Foundation Report

Thomas Creek, Richardson Gap Road
(Shimanek) Covered Bridge
County Bridge No. BR0637-0070
ODOT Bridge No. 12965, Key No. 20314

Linn County, Oregon

Prepared for:

Linn County Road Department
Albany, Oregon

February 12, 2019

Professional
Geotechnical
Services

Foundation Engineering, Inc.

Foundation Engineering, Inc.
Professional Geotechnical Services

Kevin Groom, P.E.
Linn County Road Department
3010 Ferry Street SW
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February 13, 2019

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(Shimanek) Covered Bridge
County Bridge No. BR0637-0070
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Project 2181118

Dear Mr. Groom:

We have completed the requested geotechnical investigation for the proposed rehabilitation of the Thomas Creek, Richardson Gap Road (Shimanek) Covered Bridge in Linn County, Oregon. Our report includes a description of our work, a discussion of the site conditions, a summary of laboratory testing, and a discussion of engineering analyses. Recommendations are included for site preparation, bridge foundation design, and new approach pavement design.

This report was prepared to conform to the Oregon Department of Transportation (ODOT) Geotechnical Design Manual (GDM) (2018) and the ODOT Pavement Design Guide (2019). Construction recommendations refer to sections in the Oregon Standard Specifications for Construction (2018).

It has been a pleasure assisting you with this phase of your project. Please do not hesitate to contact us if you have any questions or require further assistance.

Sincerely,

FOUNDATION ENGINEERING, INC.

Matt Mason

Matthew D. Mason, P.E.
Project Engineer

William L. Nickels, Jr., P.E., G.E.
President

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MAY 17, 1998
WILLIAM L. NICKELS JR.

EXPIRES: 12/31/20

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FOUNDATION REPORT

THOMAS CREEK, RICHARDSON GAP ROAD
(SHIMANEK) COVERED BRIDGE
LINN COUNTY, OREGON

1.0. INTRODUCTION

1.1. Project Description

The Linn County Road Department (Linn County) is planning to rehabilitate the Shimanek Covered Bridge crossing Thomas Creek at Milepost (MP) 0.70 on Richardson Gap Road (County Road No. 637) in Linn County. The bridge location is ±2.3 miles northeast of Scio and is shown on Figure 1A (Appendix A).

The existing 225-foot long, four-span bridge was constructed in 1966 and needs significant structural rehabilitation on the superstructure and the substructure. The main span is a 130.5-foot long, historic covered bridge. The south approach is a single-span, concrete slab section and the north approach is a two-span, prestressed, concrete slab section. The main span is supported on cast-in-place concrete piers (Bents 2 and 3), and the north interior bent and the abutments are supported on timber piling. Foundation rehabilitation will include replacing the foundations at all bents and providing scour protection at Bent 2 and Bent 3 using riprap.

Linn County is the project owner. Foundation Engineering, Inc. was retained by Linn County as the geotechnical consultant. Our scope of work was summarized in Exhibit A of the Engineering and Related Services Contract, County Project No. CB1803.

1.2. Purpose and Scope

The purpose of the investigation was to develop recommendations for the design and construction of repair foundations and new approaches. The scope of the geotechnical work included exploratory drilling, laboratory testing, engineering analysis, and preparation of this report.

1.3. Literature Search and Site Observations

We reviewed the 1966 drawings of the existing bridge, geologic maps, water well logs, and the on-site surface conditions prior to the subsurface investigation. The information was used to estimate the subsurface conditions and proposed drilling depths, and to provide a general overview of the site geology.

2.0. LOCAL GEOLOGY AND FAULTING

2.1. Local Geology

The bridge site is located west of the western foothills of the Western Cascade Range in the central Willamette Valley. Thomas Creek flows west at the bridge site and the confluence with the South Santiam River is ±9 miles southwest of the site.

Local geologic mapping shows the site is underlain by recent alluvial deposits of clay, silt, sand, and gravel associated with Thomas Creek (Beaulieu et al., 1974; Walker and Duncan, 1989; Yeats et al., 1996; Sherrod and Smith, 2000). The alluvium overlies nonmarine sedimentary and volcanic rocks. Older map sources refer to the rock as part of the Little Butte Volcanic Series (Beaulieu et al., 1974; Yeats et al., 1996). However, for this report, we have referred to the more recent mapping indicating the rocks are within the Tertiary Sedimentary Rocks of the Oligocene (±25 to 35 million years old) (Walker and Duncan, 1989; Sherrod and Smith, 2000). The soil profiles at the bridge site are consistent with the mapped local geology.

2.2. Seismicity and Faulting

A review of nearby faults was completed to evaluate the seismic setting and identify the potential seismic sources. The seismic sources include the Cascadia Subduction Zone (CSZ) and local crustal faults. The CSZ, located ±50 miles west of the Oregon coast shoreline, forms the plate boundary between the subducting Juan de Fuca Plate and the overriding North American Plate. The subduction zone extends ±700 miles from offshore northern California to southern British Columbia (Atwater, 1970). Geologic studies suggest the subduction zone is capable of producing a magnitude (M_W) 9 earthquake.

Numerous detailed studies of coastal subsidence, tsunami, and turbidite deposits estimate a wide range of CSZ earthquake recurrence intervals. Turbidite deposits in the Cascadia Basin have been investigated to help develop a paleoseismic record for the CSZ and estimate recurrence intervals for interface earthquakes (Adams, 1990; Goldfinger et al., 2012). A study of turbidites from the last ±10,000 years suggests the return period for interface earthquakes varies with location and rupture length. That study estimated an average recurrence interval of ±220 to 380 years for an interface earthquake on the southern portion of the CSZ, and an average recurrence interval of ±500 to 530 years for an interface earthquake extending the entire length of the CSZ (Goldfinger et al., 2012). However, older, deep-sea cores have been re-examined and the findings may indicate greater Holocene stratigraphy variability along the Washington coast (Atwater et al., 2014). Additional research by Goldfinger for the northern portion of the subduction zone suggests a recurrence interval of ±340 years for the northern Oregon Coast (Goldfinger et al., 2016).

Numerous northeast and northwest-trending concealed and inferred crustal faults are located within ±6 to 20 miles of the project site. (Walker and Duncan, 1989; Yeats et al., 1996; Sherrod and Smith, 2000). However, none of these faults show any evidence of movement in the last ±1.6 million years (Geomatrix Consultants, 1995; USGS, 2006). Six potentially active Quaternary (<1.6 million years or less) crustal fault zones have been mapped within ±40 miles of the site (Geomatrix Consultants, 1995; Personius et al., 2003; USGS, 2006; Niewendorp, 2014) and are listed in Table 1. Additional fault information can be found in the literature (Personius et al., 2003; USGS, 2006).

Table 1. Potentially Active Quaternary Crustal Faults within ±40 miles of the Thomas Creek-Richardson Gap Road (Shimanek) Covered Bridge

Fault Name	Length (miles)	Last Known Activity	Distance from Site (miles)	Slip Rate (mm/yr)
Mill Creek (#871)	±11	<1.6 million years	±11 NW	<0.20
Waldo Hills (#872)	±8	<1.6 million years	±14 NW	<0.20
Mount Angel (#873)	±19	<15,000 years	±21 N-NE	0.067*
Owl Creek (#870)	±9	<750,000 years	±21 SW	<0.20
Corvallis (#869)	±25	<1.6 million years	±23 SW	<0.20
Canby-Molalla (#716)	±31	<15,000 years	±29 NE	<0.20

Notes:
1. Fault data is based on the USGS, 2006 and USGS, 2008.
2. *From Table H-1 (Petersen et al., 2008).

Of the listed faults, all but the Corvallis fault are considered USGS Class A faults. Class A faults have geologic evidence supporting tectonic movement in the Quaternary, known or presumed to be associated with large-magnitude earthquakes (Personius et al., 2003). The Corvallis fault is considered a Class B fault by the USGS. Class B faults are of non-tectonic origin (e.g. volcanic activity) or demonstrate less evidence of tectonic displacement (Personius et al., 2003). All the faults listed are also found on the State of Oregon Active Fault list (Niewendorp, 2014).

3.0. SUBSURFACE EXPLORATION AND CONDITIONS

3.1. Exploration

Three exploratory boreholes (BH-1 through BH-3) were drilled at the site between November 19 and 21, 2018, using a CME 55, track-mounted drill rig drilling with mud-rotary and HQ coring methods. BH-1 was drilled ±13.5 feet north of the north abutment (Bent 5) and BH-2 was drilled ±26 feet south of the south abutment (Bent 1). BH-3 was drilled in the northeast quadrant of the site, ±10 feet north and ±8 feet east of Bent 3. The approximate borehole locations are shown on Figure 2A (Appendix A).

The boring locations were not surveyed. However, the boring elevations were estimated based on the elevations indicated on the preliminary plan and profile sheet provided by Linn County.

Disturbed soil samples were obtained in each boring at ±2.5 and 5-foot intervals until coreable bedrock was encountered. Samples were collected using a 2-inch diameter, split-spoon sampler in conjunction with the Standard Penetration Test (SPT). The SPT, which is performed when the split-spoon is driven, provides an indication of the relative stiffness or density of the soil (ASTM D1586). The number of blows required to drive the sampler the final 12 inches of an 18-inch drive is recorded and represents the standard penetration resistance or N-value in blows per foot (bpf). One relatively undisturbed sample was also obtained by pushing a thin-walled Shelby tube at a depth of ±5 feet in BH-3.

Continuous HQ-sized coring was completed from ±26 to 36 feet in BH-1 and from ±18 to 38 feet in BH-3. Coring was attempted in BH-2 from ±22 to 26 feet. However, the coring was discontinued due to an obstruction in the core barrel and other equipment issues. Therefore, in lieu of coring, mud-rotary drilling and SPT tests were used to characterize the bedrock conditions in BH-2 to a depth of ±45.3 feet.

The borings were continuously logged by a Foundation Engineering representative. The collected samples were sealed to avoid moisture loss and transported to our office for further examination and potential testing. The soil and rock profiles encountered in the borings are shown in the boring logs (Appendix B) and are discussed below. The final logs were prepared based on a review of the field logs, the results of the laboratory testing, and an examination of the samples in our office.

3.2. Subsurface Conditions

A general discussion of the subsurface conditions is presented below. A more detailed description of the soil conditions encountered in each boring is summarized on the appended logs.

3.2.1. Bent 1 (South Abutment) – BH-2: The paved surface at BH-2 lies at ±El. 369.5. The pavement section consists of ±4 inches of asphaltic concrete (AC) over ±20 inches of dense, ±1½-inch minus crushed rock (base rock). The pavement section is underlain by embankment fill consisting of very dense silty gravel with some sand to ±5 feet (±El. 364.5).

The embankment fill is underlain by alluvium consisting of stiff, medium to high plasticity clayey silt to ±14 feet (±El. 355.5) followed by dense to very dense silty gravel with some sand to ±20 feet (±El. 349.5), the approximate bedrock surface.

Silty sandstone (Continental Sedimentary Rocks) extends below the alluvium to ±40 feet (±El. 329.5). The silty sandstone is highly weathered and extremely soft (R0) from ±20 to 26 feet (±El. 343.5) and slightly weathered to fresh and very soft (R1) from ±26 to 40 feet. The silty sandstone is underlain by slightly weathered, very soft to soft (R1 to R2) volcaniclastic sandstone (Oligocene Sedimentary Rocks) to ±45.3 feet (±El. 324.2), the limits of the exploration.

One core run was completed in the bedrock with no recovery. No recovery was due to an obstruction in the core barrel that compromised the core. Therefore, drilling was switched back to mud-rotary drilling. Rock strength and type were inferred from the recorded SPT N-values and observations of the recovered SPT samples. The N-values are consistent with those obtained in the surficial rock surface of BH-1 and BH-3.

