December 2, 2020
HWA Project No. 2019-173-21

Environmental Science Associates
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Seattle, Washington 98107

Attn: Ms. Alaina Floor, EIT

Subject: **GEOTECHNICAL LETTER REPORT**
Langlois Creek Culvert Replacement
Snoqualmie Valley Watershed Improvement District
Carnation, Washington

Dear Alaina:

At your request, HWA GeoSciences Inc. (HWA) has completed a geotechnical engineering study for the project that will replace four culverts along Langlois Creek south of Carnation, Washington. For this study, HWA reviewed available subsurface data for the project, conducted a site reconnaissance, drilled one boring, performed engineering analyses, and prepared this letter report. This letter report presents our observations and interpretations of the site geology as well as our conclusions and recommendations based on the results of the engineering study.

PROJECT BACKGROUND

The Snoqualmie Valley Watershed Improvement District (SVWID) would like to replace four existing culverts along Langlois Creek with structures enabling fish passage. One culvert, designated #101SC-07, crosses underneath the historic railroad embankment, which supports the Snoqualmie Valley Trail and is owned by King County Parks. The three remaining culverts support farming access roads, two of which (designated #933062 and #933064) are on Remlinger Farm property and one (designated #933063) that supports a Remlinger Farm access road on Puget Sound Energy (PSE) property.

Each of the culverts is to be replaced with a 4-sided precast concrete box culvert structure. The installation will require excavation into the road/trail embankment down to the base elevation of the box culverts. This includes excavation through the existing Snoqualmie Valley Trail embankment that is about 14 to 16 feet above the ground surface of the surrounding river valley. Work associated with the installation will include temporary stream diversions and other ground water control during excavations, removal of the existing culverts, excavation down to the proposed base of the new culverts, placement of the structures and backfill to restore the embankments.

FIELD INVESTIGATION

HWA drilled one borehole, designed BH-1, from the top of the trail embankment on March 31, 2020. Drilling was performed by Gregory Drilling, Inc. under subcontract to HWA, with a medium-sized track-mounted CME-55LCX rig using hollow stem augers. The boring extended to a depth of about 40.5 feet below the ground surface (bgs) and was terminated due to refusal within gravelly soils. The location of the boring is shown on the Site and Exploration Plan, Figure 2A-2D.

Soil samples were collected at 2½- to 5-foot depth intervals using Standard Penetration Test (SPT) sampling methods. SPT testing consisted of using a 2-inch outside diameter, split-spoon sampler driven with a 140-pound autohammer. 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 blow. 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.

All explorations were drilled under the full-time supervision and observation of a geotechnical engineer from HWA. Soil samples obtained from the explorations were classified in the field and representative portions were placed in plastic bags. These soil samples were then taken to our Bothell, Washington laboratory for further examination and laboratory testing.

Pertinent information including soil sample depths, stratigraphy, engineering characteristics, and ground water occurrence was recorded and used to develop logs of each of the explorations. A Legend of Terms and Symbols used on Exploration Logs is presented on Figure A-1, and the borehole log is presented on Figure A-2.

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

LABORATORY TESTING

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

EXISTING SUBSURFACE DATA

HWA reviewed data previously obtained by the United Stated Department of Agriculture (USDA) for two of the four culverts. This data was provided in their geotechnical investigation reports, which are listed below and are provided in Appendix C.

- USDA, 2019, *ENG – Trip Report, Remlinger Farms Bridge – Site 1, King County, WA,* submitted to Larry Johnson, State Conservation Engineer, dated June 25, 2019.
    - This report provides the results of two borings, designated DH-1 and DH-2, drilled about 135 feet east of the existing culvert #933063 on the PSE property and 210 feet east of the Snoqualmie Valley Trail. Boring DH-1 was extended about 18 feet bgs and DH-2 extended 16.5 feet. Both were terminated due to heave.
- USDA, 2019, *ENG – Trip Report, Remlinger Farms Bridge – Site 2, King County, WA,* submitted to Larry Johnson, State Conservation Engineer, dated June 26, 2019.
    - This report provides the results of two borings, designated DH-1 and DH-2, drilled on a paved access road about 2,800 feet east of the Snoqualmie Valley trail and about 800 feet north of NE 24th Street. These explorations were conducted immediately north and south of the proposed culvert #933064. Boring DH-1 was drilled to 13 feet below the ground surface (bgs) and DH-2 extended 15 feet bgs. DH-1 was terminated due to rig being out of plumb and DH-2 was terminated due to heave.

GENERAL GEOLOGIC CONDITIONS

The project is located within the Cascade foothills, at the eastern margin of the Puget Lowlands. The Puget Lowlands have repeatedly been occupied by a portion of the continental glaciers that developed during the ice ages of the Quaternary period. During at least six 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 Tenino, 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 Lowlands in lakes and valleys.

Geologic information for the project area was obtained from the *Geologic Map of the Carnation 7.5-Minute Quadrangle, King County, Washington* (Dragovich et al, 2010). According to this 1:24,000 scale map, Quaternary glacial recessional outwash of Fraser Glaciation age, in particular glaciolacustrine and glacial fluvial outwash, are mapped to the south of the project alignment, and Quaternary alluvial fan deposits are mapped to the north of the project alignment. The project area is located at the terminus of an east to northwest-trending valley, terminating in the Carnation Basin on the Tolt River Delta. Langlois Creek drains through the valley. Topographic highs to the north and south of the valley are mapped as Quaternary glacial recessional outwash and Quaternary ice contact deposits.

SITE CONDITIONS

The western limit of the project is at the Langlois Creek stream crossing of the Snoqualmie Valley Trail. At this point, Langlois Creek branches off and part of the creek flows north to the Tolt River and the other part flows southwest to the Snoqualmie River. Most of the site is relatively flat at about Elev. 82 to 84 feet above mean sea level (AMSL). The elevation does begin to gently rise to the east where the creek flows out from the base of the foothills east of Carnation.

The existing 24-inch diameter ductile iron pipe (culvert #101SC-07) that crosses under the Snoqualmie Valley Trail is at the base of the historic railroad embankment, which is built up about 14 to 16 feet above the surrounding river valley. The top of the embankment is at approximately Elev. 98 feet AMSL with the surrounding valley at about Elev. 82 to 84 feet AMSL. The culvert invert ranges from about Elev. 76.3 feet AMSL on the downstream end to about Elev. 77.1 feet AMSL on the upstream end. The side slopes of the railroad embankment in this section range from about 1½ to 2 horizontal to 1 vertical.

Located about 40 feet upstream of culvert #101SC-07, culvert #933063 crosses under the Remlinger Farms access road that is located on PSE property. This culvert consists of a 24-inch concrete pressure pipe with an invert at about Elev. 76.6 feet AMSL on the downstream end and about Elev. 76.9 feet AMSL on the upstream end. The top of the access road at this location is at about Elev. 84 feet AMSL.

Further upstream are culverts #933062 and #933064. Culvert #933062 is located upstream of the trail about 1,300 feet and culvert #933064 is upstream another 1,500 feet beyond that. Both culverts consist of a 36-inch diameter corrugate metal pipe (CMP). At culvert #933062, the roadway elevation is at about Elev. 82 feet AMSL. The invert elevations for this culvert are about Elev. 77.2 feet AMSL at the downstream end, while the upstream end is lower at about Elev. 76.8 feet AMSL. The elevation of Langlois Creek increases between culvert #933062 and culvert #933064, the latter of which is near the base of the hills along the east side of the Snoqualmie Valley. The approximate roadway elevation here is Elev. 100 feet AMSL. At culvert #933064, the downstream invert is at about Elev. 91.7 feet AMSL and the upstream invert is at about Elev. 91.5 feet AMSL.

SUBSURFACE CONDITIONS

SOIL CONDITIONS

The soils encountered in our explorations consist of fill and alluvium. Further descriptions of the soils encountered in the explorations performed for the culvert projects are presented below in order of deposition, beginning with the most recently deposited. The exploration logs in Appendix A provide more detail of subsurface conditions observed at specific locations and depths.

- **Fill:** Fill was observed in each of the borings except boring DH-2 at culvert #933063. The fill was fully penetrated in all but boring DH-1 at culvert #933064, with depths ranging from about 8.5 to 20 feet. The fill consisted of very loose to medium dense, gravelly, slightly silty to silty sand at culvert #933063. The fill consisted of loose to medium dense, silty, gravelly sand to sandy gravel at culverts #101SC-07 and #933064. These deposits ranged from fresh to oxidized, and contained organics and woody debris. No observations of fill depth were observed at culvert #933062 as no borings were conducted at that location.
- **Alluvium:** Each of the borings, except for boring DH-1 at culvert #933064, encountered silt and silty sand alluvium with varying amounts of gravel beneath the fill. The relative density varied from very loose to medium dense, for granular materials and stiff for silt. The alluvium extended the full depth of the boreholes. The deposits ranged from fresh to oxidized and contained, peat, organics and woody debris. Wood was encountered in boring DH-2 at culvert #933063 from about 13.5 to 15 feet and was interpreted as a buried log. The alluvium is interpreted from the boreholes and site geomorphology to be floodplain deposits, resulting from bank overflow from Langlois Creek and the Tolt River.

