Foundation Report

Mill Creek – Folsom Road Bridge Replacement
Bridge No. 12792, Key No. 20306

Linn County, Oregon

Prepared for:

Linn County Road Department
Albany, Oregon

July 7, 2021

Professional Geotechnical Services

Foundation Engineering, Inc.

Foundation Engineering, Inc.
Professional Geotechnical Services

Kevin Groom, P.E.
Linn County Road Department
3010 Ferry Street SW
Albany, Oregon 97322

July 7, 2021

Mill Creek – Folsom Road Bridge Replacement
Bridge No.: 12792, Key No.: 20306
Foundation Report
Linn County, Oregon

Project No.: 2211021

Dear Mr. Groom:

We have completed the requested geotechnical investigation for the proposed replacement of the Mill Creek – Folsom Road Bridge in Linn County, Oregon. Our report includes a description of our work, discussion of the site conditions, summary of laboratory testing, and discussion of engineering analyses. Recommendations are included for site preparation, bridge foundation design and approach pavements.

This report was prepared to conform to the Oregon Department of Transportation (ODOT) Geotechnical Design Manual (GDM) (ODOT, 2018) and the ODOT Pavement Design Guide (PDG) (ODOT, 2019). Construction recommendations refer to sections in the Oregon Standard Specifications for Construction (ODOT, 2021).

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

REGISTERED PROFESSIONAL ENGINEER
60776PE
DIGITALLY SIGNED 2021.09.15 09:19:20 -07'00'
OREGON
MAY 17, 1999
WILLIAM L. NICKELS JR.
RENEWS: 12-31-2022

MDM/WLN/mm
enclosures

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

FOUNDATION REPORT

MILL CREEK – FOLSOM ROAD BRIDGE REPLACEMENT
LINN COUNTY, OREGON

1.0. INTRODUCTION

1.1. Project Description

The Linn County Road Department (Linn County) is planning to replace the bridge crossing Mill Creek at Milepost (MP) 0.65 on Folsom Road. The site location is provided on Figure 1A (Appendix A).

The existing bridge is a ±53-foot long, three-span structure supported on timber piling. Preliminary plans indicate the replacement bridge will be a 63.75-foot long, single-span, prestressed concrete bridge constructed along the same alignment. The site layout of the existing bridge with an overlay of the proposed replacement structure is provided on Figure 2A (Appendix A).

Linn County is the project owner, and Foundation Engineering, Inc. was retained by the County as the geotechnical consultant. Our scope of work was summarized in Exhibit A of the Engineering and Related Services Contract (County Project No. CB1801) dated March 1, 2021.

1.2. Purpose and Scope

The purpose of the investigation was to develop recommendations for the design and construction of the replacement structure foundations and 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 available geologic maps and water well logs 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 within the southern extent of the Willamette Valley, between the Coast Range and Cascade Range. The site is located on Mill Creek which flows to the north to the confluence with the South Santiam River ±2 miles north of the project site. However, due to the meandering nature of the South Santiam River, the closest point to the river from the site is less than a mile east.

Local geologic mapping indicates the site is underlain by alluvial deposits associated with the South Santiam River (Beaulieu et al., 1974; Walker and Duncan, 1989; Yeats et al., 1996). We estimate deep alluvial deposits in this portion of the Valley. Based on local water well logs, deposits of clay, sand, and gravel extend to ±82 feet, the maximum depth of the exploration. The local geologic mapping and cross-sections suggest the alluvial deposits are underlain by basaltic flow rock and volcanics of the Little Butte Formation at an unknown depth (Beaulieu et al., 1974).

The subsurface conditions encountered in our borings are consistent with the mapped geology. Details are provided in the Subsurface Conditions section of this report and on the boring logs provided in Appendix B.

2.2. Seismicity and Faulting

We completed a literature review of nearby faults to evaluate the seismic setting and identify the potential seismic sources. The USGS website includes an interactive deaggregation tool, which allows evaluation of the contribution of the various seismic sources to the overall seismic hazard (USGS, 2014). The USGS interactive deaggregation indicates the seismic hazard at the site is dominated by the Cascadia Subduction Zone (CSZ) (USGS, 2014). Crustal fault sources also represent a small percentage of the seismic hazard. A discussion of these earthquake sources is provided below.

2.2.1. Cascadia Subduction Zone. The site is ±120 miles east of the surface expression of the Cascadia Subduction Zone (CSZ). The CSZ is a converging, oblique plate boundary where the Juan de Fuca plate is being subducted beneath the western edge of the North American continent. The CSZ extends ±700 miles from central Vancouver Island in British Columbia, Canada through Washington and Oregon to Northern California (Atwater, 1970).

Available information indicates the CSZ is capable of generating earthquakes within the descending Juan de Fuca plate (intraplate) and along the inclined interface between the two plates (interface) (Weaver and Shedlock, 1996). CSZ intraslab earthquakes are estimated to have a moment magnitude (M_w) between 6.9 and 7.2, and CSZ interface earthquakes are estimated to have a M_w between 8 and 9.2.

The most recent CSZ interface earthquake occurred ±321 years ago on January 26, 1700 (Nelson et al., 1995; Satake et al., 1996). A 2012 study of turbidites from the last ±10,000 years suggests the return period for CSZ interface earthquakes varies with location and rupture length (Goldfinger et al., 2012). That study estimated an average recurrence interval of ±220 to 380 years for a CSZ 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. More recent research for the northern portion of the subduction zone suggests a recurrence interval of ±340 years for the northern Oregon Coast (Goldfinger et al., 2016).

No CSZ intraslab earthquakes have been recorded in Oregon in modern times. However, the Puget Sound region of Washington State has experienced three CSZ intraslab events in the last ±72 years, including a surface wave magnitude (M_s) 7.1 event in 1949 (Olympia), a M_s 6.5 event in 1965 (Seattle/Tacoma) (Wong and Silva, 1998), and a M_w 6.8 event in 2001 (Nisqually) (Dewey et al., 2002).

2.2.2. Crustal Faults. Crustal earthquakes occur within the North American Plate, typically at depths of ±6 to 12 miles, and dominate Oregon's seismic history. USGS classifies the crustal faults as follows (Personius et al., 2003):

- ***Class A*** - Faults with geologic evidence supporting tectonic movement in the Quaternary known or presumed to be associated with large-magnitude earthquakes.
- ***Class B*** - Faults with geologic evidence that demonstrates the existence of a fault or suggests Quaternary deformation, but either: 1) the fault might not extend deep enough to be a potential source of significant earthquakes or 2) the current evidence is too strong to confidently classify the fault as a Class C but not strong enough to classify it as a Class A.
- ***Class C*** – Faults with insufficient evidence to demonstrate 1) the existence of a tectonic fault, or 2) Quaternary movement or deformation associated with the feature.
- ***Class D*** – Geologic evidence indicates the feature is not a tectonic fault.

Geologic maps and the USGS interactive maps indicate no faults are mapped beneath the site (Beaulieu et al., 1974; Walker and Duncan, 1989; Yeats et al., 1996; USGS, 2006b). Five crustal faults have been mapped within ±10 miles of the site; however, only the Turner and Mill Creek faults (Class A) show any evidence of movement in the last ±1.6 million years (Palmer, 1983; Yeats et al., 1996; USGS, 2006a, b).

3.0. SUBSURFACE EXPLORATION AND CONDITIONS

3.1. Exploration

Two exploratory boreholes (BH-1 and BH-2) were drilled at the site on April 1 and 2, 2021, using a CME 75, truck-mounted drill rig utilizing mud-rotary drilling methods. BH-1 was drilled in the westbound travel lane, ±4 feet west of the existing west abutment. BH-2 was drilled in the westbound travel lane, ±7 feet east of the existing east abutment. The approximate borehole locations are shown on Figure 2A.

Disturbed soil samples were obtained in each boring using a 2-inch diameter, split-spoon sampler at ±2.5-foot intervals to ±15 to 20 feet and at ±5-foot intervals thereafter. The Standard Penetration Test (SPT) (ASTM D1586), which is performed when the split-spoon is driven, provides an indication of the relative stiffness or density of the soil. The number of blows required to drive the sampler the final 12 inches of an 18-inch long drive is recorded and represents the standard penetration resistance or N-value in blows per foot (bpf). A relatively undisturbed sample was also obtained at ±10 feet in BH-2 by pushing a thin-walled Shelby tube (ASTM D1587). The samples collected were sealed to avoid moisture loss and transported to our office for further examination and potential testing.

The borings were continuously logged during drilling by a Foundation Engineering representative. 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. The boring logs are provided in Appendix B.

3.2. Subsurface Conditions

The following provides a general discussion of the subsurface conditions encountered in the borings. Additional details are provided in the boring logs (Appendix B). The elevations shown on the logs were estimated from the preliminary plan and elevation sheet provided by Linn County.

3.2.1. ***<u>Bent 1 (West Abutment) – BH-1</u>***. The pavement surface elevation at BH-1 is ±El. 263. The pavement section consists of ±3 inches of asphaltic concrete pavement (ACP) over ±16 inches of medium dense crushed rock (base rock). The base rock is underlain by fill consisting of very loose silty sand to ±5 feet, followed by very loose gravel with a trace of silt and sand to ±6.5 feet (±El. 256.5).

