APPENDIX 3

GEOTECHNICAL INVESTIGATION REPORT TAYLOR LANE BRIDGE REPLACEMENT

Report of
Geotechnical Investigation Report
Taylor Lane Bridge Replacement
Taylor Lane
Umatilla County, Oregon

CGT Project Number B2301527

Prepared for

Chad McKinney, P.E.
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19803 North Creek Parkway
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August 10, 2023

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Dear Mr. McKinney:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation for the proposed Taylor Lane Bridge Replacement project. The site is located at Taylor Lane in Umatilla County, Oregon. We performed our work in general accordance with CGT Proposal BGC.565, dated November 28, 2022. Written authorization for our services was received on June 29, 2023 via Tetra Tech Purchase Order No. 1199744.

We appreciate the opportunity to work with you on this project. Please contact us at (541) 330-9155 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

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1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation for the proposed Taylor Lane Bridge Replacement project. The site is located at Taylor Lane in Umatilla County, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Information

CGT developed an understanding of the proposed project based on our correspondence with our client and review of the provided Request for Proposal (RFP) dated November 16, 2022, prepared by Tetra Tech. Based on our review, we understand the project will include:

- Demolition of the existing, approximately 40-feet long Taylor Lane Bridge, which currently spans Birch Creek.
- Construction of a new, longer bridge along Taylor Lane to span Birch Creek. We understand the new bridge will be between 100 and 200 feet long, and maintain the existing alignment of Taylor Lane.
- We anticipate the new bridge will be single-span, and will not include the placement of structural elements within the creek. The new bridge design will incorporate rigid retaining wall abutments to retain the approaches, and shallow spread foundations. Design of retaining walls and shallow foundations will rest with others.
- We anticipate the bridge will be surfaced with Portland Cement Concrete (PCC) or asphalt concrete (AC) pavements. Design of new pavement sections will rest with others.
- We anticipate stormwater collected from new impervious areas of the site will be collected and routed to the nearest storm drain or other suitable discharge point. Infiltration testing was not requested as part of this assignment. Design of infiltration facilities will rest with others.
- Although no grading plans have been provided, we anticipate permanent grade changes will include cuts and fills up to 15 in depth relative to existing grades.

1.2 Scope of Services

Our scope of work included the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities within a 20-foot radius of our explorations at the site.
- Explore subsurface conditions at the site by advancing four drilled borings to depths of up to about 31 feet below ground surface (bgs). Details of the subsurface investigation are presented in Appendix A.
- Classify the soils encountered in the explorations in general accordance with ASTM D2488 (Visual-Manual Procedure). Rock encountered in the explorations was classified in general accordance with procedures outlined in ASTM D5878.
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.
- Provide recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Provide a qualitative evaluation of seismic hazards at the site, including earthquake-induced liquefaction, landsliding, and surface rupture due to faulting or lateral spread.
- Provide geotechnical recommendations for site preparation and earthwork.

- Provide geotechnical engineering recommendations for use in design and construction of bridge abutment shallow foundations, rigid retaining walls, and pavements.
- Provide this written report summarizing the results of our geotechnical investigation and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Site Geology

Based on available geologic mapping[1] of the area, the Taylor Lane Bridge site is underlain by Holocene age alluvium (Qal) deposited by Birch Creek. The alluvium consists of unconsolidated silt, sand and gravel deposited from the channels and flood plains of the still active creek. Based on our review of publically available well logs in the vicinity of the project site, we anticipate that the alluvium deposits are underlain by Grande Ronde basalt flows at depths ranging from approximately 10 to 30 feet bgs. The Grande Ronde basalt is part of the Columbia River Basalt Group and consists of a monotonous flow-on-flow sequence of blue to black basalt flows which can weather to produce an orange-brown, blocky basalt structure. We anticipate that the Grande Ronde Basalt extends to depths of up to 300 feet bgs or deeper within the vicinity of the project site.

2.2 Site Surface Conditions

The existing Taylor Lane Bridge spanned Birch Creek, which was a north flowing tributary of the Umatilla River located about 1,500 feet due north of the project site. The Taylor Lane Bridge was approximately 40 feet long, gravel surfaced and was bordered by low-laying agricultural areas to the west. On each end of the bridge, the existing bridge wingwalls consisted of stacked concrete blocks. A gravel-surfaced road (Taylor Lane) provided access to the project site. Site layout and surface conditions at the time of our field investigation are shown on the attached Site Plan (Figure 2A) and Site Photographs (Figure 3).

2.3 Subsurface Conditions

2.3.1 <u>Subsurface Investigation & Laboratory Testing</u>

Our subsurface investigation consisted of four borings (B-1 through B-4) completed on July 10 and 11, 2023. The approximate exploration locations are shown on the Site Plan, attached as Figure 2. In summary, the borings were advanced to depths ranging from about 26 to 31 feet bgs. Details regarding the subsurface investigation, logs of the explorations, and results of laboratory testing are presented in Appendix A. Subsurface conditions encountered during our investigation are summarized below.

2.3.2 <u>Subsurface Materials</u>

Logs of the explorations are presented in Appendix A. The following describes each of the subsurface materials encountered at the site.

<u>Undocumented Poorly Graded Gravel with Silt Fill (GP FILL)</u>
Poorly graded gravel with silt fill (base rock) was encountered at the surface of each of the borings (B-1 through B-4). Undocumented fill refers to materials placed without (available) records of subgrade conditions or evaluation of compaction. The poorly graded gravel with silt fill was typically gray, dry, subangular to

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[1] Madin, I.P, and Gietgey, R.P., 2007, Preliminary geologic map of the Umatilla Basin, Morrow and Umatilla Counties, Oregon: Oregon Department of Geology and Mineral Industries, Open-File Report O-07-15, scale 1:24,000.

angular, up to about ¾-inch in diameter, and contained nonplastic fines. This base rock material extended to depths of about 3 to 4 inches bgs in each of the borings.

Silty Sand (SM)
Native silty sand was encountered underlying the base rock in boring B-1and B-4, extending up to 7½ feet bgs. This soil was typically loose to medium dense, brown to gray, dry to wet, fine- to medium-grained, subrounded to subangular, contained nonplastic fines and a varying amount of subrounded to round gravel up to ¾-inch in diameter.

Silty, Clayey Sand (SC-SM)
Underlying the base rock in boring B-2 we encountered native silty, clayey sand which extended to a depth of 7½ bgs. The silty, clayey sand was loose to medium dense, brown, dry, subangular to subrounded, fine- to medium-grained, and contained nonplastic fines and trace subangular to subrounded gravel up to ½-inch in diameter.

Sandy Silt and Silt with Sand (ML)
Native sandy silt and silt with sand were encountered within boring B-1 underlying the native silty sand (SM) extending between 5 and 7½ feet bgs. Within boring B-3, the sandy silt was encountered underlying the surface base rock, extending to 5 feet bgs. These soils were typically medium stiff to very stiff, brown to gray, nonplastic, and contained a varying amount of subangular to subrounded, fine- to medium-grained sand and gravel up to ¾-inch in diameter.

Poorly Graded Gravel (GP) and Poorly Graded Gravel with Silt and Sand (GP-GM)
Native poorly graded gravel was encountered within each of the borings (B-1 through B-4), underlying the native sandy silt (ML), the native silty, clayey sand (SC-SM), and the native silty sand (SM). This material extended between depths of about 5 to 15 feet bgs. The poorly graded gravel was typically medium dense to dense, multicolored (brown, gray and red), subangular to angular, up to 1 inch diameter, and contained a varying amount of fine- to medium-grained sand and nonplastic fines.

The soils encountered during our subsurface investigation were consistent with the lower Clackamas River terrace deposits described in Section 2.1. The native silty sand (SM), sandy silt and silt with sand (ML), and silty, clayey sand (SC-SM) have a high percentage of fines, have similar index properties, and are referred to as “Stream alluvium” throughout the remainder of this report.

2.3.3 Groundwater

The borings were advanced using the mud rotary (wet) drilling method, which precluded direct observation of groundwater during advancement of the borings. To determine approximate regional groundwater levels in the area, we researched well logs available on the Oregon Water Resources Department (OWRD)[2] website for wells located within Sections 13 and 24, Township 02 North, Range 31 East, Willamette Meridian. Our review indicated that groundwater levels in the area generally ranged from about 50 to 75 feet bgs. We anticipate more shallow water zones exist at or near the current water levels of Birch Creek. It should be noted groundwater levels can vary depending on local topography and proximity to nearby bodies of water, such as Birch Creek. In addition, the groundwater levels reported on the OWRD logs often reflect the

[2] Oregon Water Resources Department, 2023. Well Log Records, *accessed August 2023*, from OWRD web site: http://apps.wrd.state.or.us/apps/gw/well_log/.

purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the project site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors.

