FINAL GEOTECHNICAL REPORT
Bridge B-15 Moses Road
Okanogan County, Washington

Prepared for: Nicholls Kovich Engineering, PLLC

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Introduction

This report presents the results of a geotechnical engineering study performed by Aspect Consulting, LLC (Aspect) for the Bridge B-15 Moses Road replacement project in Okanogan County, Washington. Our services were provided in support of an engineering study by Nicholls Kovich Engineering, PLLC (NKE) for the Okanogan County Department of Public Works (County).

The project involves the removal and replacement of the existing Bridge “B-15” (bridge) along Joe Moses Road spanning over the Little Nespelem River in Section 33, Township 31 N, Range 31 E in Okanogan County, Washington. The project location is shown on Figure 1, Site Location Map.

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

The project site (Site) is located along Joe Moses Road at Milepost 2.15, located about 5 miles southeast of Nespelem, Washington. Joe Moses Road crosses the Little Nespelem River (river) in the northwest to southeast direction via an existing 51-foot-span bridge. The existing bridge is comprised of a concrete deck and timber-pile-supported abutments and a center pier. The existing bridge currently has a 5-ton weight restriction due to structural deficiencies. The existing bridge deck sits at about Elevation 1905 feet at the north end and Elevation 1907 feet at the south end.

The existing bridge will be removed and replaced with a new prestressed-concrete bridge with voided slab girders. The new bridge is planned to have a 68-foot span that is about 32 feet wide and will be supported on abutments on each side of the river, with no center pier. The centerline of the new bridge will be in about the same location as the existing bridge. The new bridge deck is planned to sit at about Elevation 1906 feet at the north end and Elevation 1908 at the south end.

We understand the new bridge will be designed in accordance with the current American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (BDS) (AASHTO, 2014) and LRFD methodology.
3 Site Conditions

3.1 General Geology

The geology of the area is mapped as alluvium deposits (Qa) of silt, sand and gravel deposited by the Little Nespelem River and older glacial drift deposits (Qgd) of clay, silt, sand and gravel deposited by retreating glacier ice meltwater, outwash, and ice-contact environments (Joseph, 1990).

Fill (Af) is not mapped at the site, but is likely present from roadway embankment and bridge construction.

Soil units encountered in soil boring explorations completed at the site are described in more detail below in Section 3.5, Subsurface Conditions.

3.2 Seismicity

Rocks in the Project area were deformed by tectonic uplift of the Cascade Mountains, beginning in the late Eocene epoch (approximately 37 million years ago), resulting from collision of oceanic tectonic plates with the North American continental plate. The Cascadia Subduction Zone (CSZ) marks the seismically-active margin between these crustal plates.

The Site is located within this active seismic zone and is subject to earthquakes on shallow crustal faults, in addition to those within the CSZ. A shallow crustal earthquake with an estimated magnitude of 6.8 occurred in 1872, south of the Site near the town of Entiat (Bakun et al., 2002). The fault responsible for the 1872 earthquake has not been identified but is considered capable of producing future earthquakes. According to the U.S. Geological Survey (USGS), the largest future earthquakes within the region would likely be generated by shallow crustal faults and could exceed magnitude 7.

Hazards associated with the CSZ include deep earthquakes and subduction zone earthquakes. Deep earthquakes, which occur from tensional rupture of the sinking oceanic plate, typically have magnitude 7.5 or less and occur approximately every 10 to 30 years. The Site area is generally protected from strong shaking caused by these earthquakes due to the great depth to the hypocenter.

Subduction zone earthquakes occur due to rupture between the subducting oceanic plate and the overlying continental plate. These earthquakes typically have magnitude up to 9 and a recurrence interval on the order of 500 years. The last great subduction zone earthquake in Washington occurred about 300 years ago.

Due to the lengthy recurrence intervals between large seismic events, the potential for strong ground shaking is considered low during the life of the proposed Project, but must be considered for design of the structure, as required by AASHTO BDS (AASHTO, 2014).
3.3 Surface

The Site consists of a paved road (Joe Moses Road) surrounded by undeveloped land. Vegetation at the Site consists of shrubs and grass. Joe Moses Road crosses the Little Nespelem River via Bridge B-15 in the northwest to southeast direction. The Little Nespelem River channel thalweg is at approximately 1895 to 1896 feet.

Topography of the Site and location of the existing bridge is illustrated on Figure 2.

3.4 Field and Laboratory Investigations

3.4.1 Soil Borings

We completed two soil borings designated B-1 and B-2 on the north and south side, respectively, of Bridge B-15. The drilling was performed on January 12 and 13, 2016, by Haz-Tech Drilling under subcontract to Aspect using mud-rotary auger drilling methods. Disturbed soil samples were obtained using Standard Penetration Test (SPT) in accordance with ASTM D1586 at 2.5-foot intervals within the upper 20 feet of drilling and 5-foot intervals thereafter using a 140-pound hammer falling 30 inches and 2.0- and 3.0-inch outer-diameter split-spoon samplers. The number of blows for each 6-inch interval of split-spoon sampler advancement was 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. Boring B-1 and B-2 were drilled and sampled to depths of 76.5 and 61.5 feet, respectively.

An Aspect geotechnical engineer was present throughout the drilling to observe the drilling procedure, assist in sampling, and to prepare descriptive logs of the exploration. Soils were classified in general accordance with ASTM D2488, 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.

Appendix A contains the boring logs for B-1 and B-2.

3.4.2 River Bank and Channel Samples

We collected soil samples from the Little Nespelem River bank and channel for characterization and scour analysis completed by NKE. Samples were collected using hand tools approximately 25 feet upstream and 25 feet downstream of the existing bridge location.

3.4.3 Laboratory Testing

Select soil samples collected from the soil borings, and samples collected from the Little Nespelem River bank and channel, were submitted to a soil testing lab to determine the selected properties. The host of laboratory testing included moisture content, grain size
analysis, plasticity (Atterberg Limits) and organic content. The results of the geotechnical laboratory testing are provided in Appendix B.

3.5 Subsurface Conditions

Subsurface conditions at the Site were inferred from the field and laboratory investigations accomplished for this study, visual reconnaissance of the Site, and review of applicable geologic literature. More detailed soils descriptions are presented on the boring log in Appendix A. The following section presents more detailed subsurface information organized from the upper to the lower soil types.

3.5.1 Stratigraphy

In general, the descriptions from geologic maps and literature for the area corresponded with the conditions encountered in the soil borings completed for the Project. Subsurface soils can be grouped into three units consisting of fill (Af), alluvium (Qa), and glacial drift (Qgd) deposits. Bedrock was not encountered in the soil borings. Details of the composition and distribution of these units are presented in more detail below.