The fill is underlain by alluvium to ±60.5 feet (±El. 202.5), the limits of the exploration. The alluvium includes medium dense silty sand to ±9 feet, followed by gravel with variable amounts of silt and sand to ±60.5 feet. The gravel was medium dense from ±9 to 15 feet, dense to very dense from ±15 to 35 feet, and very dense below ±35 feet (±El. 228.0).

3.2.2. ***<u>Bent 2 (East Abutment) – BH-2</u>***. The pavement surface elevation at BH-2 is ±El. 262.5. The pavement section consists of ±3.5 inches of ACP over ±15 inches of medium dense crushed rock (base rock). The base rock is underlain by fill consisting of loose silty sand to ±5 feet (±El. 257.5).

The fill is underlain by alluvium to ±75.3 feet (±El. 187.2), the limits of the exploration. The alluvium includes very loose to loose silty sand to 13.5 feet (±El. 249) followed by gravel with variable amounts of silt and sand to ±75.3 feet. The gravel is medium dense from ±13.5 feet to 30 feet and very dense below ±30 feet (±El. 232.5).

3.3. Groundwater

Mud-rotary drilling precluded an accurate determination of the groundwater level in the borings at the time of drilling. However, the water level in Mill Creek, as measured from the existing bridge, was ±11.3 feet (±El. 251.5) below the road surface on April 1, 2021. We anticipate the groundwater level in the vicinity of the bridge fluctuates seasonally and corresponds approximately to the water level in the creek.

4.0. FIELD AND LABORATORY TESTING

4.1. Laboratory Testing

Laboratory testing included moisture content determinations (ASTM D2216) and percent fines (ASTM D1140) tests to classify the soils and estimate their overall engineering properties. The results are summarized in Table 1C (Appendix C). The moisture contents are also shown on the appended logs.

4.2. DCP Testing

In-situ, Dynamic Cone Penetrometer (DCP) testing (ASTM D6951) was completed in conjunction with the drilling to estimate the resilient modulus (M_R) of the subgrade 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 (2019) 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 Miller 400A 4-pin soil resistance meter (ASTM G57). The resistivity test was completed ±20 feet north of the east abutment along the north road shoulder. 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).

The soil samples were generally too coarse for pH testing. A pH test (ASTM G51) was completed on sample SS-1-1 at a depth of ±2.5 to 4 feet. The results, summarized in Table 4C (Appendix C), indicate a neutral soil condition with a pH value of 6.7.

5.0. HYDRAULICS/SCOUR

Due to low flows, a hydraulic study is not planned for this project. We understand potential scour at the bridge abutments is not a design consideration.

6.0. SEISMIC ANALYSIS AND EVALUATION

6.1. Bedrock Acceleration and Site Response

Response spectra for the bridge site was developed based on the current ODOT Geotechnical Design Manual (GDM) "life-safety" and "operational" criteria (ODOT, 2018). The "life-safety" (i.e., no collapse) seismic performance criteria is based on probabilistic earthquake ground motions having a 1,000-year average return period. The "operational" (i.e., remain in service) criteria is based on a full-rupture Cascadia Subduction Zone Earthquake (CSZE).

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 (AASHTO, 2017) with modifications recommended in the 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 (Petersen 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 site correspond most closely to a Site Class D. The scaling factors were selected based on ODOT GDM Tables 6.2-A, 6.2-B, and 6.2-C. The response spectra and design parameters are shown on Figure 3A (Appendix A).

The ground motions for the CSZE operational response spectrum were obtained using the Portland State University (PSU) Acceleration Response Spectra website (PSU, 2017). We inputted the latitude and longitude coordinates for the project site and an estimated an average shear wave velocity for the upper 30 meters (V_{s30}) of the soil profile. We estimated a V_{s30} of 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 or low plasticity silt subjected to intense ground shaking. Predominantly very loose to loose silty sand was encountered from ±1.6 to 9 feet (±El. 2161.4 to El. 354.0) in BH-1 and from ±1.5 to 13.5 feet (±El. 261 to El. 249.0) in BH-2. However, due to the fines content within this layer, 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 in the creek. If liquefaction were to occur during the design earthquake, the material would densify and result in minor approach fill settlement.

The replacement bridge structure will be supported on deep foundations that bypass the silty sand and extend into very dense gravel that is resistant to seismically-induced liquefaction. Therefore, liquefaction-induced settlement of the structure foundations is not a design concern.

7.0. FOUNDATION ANALYSIS AND DESIGN RECOMMENDATIONS

7.1. Discussion of Proposed Foundations

Deep foundations (i.e., driven piles or drilled shafts) are recommended for the new bridge based on the subsurface conditions encountered in the borings. Driven piles are recommended since they cost less and are easier to install compared to drilled shafts. Driven, PP12.75x0.375 pipe piles were selected in consultation with Linn County. Details of pile analysis are provided below.

7.2. Foundation Loads

The new bridge will be supported on a row of five piles per bent with a center-to-center spacing of 7.5 feet. Linn County provided a factored (Strength I) load of 187.4 kips/pile.

7.3. Driven Pile Analysis and Design

7.3.1. Pile Type and Material Specifications. Recommendations presented herein assume PP12.75x0.375 (ASTM A252, Grade 3) sections will be used. We recommend driving the piles open-ended with inside-fitting cutting shoes to facilitate penetration to the minimum tip elevation and the formation of a soil plug, and to reduce the risk of tip damage in the dense to very dense gravel. The recommended pile properties are summarized in Table 1.

Table 1. PP12.75x0.375 Properties

Steel Grade	ASTM A252 (Grade 3)
Yield Stress (F_y)	45 ksi
Area Steel (A_s)	14.6 in^2
Nominal Structural Resistance[1]	657 kips
End Condition	Open-ended with inside-fitting cutting shoe

Note: Nominal structural resistance (P_n) is calculated as 0.66^{λ} (F_y x A_s), where $\lambda = 0$ for fully embedded piles.

7.3.2. Downdrag. At least ±½ inch of ground settlement around the pile is typically required to induce downdrag on deep foundations following their installation. The plan and profile drawing indicates the new bridge will be constructed along the current alignment with minor approach widening to accommodate the new bridge width. Up to ±2 feet of new fill will be required for the widening. Given the limited fill depth and the dense to very dense soil conditions below the new pile caps, settlement of the approaches is expected to be negligible and downdrag is not a design concern.

7.3.3. ***Nominal and Factored Axial Resistance**.* The borings indicated similar subsurface conditions along the centerline of the bridge. Therefore, for simplicity, we evaluated the nominal and factored axial resistances for driven piles at both abutments using a composite soil profile based on the conditions encountered in BH-1 and BH-2.

Strength parameters for the foundation soils were estimated based on available correlations with the SPT N-values. The nominal axial resistance is based on skin friction along the length of the driven pile and end-bearing at the pile tip. The end-bearing calculation assumed a soil plug will form at a typical depth of ±20d (where d is the pile diameter). The actual depth of the plug formation is expected to vary between piles.

Axial pile analysis was completed using the AASHTO (2017) Load Resistance Factor Design (LRFD) approach. The factored resistances are based on an AASHTO LRFD resistance factor (ϕ) of 0.4, assuming the FHWA Gates equation will be used to establish the final driving criteria per Sections 00520.20(d) and 00520.42(b) of the ODOT Standard Specifications for Construction (ODOT, 2021). A nominal axial resistance of 468.6 kips is required based on the required factored resistance (187.4 kips) and a ϕ factor of 0.4. The estimated nominal and factored axial resistance versus elevation for a PP12.75x0.375 pile section at Bent 1 and Bent 2 are shown in Figure 4A (Appendix A).

7.3.4. ***Minimum/Estimated Pile Tip Elevations**.* The minimum tip elevation was selected at a tip depth of ±20d, where a soil plug will typically form in relatively small diameter piles. We typically observe a significant increase in the driving resistance once the soil plug forms, and soon after the piles should achieve the required axial resistance of ±187 kips. The estimated tip elevation was selected based our calculations of where the pile will achieve required factored axial resistance (see Figure 4A). The minimum and estimated tip elevations are provided in Table 2.

Finished pile lengths were estimated based on the estimated tip elevations, and the pile cut-off elevations provided by Linn County. The finished pile lengths do not include additional stickup required for driving. We presume the contractor will select the delivered lengths to provide adequate stickup for driving. The bottom of cap, pile cut-off elevations, and estimated pile lengths are also provided in Table 2.

Table 2. Minimum/Estimated Tip Elevations and Estimated Finished Pile Lengths

Bent	Bottom of Cap Elevation [1] (ft)	Cut-Off Elevation[1] (ft)	Min. Tip Elevation (ft)	Est. Tip Elevation (ft)	Est. Finished Pile Length[2] (ft)
1	257.2	258.5	235.0	229.5	29
2	257.2	258.5	235.0	229.5	29

Notes: 1. Bottom of cap and cut-off elevations were provided by Linn County.
2. Estimated finished pile lengths are based on the cut-off and estimated tip elevations. These lengths do not include additional stickup during driving.

7.3.5. Nominal Uplift Resistance. The nominal uplift resistance for a PP12.75x0.375 pile was calculated based on the estimated skin resistance mobilized in the soil above the minimum tip elevation. A nominal uplift resistance of 40 kips per pile is recommended for design.

7.3.6. Pile Settlement. The piles will be driven into very dense gravel which has relatively low compressibility. Therefore, pile settlement is expected to be less than ½ inch, and limited to the elastic compression of the pile, and to the displacement required to mobilize the skin resistance.