3.2.2. Bent 3 (North Covered Bridge Pier) – BH-3: The ground surface at BH-3 lies at ±El. 359.0. The ground surface is covered with ±12 inches of medium dense, ±1-inch minus crushed rock (fill). Alluvium extends below the fill and consists of loose silty sand to ±6.5 feet (±El. 352.5), dense to very dense silty gravel with some sand to ±10 feet (±El. 349.0), and medium dense to very dense sandy gravel with some silt to ±15.5 feet (±El. 343.5), the approximate bedrock surface.

Silty sandstone (Continental Sedimentary Rocks) extends below the alluvium to ±28.5 feet (±El. 330.5). The silty sandstone is highly weathered and extremely soft (R0) from ±15.5 to 20.8 feet (±El. 338.2) and slightly weathered and very soft (R1) from ±20.8 to 28.5 feet. The silty sandstone is underlain by slightly weathered to fresh, very soft to soft (R1 to R2) volcaniclastic sandstone (Oligocene Sedimentary Rocks) to ±38 feet (±El. 321.0), the limits of the exploration.

Four core runs were completed in the bedrock. Percent recovery ranged from 45 to 100% and the RQD ranged from 41 to 82%. The RQD values indicate close to moderately close jointing. Core photos are provided in Appendix B.

3.2.3. Bent 5 (North Abutment) – BH-1: The paved surface at BH-1 lies at ±El. 369.0. The pavement section consists of ±6 inches of asphaltic concrete (AC) over ±24 inches of dense, ±1½-inch minus crushed rock (base rock). The pavement section is underlain by embankment fill consisting of dense silty gravelly sand to ±7.5 feet (±El. 361.5).

The embankment fill is underlain by alluvium consisting of loose silty sand to ±18 feet (±El. 351.0) followed by dense silty gravel with some sand to ±22 feet (±El. 347.0), the approximate bedrock surface.

Silty sandstone (Continental Sedimentary Rocks) extends below the alluvium to ±32.7 feet (±El. 336.3). The silty sandstone is highly weathered and extremely soft (R0) from ±22 to 26.2 feet and slightly weathered to fresh and very soft to soft (R1 to R2) from ±26.2 to 32.7 feet. The silty sandstone is underlain by slightly weathered to fresh, very soft to soft (R1 to R2) volcaniclastic sandstone (Oligocene Sedimentary Rocks) to ±36 feet (±El. 333.0), the limits of the exploration.

Four core runs were completed in the bedrock. Percent recovery ranged from 0 to 100% and the RQD ranged from 0 to 93%. The core photos are provided in Appendix B.

3.3. Ground Water

Mud-rotary drilling precluded an accurate determination of the ground water level in the borings at the time of drilling. However, the water level in Thomas Creek, as measured near the middle of the main span, was ±21 feet (±El. 349.0) below the deck on November 19, 2018. We anticipate the ground water level in the vicinity of the bridge fluctuates seasonally and corresponds approximately to the water level in the creek.

4.0. LABORATORY AND FIELD TESTING

4.1. Laboratory Testing

Laboratory testing on the alluvium included natural water contents (ASTM D2216), percent fines determinations (ASTM D1140) and Atterberg limits (ASTM D4318) testing to classify the soils and estimate their engineering properties. The results are summarized in Table 1C (Appendix C). The water content determinations are also shown on the boring logs.

Four unconfined compression tests (ASTM D7012-C) were run on samples of silty sandstone to estimate the unconfined compressive strength (q_u) of the bedrock. The tests indicate q_u values in the range of 854 to 1,799 lb/in^2 (psi). These values are consistent with a rock hardness of R1 to R2. The stress versus strain plots of the four samples tested are summarized in Figures 1C through 4C (Appendix C).

4.2. DCP Testing

In-situ Dynamic Cone Penetrometer (DCP) testing (ASTM D6591) was completed in conjunction with the borings to estimate the subgrade resilient modulus (M_R) for pavement design. The DCP test includes driving the cone of the DCP apparatus into the subgrade or base rock using a drop hammer. The penetration versus blow count is recorded in millimeters per blow (mm/blow) as the DCP value. The Oregon Department of Transportation (ODOT) Pavement Design Guide (PDG 2011) provides a correlation for estimating the in-situ resilient modulus from results of the DCP testing. The DCP test results and the correlated M_R values are summarized in Table 2C (Appendix C).

4.3. Resistivity and pH Testing

In-situ resistivity testing was completed using a Nilsson 400, 4-pin, soil resistance meter (ASTM G57). The resistivity test was completed immediately north of BH-3. The approximate location is shown on Figure 2A. The 4-pin resistance meter provides an estimate of the average resistivity of a soil profile extending to a depth equal to the spacing between the pins. The resistivity tests were performed with the pins spaced at ±5, 10, and 15 feet. The resistivity values are summarized in Table 3C (Appendix C).

Three pH tests (ASTM G51) were completed on samples obtained from ±7.5 to 14 feet in BH-1 and BH-2. The test values are summarized in Table 4C (Appendix C).

5.0. HYDRAULICS/SCOUR

A hydraulic and scour study was not available at the time this report was prepared. However, we understand scour mitigation is a key component of the rehabilitation work for the interior bents (Bents 2 and 3). The scour mitigation option initially considered installing sheet piles around the piers. However, based on the relatively shallow depth to bedrock that would limit the penetration of the sheet piles, this option was eliminated from further consideration. Therefore, the proposed scour mitigation is to place Class 700 riprap around the interior bents.

6.0. SEISMIC ANALYSIS AND EVALUATION

6.1. Bedrock Acceleration and Site Response

Response spectra for the site were developed based on the 2018 ODOT Geotechnical Design Manual (GDM) “life-safety” and “operational” criteria. The “life-safety” (i.e., no collapse) seismic performance criteria assumes earthquake ground motions having a 1,000-year average return period. The “operational” (i.e., remain in service) criteria assumes a full-rupture Cascadia Subduction Zone Earthquake (CSZE) event. The response spectra and design parameters are shown on Figure 3A (Appendix A).

The ground motions for the 1,000-year return period life-safety response spectrum were developed using the General Procedure in the AASHTO LRFD Bridge Design Specifications (2014), with modifications recommended in the 2017 ODOT GDM. The ground motion parameters, including peak ground accelerations (PGA), short period (0.2 second) spectral accelerations (S_s), and long period (1.0 second) spectral accelerations (S_1) on bedrock were calculated using the ODOT ARS V 2014.16 spreadsheet, which is based on the 2014 USGS seismic hazard maps (Peterson et al., 2014). Following the AASHTO General Procedure, the spectral accelerations on bedrock were scaled to the ground surface using F_{pga}, $F_{a,}$ and F_v values appropriate for the Site Class. The Site Class accounts for the average subsurface conditions within 100 feet of the ground surface. The subsurface conditions at the bridge site correspond most closely to a Site Class D based on the subsurface profile and depth-averaged SPT N-values documented in the borings. The scaling factors were selected based on ODOT GDM Tables 6.2-A, 6.2-B, and 6.2-C.

The ground motions for the CSZE operational response spectrum were obtained using the Portland State University (PSU) Acceleration Response Spectra website (PSU, 2017). The website requires inputting latitude and longitude coordinates for the project site and an assumed average shear wave velocity to a depth of ±30 meters (V_S30). We assumed a V_S30 equal to 270 meters/second based on a Site Class D soil profile.

6.2. Liquefaction, Settlement and Lateral Spread

Liquefaction is typically observed in saturated deposits of loose sand and non-plastic to low plasticity silt (i.e., PI less than 6) subjected to intense ground shaking. Loose silty sand was encountered below the approach fill in BH-1 (near Bent 5) from ±7.5 to 18 feet (±El. 361.5 to El. 351.0) and from in BH-3 (near Bent 3) ±1 to 6.5 feet (±El. 358.0 to El. 352.5) in BH-3. However, due to the fines content we do not believe this material poses a significant liquefaction and lateral spread hazard even if the material were to become saturated during periods of higher water levels in the creek. If liquefaction were to occur during the design earthquake, the material would densify and result in several inches of approach fill settlement. The bridge structure will be supported on deep foundations that bypass the silty sand, so liquefaction-induced settlement of the structure foundations is not a hazard.

7.0. FOUNDATION ANALYSIS AND DESIGN RECOMMENDATIONS

7.1. Discussion of Foundations Options

Deep foundations are recommended for the reconstructed bridge supports due to the existing scour hazard and the presence of loose and compressible soils at shallow depths. Driven piles or grouted-in-place (GIP) piles were considered as foundation options. Driven piles could be installed. However, there is a risk the piles will not penetrate the bedrock far enough to provide lateral support. Therefore, to eliminate this risk, GIP piles socketed into bedrock were selected in consultation with Linn County.

7.2. Foundation Loads

The number of piles per bent and the corresponding maximum (factored) pile loads are summarized in Table 2.

Table 2. Foundation Load Summary

Bent	Number of Piles per Bent	Maximum Factored Pile Load (kips)
1	4	108.8
2	7	177.6
3	7	177.6
4	4	153.6
5	4	108.8

7.3. GIP Pile Analysis and Design

7.3.1. Pile Type and Material Specifications. In consultation with Linn County, PP12.75x0.375 piles (ASTM A252) were selected. The recommended pile properties are summarized in Table 3. The piling will be set in 24-inch diameter, predrilled holes and grouted in place. The grout surrounding the piles should have a minimum 28-day compressive strength of 3,000 psi and material properties consistent with ODOT Section 02080.40 (Portland Cement Grout).

Table 3. Recommended Pile Properties

Pile Properties	PP12.75x0.375
Steel Grade	ASTM A252, Grade 3
Yield Stress (F_y)	45 ksi
Area Steel (A_s)	14.6 in^2
Nominal Structural Resistance (F_y x A_s)	657 kips

7.3.2. Downdrag. At least ± ½ inch of ground settlement around the pile is typically required to induce downdrag loads on deep foundations. The reconstructed bridge will be constructed with a similar alignment and grade relative to the existing roadway. Therefore, little or no increase to the overburden pressure is expected at the abutments and downdrag from embankment settlement is not a design concern.

If liquefaction (or densification) induced settlement were to occur from a seismic event, the loads imparted would be negligible because of the low strength of the overburden soil compared to the relatively high axial resistance that will be mobilized in the rock socket. Therefore, downdrag from liquefaction-induced settlement is not a design consideration.

7.3.3. Nominal and Factored Axial Resistance. Axial analysis for the GIP piles was completed using the AASHTO (2018) Load Resistance Factor Design (LRFD) approach. The analysis is discussed in the following sections.

The nominal axial resistance of the GIP portion of the foundation system was estimated per the FHWA Drilled Shafts: Construction Procedures and LRFD Design Methods manual (2010), for drilled shafts socketed into intermediate geomaterials and rock. The nominal axial resistance was calculated using the side friction resistance mobilized at the contact between the grout and the sidewalls of the predrilled holes. Side friction in the soil above the bedrock was neglected. End-bearing resistance was also neglected due to the displacement required to mobilize end-bearing and the potential difficulties in cleaning out the bottom of the predrilled holes.

The bedrock profile consists of ±4 to 6 feet of extremely soft (R0), highly weathered silty sandstone underlain by very soft (R1), slightly weathered silty sandstone. Due to the relatively uniform rock profile, we used an average profile to model the rock conditions for foundation design at each bent location. Our model consists of ±5 feet of extremely soft (R0) silty sandstone followed by very soft (R1) silty sandstone. Very soft to soft (R1 to R2) volcaniclastic sandstone was encountered at greater depths across the site, but is deeper than the required rock socket depth.