3.5.1.1 Fill (Af)

Fill was encountered from just below a 6-inch-thick layer of asphalt concrete to a total depth of about 7 feet below the ground surface (bgs). The fill consists of a 1.5-foot-thick layer of medium dense, brown, gravelly sand (SP)\(^1\) interpreted to be roadway base course, overlying loose to medium dense, moist, brown, slightly silty to silty sand with variable gravel content (SP-SM, SM) and loose, moist, brown, very sandy, non-plastic silt (ML). The fill was placed to construct the existing bridge approach embankments.

The SPT\(^2\) blow counts from the explorations in the fill ranged from 4 to 26 blows per foot, indicating the fill was typically loose to medium dense. The majority of the fill can generally be expected to have low to moderate shear strength, and moderate elastic compressibility.

3.5.1.2 Alluvium (Qa)

Below the fill, we encountered alluvium deposits associated with the Little Nespelem River. The alluvium deposits generally consisted of loose to medium dense, moist to wet, light brown, well-graded sand with variable silt content (SW, SW-SM) and poorly-graded sandy gravel (GP).

The SPT blow counts from the explorations in the alluvium ranged from 5 to 26 blows per foot, indicating the density of the alluvium varies from loose to medium dense. The majority of the alluvium can generally be expected to have low to moderate shear strength and moderate elastic compressibility characteristics under new loads.

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\(^1\) 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.
3.5.1.3 Glacial Drift (Qgd)

Below the alluvium, we encountered glacial drift deposits (Qgd) extending to the maximum depth explored of 76.5 feet in B-1, and 61.5 feet in B-2.

To a depth of about 25 to 30 feet in B-1, and 20 to 25 feet in B-2, the glacial drift deposits generally consisted of loose to medium dense soils described as wet, slightly silty to very silty sand with variable gravel content (SP-SM, SM), and non-plastic sandy silt (ML) with trace organic material.

We observed the glacial drift deposits consistently ranged from medium dense to very dense below a depth of about 25 to 30 feet at B-1, and 20 to 25 feet at B-2, and generally consisted of clean to silty sand with variable gravel content (SP, SP-SM, SM); non-plastic very sandy silt (ML); and silty, very sandy gravel (GP-GM). Although not encountered in the borings, it is possible that cobbles and boulders may also present in this soil unit.

Occasional layers of fine-grained cohesive soil were observed in the soil borings and were interbedded between the sand and gravel glacial drift deposits. A 5-foot-thick layer of very stiff, moist, gray, slightly sandy, silty clay (CL) was encountered from 35 to 40 feet, and a 1-foot-thick layer of hard, very moist, brown, very sandy, slightly clayey silt (ML) from 70 to 71 feet in B-1. A 3-foot-thick layer of very stiff, moist, gray clay (CL) was encountered from 47 to 50 feet in B-2.

We suspect the blow counts of SPT samples taken in the glacial drift deposits may be overstated in some cases due to the relatively high gravel content at some depths and have made note of this suspicion on the boring logs in Appendix A.

The loose to medium dense sand and gravel glacial drift deposits can generally be expected to have a moderate shear strength and moderate elastic compressibility characteristics under new loads. The medium dense to very dense sand and gravel, and very stiff to hard clay and silt glacial drift deposits can generally be expected to have a high shear strength and low elastic compressibility under new loads.

3.5.2 Groundwater

Groundwater level was observed during drilling at a depth of 10 feet bgs in B-1, and 14 feet bgs in B-2, equating to a Site groundwater elevation of about Elevation 1893 to 1895 feet. The groundwater level was below the river water level at the time of drilling, which we estimated at about Elevation 1898 feet. The groundwater level observed during drilling is likely lower than the true groundwater level, because the drilling equipment tends to impede the flow of groundwater into the borehole, and the boring was not left open long enough for the water level to stabilize. For design purposes, the groundwater level at the Site should be assumed to be at/near the river level. The groundwater level is expected to fluctuate by many feet with seasonal changes and water levels in the river.
4 Conclusions and Recommendations

4.1 General

To provide for scour protection, the base of the bridge foundations must be below the river thalweg. This would require a significant amount of excavation and dewatering within moderate to high permeability soils to construct shallow foundations to support the bridge. Therefore, we recommend the bridge be supported on deep foundations.

The following sections present the results of our studies and geotechnical engineering analyses. The recommendations are meant to support engineering design activities.

4.2 Earthquake Engineering

4.2.1 Ground Motion

The AASHTO BDS response spectrum for design is based on local seismicity and soil conditions. The seismicity is represented by the acceleration coefficient, $A_s$, which represents the peak ground acceleration (PGA) based on established seismic risk models adjusted for Site conditions.

The USGS completed regional probabilistic ground motion studies to establish the PGA for various recurrence intervals equating to 7 percent occurrence in 75 years (approximately a 975-year return period event) (USGS, 2008).

The AASHTO BDS expresses the effects of site-specific subsurface conditions on the ground motion response in terms of the Site Coefficients. The Site Coefficient accounts for the seismic response of the soil profile and is based on the density and stiffness of the soil profile underlying the site. The Soil Type can be correlated to the average standard penetration resistance (NSPT) in the upper 100 feet of the soil profile. We characterize the site as AASHTO Site Class D.

AASHTO BDS Site Coefficients for Site Class D have been utilized to adjust the mapped PGA, and spectral accelerations at periods of 0.2 ($S_s$) and 1.0 ($S_1$) seconds at the Site, as shown in Table 1.

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Recommended Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Peak Ground Acceleration ($A_s$)</td>
<td>0.17g (Site Class D)</td>
</tr>
<tr>
<td>Design Short Period Spectral Acceleration ($S_{0.2}$)</td>
<td>0.41g (Site Class D)</td>
</tr>
<tr>
<td>Design 1-Second Period Spectral Acceleration ($S_{1}$)</td>
<td>0.20g (Site Class D)</td>
</tr>
<tr>
<td>Magnitude ($M_w$)</td>
<td>6.0</td>
</tr>
</tbody>
</table>
4.2.2 Seismic Hazards

Earthquake-induced hazards that are relevant to the Site include fault rupture, soil liquefaction, and associated effects. As discussed in the following sections, neither of these represent major design considerations.

4.2.2.1 Surficial Fault Rupture

Faults that could produce surface rupture are not mapped in the Project area, are not well-defined, and are thought to have recurrence intervals in the range of one to several thousand years. The current state of engineering practice in Washington State is such that surface fault rupture is only considered in extraordinary cases with established evidence or high likelihood that a fault is present within the Project area. This situation is not the case for the Site. In our opinion, the relative risk of fault rupture at the surface of the Site is low.

4.2.2.2 Soil Liquefaction Susceptibility

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, and sand boils, any of which could result in significant 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.

We evaluated liquefaction potential based on the design seismic event shown in Table 1. Our evaluations were conducted utilizing WSlq, a program that was created as part of an extended research Project supported by the Washington State Department of Transportation (WSDOT), and liquefaction models by Youd and Idriss (2001) and Idriss and Boulanger (2008).