GROUND WATER CONDITIONS

Ground water was observed during drilling in each of the boreholes drilled for the project. At culvert #101SC-07, the location of BH-1, ground water was noted at about 20 feet below the top of the trail. At culvert #933063, ground water was noted in both borings as free water at about 3.5 feet bgs. At #933064, ground water was noted in both borings as free water at about 11.5 feet bgs. Note that the ground water levels observed in borings are typically higher than those observed during drilling. We expect ground water levels will vary depending on location, streamflow levels, season, and the relative abundance of precipitation. Prospective contractors should be prepared to encounter and manage the ground water table observed within the alluvial soils at the site.

CONCLUSIONS & RECOMMENDATIONS

GENERAL

The four existing culverts will be replaced with precast concrete box culverts to provide fish passage. The proposed culvert and wing wall structures can be founded on the non-organic alluvial gravels, sands, and silts the underlie the site following proper subgrade preparation for the culvert.

Although the alluvium at the site is potentially liquefiable, the proposed improvements will provide sufficient support to limit impacts of liquefaction to the culverts and wing walls. However, some repair of the roadways and slopes at the abutments may be necessary following a moderate to large event.

The slopes will need to meet the OSHA requirements for sloped excavations. The fill and alluvium will classify as Type C soils and should be sloped no steeper than 1.5H:1V, provided they are adequately dewatered. The Contractor would be responsible to maintain safe slopes during the excavation and slopes may need to be adjusted in the field based on the conditions encountered at the time of the work.

To reduce the likelihood of loosening of the bearing soil we recommend the ground water be maintained at least 2 feet below the bottom of the excavation. This will likely require the use of dewatering wells, although it is possible that dry weather may allow the contractor to dewater using a sump and pump system.

The fill within the existing Snoqualmie Valley Trail embankment generally consists of gravelly sand to sandy gravel and is suitable for reuse a structural fill. Alluvial soils that are non-organic and have silt contents less than 30 percent may be reused as backfill provided they are placed within 3 percent of their optimum moisture content for compaction.

SEISMIC DESIGN PARAMETERS

Culvert structures for roadways and trails are commonly designed in accordance with the current American Association of State and Highway Transportation Official's (AASHTO) specifications, including the *LRFD Bridge Design Specifications* (AASHTO, 2017) and the seismic design parameters provided per the Washington State Department of Transportation (WSDOT) *Bridge Design Manual*. The AASHTO Code and the WSDOT BDM do not require consideration of seismic effects for the design of box culverts and buried structures; however, to assess the potential impacts of a large seismic event on the proposed culverts, walls, and slopes, we determined the associated seismic design parameters for design of the structures and evaluated the potential for liquefaction to occur at the site.

Seismic Design Acceleration Coefficients

Earthquake loading for seismic evaluation was developed in accordance with 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)* (WSDOT, 2019). 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 explorations and understanding of site geology, it is our opinion that the proposed walls are underlain by soils that would be consistent with Site Class D if no liquefiable materials were present at the site. Due to the presence of liquefiable materials, the site would classify as Site Class F. This would typically require a site-specific analysis; however, since culverts like this are mostly buried structures the fundamental periods are typically near zero and as a result site class is generally allowed to be determined based on the underlying soil conditions without the need of a site-specific analysis. As a result, design values associated with Site Class D would be appropriate for the purposes of our seismic evaluation.

The peak ground acceleration for the design level event (equal to a return period of 1,033 years) was obtained using BridgeLink, a program developed by WSDOT to incorporate the probabilistic seismic hazard parameters from the *2014 Updates to the National Hazard Maps* (Peterson, et al., 2014) as well as adopt the peak ground acceleration site coefficient (F_{pga}) provided in Table 11.8-1 of the ASCE 7-16. From BridgeLink, we obtained a mapped PGA of 0.368 g for the site. The corresponding site application factor for PGA (F_{pga}) for Site Class D is 1.232. Based on these values and using the General Procedure provided by AASHTO, we recommend using a design peak ground acceleration, designated the Acceleration Coefficient (A_s) by AASHTO, of 0.453 g for liquefaction susceptibility, wall design, and slope stability evaluations.

LIQUEFACTION CONSIDERATIONS

Liquefaction is a temporary loss of soil shear strength due to earthquake shaking. Loose to medium dense, saturated cohesionless soils are susceptible to earthquake-induced liquefaction; as well as low-plasticity silts and clays. To evaluate the liquefaction susceptibility of the soils, the simplified procedure originally developed by Seed and Idriss (1971), updated by Youd et. Al., (2001), and by Idriss and Boulanger (2004, 2006) was used. Result of these analyses are provided in the following sections.

Liquefaction for Culvert #101SC-07 at Snoqualmie Valley Trail

The subsurface conditions for culvert #101SC-07 are represented by the soils observed in boring BH-1. Analyses for this boring indicate that loose to medium dense, alluvial materials below the water table could liquefy during a moderate to large earthquake down to about Elev. 68 feet AMSL. We estimate that the static water levels could be near the Ordinary High-Water Level, which is estimated to be near Elev. 79 feet. If left in place, approximately 10 to 11 feet of soil would likely liquefy below the trail, resulting in differential vertical settlement and areas of slope instability. However, the installation of the proposed culvert and walls will remove most of the liquefiable materials, replacing them with the concrete culvert structure as well as non-liquefiable structural fill. Given the installation of the culverts and placement of structural fill, we consider this culvert to have a low potential for settlement or slope instability due to liquefaction. Additionally, the continuous and uniform structure of the new culvert will help to mitigate any differential settlements that may occur across the base of the culvert due the seismically induced settlements.

Liquefaction for Culverts #933062, #933063, and #933064

Liquefaction for the three culverts east of the trail was evaluated using the borings performed by the USDA in June 2019. Near culvert #933063, the borings were terminated in liquefiable soils at about 18 feet. Near culvert #933064, the borings were terminated in medium dense to dense gravel; however, the borings did not extend below the base of the proposed culvert, so it is not clear if soils below the base of the culvert are liquefiable. We anticipate that liquefaction induced settlement within the alluvial soils at each of the culvert locations, will range widely, from of the order of a few inches to a foot. The box culverts themselves will generally span over areas experiencing these settlements and will help to manage differential settlements that may be experienced. Some tilting of the culverts could occur; however, this will not likely impact the structural integrity of the box culvert structures. Some slumping of the streambanks upstream and downstream of the culverts, as well as rotational failures near the abutments may occur. This may require rebuilding of the roads up to the culverts or repair of the streambanks following a moderate to large earthquake.

SLOPE STABILITY

Trail Embankment Slopes

The current embankment slopes at the location of the existing culvert under the trail have become oversteepened with time. North and south of the culvert, the trail embankment slopes are about 2H:1V. Near the culvert the slopes are nearly 1.3H:1V. These slopes have likely steepened such that they are near the maximum angle that can be maintained for static conditions. Where the toes of the existing slopes have been eroded, seismic loading from a moderate to large earthquake is likely to result in slumping of the trail embankment. To provide adequate slope stability of the embankments, we recommend rebuilding the slopes to 2H:1V for the trail embankment once culvert installation is complete.

Stream Bank Slopes

The existing stream banks along Langlois Creek have several locations where the slopes are steeper than 1H:1V. These banks are generally less than about 4 feet high and are not likely to result in significant slope failures; however, the tops of the streambanks are likely to retreat episodically over time if left at their current slope.

In the areas where the streambank will be reconstructed, we recommend the slopes be regraded to no steeper than 2H:1V, except at points where the slopes tie into the existing streambank. To prevent future erosion of prepared embankment slopes the use of large aggregate or rip rap is often recommended. If such protection is determined to be necessary, the size of and material consistency of the gravel or rip rap should be determined by a person knowledgeable of the site stream velocities and experience in selecting appropriately sized rock for the embankment protection in this area.

CULVERT FOUNDATION DESIGN

The base of the culverts can be founded on the existing alluvial materials encountered in the borings. We recommend using a net allowable bearing capacity of 2,000 psf for the culvert subgrades at the proposed culverts #101SC-07, #933062, #933063, and #933064. For short-term wind and seismic loading conditions, these allowable bearing pressures may be increased by 33 percent.

CULVERT SUBGRADE PREPARATION

Culvert subgrades should be prepared by excavating 12 inches below the base of the culvert with a backhoe or excavator employing a smooth edge bucket to limit soil disturbance. If unsuitable soils are encountered at the base of the excavation, the excavation should extend up to two feet deeper than the proposed culver base and that at least 12 inches of the excavated soils should be replaced with 2- to 4-inch quarry spalls, rammed into place to achieve a dense condition. We recommend an HWA geotechnical engineer, or their representative, be present during foundation excavation and base preparation to verify that the base has been suitably prepared.

A non-woven geotextile meeting the requirements of Section 9-33.2(1) Table 3 of the WSDOT *Standard Specifications* (WSDOT, 2020), should be placed over the approved subgrade and the 2- to 4-inch quarry spalls followed by 12 inches of Crush Surfacing Base Course (CSBC) meeting the requirements of Section 9-3.9(3) of the WSDOT *Standard Specifications* (WSDOT, 2020). The CSBC should be compacted to at least 95 percent of its maximum dry density as determined by ASTM D 1557 (Modified Proctor).

Box culvert foundations constructed in accordance with these recommendations should experience no more than 1 inch of differential or total static (non-seismic) settlement.