Poor quality rock in the extremely weathered (R0) silty sandstone precluded laboratory strength testing for this zone. Therefore, we assumed an unconfined compressive strength (q_u) of 100 lb/in^2 (psi) for this zone based on the recorded SPT N-values. Laboratory testing on the underlying, very soft (R1) silty sandstone indicate q_u values in the range of ±854 to 1,799 psi. We selected a q_u value of 750 psi for design to account for the variability of the rock strength with depth and along the bridge alignment.

Using the rock strength values and a 24-inch diameter rock socket, we calculated the nominal axial resistance for the GIP piles. An AASHTO resistance factor (ϕ) of 0.55 (for side resistance in rock) was applied to the nominal axial resistance to estimate the required tip elevations.

Using the rock strength values discussed above, we calculated the nominal and factored axial resistances for a 24-inch diameter rock socket. The factored resistance was estimated based on an AASHTO resistance factor (ϕ) of 0.55 for side resistance in rock. The nominal and factored axial resistance per pile for rock socket lengths ranging from 6 to 9 feet are provided in Table 4. This range captures the maximum factored loads at each bent provided in Table 2.

Table 4. Axial Resistances versus Rock Socket Length

Rock	Rock Socket Length (ft)	Nominal Axial Resistance (kips)	Factored Axial Resistance (kips)
R1 Silty Sandstone	6	128	72
R1 Silty Sandstone	7	223	124
R1 Silty Sandstone	8	318	176
R1 Silty Sandstone	9	413	229

7.3.4. Minimum/Estimated Pile Tip Elevations. Based on the factored loads provided in Table 2, our analyses indicate the predrilled, GIP piles will develop the required axial resistance with rock socket depths ranging from 6 to 9 feet. Recommended tip elevations and pile lengths, and estimated rock socket lengths for the predrilled, GIP piles are summarized in Table 5.

Table 5. Estimated Tip Elevations and GIP Pile Lengths Set in 24-inch Diameter Predrilled Holes

Bent	[1]Cut-off Elevation (feet)	[2]Estimated Bedrock Elevation (feet)	Estimated Tip Elevation (feet)	Required Rock Socket Depth (feet)	Estimated Pile Length (feet)
1	369.4	El. 349.5	342.5	7.0	29.0
2	352.0	El. 349.5	340.5	9.0	14.0
3	352.0	El. 343.5	334.5	9.0	20.0
4	366.5	El. 343.5	335.5	8.0	33.0
5	368.8	El. 347.0	340.0	7.0	31.0

Notes: 1. Cut-off elevations provided by Linn County.
2. Estimated bedrock elevation at Bent 1 and 2 is based on BH-2. Estimated bedrock elevation at Bent 3 and 4 is based on BH-3. Estimated bedrock elevation at Bent 5 is based on BH-1.
3. Estimated pile length includes 2 to 3 feet of extra length to account for possible variation in the bedrock surface.

7.3.5. Nominal Uplift Resistance. Uplift resistance will be mobilized in the side resistance that develops along the rock socket portion of the GIP pile. An LRFD ϕ factor of 0.4 should be applied to the nominal axial resistance reported in Table 4 for Strength Limits analysis.

7.3.6. Pile Settlement. The GIP piles will be grouted into bedrock. Therefore, settlement is expected to be limited to the displacement required to mobilize the side resistance and elastic compression of the pile (i.e., less than 0.1 inch).

7.3.7. Lateral Analysis. Lateral analysis was not required for the rehabilitation project.

8.0. APPROACHES AND EMBANKMENTS

8.1. Embankment Construction and Settlement

The rehabilitated bridge will remain along the existing horizontal and vertical alignments. New approach construction will be limited to that required to rebuild the approaches and widen the shoulders at the north approach around Bent 5. Based on the absence of new fill at the south abutment and the limited fill placement at the north abutment, settlement of the new approaches is not a design concern.

8.2. Approach Pavements

The following provides a discussion of the pavement analysis for the reconstructed approaches. The analysis and recommendations provided herein are based on the ODOT Pavement Design Guide (2019).

8.2.1. Subgrade. The existing approaches include crushed rock (base rock) to a depth of ±2 to 2.5 feet, followed by embankment fill consisting of silty gravelly sand at the south abutment and silty gravel with some sand at the north abutment. DCP testing indicated a subgrade resilient modulus (M_r) of 15,986. A subgrade M_r of 15,000 was used for our design calculations to account for possible subgrade variation across the site. An M_r value of 20,000 psi was assumed for new Base Aggregate, consistent with ODOT PDG (2019) design recommendations.

8.2.2. Traffic Data. Results of a 2012 traffic study completed by Linn County indicates combined (i.e., two-way), average daily traffic (ADT) of 1,154 vehicles with 13.5 percent trucks. An annual growth rate of 1.76 percent was reported in the project prospectus. We applied the annual growth rate to the 2012 ADT to calculate the 2020 ADT (assumed project completion date) of 1,327 vehicles and the 2050 ADT (30-year design life) of 2,240 vehicles. A directional factor of 55% was applied to the two-way ADT.

We assumed the percentage of truck traffic would remain at 13.5 percent throughout the pavement design life and used a range of FHWA truck classifications (Class I to Class 10) to calculate the annual ESAL value. The tabulated distribution is provided in Appendix D.

A 30-year Equivalent Single Axle Load (ESAL) value of 1,091,733 was calculated based on the above traffic information.

8.2.3. Pavement Design. We used the ODOT PDG (2019) procedure for design and assumed the following parameters:

- reliability of 85%
- overall deviation of 0.49
- initial serviceability of 4.2
- terminal serviceability of 2.5

- layer coefficient of 0.42 for new AC
- layer coefficient of 0.10 for Base Aggregate
- subgrade resilient modulus, M_R, of 15,000 psi
- drainage coefficient of 1.0
- 30-year design life

The following steps were taken to determine the minimum pavement section:

1. The required Structural Number (S_N) for the AC surface course was determined based on the design traffic and the ODOT-recommended resilient modulus of 20,000 psi for Base Aggregate. The AC thickness was determined assuming a layer coefficient of 0.42 and a drainage coefficient of 1.0.

2. The required S_N for the Base Aggregate was determined by subtracting the S_N for the AC (Step 1) from the total required S_N, for the pavement section. The minimum thickness of Base Aggregate was calculated assuming a layer coefficient of 0.10 and drainage coefficient of 1.0 for Base Aggregate. A resilient modulus of 15,000 psi was assumed for the subgrade based on available correlations and the results of DCP testing.

Our calculations indicate a minimum pavement section of 6 inches of AC over 2 inches of Base Aggregate is required. This section is less than the County minimum standard of 6 inches AC over 12 inches of Base Aggregate. Therefore, the County minimum standard section is recommended.

8.3. Abutment Walls and Wing Walls

8.3.1. Static Wall Pressures. Preliminary drawings provided by Linn County indicate the abutment and wing walls will have a maximum height of 5 feet. The wing walls will extend back 8 feet (perpendicular) from the abutment walls.

We assume Granular Structure Backfill (Section 00510.13) will be used in the zone behind the walls. A friction angle of 34 degrees and a unit weight of 125 pcf were assumed for the wall backfill. Drained conditions were also assumed.

A lateral deflection of at least 0.001*H (where H is the height of the wall) is required for the walls to mobilize an active earth pressure condition within the granular wall backfill. For a 5-foot tall wall, the deflection is less than 0.1 inch. Typically, abutment walls deflect enough for the active earth pressure condition. However, integral abutment walls or wing wall-to-abutment wall corners may not be free to deflect. Therefore, earth pressures for both the active and at-rest condition are provided.

For restrained abutment walls, we recommend using an at-rest earth pressure coefficient (k_o) of 0.44. The nominal lateral earth pressure on restrained walls may be estimated using an at-rest equivalent fluid density of 55 pcf.

For unrestrained abutment walls (able to deflect or rotate at least 0.001*H), we recommend using an active earth pressure coefficient (k_a) of 0.28. The nominal lateral earth pressure on unrestrained walls may be estimated using an equivalent fluid density of 35 pcf.

AASHTO (2018) recommends calculating the traffic loads applied to the top of the abutment walls using an equivalent soil surcharge. For an abutment height of 5 feet, a minimum surcharge height of 4 feet is recommended. Using a unit weight of 125 pcf and a surcharge height of 4 feet results in a nominal uniform surcharge pressure of 500 psf.

Applying the at-rest pressure coefficient of 0.44 results in an additional, nominal, uniform lateral pressure of 220 psf for restrained walls. Applying the active pressure coefficient of 0.28 results in an additional, nominal, uniform lateral pressure of 140 psf for unrestrained walls.

An equivalent soil surcharge of 2 feet and active earth pressure conditions are recommended for wing wall design. Using a unit weight of 125 pcf and a surcharge height of 2 feet results in a nominal uniform surcharge pressure of 250 psf. Applying the active pressure coefficient of 0.28 results in an additional, nominal, uniform lateral pressure of 70 psf on the wing walls.

8.3.2 Seismic Lateral Earth Pressures. The ODOT GDM (2018) requires walls that affect the performance or structural integrity of the bridge be designed for a peak horizontal acceleration corresponding to a 1,000-year return period. For the 1,000-year return period seismic event, we used a design horizontal acceleration (k_h), equal to one-half of the estimated ground surface acceleration (A_s) of 0.30g. A_s is calculated using the USGS PGA (on rock) of 0.22g and multiplying it by the AASHTO site factor (F_{pga}) of 1.38 for an AASHTO Site Class D soil profile.

Mononobe-Okabe analysis was used to calculate a seismic active earth pressure coefficient (k_{ae}). For the analyses, the peak horizontal ground acceleration (k_h) and corresponding seismic lateral earth pressure coefficient (k_{ae}) depend upon the allowable lateral deflection of the wall during an earthquake. The allowable seismic wall displacement was assumed to be ±1 to 2 inches. Assuming 5-foot high abutment and wing walls, the seismic force may be modeled using an additional uniform pressure of 29 psf.

A summary of the calculated abutment and wing wall static and seismic lateral earth pressures is provided in Table 6.

Table 6. Lateral Earth Parameters for Abutment and Wing Wall Design

Parameter	Source	Value	γ_p
At Rest Earth Pressure Coefficient, k_o	$1\text{-}\sin(\phi)$	0.44	
Active Earth Pressure Coefficient, k_a	$\tan^2(45 - \phi/2)$	0.28	
At-Rest Equivalent Fluid Density	$k_o * \gamma_{backfill}$	55 pcf	1.35
Active Equivalent Fluid Density	$k_a * \gamma_{backfill}$	35 pcf	1.50
Traffic Load Surcharge for Abutment Walls (At Rest)	(500 psf*k_o)	220 psf	1.35/1.75
Traffic Load Surcharge for Abutment Walls (Active)	(500 psf*k_a)	140 psf	1.35/1.75
Traffic Load Surcharge for Wing Walls (Active)	(250 psf*k_a)	70 psf	1.35/1.75
Seismic Pressure for Wall backfill for 1,000-year event (assumes 1 to 2-inch displacement)	Mononobe-Okabe	39 psf	1.00

The appropriate load factors (γ_p) provided in AASHTO Table 3.4.1-2 should be applied to the preceding nominal pressures to estimate the factored lateral earth loads. Selection of the appropriate load factors are dependent on the load case being analyzed. AASHTO (2018) recommends a load factor of 1.35 for at-rest earth loads and 1.5 for active earth loads. For the traffic load surcharge, a load factor of 1.75 is recommended for Strength I and 1.35 for Strength II and V.

9.0. CONSTRUCTION RECOMMENDATIONS

9.1. Specifications

All specification sections contained herein refer to the Oregon Standard Specifications for Construction (2018). It is also assumed these specifications will be referred to for general or specific items not addressed in this report.

9.2. GIP Piles

Individual GIP piles should be monitored throughout construction by a design team representative to provide QA/QC during drilling and concreting. Monitoring of the drilling should follow the same requirements as drilled shaft excavations (including Sections 00512.40, 00512.41, 00512.42 and 00512.43). Additional recommendations are as follows:

Equipment. The pile installation contractor should provide the necessary equipment to predrill the 24-inch diameter holes to the required elevation and install the piles in general accordance with the recommendations provided herein.