7.3.7. Lateral Analysis. We have assumed the lateral analysis, if needed, will be completed by County engineers using the LPILE computer program. The recommended LPILE input parameters for each bent are summarized in Table 1A (Appendix A).

7.3.8. Driving Criteria and Driveability Analysis. The FHWA Gates equation was used to estimate a range of hammer field energies required to drive the piles to a nominal axial resistance of 468.6 kips with a final driving resistance in the range of 2 to 10 blows per inch. The analysis indicates a rated hammer field energy in the range of ±26.4 to 62.3 ft-kips will be required. However, the Oregon Standard Specifications for Construction (ODOT, 2021) Table 00520-1 requires a minimum field energy of 30 ft-kips for the indicated nominal driving resistance. Therefore, the recommended range of rated hammer field energies is 30 to 62.3 ft-kips. The actual final driving resistance should be established using the FHWA Gates equation after the hammer information is submitted by the contractor.

7.3.9. Potential Obstructions. We observed no potential obstructions. However, the contractor should anticipate hard driving upon encountering the dense to very dense gravel stratum. Preboring should not be required. Jetting is not recommended.

7.3.10. Set Period and Redriving. In the event the required axial resistance is not achieved at the estimated tip elevation, the contractor should stop driving and allow the piles to set for a period of at least 24 hours before re-striking.

7.3.11. Tip Protection. We recommend driving the piles open-ended with inside-fitting cutting shoes to facilitate penetration to the minimum tip elevation and the formation of a soil plug, and to reduce the risk of tip damage in the dense to very dense gravel.

8.0. APPROACHES AND EMBANKMENTS

8.1. Embankment Construction and Settlement

The new bridge will be constructed along the same horizontal and vertical alignments. Embankment widening will be required to accommodate the increased width of the new bridge. Based on the topographic data, it appears that up to ±2 feet of new fill be required to widen the approaches. Observation of the materials near the embankment toe and the subsurface conditions encountered in the borings suggest the soils consist of very loose to medium dense silty sand followed by medium dense grading to very dense gravel. Based on these conditions, we expect little or no new settlement of the embankments. Furthermore, if any settlement occurs, we expect it will happen concurrent with fill placement and finish grading prior to paving.

8.2. Approach Pavements

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

8.2.1. Subgrade. The existing approaches include base rock to a depth of ±1.5 feet, followed by embankment fill consisting predominantly of silty sand to ±6.5 feet at the west abutment and to a depth of ±5 feet at the east abutment. DCP testing indicated subgrade resilient moduli (M_R) values of 8,503 psi in BH-1 and 9,711 psi in BH-2. Therefore, a subgrade M_R value of 7,500 psi was selected for design, assuming the subgrade will be compacted during construction. An M_R value of 20,000 psi was assumed for new Base Aggregate, consistent with ODOT PDG (ODOT, 2019) design recommendations.

8.2.2. Traffic Data. The project prospectus indicates an average daily traffic (ADT) of 60 vehicles in 2010 and a projected 2030 ADT of 70 vehicles. These ADT values result in a 20-year expansion factor of 1.17 and an annual growth rate of 0.77%. For design, we used the annual growth rate to estimate the 2022 ADT (assumed project completion date) of 64 vehicles and the 2052 ADT (30-year design life) of 80 vehicles. We assume the data represents 2-way traffic. Therefore, we applied a directional factor of 55% to obtain the design 1-way traffic.

The project prospectus indicates the provided ADT includes 21.4% truck traffic. For design, we assumed the truck traffic would remain consistent and used a range of truck classifications based on ODOT and FHWA. We used the traffic data and ODOT and AASHTO conversion factors to estimate a 30-year Equivalent Single Axle Load (ESAL) value of 114,618.

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

- Reliability of 75%
- 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 7,500 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 7,500 psi was assumed for the subgrade based on available correlations and the results of DCP testing.

The calculations indicate a minimum pavement section of 4 inches of AC over 12 inches of Base Aggregate is required. This is less than the County minimum standard of 6 inches of AC over 12 inches of Base Aggregate. We anticipate the County minimum standard section will be used.

8.3. Abutment Walls and Wing Walls

Drawings provided by Linn County indicate the abutment and wing walls will have a maximum height of 5.5 feet. The wing walls will be perpendicular to the abutments and will extend back 10 feet from the abutment wall. We assume Granular Wall Backfill (Section 00510.12) or Granular Structure Backfill (Section 00510.13) will be used to backfill 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 5.5-foot tall wall, the required deflection is less than ±0.1 inch. Typically, abutment walls deflect to mobilize active earth conditions. However, integral abutment walls or wing wall to abutment wall corners may not be free to deflect. Therefore, we calculated the lateral earth pressures for both the active and the at-rest condition. We assume the structural designer will select the appropriate lateral earth pressure based on the flexibility of the structure. The resultant of the active and at-rest earth pressures will act at H/3 above the base of the wall, where H is wall height.

For restrained 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 walls (walls free to rotate), 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 (2017) recommends calculating the traffic loads applied to the top of the abutment walls using an equivalent soil surcharge. For an abutment height of 5.5 feet, a minimum surcharge height of 3.9 feet is recommended. Using a unit weight of 125 pcf and a surcharge height of 3.9 feet results in a nominal uniform surcharge pressure of 487.5 psf.

Applying the at-rest pressure coefficient of 0.44 results in an additional, nominal, uniform lateral pressure of 214.5 psf for restrained walls. Applying the active pressure coefficient of 0.28 results in an additional, nominal uniform lateral pressure of 136.5 psf for unrestrained walls. The resultant of the uniform traffic surcharge pressure acts at H/2 above the base of the wall.

The project plans indicate approach panels will be used. For this condition, a reduction of traffic loads on abutment walls and wing walls may be taken. Assuming a reduction factor of 0.5, we recommend a nominal uniform lateral pressure of 107 psf for restrained walls and 68 psf for unrestrained walls.

An equivalent soil surcharge of 2 feet and active earth pressure conditions are recommended for designing the wing walls. 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. With an assumed reduction factor of 0.5 (for approach panels), the resulting uniform lateral pressure is reduced to 35 psf.

The ODOT GDM 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 earthquake. For the 1,000-yr return period earthquake, we calculated a peak ground surface acceleration (A_s) of 0.32g based on the USGS PGA (on rock) of 0.24g and an AASHTO site factor (F_{pga}) of 1.36 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 the corresponding seismic lateral earth pressure coefficient (k_{ae}) depend upon the allowable lateral deflection of the wall during an earthquake. We used a k_h of 0.16g corresponding to $0.5A_s$, assuming an allowable wall displacement of ±1 to 2 inches. The calculations indicate the seismic force on the walls may be modeled using an additional uniform pressure of ±34 psf. The resultant of the uniform seismic earth pressure acts at H/2 above the base of the walls. The seismic earth pressure should be combined with the static active earth pressure calculated using an equivalent fluid density of 35 pcf.

A summary of the calculated abutment and wing wall lateral earth pressures is provided in Table 3.

Table 3. Lateral Earth Parameters for Abutment and Wing Wall Design

Parameter	Source	Value	γ_p
At Rest Earth Pressure Coefficient, k_o	$1-\sin(\phi)$	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)	$0.5(487.5\text{ psf}*k_o)$	107 psf	1.35/1.75
Traffic Load Surcharge for Abutment Walls (Active)	$0.5(487.5\text{ psf}*k_a)$	68 psf	1.35/1.75
Traffic Load Surcharge for Wing Walls (Active)	$0.5(250\text{ psf}*k_a)$	35 psf	1.35/1.75
Seismic Pressure for Wall backfill for 1,000-year event (assumes 1 to 2 inch displacement)	Mononobe-Okabe	34 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 (2017) 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 (ODOT, 2021). It is also assumed these specifications will be referred to for general or specific items not addressed in this report.

9.2. Driven Piles

The specifications for piles and pile driving should follow the requirements of Section 00520. A monitoring program is recommended during construction to confirm all pile driving criteria are followed. We anticipate a construction inspector will log each pile for driving resistance and hammer efficiency. The driving criteria should be established using the FHWA Gates equation prior to construction once the pile hammer has been selected by the contractor. Driving should be discontinued once the pile meets the required driving resistance (between 2 and 10 blows/inch (bpi) for 3 consecutive inches) at or below the minimum tip elevation.

9.3. Falsework Support

We anticipate any required falsework or temporary structural supports will be designed by the contractor.

9.4. Excavations/Shoring/Dewatering

We anticipate excavations up to ±6 feet deep will be required for construction of the abutments and wing walls. The excavations will extend primarily through the embankment fill consisting of a mixture of silty sand and gravel.

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, or dry, raveling soils during the summer months. Plastic sheeting may be used to protect slopes that are required to remain open for an extended period of time. Dewatering is not anticipated for excavations that extend to the bottom of the proposed pile caps.

9.5. Approach Embankments

The approach work will include limited fill placement to accommodate the new bridge dimensions, excavations for the new abutments, and reconstructing the approach pavements. The following construction recommendations are based on the requirements of Section 00330.