LATERAL EARTH PRESSURES ON CULVERT WALLS

Where the walls are constrained against rotation, the earth pressures should be computed using at-rest earth pressures. A cut and cover, four-sided box culvert would be considered to be constrained against rotation. The walls of the culvert should be designed for a lateral pressure equal to an equivalent fluid pressure of 60 pounds per cubic foot (pcf). For sliding, lateral earth pressures will be resisted by equal and opposite loading on the far side of the culvert.

WING WALL DESIGN CONSIDERATIONS

Lateral Earth Pressures

We assume that the proposed wing walls will consist of precast concrete walls that will be designed by the Contractor. For wall design we recommend designing the walls using the equivalent fluid pressures provided in Table 1. Seismic earth pressures are provided based on a horizontal seismic acceleration coefficient k_h of one-half the peak ground acceleration or 0.227 g and a vertical seismic acceleration coefficient k_v of 0.0 g.

Table 1. Recommended Equivalent Fluid Pressures for Wing Walls

Static Pressures for Level Backslope	Static Pressures for 2H:1V Backslope	Seismic Pressures for Level Backslope	Seismic Pressures for 2H:1V Backslope
42 pcf*	55 pcf	56 pcf	83 pcf

* pcf = pounds per cubic foot

Minimum Embedment

We recommend wing walls have an embedment depth in front of the wall of at least 2 feet.

Bearing Capacity

We recommend using a net allowable bearing capacity of 2,000 psf for prepared wing wall subgrades at culverts #101SC-07 #933062, #933063, and #933064. For short-term wind and seismic loading conditions, these allowable bearing pressures may be increased by 33 percent.

Sliding Resistance

An ultimate coefficient of friction equal to 0.7 times the effective stress at the base of the wall can be used for sliding resistance if founded on at least 12 inches of CSBC. A factor of safety of 1.5 should be applied to the sliding resistance.

Drainage

Drainage should be provided behind the walls. The drains should consist of a 4- or 6-inch diameter, perforated, plastic pipes, bedded and backfilled with Gravel Backfill for Drains, as specified in Section 9-03.12(4) of the *WSDOT Standard Specifications* (WSDOT, 2020). The pipes should slope to drain to a suitable outlet.

WING WALL SUBGRADE PREPARATION

Wing wall subgrades should be prepared by excavating 24 inches below the base of the walls with a backhoe or excavator employing a smooth edge bucket to limit soil disturbance. If unsuitable soils are encountered at the base of the excavation, the excavation should extend up to two feet deeper than the base of wingwall foundation and the excavated soils should be replaced with at least 12 inches of 2- to 4-inch quarry spalls, rammed into place to achieve a dense condition. We recommend an HWA geotechnical engineer, or their representative, be present during foundation excavation and base preparation to verify that the base has been suitably prepared.

A non-woven geotextile meeting the requirements of Section 9-33.2(1) Table 3 of the WSDOT *Standard Specifications* (WSDOT, 2020), should be placed over the approved subgrade and the 2- to 4-inch quarry spalls followed by at least 12 inches of Crush Surfacing Base Course (CSBC) meeting the requirements of Section 9-3.9(3) of the WSDOT *Standard Specifications* (WSDOT, 2020). The CSBC should be placed in lifts of no more than 8 inches thick and compacted to at least 95 percent of its maximum dry density as determined by ASTM D 1557 (Modified Proctor).

Wall subgrade and leveling pads constructed in accordance with these recommendations should experience no more than 1 inch of differential or total settlement.

EXCAVATIONS

Temporary excavations up to approximately 15 feet below the surrounding grade (and up to 30 feet below the top of the trail) will be made for emplacement of culverts and wing walls. For worker safety the sides of the excavations will need to be sloped or shored. Due to the presence of gravelly soils installation of sheet pile shoring is likely impractical.

All temporary cuts in excess of 4 feet in height should be sloped in accordance with Part N of Washington Administrative Code (WAC) 296-155, or be shored. The existing fill and alluvial soils at the site classify as Type C Soil; temporary unsupported excavations in Type C soils may be no steeper than 1.5H:1V (horizontal:vertical). The recommended maximum allowable temporary cut slope inclinations are applicable to temporary excavations above the water table only.

DEWATERING DURING CONSTRUCTION

Dewatering of excavations will likely be necessary, even during the dry season, to prevent loosening of bearing soils. Should loosening occur, the soils could become susceptible to quick conditions and/or post-construction settlement. The contractor should perform dewatering such that the ground water level is maintained at least two feet below the bottom of the deepest portion of the excavations for the duration that the excavation remains open. We anticipate that wells or a well point system will be needed. Dry weather may allow the contractor to dewater using a sump pump system; however, the method must maintain the water level below the base of the excavation for the duration that the excavation is open.

Dewatering during construction should be the responsibility of the contractor. The contractor should be required to submit a dewatering plan and calculations for approval prior to the initiation of excavation. The plan should include a monitoring well installed to verify that the water table at the excavation area is sufficiently withdrawn (at least 2 feet below the bottom center of the excavation) prior to onset of the excavation.

STRUCTURAL FILL AND COMPACTION

Fill materials placed to reconstruct the trail embankment, roadways and backfill behind walls should consist of granular soils having a maximum particle size of 4 inches, and be free of any organics, or other deleterious materials. The existing sand and gravel within the Snoqualmie Valley Trail embankment are likely suitable to reuse as structural fill, as well as non-organic sands to silty sands that make up the alluvial soils. Organic-rich soils and soils with high silt contents (i.e. fines greater than 30 percent) will be unsuitable for reuse as structural fill. Soil excavated from below the ground water table will likely require drying prior to compaction. Prior to placement the engineer should verify that the proposed soils are suitable for reuse. Where import materials are required, we recommend using material meeting the requirements of Gravel Borrow, as described in Section 9-03.14(1) of the WSDOT *Standard Specifications* (WSDOT, 2020). Fines should be non-plastic.

Structural fill should be uniformly moisture conditioned to within about 3 percent of optimum moisture content prior to placement. Properly prepared backfill should be placed in uniform loose lifts not to exceed 12 inches and densely compacted in a systematic manner using appropriately sized compaction equipment to achieve at least 95 percent of the maximum dry density as determined using ASTM D 1557 (Modified Proctor). Smaller loose lifts may be necessary to achieve compaction where handheld compaction equipment such as jumping jacks, hoe-packs or plate compactors are used. The contractor should develop compaction methods that consistently produce adequate compaction levels.

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 of unsuitable and/or softened soil should be followed promptly by placement and compaction of clean structural fill. The size and type of construction equipment used may need to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic.

- For wet weather conditions, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight of the portion of the fill material passing the ¾-inch sieve. The fines should be non-plastic. It should be noted this is an additional restriction on the structural fill materials specified.

- The ground surface within the construction area should be graded to promote surface water run-off and to prevent ponding.

- Within the construction area, the ground surface 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 infiltration.

- Bales of straw and/or geotextile silt fences should be strategically located to control erosion and the movement of soil.

CONDITIONS AND LIMITATIONS

We have prepared this report for Environmental Science Associates, Inc. and Snoqualmie Valley Watershed Improvement District for use in design 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 provide geotechnical construction monitoring, testing, and consultation 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.

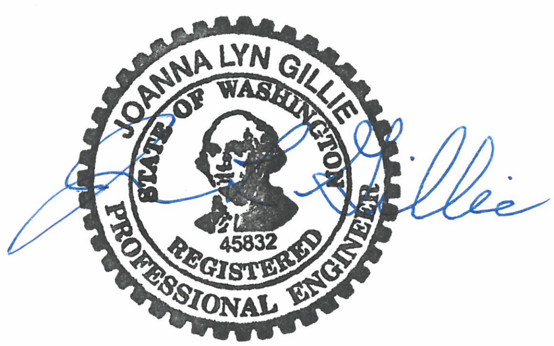
Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in the area at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of hazardous substances in the soil 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 the opportunity to be of service. Should you have any questions regarding this report, or require additional services, please contact us.

Sincerely,

HWA GeoSciences Inc.



JoLyn Gillie, P.E.
Geotechnical Engineer, Principal

Attachments:

Figure 1	Vicinity Map
Figure 2A-2D	Site and Exploration Plan
Appendix A	Borehole Log
Appendix B	Laboratory Testing
Appendix C	USDA Letter Reports for Culverts #933063 and #933064

REFERENCES

American Associate of State Highway and Transportation Officials (AASHTO), 2011, *Guide Specifications for LRFD Seismic Bridge Design*, 2nd Edition, Washington D.C.

AASHTO, 2017, *LRFD Bridge Design Specifications*, 8th Edition, Washington D.C.

