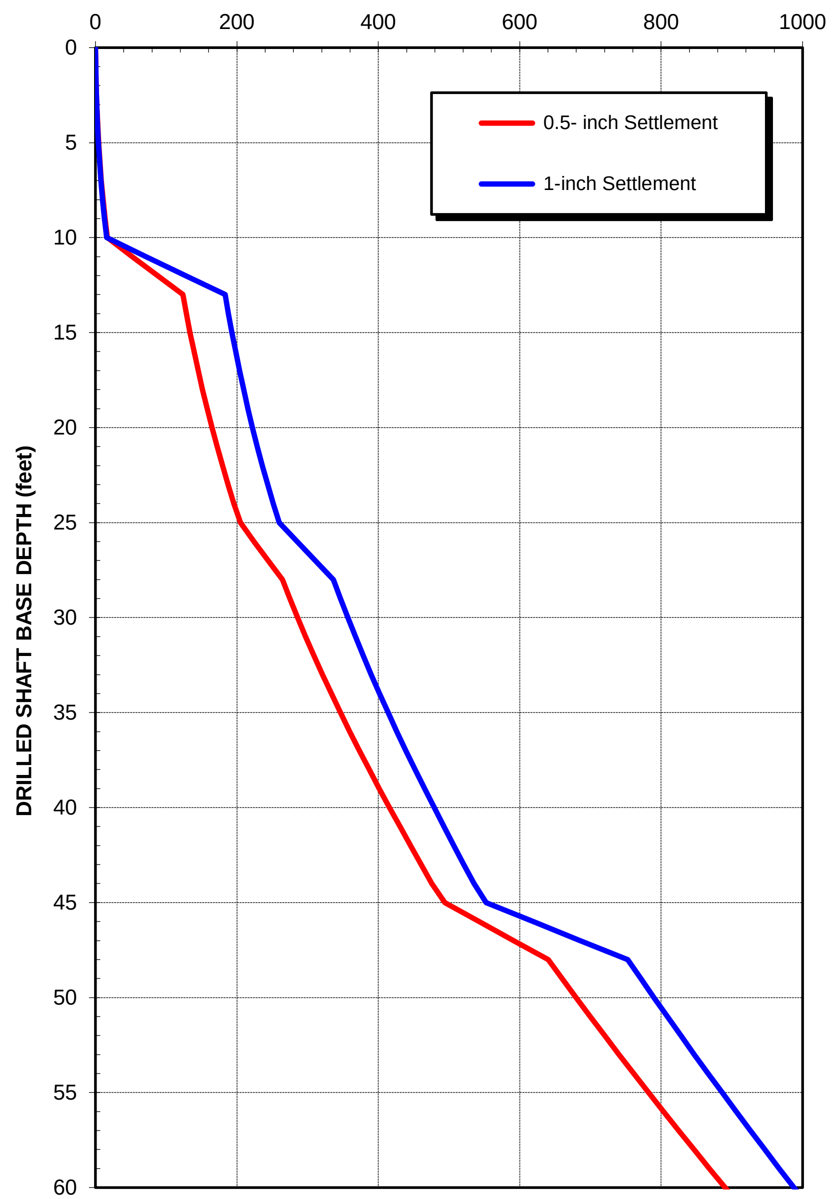
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
H-1-88, H-2-88, H-3-88