9.5.1. ***Subgrade Preparation*.** Excavations required for embankment widening should be completed in accordance with Section 00330.41. Soft or loose subgrade, if encountered, may be mitigated by moisture-conditioning and compacting the subgrade, or by overexcavating and replacing the unsuitable material. Replacement materials may consist of Selected Granular Backfill (00330.14), Selected Stone Backfill (00330.15), or Stone Embankment Material (Section 00330.16). If Stone Embankment material is specified, a special provision limiting the maximum aggregate size to 6 inches should be included in the project specifications.

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, 10 yd^3 dump truck, or other approved construction vehicle, prior to placing the Subgrade Geotextile and Base Aggregate to identify any soft areas. Any soft or pumping subgrade should be reworked and compacted, or overexcavated and replaced with additional Base Aggregate.

9.5.2. ***<u>Approach Pavements</u>.*** Based on the ODOT PDG (ODOT, 2019), the pavement mix design for new ACP should consist of Level 2, ½-inch Dense-Graded ACP Wearing Course with PG 64-22 binder. Lift thicknesses of between 2 and 3 inches should be planned. Section 10.4 (Table 24) of the ODOT PDG (ODOT, 2019) guidelines indicates the project location does not mandate the use of anti-stripping additives in the ACP.

The Base Aggregate should conform to the material requirements of Section 02630 and grading requirements of Table 02630-1. A Subgrade Geotextile for Separation meeting the requirements in Section 02320.20 is recommended between the prepared subgrade and the Base Aggregate.

9.5.3. ***<u>Embankment Fill</u>.*** 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 permanent slopes constructed during dry weather at 2(H):1(V), or flatter. Stone Embankment Material (Section 00330.16) may be required if construction occurs during wet weather or if steeper slopes are required. New fill placed for the widening of existing embankments should be placed on properly stripped and benched slopes in accordance with ODOT Standard Detail DET2100 or DET2101, as appropriate.

9.5.4. ***<u>Abutments and Wing Walls</u>.*** Placement and compaction of imported fill behind the abutment walls and wing walls should be completed using light, vibratory equipment within 4 feet of the wall. Granular Wall Backfill (00510.12) or Granular Structure Backfill (00510.13) should be used behind the walls.

10.0. LIMITATIONS

10.1. Construction Observation/Testing

We recommend a Foundation Engineering representative be present during construction to observe the pile driving and subgrade preparation for the new approaches. 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 Mill Creek – Folsom Road Bridge Replacement 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 groundwater 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.

11.0 REFERENCES

AASHTO, 1993, *AASHTO Guide for Design of Pavement Structures*: American Association of State Highway and Transportation Officials (AASHTO).

AASHTO, 2017, *AASHTO LRFD Bridge Design Specifications - Eighth Edition:* American Association of State Highway and Transportation Officials (AASHTO).

Atwater, T., 1970, *Implications of Plate Tectonics for the Cenozoic Tectonic Evolution of Western North America:* Geological Society of America (GSA), Bulletin 81, p. 3513-3536.

Beaulieu, J. D., Hughes, P. W., and Mathiot, R. K., 1974, *Environmental Geology of Western Linn County, Oregon:* Oregon Department of Geology and Mineral Industries (DOGAMI), Bulletin 84, 117 p.

Dewey, J. W., Hopper, M. G., Wald, D. J., Quitoriano, V., and Adams, E. R., 2002, *Intensity Distribution and Isoseismal Maps for the Nisqually, Washington, Earthquake of 28 February 2001:* U.S. Geological Survey (USGS), Open-File Report 02-346, 57 p.

FHWA, 2016, *Design and Construction of Driven Pile Foundations - Volume 1.* Federal Highway Administration (FHWA), Publication No. FHWA-NHI-16-009, September 2016.

Goldfinger, C., Galer, S., Beeson, J., Hamilton, T., Black, B., Romsos, C., Patton, J., Nelson, C. H., Hausmann, R., and Morey, A., 2016, *The Importance of Site Selection, Sediment Supply, and Hydrodynamics: A Case Study of Submarine Paleoseismology on the Northern Cascadia Margin, Washington, USA:* Marine Geology, In Press, http://dx.doi.org/10.1016/j.margeo.2016.06.008.

Goldfinger, C., Nelson, C. H., Morey, A. E., Johnson, J. R., Patton, J., Karabanov, E., Gutierrez-Pastor, J., Eriksson, A. T., Gracia, E., Dunhill, G., Enkin, R. J., Dallimore, A., Vallier, T., and 2012, *Turbidite Event History - Methods and Implications for Holocene Paleoseismicity of the Cascade Subduction Zone:* U.S. Geologic Survey (USGS), Professional Paper 1661-F, 170 p., 64 figures, http://pubs.usgs.gov/pp/pp1661/f.

Nelson, A. R., Atwater, B. F., Bobrowsky, P. T., Bradley, L.-A., Claque, J. J., Carver, G. A., Darienzo, M. E., Grant, W. C., Drueger, H. W., Sparks, R., Stafford, T. W., Jr., and Stulver, M., 1995, *Radiocarbon Evidence for Extensive Plate-boundary Rupture About 300 Years Ago at the Cascadia Subduction Zone*: Letters to Nature, vol. 378, no. 23, p. 372-374.

ODOT 2016, *ODOT ARS V2014.16 Response Spectra Spreadsheet*, Program developed for the Oregon Department of Transportation (ODOT), by Portland State University Department of Civil and Environmental Engineering.

ODOT, 2018, *Geotechnical Design Manual (GDM):* Oregon Department of Transportation (ODOT), Geo-Environmental Section

ODOT, 2021, *Oregon Standard Specifications for Construction*, Oregon Department of Transportation, Highway Division*.*

ODOT, 2019, *ODOT Pavement Design Guide:* Oregon Department of Transportation (ODOT), Pavement Services Unit.

Palmer, A. R., 1983, *The Decade of North American Geology - 1983 Geologic Time Scale:* Geology, vol. 11, p. 503-504, September 1983.

Personius, S. F., Dart, R. L., Bradley, L.-A., and Haller, K. M., 2003, *Map and Data for Quaternary Faults and Folds in Oregon:* U.S. Geological Survey (USGS), Open-File Report 03-095, v.1.1, Scale: 1:750,000, 507 p.

USGS, 2006, *Quaternary Fault and Fold Database for the United States - Oregon:* U.S. Geological Survey (USGS), accessed May 2019, web site: http://earthquake.usgs.gov/hazards/qfaults.

USGS, 2014, *Earthquake Hazards Program, Interactive Deaggregations, Dynamic Conterminous U.S. 2014 (v.4.2.0):* U.S. Geological Survey (USGS), 5% in 50 years return period (975 years) PGA spectral acceleration for bridges, latitude/longitude search, reference material has no specific release date, accessed April 2021, website: https://earthquake.usgs.gov/hazards/interactive/index.php.

Walker, G. W., and Duncan, R. A., 1989, *Geologic Map of the Salem 1° by 2° Quadrangle, Western Oregon:* U. S. Geological Survey (USGS), Miscellaneous Investigations Series Map I-1893, Scale= 1:250,000.

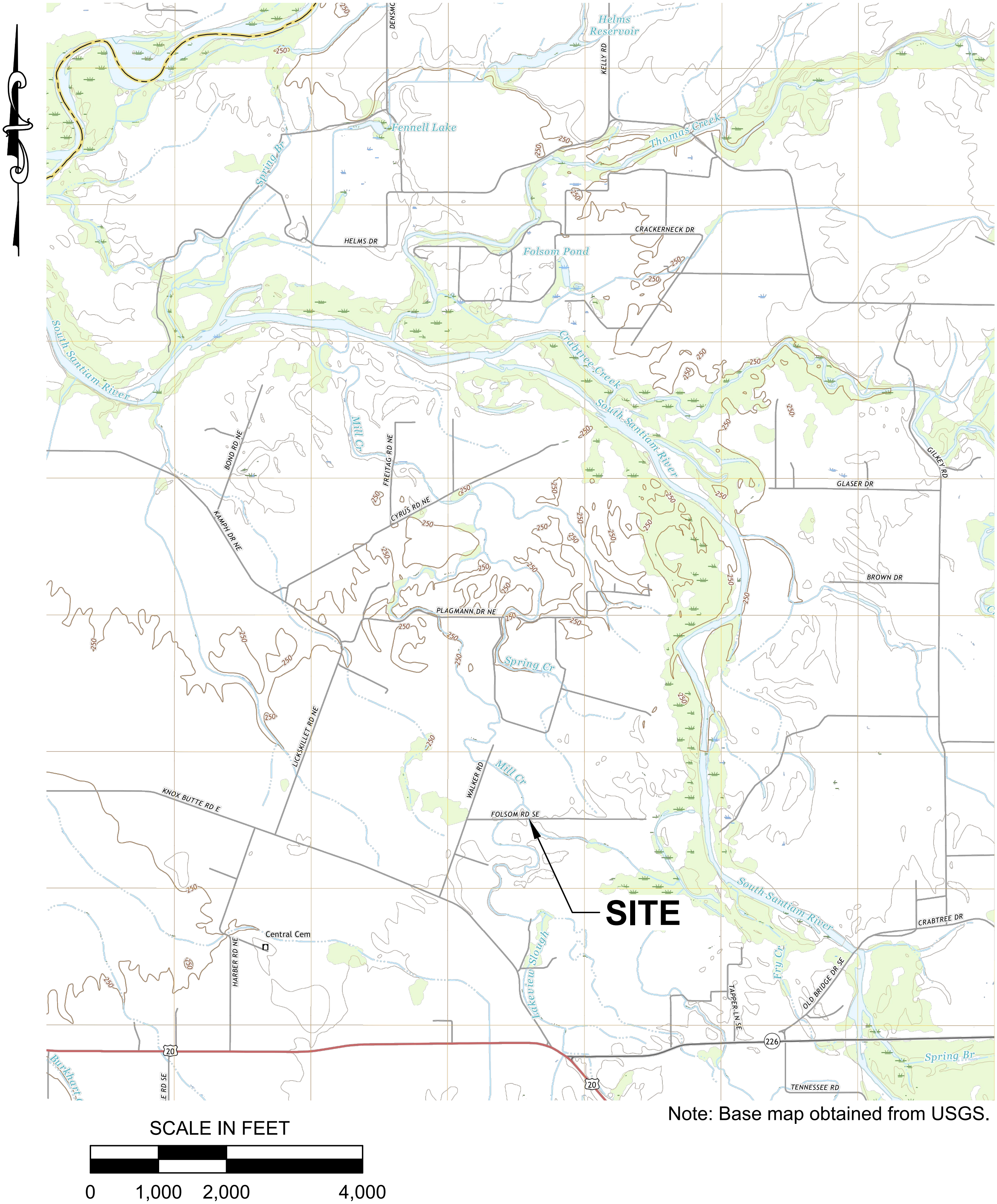
Wong, I. G., and Silva, W. J., 1998, *Earthquake Ground Shaking Hazards in the Portland and Seattle Metropolitan Areas*: *in* Dakoulas, P., Yegian, M., and Holtz, R. D., eds., *Geotechnical Earthquake Engineering and Soil Dynamics III*, American Society of Civil Engineers (ASCE), Geotechnical Special Publication vol. 1, no. 75, p. 66-78.