Our analyses indicate that the risk of liquefaction at the Site is low and is not a design consideration.

4.3 River Bank and Approach Embankments

We understand that the roadway approach embankments will be raised by less than 1 foot and will be widened using sliver fills placed atop existing embankment side slopes. Embankment settlement beneath the added fill is expected to occur quickly as the fill is placed because the underlying Site soils are comprised of sand, gravel and low-plasticity silt.

We understand the river bank slopes beneath the bridge will be on the order of 2H:1V (horizontal to vertical) in steepness with some rip rap revetment. We conclude that this slope angle will be sufficiently stable under static and seismic loading conditions, with factors of safety against global slope instability of greater than 1.5 and 1.1, respectively.

We understand roadway embankment widening and right-of-way restrictions will require side slopes as steep as 1.5H:1V that will range in height from 2 or 8 feet tall. We recommend sliver fills placed to grade existing slopes to angles steeper 2H:1V (but not
steeper than 1.5H:1V) and taller than about 3 feet be constructed of well-compacted angular crushed rock or Crushed Surfacing Base Course as defined in Section 9-03.9(3) of WSDOT Standard Specifications (M41-10) (WSDOT, 2016). We recommend existing embankment side slopes be cleared and grubbed of organic and deleterious material and notched-and-keyed along the existing slope and at the toe prior to placing the sliver fills. At a minimum, the crushed rock sliver fill should comprise the outer 3 feet of these steeper-sloped-areas and be well-compacted throughout and at the face.

4.4 Foundations Alternatives

Given the soil conditions and low seismicity, both shallow and deep foundations are geotechnically feasible. However, in consideration of potential scour depth and expected bearing-level for spread footings, we think shallow spread footing foundation preparation would require excavation below the river level. This would require dewatering of moderate to high permeability sand and gravels, and/or use of water-tight shoring.

Deep foundations would eliminate the need for large/deep excavations and construction dewatering. Driven piles are installed relatively quickly and there are practical ways to verify their capacity in the field during construction. Driven piles commonly used include steel H-piles, steel pipe piles (driven, steel-walled pipes that are in-filled with concrete and are also known as cast-in-place concrete piles), and precast, prestressed concrete piles. Given the rural Project location, vibration damage to adjacent facilities from pile driving vibration does not pose a concern.

Steel pipe piles have advantages in that they are durable, easy to splice, and if driven closed-ended, can be inspected from the interior after driving. They have also been shown to resist cyclic loads more effectively than precast, prestressed concrete piles. Precast, prestressed concrete piles are also somewhat brittle, must be handled carefully using multi-point rigging, and are difficult to splice. Precast concrete piles also require significant lead time for casting and curing. Low-displacement steel H-piles will not develop as much axial capacity compared to closed-ended pipe piles. Additionally, because of the asymmetry of H-piles, they can more easily stray from plumb when driving through dense, coarse (gravels or larger) materials compared with pipe piles.

Based on our experience, closed-ended, concrete-filled steel pipe piles would be cost effective and practical to construct at these relatively remote bridge sites and in these subsurface conditions. We recommend 18-inch-diameter, 1/2-inch wall thickness closed-end steel pipe piles. This steel pipe pile size is common and expected to be readily available. The use of 1/2-inch (rather than 3/8-inch) wall thickness pipe material will increase the likelihood of successfully advancing the piles without damage through layers of medium dense to dense sand and gravel.

4.4.1 Bridge Abutment Loads

The design concept for the new bridge includes a concrete, single-span, pile-supported bridge structure. According to NKE, the preliminary bridge design includes a 68-foot span supported by abutments on each side of the river. At the time of this report, we understand the unfactored load per abutment is about 575 kips.
4.4.2 Scour Depth

Hydraulic modeling results provided by NKE indicate scour action at the new bridge abutments could incise bank soils down as low as approximately Elevation 1894.4 feet. Axial and lateral pile resistance should be neglected through the scour zone from top of pile to scour elevation.

Pile side resistance developed over the scour zone ($R_{scour}$) should be accounted for during pile installation as it must be overcome during pile driving through the future scour zone soils. We estimate the side resistance over the scour zone, $R_{scour}$, is equal to approximately 6 kips for a driven, closed-end, 18-inch-diameter steel pipe pile.

4.4.3 Minimum Pile Penetration

We recommend that the piles be advanced to minimum tip Elevation 1872 feet at the north abutment and Elevation 1873.5 at the south abutment; about 28 feet below proposed bottom of pile cap at (Elevation 1900 and 1901.5 feet, respectively) to establish lateral fixity, considering fully-scoured conditions. Piles may need to be driven deeper than the minimum pile tip elevation to develop the required geotechnical resistance. Actual pile depths will need to be evaluated in the field during driving.

4.4.4 Driven Pile Axial Resistance

Axial pile resistance analyses were completed for a driven, closed-end, 18-inch-diameter, steel pipe pile in accordance with AASHTO BDS guidelines. The analyses were performed using the Federal Highway Administration (FHWA, 2007) Driven Analysis Program, using the soil conditions observed in geotechnical boring B-1 and B-2, and our engineering judgement.

The results of our axial resistance analyses are nominal (ultimate) resistances for both bearing (compression) and uplift (tension) resistances for a single pile or pile groups with a minimum center-to-center pile spacing of $2.5B$, where $B$ is pile diameter. The estimated nominal resistance is a sum of the frictional resistance along the side of the pile and the end resistance (considering fully-scoured conditions) and is shown on Figure 3, Driven Pile Axial Resistance.

The applicable Resistance Factors corresponding to the strength, service, and extreme limit states are shown in Table 2 and can be used in conjunction with Figure 3 to determine estimated strength, service, and extreme geotechnical resistances at various pile embedment depths. Pile embedment was assumed to begin at about Elevation 1901 feet. Pile skin resistance was neglected through the scour zone extending down to Elevation 1894.4 feet.

It is important to recognize that the nominal resistances shown on Figure 3 are estimates based on static analysis methods from geotechnical borings and AASHTO BDS methodologies. It is possible that soil conditions may vary locally at pile locations. Resistances should be confirmed by field observations made during driving as discussed in Section 5.3, Geotechnical Monitoring of Driven Piles.
### Table 2 – Resistance Factors for Driven Pile Design

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Resistance Factor, $\phi$</th>
<th>Bearing Resistance, $\phi_{\text{stat}}$(^{(1)})</th>
<th>Bearing Resistance, $\phi_{\text{dyn}}$(^{(2)}), Uplift, $\phi_{\text{up}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength</td>
<td>0.45</td>
<td>0.50(^{(3)})/0.55(^{(4)})</td>
<td>0.35</td>
</tr>
<tr>
<td>Service</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**Notes:**

1. Applies to nominal resistance as determined by static analysis methods (see Figure 3).
2. Applies to nominal resistance as determined by dynamic analysis methods during pile driving.
3. Assumes wave equation analysis without pile dynamic measurements or load test but with field confirmation of hammer performance.
4. Assumes the WSDOT driving formula will be used as the basis for the dynamic analysis and pile driving construction control.