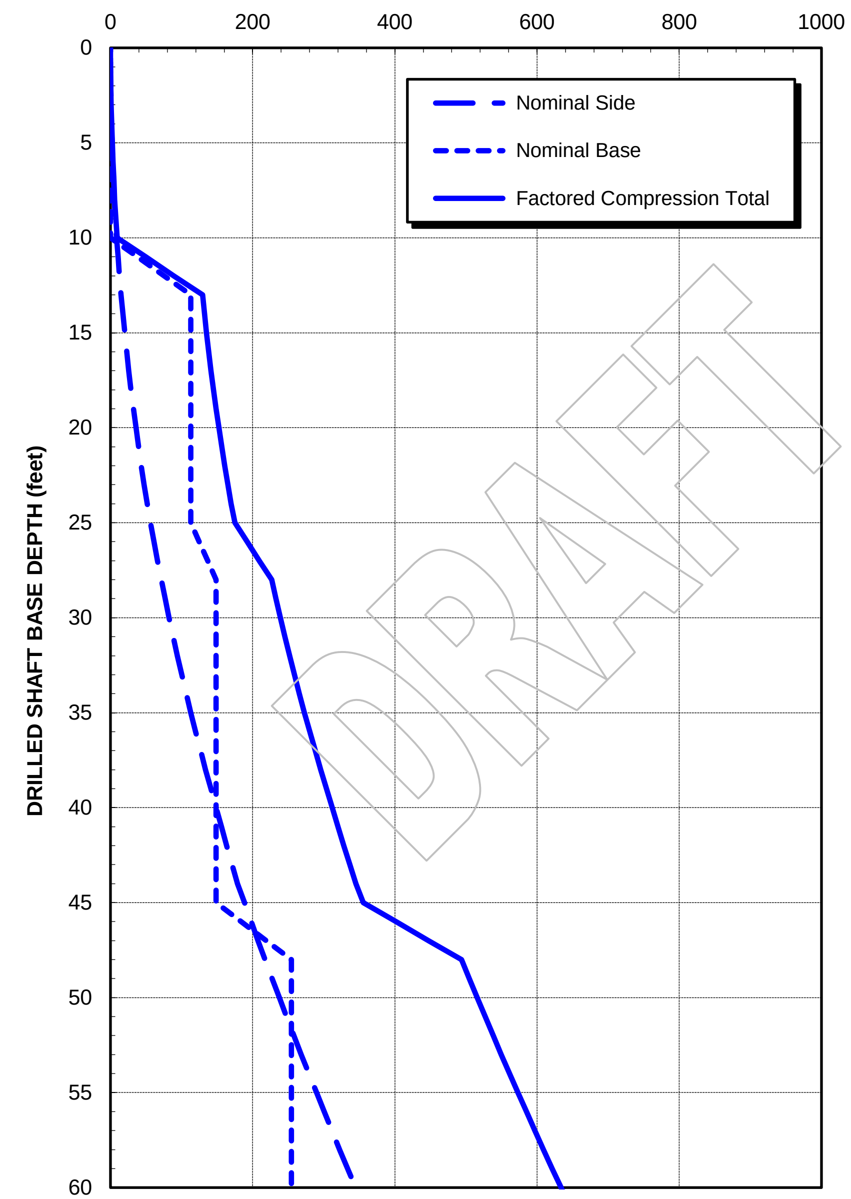
SERVICE LIMIT

NOMINAL RESISTANCE (kips)