Yeats, R. S., Graven, E. P., Werner, K. S., Goldfinger, C., and Popowski, T. A., 1996, *Tectonics of the Willamette Valley, Oregon*: *in* Roger, A. M., Walsh, T. J., Kockelman, W. J., and Priest, G. R., eds., *Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest*, U.S. Geological Survey (USGS), Professional Paper 1560, p. 183-222.

Appendix A

Figures

Foundation Engineering, Inc.
Professional Geotechnical Services



Note: Base map obtained from USGS.

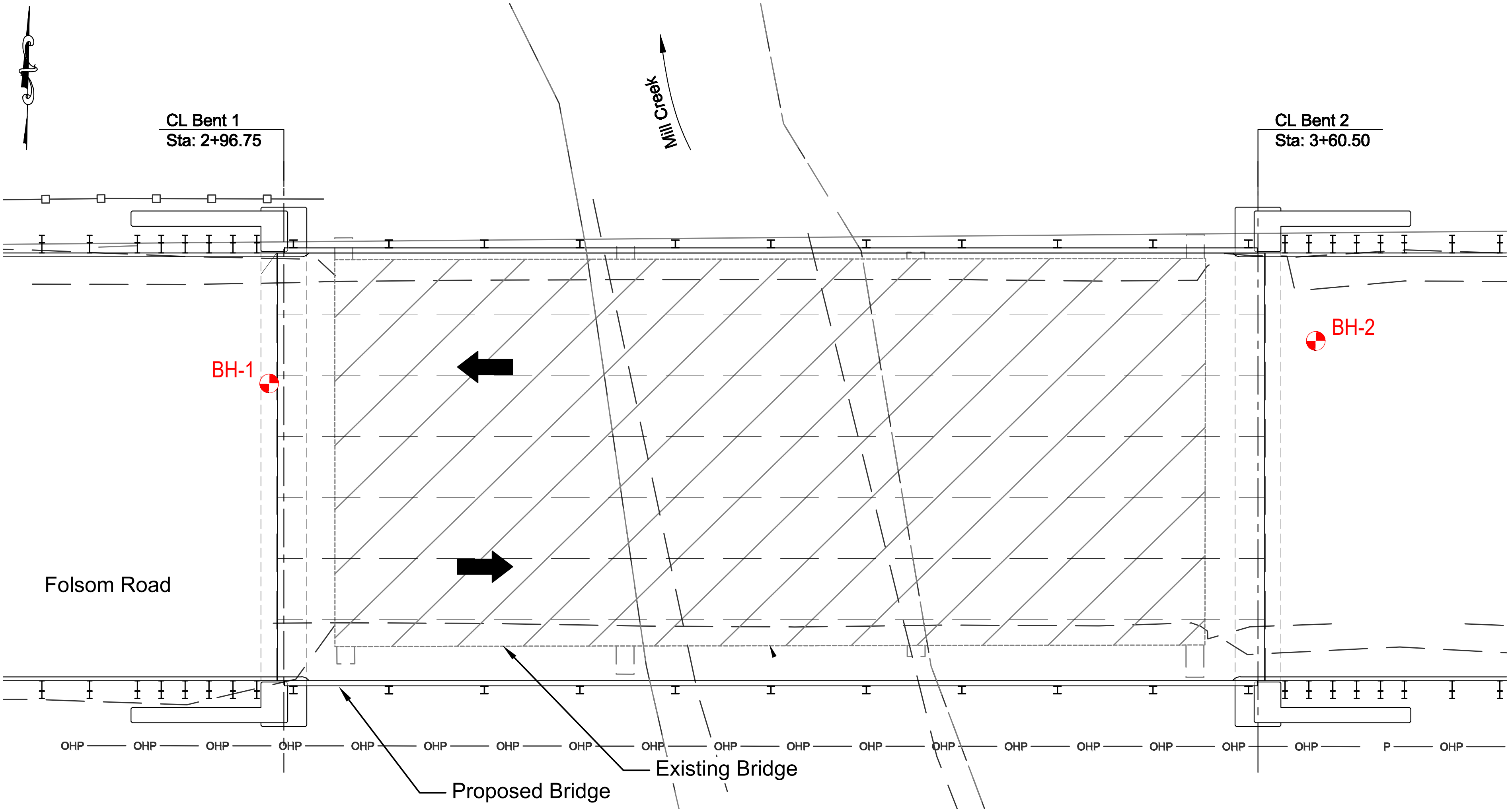
Foundation Engineering, Inc.
Professional Geotechnical Services

PROJECT NO.	DATE:	DRAWN BY:
2211021	Mar. 1, 2021	MDM

VICINITY MAP

MILL CREEK - FOLSOM ROAD BRIDGE REPLACEMENT
LINN COUNTY, OREGON

FIGURE NO.
1A



NOTES:
1. BORING LOCATIONS WERE NOT SURVEYED AND WERE ESTABLISHED USING A TAPE MEASURE REFERENCING EXISTING LANDMARKS. LOCATIONS ARE APPROXIMATE.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE DRAWING WAS PROVIDED BY LINN COUNTY ROAD DEPARTMENT.

Foundation Engineering, Inc.
Professional Geotechnical Services

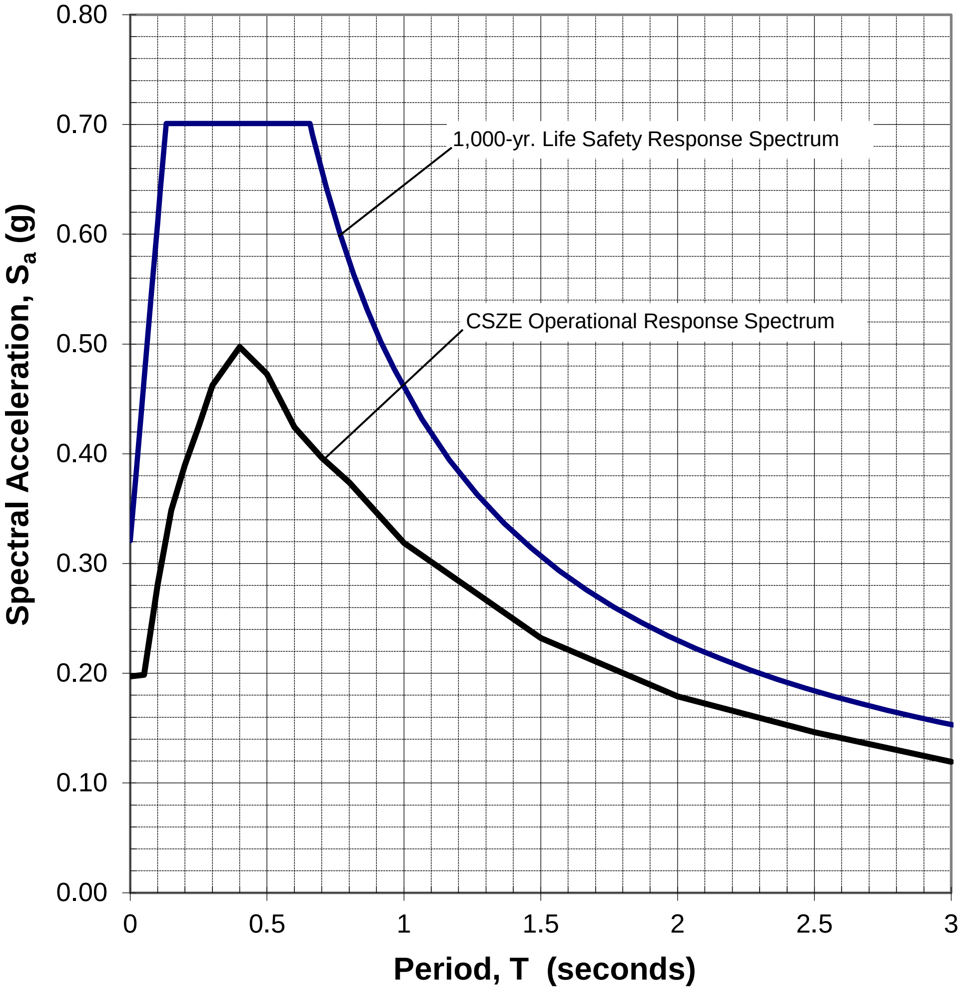
PROJECT NO.	DATE:	DRAWN BY:
2211021	Jul. 6, 2021	MDM