3.0 SEISMIC CONSIDERATIONS

3.1 Seismic Design

Based on the results of the explorations and review of geologic mapping, we have assigned the site as Site Class C for the subsurface conditions encountered. Earthquake ground motion parameters for the site were obtained in accordance with the United States Geological Survey (USGS) Seismic Design Values for Buildings - Ground Motion Parameter Calculator[3]. The site Latitude 45.651874° North and Longitude 118.878801° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

Table 1 Seismic Ground Motion Values (2009 AASHTO Guide)

Parameter		Value
Mapped Acceleration Parameters	Peak Ground Acceleration, PGA	0.138g
	Spectral Acceleration, 0.2 second (S_s)	0.323g
	Spectral Acceleration, 1.0 second (S_1)	0.123g
Coefficients (Site Class C)	Site Coefficient, 0 sec. (F_{PGA})	1.200
	Site Coefficient, 0.2 sec. (F_A)	1.200
	Site Coefficient, 1.0 sec. (F_V)	1.677
Design Spectral Response Accelerations	Design Spectral Acceleration, 0 seconds (A_S)	0.166g
	Design Spectral Acceleration, 0.2 seconds (S_{DS})	0.387g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.206g

3.2 Seismic Hazards

3.2.1 Liquefaction

In general, liquefaction occurs when deposits of loose/soft, saturated, cohesionless soils, generally sands and silts, are subjected to strong earthquake shaking. If these deposits cannot drain quickly enough, pore water pressures can increase, approaching the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. When the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil approaches zero, and the soil can liquefy. The liquefied soils can undergo rapid consolidation or, if unconfined, can flow as a liquid.

For fine-grained soils, susceptibility to liquefaction is evaluated based on penetration resistance and plasticity, among other characteristics. Criteria for identifying non-liquefiable, fine-grained soils are constantly

[3] United States Geological Survey, 2023. Seismic Design Parameters determined using: "U.S. Seismic Design Maps Web Application," accessed *August 2023*, from the USGS website *http://earthquake.usgs.gov*. 2009 AASHTO Guide Specifications selected as reference document.

evolving. Current practice to identify non-liquefiable, fine-grained soils is based on moisture content and plasticity characteristics of the soils[4,5]. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on penetration resistance, as measured using SPTs, CPTs, or Becker Hammer Penetration tests (BPTs).

Based on their lack of saturated conditions, the soils encountered within our explorations are considered non-liquefiable within the depths explored. The basalt bedrock is not susceptible to liquefaction.

3.2.2 Slope Instability

We did not observe any obvious signs of past or on-going slope instability along the sides of Birch Creek. Review of the Statewide Landslide Information Database for Oregon (SLIDO), available at the DOGAMI website[6], shows no historic or prehistoric landslides at or in the immediate vicinity of the site. Given the lack of evidence of previous landslides in the vicinity, the risk of seismically-induced slope instability occurring at the site is considered low.

3.2.3 Surface Rupture

3.2.3.1 Faulting

Although the site is situated in a region of the country with known active faults and historic seismic activity, no known faults exist on or immediately adjacent to the site. Therefore, the risk of surface rupture at the site due to faulting is considered very low.

3.2.3.2 Lateral Spread

Surface rupture due to lateral spread can occur on sites underlain by liquefiable soils that are located on or immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spread, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Based on the relatively level topography at the site and the non-liquefiable nature of the soils at the site, the risk of damage associated with lateral spread is negligible.

4.0 CONCLUSIONS

Based on the results of our field explorations and analyses, the site may be developed as described in Section 1.1 of this report, provided the recommendations presented in this report are incorporated into the design and development. The primary geotechnical consideration for this project includes the presence of sandy and silty overburden soils that may be susceptible to sloughing during the excavation of site cuts necessary to achieve foundation bearing elevations, as described later in this report.

As indicated in Section 2.3 above, we encountered sandy and silty "stream alluvium" soils (SM, ML, SC-SM) within each of the borings to depths of about 10 feet bgs. These soils were generally loose to medium dense in terms of relative density and stiff in terms of consistency, and were underlain by relatively dense, poorly graded gravel (GP, GP-GM) and basalt bedrock (RX). Recognizing that current plans call for the embedment

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[4] Seed, R.B. et al., 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. Earthquake Engineering Research Center Report No. EERC 2003-06.

[5] Bray, Jonathan D., Sancio, Rodolfo B., et al., 2006. Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, Volume 132, Issue 9, September 2006.

[6] Oregon Department of Geology and Mineral Industries, 2019. Statewide Landslide Information Database for Oregon (SLIDO), *accessed July 2023*, from DOGAMI web site: https://gis.dogami.oregon.gov/maps/slido/.

of shallow foundations associated within the new bridge to depths of about 16 feet bgs, we anticipate that excavation of these sandy and silty overburden soils will be required to achieve finished site grades. Geotechnical recommendations for temporary excavations are presented in Section 5.2 of this report.

5.0 RECOMMENDATIONS: SITE WORK

The recommendations presented in this report are based on the information provided to us, results of our field investigation and analyses, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. CGT should be consulted for further recommendations if the design of the proposed development changes and/or variations or undesirable geotechnical conditions are encountered during site development.

5.1 Site Preparation

5.1.1 Demolition

Demolition of existing bridge, bridge abutments, and appurtenant structures should include complete removal of all structural elements, including foundations and concrete block wingwalls. Abandoned buried utilities should similarly be removed or grouted full. Concrete or asphalt concrete debris resulting from should be hauled off site for disposal.

5.1.2 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new bridge, wingwalls, pavements, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 5.4 this report. Buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 5.4 of this report.

5.1.3 Subgrade Preparation - Pavement Areas & Areas to Receive Structural Fill

5.1.3.1 Dry Weather Construction

After site preparation as recommended above, but prior to placement of structural fill and/or aggregate base, the geotechnical engineer or their representative should observe the exposed subgrade soils in order to identify areas of excessive yielding through either proof rolling or probing. Proof rolling of subgrade soils is typically conducted during dry weather using a fully-loaded, 10- to 12-cubic-yard, tandem-axle, tire-mounted, dump truck or equivalent weighted water truck. Areas of limited access or that appear too soft or wet to support proof rolling equipment should be evaluated by probing. During wet weather, subgrade preparation should be performed in general accordance with the recommendations presented in Section 5.3 of this report. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2 of this report.

5.1.3.2 Wet Weather Construction

Preparation of subgrade soils during wet weather should be in conformance with Section 5.3 of this report. As indicated therein, increased base rock sections and a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade.

5.1.4 Freezing Weather Considerations

For construction that occurs during extended periods of sub-freezing temperatures, the following special provisions are recommended:

- Structural fill should not be placed over frozen ground.
- Frozen soil should not be placed as structural fill.
- Fine-grained soils should not be placed as structural fill during sub-freezing temperatures.

Identification of frozen soils at the site should be in accordance with ASTM D4083-01 “Standard Practice for Description of Frozen Soils” or other approved method. The geotechnical engineer can aid the contractor with supplemental recommendations for earthwork that will take place during extended periods of sub-freezing weather, as required.

5.1.5 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations.

5.2 Temporary Excavations

5.2.1 Overview

Within the native stream alluvium soils (SM, ML, SC-SM) and the underlying gravelly soils (GP, GP-GM), conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated site cuts as described earlier in this report. However, where temporary excavations extend into basalt bedrock (RX), we anticipate hydraulic hammering will likely be required to facilitate its removal. All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A “competent person”, as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT’s current role on the project does not include review or oversight of excavation safety.

5.2.2 OSHA Soil Type

For use in the planning and construction of temporary excavations up to 16 feet in depth, an OSHA soil type “C” may be used for the native, stream alluvium soils (SM, ML, SC-SM). A soil type “C” may similarly be used for fractured or jointed basalt encountered within the upper elevations of the borings that does not appear stable, otherwise a “Stable Rock” type may be used.

5.2.3 Dewatering

Although groundwater was not encountered within our borings advanced at the site on July 10 and 11, 2023, depending on the time of year construction occurs and the proximity of the planned excavations to Birch Creek, we anticipate that groundwater *may* be encountered within planned excavation depths. Recognizing that excavations associated with the bridge abutments will likely extend below the surface of Birch Creek, and subsurface conditions at these depths consist of basalt bedrock, perched groundwater may be encountered. If perched groundwater is encountered, pumping from sumps may be effective in removing groundwater within localized excavations at the site. Pumping from multiple well points will likely be required for larger excavations and those extending below the perched groundwater level. Alternatively, cofferdams should be installed at the locations of the bridge abutments to ensure construction will occur “in the dry”. Design of cofferdams, if incorporated, will rest with others.

5.2.4 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet bgs in the native, Stream alluvium (SM, ML, SC-SM), the native poorly graded gravel (GP, GP-GM), and the basalt bedrock (RX) encountered at the site. If caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions. A discussion of dewatering of temporary excavations is presented in Section 5.2.3. If groundwater is present at the base of utility excavations, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 5.4.3 of this report.

5.2.5 Excavations Near Foundations

Excavations near footings should not extend within a 1 horizontal to 1 vertical (1H:1V) plane projected out and down from the outside, bottom edge of the footings. In the event excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

5.2.6 Temporary Shoring Systems

5.2.6.1 Overview

Selection of the type and design of shoring systems used at this project site will rest with others. As design concepts are being established, the geotechnical engineer should be contacted to review the proposed shoring system(s) and provide geotechnical recommendations in addition to those presented in the following paragraph, if warranted.