Potential Obstructions. Based on our explorations, the overburden material overlying the bedrock consists of silt to gravel-size materials. Therefore, we believe the risk of potential obstructions is low.

Casing. Temporary casing may be installed to the rock line, as required.

Predrilling. Predrilling of the bedrock and any remaining overburden material will be required to provide the minimum tip elevation indicated in Table 5. The hole drilled for the rock socket should have a minimum diameter of 24 inches. All loose material should be removed from the base of the excavation prior to setting the piles and placing grout.

Predrilling will require drilling in extremely soft to very soft (R0 to R1) bedrock. Laboratory tests indicated unconfined compressive strengths of the bedrock of up to ±1,800 psi. The drilling subcontractor is responsible for reviewing the boring logs and other available information to determine the most appropriate drilling equipment and tooling. Photos of the rock core are included in Appendix B, and samples of the rock core are available for viewing by contractors bidding on the work.

Grout and Grout Placement. The annulus between the piles and the predrilled holes shall be filled with 3,000 psi grout consistent with Section 02080.40.

Preboring will extend below the creek level. We anticipate ground water will infiltrate into the holes during drilling. Therefore, wet construction methods will be required, and the grout will have to be placed by tremie below the water. Contractors shall capture water and cuttings displaced during grout placement and dispose of it away from the site.

Additional Recommendations. The following recommendations should be incorporated into the special provisions to augment the standard specifications for predrilled GIP piles. These recommendations are provided to give geotechnical input into the development of the special provisions.

- Include a note in the special provisions notifying the contractor of the availability of this report, the boring logs and the rock core samples in our office.
- Require the contractor to submit a pile installation plan. This plan should at a minimum include: proposed drilling equipment, a sequence of drilling and casing installation, proposed methods for cleaning the borehole, procedure for pile installation, and proposed grout mix and placement.
- Predrill to the required minimum pile tip elevations identified in Table 5. A Foundation Engineering representative should be on site during construction to confirm the rock surface elevation at each bent.

- Require the holes be located within the location tolerance shown on the plans. In addition, the completed excavation should not vary from vertical by more than 2% of the excavation depth.

- Require the contractor to clean the bottom of the hole with a cleanout bucket or air lift so that no more than 1 inch of loose material remains in the bottom of the excavation.

- Require the piles be fitted with exterior spacers that will keep the pile in the approximate center of the hole.

- Do not place the pile in the hole until a Foundation Engineering representative has evaluated and approved the excavation.

- The grout should be placed with a tremie pipe near the bottom of the pile. Pump the grout in a continuous manner until clean grout flows from the top of the cased hole. Other methods of grout placement may be considered as part of the installation plan. Maintain 5 feet of head on the grout during extraction of the tremie pipe and casing.

- Require the pile be restrained from moving vertically or horizontally during grouting and casing extraction.

9.2. Temporary Detour Structure

We understand Richardson Gap Road will be closed during construction. Therefore, a temporary detour structure will not be required.

9.3. Excavations/Shoring/Dewatering

We anticipate excavations up to ±5 feet deep will be required for construction of Bent 1 and Bent 5. The excavations will extend through the pavement sections followed by embankment fill consisting of dense mixtures of silt, sand and gravel. Excavations at interior Bents 2 and 3 will extend ±4 feet deep to accommodate construction of the pile caps.

Temporary slopes no steeper than 1.5(H):1(V) should be planned, unless shored. Flatter slopes will be required to control erosion and sloughing during wet weather. Dewatering will be required at Bent 2 and/or Bent 3 if river levels are higher than ±El. 350 at the time of construction.

9.4. Approach Embankments

The approach work will include minor fill placement to accommodate shoulder widening at the north abutment. The following construction recommendations are based on the requirements of Section 00330.

9.4.1. Subgrade Preparation. Excavations should be completed in accordance with Section 00330.41. Soft or loose subgrade, if encountered, may be mitigated by moisture-conditioning and re-compacting the subgrade, or by overexcavating and replacing the unsuitable material with imported material. If practical, existing granular fill (e.g., base rock) and AC grindings (ground to particle size of 3 inches or less) may be re-used in overexcavation areas.

Moisture-conditioning and subgrade compaction should be completed in accordance with Section 00330.43. Beneath new pavements, the finished subgrade should be proof-rolled with a loaded dump truck or other approved construction vehicle prior to placing Base Aggregate to identify any soft areas. Any soft or pumping subgrade should be reworked or overexcavated and replaced with additional Base Aggregate.

9.4.3. Embankment Fill. The limited embankment and/or approach construction should be completed in accordance with Section 00330.42. The embankment material may consist of Selected Granular Backfill (00330.14) or Selected Stone Backfill (00330.15) for slopes constructed during dry weather at 2(H):1(V), or flatter. Stone Embankment (Section 00330.16) may be required, if construction occurs during wet weather or if steeper slopes are required.

All fills required for permanent embankment widening should be placed on properly stripped and benched slopes in accordance with ODOT Standard Embankment Construction Detail, DET2100.

9.4.4. Abutments and Wing Walls. Placement and compaction of imported fill behind the abutment walls and wing walls should be completed using light, vibratory equipment within 5 feet of the wall. Granular Wall Backfill (00510.12) should be used behind these walls.

9.5. Approach Pavement Design

The recommended approach pavement section thickness is 6 inches of AC over 12 inches of Base Aggregate. Based on the ODOT PDG (2019), the pavement mix design for new AC should consist of the following:

- 2-inch thick, Level 2, ½-inch Dense-Graded HMAC Wearing Course with PG 64-22 binder.
- 4-inch thick, Level 2, ½-inch Dense-Graded HMAC Base Course (Two, 2-inch thick lifts) with PG 64-22 binder.

Section 10.4 (Table 5) of the ODOT PDG (2019) indicates the project location does not require the use of anti-stripping additives in the HMAC.

The Base Aggregate should conform to the material requirements of Section 02630 and the grading requirements should conform to Table 02630-1 (¾″ - 0).

The Subgrade Geotextile (separation) should conform to the property requirements of Table 02320-4 for woven geotextile.

9.5. Temporary Detour Structure

We anticipate the road will be closed during construction. Therefore, a temporary detour structure will not be required.

10.0. LIMITATIONS

10.1. Construction Observation/Testing

We recommend a Foundation Engineering representative be present during construction to observe the pile installation, excavations for wall footings, and subgrade preparation. Any geotechnical engineering judgment in the field should be provided by one of our representatives. ODOT specified QA/QC testing should be performed on all foundations, compacted fills, subgrade, base rock, and asphalt pavement.

10.2. Variation of Subsurface Conditions, Use of Report, and Warranty

The analysis, conclusions, and recommendations contained herein assume the subsurface profiles encountered in the borings are representative of the site conditions. The above recommendations assume we will have the opportunity to review final drawings and be present during construction to confirm the assumed foundation conditions. No changes in the enclosed recommendations should be made without our approval. We will assume no responsibility or liability for any engineering judgment, inspection, or testing performed by others.

This report was prepared for the exclusive use of the Linn County Road Department and their design consultants for the Thomas Creek, Richardson Gap Road (Shimanek) Covered Bridge rehabilitation project in Linn County, Oregon. Information contained herein should not be used for other sites or for unanticipated construction without our written consent. This report is intended for planning and design purposes. Contractors using this information to estimate construction quantities or costs do so at their own risk. Our services do not include any survey or assessment of potential surface contamination or contamination of the soil or ground water by hazardous or toxic materials. We assume those services, if needed, have been completed by others.

Our work was done in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

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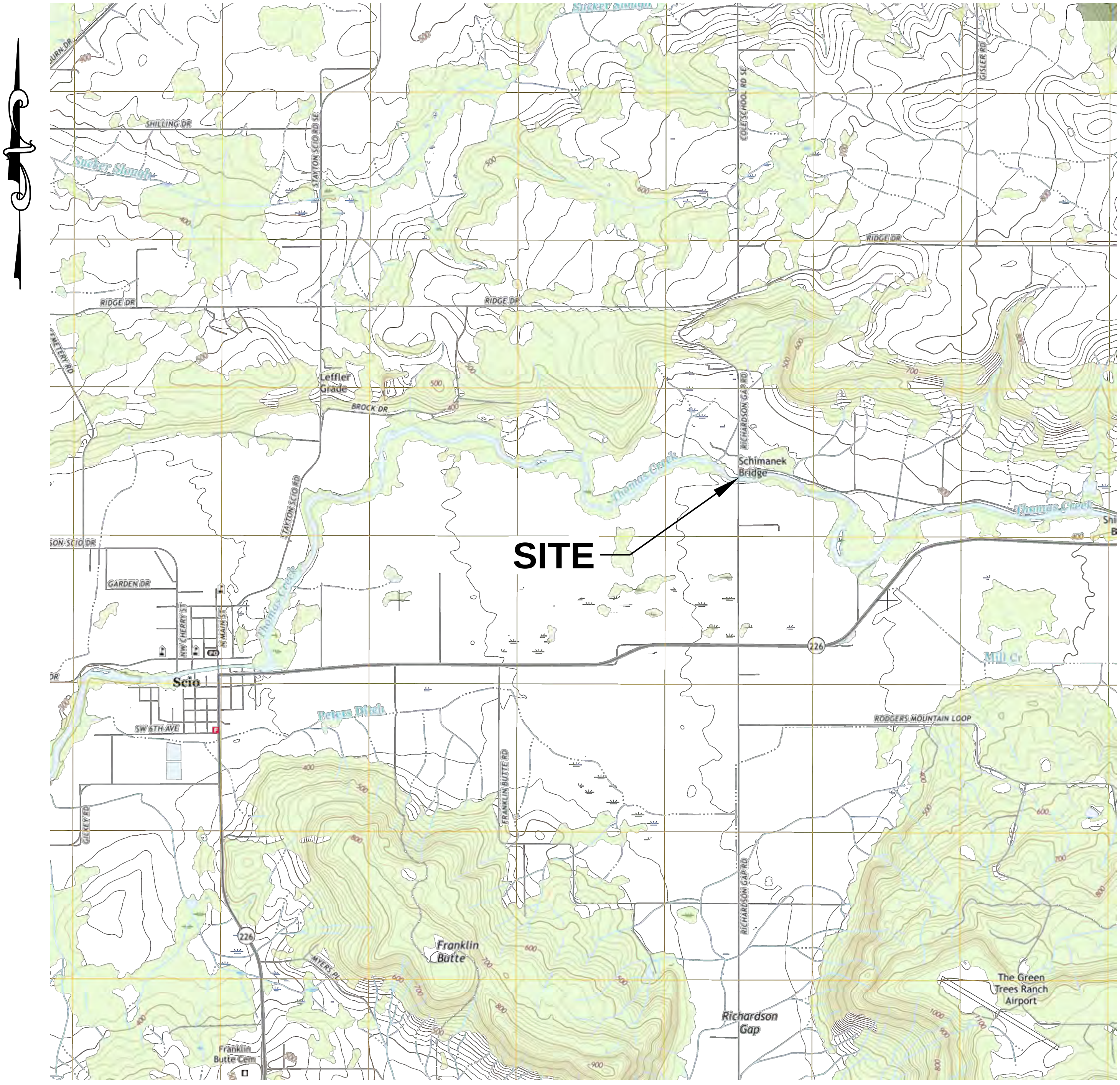
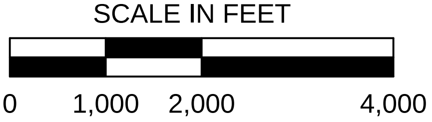
Appendix A