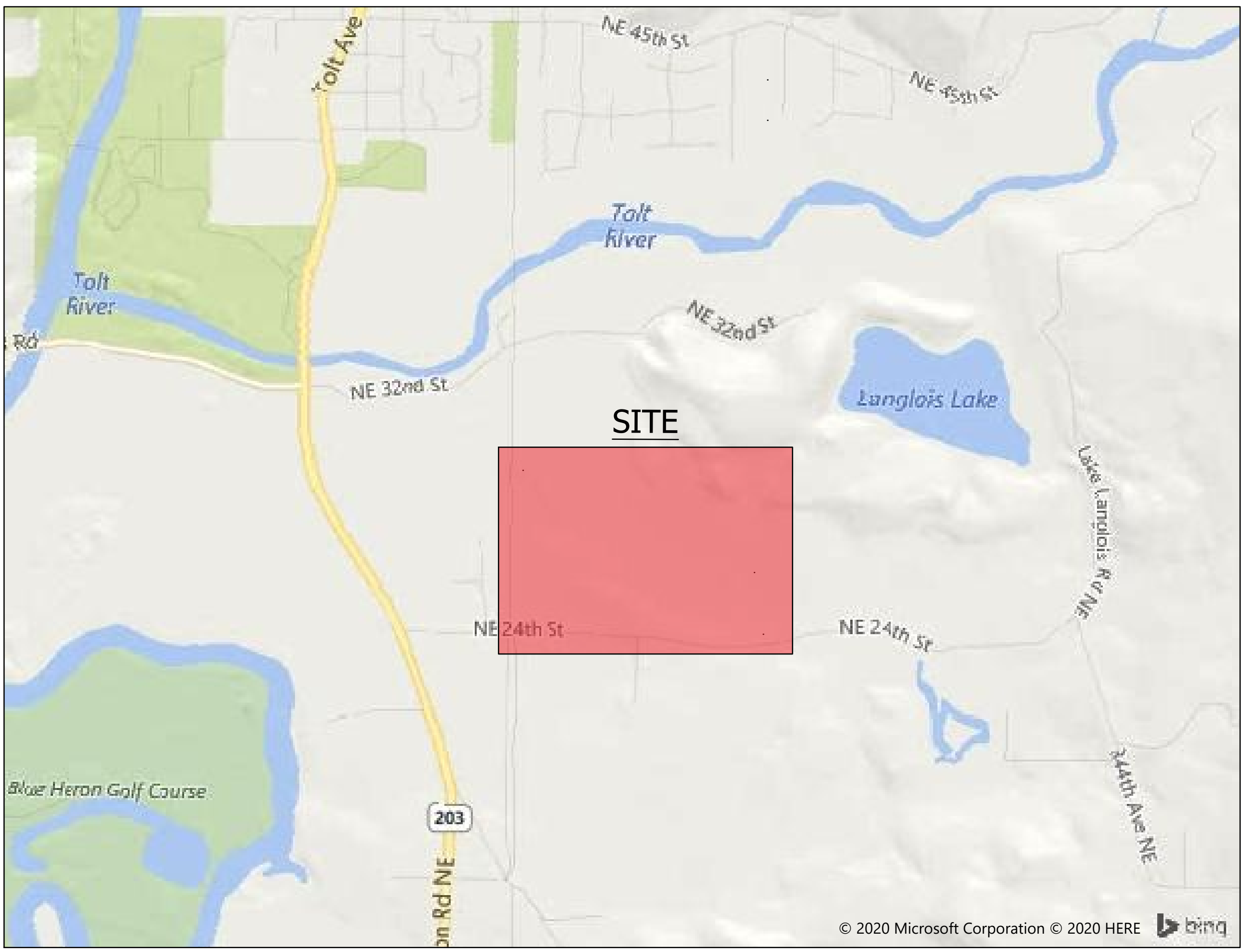
Dragovich et al, 2010, Geologic *Map of the Carnation 7.5-Minute Quadrangle, King County, Washington*.

WSDOT, 2019, *Bridge Design Manual (LRFD)*, M 23-50.19, July 2019.

WSDOT, 2020, *Standard Specifications for Road, Bridge, and Municipal Construction*, Washington State Department of Transportation.

SITE MAP

0 300' 600' 900' 1200'
SCALE: 1" = 600'
N



VICINITY MAP

0 1000' 2000' 3000' 4000'
SCALE: 1" = 2000'
N

HWA GEOSCIENCES INC.
DBE/MWBE

SITE AND VICINITY MAP

LANGLOIS CREEK CULVERT REPLACEMENT
CARNATION, WASHINGTON

FIGURE NO.: 1	
DRAWN BY: CF	CHECK BY: JG
PROJECT # 2019-173-21	

C:\USERS\CFRY\DESKTOP\2019-173-21 LANGLOIS CREEK CULVERTS\CAD\2019-173-21 LANGLOIS CREEK CULVERTS.DWG <1> Plotted: 5/4/2020 11:50 AM

LANGLOIS CREEK CULVERTS

Scale: 1" = 200'-0"

EXPLORATION LEGEND

- BH-1 — BOREHOLE DESIGNATION AND APPROX. LOCATION (CURRENT STUDY)
- DH-1-933063 — BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 25, 2019 FOR CULVERT 933063
- DH-1-933064 — BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 26, 2019 FOR CULVERT 933064

A A' GEOLOGIC CROSS SECTION

0 100 200 300 400

SCALE: 1" = 200'

BASE MAP PROVIDED BY: BING AND SURVEYOR

HWA GEOSCIENCES INC. DBE/MWBE

LANGLOIS CREEK CULVERT REPLACEMENT CARNATION, WASHINGTON	SITE & EXPLORATION PLAN	DRAWN BY: CF	FIGURE NO.: 2A
		CHECK BY: JG	PROJECT NO.: 2019-173-21

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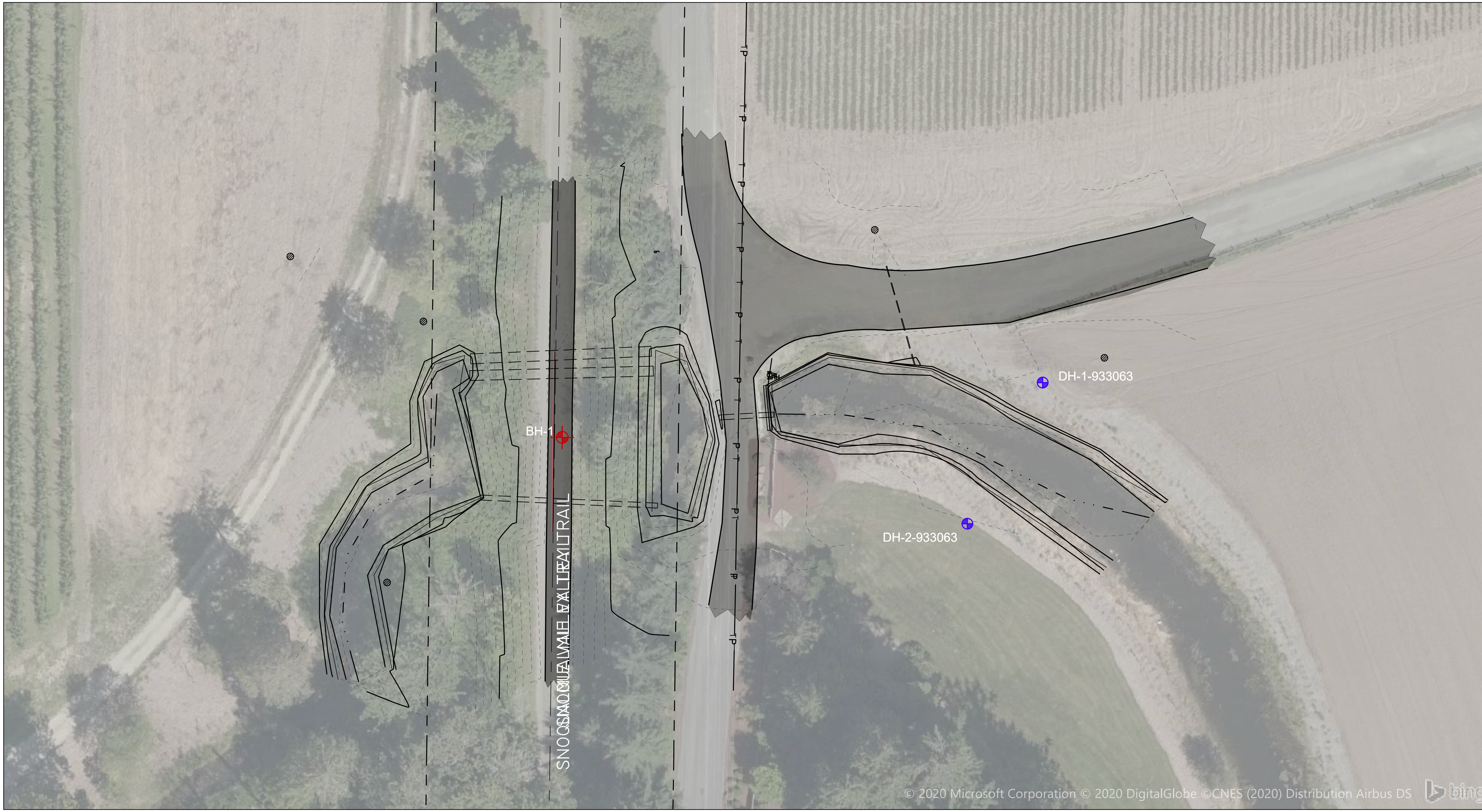

LANGLOIS CREEK CULVERTS 101SC-07 AND 933063

Scale: 1" = 40'-0"

EXPLORATION LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROX. LOCATION (CURRENT STUDY)

DH-1-933063 BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 25, 2019 FOR CULVERT 933063

DH-1-933064 BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 26, 2019 FOR CULVERT 933064

A A' GEOLOGIC CROSS SECTION

BASE MAP PROVIDED BY: BING AND SURVEYOR

N

0 20 40 60 80

SCALE: 1" = 40'