5.2.6.2 Soil Parameters

Soil parameters typically required as part of design of temporary or permanent shoring systems are presented in the following table. The parameters presented therein were based on the results of the laboratory testing performed on selected samples, published correlations with SPT N-values, and experience with similar soils.

Table 2 Soil Parameters Recommended for Preliminary Shoring Design

Parameter[1]	Subsurface Material[2]			
	Silty Sand (SM)	Sandy Silt & Silt with Sand (ML)	Poorly Graded Gravel (GP, GP-GM)	Basalt Bedrock (RX)
Effective Unit Weight, γ'[3]	120 pcf	110 pcf	135 pcf	150 pcf
Internal Angle of Friction, ϕ'	34°	30°	38°	50°
Effective Cohesion, c’	0 psf	0 psf	0 psf	3,000 psf
Ultimate Coefficient of Active Pressure, K_a	0.28	0.33	0.23	0.13
Ultimate Coefficient of At Rest Pressure, K_o	0.44	0.57	0.38	0.23
Ultimate Coefficient of Passive Pressure, K_p	3.5	3.0	4.4	7.7

[1] If additional soil parameters are required for design, the geotechnical engineer should be consulted.
[2] Refer to the attached boring logs for layer thicknesses across the site.
[3] The groundwater level (phreatic surface) is anticipated to be at depths in excess of 50 feet bgs at the site.
[4] Measured uni-axial compressive strengths are shown on the attached boring logs.

5.3 Wet Weather Considerations

Notwithstanding the generally arid conditions of Umatilla County, soil conditions should be evaluated in the field by the geotechnical engineer's representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

5.3.1 Overview

Due to their fines content, the near-surface, native stream alluvium soils (SM, ML, SC-SM) are moisture sensitive and susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. Site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical engineer or their representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2 of this report.

5.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should meet the requirements presented in the current Oregon Department of Transportation Standard Specification for Construction (ODOT SSC), Section 02320.

5.3.3 Granular Working Surfaces (Haul Roads & Staging Areas)

Haul roads subjected to repeated heavy, tire-mounted, construction traffic (e.g. dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas, 12 inches of imported granular material is typically sufficient. Additional granular material or geo-grid reinforcement may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should be in conformance with Section 5.4.2 and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric (Section 5.3.2) prior to placement of the imported granular material. The imported granular material should be placed in a single lift (up to 24 inches deep) and compacted using a smooth-drum, non-vibratory roller until well-keyed.

5.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). Samples of the proposed fill materials should be submitted to the geotechnical engineer a minimum of 5 business days prior their use on site[7]. The geotechnical engineer or their representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests and/or proof roll tests with

[7] Laboratory testing for moisture density relationship (Proctor) is required. Tests for gradation may be required.

suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed. The following table presents specific recommendations for frequency of density testing (where practical) of various fill designations.

Table 3 Guidelines for Frequency of Density Testing of Structural Fill Materials

Fill Designation	Recommended Frequency of Density Tests[1]	
	Maximum Depth Interval	Area-Wide
General Structural Fill (Mass Grading)	Test every 2 vertical feet	At least one density test per 4,000 feet^2 of fill area
Retaining Wall Backfill	Test every 2 vertical feet	At least one density test per 100 linear feet of backfill
Utility Trench Backfill	Test every 2 vertical feet	At least one density test per 100 feet of trench line
Pavement Base Rock	Test at surface of section	At least one density test per every 100 feet of roadway

[1]Or as required by the local jurisdiction, where located in the public right of way.

5.4.1 On-Site Soils – General Use

5.4.1.1 Native Stream Alluvium (SM, ML, SC-SM)

Re-use of these soil as structural fill may be difficult because these soils are sensitive to small changes in moisture content and may be difficult, if not impossible, to adequately compact during wet weather. We anticipate the moisture content of these soils will be higher than the optimum moisture content for satisfactory compaction. Therefore, moisture conditioning (drying) should be expected in order to achieve adequate compaction. If used as structural fill, this soil should be free of organic matter, debris, and particles larger than 4 inches. When used as structural fill, these soils should be placed in lifts with a maximum pre-compaction thickness of about 8 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor).

5.4.1.2 Poorly Graded Gravel (GP-GM FILL, GP, GP-GM)

Re-use of the on-site, relatively clean, gravelly fill and gravelly soils as structural fill is feasible, provided these materials are kept clean of organics, debris, and particles larger than 4 inches in diameter. If reused as structural fill, these materials should be prepared in general accordance with Section 5.4.2.

5.4.1.3 Basalt Bedrock (RX)

Re-use of excavated basalt bedrock (RX) as structural fill is feasible, provided it can be processed (crushed, or blended with imported granular material) to achieve a fill that is fairly well graded between coarse and fine. The maximum particle size should be limited to about 4 inches. If used as structural fill, the processed material should be prepared in conformance with that recommended for imported granular structural fill in Section 5.4.2 of this report.

If the on-site materials cannot be properly moisture-conditioned and/or processed, we recommend using imported granular material for structural fill.

5.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 10 percent material passing the

U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½ inches. The percentage of fines can be increased to 15 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. As a guideline, grading of this material with particles up to about 4 inches in diameter may follow that presented in the following table.

Table 3 Guideline Gradation for Imported Coarse-Grained Granular Fill

Sieve Size	% Passing
4 inches	100
3 inches	88 – 100
¾-inch	70 – 90
U.S. Standard No. 4	40 – 60
U.S. Standard No. 40	20 – 40
U.S. Standard No. 200	Dry Weather: Less than 15
	Wet Weather: Less than 8

Imported granular fill material should be compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by periodic deflection (proof roll) testing in accordance with ODOT Test Method 158, where accepted by the geotechnical engineer. Proof roll tests should be performed at maximum intervals of every 1 vertical foot as the fill is being placed.

5.4.3 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of a minimum of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

5.4.4 Trench Backfill Material

On-Site Soils: If on-site soils are considered and accepted by the local jurisdiction for re-use as utility trench backfill at the site, those soils should be prepared in conformance with Section 5.4.1 of this report.

Imported Granular Material: Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch-thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 4 Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas[1,2]	Landscaping Areas
Pipe Base and Within Pipe Zone	90% ASTM D1557 or pipe manufacturer's recommendation	88% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	92% ASTM D1557	90% ASTM D1557
Within 3 Feet of Design Subgrade	95% ASTM D1557	90% ASTM D1557

1 Includes proposed bridge, pavement areas, structural fill areas, exterior hardscaping, etc.
2 Or as specified by the local jurisdiction where located in the public right of way.

5.4.5 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, ODOT SSC. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength. If CLSM is considered for use on this site, please contact the geotechnical engineer for site-specific and application-specific recommendations.

5.5 Permanent Slopes

5.5.1 Overview

Permanent cut or fill slopes constructed at the site, if any, should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of slopes.

5.5.2 Placement of Fill on Slopes

New fill should be placed and compacted against horizontal surfaces. Where slopes exceed 5H:1V, the slopes should be keyed and benched prior to structural fill placement in general accordance with the attached Fill Slope Detail, Figure 4. If subdrains are needed on benches, subject to the review of the CGT geotechnical representative, they should be placed as shown on the attached Fill Slope Detail. In order to achieve well-compacted slope faces, slopes should be overbuilt by a few feet and then trimmed back to proposed final grades. A representative from CGT should observe the benches, keyways, and associated subdrains, if needed, prior to placement of structural fill.

6.0 RECOMMENDATIONS: BRIDGE ABUTMENTS

6.1 Design Considerations

Based on our correspondence, we understand that bridge abutment foundation design will be based on the current (2020) AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications, Ninth Edition manual. That manual is hereafter referred to as the "AASHTO manual". The following recommendations are presented relative to that manual and modeling the abutment foundations as follows:

- Founded as recommended in Section 6.2.1 of this report,
- Established at or very near elevations of 954.1 feet and 951.8 feet,
- Embedded a minimum of 1 foot into the basalt bedrock (RX).

In the event dimensioning of the abutment foundations vary from that presented herein, the geotechnical engineer should be consulted to review the updated design plans.

6.1.1 <u>Service Limit State</u>

Foundation design at the service limit state shall include estimated settlements, horizontal movements, overall stability, and scour at the design flood event. The recommendations presented herein for use in design address maximum permissible net contact stress and estimated vertical movement (settlements).

6.1.1.1 Maximum Permissible Net Contact Stress
For abutment foundations founded as recommended in Section 6.2.1.1 below, we recommend the maximum permissible net contact stress, q_{pn}, be assigned as 8,000 psf. In accordance with Section 10.5.2.2 of the AASHTO manual, this net contact stress (bearing pressure) should consider all applicable factored loads in the Service I Load Combination specified in Table 3.4.1-1 of that manual. This contact stress should include the weight (dead load) associated with backfill placed above foundation. Recognizing the cohesionless nature of the bearing soils, we recommend factored transient loads be included in this calculation. A resistance factor (φ_b) of 1.0 is recommended for service limit design based on Section 10.5.5.1 of the referenced AASHTO manual.