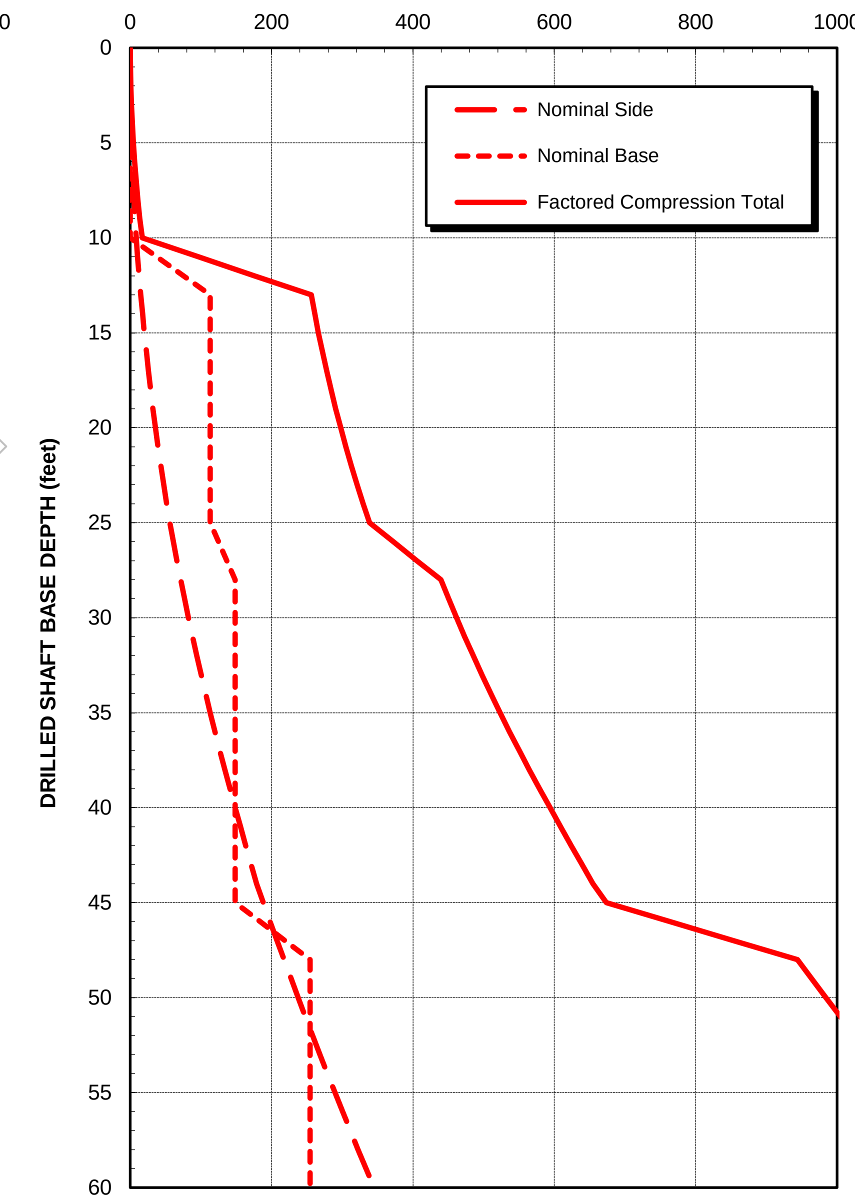
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (kips)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

Green to Cedar River Trail South Segment
BNSF Pedestrian Bridge
Black Diamond, WA

ESTIMATED AXIAL SHAFT RESISTANCE
3-FOOT-DIAMETER SHAFT

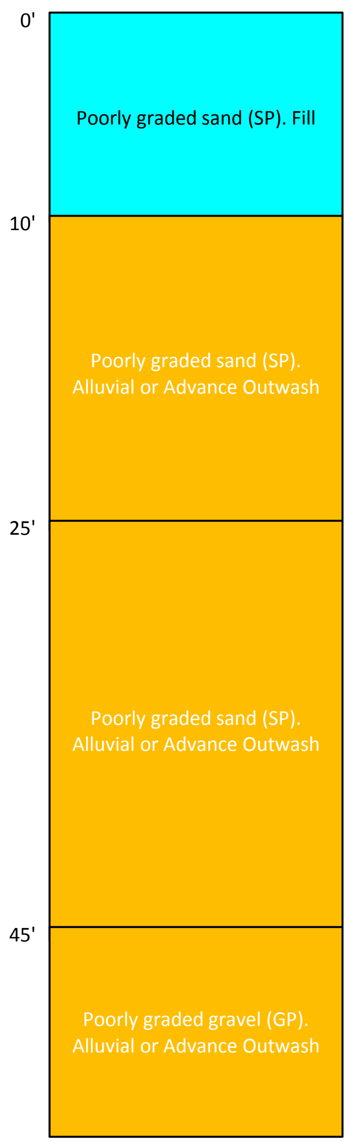
January 2019 101541-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

