



Oregon State University

COMPREHENSIVE RESERVE INVITATION TO BID (ITB) #2026-021520

SOAP CREEK RANCH BRIDGE REPLACEMENT

ADDENDUM NO. 01

ISSUE DATE: May 14, 2026

CONTRACT ADMINISTRATOR:
Shoshana Shabazz, Purchasing Analyst
Construction Contracts Administration
Email: ConstructionContracts@oregonstate.edu

This Addendum is hereby issued to inform you of the following revisions and or clarifications to the above-referenced ITB and/or the Contract Documents for the Project, to the extent they have been modified herein. Any conflict or inconsistency between this Addendum and the Solicitation Document or any previous addenda will be resolved in favor of this Addendum. Bids shall conform to this Addendum. Unless specifically changed by this Addendum, all other requirements, terms and conditions of the Solicitation Document and or Contract Documents, and any previous addenda, remain unchanged and can be modified only in writing by OSU. The following changes are hereby made:

MODIFICATIONS:

**Item 1 Section 7.6
Submission
of Bid** **ADD** 7.6.5 All Bid submissions subject line shall read exactly: ITB 2026-021520 - Soap Creek Ranch Bridge Replacement - Bid Submission – Firm Name. The Bidder shall be responsible for using the exact email subject-line format required in this ITB for Bid submissions. OSU shall not be responsible for communications or submissions that are not received, not identified, misrouted, or not reviewed because the Bidder failed to use the required email format. Any delay, omission, or missed submission resulting from an incorrect or incomplete subject line shall be the sole responsibility of the Bidder.

TECHNICAL SPECIFICATIONS:

Item 2 ADD Special Inspection and Testing Specifications attached to this addendum.

REFERENCE/SUPPLEMENTAL MATERIALS:

Item 3 ADD Geotechnical Memorandum dated September 10, 2025 attached to this addendum.

QUESTIONS:

Item 4 Question Does Buy America apply for this project?
Answer No, it does not apply to this Project.

Item 5 Question Is the Fabricator of this bridge required to have an AISC certification for this bridge work?
Answer No, not required.

Item 6 Question What coating system will be needed and is there any certifications required?
Answer See Structural Working Drawings.

Item 7 Question Is there any hardware specification?
Answer See Structural Working Drawings.

Item 8 Question Are there any Material restrictions. Does this job Buy America trade deals, domestic melt and manufacture, or is import allowed?
Answer See question and answer 02 above and Structural Working Drawings, 3.00 General Notes, Materials.

- Item 9** **Question** What weld code is required on this Project? D1.1? D1.5?
Answer D1.5. See Structural Working Drawings, 3.00 General Notes, Materials.
- Item 10** **Question** Are there any weld tracking or special inspections required?
Answer See attached Special Inspection and Testing Specifications included in this addendum.
- Item 11** **Question** Will a work schedule need to be documented and present at the time of bid?
Answer Not at time of bid. The in-water work period for this Project is July 1 through October 31, 2026.
- Item 12** **Question** I have been working with a well-known and reputable steel bridge fabricator on this project. They have informed me that the steel stringer members called out for this project (W40x211 A588) is only available from one mill in the United States. The next rolling for that member size is in August of this year and that rolling is full and not accepting any new orders. There currently is not another rolling scheduled for that member size, but they thought it might be around November of this year. This obviously poses a big problem for procuring and fabricating the steel members.

As an alternative, I inquired about foreign steel (see my Q&A about Buy America from yesterday). They told me that a foreign rolling would likely be as far out as the next domestic rolling. So, even if foreign is allowed, we still most likely will not be able to source the material in time.

The last option is to source the material in a warehouse somewhere, but it is highly unlikely that we'll be able to find full length members (30-50ft is far more common), especially if weathering steel (A588) is required.

Questions (solutions):

- 1) Are there any other member sizes that could be used as an alternative?
- 2) If yes to #1 above, will A588 still be required?
- 3) Will welded splices in the stringers be permitted?

Answer Alternative member sizes, material grades, and welded splice details may be submitted for review as an approved equal request. Proposed approved equal requests shall meet or exceed the structural and durability requirements of the Project Documents. Also See Section 01 25 00 Product Substitution Procedures.

- Item 13** **Question** Will trade access to the west side of the bridge be allowed via alternative access roads or will this be limited to ranch and OSU personal only requiring a temp bridge to be constructed for construction access to west side?
- Answer** There would be limited access to west of ranch on a gravel road off of Tampico. Unknown condition and suitable for smaller vehicles only. Primary construction access is Beef Barn Road east approach with construction staged accordingly.
- Item 14** **Question** It is observed in note 1.2 regarding the displacement of Native Americans, are there any known artifacts in this area, additional requirements, or stipulations on the part of the contractor pertaining to this?
- Answer** No known artifacts in area of project work.
- Item 15** **Question** Has the creek maintained a consistent water level during the time of year construction will take place? Do we know what the WSE was on +/- July/August 1, 2025?
- Answer** See Geotechnical Memorandum dated September 10, 2025 included in this addendum.
- Item 16** **Question** Note B.22 relating to owner requested project management software being required. Do you know which project management software is desired?
- Answer** Not required for this project.
- Item 17** **Question** Is there a water spigot available for use nearby for temp irrigation measures for maintaining plantings after construction complete until they are established?
- Answer** Yes, twenty feet plus/minus north of the bridge area of work.
- Item 18** **Question** Is there a geotechnical report available?
- Answer** See Geotechnical Memorandum dated September 10, 2025 included in this addendum.

SPECIAL INSPECTION & TESTING SPECIFICATION

Project: 14'-0" x 80'-0" Replacement Bridge

Location: Beef Barn Lane, Benton County, Oregon

Prepared by: Structural Engineer of Record

CODES & STANDARDS

- **American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications (9th Ed. or as adopted by ODOT)**
 - **AASHTO LRFD Bridge Construction Specifications**
 - **Oregon Department of Transportation Standard Specifications for Construction (latest)**
 - **American Welding Society / AASHTO AWS D1.5 Bridge Welding Code**
 - **Research Council on Structural Connections Specification for Structural Joints Using High-Strength Bolts**
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1. GENERAL

- 1. Inspection and testing shall be performed by qualified independent agencies approved by the Owner/Engineer.**
 - 2. Contractor shall provide 48-hour notice for required inspections.**
 - 3. Work shall not proceed beyond designated hold points without inspection approval.**
 - 4. Nonconforming work shall be corrected and re-inspected.**
 - 5. Inspection reports shall be submitted to the Engineer of Record.**
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2. REQUIRED SPECIAL INSPECTIONS / HOLD POINTS

2.1 REBAR INSPECTION (PRIOR TO FORM CLOSURE)

Inspection required prior to concrete placement.

Verify:

- **Reinforcing steel size, spacing, and grade**
 - **Clear cover and support (chairs/ties)**
 - **Lap splices, hooks, and development lengths**
 - **Cleanliness and condition of reinforcement**
 - **Placement of dowels and embeds**
-

2.2 PRE-POUR INSPECTION (CONCRETE)

Inspection required immediately prior to each concrete placement.

Verify:

- **Forms, dimensions, alignment, and elevations**
- **Bearing seat elevations and geometry**
- **Embedded items (anchor rods, plates, inserts)**
- **Construction joints and surface preparation**
- **Access and placement procedures**

Concrete testing agency shall be present at start of placement.

2.3 CONCRETE TESTING & PLACEMENT OBSERVATION

Testing and observation during placement:

- **Slump — ASTM C143**
- **Air content — ASTM C231/C173**
- **Temperature — ASTM C1064**
- **Compression cylinders — ASTM C31/C39**
- **Unit weight (if required) — ASTM C138**

Observe:

- **Placement and consolidation**
 - **Finishing operations**
 - **Curing procedures**
-

2.4 STRUCTURAL STEEL INSPECTION (ERECTION)

Inspection required during steel erection.

Verify:

- **Member sizes, camber, and alignment**
 - **Bearing conditions and seat fit-up**
 - **Connection fit-up and hole alignment**
 - **Installation of bolts and temporary supports**
-

2.5 HIGH-STRENGTH BOLTING INSPECTION

Verify:

- **Bolt type, grade, and installation method**
- **Washer placement and orientation**
- **Proper tightening method (snug-tight or pretensioned)**
- **Periodic verification of bolt tension**

2.6 WELDING INSPECTION (CWI REQUIRED)

Inspection shall be performed by a Certified Welding Inspector (CWI) in accordance with AASHTO/AWS D1.5.

Pre-Welding:

- **Verify welder qualifications**
- **Verify WPS and joint preparation**

During Welding:

- **Monitor preheat and welding procedures**
- **Verify consumables**

Post-Welding:

- **Visual inspection of welds**
- **Inspect for defects (cracks, undercut, porosity)**

NDT (if specified):

- **MT, UT, or RT for critical welds**

2.7 BEARINGS & ANCHORAGE

Verify:

- **Bearing placement, orientation, and seating**
 - **Anchor rod size, embedment, and alignment**
 - **Grout pad placement and condition**
-

3. DOCUMENTATION

Provide:

- **Daily inspection reports**
 - **Concrete test reports**
 - **Steel certifications**
 - **Bolt installation verification**
 - **Welding inspection and NDT reports**
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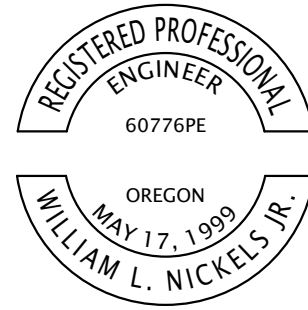
4. ACCEPTANCE

Work shall be accepted when:

- **All required inspections are completed**
- **Test results meet specifications**
- **All deficiencies are corrected**



Date: September 10, 2025
To: Josh Hackenbruck
 Oregon State University
 College of Agricultural Sciences
From: William L. Nickels, Jr., P.E., G.E.
Subject: Foundation Report
Project: Soap Creek Bridge Replacement
Project No.: 2251001



EXPIRES: 12-31-2026

We have completed the requested geotechnical investigation for the above-referenced project. This memorandum includes a description of our work, a discussion of the site conditions, and recommendations for foundation design and construction.