6.1.1.2 Vertical Settlements
For abutment foundations founded as recommended in Section 6.2.1.1 below, total settlements of abutment foundations are estimated to be less than 1 inch. Differential settlements between abutments are anticipated to be less than ½-inch. Settlements are anticipated to occur very quickly during construction (as loads are applied).

6.1.2 <u>Strength Limit State</u>

6.1.2.1 Nominal Bearing Resistance
For purposes of design and planning, the nominal bearing resistance, q_n, for shallow foundations situated in relatively level ground conditions may be based on the following equation[8]:

$$q_n = [c * N_c * s_c * i_c] + [\gamma_t * D_f * N_q * s_q * d_q * i_q * C_{wq}] * [½ * \gamma_t * B * N_\gamma * s_\gamma * i_\gamma * C_{w\gamma}]$$

where:	q_n = Nominal base resistance of foundations and considering strength limit state design (psf).

[8] Equation 10.6.3.1.2a-1 of AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020.

c = cohesion (psf)
N_c = Cohesion term bearing capacity factor (dimensionless)
s_C, s_q, s_γ = Footing shape correction factors (dimensionless)
i_C, i_q, i_γ = Load inclination correction factors (dimensionless)
D_f = Footing embedment depth (feet)
N_q = Unit weight term bearing capacity factor (dimensionless)
d_q = Correction factor to account for the shearing resistance along failure surface (dimensionless)
γ_t = Total (moist) unit weight below the depth of the footing (pcf)
B = Footing width (feet)
N_γ = Unit weight (footing width) term bearing capacity factor (dimensionless)
C_{wq}, $C_{w\gamma}$ = Correction factors to account for the location of the groundwater table (dimensionless)

The following table presents recommendations for the geotechnical soil parameters, based on the results of the investigation, the assumption that the bearing soils consist of imported granular structural fill, and abutment foundation dimensioning described above.

Table 5 Recommended Soil Parameters for Nominal Bearing Resistance Calculations

Soil Parameter	Recommended Value	Comments
c	0 psf	Site soils are cohesionless
N_c	0	Site soils are cohesionless (not a required calculation)
s_c	0	Site soils are cohesionless (not a required calculation)
i_c , i_q , i_γ	1.0	Modest embedment is anticipated for site walls. For walls with modest embedment, load inclination factors may be omitted per design per Section 10.6.3.1.2a of the AASHTO manual.
γ_t	135 pcf	Anticipated condition: Granular fill placed on the basalt bedrock (or foundations set directly on basalt bedrock) per Section 6.2.1.1 of this report.
D_f	24 inches (minimum)	A minimum embedment depth of 24 inches relative to the lowest adjacent, permanent grade is recommended.
N_q	48.9	Based on an internal angle of friction (ϕ') of 38 degrees for the compacted granular fill below foundations
s_q	1 + (B / L * tan ϕ')	Per Table 10.6.3.1.2a-3 of the AASHTO manual. L = Length of retaining wall in feet. ϕ' = 38 degrees.
d_q	1.0	Per Section 10.6.3.1.2a of the AASHTO manual.
C_{wq}, $C_{w\gamma}$	1.0	Groundwater is anticipated at significant depth at this site.
B	To be determined by foundation designer	Foundation width to be determined by designer based on review of service limit and strength limit states.
N_γ	78	Anticipated condition: Granular fill placed on the basalt bedrock (or foundations set directly on basalt bedrock) per Section 6.2.1.1 of this report.
s_γ	1 – 0.4 * (B / L)	Per Table 10.6.3.1.2a-3 of the AASHTO manual. L = Length of retaining wall in feet.

A bearing resistance factor (φ_b) of 0.45 is recommended for strength limit design based on Table 10.5.5.2.2-1 of the referenced AASHTO manual.

6.1.2.2 Sliding Resistance

An ultimate (unfactored) coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction. A sliding resistance factor (φ_T) of 0.80 is recommended for strength limit design based on Table 10.5.5.2.2-1 of the referenced AASHTO manual.

6.1.2.3 Passive Resistance

A maximum ultimate (unfactored) passive (equivalent fluid) earth pressure of 300 pounds per cubic foot (pcf) is recommended for design of footings cast neat into the basalt bedrock (RX) or confined by granular structural fill that is properly placed and compacted during construction. Development of passive resistance assumes that some lateral movement of the foundation is allowed into the surrounding soil, thereby developing a passive soil wedge. In order to develop the above capacity, the following should be understood:

1) Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2) The adjacent grade must be level,
3) The static ground water level must remain below the base of the footings throughout the year.
4) Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

A passive earth resistance factor (φ_{ep}) of 0.50 is recommended for strength limit design based on Table 10.5.5.2.2-1 of the referenced AASHTO manual.

6.1.3 Lateral Earth Pressures

For abutment walls founded, backfilled, and drained[9] as recommended in Section 7.2.2 below, the following table presents unfactored soil parameters recommended for use in design.

Table 6 Design Parameters for Rigid Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)[1]	Dynamic Active Thrust Force (P_E)[1,2]	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i = 0)	29 pcf	(3 pcf)*H^2	0.22*q
Restrained from Rotation	Level (i = 0)	52 pcf	(1 pcf)*H^2	0.38*q

[1] Refer to the attached Figure 5 for a graphical representation of static and seismic loading conditions. Seismic resultant force acts at 0.6H above the base of the wall.

[2] Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual.

The above design recommendations are based on the assumptions that:

- The walls consist of concrete cantilevered retaining walls (β = 0 and δ = 24 degrees, see Figure 4).
- The walls are 12 feet or less in height.

[9] If site retaining walls will be constructed without retaining wall drains, the geotechnical engineer should be consulted.

- The backfill is drained and consists of imported granular structural fill (ϕ = 38 degrees).
- No line load or point load surcharges are imposed behind the walls.
- The grade behind the wall is level, or sloping down and away from the wall, for a distance of 10 feet or more from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

6.1.4 Surcharge Loads

Where present, surcharges from adjacent site features (i.e. buildings, slabs, pavements, etc.) should be evaluated in design of retaining walls at the site. Methods for calculating lateral pressures on rigid retaining walls from strip, line, and vertical point loads are presented on the attached Figure 6.

6.2 Construction Considerations

6.2.1 Abutment Foundations

6.2.1.1 Subgrade Preparation

Satisfactory subgrade support for abutment foundations can be obtained from the native basalt bedrock (RX), or imported granular structural fill that is properly placed and compacted on the bedrock during construction. The geotechnical engineer or their representative should be contacted to observe subgrade conditions prior to placement of foundation forms, reinforcement steel, or granular fill (if required). Although not anticipated, if soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 5.4.2 of this report. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

6.2.1.2 Minimum Footing Width & Embedment

We recommend continuous footings have a minimum width of 24 inches. All footings should be founded at least 24 inches below the lowest, permanent adjacent grade. This embedment should be measured from the lowest, permanent grade (adjacent to the footing) to bottom of footing elevation.

6.2.1.3 Horizontal Setback from Descending Slopes

Foundations constructed within or near descending slopes should be setback a minimum of 5 feet from the slope surface. This distance should be measured between the face of the slope and the bottom, outside edge of the respective foundation. Organic topsoil and loose surface soils (if present) should not be included when determining this distance. The geotechnical engineer or his representative should be contacted to observe foundation subgrade conditions and confirm this recommended minimum setback is achieved.

6.2.2 Abutment Wall Drainage & Backfill

6.2.2.1 Wall Drains

The recommendations presented in Section 7.2.2 of this report are appropriate for the upslope (embankment) side of bridge abutments, as applicable.

6.2.2.2 Wall Backfill Material(s)

The recommendations presented in Section 7.2.3 of this report are appropriate for the upslope (embankment) side of bridge abutments, as applicable.

7.0 RECOMMENDATIONS: SITE RETAINING WALLS

7.1 Design Considerations

The following recommendations are presented relative to that manual and modeling site retaining wall foundations as follows:

- Founded on a minimum of 6 inches of imported granular structural fill as recommended in Section 7.2.1.1 of this report,
- Site grades will be relatively level or gently ascending in front of the retaining walls, and

In the event the above assumptions are incorrect, the geotechnical engineer should be consulted to review the updated design plans.

7.1.1 Service Limit State

Foundation design at the service limit state shall include estimated settlements, horizontal movements, overall stability, and scour at the design flood event. The recommendations presented herein for use in design address maximum permissible net contact stress and estimated vertical movement (settlements). We anticipate the maximum permissible, post-construction settlement for site retaining wall foundations is 1-inch. If post-construction settlements of site retaining walls need to be reduced, the geotechnical engineer should be consulted.

7.1.1.1 Maximum Permissible Net Contact Stress

For retaining wall foundations founded as recommended .in Section 6.2.1.1 below, we recommend the maximum permissible net contact stress, q_{pn}, be assigned as 5,000 psf. In accordance with Section 10.5.2.2 of the AASHTO manual, this net contact stress (bearing pressure) should consider all applicable factored loads in the Service I Load Combination specified in Table 3.4.1-1 of that manual. This contact stress should include the weight (dead load) associated with backfill placed above the foundation. A resistance factor (φ_b) of 1.0 is recommended for service limit design based on Section 10.5.5.1 of the referenced AASHTO manual.