#### 4.4.5 Driven Pile Lateral Resistance

Driven pile lateral resistance is developed from the material stiffness of the pile itself and the embedment soils. The magnitude of resistance depends on the type of center-to-center pile spacing, pile head fixity condition, and tolerable deflections. We analyzed a concrete-filled 18-inch-diameter pipe pile using the computer program LPILE (Ensoft, 2013). From NKE, we understand the center-to-center spacing pile spacing may be as close as 6 feet, equal to 4B, where B is pile diameter.

We considered the steel pile wall thickness to be reduced by from 1/2-inch to 3/8-inch steel to account for possible long-term corrosion. Per NKE, the pile core was modeled as being in-filled with 4,000 psi structural concrete with six, #6 size steel vertical reinforcing bars extending from the pile head to a depth of 25 feet. Pile head deflections of ¼- and 2-inches, and fixed- and free-pile head conditions were considered. The embedment soils were modeled in LPILE using the parameters shown in Table 3 below. Lateral resistance over the scour zone was neglected.

Based on our analysis, we conclude the piles should be driven to minimum tip Elevation of 1872 feet at the north abutment and 1873.5 feet at the south abutment to establish a reasonable degree of pile lateral fixity.
Table 3 – Lateral Pile Analysis Soil Parameters

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil Type</th>
<th>Top of Layer Elevation (ft)</th>
<th>Bottom of Layer Elevation (ft)</th>
<th>Effective Unit Weight, $\gamma'$ (pcf)</th>
<th>Internal Friction Angle, $\phi$ (degrees)</th>
<th>Cohesion (psf)</th>
<th>Strain at 50%, $\varepsilon_{50}$</th>
<th>P-y Modulus, (pci)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sand (API)</td>
<td>1895</td>
<td>1880</td>
<td>62</td>
<td>32</td>
<td>0</td>
<td>Not Applicable</td>
<td>70</td>
</tr>
<tr>
<td>2</td>
<td>Sand (API)</td>
<td>1880</td>
<td>1875</td>
<td>62</td>
<td>34</td>
<td>0</td>
<td>Not Applicable</td>
<td>80</td>
</tr>
<tr>
<td>3</td>
<td>Sand (API)</td>
<td>1875</td>
<td>1870</td>
<td>67</td>
<td>38</td>
<td>0</td>
<td>Not Applicable</td>
<td>140</td>
</tr>
<tr>
<td>4</td>
<td>Stiff Clay w/Free Water</td>
<td>1870</td>
<td>1865</td>
<td>52</td>
<td>0</td>
<td>3,000</td>
<td>0.005</td>
<td>1,000</td>
</tr>
<tr>
<td>5</td>
<td>Sand (API)</td>
<td>1875</td>
<td>1825</td>
<td>67</td>
<td>38</td>
<td>0</td>
<td>Not Applicable</td>
<td>140</td>
</tr>
</tbody>
</table>

Pile group interaction effects were taken into account by applying a P-multiplier value ($P_m$) of 0.90 per Table 10.7.2.4-1 of the AASHTO BDS (AASHTO, 2014), reflecting pile spacing of about 4B. The results of the lateral pile analysis are shown on Figures 4, 5, and 6 plotting pile elevation versus deflection, shear resistance, and moment, respectively, for this proposed spacing. The plots are provided for normal (static) conditions and are nominal resistances (unfactored).

4.4.6 Drivability

In our opinion, the Site is suitable for pile driving. Once a contractor has been selected for pile installation a wave equation analysis should be completed and submitted by the contractor to determine if the contractor’s proposed pile driving system is capable of driving the piles in accordance with WSDOT Standard Specifications Section 6-05.3. Based on a preliminary drivability analysis, we recommend an open-ended diesel pile driving hammer with a minimum rated energy of 45,000 to 50,000 foot-pounds. Because the piles will be advanced into very dense sand and gravel, the contractor should anticipate difficult/hard driving to achieve minimum tip elevation.

At a minimum, we recommend that driven pile resistance be confirmed using the WSDOT pile driving formula shown in WSDOT Standard Specifications Section 6-05.3.

4.5 Bridge Abutment and Wingwall Design

4.5.1 Backfill and Drainage Material

Measures should be taken to prevent buildup of hydrostatic pressure behind abutments and wingwalls. Abutment and wingwall backfill materials should consist of material meeting the requirements of Gravel Backfill for Walls as specified in Section 9-03.12(2) of the WSDOT Standard Specifications. Placement and compaction of fill behind walls shall be in accordance with Section 6-13.3(7) of the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction.
Over-compaction of the backfill behind walls should be avoided. We recommend compacting backfill behind walls to approximately 95 percent of MDD as determined in accordance with WSDOT Standard Specifications Section 2-03.3(14), except within a few feet of the back of the walls the compaction requirement may be reduced to 92 percent of MDD. Heavy compactors and large pieces of construction equipment should not operate within several 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.

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

4.5.2 Lateral Earth Pressures

Lateral earth pressures acting behind the abutment and wingwalls are presented in Table 4 below and represent the active, at-rest, passive, and seismic.

<table>
<thead>
<tr>
<th>Earth Pressure Condition</th>
<th>Earth Pressure Coefficient</th>
<th>Equivalent Fluid Weight(^{(1)}) (pcf)</th>
<th>Earth Pressure(^{(2)}) (psf)</th>
<th>Surcharge Pressure (psf)</th>
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<tr>
<td>Active (K(_a))(^{(3)})</td>
<td>0.28</td>
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<td>55H</td>
<td>0.44S(^{(8)})</td>
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<td>450(^{(5)})</td>
<td>450D(^{(5),(6),(7)})</td>
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<td>-</td>
<td>-</td>
<td>7.0 H</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes:

1. Granular backfill placed as compacted structural fill with a unit weight of 125 pcf is assumed.
2. Static earth pressures result in a triangular pressure distribution along the height of the wall. Seismic earth pressures result in a uniform rectangular pressure distribution along the height of the wall (H) with the resultant acting at a height of 0.5H above base of the wall.
3. To invoke the active conditions, the wall must rotate about the base with a lateral movement at the top of the wall of approximately 0.002H, where H is the height of the wall.
4. To invoke the passive conditions, the wall must move into the backfill with a lateral movement of approximately 0.020H.
5. Ultimate 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.
7. Passive pressure should be ignored within 24 inches below finish grade.
8. Resulting uniform surcharge acting along the height of the wall, where S is the surcharge pressure. Seismic and surcharge pressures are typically not considered concurrently in design unless specific conditions dictate otherwise.
9. The seismic pressures were calculated in accordance with Chapter 11 and Appendix A11.1.1.1 of the AASHTO BDS where the wall is capable of displacements of 1.0 to 2.0 inches of more during the seismic event using the design earthquake parameters shown in Tables 2 and 3.
10. The at-rest seismic pressure assumes wall is not free to translate or move during the seismic event.
4.6 Earthwork

Based on the explorations performed on Site and our understanding of the Project, it is our opinion that the contractor should be able to complete Site earthwork with standard construction equipment.