Figures

Foundation Engineering, Inc.
Professional Geotechnical Services





DATE NOV. 2018
DWN. mdm
APPR.
REVIS.
PROJECT NO.
2181118

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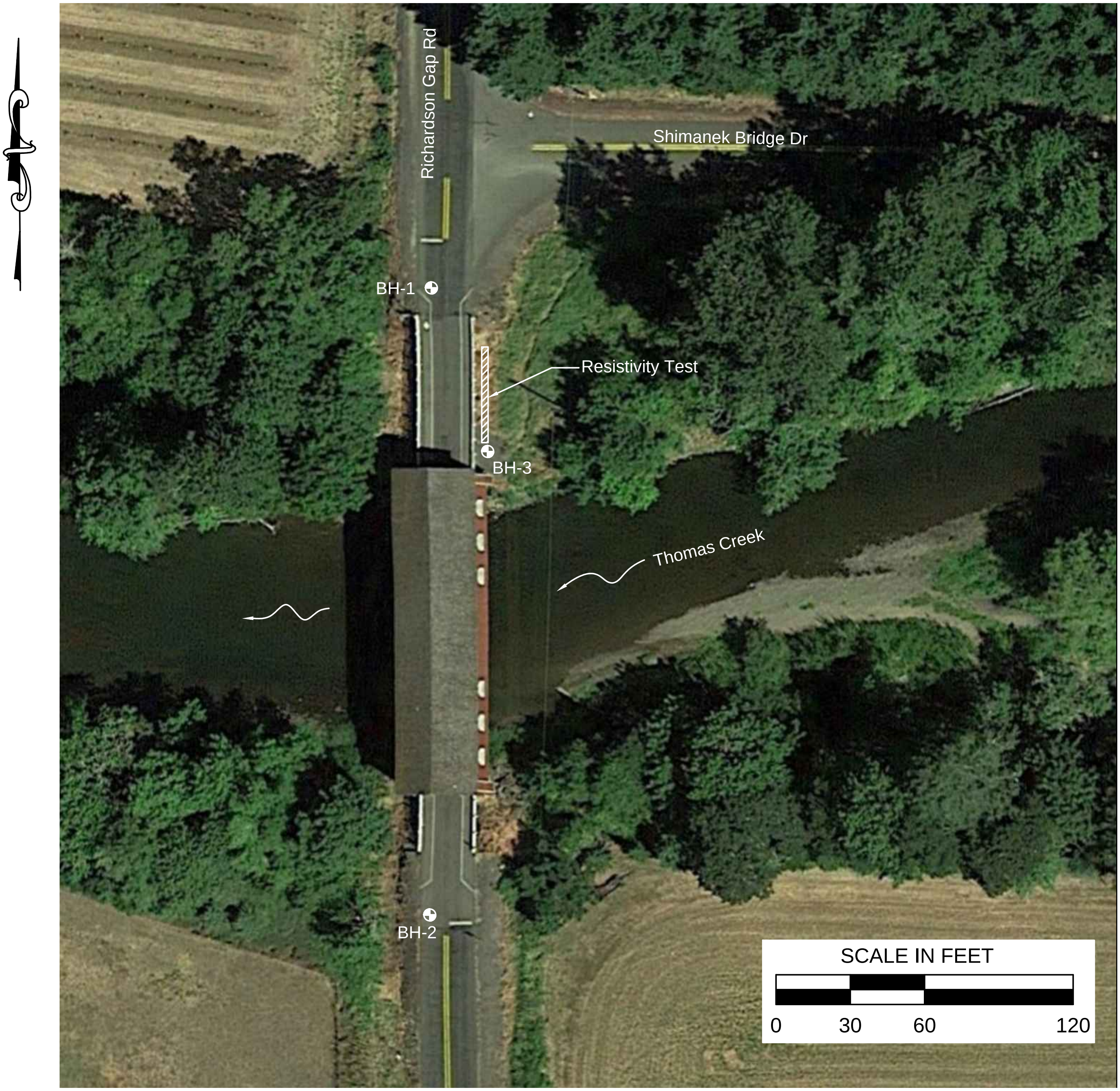
VICINITY MAP

THOMAS CREEK, RICHARDSON GAP ROAD (SHIMANEK) COVERED BRIDGE
LINN COUNTY, OREGON

FIGURE NO.
1A

FILE NAME: Fig1A



NOTES:

1. BORING LOCATIONS WERE NOT SURVEYED AND WERE ESTABLISHED USING A TAPE MEASURE REFERENCING EXISTING LANDMARKS. LOCATIONS ARE APPROXIMATE ONLY.
2. SEE MEMO FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. AERIAL IMAGE OBTAINED FROM GOOGLE EARTH.

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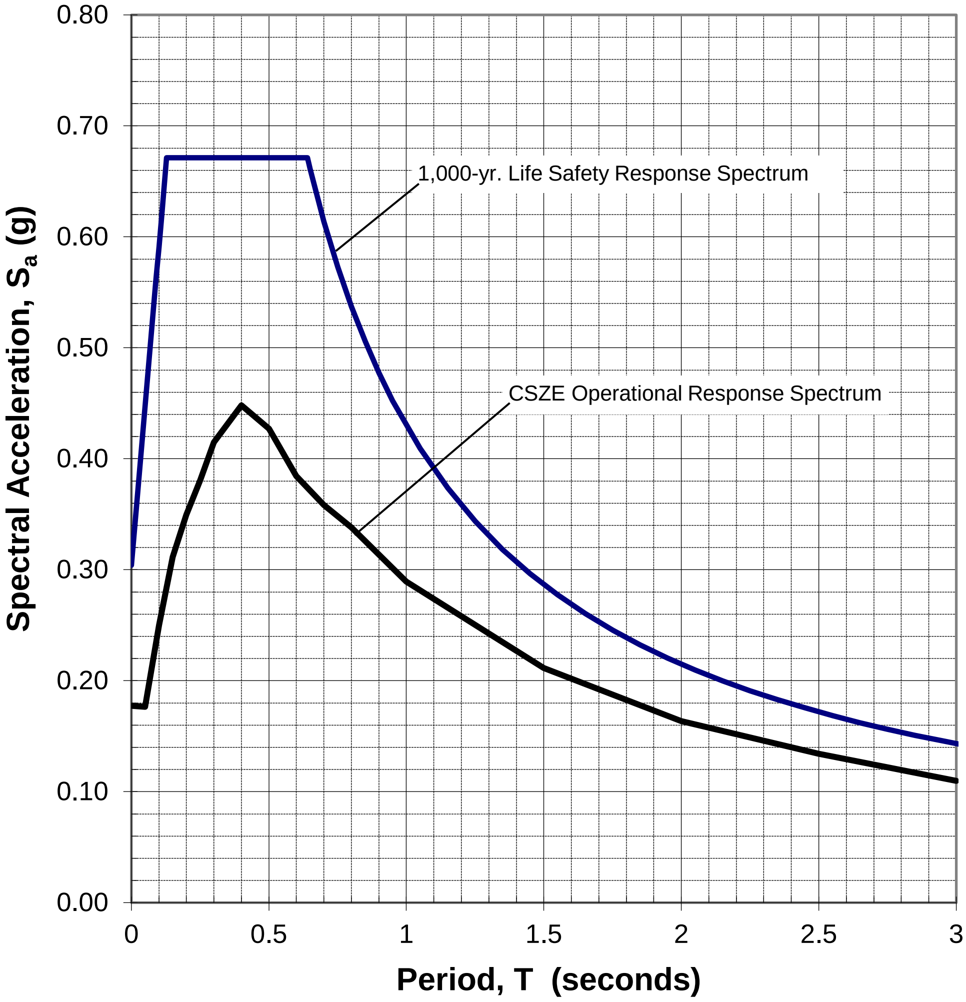
BORING LOCATIONS

THOMAS CREEK, RICHARDSON GAP ROAD (SHIMANEK) COVERED BRIDGE
LINN COUNTY, OREGON

FIGURE NO.
2A

FILE NAME: Fig2A



Notes:

1. The 1,000-yr. Life Safety Design Response Spectrum is based on AASHTO 2014 Section 3.10.3 using the following parameters:

Site Class= D Damping = 5%
PGA = 0.22 F_{pga} = 1.38 A_s = 0.30
S_S = 0.47 F_a = 1.42 S_{DS} = 0.67
S_1 = 0.19 F_v = 2.21 S_{D1} = 0.43

PGA, S_S and S_1 values are based on USGS 2014 seismic hazard maps and were obtained using the ODOT ARSV2014.16.xls spreadsheet. F_{pga}, F_a, and F_v were established based on ODOT GDM 2016, Tables 6.2-A, 6.2-B and 6.2-C using the selected PGA, S_S, and S_1 values.

2. The CSZE values were obtained using the PSU CSZ calculator assuming Vs30 = 270 m/s consistent with the average assumed shear wave velocity for a Site Class D profile.

3. Site location: Latitude 44.7157, Longitude -122.8044.

FIGURE 3A
LIFE SAFETY AND OPERATIONAL DESIGN CRITERIA RESPONSE SPECTRA
Thomas Creek, Richardson Gap Road (Shimanek) Covered Bridge
Linn County, Oregon
Project No. 2181118

Appendix B

Boring Logs and Rock Core Photos

Professional
Geotechnical
Services

Foundation Engineering, Inc.

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for each boring or test pit by our field representative. The log contains information concerning sampling depths and the presence of various materials such as gravel, cobbles, and fill, and observations of ground water. It also contains our interpretation of the soil conditions between samples. The final logs presented in this report represent our interpretation of the contents of the field logs and the results of the sample examinations and laboratory test results. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

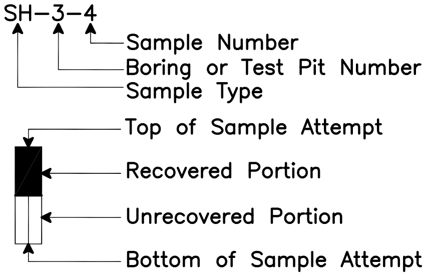
VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

The final log and related information depict subsurface conditions only at the specific location and on the date indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ. Actual foundation or subgrade conditions should be confirmed by us during construction.

TRANSITION BETWEEN SOIL OR ROCK TYPES

The lines designating the interface between soil, fill or rock on the final logs and on subsurface profiles presented in the report are determined by interpolation and are therefore approximate. The transition between the materials may be abrupt or gradual. Only at boring or test pit locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes thereon.

SAMPLE OR TEST SYMBOLS



C – Pavement Core Sample
CS – Rock Core Sample
OS – Oversize Sample (3-inch split-spoon)
S – Grab Sample
SH – Thin-walled Shelby Tube Sample
SS – Standard Penetration Test Sample (split-spoon)

▲ Standard Penetration Test Resistance equals the number of blows a 140 lb. weight falling 30 in. is required to drive a standard split-spoon sampler 1 ft. Practical refusal is equal to 50 or more blows per 6 in. of sampler penetration.

● Water Content (%).

UNIFIED SOIL CLASSIFICATION SYMBOLS

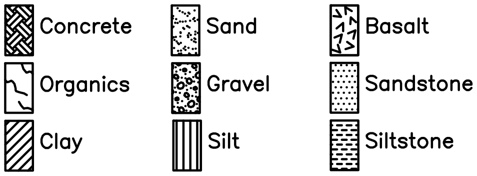
G – Gravel
S – Sand
M – Silt
C – Clay
Pt – Peat

W – Well Graded
P – Poorly Graded
L – Low Plasticity
H – High Plasticity
O – Organic

FIELD SHEAR STRENGTH TEST

Shear strength measurements on test pit side walls, blocks of soil or Shelby tube samples are typically made with Torvane or Field Vane shear devices.