BOREHOLE LOCATIONS

MILL CREEK - FOLSOM ROAD BRIDGE REPLACEMENT
LINN COUNTY, OREGON

FIGURE NO.
2A

FILE:FIG 2A (7-2-21).dwg



Notes:

1. The 1,000-yr. Life Safety Design Response Spectrum is based on AASHTO 2017 Section 3.10.3 using the following parameters:

Site Class= D Damping = 5%

PGA =	0.24	F_{pga} =	1.36	A_s =	0.32
S_S =	0.50	F_a =	1.40	S_{DS} =	0.70
S_1 =	0.21	F_v =	2.18	S_{D1} =	0.46

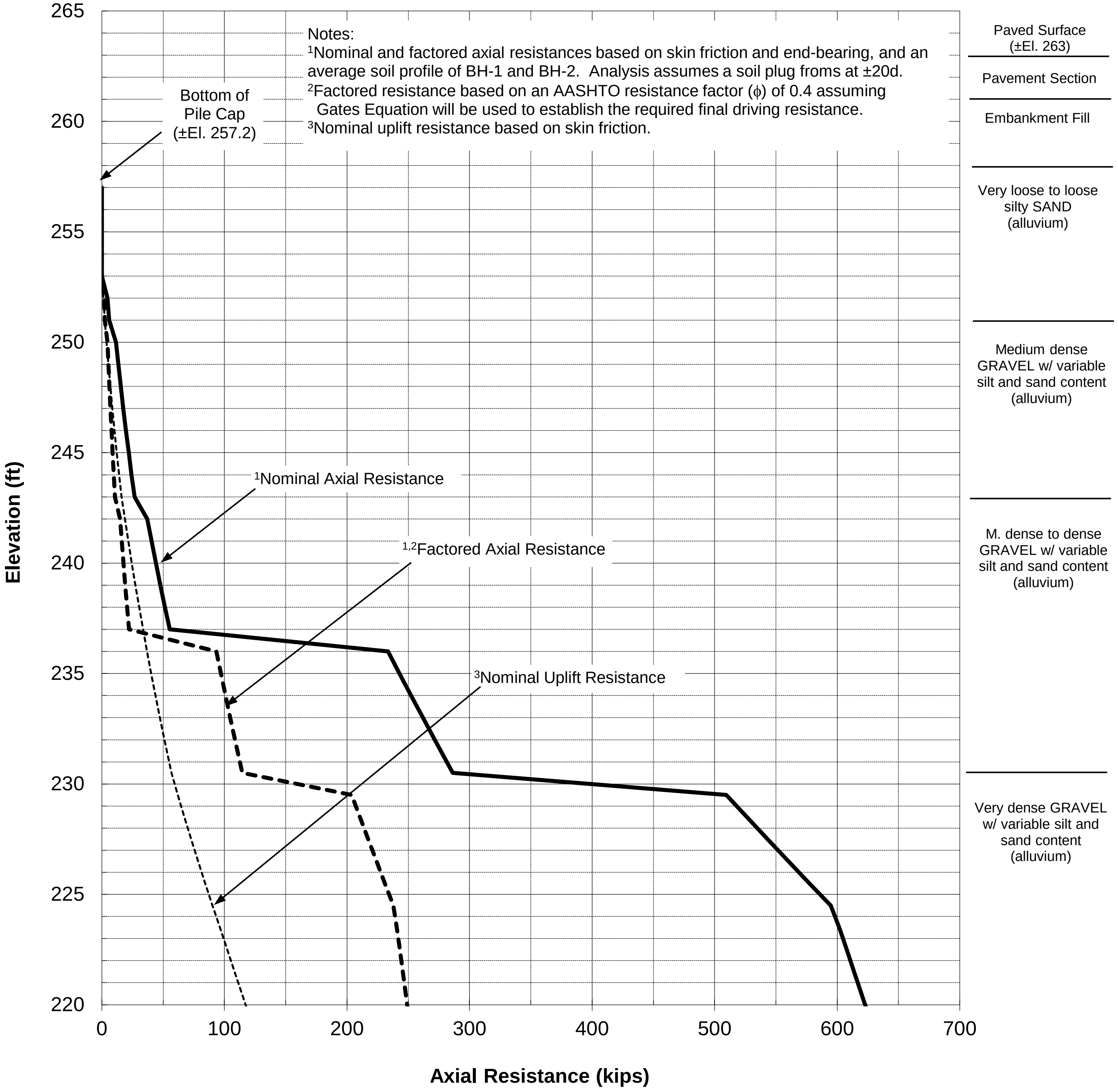
PGA, S_S and S_1 values are based on USGS 2014 seismic hazard maps and were obtained using the ODOT ARSV2014.16.xls spreadsheet. F_{pga}, F_a, and F_v were established based on ODOT GDM 2018, Tables 6.2-A, 6.2-B and 6.2-C using the selected PGA, S_S, and S_1 values.

2. The CSZE values were obtained using the PSU CSZ calculator assuming V_{s30} = 270 m/s consistent with the average assumed shear wave velocity for a Site Class D profile.

3. Site location: lattitude 44.6449, longitude -122.9547.

FIGURE 3A
LIFE SAFETY AND OPERATIONAL DESIGN CRITERIA RESPONSE SPECTRA

Mill Creek - Folsom Road Bridge Replacement
Linn County, Oregon
Project No. 2211021



Notes:

[1] Nominal and factored axial resistances based on skin friction and end-bearing, and an average soil profile of BH-1 and BH-2. Analysis assumes a soil plug froms at ±20d.

[2] Factored resistance based on an AASHTO resistance factor (ϕ) of 0.4 assuming Gates Equation will be used to establish the required final driving resistance.

[3] Nominal uplift resistance based on skin friction.

FIGURE 4A
AXIAL RESISTANCE vs. ELEVATION
PP12.75x0.375 Open-Ended Pile
Mill Creek - Folsom Road Bridge Replacement
Linn County, Oregon
Project No.: 2211021

Table 1A. Recommended LPILE Soil Parameters

Elevation (ft)	Depth (ft)	Soil Description	LPILE p-y Criteria	γ' (pcf)	k (pci)	ϕ' (°)
257.2 253.0	0 4.2	Very loose to loose silty SAND	SAND (Reese)	115 115	25 25	32 32
253.0 251.0	4.2 6.2	Very loose to loose silty SAND	SAND (Reese)	52.6 52.6	20 20	32 32
251.0 243.0	6.2 14.2	Medium dense GRAVEL, w/ variable silt & sand content	SAND (Reese)	62.6 62.6	60 60	35 35
243.0 230.5	14.2 26.7	Medium dense to dense GRAVEL, w/ variable silt & sand content	SAND (Reese)	67.6 67.6	125 125	38 38
230.5 200.0	26.7 57.2	Very dense GRAVEL, w/variable silt & sand content	SAND (Reese)	67.6 67.6	125 125	42 42

Notes: 1. Subsurface profile interpreted based on conditions encountered in BH-1 and BH-2.
2. Top elevation of the soil profile corresponds to the bottom elevation of the pile cap.
3. Assumes ground water table at El. 253.0.

Appendix B

Boring Logs

Foundation Engineering, Inc.
Professional Geotechnical Services

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.

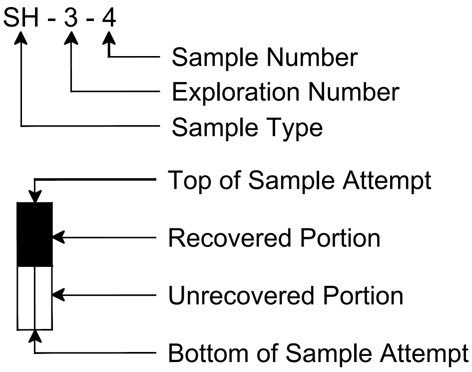
VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

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 O.D. split-spoon)
S - Grab Sample
SH - Thin-walled Shelby Tube Sample
SS - Standard Penetration Test Sample
(2-inch O.D. 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 (%)

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.

WATER TABLE

▼	Water Table Location
(1/31/16)	Date of Measurement

TYPICAL SOIL/ROCK SYMBOLS

▩	Concrete	▥	Silt	▨	Basalt
▢	Organics	▒	Sand	▒	Sandstone
▨	Clay	▩	Gravel	▤	Siltstone

UNIFIED SOIL CLASSIFICATION SYMBOLS

G - Gravel	W - Well Graded
S - Sand	P - Poorly Graded
M - Silt	L - Low Plasticity
C - Clay	H - High Plasticity
Pt - Peat	O - Organic

Foundation Engineering, Inc.
Professional Geotechnical Services

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.