7.1.1.2 Vertical Settlements

For retaining wall foundations founded as recommended in Section 6.2.1.1 below, total settlements of abutment foundations are estimated to be less than 1-inch. Settlements are anticipated to occur very quickly during construction (as loads are applied).

7.1.2 Strength Limit State

7.1.2.1 Nominal Bearing Resistance

For purposes of design and planning, the nominal bearing resistance, q_n, for shallow foundations situated in relatively level ground conditions may be based on the following equation[10]:

[10] Equation 10.6.3.1.2a-1 of AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020.

$$q_n = [c * N_c * s_c * i_c] + [\gamma_t * D_f * N_q * s_q * d_q * i_q * C_{wq}] * [½ * \gamma_t * B * N_\gamma * s_\gamma * i_\gamma * C_{w\gamma}]$$

where:	q_n = Nominal base resistance of foundations and considering strength limit state design (psf).
	c = cohesion (psf)
	N_c = Cohesion term bearing capacity factor (dimensionless)
	s_c, s_q, s_γ = Footing shape correction factors (dimensionless)
	i_c, i_q, i_γ = Load inclination correction factors (dimensionless)
	D_f = Footing embedment depth (feet)
	N_q = Unit weight term bearing capacity factor (dimensionless)
	d_q = Correction factor to account for the shearing resistance along failure surface (dimensionless)
	γ_t = Total (moist) unit weight below the depth of the footing (pcf)
	B = Footing width (feet)
	N_γ = Unit weight (footing width) term bearing capacity factor (dimensionless)
	C_{wq}, $C_{w\gamma}$ = Correction factors to account for the location of the groundwater table (dimensionless)

The following table presents recommendations for the geotechnical soil parameters, based on the results of the investigation, the assumption that the bearing soils consist of imported granular structural fill, and retaining wall foundation dimensioning described above.

Table 7 Recommended Soil Parameters for Nominal Bearing Resistance Calculations

Soil Parameter	Recommended Value	Comments
c	0 psf	Site soils are cohesionless
N_c	0	Site soils are cohesionless (not a required calculation)
s_c	0	Site soils are cohesionless (not a required calculation)
i_c , i_q , i_γ	1.0	Modest embedment is anticipated for site walls. For walls with modest embedment, load inclination factors may be omitted per design per Section 10.6.3.1.2a of the AASHTO manual.
γ_t	135 pcf	Anticipated condition: Granular fill placed on acceptable native soils per Section 7.2.1.1 of this report.
D_f	24 inches (minimum)	A minimum embedment depth of 24 inches relative to the lowest adjacent, permanent grade is recommended.
N_q	48.9	Based on an internal angle of friction (ϕ') of 38 degrees for the minimum 6 inches of compacted granular fill below foundations
s_q	1 + (B / L * tan ϕ')	Per Table 10.6.3.1.2a-3 of the AASHTO manual. L = Length of retaining wall in feet. ϕ' = 38 degrees.
d_q	1.0	Per Section 10.6.3.1.2a of the AASHTO manual.
C_{wq}, $C_{w\gamma}$	1.0	Groundwater is anticipated at significant depth at this site.
B	To be determined by wall designer	Retaining wall width to be determined by designer based on review of service limit and strength limit states.
N_γ	78	Anticipated condition: Granular fill placed on the basalt bedrock per Section 6.2.1.1 of this report.
s_γ	1 – 0.4 * (B / L)	Per Table 10.6.3.1.2a-3 of the AASHTO manual. L = Length of retaining wall in feet.

A bearing resistance factor (φ_b) of 0.45 is recommended for strength limit design based on Table 10.5.5.2.2-1 of the referenced AASHTO manual.

7.1.2.2 Sliding Resistance

An ultimate (unfactored) coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on imported granular structural fill (crushed rock) that is properly placed and compacted during construction. A sliding resistance factor (φ_T) of 0.80 is recommended for strength limit design based on Table 10.5.5.2.2-1 of the referenced AASHTO manual.

7.1.2.3 Passive Resistance

A maximum ultimate (unfactored) passive (equivalent fluid) earth pressure of 300 pounds per cubic foot (pcf) is recommended for design of footings cast neat into the basalt bedrock (RX) or confined by granular structural fill that is properly placed and compacted during construction. Development of passive resistance assumes that some lateral movement of the foundation is allowed into the surrounding soil, thereby developing a passive soil wedge. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should <u>not</u> be considered when calculating passive resistance.

A passive resistance factor (φ_{ep}) of 0.50 is recommended for strength limit design based on Table 10.5.5.2.2-1 of the referenced AASHTO manual.

7.1.3 Lateral Earth Pressures

For concrete cantilevered retaining walls founded, backfilled, and drained[11] as recommended in Section 7.2 below, the following table presents unfactored soil parameters recommended for use in design.

Table 8 Design Parameters for Concrete Cantilevered Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)[1]	Dynamic Active Thrust Force (P_E)[1,2]	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i = 0)	29 pcf	(3 pcf)*H^2	0.22*q
Restrained from Rotation	Level (i = 0)	52 pcf	(1 pcf)*H^2	0.38*q

[1] Refer to the attached Figure 5 for a graphical representation of static and seismic loading conditions. Seismic resultant force acts at 0.6H above the base of the wall.

[2] Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual.

[11] If site retaining walls will be constructed without retaining wall drains, the geotechnical engineer should be consulted.

The above design recommendations are based on the assumptions that:

- The walls consist of concrete cantilevered retaining walls (β = 0 and δ = 24 degrees, see Figure 4).
- The walls are 12 feet or less in height.
- The backfill is drained and consists of imported granular structural fill (ϕ = 38 degrees).
- No line load or point load surcharges are imposed behind the walls.
- The grade in front of the walls is level or ascending for a distance of at least 5 feet from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

7.1.4 Surcharge Loads

Where present, surcharges from adjacent site features (i.e. buildings, slabs, pavements, etc.) should be evaluated in design of retaining walls at the site. Methods for calculating lateral pressures on rigid retaining walls from strip, line, and vertical point loads are presented on the attached Figure 6.

7.2 Construction Considerations

7.2.1 Retaining Wall Foundations

7.2.1.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations associated with site retaining walls can be obtained from a minimum of 6 inches of granular structural fill that is property placed and compacted on:

- The native, medium dense/medium stiff to better stream alluvium (SM, ML, SC-SM), or
- The native, medium dense to better poorly graded gravel (GP, GP-GM), or
- The native, medium strong to very strong (R3-R5) basalt bedrock.

The geotechnical engineer or their representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 5.4.2 of this report. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

7.2.1.2 Minimum Footing Width & Embedment

We recommend continuous footings have a minimum width of 24 inches. All footings should be founded at least 24 inches below the lowest, permanent adjacent grade. This embedment should be measured from the lowest, permanent grade (adjacent to the footing) to bottom of footing elevation.

7.2.1.3 Horizontal Setback from Descending Slopes

Foundations constructed within or near descending slopes should be setback a minimum of 5 feet from the slope surface. This distance should be measured between the face of the slope and the bottom, outside edge of the respective foundation. Organic topsoil and loose surface soils (if present) should not be included when determining this distance. The geotechnical engineer or his representative should be contacted to observe foundation subgrade conditions and confirm this recommended minimum setback is achieved.

7.2.2 Wall Drains

We recommend placing retaining wall drains at the base elevation of the heel of retaining wall footings. Retaining wall drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Retaining wall drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer or their representative should be contacted to observe the drains prior to backfilling. Roof or area drains should not be tied into retaining wall drains.

7.2.3 Wall Backfill Material

Retaining walls should be backfilled with imported granular structural fill in conformance with Section 5.4.2 contain less than 10 percent passing the U.S. Standard No. 200 Sieve. The backfill should be compacted to a minimum of 90 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). When placing fill behind walls, care must be taken to minimize undue lateral loads on the walls. Heavy compaction equipment should be kept at least "H" feet from the back of the walls, where "H" is the height of the wall. Light mechanical or hand tamping equipment should be used for compaction of backfill materials within "H" feet of the back of the walls.

8.0 RECOMMENDATIONS: PAVEMENTS

8.1 Flexible (Asphalt Concrete) Pavements

8.1.1 Subgrade Preparation

Satisfactory subgrade support for pavements constructed on-grade may consist of the native, medium dense/medium stiff to better stream alluvium (SM, ML, SC-SM), the native medium dense to better gravelly soils (GP, GP-GM), or new structural fill that is properly placed and compacted on these materials during construction. Subgrade preparation of pavements should be in conformance with Section 5.1.3 of this report. Pavement subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

8.1.2 Design Sections

Design of pavement sections was not requested as part of this current assignment. CGT would be pleased to prepare site-specific pavement section designs, upon request, for an additional fee.

8.2 Additional Considerations

8.2.1 Drainage

Subsurface drains should be connected to the nearest storm drain, on-site infiltration system (to be designed by others) or other suitable discharge point. Paved surfaces and grading near or adjacent to the bridge abutments should be sloped to drain away from the bridge. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains, retaining wall drains, or onto site slopes.

8.2.2 Expansive Potential

The near surface native soils consist of generally granular soils with low plasticity fines and low plasticity fine-grained soils. Based on experience with similar soils in the region, these soils are not considered susceptible to appreciable movements from changes in moisture content. Accordingly, no special considerations are required to mitigate expansive potential of the near surface soils at the site.