There are numerous values in geotechnical investigations that are approximate, including calculated values, measured lengths and heights, soil and rock layer depths and elevations, and strength measurements. For brevity, the symbol “±” is used throughout this report to represent the words “approximate” or “approximately” when discussing approximate values.

BACKGROUND

Oregon State University (OSU) is planning to replace the existing bridge on Beef Barn Road where it crosses Soap Creek in Benton County. The bridge location is shown on the Vicinity Map (Figure 1A, Appendix A).

The existing bridge is a ±75-foot long, three-span timber structure supported on timber piles. The replacement bridge will be an 80-foot long, 14-foot wide bridge supported on spread footings. The approaches will be raised by 3.4 feet at the west abutment and by 4.2 feet at the east abutment.

OSU is the project owner. CSE Engineering, Inc. is the structural designer and River Design Group (RDG) is the hydraulic consultant. Foundation Engineering, Inc. (FE) was retained by OSU as the geotechnical consultant. Details of our scope of work were provided in a proposal dated December 27, 2024, and authorized by Supplement No.: 2025-017841, dated January 30, 2025.

LOCAL GEOLOGY AND FAULTING

Local Geology

The bridge site is within the transition from the Willamette Valley to the eastern foothills of the Coast Range, which is a north-south-trending low-lying sedimentary range separating the Pacific Coast to the west from the Willamette Valley to the east. The bridge crosses over the northeast flowing Soap Creek, which continues ± 5 miles north and east to the confluence with the Luckiamute River and a short length northeast to the Willamette River. The creek lies between Forest Peak and Vineyard Hill ridges which trend northeast.

The local geology at the bridge site consists of young alluvial deposits consisting of silt, sand, and gravel associated with Soap Creek (Wiley, 2009). The alluvium is underlain by Siletz River Volcanics. The Siletz River Volcanics consist of basalt flows, pillow basalt, and pillow breccia and the upper portion of the unit includes basaltic siltstone, sandstone, tuff, and conglomerate (Wiley, 2009).

Our borings encountered fill, fine- to coarse-grained alluvium, minor residual soil, and basalt bedrock. Both borings encountered basalt bedrock at relatively similar depths. The subsurface conditions encountered in the borings are consistent with the mapped geology. The bedrock encountered in the borings was designated as the Siletz River Volcanics on the boring logs based on the local geologic mapping (Wiley, 2009). Additional details of the conditions encountered in each boring are provided in the Subsurface Conditions section below and on the boring logs in Appendix B.

Seismicity and Faulting

We completed a literature review of nearby faults to evaluate the seismic setting and identify the potential seismic sources. Available information indicates the seismic hazard at the site is affected by the Cascadia Subduction Zone (CSZ) and crustal fault sources. A discussion of these earthquake sources is provided below.

Cascadia Subduction Zone (CSZ). The bridge site is ± 103 miles east of the surface expression of the 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 (intraslab) and along the inclined interface between the Juan de Fuca and North American 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 ± 325 years ago on January 26, 1700 (Nelson et al., 1995; Satake et al., 1996). A 2012 study of turbidites from the last $\pm 10,000$ 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 ± 76 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).

Crustal Faults. Crustal earthquakes dominate Oregon's seismic history. Crustal earthquakes occur within the North American Plate, typically at depths of ± 6 to 12 miles. No potentially active faults are mapped beneath the site (Walker and Duncan, 1989; Yeats et al., 1996; Wiley, 2009; USGS, 2022). Some concealed and approximate crustal faults are located within ± 10 miles of the site including the southern extension of the Corvallis fault (Class B). Only the Corvallis fault shows evidence of potential movement in the last ± 1.6 million years (Palmer, 1983; Yeats et al., 1996; USGS, 2016, 2022). The closest Class A fault is the Owl Creek fault located ± 6 miles to the south-southeast of the site (USGS, 2022). USGS Class A and Class B faults are defined 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 demonstrate the existence of a fault or suggests Quaternary deformation but might not be a potential source of significant earthquakes or evidence too strong to be a Class C but not enough to assign the fault to Class A.

The USGS website includes an interactive deaggregation tool, which allows evaluation of the contribution of the various seismic sources to the overall seismic hazard. The interactive deaggregation indicates the primary source affecting the seismic risk at the site is the CSZ (USGS, 2014).

FIELD EXPLORATION

We drilled two borings at the site between February 27 and February 28, 2025, using a CME 850 track-mounted drill rig operated by Crisman Pacific Strata Drilling, LLC. BH-1 was drilled in the approximate center of Beef Barn Road near the east abutment (Bent 1) and BH-2 was drilled in the approximate center of Beef Barn Road near the west abutment (Bent 2). The approximate boring locations are shown in Figure 2A (Appendix A).

The borings were advanced to a depth of ± 41 feet below the existing gravel surface using mud-rotary drilling and HQ wire-line, rock coring techniques. Disturbed soil and rock samples were obtained using a split-spoon sampler in conjunction with the Standard Penetration Test (SPT). The SPT, which is performed when the split-spoon sampler is driven, provides an indication of the stiffness or density of the soil. The sampler is driven three consecutive 6-inch increments and the blows required to drive the sampler through each increment are recorded. The number of blows required to drive the sampler the final 12 inches represents the standard penetration resistance or N-value in blows per foot (bpf). SPT refusal is defined by a penetration resistance exceeding 50 blows in a 6-inch increment. Sampling was conducted at various intervals to the approximate bedrock surface. Continuous, HQ-size rock coring was completed below ± 22.5 feet in BH-1 and ± 24 feet in BH-2.

Upon completion of the drilling, the boreholes were backfilled with bentonite in accordance with Oregon Water Resources Department (OWRD) guidelines. The backfill was capped with gravel.

The borings were continually logged by a Professional Engineer from FE during drilling. The final logs (Appendix B) were prepared based on a review of the field logs, the laboratory test results, and an examination of the samples in our office. Additional information regarding the boring logs and the definition of symbols and descriptive terms used in the logs are provided in the *Important Information and Symbols*, the *Common Soil Description Terms*, and the *Common Rock Description Terms* sheets included in Appendix B. The sheets also include a discussion of the interpretation of the subsurface profiles at the boring locations and the potential for inherent variations in the subsurface conditions across the site.

SURFACE AND SUBSURFACE CONDITIONS

A general discussion of the surface and subsurface conditions encountered in the borings is presented below. More detailed descriptions of the subsurface conditions encountered in each exploration are summarized in the appended boring logs.

Borehole 1 (Bent 1)

BH-1 was drilled near the center of Beef Barn Road, ± 40 feet east of the center of the existing bridge. The gravel surface elevation at BH-1 was estimated to be \pm El. 307.0.

Drilling encountered ± 2 feet of gray, dense, ± 2 -inch minus crushed rock (fill) followed by soft to stiff, low plasticity silt (alluvium) to ± 8 feet (\pm El. 299.0). Medium stiff to very stiff, high plasticity clay (alluvium) extends below the silt to ± 21 feet (\pm El. 286.0). The clay is underlain by medium stiff, low plasticity sandy silt (possible residual soil) to ± 21.5 feet (\pm El. 285.5), the approximate bedrock surface. We believe this material may be residual soil, which is rock that has weathered in place to a soil-like consistency.

Gray, slightly weathered, weak to strong (R2 to R4) basalt (Siletz River Volcanics) extends below the residual soil to ± 52 feet (\pm El. 255.0), the maximum depth of the exploration. Six core runs were completed in the basalt from ± 21.5 to 52 feet. Percent recovery ranged from ± 82 to 100 percent and the Rock Quality Designation (RQD) ranged from 76 to 96 percent.

Borehole 2 (Bent 2)

BH-2 was drilled near the center of Beef Barn Road, ± 40 feet west of the center of the existing bridge. The gravel surface elevation at BH-2 was estimated to be \pm El. 307.0.

Drilling encountered ± 4.5 feet of gray, medium dense, ± 2 -inch minus crushed rock (fill) followed by stiff to very stiff, high plasticity clay (alluvium) to ± 15 feet (\pm El. 292.0). The clay is underlain by medium dense, silty sandy gravel (alluvium) to ± 20 feet (\pm El. 287.0) followed by medium dense, silty rock fragments (residual soil) to 22 feet (\pm El. 285.0), the approximate bedrock surface.

Gray, slightly weathered, moderately strong to strong (R3 to R4) basalt (Siletz River Volcanics) extends below the residual soil to ± 41 feet (\pm El. 266.0), the maximum depth of the exploration. Four core runs were completed in the basalt from ± 24 to 41 feet. Percent recovery ranged from ± 90 to 100 percent and the Rock Quality Designation (RQD) ranged from 55 to 89 percent.