Appropriate erosion and sedimentation control measures should be in accordance with the local best management practices (BMPs) and should be implemented prior to beginning earthwork activities.

Cobbles and boulders may be encountered during construction and the contractor should be prepared to deal with them.

4.6.1 Temporary Excavation 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.

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 if precipitation is expected.

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.

4.6.2 Structural Fill and Compaction

We estimate much of the materials excavated for the Project may be suitable as structural fill. On-site soils may be made into suitable structural fill by moisture conditioning to near the optimum moisture content. Excavated material should be visually inspected by the geotechnical engineer to determine its potential use as backfill. Excavated material that is unsuitable as structural fill may be suitable as backfill for unimproved areas that are not susceptible to differential settlement over time.

In general, suitable structural fill material for the Project is fill placed within 3 percent of its optimum moisture content per Standard Specifications Section 2-03.3(14) and does not contain deleterious materials, or greater than 5 percent organics.

On-site or import material for the roadway fill embankment should be in general conformance with Section 9-03.14(3), Common Borrow, of the WSDOT Standard Specifications. In wet weather conditions, or situations requiring a more free-draining backfill, Gravel Borrow in accordance with Section 9-03.14(1) of the WSDOT Standard Specifications should be imported for use as fill.

The outer edges of relatively steeper sloped portions (steeper than 2H:1V, but not steeper than 1.5H:1V) of the roadway embankment fill should consist of compacted angular
crushed rock or Crushed Surfacing Base Course as defined in Section 9-03.9(3) of WSDOT Standard Specifications (M41-10) (WSDOT, 2016), as discussed in Section 4.3.

Structural fill for roadway embankments and around abutments and wingwalls (except for within a few feet of the back of the walls) should be compacted to at least 95-percent of the maximum dry density of the material as determined by ASTM D1557 (modified Proctor). Structural fill within a few feet of the back of abutments and wingwalls should be compacted to at least 92-percent of the maximum dry density of the material as determined by ASTM D1557 (modified Proctor).

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.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with a high percentage of silt or clay are particularly susceptible to becoming too wet, and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.
5 Construction Considerations

5.1 General

All pile 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.

5.2 Driven Pile Installation

In general, pile driving construction should follow the guidelines set forth in WSDOT Standard Specifications Section 6-05.

Installation of piles may be impacted by the potential presence of obstructions (such as boulders). 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.

5.3 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.

We recommend the contract include the requirement that one production pile per abutment be driven as 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 at each abutment.

5.4 Site Subsurface Variation and Pile Lengths

Local variations of subsurface conditions along the bridge alignment should be expected. The lengths of certain piles may need to be adjusted in the field based on conditions encountered during driving. Variable pile lengths should be anticipated, and contingency provisions should be included in the contract documents to facilitate adjustment in payments to the contractor based on actual lengths of piles installed.
6 References

American Association of State Highway and Transportation Officials (AASHTO), 2014, LRFD Bridge Design Specifications, Customary U.S. Units.


Ensoft, Inc., 2013, LPILE plus v7.02 for Windows Analysis program.


U.S. Department of Transportation Federal Highway Administration (FHWA), 2007, Driven v1.2 Analysis program.

Washington State Department of Transportation (WSDOT), 2016, Standard Specifications for Road, Bridge and Municipal Construction, Document M 41-10.


7 Limitations

Work for this Project was performed for Nicholls Kovich Engineering, PLLC (Client), and this report was 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 are intended solely for the Client and apply only to the services described in the Agreement with Client. Any use or reuse by Client for purposes outside of the scope of Client’s Agreement is at the sole risk of Client and without liability to Aspect Consulting. Aspect Consulting shall not be liable for any third parties’ use of the deliverables provided by 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
Site and Exploration Map
Bridge B-15 Moses Road
Okanogan County, Washington

Legend

Boring Location

Source: Base map provided by Nicholls Kovich Engineering, PLLC.
Resistances for 18” OD closed-ended steel pipe pile

- Pile head assumed at Elevation 1901 feet.
- Scour Elevation assumed at Elevation 1894.4 feet.
Figure 4
Lateral Pile Deflection Profile
Bridge B-15 Moses Road
Project No. 150364, Okanogan County, WA
Figure 5
Lateral Pile Shear Profile
Bridge B-15 Moses Road
Project No. 150364, Okanogan County, WA
Figure 6
Lateral Pile Moment Profile
Bridge B-15 Moses Road
Project No. 150364, Okanogan County, WA

1/4-inch Head Deflection, Fixed Head Condition
2-inch Head Deflection, Fixed Head Condition
1/4-inch Head Deflection, Free Head Condition
2-inch Head Deflection, Free Head Condition
APPENDIX A

Subsurface Explorations
A.1 Field Exploration Program

A.1.1 Geotechnical Borings

On January 12 and 13, 2016, we performed a site reconnaissance and completed two geotechnical soil borings, B-1 and B-2, to a depth of 76.5 and 61.5 feet below the existing ground surface, respectively, using mud-rotary auger drilling techniques. B-1 was completed on the northwest side of the existing bridge using a truck-mounted rotary drill rig. B-2 was completed on the southeast side of the bridge using a relatively lightweight D25 Skid Rotary drill rig to cross over the existing bridge with that currently has a 5-ton load restriction. The location of the borings are shown on Figure 2.

Sampling was completed at selected depth intervals using the Standard Penetration Test (SPT) in general accordance with ASTM method D1586. 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. Sampling in B-1 was completed using an automatic-trip hammer. Sampling in B-2 was completed using rope-and-cathead manual hammer. A 3-inch-diameter thin-wall Shelby tube soil sample was also collected at a depth of 45 to 46 feet in B-2.

An Aspect Consulting representative 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.

A.1.2 Bank and Channel Soil Samples

Four bulk soil samples were collected from the bank and channel of the river about 25 feet upstream and 25 feet downstream of the bridge location using hand tools. The soil samples were submitted to a geotechnical testing laboratory for grain size analysis. The grain size analysis results are shown in Appendix B.
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.