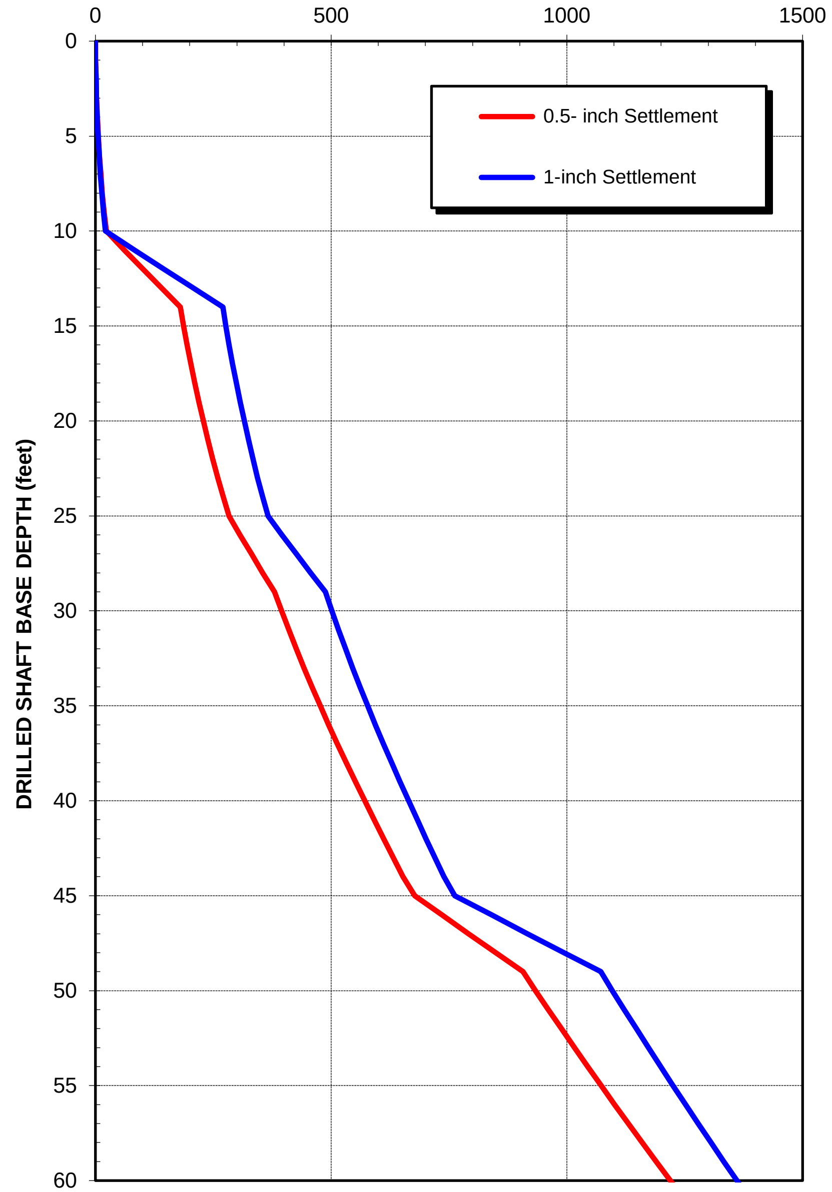
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
H-1-88, H-2-88, H-3-88