HWA GEOSCIENCES INC. DBE/MWBE

LANGLOIS CREEK CULVERT REPLACEMENT CARNATION, WASHINGTON	SITE & EXPLORATION PLAN	DRAWN BY: CF	FIGURE NO.: 2B
		CHECK BY: JG	PROJECT NO.: 2019-173-21

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EXPLORATION LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROX. LOCATION (CURRENT STUDY)

DH-1-933063 BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 25, 2019 FOR CULVERT 933063

DH-1-933064 BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 26, 2019 FOR CULVERT 933064

A A' GEOLOGIC CROSS SECTION

BASE MAP PROVIDED BY: BING AND SURVEYOR

LANGLOIS CREEK CULVERT 933062

Scale: 1" = 40'-0"

N

0 20 40 60 80

SCALE: 1" = 40'

HWA GEOSCIENCES INC. DBE/MWBE

LANGLOIS CREEK CULVERT REPLACEMENT
CARNATION, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2C
CHECK BY:	PROJECT NO.:
JG	2019-173-21

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EXPLORATION LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROX. LOCATION (CURRENT STUDY)

DH-1-933063 BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 25, 2019 FOR CULVERT 933063

DH-1-933064 BOREHOLE DESIGNATION AND APPROX. LOCATION USDA JUNE 26, 2019 FOR CULVERT 933064

A A' GEOLOGIC CROSS SECTION

BASE MAP PROVIDED BY: BING AND SURVEYOR

LANGLOIS CREEK CULVERT 933064

Scale: 1" = 40'-0"

N

0 20 40 60 80

SCALE: 1" = 40'

HWA GEOSCIENCES INC. DBE/MWBE

LANGLOIS CREEK CULVERT REPLACEMENT
CARNATION, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY: CF	FIGURE NO.: 2D
CHECK BY: JG	PROJECT NO.: 2019-173-21

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APPENDIX A

BOREHOLE LOG

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 More than 50% Retained on No. 200 Sieve Size	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
	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
PID	Photoionization Device Reading
PP	Pocket Penetrometer Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
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
- 3-1/4" OD Split Spoon with Brass Rings
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- Non-standard Penetration Test (3.0" 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.

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CARNATION, WASHINGTON

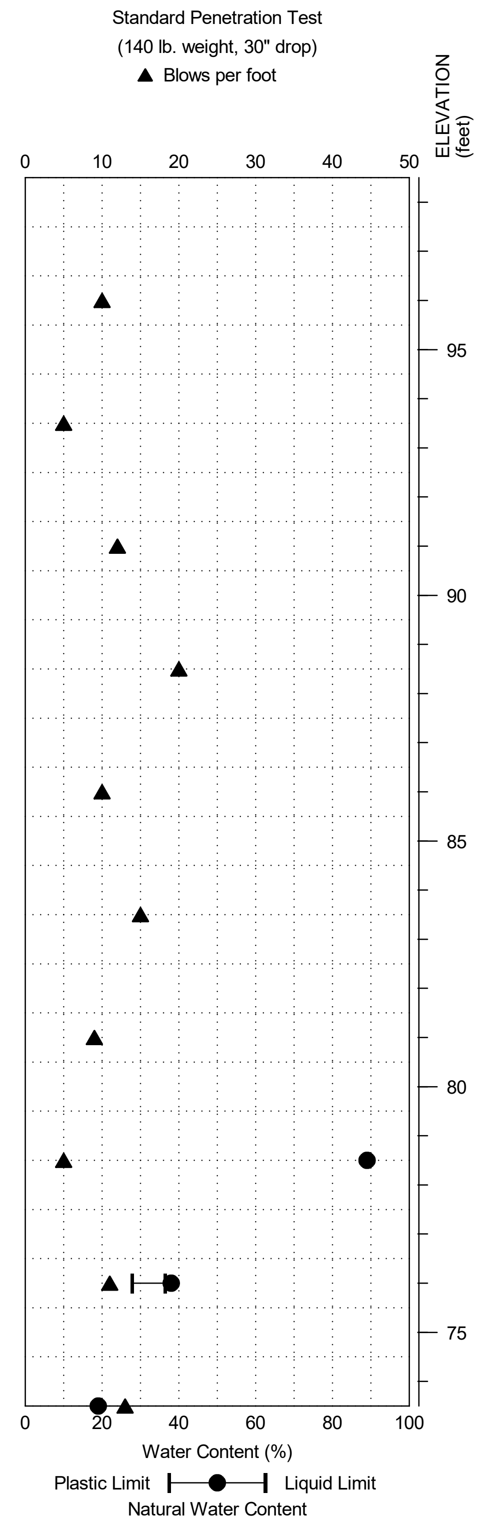
LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2019-173-21 FIGURE: A-1

DRILLING COMPANY: Gregory Drilling Inc.
DRILLING METHOD: CME-55LCX, 4.25" ID HSA
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/31/2020
DATE COMPLETED: 3/31/2020
LOGGED BY: Z. Ngoma
SURFACE ELEVATION: 98.5 ± feet

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	SP SM	Medium dense, olive-brown, slightly silty, SAND, moist, with ecomposed wood fragments. **(FILL)**				
			S-1	2-4-6		
5	SM	Loose to medium dense, olive-brown, silty, gravelly to very gravelly, SAND, moist, with decomposed wood fragments.	S-2	5-2-3		
		No recovery. Gravelly drilling around 8 feet and abundant gravel in cuttings. Gravel is sub-rounded.	S-3	3-6-6		
10		Blow counts possibly overstated due to gravel particles.	S-4	5-13-7		
	SP SM	Medium dense, dark yellow-brown, gravelly, slightly silty to silty, SAND, moist.	S-5	3-4-6		
15		Becomes olive-brown.	S-6	7-7-8		
		Gravelly drilling around 17 feet and abundant gravel in cuttings. Gravel is sub-rounded. Becomes loose and dark olive-brown.	S-7	4-5-4		
20	SM	Loose, olive-brown, silty SAND interbedded with sandy PEAT, moist. **(ALLUVIUM)**	S-8	2-2-3		∇
	ML	Stiff, light yellow-brown, SILT, wet. Interbedded with fine sandy silt and poorly graded sand. Minor organics.	S-9	1-2-9	AL	
25						



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.

LANGLOIS CREEK CULVERT REPLACEMENT
CARNATION, WASHINGTON

BORING:
BH-1

PAGE: 1 of 2

PROJECT NO.: 2019-173-21 FIGURE: A-2

DRILLING COMPANY: Gregory Drilling Inc.
DRILLING METHOD: CME-55LCX, 4.25" ID HSA
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/31/2020
DATE COMPLETED: 3/31/2020
LOGGED BY: Z. Ngoma
SURFACE ELEVATION: 98.5 ± feet

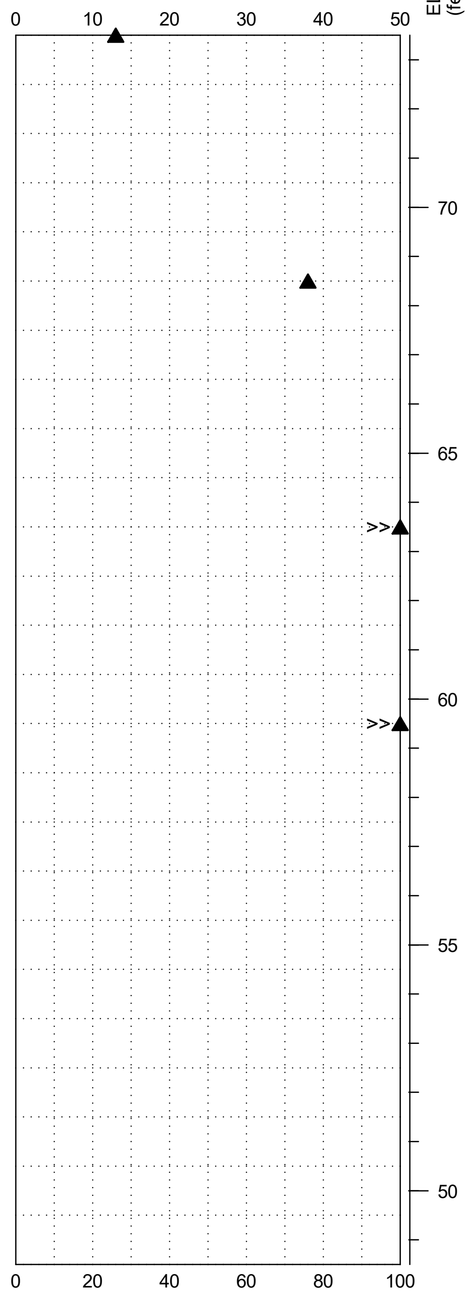
DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
25		SM	Medium dense, dark gray to yellow-brown, gravelly to very gravelly, silty SAND, wet.		S-10	8-7-6	GS	
			Lenses of soil with abundant organics observed in sample at 25 feet.					
30			Becomes dense.		S-11	10-20-18		
			Gravelly drilling below 32 feet, with abundant gravel in cuttings. Gravel is sub-rounded.					
35			Becomes very dense and dark gray.		S-12	21-25-28		
			Becomes light yellow-brown.		S-13	27-45-37		
40								

Borehole terminated at 40.5 feet below ground surface (bgs) due to refusal.
Ground water seepage encountered at 20 feet bgs during exploration.

45

50

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



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.

LANGLOIS CREEK CULVERT REPLACEMENT
CARNATION, WASHINGTON

BORING:
BH-1

PAGE: 2 of 2

PROJECT NO.: 2019-173-21 FIGURE: A-2

APPENDIX B

LABORATORY TESTING

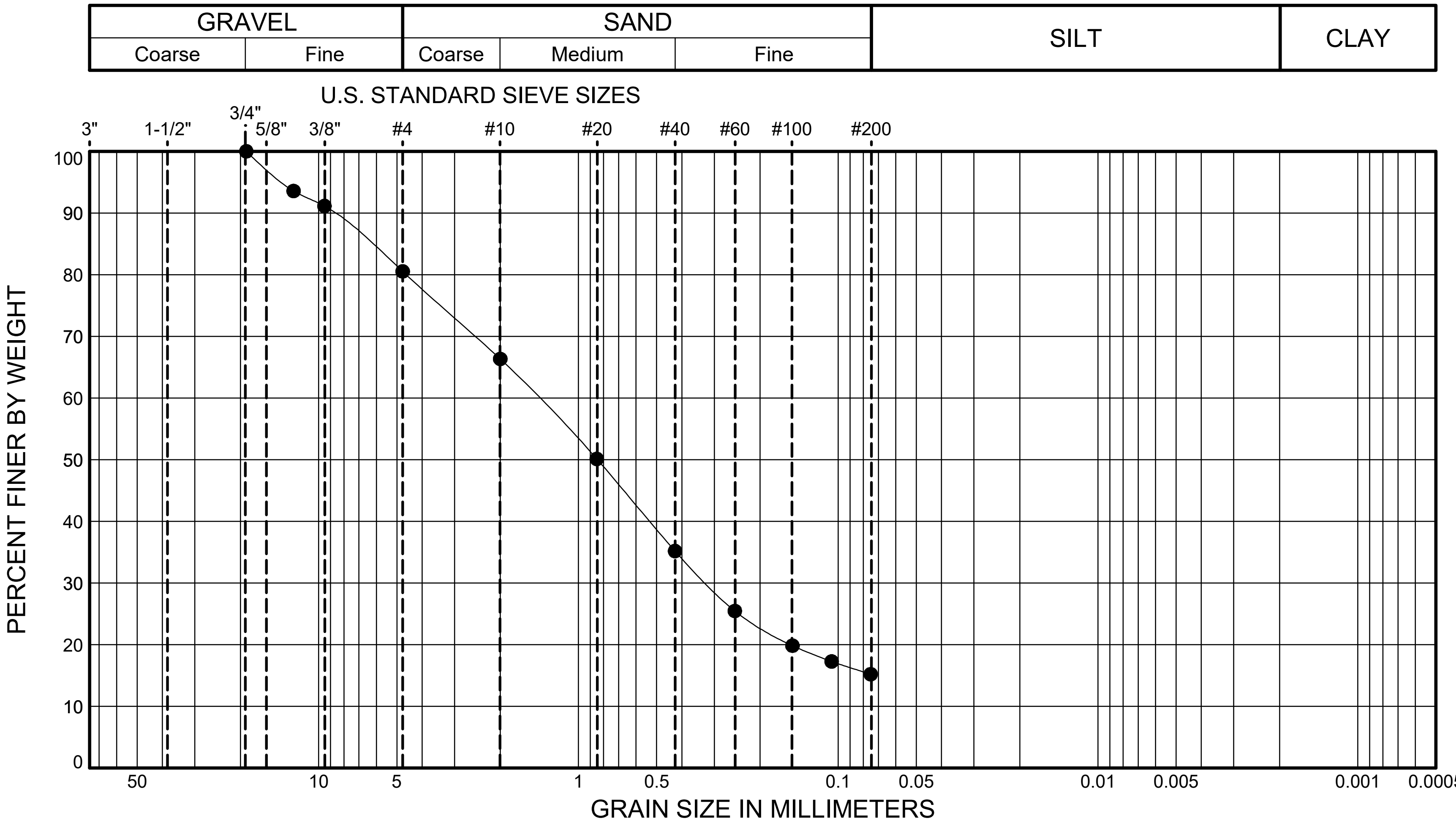
Laboratory Testing

Representative soil samples obtained from the exploration were placed in plastic bags to prevent loss of moisture and transported to our Bothell, Washington laboratory for further examination and testing. Laboratory tests were conducted on selected soil samples to characterize relevant engineering and index properties of the site soils. The laboratory testing program was performed in general accordance with appropriate ASTM Standards, as outlined below.

Moisture Content of Soil: The moisture content of selected soil samples (percent by dry mass) was determined in general accordance with ASTM D 2216. The results are shown at the sampled intervals on the appropriate summary logs in Appendix A.

Particle Size Analysis of Soils: Selected granular samples were tested to determine the particle size distribution of material in accordance with ASTM D 6913 (wash sieve or wash sieve and hydrometer methods). The results are summarized on the attached Particle-Size Distribution report (Figure B-1), which also provide information regarding the classification of the samples and the moisture content at the time of testing.

Liquid Limit, Plastic Limit, and Plasticity Index of Soils (Atterberg Limits): One selected sample was tested using method ASTM D 4318, multi-point method. The results are reported on the attached Liquid Limit, Plastic Limit, and Plasticity Index reports found on Figure B-2.



SYMBOL	SAMPLE		DEPTH (ft.)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-1	S-10	25.0 - 26.5	(SM) Olive-brown, silty SAND with gravel	19				19.5	65.3	15.2

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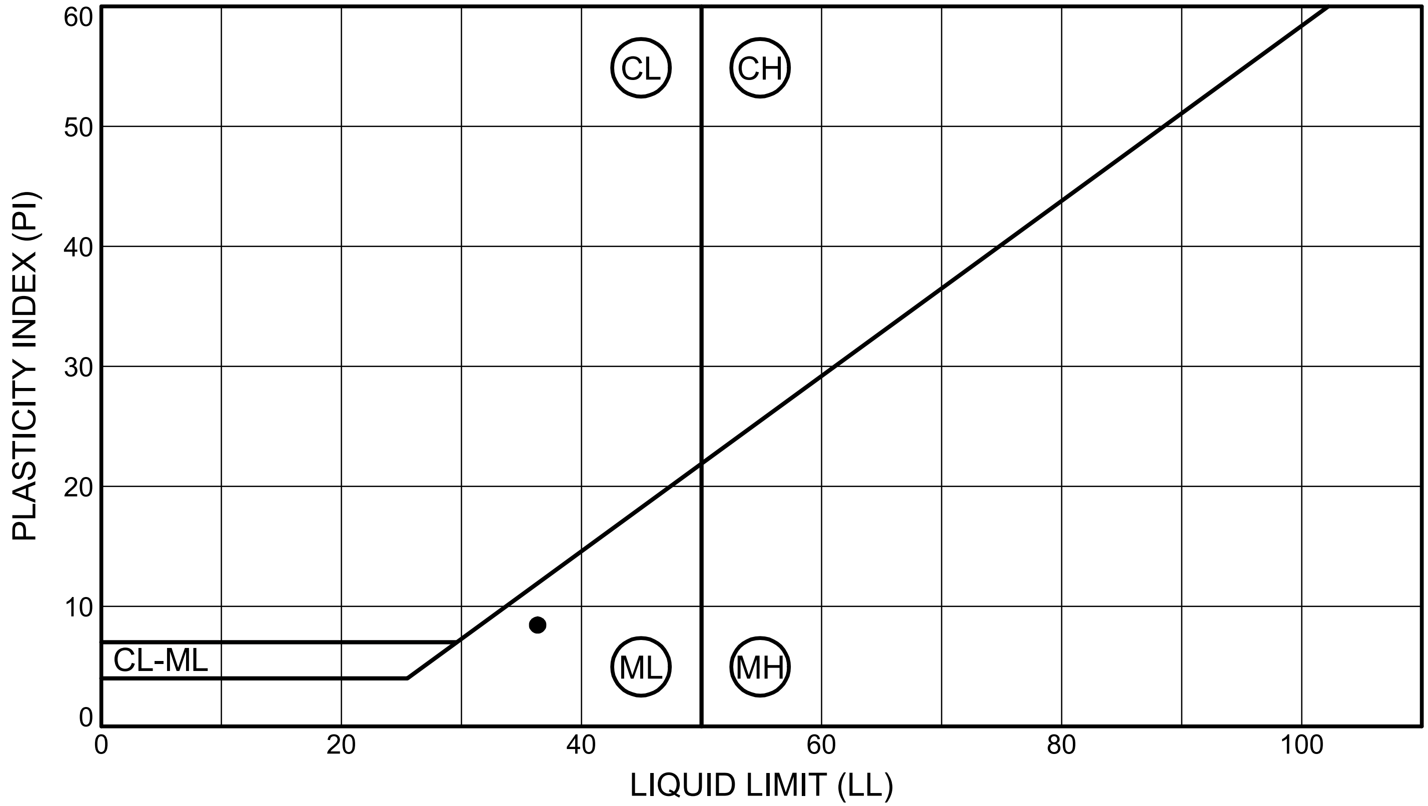
REPLACEMENT
CARNATION, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D6913

PROJECT NO.: 2019-173-21 FIGURE: B-1



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-1	S-9	22.5 - 24.0	(ML) Dark gray, sandy SILT	38	36	28	8	

HWA GEOSCIENCES INC.

LANGLOIS CREEK CULVERT
REPLACEMENT
CARNATION, WASHINGTON

LIQUID LIMIT, PLASTIC LIMIT AND
PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

PROJECT NO.: 2019-173-21 FIGURE: B-2

APPENDIX C

USDA LETTER REPORTS FOR CULVERTS #933063 AND #933064

USDA

United States
Department of
Agriculture

Natural
Resources
Conservation
Service

316 W. Boone Ave, Suite 450
Spokane, WA 99201-2348

Phone 509-323-2900
Fax 509-323-2979

June 25, 2019

SUBJECT: ENG – Trip Report
Remlinger Farms Bridge – Site 1
King County, WA

File Code: 210-0

TO: Larry Johnson
State Conservation Engineer
Spokane, WA

PURPOSE

On June 19, 2019, we visited the Remlinger Farms site to explore geotechnical foundation conditions for removal of a culvert and replacement with a bridge. The bridge will improve conditions for aquatic organism passage along Langlois Creek. The site is located south of NE 32nd St. on the east side of an abandoned railroad grade (Snoqualmie Valley Trail) where Langlois Creek crosses under a paved farm lane (Figure 1).

PARTICIPANTS

Joe Gasperi, State Geologist, Spokane, WA
Wes Durheim, Civil Engineering Technician, Spokane, WA
Jim Farley, Civil Engineering Technician, Olympia, WA
Baylee McGinnis, Pathways Intern – Geology, Spokane, WA
Dillon Sprague, Pathways Intern – Engineering, Spokane, WA

DATA COLLECTION

We drilled two exploratory holes, DH1 and DH2. The general location of the drill holes is shown in Figure 2. Figures 3 and 4 show the drill rig set up at DH1 and DH2 respectively.

Figure 1. View looking west of existing crossing.

Figure 2. Aerial image of Remlinger Farms site 1 with the approximate location of the exploratory drill holes. North is towards top of page.

Figure 3. View looking east of drill rig set up at DH1.

Figure 4. View looking northwest of drill rig set up at DH2.

Drill Hole 1

SPT	Number of Blows
3.5 to 4.0 feet	4
4.0 to 4.5 feet	2
4.5 to 5.0 feet	1
Blows per foot, N =	3

3.5 to 5.0 feet: SM/SP, silty sand to poorly-graded sand, saturated, free water, oxidized, red-brown, very loose relative density (Figure 5). Sample collected for laboratory classification.

Figure 5. View of sample recovered from a depth of 3.5 to 5.0 feet.

SPT	Number of Blows
6.5 to 7.0 feet	1
7.0 to 7.5 feet	1
7.5 to 8.0 feet	1
Blows per foot, N =	2

6.5 to 8.0 feet: SP/SM, poorly-graded sand to silty sand, saturated, free water, oxidized, red-brown, very loose relative density to 9.2 feet, ML, silt, saturated, reduced, blue-gray, very soft consistency below 9.2 feet (Figure 6). Sample collected for laboratory classification.

Figure 6. View of sample recovered from a depth of 6.5 to 8.0 feet. Note: The sample bags are mislabeled in the photograph. They should read 6.5 to 7.2 feet and 7.2 to 8.0 feet. The error was identified in the field and corrected after the photo was taken.

SPT	Number of Blows
8.5 to 9.0 feet	sank
9.0 to 9.5 feet	0
9.5 to 10.0 feet	1
Blows per foot, N =	1

8.5 to 10.0 feet: ML/SM, silt to silty sand with organic matter, saturated, free water, reduced, blue-gray, very soft consistency (Figure 7). Sample collected for laboratory classification.

Figure 7. View of sample recovered from a depth of 8.5 to 10.0 feet.

SPT	Number of Blows
11.5 to 12.0 feet	1
12.0 to 12.5 feet	1
12.5 to 13.0 feet	1
Blows per foot, N =	2

11.5 to 13.0 feet: ML/SM, silt to silty sand with organic matter, saturated, free water, reduced, blue-gray, very loose relative density (Figure 8). Sample collected for laboratory classification.

Figure 8. View of sample recovered from a depth of 11.5 to 13.0 feet.

SPT	Number of Blows
13.5 to 14.0 feet	sank
14.0 to 14.5 feet	2
14.5 to 15.0 feet	2
Blows per foot, N =	4

13.5 to 15.0 feet: ML, silt, saturated, free water, reduced, blue-gray, soft consistency (Figure 9). Sample collected for laboratory classification.

Figure 9. View of sample recovered from a depth of 13.5 to 15.0 feet.

SPT	Number of Blows
16.5 to 17.0 feet	sank
17.0 to 17.5 feet	2
17.5 to 18.0 feet	1
Blows per foot, N =	3

16.5 to 18.0 feet: ML/SM, silt to silty sand, saturated, free water, reduced, blue-gray, very loose relative density (Figure 10). Sample collected for laboratory classification.

Figure 10. View of sample recovered from a depth of 16.5 to 18.0 feet.

*Stopped drilling 18.0 feet due to 21-inches of soil heave following the Standard Penetration Test. We measured standing water at a depth of 4.0 feet below ground surface once the hollow-stem auger was removed.

Drill Hole 2

SPT	Number of Blows
3.5 to 4.0 feet	4
4.0 to 4.5 feet	4
4.5 to 5.0 feet	5
Blows per foot, N =	9

3.5 to 5.0 feet: SM/ML, silty sand to silt with sand, saturated, free water, reduced, blue-gray, loose relative density (Figure 11). Sample collected for laboratory classification.

Figure 11. View of sample recovered from a depth of 3.5 to 5.0 feet.

SPT	Number of Blows
6.5 to 7.0 feet	2
7.0 to 7.5 feet	2
7.5 to 8.0 feet	1
Blows per foot, N =	3

6.5 to 8.0 feet: SM/ML, silty sand to silt with organic matter, saturated, free water, reduced, blue-gray, very loose relative density (Figure 12). Sample collected for laboratory classification.

Figure 12. View of sample recovered from a depth of 6.5 to 8.0 feet

SPT	Number of Blows
8.5 to 9.0 feet	sank
9.0 to 9.5 feet	3
9.5 to 10.0 feet	2
Blows per foot, N =	5

8.5 to 10.0 feet: SM/ML, silty sand to silt with sand, saturated, free water, reduced, blue-gray, loose relative density (Figure 13). Sample collected for laboratory classification.

Figure 13. View of sample recovered from a depth of 8.5 to 10.0 feet.

SPT	Number of Blows
11.5 to 12.0 feet	3
12.0 to 12.5 feet	2
12.5 to 13.0 feet	1
Blows per foot, N =	3

11.5 to 13.0 feet: ML/SM, silt with sand to silty sand, saturated, free water, reduced, blue-gray, very loose relative density (Figure 14). Sample collected for laboratory classification.

Figure 14. View of sample recovered from a depth of 11.5 to 13.0 feet.

SPT	Number of Blows
13.5 to 14.0 feet	4
14.0 to 14.5 feet	3
14.5 to 15.0 feet	9
Blows per foot, N =	12

13.5 to 15.0 feet: Wood, we drilled through an old log buried in the floodplain (Figure 15). Sample collected for laboratory classification.

Figure 15. View of sample recovered from a depth of 13.5 to 15.0 feet.

*Stopped drilling at a depth of 16.5 feet due to 10-inches of soil heave inside the hollow-stem auger prior to the Standard Penetration Test. We measured standing water at a depth of 4.0 feet below ground surface once the hollow-stem auger was removed.

FINDINGS

1. Seasonal High Water Table

Langlois Creek has been channelized and the surrounding farm ground modified to benefit the farming operation. It is possible that the soil in the upper 7.2 feet of DH1 was placed to fill a low spot in the field and has yet to develop redoximorphic features. Even though the soils are saturated, contain free water, and lie below the water level in the channel they don’t exhibit reduced conditions until a depth of 7.2 feet. The soils in DH2 are fully reduced at a depth of 3.5 feet and may better represent native soil conditions given their location across the channel away from the farm field. We did not sample the interval from the ground surface to a depth of 3.5 feet. The seasonal high water table occurs somewhere within this depth interval. For design purposes, assume a seasonal high water table of 1.0 feet below ground surface.

2. Bearing Capacity
The foundation soils consist of up to 18.0 feet of very loose to loose relative density poorly-graded sand and silty sand and very soft to soft consistency silt along with layers of organic matter and woody debris. For design purposes, these soils require improvement before they can support a load without failing. They lack the required consolidation needed to make a recommendation for presumptive bearing capacity.

3. Differential Settlement
The site has a high potential for differential settlement given the minimal consolidation of the in-place soils.

4. Liquefaction Potential
According to the USGS NSHM 2014 Dynamic edition, the site has a 10% probability in 50 years of experiencing a peak ground acceleration (pga) of 0.38g assuming an average shear-wave velocity in the top 30 meters of 180 meters/second, NEHRP Site Class D/E.

This site has a high potential for failure due to liquefaction because of the poorly-consolidated, saturated, sands, silty sands and silts to a depth of at least 18.0 feet in the foundation. Seismic loading would likely cause the foundation soils to lose shear strength and flow toward the stream channel leading to a slumping or rotational failure of the bridge abutments and a collapse of the bridge. As a routine part of Operations & Maintenance activities, the bridge should be inspected following large earthquakes.

5. Cultural Resources
We did not observe cultural resource remains in the auger cuttings or split-barrel samples.

JOE GASPERI
Geologist

cc: Amy Hendershot, District Conservationist, Puyallup, WA
Rob Ryan, Agronomist, Renton, WA
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June 26, 2019

SUBJECT: ENG – Trip Report
Remlinger Farms Bridge – Site 2
King County, WA

File Code: 210-0

TO: Larry Johnson
State Conservation Engineer
Spokane, WA

PURPOSE

On June 20, 2019, we visited the Remlinger Farms site to explore geotechnical foundation conditions for removal of a culvert and replacement with a bridge (Figure 1). The bridge will improve conditions for aquatic organism passage along Langlois Creek. The site is located on an asphalt paved farm road north of NE 24th Street on the southeast corner of the property.

PARTICIPANTS

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DATA COLLECTION

We drilled two exploratory holes, DH1 and DH2. The general location of the drill holes is shown in Figure 2. Figures 3 and 4 show the drill rig set up at DH1 and DH2 respectively.


Figure 1. View looking east of existing crossing.

Figure 2. Aerial image of Remlinger Farms site 2 with the approximate location of the exploratory drill holes. North is towards top of page.

Figure 3. View looking north of drill rig set up at DH1.

Figure 4. View looking northwest of drill rig set up at DH2.

Drill Hole 1
This drill hole is located on a slope that required us to attempt angle drilling. The site is also situated about 10 to 12 feet above the channel bottom. Providing for a stable drill hole required drilling to a deeper depth than usual before performing the first Standard Penetration Test.

SPT	Number of Blows
6.5 to 7.0 feet	3
7.0 to 7.5 feet	3
7.5 to 8.0 feet	6
Blows per foot, N =	9

6.5 to 8.0 feet: ML/GM, silt with gravel to silty gravel, oxidized, red-brown, moist, loose relative density, a piece of gravel plugged the shoe during the Standard Penetration Test (Figure 5). Sample collected for laboratory classification.

Figure 5. View of sample recovered from a depth of 6.5 to 8.0 feet.

SPT	Number of Blows
8.5 to 9.0 feet	2
9.0 to 9.5 feet	3
9.5 to 10.0 feet	5
Blows per foot, N =	8

8.5 to 10.0 feet: GM, silty gravel with tree root, moist, oxidized, red-brown, loose relative density, a piece of gravel plugged the shoe during the Standard Penetration Test (Figure 6). Sample collected for laboratory classification.

Figure 6. View of sample recovered from a depth of 8.5 to 10.0 feet.

SPT	Number of Blows
11.5 to 12.0 feet	8
12.0 to 12.5 feet	17
12.5 to 13.0 feet	5+
Blows per foot, N =	22+

11.5 to 13.0 feet: GP, poorly-graded gravel with silt, saturated, free water, oxidized, red-brown, medium relative density (Figure 7). Sample collected for laboratory classification.

Figure 7. View of sample recovered from a depth of 11.5 to 13.0 feet.

*We stopped the Standard Penetration Test because the orientation of the drill hole was too far out of plumb to effectively drive the sampler. Due to time limitations and the generally positive results, we elected not to try drilling to a great depth at this location.

Drill Hole 2

SPT	Number of Blows
3.5 to 4.0 feet	2
4.0 to 4.5 feet	9
4.5 to 5.0 feet	21
Blows per foot, N =	30

3.5 to 5.0 feet: GM/ML, silty gravel to silt with gravel, moist, oxidized, red-brown, medium relative density (Figure 8). Sample collected for laboratory classification.

Figure 8. View of sample recovered from a depth of 3.5 to 5.0 feet.

*After the first Standard Penetration Test, gravel in the foundation kicked the hole out of plumb. We backed the truck up 3 feet and started a new hole.

SPT	Number of Blows
6.5 to 7.0 feet	4
7.0 to 7.5 feet	7
7.5 to 8.0 feet	3
Blows per foot, N =	10

6.5 to 8.0 feet: GM/ML, silty gravel to silt with gravel, moist, redoximorphic soil features, red-brown/blue-gray, loose relative density (Figure 9). Sample collected for laboratory classification.

Figure 9. View of sample recovered from a depth of 6.5 to 8.0 feet

SPT	Number of Blows
8.5 to 9.0 feet	3
9.0 to 9.5 feet	5
9.5 to 10.0 feet	5
Blows per foot, N =	10

8.5 to 10.0 feet: ML/SM, silt to silty sand, saturated, free water, reduced, blue-gray, loose relative density to 9.3 feet, SM, silty sand, saturated, free water, oxidized, red-brown, loose relative density below 9.3 feet (Figure 10). Sample collected for laboratory classification.

Figure 10. View of sample recovered from a depth of 8.5 to 10.0 feet.

SPT	Number of Blows
11.5 to 12.0 feet	2
12.0 to 12.5 feet	6
12.5 to 13.0 feet	7
Blows per foot, N =	13

11.5 to 13.0 feet: SM/SP, silty sand to poorly-graded sand, saturated, free water, oxidized, red-brown, loose to medium relative density to 12.1 feet, ML, silt with gravel and sand, saturated, reduced, blue-gray, medium relative density below 12.1 feet (Figure 11). Sample collected for laboratory classification.

Figure 11. View of sample recovered from a depth of 11.5 to 13.0 feet.

SPT	Number of Blows
13.5 to 14.0 feet	6
14.0 to 14.5 feet	7
14.5 to 15.0 feet	21
Blows per foot, N =	28

13.5 to 15.0 feet: ML/GM, plastic silt with gravel to silty gravel, saturated, reduced, blue-gray, very stiff consistency (Figure 12). Sample collected for laboratory classification.

Figure 12. View of sample recovered from a depth of 13.5 to 15.0 feet.

*Stopped drilling at a depth of 15.0 feet due to 18-inches of soil heave inside the hollow-stem auger after the Standard Penetration Test. The hole collapsed as we removed the hollow-stem auger.

FINDINGS

1. Seasonal High Water Table

We observed moist, oxidized, silty/gravelly soils to a depth of 10.0 feet in DH1 and saturated, oxidized, silty/gravelly soils with free water at depths of 11.5 to 13.0 feet. These soils have a fairly high permeability and freely drain toward the creek. Given our location, on the hillslope above the channel, we may not have drilled to a depth below the bottom of the channel. The soil redoximorphic indicators suggest that water percolates through the soils profile to a depth of at least 13.0 feet and drains towards Langlois Creek. Due to the free drainage of the soils, redoximorphic soils features used to determine seasonal high water table were either weakly developed or absent in the upper 13.0 feet of the soil profile. For design purposes, assume a seasonal high water table of 10.0 feet below the ground surface at DH1.

At DH2, we observed oxidized soils to a depth of 5.0 feet, redoximorphic soil features from 6.5 to 8.0 feet, reduced soils from 8.5 to 9.3 feet, oxidized soils from 9.3 to 12.1 feet and reduced soils from 12.1 to 15.0 feet. The redoximorphic soil features indicate a seasonal high water table at a depth of 5.0 to 6.5 feet, saturated conditions below 8.0 feet. The zone of oxidized soils from 9.3 to 12.1 feet consists of relatively free draining silty sand and poorly-graded sands. Oxygenated ground water flows through this zone at a high enough rate to keep the mineral soils in an oxidized state, whereas the finer grained soils from 8.5 to 9.3 feet have a lower permeability and have developed reduced conditions. Below 12.1 feet, ground water is either ponded or flows slowly through the silty/gravelly soils allowing reduced conditions to exist. For design purposes, assume a seasonal high water table of 5.0 feet below the ground surface at DH2.

2. Bearing Capacity

At DH1, the foundation soils consist of at least 10.0 feet of loose relative density silty gravels over medium relative density poorly-graded gravels at a depth of 11.5 feet. For design purposes, assume a presumptive bearing capacity value of 0.50 tons per square foot.

At DH2, the foundation soils consist of loose to medium relative density silts, silty sands and silty gravels to a depth of at least 13.0 feet over very stiff consistency silts from 13.5 to 15.0 feet. For design purposes, assume a presumptive bearing capacity value of 0.75 tons per square foot.

3. Differential Settlement

The site has a low to moderate potential for differential settlement given the loose to medium relative density of the foundation soils. Consider leaving the bridge approaches unpaved for a year to allow for consolidation before bringing to final grade and applying an asphalt surface.

4. Liquefaction Potential

According to the USGS NSHM 2014 Dynamic edition, the site has a 10% probability in 50 years of experiencing a peak ground acceleration (pga) of 0.37g assuming an average shear-wave velocity in the top 30 meters of 180 meters/second, NEHRP Site Class D/E.

The site has a low to moderate potential for failure due to liquefaction due to the high fines content and loose to medium relative density. Seismic loading would likely cause slumping or rotational failure of the bridge abutments. As a routine part of Operations & Maintenance activities, the bridge should be inspected following large earthquakes.

5. Cultural Resources

We did not observe cultural resource remains in the auger cuttings or split-barrel samples.

JOE GASPERI
Geologist

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