TYPICAL SOIL/ROCK SYMBOLS



WATER TABLE

▼ Water Table Location

(1/31/16) Date of Measurement

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SYMBOL KEY

EXPLORATION LOGS

Explanation of Common Terms Used in Soil Descriptions

Field Identification	Cohesive Soils			Granular Soils	
	SPT*	S_u** (tsf)	Term	SPT*	Term
Easily penetrated several inches by fist.	0 – 2	< 0.125	Very Soft	0 – 4	Very Loose
Easily penetrated several inches by thumb.	2 – 4	0.125–0.25	Soft	4 – 10	Loose
Can be penetrated several inches by thumb with moderate effort.	4 – 8	0.25 – 0.50	Medium Stiff	10 – 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	8 – 15	0.50 – 1.0	Stiff	30 – 50	Dense
Readily indented by thumbnail.	15 – 30	1.0 – 2.0	Very Stiff	> 50	Very Dense
Indented with difficulty by thumbnail.	>30	> 2.0	Hard		

* SPT N–value in blows per foot (bpf)
** Undrained shear strength

Term	Soil Moisture Field Description
Dry	Absence of moisture. Dusty. Dry to the touch.
Damp	Soil has moisture. Cohesive soils are below plastic limit and usually moldable.
Moist	Grains appear darkened, but no visible water. Silt/clay will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grain surfaces. Sand and cohesionless silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is wetter than the optimum moisture content and above the plastic limit.

Term	PI	Plasticity Field Test
Non–plastic	0 – 3	Cannot be rolled into a thread at any moisture.
Low Plasticity	3 – 15	Can be rolled into a thread with some difficulty.
Medium Plasticity	15 – 30	Easily rolled into thread.
High Plasticity	> 30	Easily rolled and re–rolled into thread.

Term	Soil Structure Criteria
Stratified	Alternating layers at least ¼ inch thick.
Laminated	Alternating layers less than ¼ inch thick.
Fissured	Contains shears and partings along planes of weakness.
Slickensided	Partings appear glossy or striated.
Blocky	Breaks into small lumps that resist further breakdown.
Lensed	Contains pockets of different soils.

Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

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COMMON TERMS

SOIL DESCRIPTIONS

Explanation of Common Terms Used in Rock Descriptions

Field Identification		UCS (psi)	Strength	Hardness (ODOT)
Indented by thumbnail.	R0	< 100	Extremely Weak	Extremely Soft
Crumbles under firm blows with geological hammer, can be peeled by a pocket knife.	R1	100–1,000	Very Weak	Very Soft
Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with geological hammer.	R2	1,000–4,000	Weak	Soft
Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow of geological hammer.	R3	4,000–8,000	Medium Strong	Medium Hard
Specimen requires more than one blow of geological hammer to fracture it.	R4	8,000–16,000	Strong	Hard
Specimen requires many blows of geological hammer to fracture it.	R5	>16,000	Very Strong	Very Hard

Term	Weathering Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric.
Moderately Weathered	Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Highly Weathered (Predom. Decomp.)	Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident (relict texture). May be reduced to soil with hand pressure.

Spacing (meters)	Spacing	Spacing Term	Bedding/Foliation
< 0.06	< 2 in.	Very Close	Very Thin (Laminated)
0.06 – 0.30	2 in. – 1 ft.	Close	Thin
0.30 – 0.90	1 ft. – 3 ft.	Moderately Close	Medium
0.90 – 3.0	3 ft. – 10 ft.	Wide	Thick
> 3.0	> 10 ft.	Very Wide	Very Thick (Massive)

Vesicle Term	Volume
Some vesicles	5 – 25%
Highly vesicular	25 – 50%
Scoriaceous	> 50%

Stratification Term	Description
Lamination	<1 cm (0.4 in.) thick beds
Fissile	Preferred break along laminations
Parting	Preferred break parallel to bedding
Foliation	Metamorphic layering and segregation of minerals

RQD %	Designation	RQD %	Designation
0 – 25	Very Poor	75 – 90	Good
25 – 50	Poor	90 – 100	Excellent
50 – 75	Fair		

Rock Quality Designation (RQD) is the cumulative length of intact pieces 4 inches or longer excluding breaks caused by drilling and handling divided by run length, expressed as a percentage.

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES

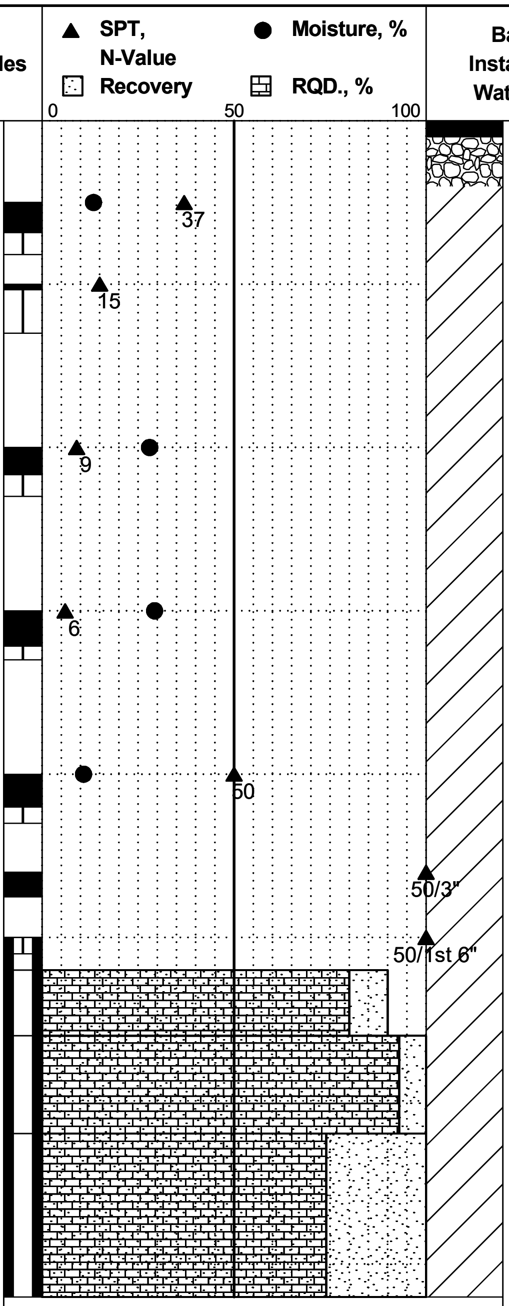
820 NW CORNELL AVENUE
CORVALLIS, OR 97330
BUS. (541) 757-7645

7857 SW CIRRUS DRIVE, BUILDING 24
BEAVERTON, OR 97008
BUS. (503) 641-1541

COMMON TERMS

ROCK DESCRIPTIONS

Soil and Rock Description and Comments	Elev. Depth	Samples
	369	
ASPHALTIC CONCRETE (±6 inches).	368.5 0.0 0.5	
CRUSHED ROCK (GP); dark grey, damp, dense, ±1½-inch minus angular rock, (base rock).	366.5 2.5	
Silty gravelly SAND (SM); dark grey, low plasticity silt, damp, dense, fine to coarse sand, fine to coarse angular gravel, (fill). Lost ±20 gallons of drilling fluid at ±5 feet.	361.5 7.5	SS-1-1 SS-1-2
Silty SAND (SM); brown, low plasticity silt, damp, loose, fine sand, (alluvium).	351.0 18.0	SS-1-3 SS-1-4
Silty GRAVEL, some sand (GM); brown, low plasticity silt, wet, dense, fine to coarse sand, fine to coarse subrounded to rounded gravel, (alluvium).	347.0 22.0	SS-1-5
Silty SANDSTONE; grey, extremely soft (R0), highly weathered, (Continental Sedimentary Rocks). Moderately weathered below ±25 feet.	342.8 26.2	SS-1-6 SS-1-7 CS-1-1
Silty SANDSTONE; grey, very soft to soft (R1 to R2), slightly weathered to fresh, close joints are planar, smooth, and open, very thin bedding, silt and sand laminations, fine sand, some mica, (Continental Sedimentary Rocks).	336.3 32.7	CS-1-2 CS-1-3 CS-1-4
VOLCANICLASTIC SANDSTONE; dark grey, very soft to soft (R1 to R2), slightly weathered to fresh, close to moderately close joints are irregular, rough, and open, find sand matrix, medium to coarse angular to subangular sand-sized volcanic clasts, (Oligocene Sedimentary Rocks). Very soft (R1) siltstone interbed from ±33.4 to 35.8 feet.	333.0 36.0	
BOTTOM OF BORING		



Capped with AC cold patch and gravel

Backfilled with bentonite chips

Project No.: 2181118

Surface Elevation: 369.0 feet (Approx.)

Date of Boring: November 20, 2018

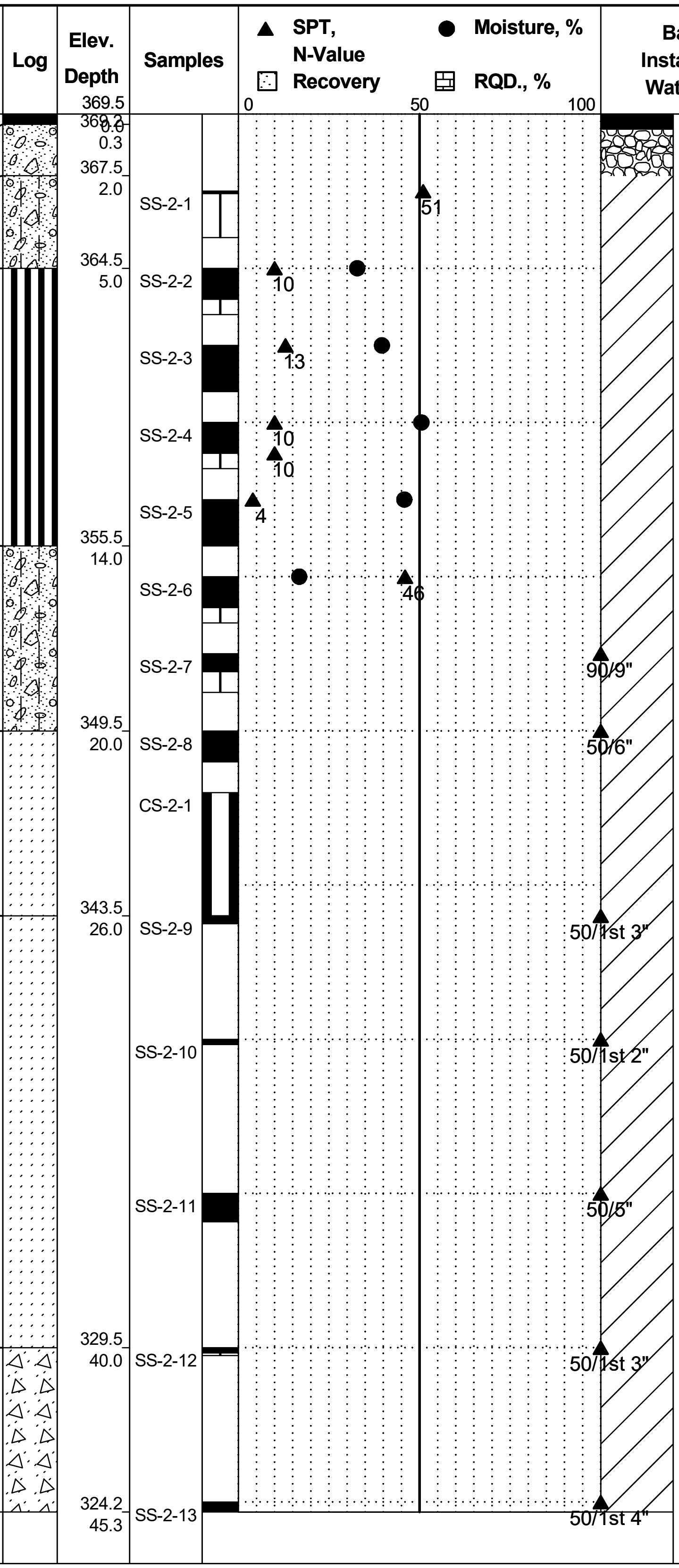
Foundation Engineering, Inc.