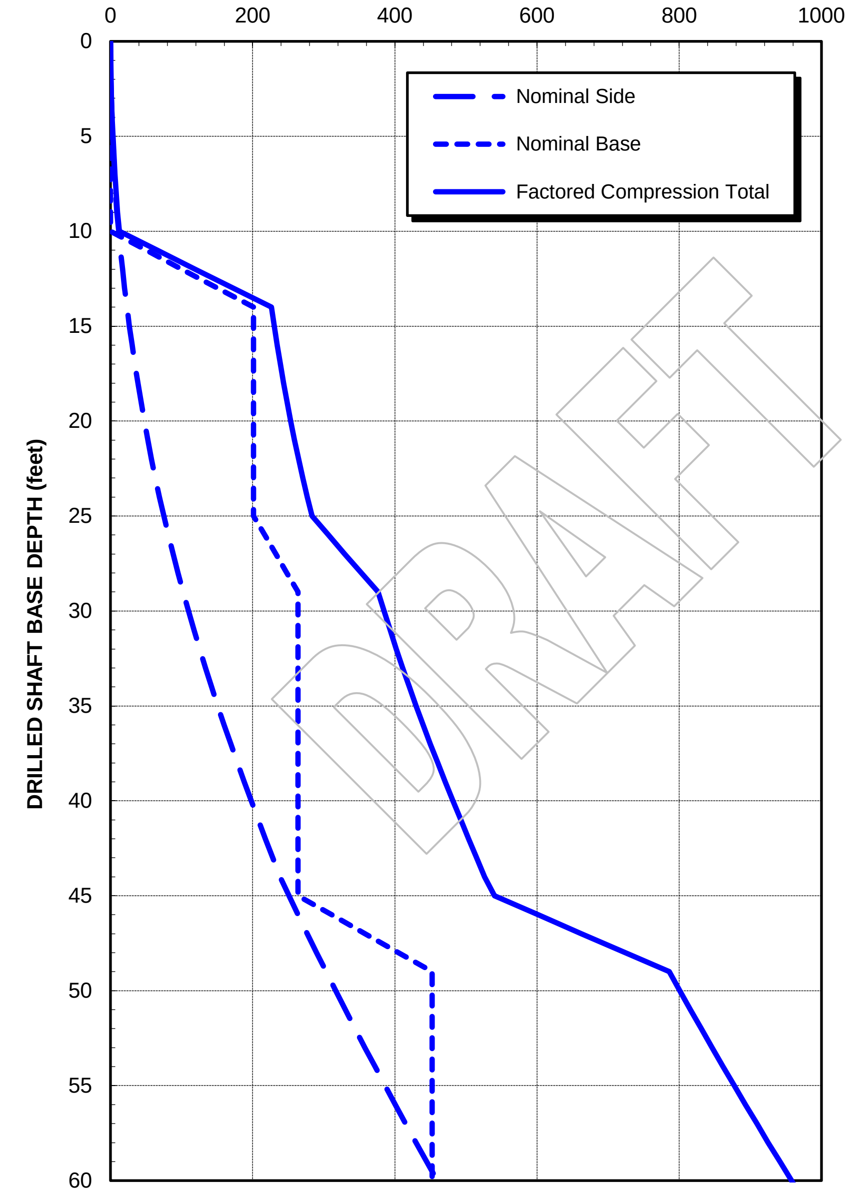
SERVICE LIMIT

NOMINAL RESISTANCE (kips)