### Exploration Log Key

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<tr>
<th>Component Definitions</th>
<th>Size Range and Sieve Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse-Grained Soils</td>
<td>More than 50% retained on No. 200 sieve</td>
</tr>
<tr>
<td>Fine-Grained Soils</td>
<td>50% or more pass No. 200 sieve</td>
</tr>
</tbody>
</table>

### Estimated Percentage

**Percentage by Weight**: 
- >30
- 20 to 30
- 15 to 20
- 10 to 15
- 5 to 10
- 2 to 5
- <2

**Modifier**: 
- Trace
- Slightly (sandy, silty, gravelly)
- Moist (sandy, silty, gravelly)
- Very (sandy, silty, gravelly)

### Moisture Content

**Dry** - Absence of moisture, dusty 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

- **Cement grout**
- **Bentonite chips**
- **Grout seal**
- **Filter pack with blank casing section**
- **Screened casing or Hydrotip with filter pack**
- **End cap**

### Density and Consistency

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<th>Type</th>
<th>Description</th>
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<td><strong>SPT</strong></td>
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<td><strong>Continuous Push</strong></td>
<td>Non-Standard Sampler</td>
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<td><strong>3.0&quot; OD Thin-Wall Tube Sampler (including Shelby tube)</strong></td>
<td>Grab Sample</td>
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<tr>
<td><strong>Portion not recovered</strong></td>
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</table>

### Miscellaneous

- **Grouted**
- **Transducer**
- **Detector**
- **SPT** | Standard Penetration Test (ASTM D-1586)
- **A** | At time of drilling
- **BGS** | Below ground surface
- **ATD** | Estimated in General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
- **C** | Consolidation
- **C** | Permeability
- **K** | Shear Strength
- **M** | Moisture Content
- **G** | Grain Size
- **A** | Atterberg Limits
- **FC** | Fines Content

### Exploration Log Key

![Exploration Log Key A1.dwg](Q:\_ACAD Standards\Standard Details\Exploration Log Key A1.dwg)
Bridge B-15 Joe Moses Road - 150364

Project Address & Site Specific Location
Moses Road Nespelem, WA 99155, North of bridge deck, 10' offset, west shoulder of roadway

Exploration Method(s)
Mud rotary

Sampling Method
Autohammer; 140 lb hammer; 30" drop

Operator
Aaron

Exploration Completion and Notes
Backfilled with bentonite chips (base course slough used for top 2'). Exploration completed with asphalt patch.

Geotechnical Exploration Log

Coordinates (Lat, Lon WGS84) 48.139, -118.929 (est.)

Exploration Number
B-1

Ground Surface (GS) Elev.
1905'

Top of Casing Elev.
NA

Depth to Water (Below GS)
10' (ATD)

Depth
Material
Description

1
Asphalt; 6 inches

2
FILL
Medium dense, moist, brown, gravelly SAND (SP); base course.

Loose, brown, gravelly, silty SAND (SM).

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Medium dense, moist to wet, light brown, very gravelly, slightly silty SAND (SW-SM); fine to coarse sand, subrounded to subangular gravel.

Loose, wet, brown and light brown, sandy GRAVEL (GP); predominantly coarse sand, subangular gravel.

Grades to medium dense. No recovery.

GLACIAL DRIFT
Medium dense, wet, gray, gravelly, silty SAND (SM).

Medium dense, wet, gray, gravelly, slightly silty SAND (SP-SM); fine to coarse sand, subrounded to subangular gravel.

Medium dense, wet, gray, very silty SAND (SM); predominantly fine sand, trace subangular to angular gravel, rare organics.
Blow count may be overstated due to gravel blocking shoe.

Grades to dense, predominantly fine to medium sand.

Grades to fine sand.

Hard, very moist to wet, brown, very sandy, slightly clayey SILT (ML); medium plasticity.

Very dense, wet, gray SAND (SP); predominantly fine sand.

Driller reported gravel based on drill action from 71' - 72' BGS.

---

**Geotechnical Exploration Log**

**Bridge B-15 Joe Moses Road - 150364**

**Project Address & Site Specific Location**
Moses Road, Nespelem, WA 99155, North of bridge deck, 10’ offset, west shoulder of roadway

**Coordinates (Lat, Lon WGS84)**
48.139, -118.929 (est.)

**Exploration Number**
B-1

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<th>Blows/foot</th>
<th>Water Content (%)</th>
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**Sample Method**
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)

**Equipment**
- Truck-mounted Rotary Drill Rig
- Autohammer; 140 lb hammer; 30" drop

**Exploration Method(s)**
- Mud rotary

**Work Start/Completion Dates**
- 1/12/2016

**Top of Casing Elev.**
- NA

**Depth to Water (Below GS)**
- 10' (ATD)

**Water Level (ATD)**
- 10' (ATD)

---

**Reported by:** SJA

---

**Logging Check**
- SJA

---

**Exploration Log**
- B-1

**Sheet 3 of 4**

---

**See Exploration Log Key for explanation of symbols**

---

**Logged by:** SJA
**Approved by:** NCS
Bridge B-15 Joe Moses Road - 150364

Moses Road Nespelem, WA 99155, North of bridge deck, 10' offset, west shoulder of roadway

Geotechnical Exploration Log

Coordinates (Lat,Lon WGS84)

48.139, -118.929 (est.)

Exploration Number

B-1

Hazard-Tech Drilling

Operator

Aaron

Sampling Method

Autohammer; 140 lb hammer; 30" drop

Top of Casing Elev.

NA

Depth to Water (Below GS)

10' (ATD)

Exploration Completion and Notes

B-1

10 20 30 40 50

Water Content (%)

14 20 21

Tests

Material Type

Description

Becomes dense.

Bottom of exploration at 76.5 ft. bgs.

See Exploration Log Key for explanation of symbols

Logged by: SJA

Approved by: NCS

Exploration log

B-1

Sheet 4 of 4
Bridge B-15 Joe Moses Road - 150364
Moses Road Nespelem, WA 99155, South of bridge deck, 15' offset, east shoulder of roadway

Geotechnical Exploration Log

Project Address & Site Specific Location

Groundwater measured downhole by driller.

Backfilled with bentonite chips (base course slough used for top 2'). Exploration completed with asphalt patch.

Blow count likely overstated due to gravel content.

Coordinates (Lat,Long WGS84): 48.139, -118.929 (est.)

Exploration Number

FILL
Medium dense, moist, gray and brown, gravelly SAND (SP); base course.

Medium dense, moist, brown, slightly gravelly, silty SAND (SM); predominantly fine to medium sand, subrounded gravel.

Loose, moist, brown, very sandy SILT (ML); predominantly fine sand, rare organics, trace gravel.

ALLUVIUM
Medium dense, moist, light brown, gravelly SAND (SW); fine to coarse sand, predominantly fine gravel, trace silt.

GLACIAL DRIFT
Loose, very moist, dark gray, sandy SILT (ML); fine sand, trace gravel, trace organics (2% by mass).