Boring Log: BH-1

Thomas Creek, Richardson Gap Road

(Shimanek) Covered Bridge

Linn County, Oregon



Depth Feet	Soil and Rock Description and Comments	Elev. Depth
0	ASPHALTIC CONCRETE (±4 inches).	369.5 0.0
0.3	CRUSHED ROCK (GP); grey, damp, dense, ±1½-inch minus angular rock, (base rock).	369.2 0.3
2	Silty GRAVEL, some sand (GM); dark grey, low plasticity silt, damp, very dense, fine to coarse sand, fine to coarse angular gravel, (fill).	367.5 2.0
5	Clayey SILT (MH); brown, medium to high plasticity, damp, stiff, (alluvium).	364.5 5.0
8	Trace fine to medium sand below ±7.5 feet.	
11	Scattered organics consisting of charcoal flecks and moist below ±10 feet.	
13	Soft and sandy below ±12.5 feet.	
14	Silty GRAVEL, some sand (GM); brown, low plasticity silt, wet, dense to very dense, fine to coarse sand, fine to coarse subrounded to rounded gravel, (alluvium).	355.5 14.0
18	Sidewall caving from ±15 to 20 feet.	
20	Silty SANDSTONE; grey, extremely soft (R0), highly weathered, fine sand, some mica, (Continental Sedimentary Rocks).	349.5 20.0
26	Silty SANDSTONE; grey, very soft (R1), slightly weathered, fine sand, (Continental Sedimentary Rocks).	343.5 26.0
40	VOLCANICLASTIC SANDSTONE; dark grey, very soft to soft (R1 to R2), slightly weathered, (Oligocene Sedimentary Rocks).	329.5 40.0
45.3	BOTTOM OF BORING	324.2 45.3

Samples	SPT, N-Value
SS-2-1	51
SS-2-2	10
SS-2-3	13
SS-2-4	10 10
SS-2-5	4
SS-2-6	46
SS-2-7	90/9"
SS-2-8	50/6"
CS-2-1	
SS-2-9	50/1st 3"
SS-2-10	50/1st 2"
SS-2-11	50/5"
SS-2-12	50/1st 3"
SS-2-13	50/1st 4"

Project No.: 2181118

Surface Elevation: 369.5 feet (Approx.)

Date of Boring: November 20, 2018

Foundation Engineering, Inc.

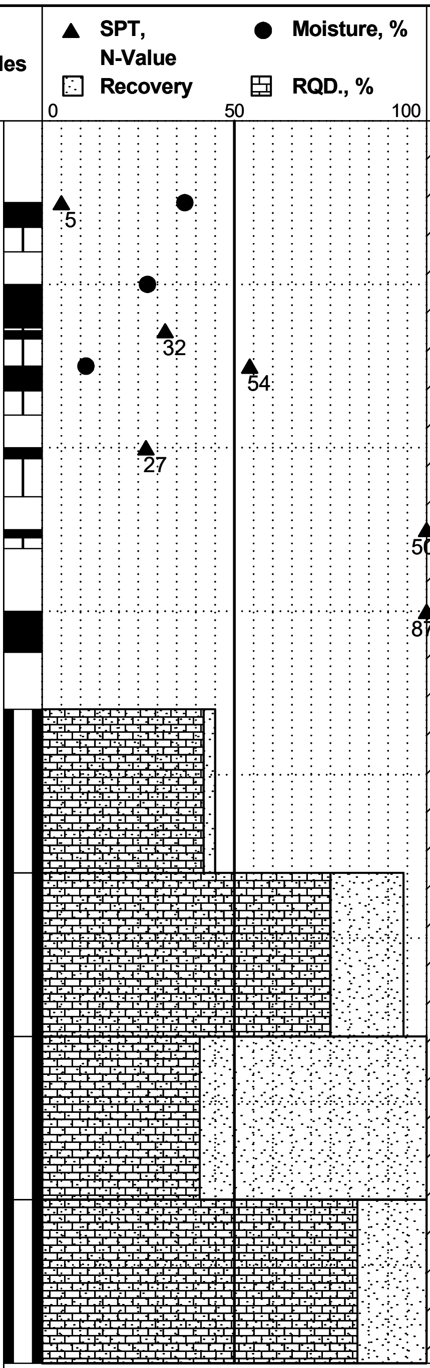
Boring Log: BH-2

Thomas Creek, Richardson Gap Road

(Shimanek) Covered Bridge

Linn County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	Backfill/ Installations/ Water Table
		359 0.0		
1	CRUSHED ROCK (GP) (±12 inches); grey, moist, medium dense, ±1-inch minus angular rock, (fill).	358.0 1.0		Backfilled with bentonite chips
2	Silty SAND, scattered organics (SM); brown, low plasticity silt, damp, loose, fine sand, organics consist of wood debris, (alluvium).		SS-3-1 SH-3-2	
7	Silty GRAVEL, some sand (GM); brown, low plasticity silt, wet, dense to very dense, fine to coarse sand, fine to coarse subrounded to rounded gravel, (alluvium).	352.5 6.5	SS-3-3 SS-3-4	
11	Sandy GRAVEL, some silt (GP-GM); brown, wet, low plasticity silt, wet, medium dense, fine to coarse sand, fine to coarse subrounded gravel, (alluvium). Very dense below ±12.5 feet. Sidewall caving from ±10 to 15 feet.	349.0 10.0	SS-3-5 SS-3-6	
16	Silty SANDSTONE; grey, extremely soft (R0), highly weathered, fine sand, some mica, (Continental Sedimentary Rocks).	343.5 15.5	SS-3-7 CS-3-1	
21	Silty SANDSTONE; grey, very soft (R1), slightly weathered, close to moderately close joints are planar, smooth, and open, very thin bedding, silt and sand laminations, fine sand, some mica, (Continental Sedimentary Rocks).	338.2 20.8	CS-3-2	
29	VOLCANICLASTIC SANDSTONE; dark grey, very soft to soft (R1 to R2), slightly weathered to fresh, very close to close joints are planar to irregular, smooth to rough, and open, fine sand matrix, medium to coarse angular to subangular sand-sized volcanic clasts, (Oligocene Sedimentary Rocks). Very close joints from ±29.4 to 29.7 feet. Some calcite veining from ±29.4 to 29.9 feet. Organics at ±37.3 feet. Matrix of fine to medium sand below ±37.8 feet. BOTTOM OF BORING	330.5 28.5 321.0 38.0	CS-3-3 CS-3-4	



Project No.: 2181118

Surface Elevation: 359.0 feet (Approx.)

Date of Boring: November 21, 2018

Foundation Engineering, Inc.

Boring Log: BH-3

Thomas Creek, Richardson Gap Road

(Shimanek) Covered Bridge

Linn County, Oregon

Photo 1B. BH-1 - Box 1

Photo 2B. BH-1 - Box 2

Photo 3B. BH-3 - Box 1

Photo 4B. BH-3 - Box 2

Photo 5B. BH-3 - Box 3

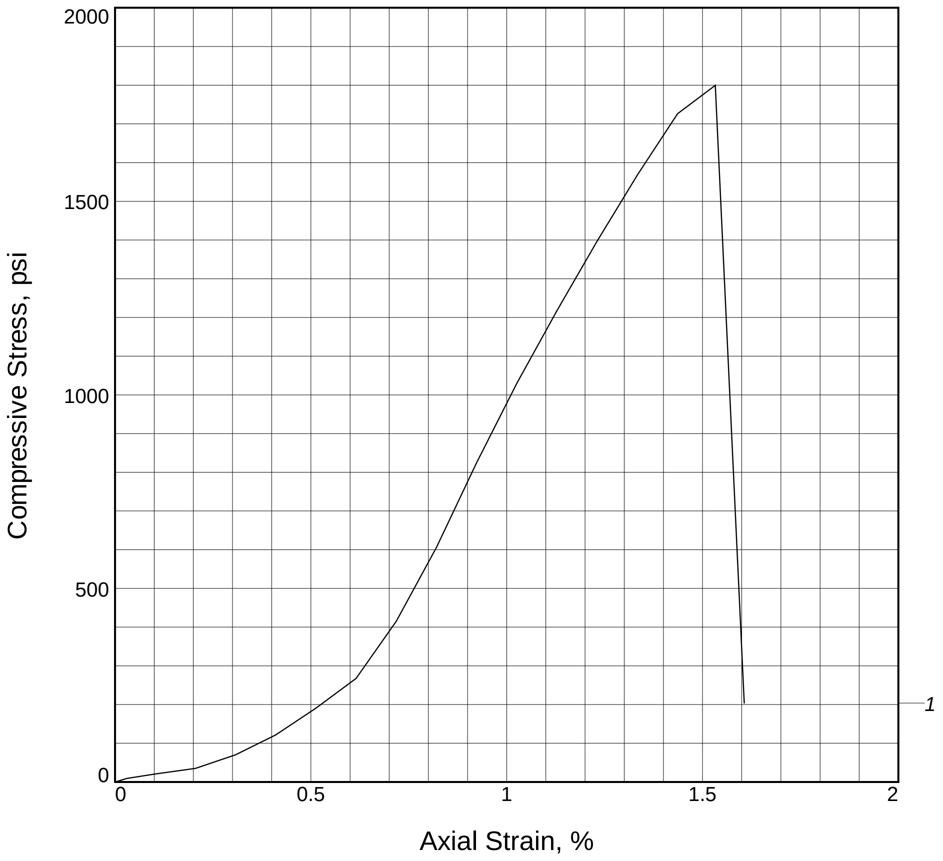
Appendix C

Field and Laboratory Test Results

Professional
Geotechnical
Services

Foundation Engineering, Inc.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	1799.60			
Undrained shear strength, psi	899.80			
Failure strain, %	1.5			
Strain rate, in./min.	0.005			
Water content, %	31.1			
Wet density, pcf	118.2			
Dry density, pcf	90.2			
Saturation, %	98.6			
Void ratio	0.8347			
Specimen diameter, in.	2.01			
Specimen height, in.	4.87			
Height/diameter ratio	2.43			

Description: Grey, soft (R2) silty sandstone

LL =	**PL =**	**PI =**	**Assumed GS=** 2.65	**Type:**

Project No.: 2186001-628
Date Sampled: 11-20-18
Remarks:

Figure 1C

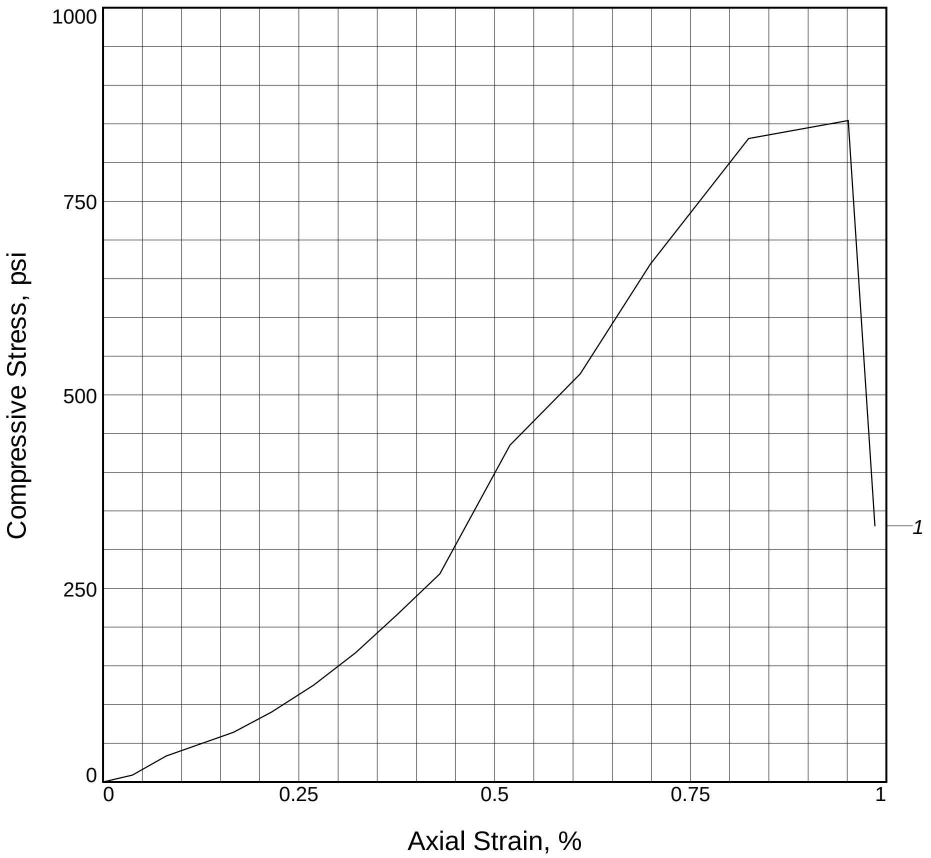
Client: Foundation Engineering, Inc.; Project No. 2181118