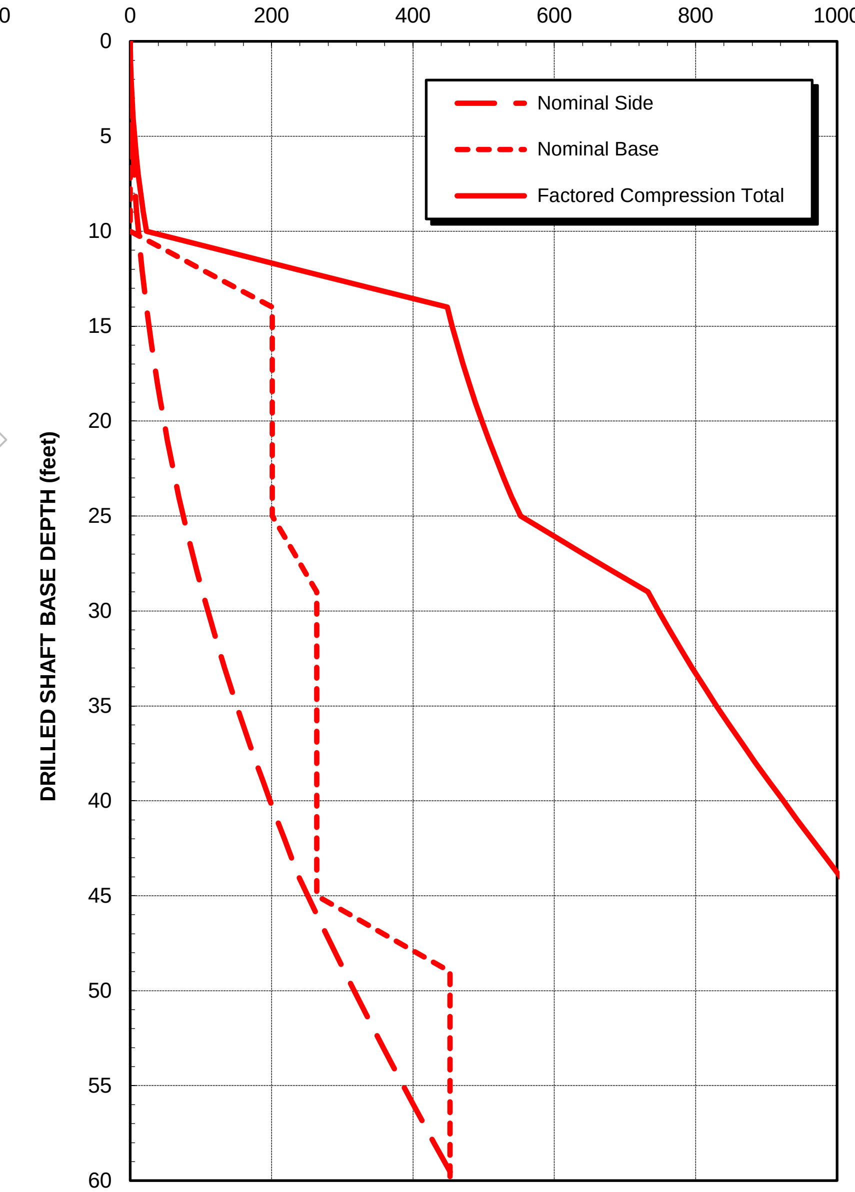
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (kips)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
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4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

Green to Cedar River Trail South Segment
BNSF Pedestrian Bridge
Black Diamond, WA

ESTIMATED AXIAL SHAFT RESISTANCE
4-FOOT-DIAMETER SHAFT

January 2019 101541-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2

LOG OF TEST BORING — **WASHINGTON STATE DEPARTMENT OF TRANSPORTATION**

S.H. ____ S.R. 169 SECTION BN RR O/C Bridge No. 169/12 Job No. L-8872

Hole No. H-1-88 Sub Section ____ Cont. Sec. 1734

Station 94+36 Offset 21' Rt. ℄ Ground El. 560.0' Contour on Layout

Type of Boring Augers Casing 4.5 x 52' W.T. El. 543.0'

Inspector ____ Date August 16, 1988 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	15		6 6 9 STD PEN 1	GP, M.C. = 0.9% Medium dense, gray, dry, slightly silty, fine to coarse sandy, sub-angular GRAVEL with cobbles. Retained 8".
5				
	20		8 12 8 STD PEN 2	Medium dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 10".
10				
	43		10 21 22 STD PEN 3	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 11".
15				
	34		10 15 19 STD PEN 4	Dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 8".
20				

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

Copy to ____

Hole No. H-1-88 Sub Section BN RR O/xing Bridge No. 169/12 Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	36		12 16 20 STD PEN 5	Dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 12".
25				
	65/6		35 65 STD PEN 6	GW Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 12".
30				
	64		21 28 36 STD PEN 7	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 18".
35				
	93		25 35 58 STD PEN 8	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 12".
40				
	70/6"		70 STD PEN 9	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 6".
45				