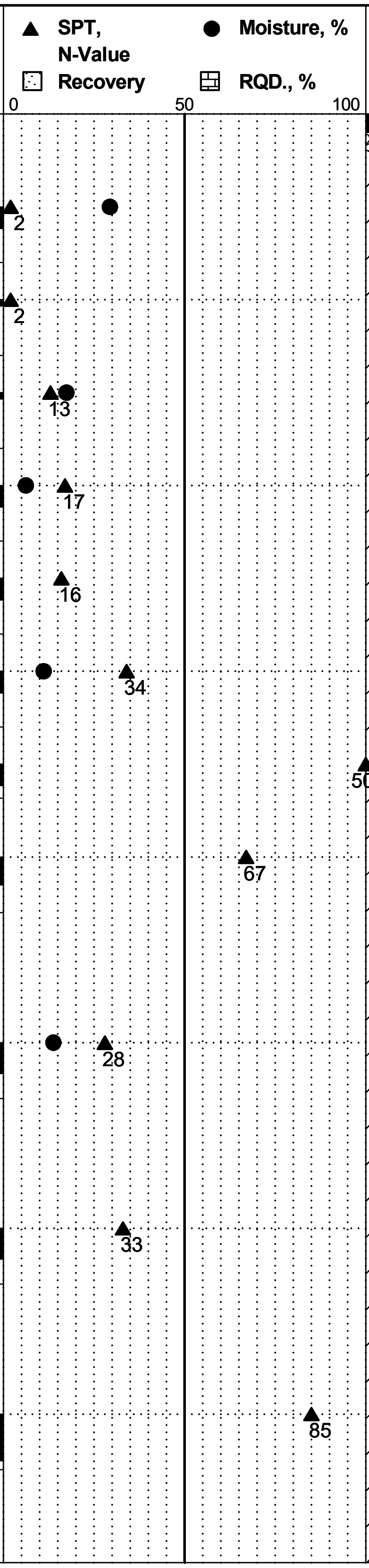
Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

Foundation Engineering, Inc.
Professional Geotechnical Services

SOIL DESCRIPTIONS

COMMON TERMS

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
	ASPHALTIC CONCRETE (±3 inches).	263 262.7 0.3			Capped with AC cold patch and gravel
1	CRUSHED ROCK (GW); grey, dry to damp, medium dense, ±1-inch minus angular rock, (base rock).	261.4 1.6			
2	Silty SAND, trace gravel (SM); brown, low plasticity silt, moist to wet, very loose, fine sand, fine subangular gravel, (fill).				
3			SS-1-1	2	
4					
5	GRAVEL, trace silt and sand (GP); grey, low plasticity silt, moist to wet, very loose, fine sand, fine subangular gravel, (fill).	258.0 5.0	SS-1-2	2	Backfilled with bentonite chips (±1 to 5 feet)
6					
7	Lost ±150 gallons of drilling fluid from ±5 to 11.5 feet.	256.5 6.5			
8	Silty SAND, trace gravel (SM); brown, low plasticity silt, moist to wet, medium dense, fine sand, fine subangular gravel, (alluvium).		SS-1-3	13	
9		254.0 9.0			
10	GRAVEL, trace silt and sand (GW); grey, low plasticity silt, moist to wet, medium dense, fine sand, fine to coarse subrounded gravel, (alluvium).		SS-1-4	17	
11					
12	Cobbles up to ±10-inch diameter from ±10 to 12 feet.				
13	Dark grey to black from ±12.5 to 13.1 feet.		SS-1-5	16	
14					
15	GRAVEL, some silt and sand (GW-GM); grey, low plasticity silt, moist to wet, dense to very dense, fine sand, fine to coarse subrounded gravel, (alluvium).	248.0 15.0	SS-1-6	34	Bentonite grout (±15 to 40 feet)
16					
17	Dark grey to black from ±17 to 25 feet.				
18			SS-1-7	50/5"	
19					
20			SS-1-8	67	
21					
22	Lost ±40 gallons of drilling fluid from ±22.5 to 23.5 feet.				
23	Cobbles up to ±6-inch diameter from ±22.5 to 23 feet and ±26 to 27 feet.				
24					
25	Grey to brown and iron-stained below ±25 feet. Medium dense at ±25 feet. Lost ±40 gallons of drilling fluid from ±26 to 27 feet.		SS-1-9	28	
26					
27					
28					
29					
30			SS-1-10	33	
31					
32					
33					
34					
35			SS-1-11	85	
36					
37					
38					



Project No.: 2211021

Surface Elevation: 263.0 feet (Approx.)

Date of Boring: April 1, 2021

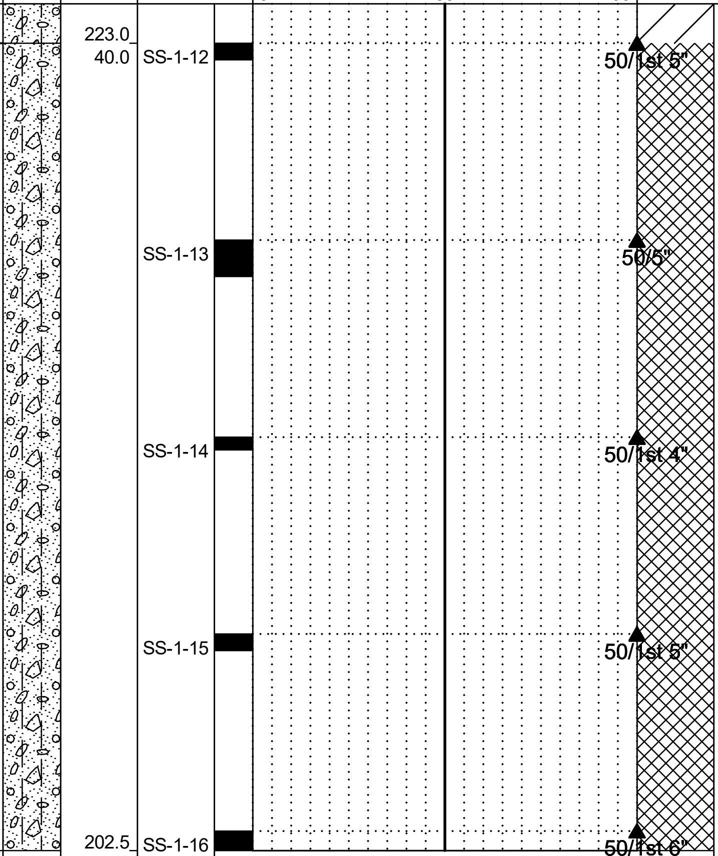
Foundation Engineering, Inc.

Boring Log: BH-1

Mill Creek - Folsom Road Bridge Replacement

Linn County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth (224)	Samples	▲ SPT, N-Value (0 – 50 – 100)	Backfill/ Installations/ Water Table
40	Sandy GRAVEL, some silt (GW-GM); blue-grey, low plasticity silt, moist to wet, very dense, fine sand, fine to coarse subrounded gravel, (alluvium).	223.0 40.0	SS-1-12	50/1st 5"	
41					
42	Lost ±20 to 30 gallons of drilling fluid from ±42 to 44 feet.				
43					
44					
45	Grey-brown and iron-stained and silty at ±45 feet.		SS-1-13	50/5"	Native soil (due to caving)
46					
47					
48					
49					
50	Grey to black at ±50 feet.		SS-1-14	50/1st 4"	
51					
52					
53					
54					
55	Blue-grey to dark grey below ±55 feet.		SS-1-15	50/1st 5"	
56					
57	Lost ±30 gallons of drilling fluid from ±57 to 59 feet.				
58					
59					
60		202.5 60.5	SS-1-16	50/1st 6"	
	BOTTOM OF BORING Note: Boring terminated at ±60.5 feet due to sidewalls caving in to ±40 feet.				



Project No.: 2211021

Surface Elevation: 263.0 feet (Approx.)

Date of Boring: April 1, 2021

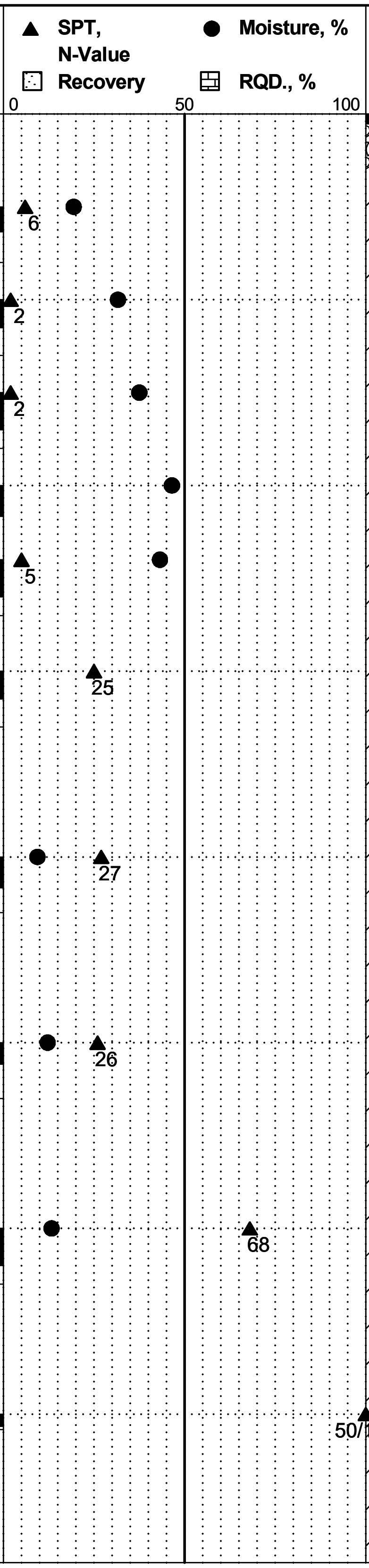
Boring Log: BH-1

Mill Creek - Folsom Road Bridge Replacement

Linn County, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
0	ASPHALTIC CONCRETE (±3½ inches).	262.5 262.2 0.3			Capped with AC cold patch and gravel
1	CRUSHED ROCK (GW); grey, dry to damp, medium dense, ±1-inch minus angular rock, (base rock).	261.0 1.5			
2	Silty SAND, trace gravel (SM); brown to grey, low plasticity silt, damp to moist, loose, fine sand, fine to coarse subangular gravel, (fill).				
3			SS-2-1	6	
5	Silty SAND (SM); brown and iron-stained, low plasticity silt, moist to wet, very loose to loose, fine sand, (alluvium).	257.5 5.0	SS-2-2	2	Backfilled with bentonite chips (±1.5 to 15 feet)
7.5			SS-2-3	2	
10			SH-2-4		
12	Some silt below ±12 feet.		SS-2-5	5	
13.5	GRAVEL, trace to some silt and some sand (GW-GM); dark grey to black, low plasticity silt, moist to wet, medium dense, fine sand, fine to coarse subrounded gravel, (alluvium).	249.0 13.5			
15			SS-2-6	25	Bentonite grout (±15 to 75 feet)
20	Grey to grey-brown below ±20 feet.		SS-2-7	27	
21	Lost ±80 gallons of drilling fluid from ±20 to 21.5 feet and ±23.5 to 25 feet.				
25	GRAVEL, some silt and sand (GW-GM); grey to brown, low plasticity silt, moist to wet, medium dense, fine sand, fine to coarse subrounded gravel, (alluvium).	237.5 25.0	SS-2-8	26	
29	Possible sand layer from ±28.5 to 29 feet.				
30			SS-2-9	68	
35	Sandy GRAVEL, some silt (GW-GM); grey-brown, low plasticity silt, moist to wet, very dense, fine sand, fine to coarse subrounded gravel, (alluvium).	227.5 35.0	SS-2-10	50/1st 5"	



Project No.: 2211021

Surface Elevation: 262.5 feet (Approx.)

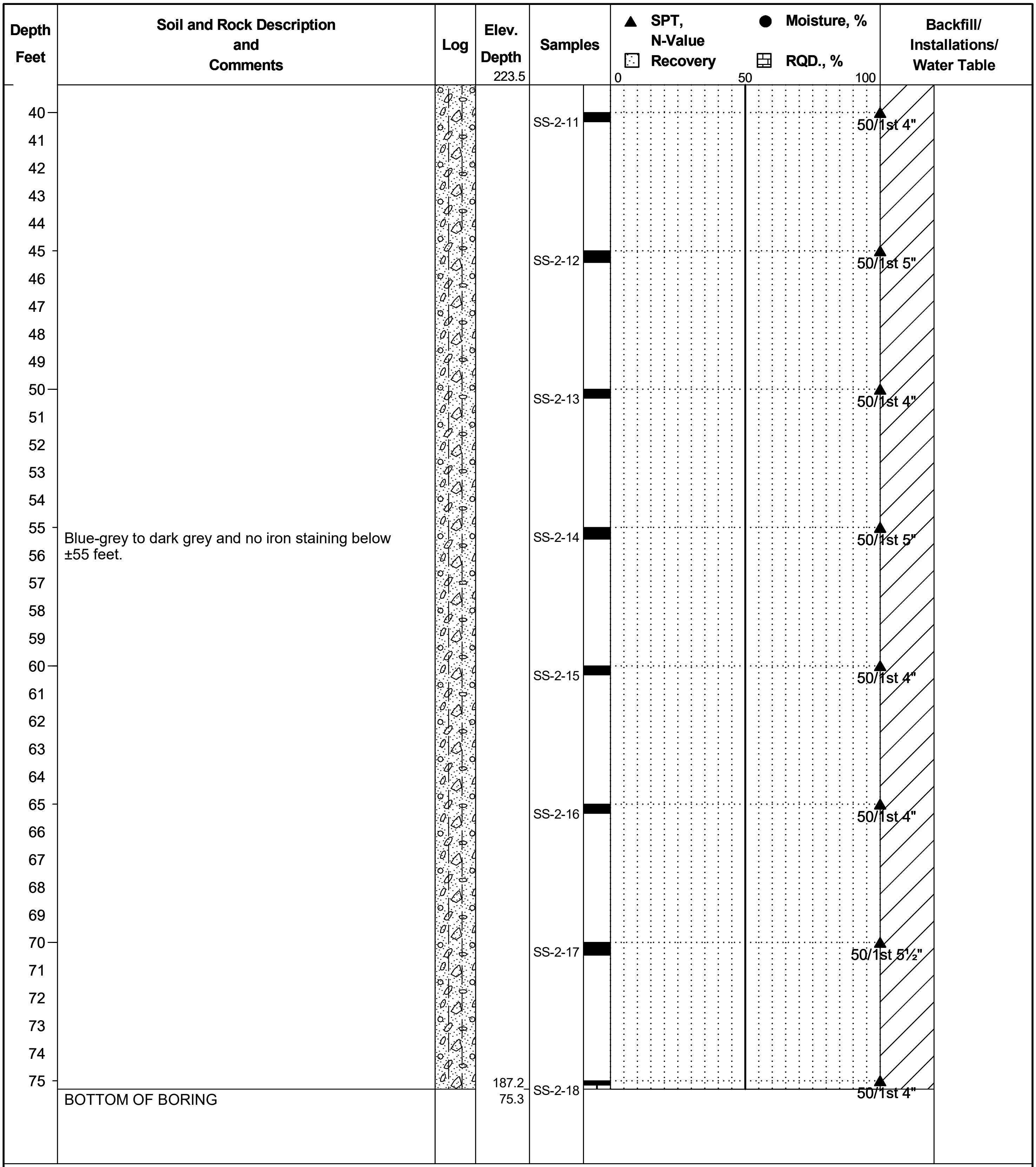
Date of Boring: April 2, 2021

Boring Log: BH-2

Mill Creek - Folsom Road Bridge Replacement

Linn County, Oregon

Foundation Engineering, Inc.



Blue-grey to dark grey and no iron staining below ±55 feet.

BOTTOM OF BORING

Project No.: 2211021

Surface Elevation: 262.5 feet (Approx.)

Date of Boring: April 2, 2021

Foundation Engineering, Inc.

Boring Log: BH-2

Mill Creek - Folsom Road Bridge Replacement

Linn County, Oregon

Appendix C

Field and Laboratory Testing

Foundation Engineering, Inc.
Professional Geotechnical Services

Table 1C. Moisture Contents (ASTM D 2216) and Percent Fines (ASTM D 1140)

Sample Number	Sample Depth (ft)	Moisture Content (%)	Percent Fines (%)
SS-1-1	2.5 – 4.0	29.4	41.3
SS-1-3	7.5 – 9.0	17.4	14.5
SS-1-4	10.0 – 11.5	6.2	0.9
SS-1-6	15.0 – 16.5	11.1	6.7
SS-1-9	25.0 – 26.5	13.8	3.8
SS-2-1	2.5 – 4.0	19.4	28.7
SS-2-2	5.0 – 6.5	31.6	45.3
SS-2-3	7.5 – 9.0	37.5	40.5
SH-2-4	10.0 – 12.0	46.5	30.0
SS-2-5	12.0 – 13.5	43.2	11.1
SS-2-7	20.0 – 21.5	9.4	3.5
SS-2-8	25.0 – 26.5	12.2	7.5
SS-2-9	30.0 – 31.5	13.3	11.9

Table 2C. Summary of DCP Test Results (ASTM D6951)

Exploration	Initial Test Depth (inches)	Soil Description	[1]Average DCP (mm/blow)	[2]Average M_r (psi)	[3]Corrected M_r (psi)
BH-1	6.0	CRUSHED ROCK (GW)	2.4	34,582	21,441
	18.0	Silty SAND, trace gravel (SM)	6.1	24.294	8,503
BH-2	6.0	CRUSHED ROCK (GW))	1.9	38,030	23,579
	18.0	Silty SAND, trace gravel (SM)	4.3	27,745	9,711

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: $M_r = 49{,}023\ (DCP)^{-0.39}$. Values may vary slightly due to rounding.
3. Corrected M_r value is based on the ODOT recommended correction factors of 0.62 for base rock and 0.35 for subgrade.

Table 3C. Summary of Resistivity Testing (ASTM G 57)

Location	Pin Spacing (ft)	Resistivity (Ω-cm)
R-1	5	9,192
	10	8,809
	15	8,618

Table 4C. pH Test Results (ASTM G 51)

Sample Number	Sample Depth (ft)	Sample Description	pH
SS-1-1	2.5 – 4.0	Silty SAND	6.7