LABORATORY TESTING

Moisture Contents, Percent Fines, Atterberg Limits Tests, and Bulk Density

The laboratory testing included moisture content determinations, Atterberg limits, bulk density and percent fines tests to help classify the soils above the bedrock according to the Unified Classification System (USCS) and estimate their overall engineering properties. The test results are summarized in Table 1C and 2C (Appendix C), and the moisture contents are also shown on the boring logs. Non-tested samples were visually classified in general accordance with the procedures outlined in ASTM D2487 and ASTM D2488. The USCS symbols shown on the logs for untested samples should be considered approximations.

Consolidation Testing

One-dimensional consolidation tests were run on Shelby tube sample SH-1-6, obtained from BH-1 at a depth of ± 15 to 17 feet, and on Shelby tube sample SH-2-5, obtained from BH-2 at a depth of ± 13 to 15 feet. The tests were completed to estimate the compressibility of the clay for use in our settlement analysis.

The test results for sample SH-1-6 suggest compression and recompression indices (C_{ce} and C_{re}) of 0.235 and 0.0067, respectively. A preconsolidation pressure (P'_c) of ± 4.0 ksf was also estimated. The test results for sample SH-2-5 suggest compression and recompression indices (C_{ce} and C_{re}) of 0.214 and 0.0075, respectively. A preconsolidation pressure (P'_c) of ± 3.4 ksf was also estimated. The test reports are shown in Figure 1C and 2C (Appendix C).

Unconfined Compression

Two unconfined compression (q_u) tests were completed on selected rock core samples to estimate the bedrock strength. The test results and stress versus strain plots are shown in Figures 3C and 4C (Appendix C). The results indicate q_u values of 3,155 and 7715 lb/in² (psi), corresponding to a rock strength of weak (R2) and medium strong (R3), respectively.

HYDRAULICS AND SCOUR

The hydraulic analysis completed by RDG indicates an ordinary high water (OHW) elevation of El. 297.0 and a 100-year (design) flood elevation of El. 303.5. The low chord at the center of the new bridge will provide 3.25 feet of clearance above the design flood elevation.

The estimated bed scour elevation is El. 292.8, which corresponds to a bed scour depth of 0.3 feet. Class 200 rip rap is planned around the abutments to protect the spread footing foundations.

SEISMIC CONSIDERATIONS

Site Response Spectrum

Response spectra for the site were developed based on the 2024 ODOT Geotechnical Design Manual (GDM) for an Operational Category of "Ordinary" that corresponds to only providing the "life safety" criteria. The "life-safety" (i.e., no collapse) seismic performance criterium assumes earthquake ground motions having a 1,000-year average return period.

The response spectra for the bridge site were developed based on the current ODOT GDM "life-safety" and "operational" criteria (ODOT, 2024). 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 outlined in the 2024 AASHTO LRFD Bridge Design Specifications (AASHTO BDS) with modifications recommended in the 2024 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 spreadsheet, which is based on the 2014 USGS seismic hazard maps.

Following the 2024 AASHTO BDS 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. Scaling factors F_{pga} , F_a , and F_v were selected based on ODOT GDM Tables 13-1, 13-2, and 13-3. The life-safety response spectrum and design parameters are shown in Figure 3A.

Ground motions for the CSZE operational response spectrum were obtained using the Portland State University (PSU) Acceleration Response Spectra website (PSU, 2017). The PSU website requires inputting latitude and longitude coordinates for the project site and an average shear wave velocity for the upper 30 meters (V_{s30}) of the soil profile. In-situ shear wave velocity (V_s) data was not available for this site. We calculated a design V_{s30} of ± 270 m/s by assigning anticipated V_s values to each soil layer using V_s data based on our experience with in-situ shear wave velocity tests on similar soils and correlations to SPT N-values based on the procedures outlined in the *Guidelines for Estimation of Shear Wave Velocity Profiles* (Wair et al. 2012). The operational response spectrum is also shown in Figure 3A.

Liquefaction

Liquefaction is typically observed in saturated deposits of loose sand and non-plastic to low plasticity silt subjected to intense ground shaking. These soil conditions were not encountered in the explorations. Therefore, the risk of liquefaction at the site is negligible.

FOUNDATION ANALYSIS AND DESIGN RECOMMENDATIONS

Discussion of Foundation Options

Bedrock was encountered within ± 22 feet of the ground surface in both borings. Conventional driven piles were considered. However, due to the planned relatively deep abutment walls, the piles (below the cap) would be short and would have very limited penetration due to the high bedrock strength. As a result, lateral pile stability would have been a design concern. We also considered predrilled and grouted-in-place piles, but this system was cost prohibitive. Therefore, spread footings were ultimately selected to support the new bridge.

Bearing Capacity

The allowable bearing capacity was estimated for new footings placed within 10 feet (\pm El. 300 to El. 301) of the finish grade on a 12-inch-thick layer of Selected Granular Backfill or Granular Structure Backfill underlain by undisturbed, stiff to very stiff clay. The granular backfill should extend to a minimum of 12 inches beyond the edges of the new footings.

Undrained shear strength values generated from field vane shear tests on the Shelby Tube samples obtained from a depth of \pm 5 to 6 feet below the existing grade ranged from 1.2 ksf to greater than 2 ksf. Therefore, we used an undrained shear strength of 1.4 ksf to estimate the allowable bearing capacity of the fine-grained soils. Our calculations suggest an allowable bearing capacity of 3,800 psf, assuming a typical factor of safety of 3. However, we recommend using an allowable bearing capacity of 3,000 psf for design based on the results of the settlement analyses (discussed below).

Settlement

A settlement analysis was performed using the computer program Settle3D in combination with the laboratory consolidation data presented above. The analyses assumes the footings will be supported on a minimum of 12 inches of granular backfill that extends a minimum of 12 inches beyond the foundation edges. A groundwater elevation of El. 297.0 was used based on the OHW elevation.

The consolidation data suggests the clay is overconsolidated. To estimate the settlement, we used a range of recompression indices (C_{re}) of 0.0067 to 0.0085 for the fine-grained soils. Predominately coarse-grained soils encountered in the soil profile of BH-2 will settle immediately as they are loaded. Therefore, we assigned elastic moduli to these soils that are consistent with reported correlations with SPT values.

The contact pressure at the base of the footings due to sustained loading was assumed to be 3,000 psf. The estimated settlements for foundations are summarized in Table 1.

Table 1. Summary of Foundation Settlement

Bent	Footing Size (feet)	Sustained Loading (psf)	Estimated Total Settlement (inches)
1	6x20	3,000	1.1
2	6x20	3,000	1.2

Sliding Resistance

We estimated a friction coefficient of 0.5 for sliding between cast-in-place (CIP) concrete footings and the granular backfill. We do not anticipate the footing to translate sufficiently to develop a passive earth pressure. Therefore, the at-rest earth pressure provided below should be used for design.

APPROACHES AND EMBANKMENTS

Embankment Settlement

The replacement bridge will be constructed along the current horizontal alignment. However, the vertical alignment of the east approach will be raised up to 3.4 feet and the vertical alignment of the west approach will be raised up to 4.2 feet. Therefore, we completed a settlement analysis to estimate the settlement induced by the new embankment fill.

Our calculations suggest that where the approach fill is thickest at the abutments, an additional $\frac{1}{4}$ inch of settlement should be anticipated. This will be in addition to the settlement values provided in Table 1.

Lateral Earth Pressures for Abutment Walls and Wingwalls

Lateral Earth Pressures. Earth pressure loads for the abutments and wingwalls were estimated based on the recommendations found in the AASHTO (2024) and the Oregon Department of Transportation Geotechnical Design Manual (ODOT GDM). We assume Granular Wall Backfill (Section 00510.12) or Granular Structural Backfill (Section 00510.13) will be used in the zone behind the walls as defined in the 2024 Oregon Standard Specifications for Construction (ODOT, 2024). For these materials, we used an internal friction angle (ϕ') of 34 degrees and a moist unit weight (γ_m) of 125 pcf. 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 active earth pressure conditions. This corresponds to ± 0.1 inch of deflection for a 10-foot tall wall. Typically, abutment walls and wingwalls deflect sufficiently to mobilize active earth conditions. However, pile caps that are pinned to the bridge may not be free to deflect. Therefore, we calculated earth pressures for both the active and at-rest conditions. We assume the structural designer will select the appropriate design conditions based on the flexibility of the structure.

For unrestrained walls (i.e., walls able to deflect or rotate at least $0.001 * H$), we recommend evaluating the static earth pressure component 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. The resultant of the active earth pressures will act at $H/3$ above the base of the wall.

For restrained walls, we recommend evaluating the static earth pressure component 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. The resultant of the at-rest earth pressures will act at H/3 above the base of the wall.

Traffic Surcharge. We calculated the traffic load applied to the abutment walls using an equivalent soil surcharge based on AASHTO (2024) Table 3.11.6.4-1. The values in AASHTO (2024) Table 3.11.6.4-1 indicate a minimum surcharge height of 3.0 feet for an abutment wall height of 10 feet. Using a unit weight of 125 pcf and a surcharge height of 3.0 feet results in a nominal uniform surcharge pressure of 375 psf. Applying k_a of 0.28 results in a nominal, uniform lateral pressure of 105 psf for unrestrained walls. Applying k_o of 0.44 results in a nominal, uniform lateral pressure of 165 psf for restrained walls. The resultant of the uniform traffic surcharge pressure acts at H/2 above the base of the wall.