9.0 RECOMMENDED ADDITIONAL SERVICES

9.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

9.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and pavement performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend the geotechnical engineer or their representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer or their representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Demolition
- Subgrade Preparation for Shallow Foundations, Structural Fills, and Pavements
- Compaction of Structural Fill, Retaining Wall Backfill, and Utility Trench Backfill
- Compaction of Asphalt Concrete for Pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

10.0 LIMITATIONS

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are forwarded to assist in the planning and design process and are not intended to be, nor should they be construed as, a warranty of subsurface conditions.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

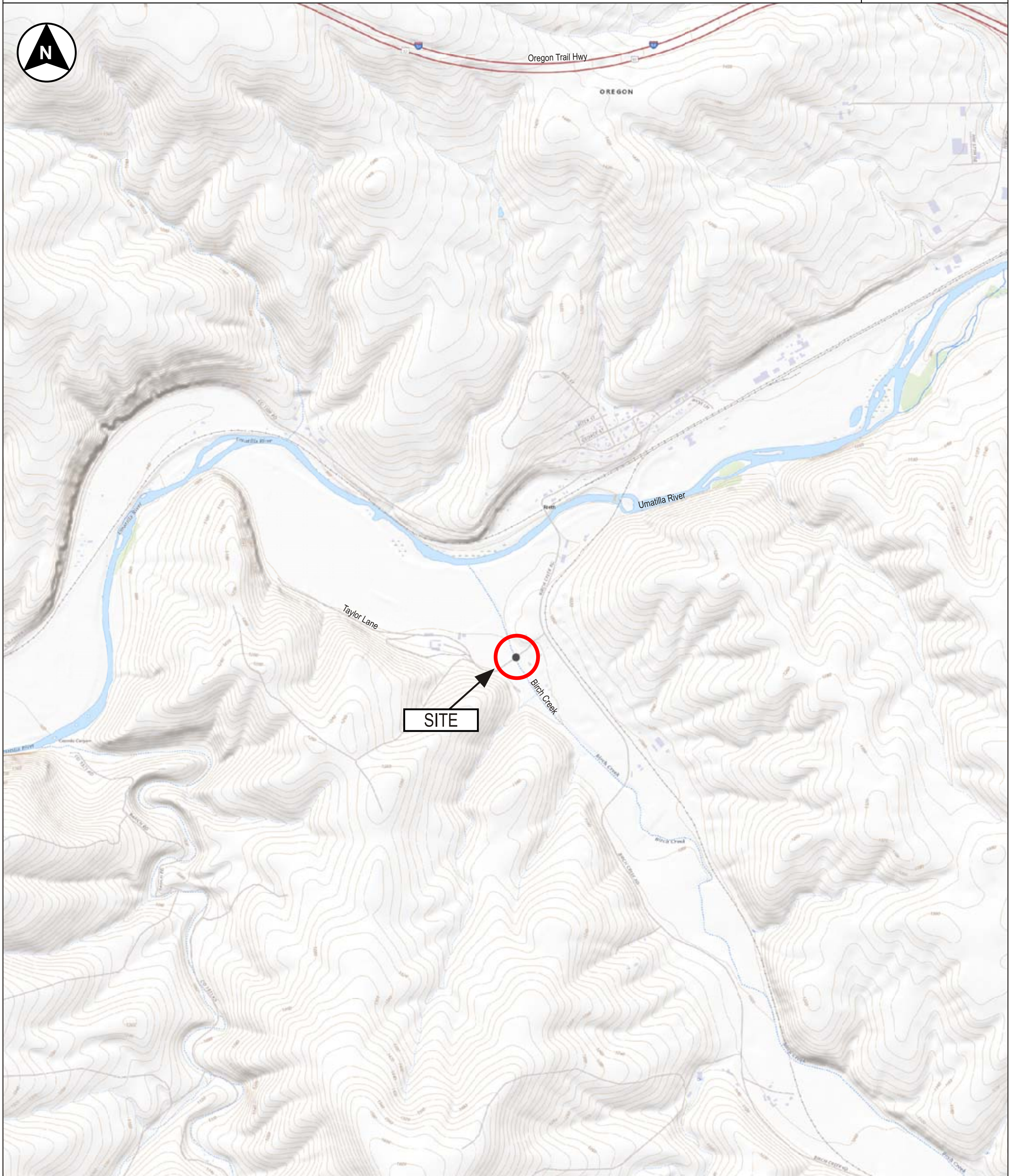
The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON
Project Number B2301527

FIGURE 1

Site Location



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Drafted by: GS

USGS Topographic base map created with The National Map, 2023, at https://apps.nationalmap.gov/viewer/

Township 02 North, Range 31 East, Section 13, Willamette Meridian

Latitude: 44.651874° North
Longitude: 118.878801° West

1 Inch = 2,000 feet

0 2000 4000

TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON
Project Number B2301527

FIGURE 2A
Site Plan

LEGEND

B-1 Boring advanced by CGT. Elevation of surface of bedrock indicated in (blue).

1 Orientation of site photographs shown on Figure 3

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Drafted by: GS

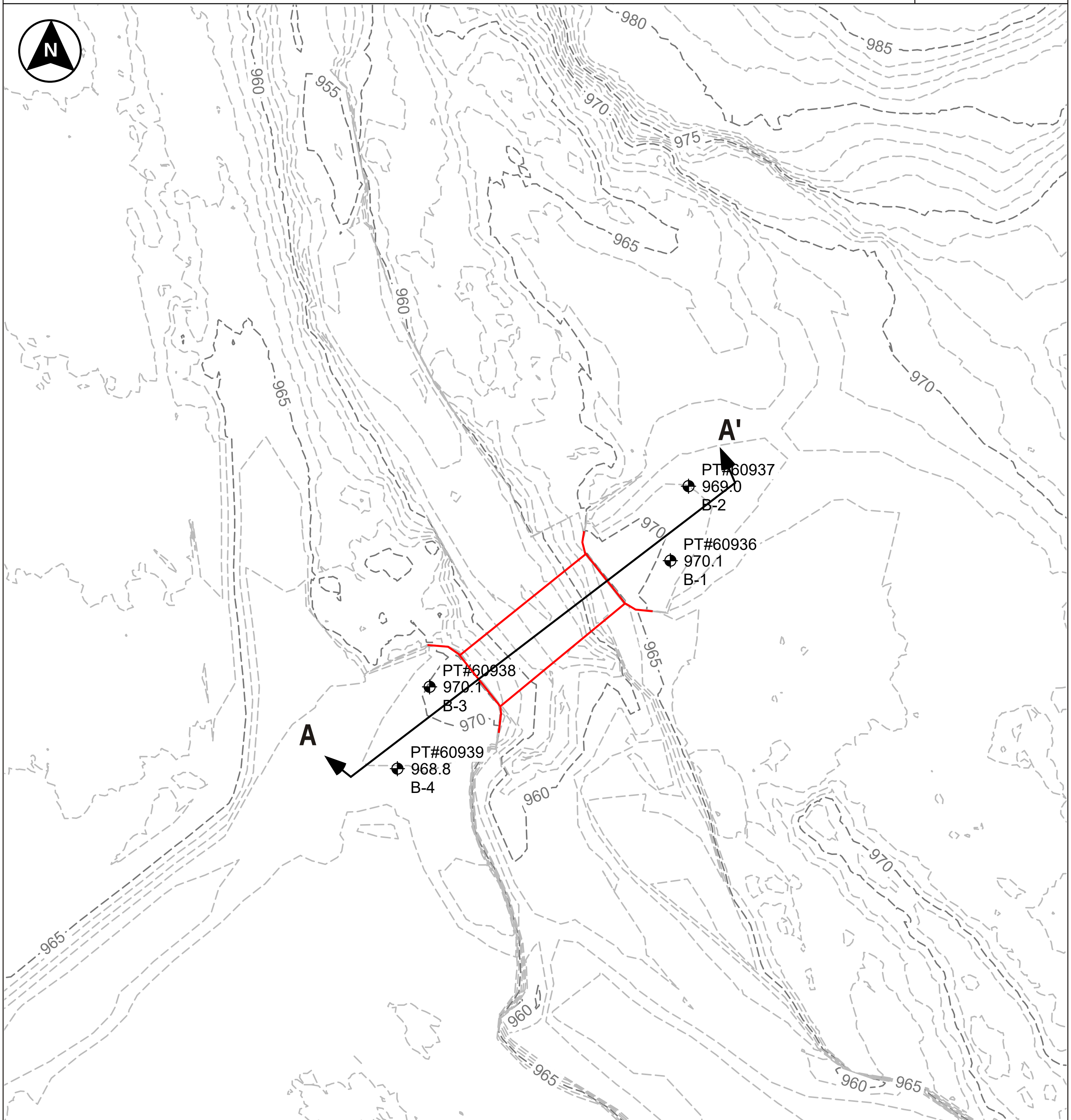
NOTES: Drawing based on observations made while on site and page 12 of Tetra Tech's "Geotechnical Investigation Scope of Work", dated November 16, 2022. All locations are approximate.