Loose, wet, dark gray, slightly gravelly, very silty SAND (SM); fine to medium sand, predominantly fine sand, fine gravel.

Very dense, wet, gray, very sandy, slightly silty GRAVEL (GP-GM); fine to coarse sand, fine gravel.

See Exploration Log Key for explanation of symbols

Logged by: SJA
Approved by: NCS

Exploration log
B-2
Sheet 1 of 3
**Bridge B-15 Joe Moses Road - 150364**

**Moses Road, Nespelem, WA 99155, South of bridge deck, 15' offset, east shoulder of roadway**

### Geotechnical Exploration Log

- **Operation:** Haz-Tech Drilling
- **Equipment:** D25 Skid Rotary Drill Rig
- **Sampling Method:** Cathead; 140 lb hammer; 30° drop
- **Operator:** Aaron
- **Exploration Method(s):** Mud rotary
- **Work Start/Completion Dates:** 1/13/2016
- **Ground Surface (GS) Elev.:** 1907'
- **B-2**
- **Depth to Water (Below GS):** 14' (ATD)

### Exploration Completion and Notes

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---

**Legend:****
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)
- Thin wall 3" (Shelby)

---

**See Exploration Log Key for explanation of symbols**

- **Logged by:** SJA
- **Approved by:** NCS

---

**Exploration log**

**Sheet 2 of 3**
Very dense, wet, gray, slightly silty SAND (SP-SM); fine to medium sand, predominantly fine sand.

Very dense, wet, gray, very sandy SILT (ML) interbedded with wet, gray, very silty SAND (SM); predominantly fine sand.

Very dense, wet, gray, slightly silty SAND (SP-SM) interbedded with laminae of hard, gray, clayey SILT (ML); fine to coarse sand.

Bottom of exploration at 61.5 ft. bgs.
APPENDIX B

Laboratory Test Results
B.1 Geotechnical Laboratory Testing

Geotechnical laboratory tests were conducted on selected soil samples collected during the field exploration program. The tests performed and the procedures followed are outlined below.

**Grain Size Analysis (G)**

Grain size analysis was analyzed in accordance with ASTM D422 on SPT soil samples collected from B-1 and B-2, and stream bank and channel grab samples. The results of the tests are presented as curves in this appendix plotting percent finer by weight versus grain size.

**Water Content Determination**

Water contents were determined in accordance with ASTM D2216 on SPT soil samples collected from B-1 and B-2, and stream bank and channel grab samples. The results of the tests are presented on the boring logs shown in Appendix A.

**Atterberg Limits**

Atterberg Limits (plasticity) were determined in accordance with ASTM D4318 on two soil samples (S-10 and S-11a) collected from B-1, and one (S-14b) collected from B-2. The test results are shown on the boring logs in Appendix A and this in appendix.

**Organic Content Determination**

Organic content determination was completed in accordance AASHTO T276 on one SPT soil sample (S-5) collect from soil boring B-2. The results of the test are shown in this appendix.
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MATERIALS TESTING & SPECIAL INSPECTION
104 East Ninth Street
Wenatchee, WA 98801
(509) 664-4843

STANDARD MECHANICAL SIEVE  ASTM C-136 or ASTM D-422

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DESCRIPTION: Silty sand with gravel

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<td>81%</td>
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TOTAL: 286.7

FIELD MOISTURE: 12.5%

REMARKS: 

TECHNICIAN: D. Nyland    PROJ. MGR. J. HILLS

Note: All sample material will be discarded after 30 days of receipt unless otherwise notified.
CSI: Construction Special Inspection

MATERIALS TESTING & SPECIAL INSPECTION

104 East Ninth Street
Wenatchee, WA 98801
(509) 664-4843

STANDARD MECHANICAL SIEVE  ASTM C-136 or ASTM D-422

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DESCRIPTION: Well graded sand with silt and gravel

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TOTAL: 271.8

FIELD MOISTURE: 11.1%

REMARKS:

TECHNICIAN: D. Nyland  PROJ. MGR. J. HILLS

Note: All sample material will be discarded after 30 days of receipt unless otherwise notified.
# STANDARD MECHANICAL SIEVE ASTM C-136 or ASTM D-422

**DESCRIPTION:** Silty Sand

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**TOTAL** 307.8

**FIELD MOISTURE:** 16.7%

**REMARKS:**

**TECHNICIAN:** D. Nyland  
**PROJ. MGR.** J.HILLS

Note: All sample material will be discarded after 30 days of receipt unless otherwise notified.
**STANDARD MECHANICAL SIEVE ASTM C-136 or ASTM D-422**

- **CLIENT:** Aspect Consulting
- **PROJECT NO:** 16-3
- **PROJECT:** Joe Moses Rd
- **LAB NO:** 16-2991
- **DATE RCVD:** 1/18/2016
- **DATE TESTED:** 1/19/2016
- **SUBMITTED BY:** Nick S
- **LOCATION:** B-2, S-2
- **SAMPLE DEPTH:** 5.0'

**DESCRIPTION:** Sandy Silt

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**TOTAL:** 392.4

**FIELD MOISTURE:** 25.5%

**REMARKS:**

**TECHNICIAN:** D. Nyland
**PROJ. MGR:** J. HILLS

*Note: All sample material will be discarded after 30 days of receipt unless otherwise notified.*
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MATERIALS TESTING & SPECIAL INSPECTION
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Wenatchee, WA 98801
(509) 664-4843

STANDARD MECHANICAL SIEVE ASTM C-136 or ASTM D-422

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DESCRIPTION: silty sand

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TOTAL: 573.3

FIELD MOISTURE: 21.7%

REMARKS: __________________________________________

TECHNICIAN: D. Nyland                       PROJ. MGR. J. HILLS

Note: All sample material will be discarded after 30 days of receipt unless otherwise notified.

StdSieve 9-1-11
LIQUID AND PLASTIC LIMITS TEST REPORT

Dashed line indicates the approximate upper limit boundary for natural soils.