Construction Surcharge. The ODOT GDM requires abutments also be designed for a minimum uniform live load surcharge of 250 psf to account for potential construction surcharge loading. Applying a k_a of 0.28 for the active case results in a nominal uniform lateral pressure of 70 psf. Applying a k_o of 0.44 results in a nominal uniform lateral pressure of 110 psf. The resultant of the uniform traffic surcharge pressure acts at H/2 above the base of the wall.

The construction surcharge load will precede the traffic surcharge loads described above and should not be combined with those loads. Construction loads are also assumed to not act concurrently with seismic loads.

Seismic Loading. The ODOT GDM requires walls that affect the performance or structural integrity of the bridge be designed for seismic loading corresponding to an earthquake having 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 0.5 times the estimated ground surface acceleration (A_s) of 0.38g. A_s is based on the USGS PGA (on rock) of 0.29g and an AASHTO (2024) site factor (F_{pga}) of 1.31 for a 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. We used a k_h of 0.19g 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 calculated as $7.9 \text{ psf} * H$, where H is the wall height. 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.

The recommended static and seismic lateral earth pressures are summarized in Table 2.

Table 2. Lateral Earth Parameters for Abutment and Wing Wall Design

Parameter	Source	Value
Active Earth Pressure Coefficient, k_a	$\tan^2(45 - \phi/2)$	0.28
At-Rest Earth Pressure Coefficient, k_o	$1 - \sin(\phi)$	0.44
Active Equivalent Fluid Density	$k_a * \gamma_{\text{backfill}}$	35 pcf
At-Rest Equivalent Fluid Density	$k_o * \gamma_{\text{backfill}}$	55 pcf
Traffic Load Surcharge for Abutment Walls (Active)	$375.0 \text{ psf} * k_a$	105 psf
Traffic Load Surcharge for Abutment Walls (At Rest)	$375.0 \text{ psf} * k_o$	165 psf
Construction Surcharge Load (active)	$250 * k_a$	70 psf
Construction Surcharge Load (at-rest)	$250 * k_o$	110 psf
Seismic Earth Pressure for Wall Backfill for 1,000-year event (assumes 1 to 2 inches of displacement)	Mononobe-Okabe	$7.9 * H \text{ psf}$

The appropriate load factors (γ_p) provided in AASHTO Table 3.4.1-2 should be applied to the nominal pressures to estimate the factored lateral earth loads. Selection of the appropriate load factors are dependent upon the load case being analyzed.

CONSTRUCTION RECOMMENDATIONS

Specifications

All specifications refer to the 2024 Oregon Standard Specifications for Construction (ODOT, 2024), except as noted. It is also assumed these specifications will be referenced for general or specific items not addressed in this report.

Spread Footing Subgrade Preparation

Perform the work in general accordance with Preparation of Foundations (Section 00510.46) with the following additions:

1. Excavate for the foundations using a hoe equipped with a smooth bucket. The excavation should extend a minimum of 12 inches beyond the outside edge of the foundation.
2. Overexcavate soft material exposed at the foundation subgrade elevation. The subgrade is required to have a minimum undrained shear strength of 1.4 ksf. The subgrade strength shall be verified by a FE representative prior to placing the Dense Graded Base Aggregate.

3. Provide a minimum of 12 inches of compacted, Dense Graded Base Aggregate beneath the footings. The aggregate should meet the requirements of Section 2630.10 for 1"-0. The Base Aggregate should extend at least 12 inches beyond the edge of the footing.
4. Moisture condition and compact the Base Aggregate in loose lifts not exceeding 12 inches. Thinner lifts may be required if light or hand-operated equipment is used. Compact the material to a minimum of 95% relative compaction. The maximum dry density of ASTM D698 should be used as the standard for estimating relative compaction. Subgrade compaction is not required.

Excavations/Shoring/Dewatering

We anticipate excavations up to ± 7 feet deep will be required for construction of the new foundations. The excavations will extend through the existing embankment fill and alluvial silt and clay. Groundwater is not expected to be a consideration during the dry weather months. However, the creek level could impact the excavation work during high water flows or flood events typically occurring during the wet, winter months.

Temporary slopes no steeper than 1.5(H):1(V) should be planned, unless shored. Steeper slopes may be permitted if approved by FE during construction. Flatter slopes may be required to control erosion and sloughing during wet weather. The design of temporary shoring for temporary cut slope support, if required, should be completed in conformance with Section 16.3.26 of the ODOT GDM.

Approach Embankments

The approach work will include raising the approach grades. The following construction recommendations are based on the requirements of Section 00330.

Subgrade Preparation. Excavations should be completed in accordance with Section 00330.41. Moisture-conditioning and subgrade compaction should be completed in accordance with Section 00330.43. Soft or loose subgrade, if encountered, should be mitigated by over-excavating and replacing the unsuitable material with approved material. Surface organics should be stripped and grubbed from the existing embankment areas.

Embankment Fill. Embankment widening 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 at 2(H):1(V) or flatter during dry weather. Base Aggregate may also be used, especially in areas where the embankment thickness is less than 2 feet. Stone Embankment Material (Section 00330.16) is required for 1.5(H):1(V) finished slopes.

Abutments Walls and Wingwalls. Granular Wall Backfill (Section 00510.12) or Granular Structural Backfill (Section 00510.13) should be used to backfill the abutment walls and wingwalls. The wall backfill should be placed and compacted as specified in Section 00510.48. Compaction within 5 feet of the walls should be completed using light, vibratory compaction equipment.

LIMITATIONS

Design Review, Construction Observation, and Testing

We should be provided with the opportunity to review all drawings and specifications that pertain to foundation, embankment, and wall construction. Bridge foundation construction will require field confirmation of the subgrade conditions beneath the new foundations. We recommend FE be retained to provide those observations. Field density tests should be run on all imported, granular materials. Compaction of fill that is too coarse for density testing should be evaluated by observation of the compaction method and proof-rolling (where appropriate) with a loaded dump truck or other approved vehicle. We recommend FE be retained to provide any necessary geotechnical consultation during construction.

Variation of Subsurface Conditions, Use of This Information, 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 final design loads and be present during construction to confirm the assumed foundation conditions. No changes to 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 Oregon State University and their design consultants for the Soap Creek Bridge Replacement project in Benton County, Oregon. The 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.

It has been a pleasure assisting you with this phase of the project. Please let us know if you have questions or require further assistance.

Attachments

REFERENCES

- AASHTO, 2024, *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials (AASHTO), 10th Edition.
- 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.
- Dewey, J.W., Hopper, M.G., Wald, D.J., Quitariano, 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.
- 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, website: <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., and Vallier, T., 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, website: <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, 2024, Geotechnical Design Manual (GDM): Delivery & Operations Division, Engineering & Technical Services Branch, Oregon Department of Transportation (ODOT), May 2024.
- ODOT, 2024, *Oregon Standard Specifications for Construction, 2024*, Oregon Department of Transportation.
- 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.

- PSU, 2017, *CSZ Acceleration Response Spectrum Tool*, developed by Portland State University (PSU), accessed 2025, website: <https://csz.cee.pdx.edu>.
- Satake, K., Shimazaki, K., Tsuji, Y., and Ueda, K., 1996, *Time and Size of a Giant Earthquake in Cascadia Inferred from Japanese Tsunami Records of January 1700*: *Nature*, vol. 379, no. 6562, p. 246-249.
- 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, latitude/longitude search, reference material has no specific release date, website: <https://earthquake.usgs.gov/hazards/interactive/index.php>, accessed July 2025.
- USGS, 2016, *Quaternary Fault and Fold Database for the United States - Oregon*: U.S. Geological Survey (USGS), Class A and B faults, reference material has no specific release date, maintained limited number of metadata fields after January 12, 2017, website: https://earthquake.usgs.gov/cfusion/qfault/query_results_AB.cfm, accessed July 2025.
- USGS, 2022, *Quaternary Fault and Fold Database of the United States - Interactive Fault Map*: U.S. Geological Survey (USGS), Qfaults and NSHM 2024 Fault Source Layers, reference material has no specific release date, updated September 27, 2022 website: <https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf>, accessed July 2025.
- Wair, B. R., DeJong, J. T., and Shantz, T., 2012, *Guidelines for Estimation of Shear Wave Velocity Profiles*: Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, CA, PEER Report 2012/08, 68 p.
- 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.
- Weaver, C. S., and Shedlock, K. M., 1996, *Estimates of Seismic Source Regions from the Earthquake Distribution and Regional Tectonics in the Pacific Northwest*: 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, vol. 1, p. 285-306.
- Wiley, T.J., 2009, *Preliminary Geologic Map of the Lewisburg 7.5' Quadrangle, Benton, Linn, Polk, and Marion Counties, Oregon*: Oregon Department of Geology and Mineral Industries (DOGAMI), Open-file Report OFR O-09-05, 10 p., 1 Plate, Scale = 1:24,000.

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.