Hole No. H-1-88 Sub Section BN RR O'xing Bridge No. 169/12 Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	50/3"		43 50/ 3" ↕ STD PEN 10	Very dense, gray, wet, slightly sitly, fine to coarse sandy. sub rounded to angular GRAVEL. Retained 9".
50				
	50/2"		41 50/ 2" ↕ STD PEN 11	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub rounded to angular GRAVEL. Retained 8".
55				
				Stopped Test Boring 52' 8" below ground elevation.
				Bensealed Hole
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

LOG OF TEST BORING — WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ____ S.R. 169 SECTION BN RR O/C Bridge No. 169/12 Job No. L-8872

Hole No. H-2-88 Sub Section ____ Cont. Sec. 1734

Station 93+95 Offset 19' Lt. ℄ Ground El. 581.0' Contour on Layout

Type of Boring Augers Casing 4" x 10" x 48' W.T. El. 539.0'

Inspector ____ Date August 17, 1988 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				1' to 2' 6" cobbles and boulders
5	13		5 6 7 STD PEN 1	SP, M.C. = 4.6% Medium dense, brown, dry, slightly silty, gravelly, fine to coarse SAND. Retained 8".
10	9		4 4 5 STD PEN 2	Loose, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 2".
15	26		10 12 14 STD PEN 3	SP Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
20	67		14 30 37 STD PEN 4	Very dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 6".

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

Copy to ____

Hole No. H-2-88 Sub Section BN RR O'xing Bridge No. 169/12 Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
25	31		10 / 14 / 17 STD PEN 5	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 10".
30	35		11 / 16 / 19 STD PEN 6	SW/SM, M.C. = 4.2% Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
35	59		20 / 24 / 35 STD PEN 7	Very dense, gray,moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
40	40		12 / 20 / 20 STD PEN 8	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
45	41		18 / 20 / 21 STD PEN 9	GP/GM Dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 12".

Hole No. H-2-88 Sub Section BNRR O'xing Bridge No. 169/12 Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	65		18 STD 32 PEN 33 10	Very dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 6".
50				
				Stopped Test Boring 49' 6" below ground elevation.
				Ben Seal and Hole Plug.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

LOG OF TEST BORING WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

___ S.H. ___ S.R. 169 SECTION BN RR O/C Bridge No. 169/12 Job No. L-8872

Hole No. H-3-88 Sub Section ___ Cont. Sec. 1734

Station 95+27 Offset 27' Rt. ℄ Ground El. 579.5' Contour on Layout

Type of Boring Augers Casing 4" x 48' W.T. El. 537.5'

Inspector ___ Date August 18, 1988 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				0' to 3' Cobbles and boulders
	14		6 STD 7 PEN 7 1	SW/SM, M.C. = 6.2% Medium dense, brown, dry, silty, gravelly, fine to coarse SAND. Retained 8".
5				
	7		3 STD 3 PEN 4 2	Loose, brown, dry, silty, gravelly, fine to coarse SAND. Retained 3".
10				
	28		11 STD 14 PEN 14 3	Dense, gray, dry, slightly silty, gravelly, fine to coarse SAND. Retained 9".
15				
	29		9 STD 11 PEN 18 4	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 10".
20				

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

Copy to ___

Hole No. H-3-88 Sub Section BN RR O'xing Bridge No. 169/12 Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	33		11 STD 16 PEN 17 5	Dense, gray,moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
25				
	34		12 STD 16 PEN 18 6	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 10".
30				
	42		14 STD 20 PEN 22 7	Dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
35				
	77		14 STD 27 PEN 50 8	Very dense, gray, moist, slightly silty, gravelly, fine to coarse SAND. Retained 12".
40				
	50/3"		50/ STD 3" PEN 9	SW/SM Very dense, brown, wet, silty, gravelly, fine to coarse SAND. Retained 3".
45				