SOIL DATA

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<td>•</td>
<td>B-1 S-10 35' 7819-A</td>
<td>35'</td>
<td>32.2</td>
<td>17</td>
<td>24</td>
<td>7</td>
<td>CL</td>
<td></td>
</tr>
</tbody>
</table>

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting LLC
Project: Bridge B-15 Moses Road

Project No.: 150364-02/08-175

Figure

Tested By: B.H
Checked By: JAM
LIQUID AND PLASTIC LIMIT TEST DATA

Client: Aspect Consulting LLC
Project: Bridge B-15 Mosas Road
Project Number: 150364-02/08-175
Location: B-1 S-10
Depth: 35’
Material Description: B-1 S-10 35’
USCS: CL
Sample Number: 7819-A

Tested by: B.H
Checked by: JAM

<table>
<thead>
<tr>
<th>Run No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
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<tbody>
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<td>Wet+Tare</td>
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<tr>
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<td>26</td>
<td>21</td>
<td>18</td>
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<td></td>
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</tr>
<tr>
<td>Moisture</td>
<td>24.4</td>
<td>24.5</td>
<td>24.9</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
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Liquid Limit = 24
Plastic Limit = 17
Plasticity Index = 7
Natural Moisture = 32.2
Liquidity Index = 2.2

Plastic Limit Data

<table>
<thead>
<tr>
<th>Run No.</th>
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</thead>
<tbody>
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<td>Tare</td>
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<tr>
<td>Moisture</td>
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Natural Moisture Data

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<thead>
<tr>
<th>Wet+Tare</th>
<th>Dry+Tare</th>
<th>Tare</th>
<th>Moisture</th>
</tr>
</thead>
<tbody>
<tr>
<td>565.2</td>
<td>431.3</td>
<td>15.9</td>
<td>32.2</td>
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</table>

Hayre McElroy & Associates, LLC
LIQUID AND PLASTIC LIMITS TEST REPORT

Dashed line indicates the approximate upper limit boundary for natural soils

SOIL DATA

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>SOURCE</th>
<th>SAMPLE NO.</th>
<th>DEPTH</th>
<th>NATURAL WATER CONTENT (%)</th>
<th>PLASTIC LIMIT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>PLASTICITY INDEX (%)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 S-11a 40'-40.5'</td>
<td>7819-B</td>
<td>40'-40.5'</td>
<td>33.9</td>
<td>16</td>
<td>33</td>
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</tbody>
</table>

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting LLC
Project: Bridge B-15 Moses Road
Project No.: 150364-02/08-175

Tested By: B.H
Checked By: JAM
**LIQUID AND PLASTIC LIMIT TEST DATA**

Client: Aspect Consulting LLC  
Project: Bridge B-15 Moses Road  
Project Number: 150364-02/08-175  
Location: B-1 S-11a  
Depth: 40'-40.5'  
Material Description: B-1 S-11a 40'-40.5'  
USCS: CL  
Sample Number: 7819-B  
Tested by: B.H  
Checked by: JAM

### Liquid Limit Data

<table>
<thead>
<tr>
<th>Run No.</th>
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<th>4</th>
<th>5</th>
<th>6</th>
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</thead>
<tbody>
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<td>Wet+Tare</td>
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<tr>
<td>Tare</td>
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<tr>
<td># Blows</td>
<td>30</td>
<td>26</td>
<td>21</td>
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</tr>
<tr>
<td>Moisture</td>
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<td></td>
</tr>
</tbody>
</table>

Liquid Limit = 33  
Plastic Limit = 16  
Plasticity Index = 17  
Natural Moisture = 33.9  
Liquidity Index = 1.1

### Plastic Limit Data

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<thead>
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<th>Run No.</th>
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</thead>
<tbody>
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<td>24.48</td>
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<td>Dry+Tare</td>
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<tr>
<td>Tare</td>
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<td>Moisture</td>
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### Natural Moisture Data

<table>
<thead>
<tr>
<th>Wet+Tare</th>
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<th>Tare</th>
<th>Moisture</th>
</tr>
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<tbody>
<tr>
<td>303.6</td>
<td>230.8</td>
<td>16.0</td>
<td>33.9</td>
</tr>
</tbody>
</table>

Hayre McElroy & Associates, LLC
LIQUID AND PLASTIC LIMITS TEST REPORT

Dashed line indicates the approximate upper limit boundary for natural soils.

SOIL DATA

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>SOURCE</th>
<th>SAMPLE NO.</th>
<th>DEPTH</th>
<th>NATURAL WATER CONTENT (%)</th>
<th>PLASTIC LIMIT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>PLASTICITY INDEX (%)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>•</td>
<td>B-2 S-14b 47'-47.5'</td>
<td>7819-C</td>
<td>47'-47.5'</td>
<td>36.6</td>
<td>20</td>
<td>43</td>
<td>23</td>
<td>CL</td>
</tr>
</tbody>
</table>

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting LLC
Project: Bridge B-15 Moses Road
Project No.: 150364-02/08-175

Tested By: B.H
Checked By: JAM
**LIQUID AND PLASTIC LIMIT TEST DATA**

Client: Aspect Consulting LLC  
Project: Bridge B-15 Moses Road  
Project Number: 150364-02/08-175  
Location: B-2 S-14b  
Depth: 47'-47.5'  
Material Description: B-2 S-14b 47'-47.5'  
USCS: CL  
Sample Number: 7819-C  
Tested by: B.H  
Checked by: JAM

### Liquid Limit Data

<table>
<thead>
<tr>
<th>Run No.</th>
<th>1</th>
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<th>6</th>
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</thead>
<tbody>
<tr>
<td>Wet+Tare</td>
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<tr>
<td>Tare</td>
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<td>13.69</td>
<td>13.71</td>
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<tr>
<td># Blows</td>
<td>30</td>
<td>25</td>
<td>21</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moisture</td>
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<td>43.5</td>
<td>43.5</td>
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</tr>
</tbody>
</table>

Liquid Limit = 43  
Plastic Limit = 20  
Plasticity Index = 23  
Natural Moisture = 36.6  
Liquidity Index = 0.7

### Plastic Limit Data

<table>
<thead>
<tr>
<th>Run No.</th>
<th>1</th>
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<th>3</th>
<th>4</th>
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</thead>
<tbody>
<tr>
<td>Wet+Tare</td>
<td>23.69</td>
<td>23.69</td>
<td>23.69</td>
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<td>Dry+Tare</td>
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<td>22.00</td>
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<tr>
<td>Tare</td>
<td>13.69</td>
<td>13.69</td>
<td>13.69</td>
<td></td>
</tr>
<tr>
<td>Moisture</td>
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<td>20.3</td>
<td>20.3</td>
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</tbody>
</table>

### Natural Moisture Data

<table>
<thead>
<tr>
<th>Wet+Tare</th>
<th>Dry+Tare</th>
<th>Tare</th>
<th>Moisture</th>
</tr>
</thead>
<tbody>
<tr>
<td>301.2</td>
<td>224.8</td>
<td>16.0</td>
<td>36.6</td>
</tr>
</tbody>
</table>

---

Hayre McElroy & Associates, LLC
SOIL DISCRITION: Clayey sand

A = mass of crucible and oven-dried soil, before ignition: 174.39
B = mass of crucible and oven-dried soil, after ignition: 173.79
C = mass of crucible: 144.69

Percent Organic Matter = \( \frac{A - B}{A - C} \times 100 \) = 2

Sample in ignition furnace at: 9:15
Sample out of ignition furnace at: 3:15

REMARKS: Oven Temp. Set at 455 C

TECHNICIAN: D. Nyland
PROJECT MGR: J. Hills

Note: All sample material will be discarded after 30 days of receipt unless otherwise notified.