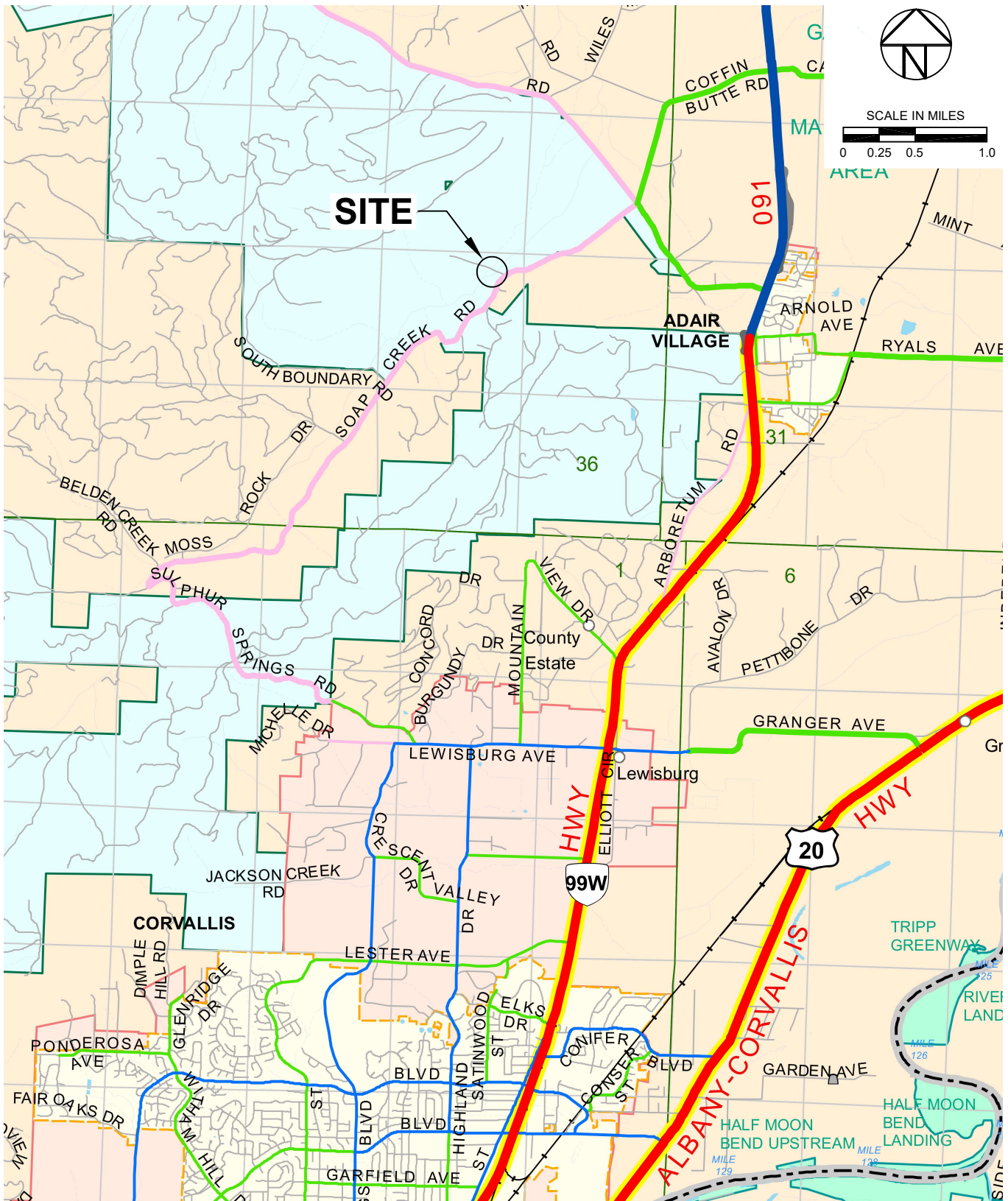
Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T.A., 1996, *Tectonics of the Willamette Valley, Oregon: in* Roger, A.M., Walsh, T.J., Kockelman, W.J., and Priest, G.R., eds., *Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest*, U.S. Geological Survey (USGS), Volume 1, Professional Paper 1560, Document and Plates 1 to 3, Scale = 1:100,000, p. 183-222.



Appendix A

Figures

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Note: Base map obtained from ODOT website.



LEGEND

⊕ BH-1 BORING NUMBER AND LOCATION

NOTES:

1. BORING LOCATIONS WERE ESTABLISHED USING A METAL TAPE AND ARE APPROXIMATE.
2. AERIAL IMAGE OBTAINED FROM GOOGLE EARTH.
3. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.

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AERIAL IMAGE AND BORING LOCATIONS

FIGURE NO.

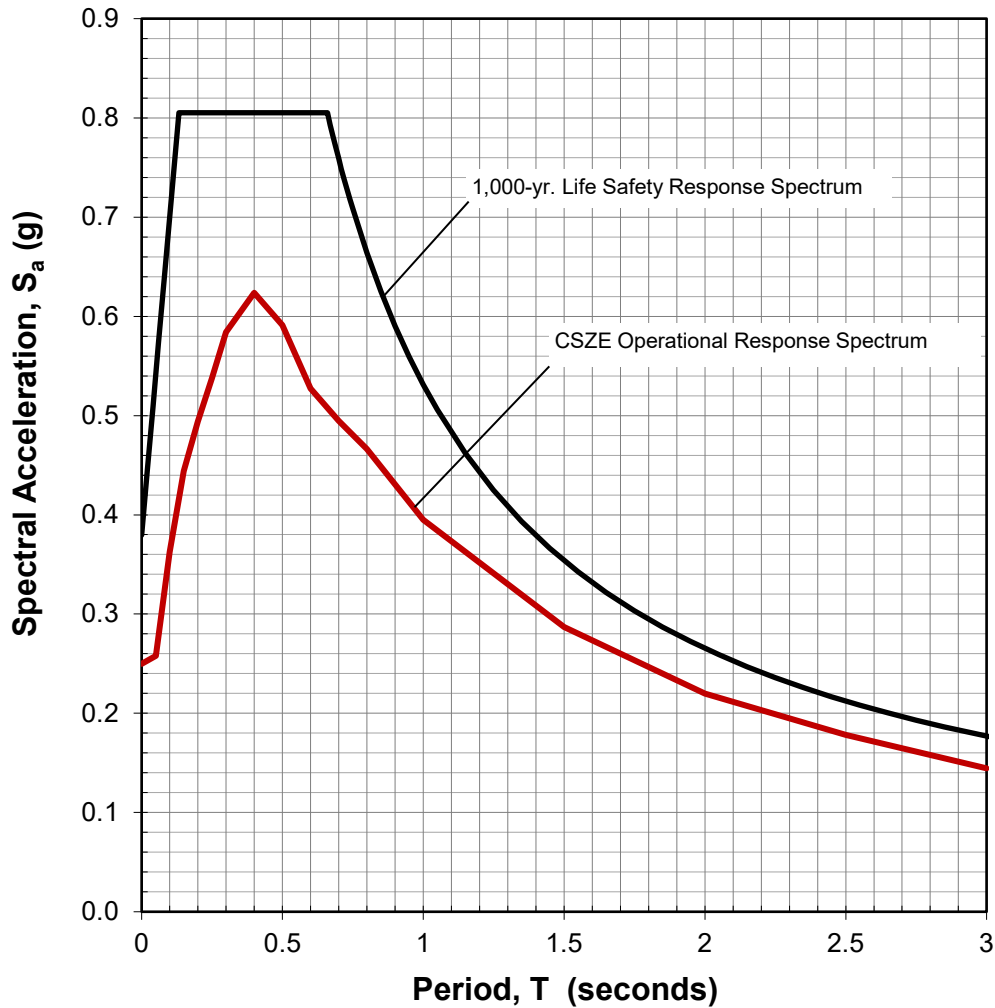
PROJECT NO.
2251001

DATE:
Apr. 14, 2025

DRAWN BY:
EJG

SOAP CREEK BRIDGE REPLACEMENT
BENTON COUNTY, OREGON

2A



Notes:

1. The 1,000-yr. Life Safety Design Response Spectrum is based on AASHTO 2020 Section 3.10.4 using the following parameters:

Site Class = D	Damping = 5%	
PGA = 0.29	$F_{pga} = 1.31$	$A_s = 0.38$
$S_S = 0.62$	$F_a = 1.31$	$S_{DS} = 0.81$
$S_1 = 0.25$	$F_v = 2.09$	$S_{D1} = 0.53$

PGA, S_S , and S_1 values are based on USGS 2014 seismic hazard maps and were obtained using the ODOT ARS spreadsheet. F_{pga} , F_a , and F_v were established based on 2024 ODOT GDM Tables 13-1 to 13-3 assuming a Site Class D profile.

2. The CSZE values were obtained using the PSU CSZE calculator assuming $V_{s30} = 270$ m/s consistent with the average assumed shear wave velocity for a Site Class D profile.
3. Site location: Latitude 44.6780, Longitude -123.2604.

FIGURE 3A
AASHTO 2024 GENERAL PROCEDURE RESPONSE SPECTRUM
AND CSZE ACCELERATION RESPONSE SPECTRUM

Soap Creek Bridge Replacement
 Benton County, Oregon
 Project No.: 2251001



Appendix B

Boring Logs

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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.

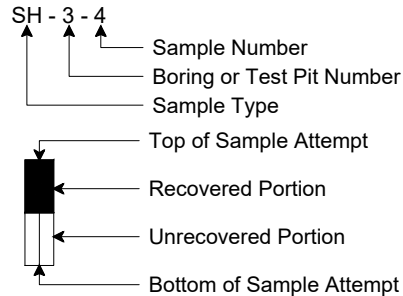
VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

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 (%)

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 |

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

- | | | | | | |
|--|----------|--|--------|--|-----------|
| | Concrete | | Silt | | Basalt |
| | Organics | | Gravel | | Sandstone |
| | Clay | | Sand | | Siltstone |

WATER TABLE

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

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.

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 (Predominately Decomposed)	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.

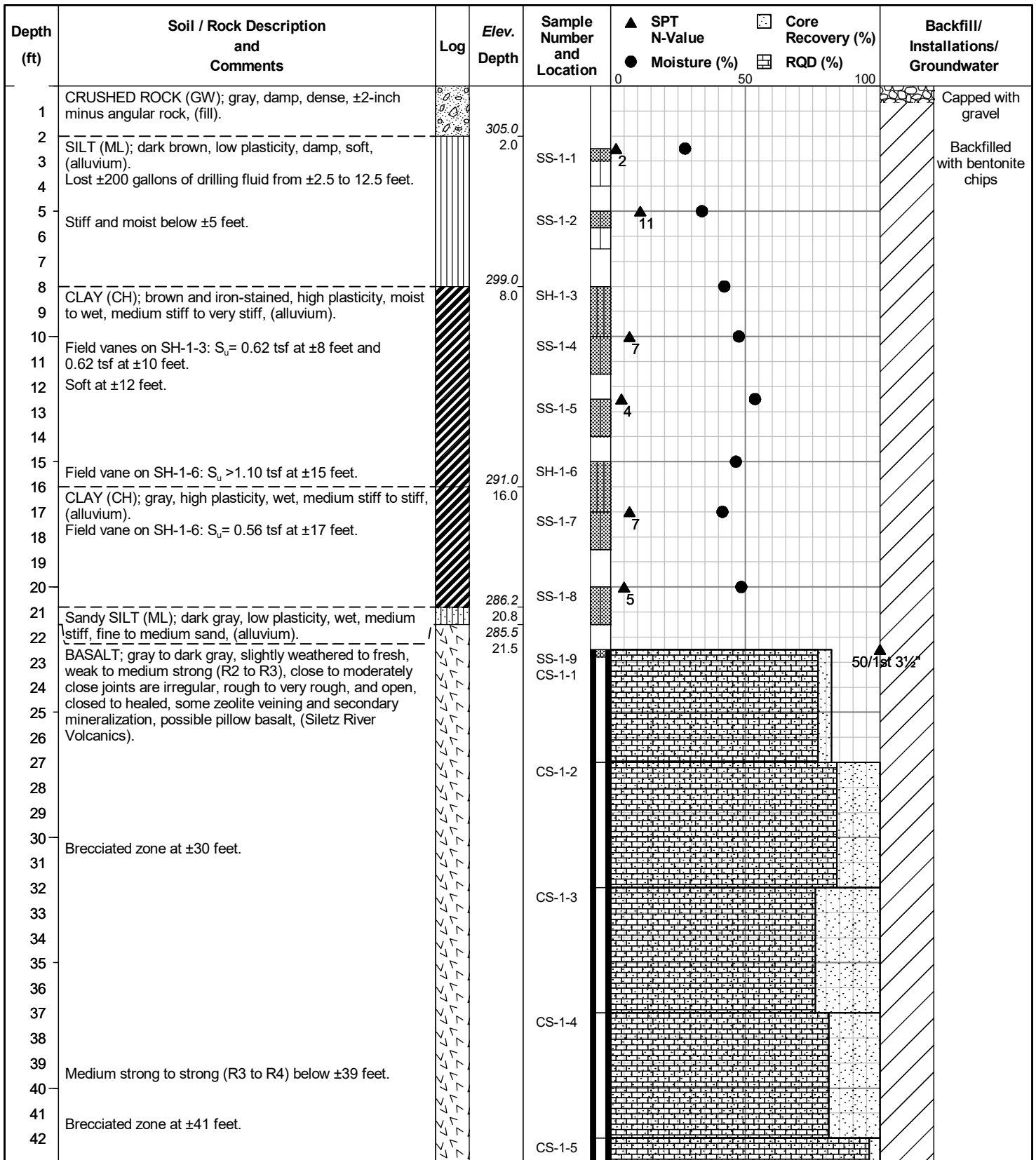
Spacing (metric)	Spacing (imperial)	Spacing Term	Bedding/Foliation
< 6 cm	< 2 in.	Very Close	Very Thin (Laminated)
6 cm - 30 cm	2 in. - 1 ft.	Close	Thin
30 cm - 90 cm	1 ft. - 3 ft.	Moderately Close	Medium
90 cm - 3.0 m	3 ft - 10 ft.	Wide	Thick
> 3.0 m	> 10 ft.	Very Wide	Very Thick (Massive)

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		


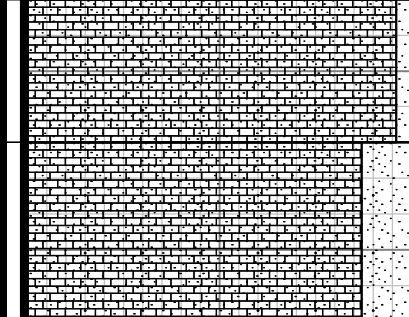
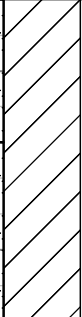

Rock Quality Designation (RQD) is the cumulative length of intact rock core pieces 4 inches or longer excluding breaks caused by drilling and handling divided by run length, expressed as a percentage.



Project No.: 2251001
 Surface Elevation: 307.0 feet (Approx.)
 Date of Boring: February 27, 2025

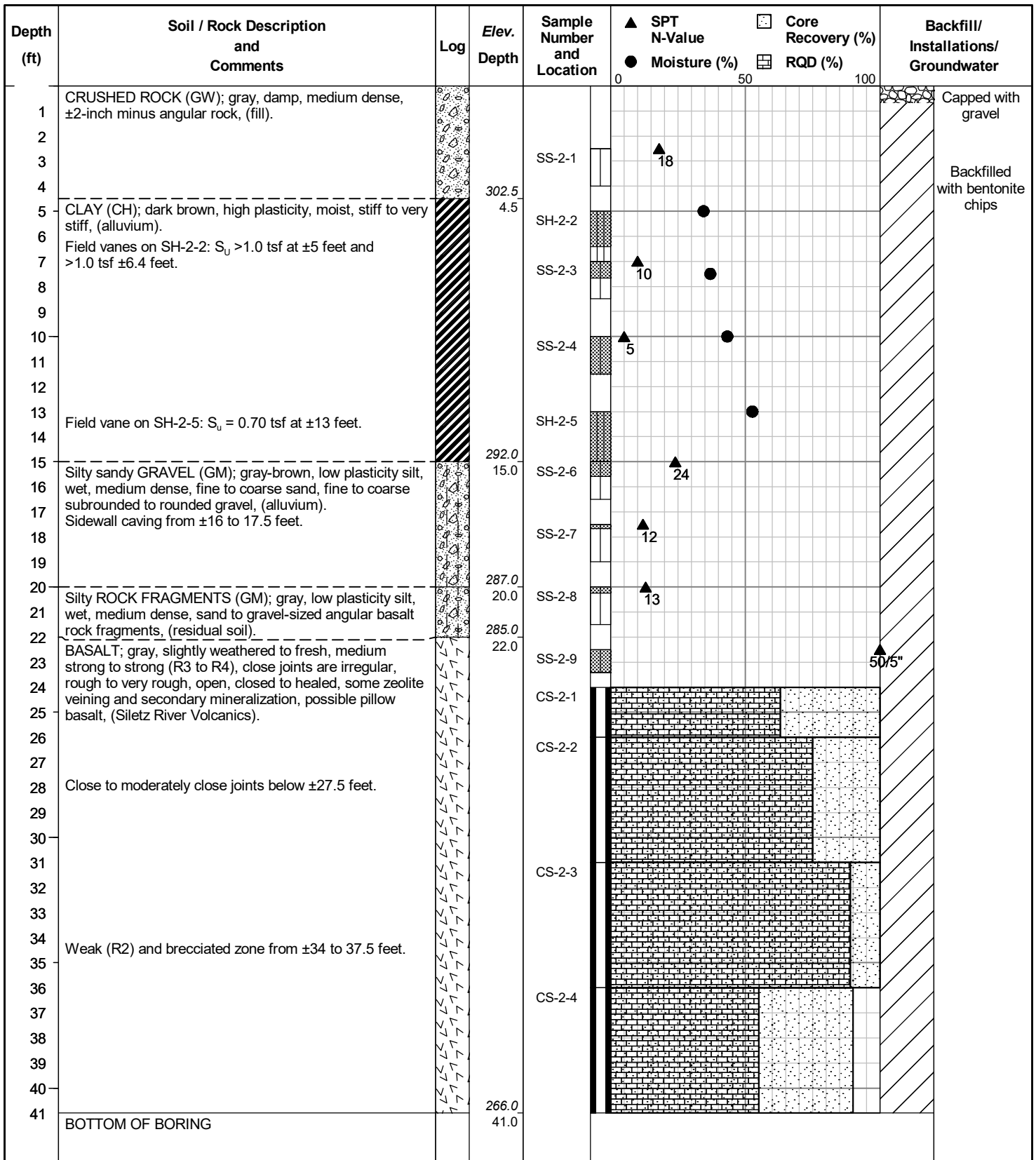
BORING LOG: BH-1
Soap Creek Bridge Replacement
Benton County, Oregon



Depth (ft)	Soil / Rock Description and Comments	Log	Elev. Depth	Sample Number and Location	▲ SPT N-Value ● Moisture (%) 0 50 100	□ Core Recovery (%) ▣ RQD (%)	Backfill/ Installations/ Groundwater
44 45 46 47 48 49 50 51 52			255.0 52.0	CS-1-6			
	BOTTOM OF BORING						

Project No.: 2251001
 Surface Elevation: 307.0 feet (Approx.)
 Date of Boring: February 27, 2025

BORING LOG: BH-1
Soap Creek Bridge Replacement
Benton County, Oregon



Project No.: 2251001
 Surface Elevation: 307.0 feet (Approx.)
 Date of Boring: February 28, 2025

BORING LOG: BH-2
Soap Creek Bridge Replacement
Benton County, Oregon



Appendix C

Field and Laboratory Test Results

*Professional
Geotechnical
Services*

Foundation Engineering, Inc.

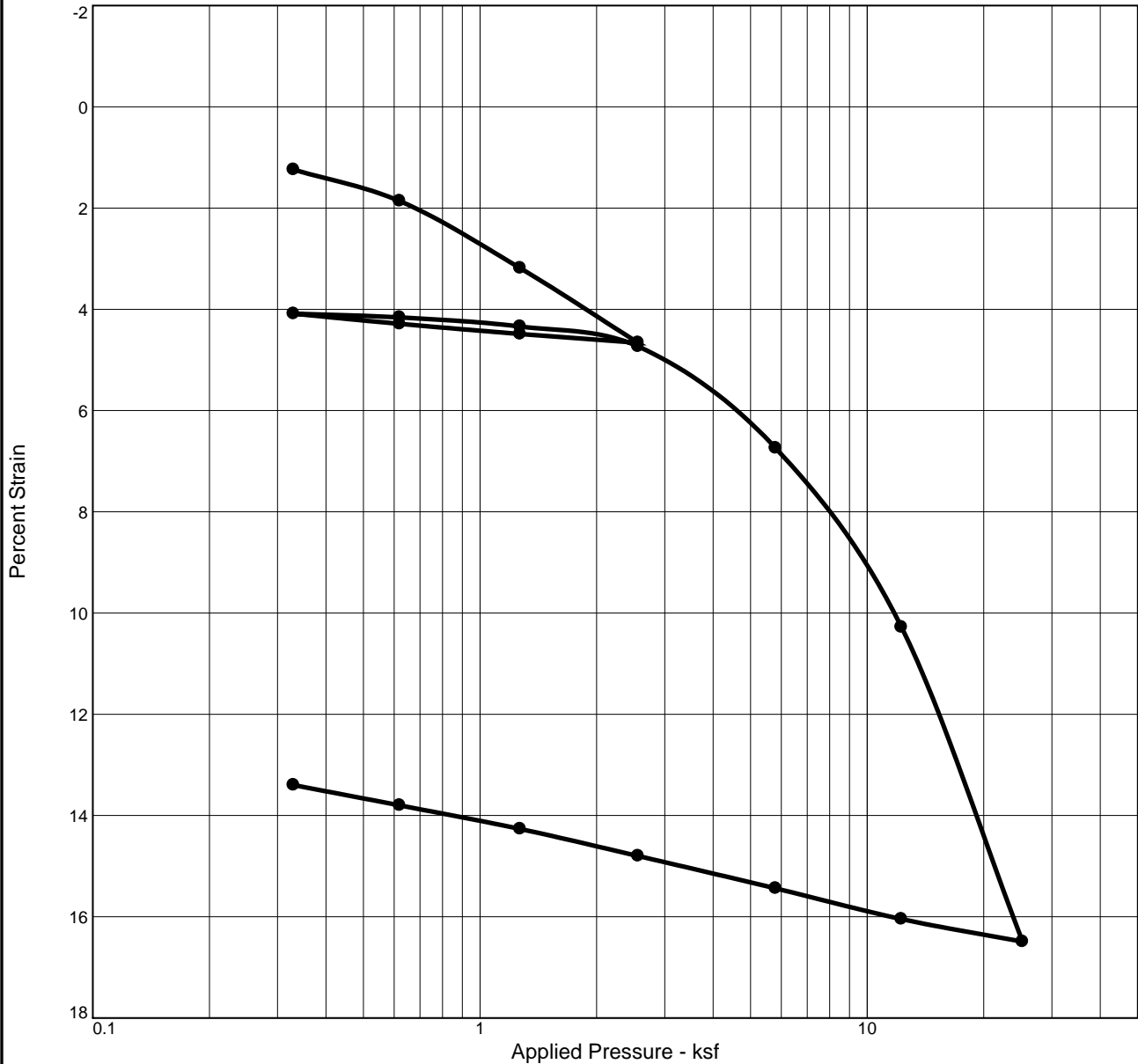
**Table 1C. Moisture Contents (ASTM D2216), Percent Fines (ASTM D1140),
 and Atterberg Limits (ASTM D4318)**

Sample Number	Sample Depth (ft)	Moisture Content (%)	Fines (%)	Atterberg Limits			USCS Classification
				LL	PL	PI	
SS-1-1	2.5 – 4.0	27.6					
SS-1-2	5.0 – 6.5	33.9					
SH-1-3	8.0 – 10.0	42.2		78	25	53	CH
SS-1-4	10.0 – 11.5	47.6					
SS-1-5	12.5 – 14.0	53.6					
SH-1-6	15.0 – 17.0	46.5					
SS-1-7	17.0 – 18.5	41.5		55	21	34	CH
SS-1-8	20.8 – 21.5	48.5	66.3				
SH-2-2	5.0 – 7.0	34.5					
SS-2-3	7.5 – 9.0	37.0					
SS-2-4	10.0 – 11.5	43.3		65	26	39	CH

Table 2C. Bulk Density (ASTM D2937)

Sample Number	Sample Depth (ft)	Moisture Content %	Moist Density (pcf)	Dry Density (pcf)
SH-2-5	13.0 – 15.0	52.6	106.8	70.0

CONSOLIDATION TEST REPORT



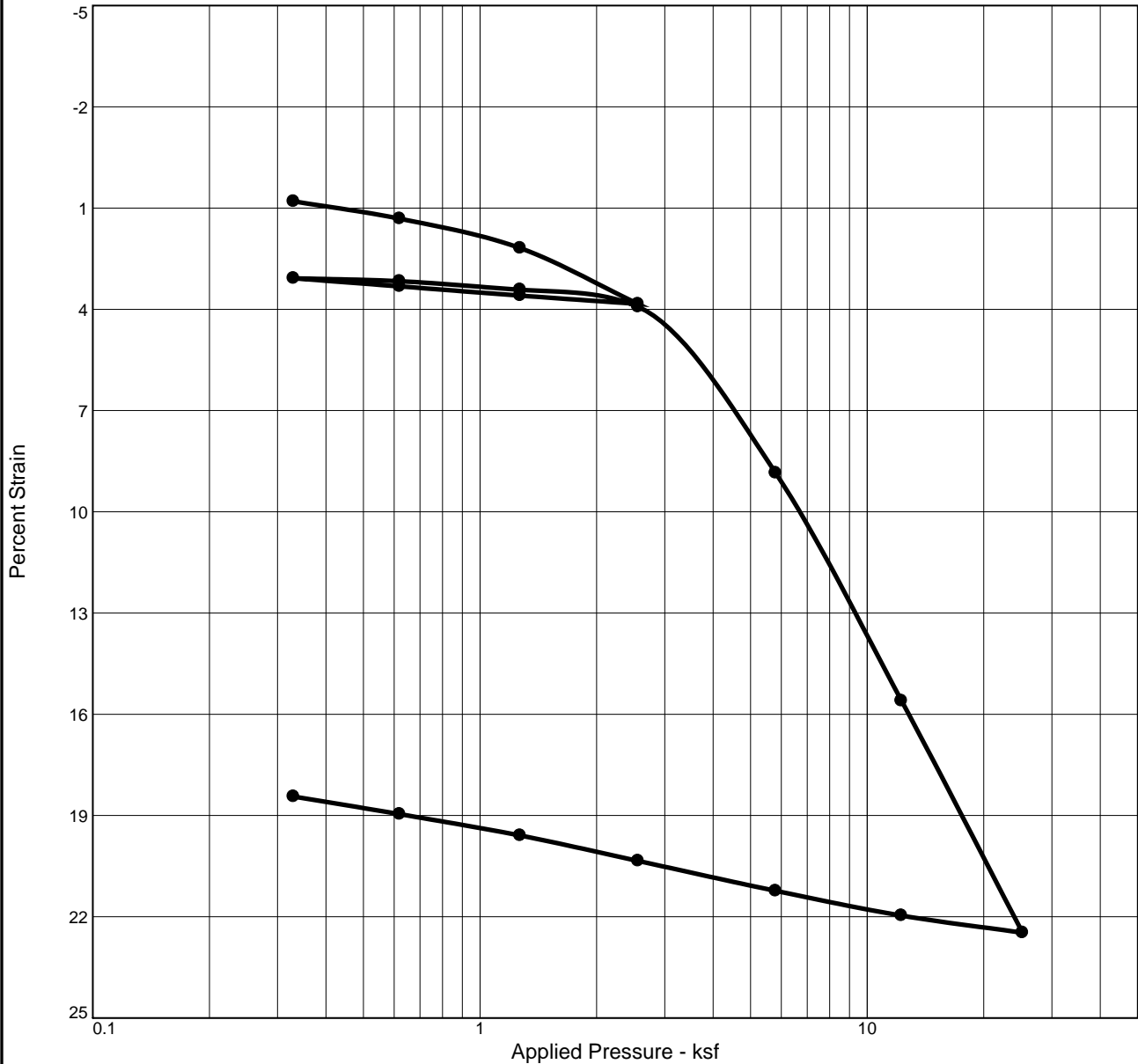
Natural Sat.	Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P _c (ksf)	C _{ce}	C _{re}	Swell Press. (ksf)	Swell %	e _o
96.6 %	41.9 %	77.0			2.65		4.0	0.007	0.235			

MATERIAL DESCRIPTION	USCS	AASHTO
Gray, medium stiff to stiff, high plasticity clay	CH	

Project No. 2255001 Client: Foundation Engineering, Inc. (Project No.: 2251001) Project: Soap Creek Bridge Replacement	Remarks:
Source of Sample: 9934 Depth: 15.0'-17.0' Sample Number: SH-1-6 FEI Testing & Inspection, Inc.	
Tangent, OR	

Figure 1C

CONSOLIDATION TEST REPORT

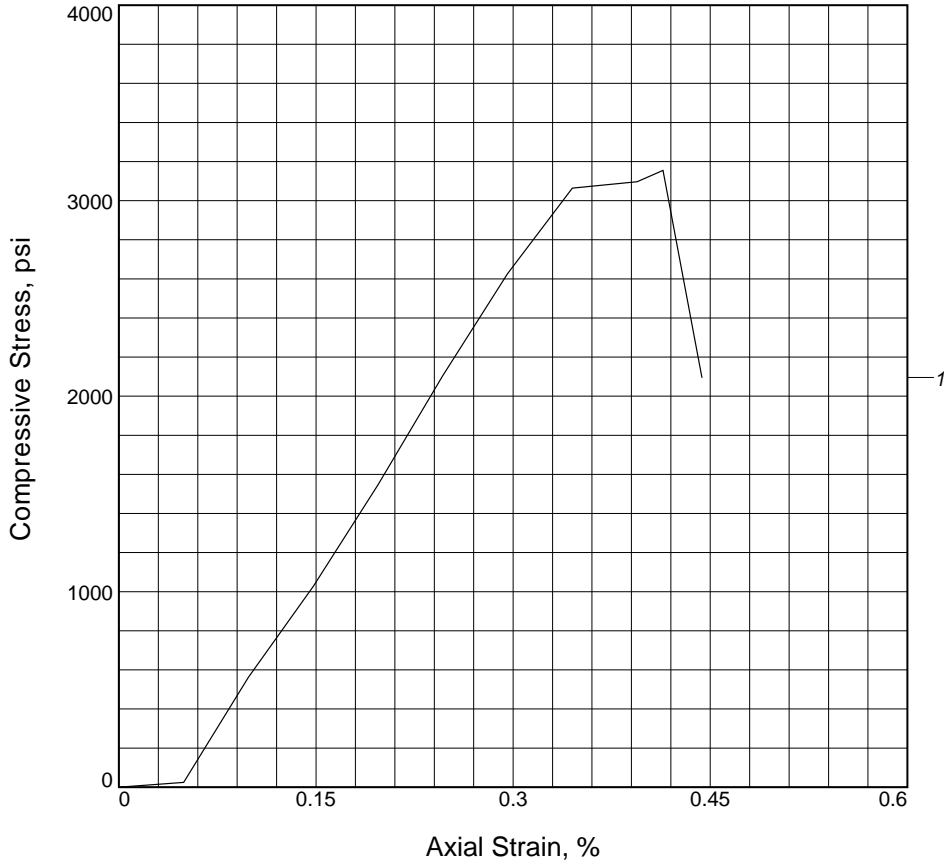


Natural Sat.	Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P _c (ksf)	C _{ce}	C _{re}	Swell Press. (ksf)	Swell %	e _o
102.9 %	51.7 %	71.0			2.65		3.4	0.214	0.008			

MATERIAL DESCRIPTION	USCS	AASHTO
Dark brown, stiff to very stiff, high plasticity clay	CH	

Project No. 2255001 Client: Foundation Engineering, Inc. (Project No.: 2251001) Project: Soap Creek Bridge Replacement Source of Sample: 9934 Depth: 13.0'-15.0' Sample Number: SH-2-5 FEI Testing & Inspection, Inc. Tangent, OR	Remarks: <div style="text-align: right;">Figure 2C</div>
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UNCONFINED COMPRESSION TEST



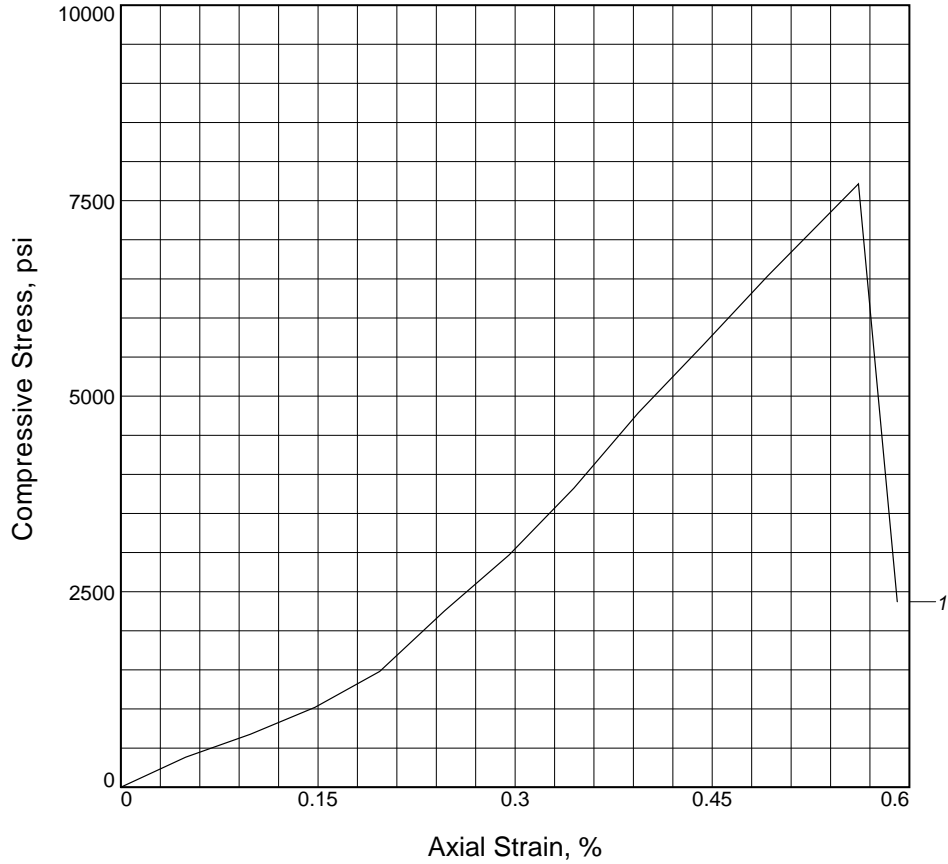
Sample No.	1			
Unconfined strength, psi	3154.65			
Undrained shear strength, psi	1577.32			
Failure strain, %	0.4			
Strain rate, in./min.	0.005			
Water content, %	4.9			
Wet density, pcf	161.5			
Dry density, pcf	154.0			
Saturation, %	89.3			
Void ratio	0.1550			
Specimen diameter, in.	2.37			
Specimen height, in.	5.07			
Height/diameter ratio	2.14			

Description: Gray, slightly weathered, weak (R2) Basalt

LL =	PL =	PI =	Assumed GS= 2.85	Type: HQ Core
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<p>Project No.: 2255001-613</p> <p>Date Sampled: 2-27-25</p> <p>Remarks:</p> <p>Figure: 3C</p>	<p>Client: Foundation Engineering, Inc. (Project No.: 2251001)</p> <p>Project: Soap Creek Bridge</p> <p>Source of Sample: 9787 Depth: 26.3'-26.7'</p> <p>Sample Number: CS-1-1</p> <hr/> <p style="text-align: center;">UNCONFINED COMPRESSION TEST FEI Testing & Inspection, Inc. Tangent, OR</p>
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UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	7714.68			
Undrained shear strength, psi	3857.34			
Failure strain, %	0.6			
Strain rate, in./min.	0.005			
Water content, %	3.5			
Wet density, pcf	168.0			
Dry density, pcf	162.4			
Saturation, %	87.8			
Void ratio	0.1148			
Specimen diameter, in.	2.38			
Specimen height, in.	5.08			
Height/diameter ratio	2.14			

Description: Gray, slightly weathered, medium strong (R3) Basalt

LL =	PL =	PI =	Assumed GS= 2.90	Type: HQ Core
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<p>Project No.: 2255001-613</p> <p>Date Sampled:</p> <p>Remarks:</p> <p>Figure: 4C</p>	<p>Client: Foundation Engineering, Inc. (Project No.: 2251001)</p> <p>Project: Soap Creek Bridge</p> <p>Source of Sample: 9787 Depth: 25.7'-26.2'</p> <p>Sample Number: CS-2-2</p> <p style="text-align: center;">UNCONFINED COMPRESSION TEST FEI Testing & Inspection, Inc. Tangent, OR</p>
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