1 Inch = 30 Feet

0 30 60

TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON
Project Number B2301527

FIGURE 2B
Site Plan



LEGEND

Existing Taylor Lane Bridge

B-1 Drilled boring locations

A A' Location of cross section shown on Figure 7

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Drafted by: PBR

NOTES: Drawing based on observations made while on site and the existing ground contours survey provided by our client. All locations are approximate.

1 Inch = 60 Feet

0 60 120

TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON
Project Number B2301527

FIGURE 3

Site Photographs

Photograph 1

Photograph 2

Photograph 3

Photograph 4

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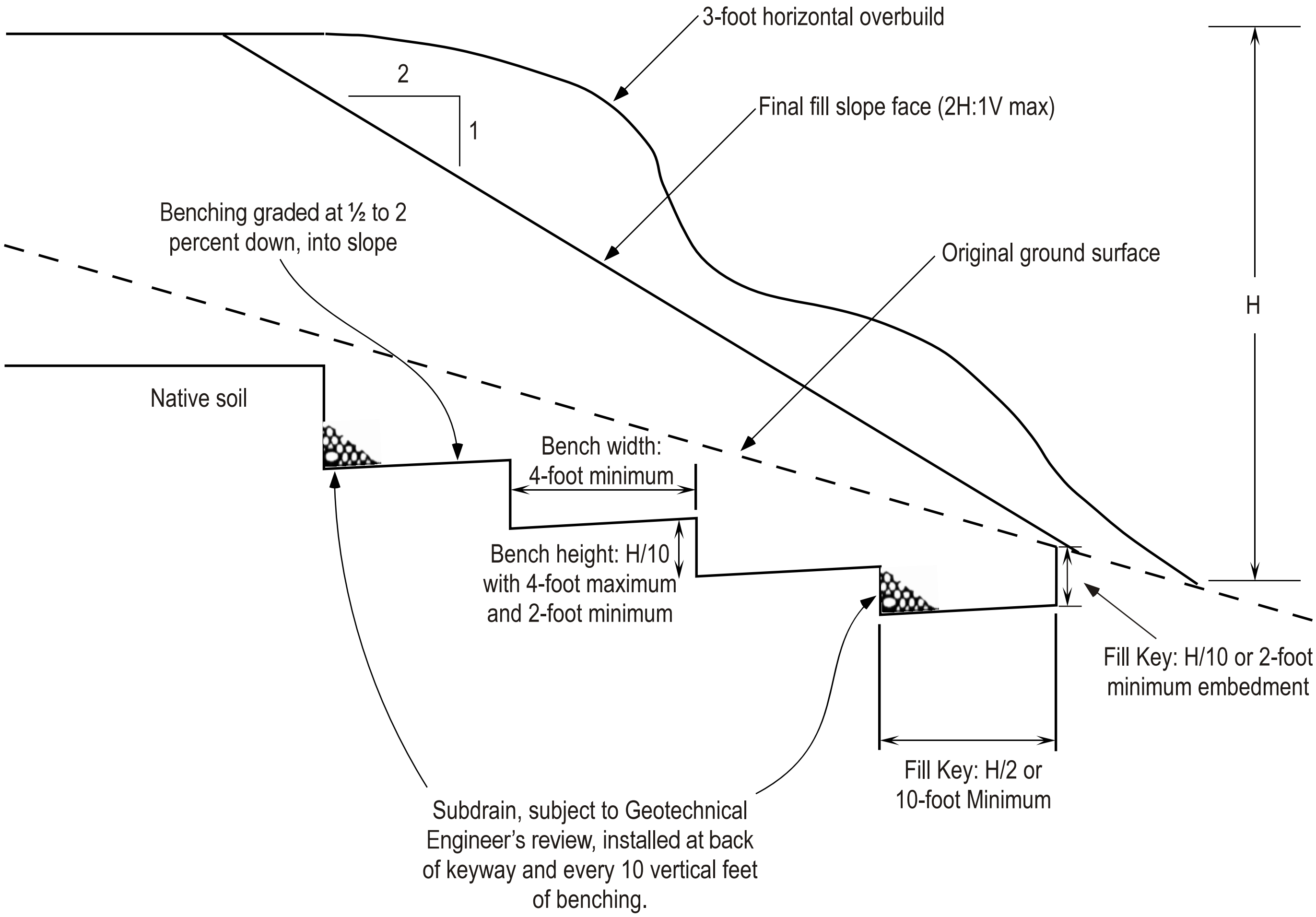
Drafted by: GS

See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.

TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON Project Number B2301527	FIGURE 4 Fill Slope Detail



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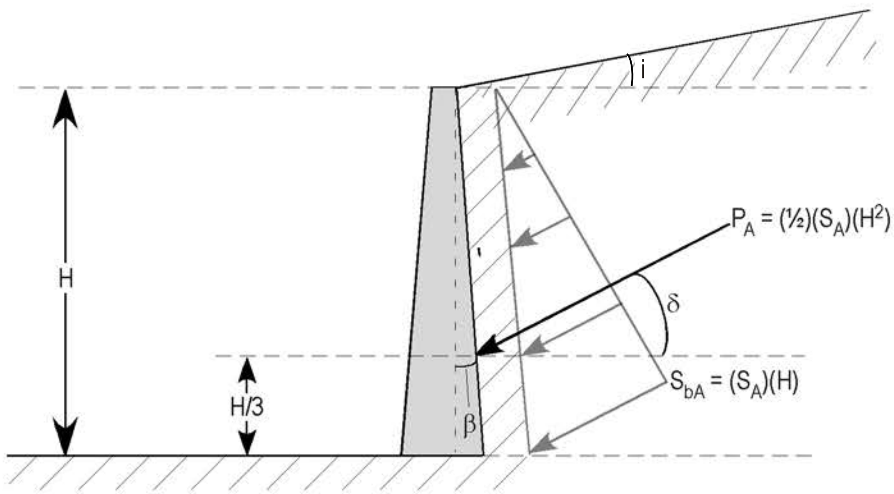
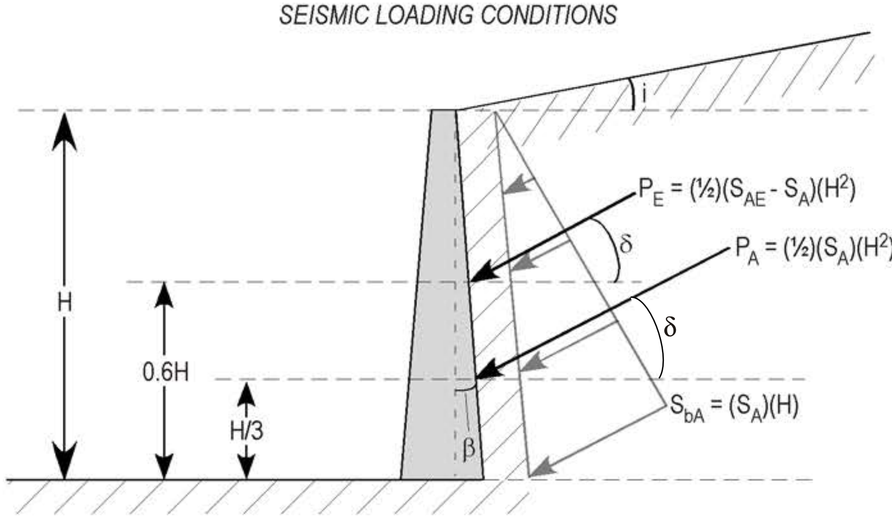
NOTE: Surfaces to receive fill with slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed as shown.

ACTIVE LATERAL PRESSURE DISTRIBUTION

STATIC LOADING CONDITIONS



SEISMIC LOADING CONDITIONS



LEGEND

S_A = Active lateral equivalent fluid pressure (lb/ft^3)*

S_{bA} = Active lateral earth pressure (static) at the bottom of wall (lb/ft^3)

S_{AE} = Active total (static + seismic) equivalent fluid pressure (lb/ft^3)*

i = Slope of backfill, relative to horizontal (degrees)**

β = Slope of back of wall, relative to vertical (degrees)**

P_A = Static active thrust force acting at H/3 from bottom of retaining wall (lb/ft)

P_E = Dynamic active thrust force acting at 0.6H from bottom of retaining wall (lb/ft)

δ = Angle from normal of back of wall (degrees). Based on friction developing between wall and backfill**

*Refer to report text for calculated values **Refer to report text for modeled/assumed values