Project: Thomas Cr., Richardson Gap Rd. (Shimanek) Covered Bridge

Source of Sample: 6416 **Depth:** 27.2-27.6'
Sample Number: CS-1-2

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	854.37			
Undrained shear strength, psi	427.19			
Failure strain, %	1.0			
Strain rate, in./min.	0.006			
Water content, %	33.9			
Wet density, pcf	112.6			
Dry density, pcf	84.1			
Saturation, %	92.9			
Void ratio	0.9667			
Specimen diameter, in.	2.39			
Specimen height, in.	5.58			
Height/diameter ratio	2.33			

Description: Grey, very soft (R1) silty sandstone

LL =	**PL =**	**PI =**	**Assumed GS=** 2.65	**Type:**

Project No.: 2186001-628
Date Sampled: 11-20-18
Remarks:

Figure 2C

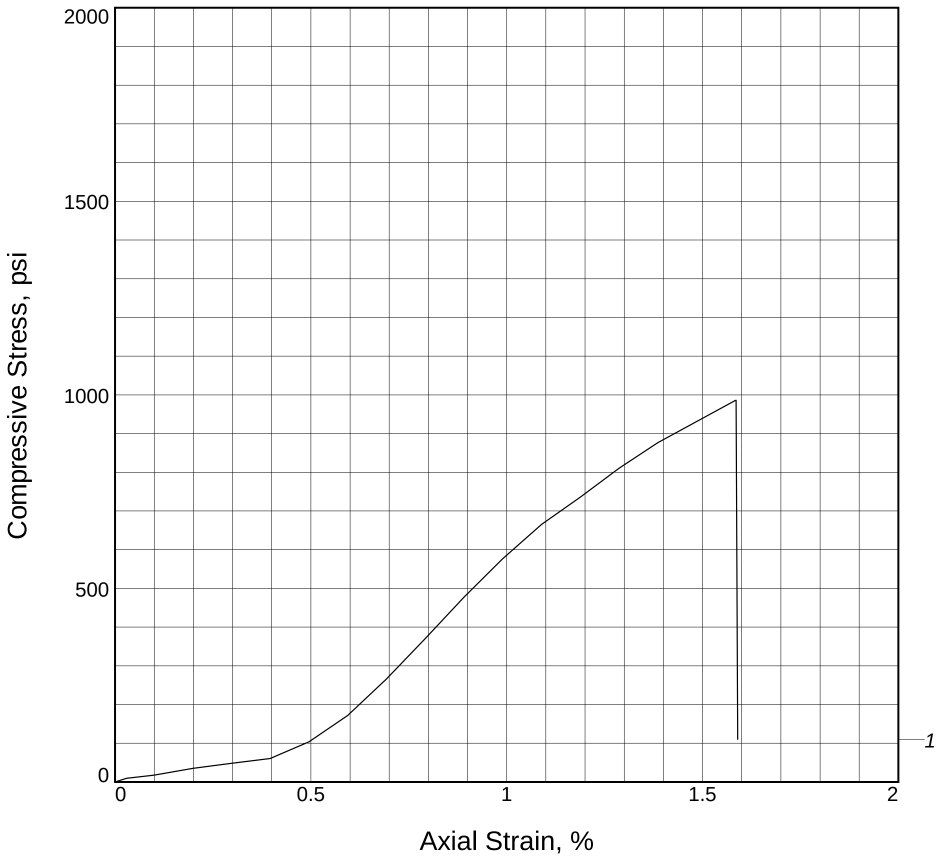
Client: Foundation Engineering, Inc.; Project No. 2181118

Project: Thomas Cr., Richardson Gap Rd. (Shimanek) Covered Bridge

Source of Sample: 6416 **Depth:** 31.0-31.5'
Sample Number: CS-1-4

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	985.84			
Undrained shear strength, psi	492.92			
Failure strain, %	1.6			
Strain rate, in./min.	0.005			
Water content, %	30.0			
Wet density, pcf	118.7			
Dry density, pcf	91.3			
Saturation, %	98.0			
Void ratio	0.8115			
Specimen diameter, in.	2.39			
Specimen height, in.	5.05			
Height/diameter ratio	2.12			

Description: Grey, very soft (R1) silty sandstone

LL =	**PL =**	**PI =**	**Assumed GS=** 2.65	**Type:**

Project No.: 2186001-628

Date Sampled: 11-20-18

Remarks:

Figure 3C

Client: Foundation Engineering, Inc.; Project No. 2181118

Project: Thomas Cr., Richardson Gap Rd. (Shimanek) Covered Bridge

Source of Sample: 6416 **Depth:** 20.8-21.3'

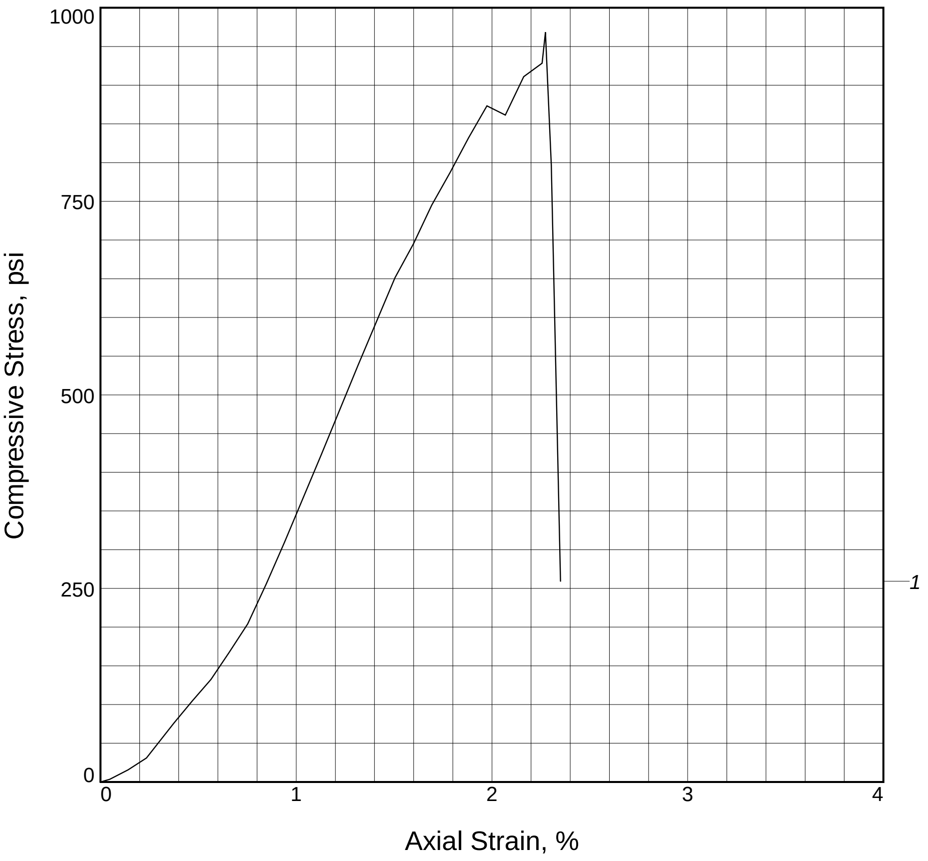
Sample Number: CS-3-1

UNCONFINED COMPRESSION TEST

FEI Testing & Inspection, Inc.

Corvallis, OR

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	968.37			
Undrained shear strength, psi	484.19			
Failure strain, %	2.3			
Strain rate, in./min.	0.005			
Water content, %	33.8			
Wet density, pcf	113.0			
Dry density, pcf	84.4			
Saturation, %	93.5			
Void ratio	0.9590			
Specimen diameter, in.	2.40			
Specimen height, in.	5.32			
Height/diameter ratio	2.22			

Description: Grey, very soft (R1) silty sandstone

LL =	**PL =**	**PI =**	**Assumed GS=** 2.65	**Type:**

Project No.: 2186001-628

Date Sampled: 11-20-18

Remarks:

Figure 4C

Client: Foundation Engineering, Inc.; Project No. 2181118

Project: Thomas Cr., Richardson Gap Rd. (Shimanek) Covered Bridge

Source of Sample: 6416 **Depth:** 23.0-23.5'

Sample Number: CS-3-2

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

Table 1C. Moisture Content, Atterberg Limits and Percent Fines

Sample Number	Sample Depth (feet)	Moisture Content (%)	LL	PL	PI	Fines (%)	USCS Classification
SS-1-1	2.5 – 4.0	13.4					
SS-1-3	10.0 – 11.5	28.0				43.1	
SS-1-4	15.0 – 16.5	29.3				30.0	
SS-1-5	20.0 – 21.5	10.8					
SS-2-2	5.0 – 6.5	32.8					
SS-2-3	7.5 – 9.0	39.6	73	42	31		MH
SS-2-4	10.0 – 11.5	50.5					
SS-2-5	12.5 – 14.0	45.8				61.6	
SS-2-6	15.0 – 16.5	16.8					
SS-3-1	2.5 – 4.0	37.1				44.1	
SH-3-2	5.0 – 6.4	27.4					
SS-3-4	7.5 – 9.0	11.4					

Table 2C. Summary of DCP Test Results (ASTM D6591)

Exploration	Initial Test Depth (inches)	Soil Description	[1]Average DCP (mm/blow)	[2]Average M_r (psi)	[3]Corrected M_r (psi)
BH-1	9.0	CRUSHED ROCK (GP)	30.	31,840	19,741
BH-2	7.0	CRUSHED ROCK (GP)	3.3	30,809	19,102
	24.0	Silty GRAVEL, some sand (GM) (approach fill)	5.2	25,784	15,986

Notes:
1. DCP (mm/blow) based on the average readings from the initial test depth.
2. M_r value based on average DCP value at the test depth and the ODOT recommended correlation: $M_r = C_f\ 49{,}023\ (DCP)^{-0.39}$. Values may vary slightly due to rounding.
3. C_f (correction factor) is based on the ODOT recommended value of 0.62 for base rock and approach fill.

Table 3C. Summary of Resistivity Testing (ASTM G57)

Location	Pin Spacing (ft)	Resistivity (Ω-cm)
North of BH-3	5	6,606
	10	9,958
	15	11,203

Table 4C. pH Test Results (ASTM G51)

Sample Number	Sample Depth (ft)	Sample Description	pH
SS-1-1	10.0 – 11.5	Silty SAND	6.3
SS-2-3	7.5 – 9.0	Clayey SILT	6.5
SS-2-5	12.5 – 14.0	Sandy SILT	6.2