Hole No. H-3-88 Sub Section BN RR O'xing Bridge No. 169/12 Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	100/3"		50 100/ 3" STD PEN 10	GW/GM Very dense, gray, wet, slightly silty, fine to coarse sandy, sub angular GRAVEL. Retained 9".
50				
				Benseal and hole plug.
				Stopped Test Boring 48' 9" below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

Date Excavated: 7/20/2015 Logged By: BHC

Equipment: TB235 Mini trackhoe Total Depth (ft) 7.5

Elevation (feet)	Depth (feet)	SAMPLE Testing Sample	SAMPLE Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				GM		Brownish-gray silty gravel with sand (dense, dry)		
	1	X	S-1						Probe = 3 to 4 inches at 1 foot
	3				GM		Brown silty gravel with sand and cobbles (very dense, dry to moist)		
	3.5	X	S-2						Oxidation staining Probe < 1 inch at 3.5 feet
	5				GP		Brown gravel with sand (very dense, moist)		
	5.5	X	S-3						

Test pit completed at 7.5 feet
No groundwater seepage observed
Moderate caving observed at 3 to 5 feet

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-01

GeoEngineers

Project: Green to Cedar Rivers Regional Trail Project
Project Location: King County, Washington
Project Number: 1813-017-00

Date Excavated: 7/20/2015

Equipment: TB235 Mini trackhoe

Logged By: BHC

Total Depth (ft) 7.0

Elevation (feet)	Depth (feet)	SAMPLE Testing Sample	SAMPLE Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Brown-gray silty fine to coarse sand with gravel and cobbles (very dense, dry)		
	1	X	S-1		GP-GM		Brown gravel with silt, sand and cobbles (very dense, dry)		Probe = 1 to 2 inches at 1 foot
	2								
	3	X	S-2						Probe = 0 to 1 inch at 3 feet
	4								
	5	X	S-3						
	6								
	7								

Test pit completed at 7 feet
No groundwater seepage observed
No caving observed

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-02

GEOENGINEERS

Project: Green to Cedar Rivers Regional Trail Project

Project Location: King County, Washington

Project Number: 1813-017-00

Date Excavated: 7/20/2015

Equipment: TB235 Mini trackhoe

Logged By: BHC

Total Depth (ft) 7.5

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				GM		Brown silty gravel with sand and cobbles (medium dense, dry)		Probe = 1 foot depth at surface Abundant roots in top 2 feet
	1	X	S-1						
	2						Becomes very dense		
	3	X	S-2						Probe = 1 inch at 3 feet
	5				SM-GM		Orange-brown silty fine to coarse sand with gravel and cobbles (very dense, dry)		
	6.5	X	S-3						

Test pit completed at 7.5 feet
No groundwater seepage observed
No caving observed

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-03

GeoEngineers

Project: Green to Cedar Rivers Regional Trail Project
Project Location: King County, Washington
Project Number: 1813-017-00

Date Excavated: 7/20/2015
Equipment: TB235 Mini trackhoe

Logged By: BHC
Total Depth (ft) 7.0

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Dark brownish-gray silty fine to coarse sand with gravel and cobbles (medium dense, dry)		Abundant roots
	1	X	S-1						
	1.5				GM		Light brown silty gravel with sand and cobbles (medium dense, dry)		
	2	X	S-2						Probe = 1 inch at 2 feet Harder digging
	3						Becomes very dense		
	4								
	5								
	6	X	S-3						
	7						Test pit completed at 7 feet No groundwater seepage observed Slight caving observed at 3 feet		

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

Log of Test Pit TP-04

GeoEngineers

Project: Green to Cedar Rivers Regional Trail Project
Project Location: King County, Washington
Project Number: 1813-017-00

Date: January 11, 2019
To: Icicle Creek Engineers, Inc.
Attn: Mr. Brian R. Beaman, PE, LEG, LHG

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report"s recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland