GEOTECHNICAL REPORT
SALMON CREEK CULVERT REPLACEMENT

West Uncas Road
Port Townsend, Washington

Prepared for: Shearer Design, LLC and
Jefferson County Public Works

Project No. 140256-01 • June 5, 2015 FINAL

Aspect
CONSULTING

earth + water

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1 Introduction

This report presents the results of a geotechnical engineering study performed by Aspect Consulting, LLC (Aspect) for the Salmon Creek Culvert Replacement Project (Project) located southwest of Discovery Bay on West Uncas Road in Jefferson County, Washington (Site). Our services were provided in support of an engineering study and design being performed by Shearer Design, LLC (Shearer) for the Jefferson County Public Works Department (County).

The Project location is the intersection of West Uncas Road and Salmon Creek as shown on Figure 1, *Site Location Map*. The purpose of the Project is to replace the existing culvert to enhance fish passage at the crossing, reduce flooding, and enhance in-stream and floodplain habitat in the vicinity of the passage.

This report summarizes the results of the completed field explorations and presents Aspect’s geotechnical engineering conclusions and recommendations.

1.1 Scope of Services and Authorization

Our scope of work included gathering and reviewing existing subsurface information near the Site; drilling and sampling exploratory borings; performing laboratory testing; completing engineering analyses to develop geotechnical conclusions and recommendations for design and considerations for construction of the Project; and preparing this report. Our work was completed in general accordance with our subconsultant agreement with Shearer Design, LLC, dated November 1, 2014.

1.2 Project Description

The existing culvert at the intersection of Salmon Creek and West Uncas Road is a 114-inch-diameter corrugated metal pipe arch culvert. The existing culvert under-crosses West Uncas Road in a west-southwest to east-northeast direction. West Uncas Road is a paved, two-lane, rural residential road, approximately 20-feet wide. Significant corrosion has been noted throughout the existing culvert during inspections, necessitating this Project.

We understand the Washington Department of Fish and Wildlife (WDFW, 2013) recommended the replacement culvert accommodate a minimum reconstructed channel width of 24 feet. The layout and location of the existing culvert, topography, and other existing Site features are shown on Figure 2, *Site and Exploration Plan*.

A short bridge is planned to replace the existing culvert. The short bridge concept includes a 28-foot-wide, 80-foot-long, single-span bridge with a reconstructed channel width of 30 feet. The short bridge concept includes provisions for 18-inch-diameter steel piles for foundation support. Stem walls at the abutments, approximately 7 feet tall, are planned from the pile caps to the bridge deck. We understand the bridge abutments are designed with a hinge connection to the superstructure that will restrain lateral movements. Reconstructed creek channel slopes with an inclination of 2H:1V (horizontal:vertical) are proposed. We assume the bridge will be designed for a HL-93 vehicular live load.

It is our assumption that the proposed construction will occur during the dry weather season and during the allowable 'in-water' work window as determined by the applicable governing agencies.

For the purposes of this study, we assume that design and construction of the improvements will be in accordance with American Association of State Highway and Transportation Officials Bridge Design Specifications (BDS) (AASHTO, 2012) and/or Washington State Department of Transportation (WSDOT) Geotechnical Design Manual (GDM) (WSDOT, 2014a). The Project vertical datum is NAVD 88 and the basis for all references to elevations contained herein.

2 Site Conditions

2.1 General Geology

The Project area is located in the Puget Lowland. The Puget Lowland is a complex tectonic environment and an area of tectonic subsidence flanked by two mountain ranges—the Cascades to the east, and the Olympics to the west. The sediments within the Puget Lowland result from repeated cycles of glacial and non-glacial deposition and erosion. During non-glacial cycles, the area was dominated by lowland forests and broad river valleys. During glacial cycles, ice sheets up to 3,000 feet thick occupied the Puget Lowland and surrounding areas and carved out the deep marine waterways and river valleys, and sculpted the uplands. Deposits from these glacial and nonglacial cycles are present in the subsurface of the Project area.

The available geologic mapping (Tabor et. al., 2011) indicates that subsurface conditions at the Site generally consist of road embankment fill, overlying recent (Holocene) alluvium deposits, overlying glacial soils from the Fraser glaciation age.

2.2 Seismicity

The Site is located in a seismically active area and is prone to seismic hazards such as liquefaction and amplified seismic response. The Site lies approximately 12 miles southwest of the Southern Whidbey Island Fault (SWIF) zone, a shallow crustal tectonic structure that is considered active (meaning it has the potential to cause earthquakes in the future). The recurrence interval of earthquakes on this fault zone is believed to be on the order of a thousand years or more.

The Site also lies within the zone of strong shaking from subduction zone earthquakes. The recurrence interval of these earthquakes is thought to be on the order of about 500 years. The most recent subduction zone earthquake occurred in 1700.

Deep intra-slab earthquakes also occur in the region every decade or two, including the 2001 Nisqually earthquake. These earthquakes are generally less severe than the shallow crustal and subduction zone earthquakes but have the potential to cause damage to older structures built before modern seismic codes were enacted, and those in liquefaction-sensitive areas.

2.3 Surface

The Site is located at the base of the eastern foothills of the Olympic Mountain range, approximately ½ mile southwest of the intersection between Highway 101 and State Route 20. The Site has moderately to steeply sloping topography toward Salmon Creek with the West Uncas Road fill embankment transecting the central portion of the Site in a north-to-south orientation. Elevations across the Site range from a high of 58 along the road surface of West Uncas Road to a low of 34 at the thalweg of Salmon Creek immediately downstream of the existing culvert. The West Uncas Road fill embankment is 15 to 20 feet tall at the existing culvert crossing location.

Salmon Creek flows toward the northeast and consists primarily of an open creek channel approximately 15 feet wide. The creek channel has steep side slopes that include riprap armoring near the existing culvert crossing. Salmon Creek flows underneath West Uncas Road through a deteriorating 114-inch-diameter corrugated metal pipe arch culvert. Topography of the Site and approximate layout of the existing stream path is illustrated on Figure 2.

2.4 Subsurface Conditions

Subsurface conditions at the project Site were inferred from the completed field explorations, review of applicable geologic literature, and our experience with the local geology. More detailed soils descriptions are presented on boring logs in Appendix A.

The following section presents more detailed subsurface information organized from the upper to the lower soil types.

2.4.1 Stratigraphy

The subsurface soils, based on the completed subsurface explorations, can be grouped into four units consisting of the following: fill, recent alluvium deposits, glacial recessional outwash, and glacial advance outwash. The geologic units encountered are generally consistent with available geologic mapping described in Section 2.1. Soil borings B-1 and B-2 were advanced near the existing culvert crossing, at the approximate locations shown on Figure 2. Selected soil samples collected from the borings were submitted to a soil testing lab to determine the selected properties of the soil samples including moisture content and grain size analysis. The results of the geotechnical laboratory testing are shown in Appendix B.

Details of the composition and distribution of these units are presented in more detail below.

Fill

We encountered roadway embankment fill at the ground surface in both borings. The fill thickness varied from 10 to 18 feet It generally consisted of loose to dense, moist, brown, clean to silty, gravelly SAND (SP, SM)[1], and scattered to trace organics.

The SPT[2] blow counts from the explorations in the fill ranged from 5 to 22 blows per foot, indicating the fill was typically medium dense. The presence of fine-grained soil (soil particles passing the No. 200 sieve) makes the road fill susceptible to disturbance during construction (it is moisture sensitive). Scattered fine organics were present throughout the fill. The majority of the fill can generally be expected to have moderate shear strength, moderate compressibility, and moderate permeability.

[1] Soil Classification per the Unified Soil Classification System (USCS). Refer to ASTM D-2488.
[2] SPT blow count refers to standard penetration test (SPT) N-values, in accordance with ASTM D-1586.

Recent Alluvium

Below roadway embankment fill, we encountered stream alluvium extending to a depth of 25 feet below ground surface (bgs) in both borings. The alluvium consisted of very loose to dense, moist to wet, brown, silty, gravelly SAND (SM) and sandy GRAVEL (GP, GM) with silt.

The SPT blow counts from the explorations in the alluvium ranged from 4 to 38 blows per foot with an average of 24 blows per foot, indicating the alluvium was typically medium dense with zones that were very loose to loose. The SPT data from boring B-2 was impacted (over-stated) by coarse gravel in the recent alluvium deposits and may not be representative of the relative density of the alluvium at that location.

The alluvium can be expected to have low to moderate shear strength, moderate compressibility, high permeability, and low to moderate moisture sensitivity. The alluvium is susceptible to liquefaction under the design seismic loading conditions.

Glacial Recessional Outwash

Below alluvium, we encountered glacial recessional outwash deposits in both borings to depths ranging from 43.5 to 50 feet bgs. The recessional outwash deposits consisted of dense to very dense, wet, brown, gravelly SAND (SW, SW-SM, SM) with silt. We encountered a layer of hard, wet, gray-brown, sandy SILT (ML) in boring B-2 from 41 to 43.5 feet bgs.

The SPT blow count from the explorations in the recessional outwash deposits ranged from 30 to 90 blows per foot, indicating the recessional outwash was dense to very dense. The recessional outwash can be expected to have moderate to high shear strength, low to moderate compressibility, high permeability, and moderate to high moisture sensitivity. Due to the relative density of the unit, the recessional outwash is not susceptible to liquefaction under the design seismic loading conditions.

Glacial Advance Outwash

We encountered glacial advance outwash underlying the recessional outwash in both borings. Both borings were terminated in the advance outwash unit at a depth of 51.5 feet bgs. The advance outwash consisted of very dense, wet, gray-brown, SAND (SP) with a gravel content of trace to gravelly.

The SPT blow counts from the explorations in the advance outwash ranged from 72 to 100 blows per foot with an average of 90 blows per foot, indicating the advance outwash was very dense. The advance outwash can be expected to have high shear strength, low compressibility, moderate to high permeability, and low moisture sensitivity. Due to the relative density of the unit, the advance outwash is not susceptible to liquefaction under the design seismic loading conditions.

2.4.2 Groundwater

We encountered groundwater in both soil borings at approximately Elevation 32. Groundwater levels may also fluctuate seasonally, with precipitation variations, and with changes in usage at and around the Site. Groundwater across the Site can be expected to generally follow the water levels within Salmon Creek and to slope gently toward the creek. For the purposes of geotechnical analyses and Project design, we recommend the design groundwater level be considered at Elevation 36, consistent with the upstream invert of the existing culvert structure.

2.4.3 Engineering Properties

The engineering properties of the subsurface soils were generalized for engineering analysis purposes. The generalized subsurface conditions at the Site are based on the limited subsurface information obtained from the completed explorations and our experience with the local geology.

An assessment of the geotechnical soil parameter for each soil unit identified is summarized in Table 1.

Table 1 – Soil Engineering Properties used for Analyses

Geologic Description	USCS Classification	SPT N-Value[1] (range & average)	Total Unit Weight (pcf)	Effective Friction Angle (degrees)
Fill	SM, SP	Range: 5-22, Average: 16	120	32
Recent Alluvium	SM, GM, GP	Range: 4-38, Average: 24	115	28-34[2]
Recessional Outwash	ML, SM, SP	Range: 30-90, Average: 55	120	35
Advance Outwash	SP	Range: 72-100, Average: 90	130	36

Notes:
1) Corrected for documented field and sampling procedures.
2) A range of effective friction angles was assumed to account for discrete loose zones within the alluvium.

2.5 Earthquake Engineering

2.5.1 Ground Motion

The AASHTO BDS response spectra for design are based on local seismicity and soil conditions. The seismicity is represented by the peak bedrock acceleration (PBA) based on established seismic risk models. The 7 percent probability of exceedance in 75-year design event (approximately 1,000-year recurrence interval) is being considered for this project. The PBA for the 1,000-year recurrence interval ground motion is 0.389g, based on United States Geologic Survey (USGS) National Seismic Hazard Map data (USGS, 2008).

The AASHTO BDS express the effects of site-specific subsurface conditions on the ground motion response in terms of the Site Factors, F_{pga}, F_a, and F_v for the Site. The Site Factors, which are determined by the Site Class, account for the seismic response of the soil profile and is based on the density and stiffness of the soil profile underlying the Site. The Site Class can be correlated to the average standard penetration resistance (NSPT) in the upper 100 feet of the soil profile. Based on our characterization of the subsurface conditions, AASHTO Site Class D should be assumed for the Site provided liquefaction mitigation as discussed in Section 3.3 is implemented into the Project design. The recommended ground motion parameters are shown below in Table 2.

Table 2 – Ground Motion Parameters

Design Parameter	Recommended Value
Site Class	D[1]
Peak Ground Acceleration (PGA)	0.389g (Site Class B)
Short Period Spectral Acceleration (S_s)	0.856g (Site Class B)
1-Second Period Spectral Acceleration (S_1)	0.315g (Site Class B)
Site Coefficient F_{pga}	1.11 (Site Class D)
Site Coefficient F_a	1.16 (Site Class D)
Site Coefficient F_v	1.77 (Site Class D)
Acceleration Coefficient (A_s)	0.432g (Site Class D)
Design Short Period Spectral Acceleration (SD_s)	0.991g (Site Class D)
Design 1-Second Period Spectral Acceleration (SD_1)	0.558g (Site Class D)

Notes:
1) Based on Table 3.4.2.1-1 of the AASHTO BDS.

2.6 Seismic Hazards

Earthquake-induced hazards that are relevant to the project Site include fault rupture, soil liquefaction and associated settlement, and lateral spreading. The following sections discuss these hazards.

Surficial Fault Rupture

The Site is located approximately 12 miles southwest of the SWIF. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of a thousand years. Due to the suspected long recurrence interval and offset of the Site from the known rupture location, the risk of surficial ground rupture is considered to be low during the expected life of the project.

Soil Liquefaction and Related Hazards

Liquefaction occurs when loose, saturated and relatively cohesionless soil deposits temporarily lose strength as a result of earthquake shaking. Potential effects of soil liquefaction include temporary loss of bearing capacity and lateral soil resistance, liquefaction-induced settlement, riverbank slope failure, and lateral spreading, any of which could result in structural damage. Primary factors controlling the development of liquefaction include intensity and duration of strong ground motion, characteristics of subsurface soil, in-situ stress conditions and the depth to groundwater.

Liquefaction evaluations were conducted with the aid of LiquefyPro, a seismically induced liquefaction and settlement analyses software program developed by CivilTech Software (2009) and WSliq, a liquefaction analysis software program that was created as part of an extended research project supported by the Washington State Department of Transportation and authored by Steve Kramer (2008). The evaluations are based on the data from the subsurface explorations for the Project.

We evaluated liquefaction potential based on the design event as summarized in Table 3. The design level event is based on the USGS National Seismic Hazard Map data to obtain the PBA and earthquake magnitude. The Peak Ground Acceleration (PGA) was determined using the methods suggested in Section 6.3.4 of the WSDOT GDM.

Table 3 – Design Level Earthquake Parameters

Seismic Event Return Period (years)	Peak Bedrock Acceleration (g)(1)	Peak Ground Acceleration (g)(2)	Earthquake Magnitude(1)	Mean Source-to-Site Distance (km)(1)
1,000	0.389	0.432	7.01	53.3

Notes:

1) Based on USGS Probabilistic Seismic Hazard Deaggregation.
2) Based on Section 6.3.4 of WSDOT GDM.

The liquefaction analyses performed indicate that liquefaction could occur at the Site for the design seismic event considered. The data from boring B-1 indicates that liquefaction is likely to occur within the recent alluvium deposits between Elevation 36 and 29. The data from boring B-2 does not indicate that liquefaction is likely to occur; however, SPT data collection in B-2 was impacted by coarse gravel in the recent alluvium deposits and may not be representative of the relative density of the alluvium at that location. Liquefaction requires soil saturation and we've assigned a design groundwater level at Elevation 36. A graphical output of the analysis using the data from soil boring B-1 is shown in Appendix C. The ground settlement indicated in Appendix C represents the lower bound of the calculated seismic settlement.

The primary consequences of liquefaction include temporary loss of soil shear strength, seismic-induced settlement, riverbank failure, lateral spreading, and sand boils. Temporary loss of soil shear strength will primarily impact the culvert replacement structure foundations and result in reduced vertical and lateral resistances for the extreme limit design state. Seismic-induced settlement will cause downdrag loads on the deep foundations (piles) and will likely distort the West Uncas Road surface. Seismically-induced settlements resulting from the design level earthquake considered were estimated using the Liquefy Pro and WSliq programs. The results of our total settlement analyses are summarized in Table 4.

The creek channel creates a free-face that may be subject to lateral spreading deformations during the design seismic event. Using procedures presented in the WSliq program, the estimated range of lateral spread at the Site is zero (0) to 2 inches.

Table 4 – Extents of Liquefaction and Estimates of Seismic-Induced Settlements

Seismic Event Return Period (years)	Estimated Elevation Range where Liquefaction is Likely (ft)	Estimated Post-Liquefaction Total Settlement[1] (inches)
1,000	36 to 29	2.5 to 12

Notes:

1) Settlement range estimated using the combined results from LiquefyPro and WSliq, settlement shown in Appendix B represents LiquefyPro results only.

Based on our understanding of the proposed culvert replacement and the depth of the anticipated liquefaction, deep foundations will be necessary for the short bridge concept to mitigate against the effects of liquefaction. The effects of the projected liquefaction will likely consist of total settlements as presented above and differential settlements up to half of the estimated total settlements.

2.6.1 Embankment Seismic Slope Stability

Due to the liquefaction hazard identified at the Site, the proposed short bridge configuration, and steepness of the existing embankment and creek channel slopes, a flow failure could develop following the design seismic event. To better understand the mechanism and impacts of a potential flow failure, we conducted a stability analysis using the computer model SLIDE (Rocscience, 2013). The residual shear strength of the liquefied soil unit (recent alluvium) was estimated using the methods suggested in the WSliq program (Kramer, 2008). Our screening-level analyses indicate that a global flow-failure of the embankment is likely for the post-earthquake condition. The potential flow-failure mass would involve the upper unsaturated nonliquefied soil (fill) behind and below the abutments. This failure will impose horizontal loads on the bridge abutments and pile supports. As the flow-failure moves past the piles towards the thalweg of the creek, there will be ground subsidence behind the abutments at the approaches. The flow-failure would be triggered by liquefaction of the alluvium soils at depth and appears relatively independent of the proposed 2H:1V inclination of the reconstructed creek channel slopes.

The short bridge is being designed such that the bridge abutments will have a hinge connection to the superstructure that will restrain lateral movements. Because both abutments will experience similar lateral loads due to soil deformation, the girders acting in compression can be utilized to stabilize each abutment wall. We conclude that the earthquake-related deformations and loads on the abutments can be resisted by an integral abutments and girders design approach.

The abutments should be designed for the at-rest and seismic earth pressures presented in Table 7.

3 Conclusions and Recommendations

3.1 General

In our opinion, the Project is feasible from a geotechnical perspective. The following sections present the results of our engineering analyses and recommendations. Applicable sections of the AASHTO BDS and WSDOT GDM were utilized in our evaluations and analyses.

The following recommendations are for earthwork, bridge foundation support, embankment stability, and other pertinent geotechnical design issues.

3.2 Earthwork

Based on the explorations performed on-Site and our understanding of the proposed Project, it is our opinion that basic excavation and grading can generally be completed with standard construction equipment.

Appropriate erosion control measures should be implemented prior to beginning earthwork activities in accordance with Jefferson County Best Management Practices (BMPs).

3.2.1 Temporary Excavation Stability and Permanent Slopes

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the Contractor. All temporary cuts in excess of 4 feet in height that are not protected by trench boxes or otherwise shored, should be sloped in accordance with Part N of Washington Administrative Code (WAC) 296-155 (WAC, 2009).

In general, soils across the Site classify as OSHA Soil Classification Type C. Temporary excavation cut slopes are anticipated to stand as steep as 1.5H:1V within the roadway fill and unsaturated recent alluvium deposits. The cut slope inclinations estimated above are applicable to excavations without groundwater seepage, or runoff, and assume dewatered conditions.

With time and the presence of seepage and/or precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. Therefore, all temporary slopes should be protected from erosion by installing a surface water diversion ditch or berm at the top of the slope. In addition, the Contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly. Vibrations created by traffic and construction equipment may cause caving and raveling of the temporary slopes. In such an event, lateral support for the temporary slopes should be provided by the Contractor to prevent loss of ground support.

Permanent slopes for the reconstructed creek channel should be no steeper than 1.5H:1V. The channel slopes should be covered with quarry spall or rip rap to armor and protect them from stream erosion.

3.2.2 Structural Fill

We estimate that portions of the material excavated for the Project may be suitable for reuse as structural fill. The soils within the roadway fill appear suitable for re-use as structural fill. Excavated material should be visually inspected by the Geotechnical Engineer to determine its potential use as structural fill.

In general, suitable structural fill material for the Project is fill placed within 3 percent of its optimum moisture content per the American Society for Testing and Materials (ASTM) ASTM D-1557 (modified Proctor test) and does not contain deleterious materials, greater than 5 percent organics, or particles larger than 3 inches in diameter. Structural fill should be placed and compacted to at least 95 percent of the maximum dry density (MDD) as determined by test method ASTM D-1557.

Imported material should be granular material with less than 15 percent fines such as Common Borrow as specified in Section 9-03.14(3) of the WSDOT Standard Specifications. In wet weather conditions or situations requiring free-draining backfill, we recommend using import material meeting the criteria for Gravel Borrow as specified in Section 9-03.14(1) of the WSDOT Standard Specifications. Class A Gravel Backfill for Foundations as specified in Section 9-03.12(1)A of the WSDOT Standard Specifications should be used for base rock underneath structures. Crushed Surfacing Base Course as specified in Section 9-03.9(3) of the WSDOT Standard Specifications should be used as base rock for new pavement.

Within a lateral distance of 3 feet of any wall, smaller, possibly hand-operated equipment should be used in conjunction with thinner soil lifts to achieve the required compaction so as not to damage the structure.

The procedure to achieve the specified minimum relative compaction depends on the size and type of compacting equipment, the number of passes, thickness of the layer being compacted, and certain soil properties. When size of the excavation restricts the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough lifts to achieve the required compaction. A sufficient number of in-place density tests should be performed as the fill is placed to verify the required relative compaction is being achieved. The frequency of the in-place density testing can be determined at the time of final design when more details of the Project grading and backfilling plans are available.

3.3 Bridge Foundations

Technically viable geotechnical options exist to support the proposed short bridge. Foundation design and selection must consider the design loads, subsurface conditions, constructability, construction impacts (nearby facilities, infrastructure and habitat), and cost.

Based on the subsurface conditions and liquefaction hazard present at the Site, we recommend utilizing a deep foundation system for the short bridge to mitigate against the effects of liquefaction. Deep foundations will bypass the liquefiable alluvium deposits, and gain foundation bearing and support from the underlying dense glacial outwash deposits. We recommend closed-end, concrete filled, steel pipe piles as the preferred deep foundation system for the short bridge.

The capacities for the steel pipe piles were analyzed and developed in accordance with the methods presented in Section 10.6 of the AASHTO LRFD Bridge Design Specifications.

3.3.1 Driven Piles

Driven piles can be installed relatively quickly and there are practical ways to verify their capacity in the field during construction. Driven piles commonly used in the Puget Sound area include steel pipe piles (driven steel walled pipes that are in-filled with reinforced concrete and are also known as cast-in-place concrete piles) and precast, prestressed concrete piles. The relative cost advantage of these two pile types fluctuates with the price of steel and concrete. Steel pipe piles have advantages in that they are durable, are easy to splice, and can be inspected from the interior after driving. They have also been shown to better resist cyclic loads more effectively than precast, prestressed concrete piles. However, they are vulnerable to corrosion unless they are appropriately protected. Precast, prestressed concrete piles are more resistant to corrosion than steel piles. However, they are somewhat brittle and they must be handled carefully, and they are difficult to splice. Concrete piles also require additional lead time for casting and curing.

For the short bridge, we recommend using 18-inch diameter closed-end steel pipe piles.

3.3.1.1 Driven Pile Axial Resistance

Axial pile resistance analyses were completed for driven, closed-end 18-inch diameter, steel pipe piles in accordance with AASHTO BDS guidelines. The analyses were performed using the Federal Highway Administration (FHWA, 2007) Driven Analysis Program.

The results of our axial resistance analyses are presented as nominal (ultimate) resistances for both bearing (compression) and uplift (tension) for a single driven pile. The estimated nominal resistances are shown on Figure D-1 in Appendix D. The computed nominal axial resistances are applicable to piles with a minimum spacing of 2.5 pile diameters, we should be consulted to consider group effects if pile spacing is less than 2.5 piles diameters.

The recommended Resistance Factors are shown in Table 6 and can be used in conjunction with Figure D-1 to determine estimated strength, service, and extreme limit state geotechnical resistances at various driven pile embedment depths. Estimating of the extreme limit state resistances should take into account the effects of the predicted liquefaction and downdrag (DD) load, described in Section 3.3.2.3. Pile embedment was assumed to begin at Elevation 45.

It is important to understand that the nominal resistances shown on Figure D-1 are *estimates* based on static analysis methods and pile resistance should be confirmed by field observations and made during driving as discussed in Section 4.3 – Geotechnical Monitoring of Driven Piles.

Table 5 – Recommended Resistance Factors for Driven Pile Design

Limit State	Resistance Factor, ϕ		
	Bearing Resistance, ϕ_{stat}[1]	Bearing Resistance, ϕ_{dyn}[2],	Uplift, ϕ_{up}
Strength	0.45	0.55[3]	0.35
Service	1.0	1.0	1.0
Extreme	1.0	1.0	1.0

Notes:

1) Applies to nominal resistance as determined by static analysis methods (see Figure D-1).
2) Applies to nominal resistance as determined by dynamic analysis methods during pile driving.
3) Assumes the WSDOT driving formula will be used as the basis for the dynamic analysis and pile driving construction control.

3.3.1.2 Minimum Pile Penetration

We recommend that the piles be driven/installed to a minimum depth of 32 feet below the existing roadway surface elevation. This equates to a recommended minimum pile tip Elevation of 21.5. Depending on the structural design requirements, piles may need to be driven/installed deeper than the minimum pile tip elevation to develop the required geotechnical resistance. Actual pile depths will need to be evaluated in the field through a combination of installation observation and dynamic or static load testing, as appropriate.

3.3.1.3 Downdrag

Liquefaction is predicted from approximately Elevation 29 to 36 at the Site. Liquefaction will cause temporary loss of support within the above elevation range and liquefaction-induced settlement will result in downdrag loads on the piles. For calculating extreme limit state pile resistance, we recommend ignoring bearing and uplift resistance within the estimated range of liquefaction. Additionally, we recommend applying an ultimate (total) downdrag load (DD) equal to 2.4 kips per inch of pile diameter to pile design. The zone of downdrag loading utilized to develop the ultimate DD load extends from the estimated top of the piles to Elevation 29. We recommend a load factor (γp_{DD}) of 1.05 be applied to the downdrag load. The recommended ultimate downdrag load applies to the pile shaft and assumes no helices would be included within or above the predicted zone of liquefaction if helical piles are used.

3.3.1.4 Pile Lateral Resistance

Lateral loading on the piles is anticipated to be minor. We understand lateral loads that occur parallel to the roadway and bridge will be transmitted through the bridge girders and utilize the passive earth pressure support against the opposite abutment for resistance. For the type and configuration of the short bridge being considered for the Project, the AASHTO BDS also do not require pile foundations to be specifically designed for lateral seismic loading. For lateral resistances at the abutment elevation, refer to the nominal passive earth pressure provided on Table 7.

3.4 Wall Considerations

Abutment and wing walls for the short bridge should be designed considering the lateral earth pressure considerations below. We assume the integral design approach for the short bridge abutments will result in at-rest earth pressure conditions and wing walls will result in an active earth pressure condition with a linear change between the restrained abutments and the unrestrained ends of the wing walls.

Imported wall backfill materials should consist of material meeting the requirements of Gravel Backfill for Walls (Section 9-03.12(2) of the WSDOT Standard Specifications). A suitable wall drainage system should be incorporated into the design to prevent buildup of excess hydrostatic pressure. If groundwater conditions prevent the practical installation of a wall drainage system, the wall should be designed to accommodate the additional loading from unbalanced hydrostatic pressure.

3.4.1 Lateral Earth Pressures

The recommended lateral earth pressures for use in design of the Project walls assume granular structural fill will be imported and placed as a horizontal backfill around the walls as described above.

Table 6 – Lateral Earth Pressure Parameters

Lateral Pressure Condition	Earth Pressure Coefficient	Equivalent Fluid Weight[1] (pcf)	Earth Pressure[2] (psf)	Surcharge Pressure (psf)
Active (K_a)	0.28	35	35H	0.28S[9]
At-Rest (K_o)	0.44	55[3]	55H[3]	0.44S[9]
Passive (K_p)[4]	4.86	350[5]	350D[5,6,7,8]	-
Active Seismic			8H[10]	
At-Rest Seismic[11]	-	-	62.5H	-
General Traffic Surcharge	-	-	-	140

Notes:

1) Granular backfill placed as structural fill with a unit weight of 125 pcf (120 pcf dry) is assumed.
2) Static earth pressures result in a triangular pressure distribution along the height of the wall. Seismic earth pressures and surcharge pressures result in a uniform pressure distribution along the height of the wall.
3) If unbalanced hydrostatic conditions are anticipated, the equivalent fluid weight should be increased to 90 pcf (or earth pressure 90H psf) for the at-rest condition.
4) To invoke the passive conditions, the wall must move into the backfill with a lateral movement of approximately 0.020H.
5) Nominal passive pressures are presented, a strength limit state resistance factor (ϕ_{ep}) of 0.50 should be applied for design.
6) Where D is the depth of embedment of wall below finish grade (including scour).

7) Passive pressure should be ignored within 18 inches below finish grade and anywhere wall foundations are adjacent to alluvium soils during the extreme limit state due to liquefaction concerns.
8) Assumes the wall foundations are adjacent to unsaturated embankment fill soils. The ultimate passive pressure should be reduced to 250 psf, if the wall foundations are adjacent to saturated alluvium or fill soils.
9) Resulting uniform surcharge acting along the height of the wall, where S is the surcharge pressure in psf.
10) Represented as a uniform (rectangular) distribution along the exposed height of the wing walls.
11) The at-rest seismic earth pressure for the bridge abutments represents the additional force increment required for embankment stabilization discussed in Section 2.6.1 and should be applied as a uniform (rectangular) pressure distribution.
12) Seismic and surcharge pressures are typically not considered concurrently in design unless specific conditions dictate otherwise.
13) Where walls transition from restrained (at-rest conditions) to unrestrained (active conditions), we recommend varying the earth pressures linearly between the two conditions.

Over-compaction of the backfill behind walls should be avoided. We recommend compacting backfill behind walls to approximately 90 percent of MDD as determined by ASTM D-1557 (Modified Proctor). Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any embedded wall to avoid the buildup of excessive lateral pressures. Compaction close to the walls should be accomplished using hand-operated vibratory plate compactors.

Live load surcharge (LS) from vehicular loading should be taken as a uniform load of 140 PSF acting against the short bridge abutments (stem walls).

Lateral forces that may be induced on the walls due to unique surcharge loads, such as heavy construction equipment, should be considered on a case-by-case basis by the structural engineer.

4 Construction Considerations

Based on our current knowledge of site conditions, we recommend that the following construction considerations be evaluated for construction of the geotechnical project elements.

- Fill and alluvium deposits underlying the Site may contain varying quantities of debris and obstructions. Although not directly observed in our explorations, our regional experience indicates obstructions may include buried debris such as large logs and/or cobbles. Obstructions may present difficulties during installation of deep foundations for the short bridge concept.
- Limited access and staging areas may constrain the movement and storage of larger construction equipment and materials. Selection of foundation elements and a bridge structure that can be quickly and efficiently installed may reduce road closure time and equipment staging needs.

The recommendations in this report are contingent upon good construction practices. The following sections present general, preliminary guidelines for consideration during construction.

4.1 General

Construction of the geotechnical project elements will be impacted by the presence of potentially wet soils and potential obstructions. These potential difficulties should be appropriately addressed in the contract documents. For example, the specifications should notify the contractor of the known presence of potential obstructions and their removal should be defined as incidental to general excavation. The Project specifications should require that the contractor provide submittals detailing the selected piles along with information regarding the respective pile driving systems and equipment that will be used for installing the piles prior to the start of construction.

All excavations, foundation installations, final abutment slope grading, and structural fill placement should be evaluated by the project geotechnical engineer and completed in accordance with the WSDOT Standard Specifications.

4.2 Driven Pile Installation

In general, pile driving construction should follow the guidelines set forth in WSDOT Standard Specifications Section 6-05. We recommend a minimum pile tip Elevation of 21.5.

Installation of piles may be impacted by the potential presence of obstructions (intact wood debris or gravels). Obstructions encountered during pile driving may cause some of the piles to be driven out-of-plumb, or to “drift” off of the design horizontal location. Also, if significant obstructions are encountered at certain locations, it may be necessary to adjust certain pile locations to avoid the obstructions. Because of this potential effect, some flexibility should be allowed in the design to enable adjustment of pile locations. In certain instances, it may be necessary to alter the size of the pile cap to accommodate the new pile locations. Any such situations which arise during construction should be evaluated on a case-by-case basis by the owner, structural engineer, and geotechnical engineer.

4.1 Geotechnical Monitoring of Driven Piles

All pile installation operations should be observed by the geotechnical engineer, or their field representative, experienced in the design and observation of driven piles foundations. It is essential that the field representative be present during pile driving to obtain blow count and hammer data to evaluate if the required nominal resistance has been developed.

Driven piles should be driven using an approved top-impact hammer or vibratory hammer. Selection of an appropriate hammer will depend on the pile types and sections selected for use on the project, the contractor's methods, and other factors. Prior to driving any piles, the contractor should submit details of the proposed pile driving system and driving criteria that can conservatively meet the required ultimate bearing capacities while preventing pile damage. The proposed pile driving system and driving criteria should meet the minimum requirements as presented in Section 6-05 of the WSDOT Standard Specifications.

For a top-impact hammer, a wave equation analysis of piles (WEAP) should be generated to guide the selection of properly sized driving equipment to ensure the selected pile section can be driven to the required resistance without damaging the pile. A WEAP analysis will also provide for a minimum penetration rate required for the pile to sufficiently develop the required resistance.

For a vibratory hammer system, there is no equivalent method for evaluating driving resistance as can be done for top-impact hammer systems. Therefore, for a vibratory hammer system, we recommend the project specifications be written to require installation of indicator piles and a full scale load test program or the use of an impact hammer (post-driving) to strike the individual piles and determine relative pile resistance.

Regardless of the hammer system chosen, the most practical way of determining pile driving conditions at the Site may be through the installation of a test pile. We recommend the contract include the requirement that one production pile be driven as a test pile in accordance with WSDOT Standard Specifications Section 6-05.3(10), so that field conditions and pile driving acceptance criteria can be developed. The owner's geotechnical engineer (not the contractor) should monitor and evaluate test pile driving, and develop acceptance criteria for the remaining production piles.

5 References

American Association of State Highway and Transportation Officials (AASHTO), 2012, LRFD Bridge Design Specifications, Customary U.S. Units.

American Society for Testing and Materials (ASTM), 2012, American Society of Testing Materials Annual Book of Standards, Vol. 4.08, West Conshohocken, Pennsylvania.

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Kramer, S., 2008, Evaluation of Liquefaction Hazards in Washington State, prepared for the Washington State Transportation Commission.

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Washington State Department of Fish and Wildlife (WDFW), 2013, Salmon Ck at West Uncas Rd Culvert Removal and Bridge Design, WDFW Region 6 Habitat Engineering Technical Assistance Memorandum.

Washington State Department of Transportation (WSDOT), 2014a, Geotechnical Design Manual M 46-03.

Washington State Department of Transportation (WSDOT), 2014b, Standard Specifications for Road, Bridge and Municipal Construction, Document M 41-10.

Washington State Legislature, 2009, Washington Administrative Code (WAC), April 1, 2009.

Limitations

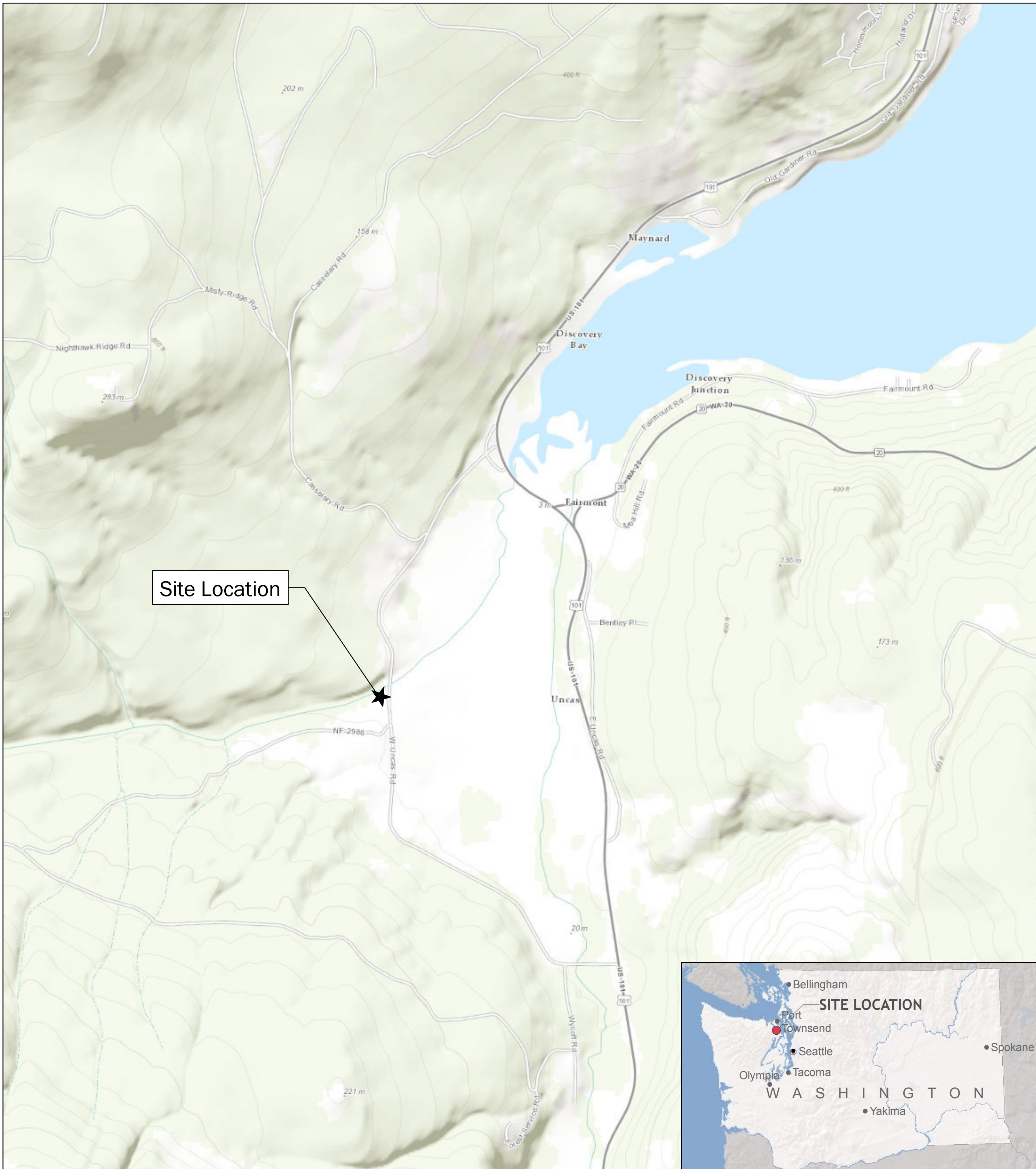
Work for this project was performed for Shearer Design, LLC and Jefferson County Public Works (Client), and this report prepared in accordance with generally accepted professional practices for the nature and conditions of work completed in the same or similar localities, at the time the work was performed. This report does not represent a legal opinion. No other warranty, expressed or implied, is made.

All reports prepared by Aspect Consulting for the Client apply only to the services described in the Agreement(s) with the Client. Any use or reuse by any party other than the Client is at the sole risk of that party, and without liability to Aspect Consulting. Aspect Consulting's original files/reports shall govern in the event of any dispute regarding the content of electronic documents furnished to others.

This report and our conclusions and interpretations should not be construed as a warranty of the subsurface conditions. Experience has shown that subsurface soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations and may not be detected by a geotechnical study. Further geotechnical evaluations, analyses, and recommendations may be necessary for the final design of this project.

If there is a substantial lapse of time between the submission of this report and the start of construction, or if conditions have changed due to construction operations at or near the Site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

FIGURES



GIS Path: Q:_GeoTech\140256 Salmon Creek Culvert Replacement\2014-12 Culvert Replacement\GIS\Site Location Map.mxd || Coordinate System: NAD 1983 HARN StatePlane Washington South FIPS 4602 Feet || Date Saved: 12/12/2014 || User: scudd || Print Date: 12/12/2014

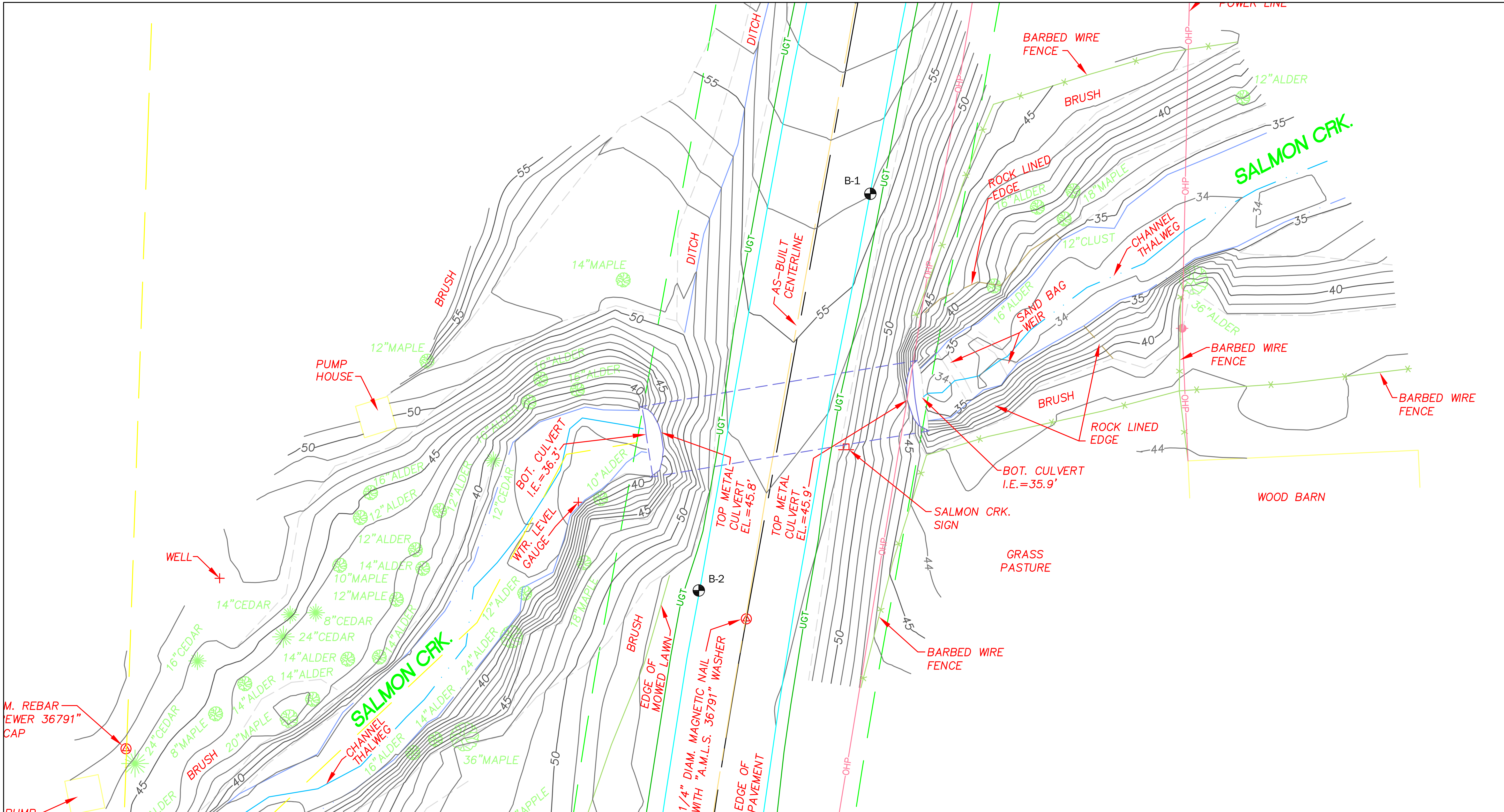
0 2,000 4,000
Feet

Site Location Map

Salmon Creek Culvert Replacement
Jefferson County, Washington

Aspect CONSULTING	DEC-2014	BY: AJH/SCC	FIGURE NO. 1
	PROJECT NO. 140256	REV BY: - - -	

Basemap Layer Credits || Sources: Esri, HERE, DeLorme, TomTom, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community



Base map provided by Van Aller Surveying, Carlsborg, Washington, October 2014.

Legend

Boring Location

0 20 40 Feet

DRAFT

Site and Exploration Map

Salmon Creek Culvert Replacement
Jefferson County, Washington

Aspect CONSULTING

JAN-2015	BY: AJH/SCC	FIGURE NO. 2
PROJECT NO. 140256	REVISED BY: -	

CAD Path: Q:_GeoTech\140256 Salmon Creek Culvert Replacement\2014-12 Culvert Replacement\140256-02.dwg 11x17 Landscape || Date Saved: Jan 05, 2015 11:22am || User: aholmson

APPENDIX A

Subsurface Explorations

A.1 Field Exploration Program

A.1.1 Geotechnical Borings

Geotechnical borings B-1 and B-2 were drilled using hollow-stem auger drilling techniques. The drilling was subcontracted to Geologic Drill, an experienced and licensed local driller. Drilling was completed with a trailer-mounted drill rig and 8-inch-diameter ($3^1/_4$-inch inside diameter) hollow-stem auger equipment. The locations of the two borings are shown on Figure 2. Borings B-1 and B-2 were both advanced to a depth of 51.5 feet bgs.

Sampling was completed at selected depth intervals using the Standard Penetration Test (SPT) in general accordance with ASTM Method D-1586. This involves driving a 2-inch outside-diameter split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling from a distance of 30 inches. The number of blows for each 6-inch interval is recorded and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance (“N”) or blow count. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils.

An Aspect geologist was present throughout the field exploration program to observe the drilling procedure, assist in sampling, and to prepare descriptive logs of the exploration. Soils were classified in general accordance with ASTM D-2488, *Standard Practice for Description and Identification of Soils* (Visual-Manual Procedure). The summary exploration log represents our interpretation of the contents of the field logs. The stratigraphic contacts shown on the individual summary logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The subsurface conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

Major Divisions		Fines	Group Symbol	Group Name
Coarse-Grained Soils - More than 50% [1] Retained on No. 200 Sieve	Gravels - More than 50% [1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines [5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines [5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50% [1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines [5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines [5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50% [1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT [2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT [2] blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

- FC = Fines Content
- G = Grain Size
- M = Moisture Content
- A = Atterberg Limits
- C = Consolidation
- DD = Dry Density
- K = Permeability
- Str = Shear Strength
- Env = Environmental
- PiD = Photoionization Detector

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

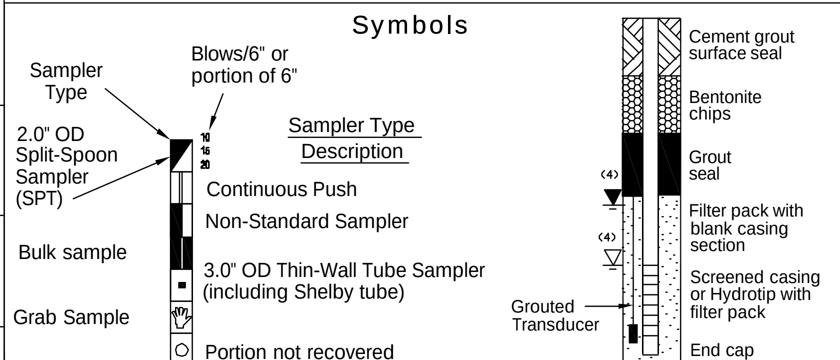
[3] Estimated Percentage

Percentage by Weight	Modifier
<5	Trace
5 to 15	Slightly (sandy, silty, clayey, gravelly)
15 to 30	Sandy, silty, clayey, gravelly)
30 to 49	Very (sandy, silty, clayey, gravelly)

Moisture Content

- Dry - Absence of moisture, dusty, dry to the touch
- Slightly Moist - Perceptible moisture
- Moist - Damp but no visible water
- Very Moist - Water visible but not free draining
- Wet - Visible free water, usually from below water table

Symbols



(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of groundwater ▽ ATD = At time of drilling ▼ Static water level (date)

(5) Combined USCS symbols used for fines between 5% and 15% as estimated in General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)

BGS = below ground surface

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Aspect CONSULTING	Exploration Log Key	DATE:	PROJECT NO.
		DESIGNED BY:	
		DRAWN BY:	FIGURE NO.
		REVISED BY:	A-1

Q:_ACAD Standards\Standard Details\Exploration Log Key A1.dwg

Boring Log

Aspect CONSULTING

Project Number	Boring Number	Sheet
140256	B-1	1 of 3

Project Name: Salmon Creek Culvert Replacement

Location: West Uncas Road, Port Townsend, Washington

Driller/Equipment: Geologic Drill / Trailer-mounted 8" OD HSA w/ Cathead

Drilling Method/Hammer: HSA / 140 lbs / 30" drop

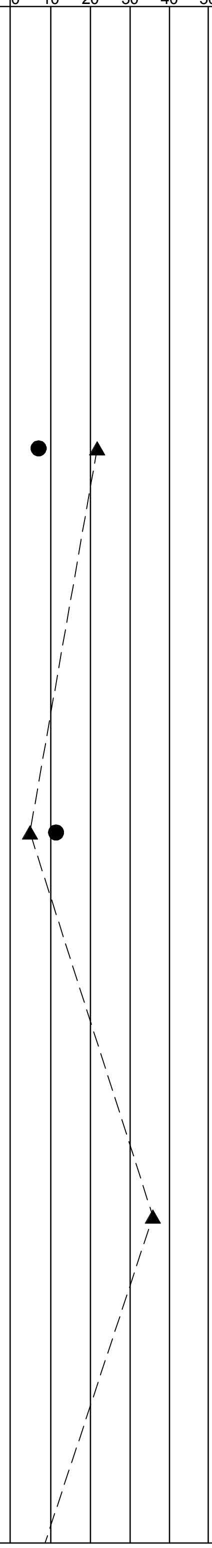
Ground Surface Elev (NAVD 88) 54

Depth to Water (ft BGS) 22.2

Start/Finish Date 11/19/2014

Depth / Elevation (feet)	Borehole Completion	Sample Type/ID	Tests	Blows/ 6"	Description	Depth (ft)
0 / 54					Asphalt roadway	
					ARTIFICIAL FILL Medium dense, moist, brown, gravelly SAND (SP); predominantly fine to medium sand, fine subangular to subrounded gravel	
1 / 53	Black-dyed concrete					1
2 / 52						2
3 / 51						3
4 / 50						4
5 / 49		S1	M	9, 14, 8	Trace silt with broken gravel in sampler	5
6 / 48						6
7 / 47						7
8 / 46	Backfilled with hydrated bentonite chips					8
9 / 45						9
10 / 44		S2	M	3, 3, 2	Becomes very loose, with trace wood in a 2" pocket at 11' bgs	10
11 / 43						11
12 / 42						12
13 / 41						13
14 / 40					"Driller indicates becomes firmer at 14' bgs	14
15 / 39		S3		18, 13, 23	Becomes dense; no recovery, gravel blocking sampler	15
16 / 38						16
17 / 37					Driller indicates gravel size increasing with depth to 18' bgs, then drilling smoothes out	17
18 / 36					**RECENT ALLUVIUM** Very loose, wet, brown, silty, gravelly SAND (SM); predominantly fine to medium sand, fine subrounded gravel, scattered pockets of sandy brown gray SILT with numerous wood organics (~25%)	18
19 / 35						19

Blows/foot ▲ Water Content % ●



Sampler Type:

○ No Recovery

◤ Standard Penetration Test (ASTM D1586)

Drilling Method:

HSA: Hollow Stem Auger

MR: Mud Rotary

Logged by: MML

Approved by: EOA

Figure No. A- 2

Project Name: Salmon Creek Culvert Replacement
Ground Surface Elev (NAVD 88) 54
Location: West Uncas Road, Port Townsend, Washington
Driller/Equipment: Geologic Drill / Trailer-mounted 8" OD HSA w/ Cathead
Depth to Water (ft BGS) 22.2
Drilling Method/Hammer: HSA / 140 lbs / 30" drop
Start/Finish Date 11/19/2014

Depth (feet)	Elevation (feet)	Borehole Completion	Sample Type/ID	Tests	Blows/ 6"	Blows/foot ▲ / Water Content % ● (0 10 20 30 40 50)	Description	Depth (ft)
							RECENT ALLUVIUM	
21	33		S4	M,G	5 2 2		Very loose, wet, brown, silty, gravelly SAND (SM); predominantly fine to medium sand, fine subrounded gravel, scattered pockets of sandy brown gray SILT with numerous wood organics (~25%)	21
22	32	▽ 11/19/2014						22
23	31							23
24	30							24
25	29						**GLACIAL RECESSIONAL OUTWASH DEPOSITS**	25
26	28		S5	M,G	9 15 15		Medium dense to dense, wet, brown, slightly silty, gravelly SAND (SW-SM); fine to coarse sand, fine subrounded gravel	26
27	27							27
28	26							28
29	25							29
30	24							30
31	23		S6	M	9 16 27		Dense, wet, brown, slightly gravelly SAND (SW); trace silt, fine to coarse sand, fine subrounded gravel	31
32	22							32
33	21							33
34	20							34
35	19							35
36	18		S7	M	11 17 33	50+	Dense to very dense, wet, brown, slightly silty, gravelly SAND (SW-SM); fine to coarse sand, fine subrounded gravel with silt increasing with depth	36
37	17							37
38	16							38
39	15							39

Sampler Type:
- ○ No Recovery
- ◪ Standard Penetration Test (ASTM D1586)

Drilling Method:
- HSA: Hollow Stem Auger
- MR: Mud Rotary

Logged by: MML
Approved by: EOA
Figure No. A- 2

Boring Log

Aspect CONSULTING

Project Number	Boring Number	Sheet
140256	B-1	3 of 3

Project Name: Salmon Creek Culvert Replacement

Location: West Uncas Road, Port Townsend, Washington

Driller/Equipment: Geologic Drill / Trailer-mounted 8" OD HSA w/ Cathead

Drilling Method/Hammer: HSA / 140 lbs / 30" drop

Ground Surface Elev (NAVD 88) 54

Depth to Water (ft BGS) 22.2

Start/Finish Date 11/19/2014

Depth (feet)	Elevation (feet)	Sample Type/ID	Tests	Blows/6"	Blows/foot (0 10 20 30 40 50)	Description	Depth (ft)
		S8		21 23 28	50+	Dense to very dense, wet, brown, slightly silty, gravelly SAND (SW-SM); fine to coarse sand, fine subrounded gravel with silt increasing with depth	
41	13						41
42	12						42
43	11						43
44	10						44
45	9	S9		23 24 24			45
46	8						46
47	7						47
48	6						48
49	5						49
50	4	S10		10 27 45	50+	**GLACIAL ADVANCE OUTWASH** Very dense, wet, brown gray, gravelly SAND (SP); fine to medium sand, fine subrounded to rounded gravel	50
51	3						51
						Bottom of boring at 51.5' bgs, groundwater encountered at 22.2' bgs	
52	2						52
53	1						53
54	0						54
55	-1						55
56	-2						56
57	-3						57
58	-4						58
59	-5						59

Sampler Type:

- No Recovery
- Standard Penetration Test (ASTM D1586)

Drilling Method:

HSA: Hollow Stem Auger

MR: Mud Rotary

Logged by: MML

Approved by: EOA

Figure No. A- 2

Aspect CONSULTING

Boring Log

Project Number	Boring Number	Sheet
140256	B-2	1 of 3

Project Name: Salmon Creek Culvert Replacement

Location: West Uncas Road, Port Townsend, Washington

Driller/Equipment: Geologic Drill / Trailer-mounted 8" OD HSA w/ Cathead

Drilling Method/Hammer: HSA / 140 lbs / 30" drop

Ground Surface Elev (NAVD 88): 51

Depth to Water (ft BGS): 19.7

Start/Finish Date: 11/19/2014

Depth (feet)	Elevation (feet)	Borehole Completion	Sample Type/ID	Tests	Blows/6"	Blows/foot	Description
0	51						Asphalt
							ARTIFICIAL FILL Dense, moist, brown, gravelly, silty SAND (SM); predominantly fine sand, fine subrounded gravel
1	50	Black-dyed concrete					
2	49						
3	48						
4	47						
5	46		S1	M	10 / 13 / 7		
6	45						
7	44						
8	43	Backfilled with hydrated bentonite chips					
9	42						
10	41		S2	M	9 / 12 / 19		**RECENT ALLUVIUM** Medium dense to dense, moist, brown, silty, sandy GRAVEL (GM); fine to coarse sand, fine to coarse gravel
11	40						
12	39						Driller indicates very gravelly between 10 to 15' bgs; a lot of coarse gravel
13	38						
14	37						
15	36		S3	M	29 / 30 / 23	50+	Sampler likely obstructed by coarse gravel, blow count not representative of formation density
16	35						
17	34						Driller indicates much less gravel starting at 16.5'bgs
18	33						
19	32						
		▽ 11/19/2014					

Blows/foot ▲ / Water Content % ● (scale 0 10 20 30 40 50)

Sampler Type:

○ No Recovery

◩ Standard Penetration Test (ASTM D1586)

Drilling Method:

HSA: Hollow Stem Auger

MR: Mud Rotary

Logged by: MML

Approved by: EOA

Figure No. A- 3

Aspect CONSULTING

Boring Log

Project Number	Boring Number	Sheet
140256	B-2	2 of 3

Project Name: Salmon Creek Culvert Replacement

Location: West Uncas Road, Port Townsend, Washington

Driller/Equipment: Geologic Drill / Trailer-mounted 8" OD HSA w/ Cathead

Drilling Method/Hammer: HSA / 140 lbs / 30" drop

Ground Surface Elev (NAVD 88) 51

Depth to Water (ft BGS) 19.7

Start/Finish Date 11/19/2014

Depth (feet)	Elevation (feet)	Sample Type/ID	Tests	Blows/6"	Blows/foot ▲	Description	Depth (ft)
20	31	S4	M,G	10 14 24		Dense, wet, brown, sandy GRAVEL (GP); trace silt, fine to coarse sand, fine subrounded to rounded gravel	
21	30						21
22	29						22
23	28						23
24	27						24
25	26	S5	M,G	3 11 22		**GLACIAL RECESSIONAL OUTWASH DEPOSIT** Dense, wet, brown, slightly gravelly, silty SAND (SM); fine to coarse sand, fine subrounded to rounded gravel, 3" pocket of sandy SILT (ML) at 26' bgs	25
26	25						26
27	24						27
28	23						28
29	22						29
30	21	S6	M	22 33 44	50+	Becomes very dense, very gravelly	30
31	20					Begin to use bentonite slurry at 30' to prevent sand heave	31
32	19						32
33	18						33
34	17						34
35	16	S7	M	17 40 50/5"	50+	Becomes gray brown	35
36	15						36
37	14						37
38	13						38
39	12						39

Sampler Type:

- No Recovery
- Standard Penetration Test (ASTM D1586)

Drilling Method:

HSA: Hollow Stem Auger

MR: Mud Rotary

Logged by: MML

Approved by: EOA

Figure No. A- 3

Boring Log

Aspect CONSULTING

Project Number	Boring Number	Sheet
140256	B-2	3 of 3

Project Name: Salmon Creek Culvert Replacement
Ground Surface Elev (NAVD 88): 51
Location: West Uncas Road, Port Townsend, Washington
Driller/Equipment: Geologic Drill / Trailer-mounted 8" OD HSA w/ Cathead
Depth to Water (ft BGS): 19.7
Drilling Method/Hammer: HSA / 140 lbs / 30" drop
Start/Finish Date: 11/19/2014

Depth (feet)	Elevation (feet)	Sample Type/ID	Tests	Blows/6"	Blows/foot	Description	Depth (ft)
		S8		17 37 37	50+	Very dense, wet, brown, SAND (SP); trace silt, fine to medium sand, fine subrounded gravel	
41	10					Hard, wet, gray brown, sandy SILT (ML); frequent very thin laminae of fine to medium sand	41
42	9						42
43	8						43
44	7					**GLACIAL ADVANCE OUTWASH** Very dense, wet, gray brown SAND (SP); trace fine subrounded to rounded gravel, predominantly fine to medium sand	44
45	6	S9		21 32 50/5"	50+		45
46	5						46
47	4						47
48	3						48
49	2						49
50	1	S10		16 21 50/5.5"	50+		50
51	0						51
52	-1					Bottom of boring at 51.5' bgs, groundwater encounterd at 19.7' bgs	52
53	-2						53
54	-3						54
55	-4						55
56	-5						56
57	-6						57
58	-7						58
59	-8						59

Blows/foot ▲ Water Content % ● (scale 0 10 20 30 40 50)

Sampler Type:
- No Recovery
- Standard Penetration Test (ASTM D1586)

Drilling Method:
HSA: Hollow Stem Auger
MR: Mud Rotary

Logged by: MML
Approved by: EOA
Figure No. A- 3

APPENDIX B

Laboratory Testing Results

B.1 Geotechnical Laboratory Testing

Laboratory tests were conducted on selected soil samples to characterize certain engineering (physical) properties of the soils within the Study Area. Laboratory testing included determination of grain-size distribution, moisture content, and plasticity. The laboratory tests were conducted in general accordance with appropriate ASTM test methods. Test procedures are discussed below.

The grain size distribution of selected samples was analyzed in general accordance with ASTM D-422, *Standard Test Method for Particle-Size Analysis of Soils* without hydrometer determination of fines content. The moisture content of selected samples was analyzed in general accordance with ASTM D-2216, *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*.

The results of the grain size distribution tests are presented as curves in Appendix B, plotting percent finer by weight versus grain size. The results of the moisture content tests are presented in tabular form in Appendix B and graphically on the boring logs in Appendix A.

Hayre McElroy & Associates, LLC

Moisture Contents

Aspect - Salmon Creek Culvert Replacement

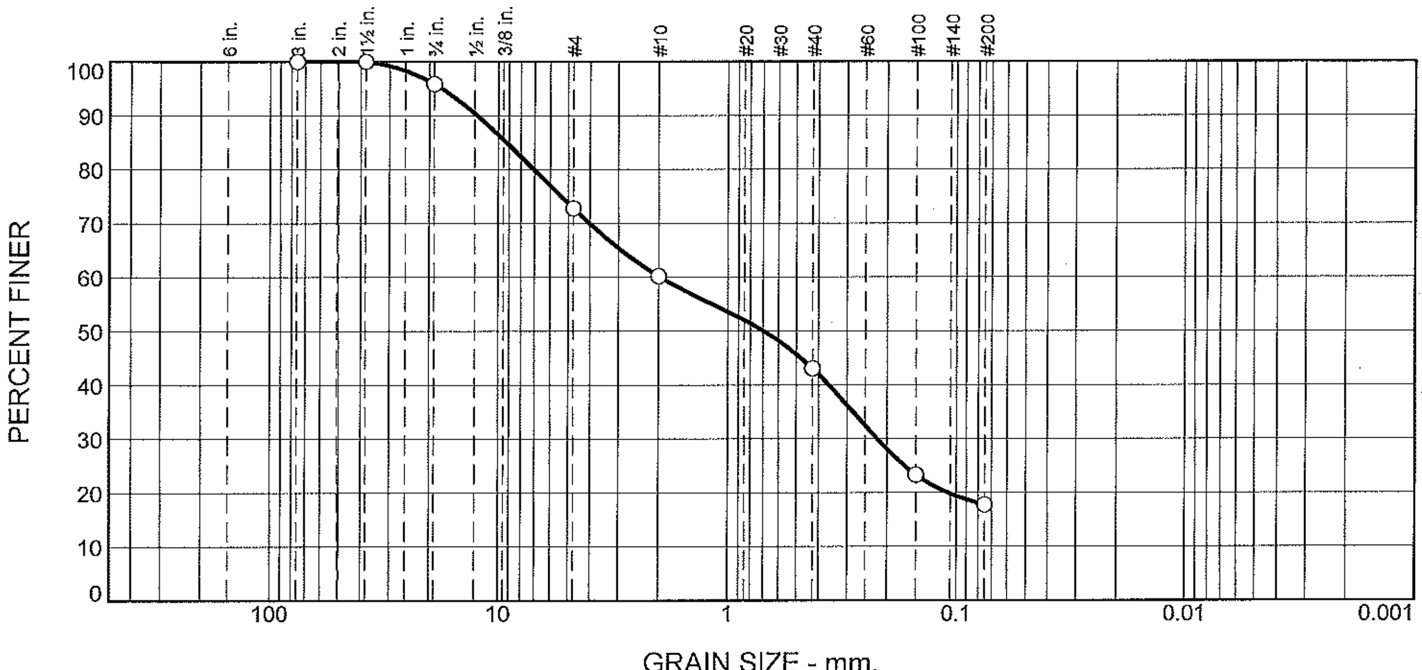
HMA Sample #	Sample # or Depth	Location	Date Received	Date of Test	Tare #	Wt of Tare	Tare+ Wet	Tare+ Dry	Moisture %
7715-1	5'	B-1 S-1	11/21/2014	11/21/2014	JZ	113.7	513.7	486.7	7.2
7715-2	10'	B-1 S-2	11/21/2014	11/21/2014	6A	114.2	447.2	412.6	11.6
7715-3	20'	B-1 S-4	11/21/2014	11/21/2014	M7	123	1374.7	1055.1	34.3
7715-4	25'	B-1 S-5	11/21/2014	11/21/2014	S7	115.3	987.6	884.5	13.4
7715-5	30'	B-1 S-6	11/21/2014	11/21/2014	M6	117.2	1202.1	1064.8	14.5
7715-6	35'	B-1 S-7	11/21/2014	11/21/2014	N7	219.3	1778.4	1587.9	13.9
7715-7	5'	B-2 S-1	11/21/2014	11/21/2014	8A	136.2	380.2	350.9	13.6
7715-8	10'	B-2 S-2	11/21/2014	11/21/2014	S4	118.4	422.60	409.70	4.4
7715-9	15'	B-2 S-3	11/21/2014	11/21/2014	5A	114.5	596.1	559.3	8.3
7715-10	20'	B-2 S-4	11/21/2014	11/21/2014	AJ^2	164.9	1021.8	927.6	12.4
7715-11	25'	B-2 S-5	11/21/2014	11/21/2014	X9	230.8	1590.6	1403.7	15.9
7715-12	30'	B-2 S-6	11/21/2014	11/21/2014	X6	196.1	1589.6	1450.7	11.1
7715-13	35'	B-2 S-7	11/21/2014	11/21/2014	1A	213.1	1298.8	1164.6	14.1

Particle Size Distribution Report

Project: Salmon Creek Culvert Replacement **Project No.:** 08-175

Client: Aspect

Location: B-1 S-4
Sample Number: 7715-3 **Depth:** 20' **Date:** 11/21/2014



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.2	23.0	12.6	17.1	25.4	17.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100.0		
1-1/2"	100.0		
3/4"	95.8		
#4	72.8		
#10	60.2		
#40	43.1		
#100	23.3		
#200	17.7		

* (no specification provided)

Material Description

Silty Sand with Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 9.2514 D_{60}= D_{50}= 0.6992
D_{30}= 0.2213 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Figure

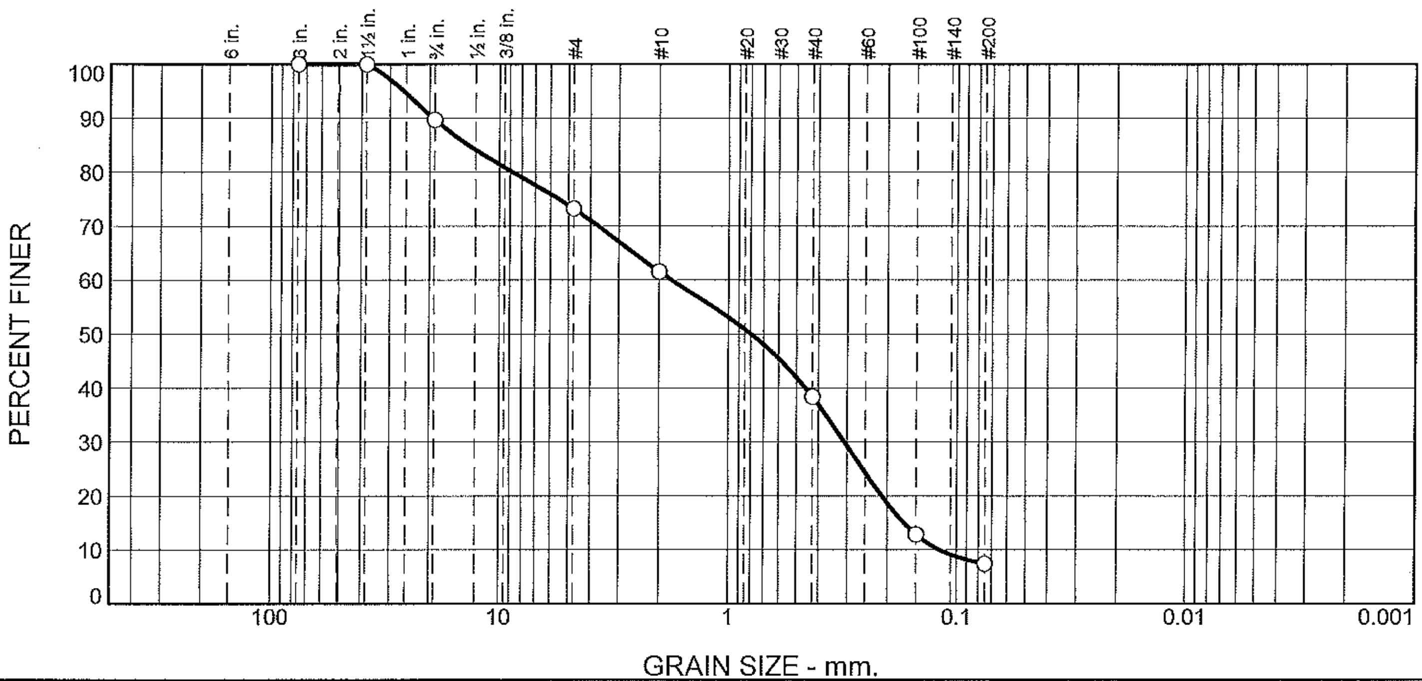
Tested By: RJF **Checked By:** JAM

Particle Size Distribution Report

Project: Salmon Creek Culvert Replacement **Project No.:** 08-175

Client: Aspect

Location: B-1 S-5
Sample Number: 7715-4 **Depth:** 25' **Date:** 11/21/2014



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.3	16.5	11.6	23.2	31.0	7.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100.0		
1-1/2"	100.0		
3/4"	89.7		
#4	73.2		
#10	61.6		
#40	38.4		
#100	12.8		
#200	7.4		

* (no specification provided)

Material Description

Sand with Silt and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 13.6607 D_{60}= D_{50}= 0.7960
D_{30}= 0.3080 D_{15}= 0.1693 D_{10}= 0.1191
C_u= C_c=

Classification

USCS= SW-SM AASHTO=

Remarks

Figure

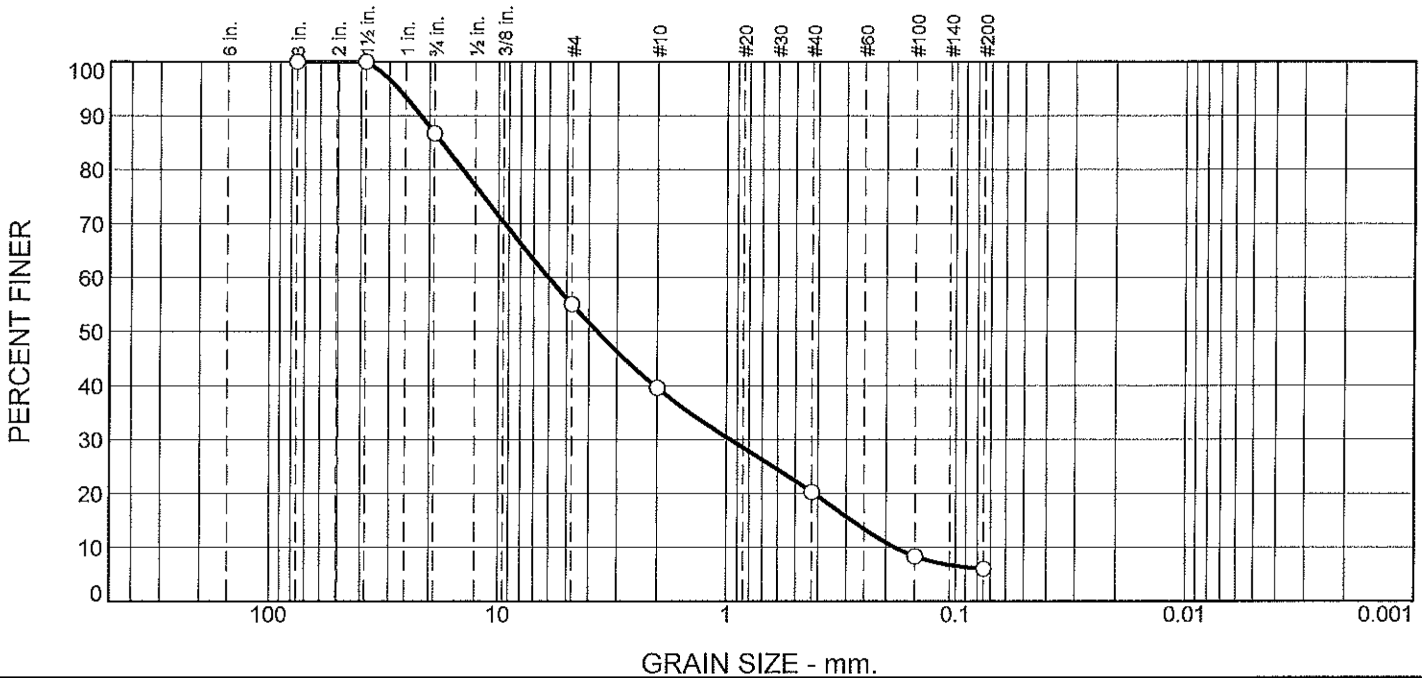
Tested By: RJF **Checked By:** JAM

Particle Size Distribution Report

Project: Salmon Creek Culvert Replacement **Project No.:** 08-175

Client: Aspect

Location: B-2 S-4
Sample Number: 7715-10 **Depth:** 20' **Date:** 11/21/2014



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.3	31.7	15.4	19.4	14.2	6.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100.0		
1-1/2"	100.0		
3/4"	86.7		
#4	55.0		
#10	39.6		
#40	20.2		
#100	8.3		
#200	6.0		

* (no specification provided)

Material Description

Sand with Silt and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 17.7228 D_{60}= 6.0217 D_{50}= 3.6835
D_{30}= 0.9665 D_{15}= 0.2841 D_{10}= 0.1839
C_u= 32.75 C_c= 0.84

Classification

USCS= SW-SM AASHTO=

Remarks

Figure

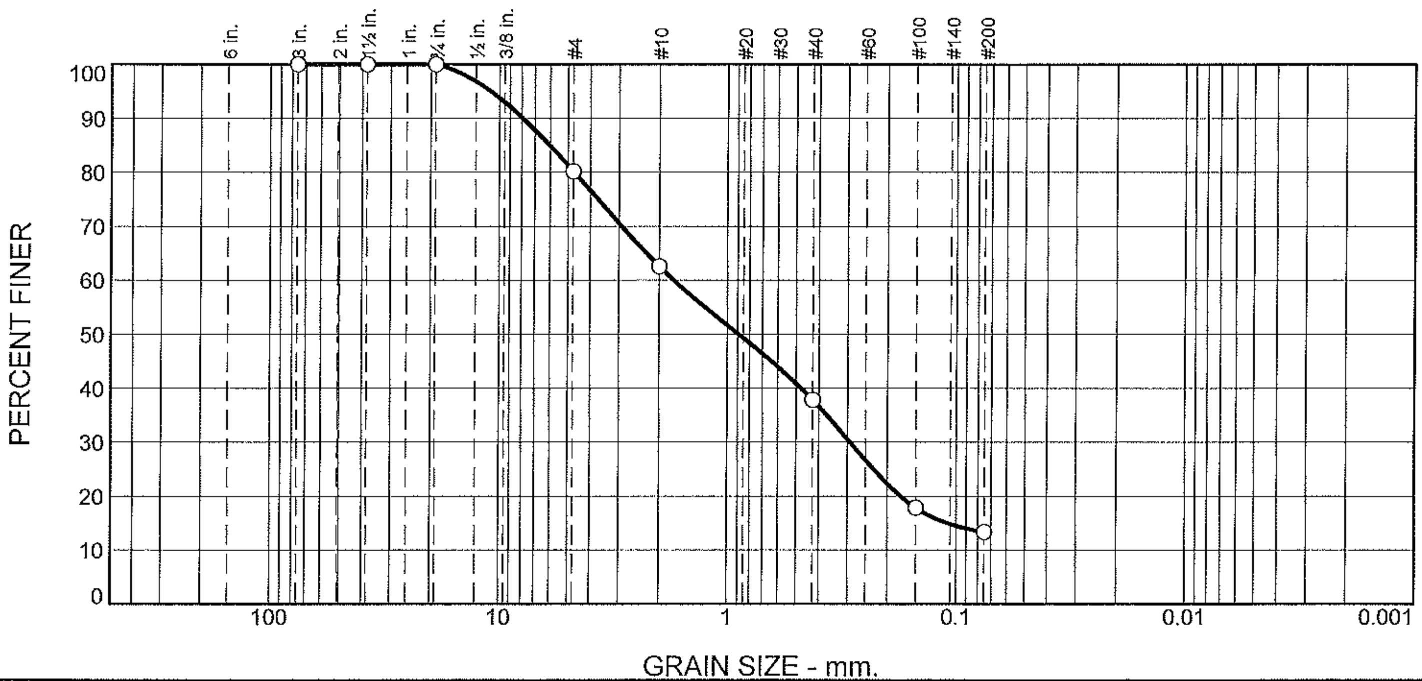
Tested By: RJF **Checked By:** JAM

Particle Size Distribution Report

Project: Salmon Creek Culvert Replacement **Project No.:** 08-175

Client: Aspect

Location: B-2 S-5
Sample Number: 7715-11 **Depth:** 25' **Date:** 11/21/2014



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	19.8	17.6	24.8	24.5	13.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100.0		
1-1/2"	100.0		
3/4"	100.0		
#4	80.2		
#10	62.6		
#40	37.8		
#100	17.8		
#200	13.3		

* (no specification provided)

Material Description

Silty Sand with Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 6.0275 D_{60}= D_{50}= 0.8913
D_{30}= 0.2925 D_{15}= 0.1096 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Figure

Tested By: RJF **Checked By:** JAM

APPENDIX C

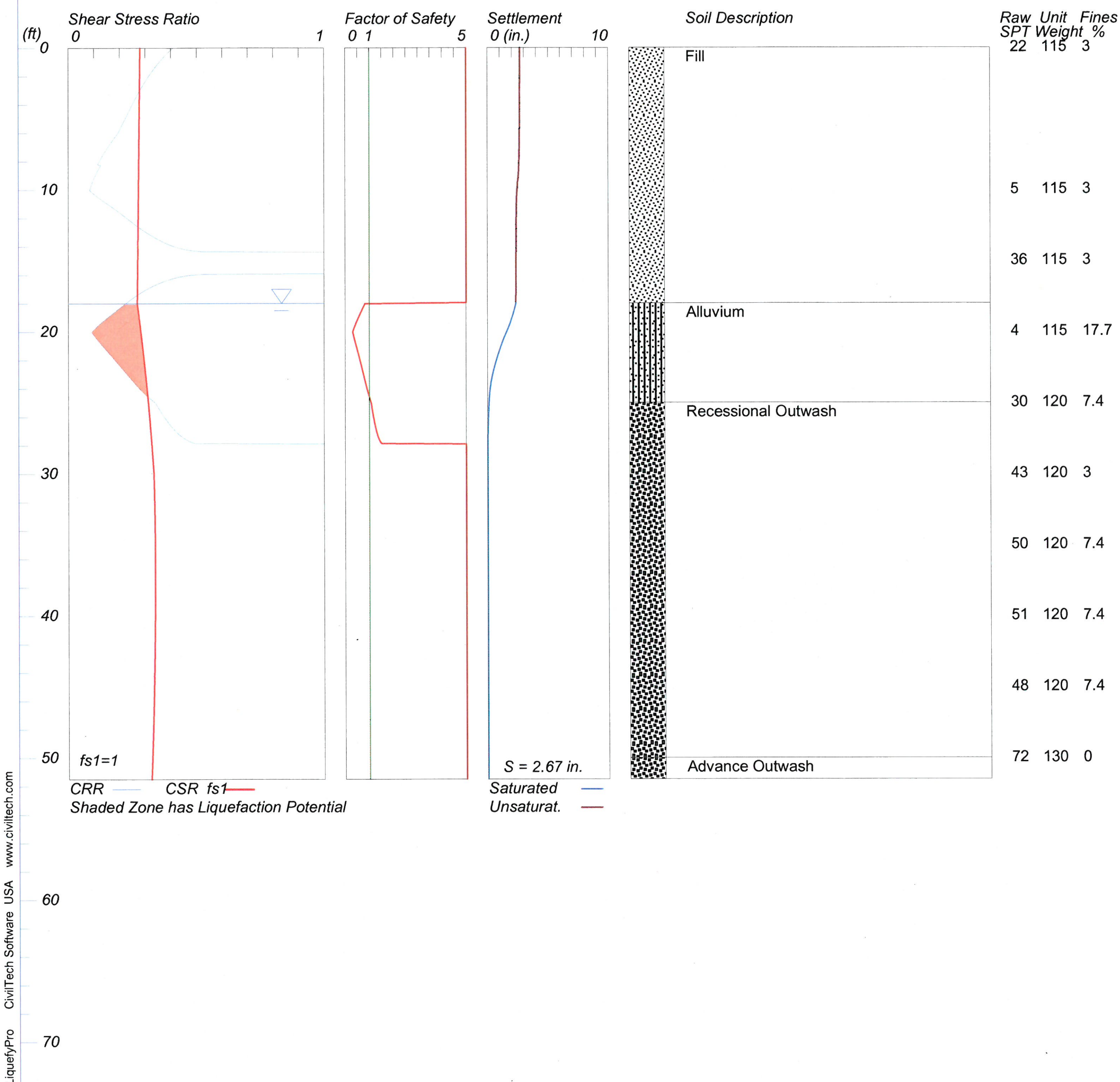
Liquefaction Analyses

LIQUEFACTION ANALYSIS

Salmon Creek Culvert Replacement

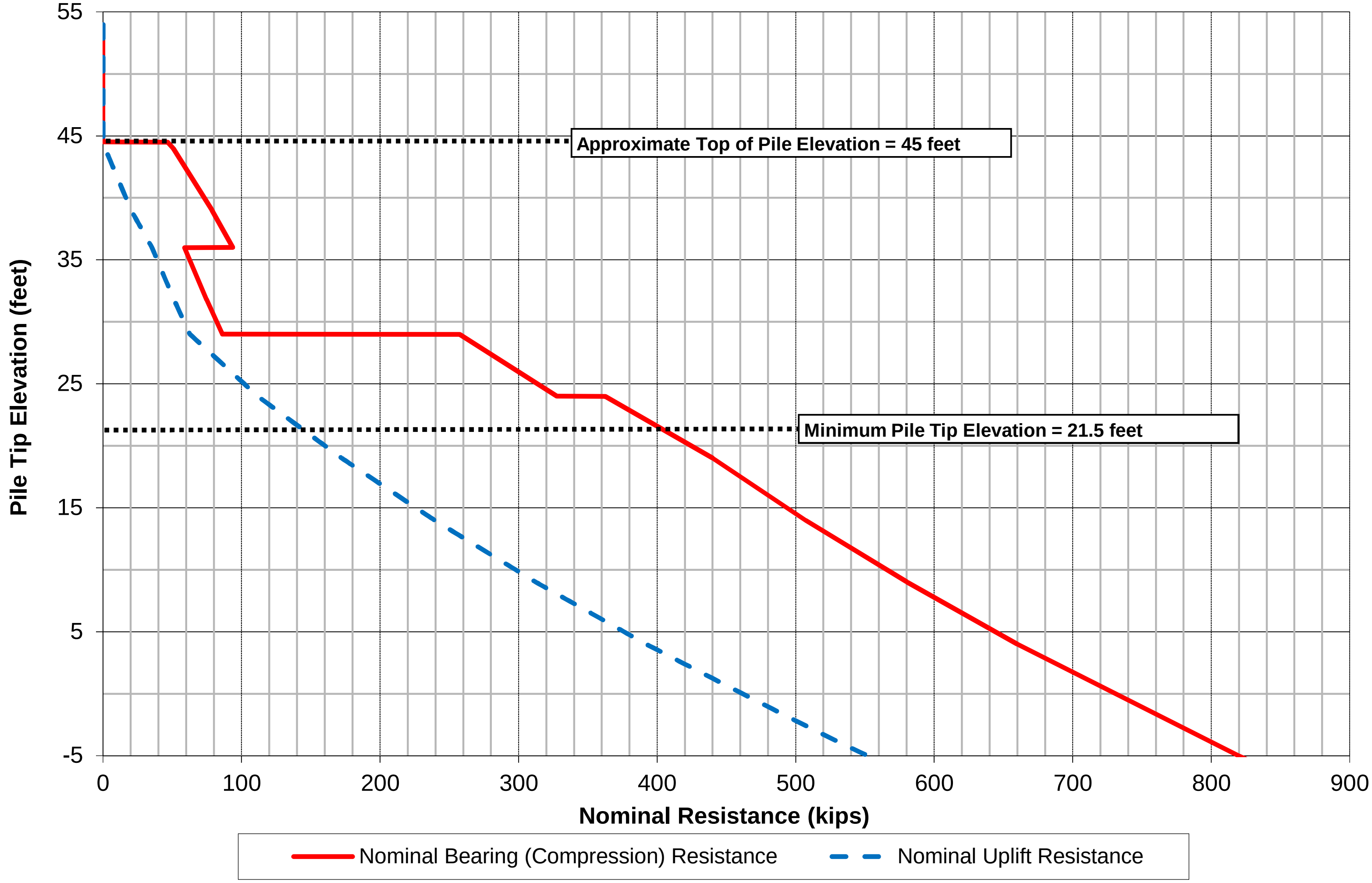
Hole No.=B-1   Water Depth=18 ft   Surface Elev.=54

Magnitude=7.01
Acceleration=0.432g



APPENDIX D

Pile Capacity Chart



Notes:
1) Nominal bearing resistance shown on this plot is unfactored and can be used with Table 6 in the text to determine the Strength, Service, and Extreme limit state pile resistances.
2) Liquefaction is predicted from Elevation 36 to 29. Nominal bearing and uplift resistance should be ignored within this elevation range for the Extreme limit state.
3) The nominal downdrag load (DD) is equal to 2.4 kips per inch of pile shaft diameter and applies from the pile top to Elevation 29 and should be applied for the Extreme limit state.

Aspect Consulting
January 2015

Figure D-1
Estimated Axial Pile Nominal Resistance -
Driven, Closed-End, 18-inch Diameter, Steel Pipe Pile