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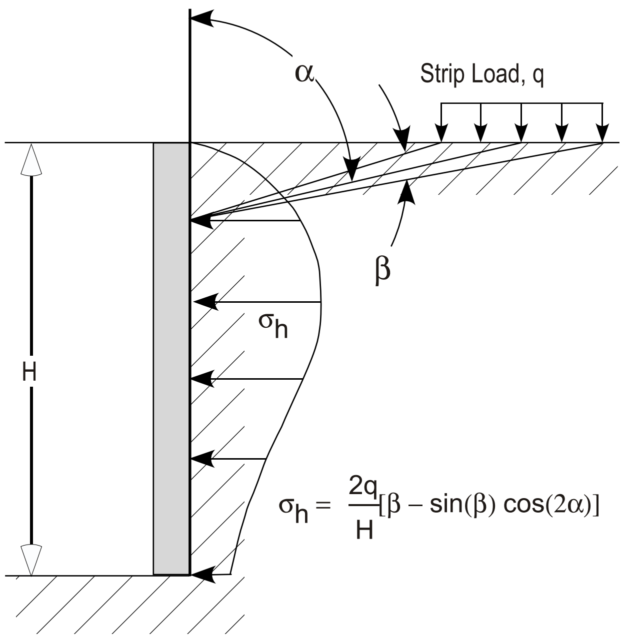
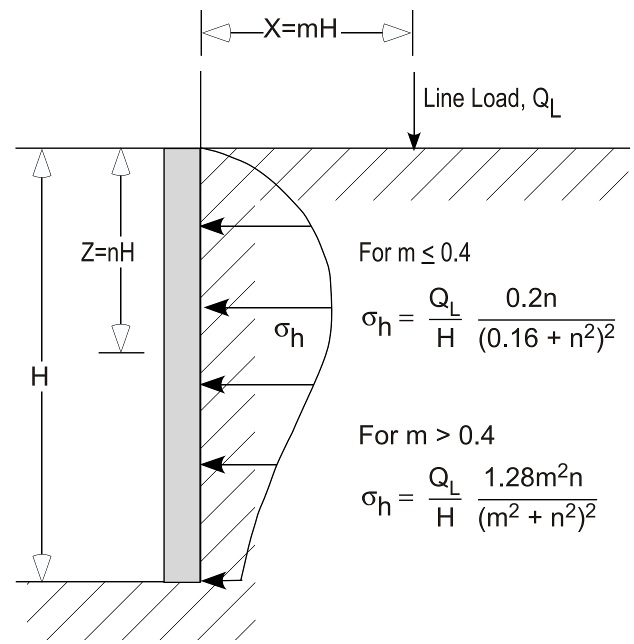
Notes

1. Uniform pressure distribution of seismic loading is based on empirical evaluations [Sherif et al, 1982 and Whitman, 1990].
2. Placement of seismic resultant force at 0.6H is based on wall behavior and model test results [Whitman, 1990].

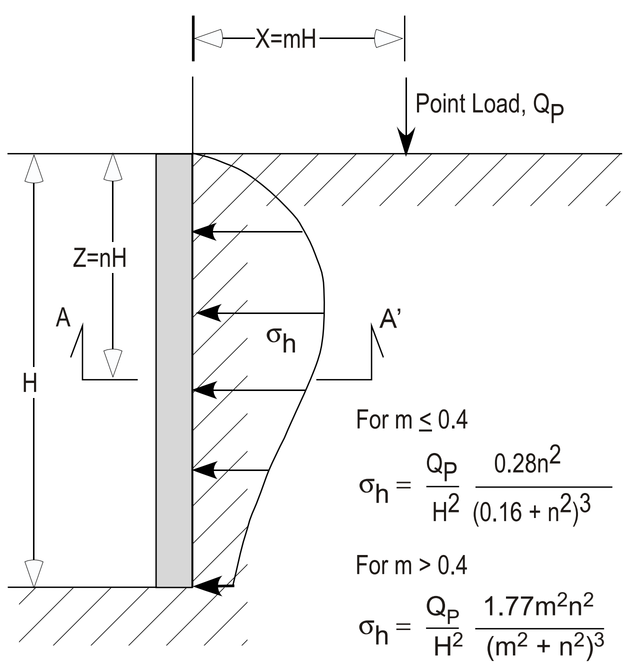
STRIP LOAD PARALLEL TO WALL[1]

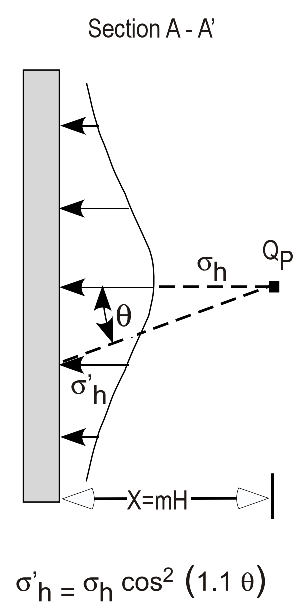


LINE LOAD PARALLEL TO WALL[2]



VERTICAL POINT LOAD[2]





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Notes: 1. Das, Principles of Geotechnical Engineering, 1990 Edition.
2. NAVFAC Design Manual 7.06.

Refer to the referenced design manuals for additional guidance. Contact CGT if there are any questions with modeling surcharge loads.

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FIGURE 7: SUBSURFACE DIAGRAM
A-A' Cross Section

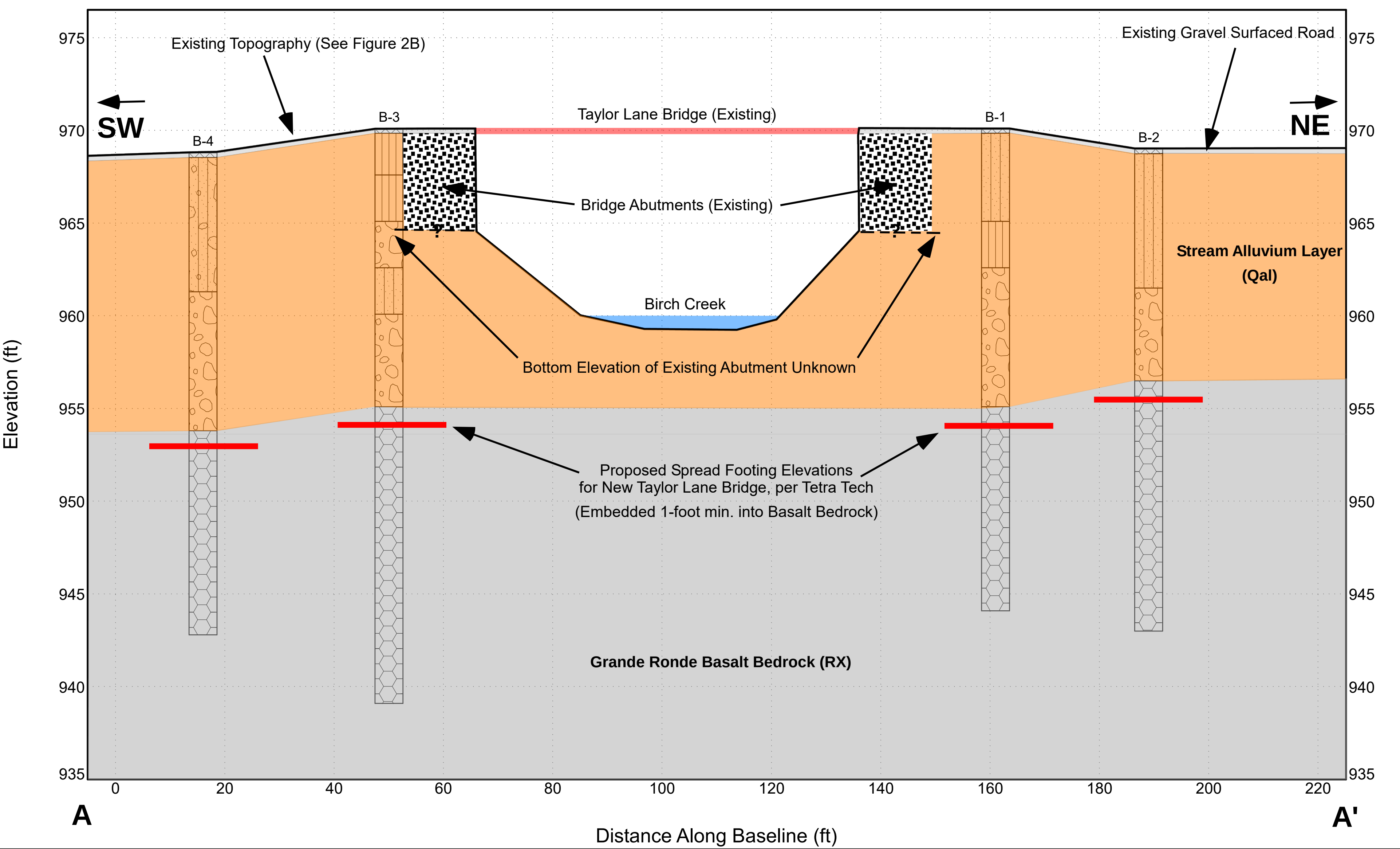
CLIENT Tetra Tech

PROJECT NUMBER B2301527

PROJECT NAME Taylor Lane Bridge Replacement

PROJECT LOCATION Taylor Lane - Umatilla County, OR

Fill (made ground)
USCS Silty Sand
USCS Silt
USCS Poorly-graded Gravel
Basalt
Silty Sand with Gravel



STRATIGRAPHY & GW - A SIZE W LEGEND B2301527 BORING LOGS.GPJ 8/7/23 DRAFTED BY: GS

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Tigard Office	**(503) 684-3460**

Appendix A: Subsurface Investigation and Laboratory Testing

Taylor Lane Bridge Replacement
Taylor Lane
Umatilla County, Oregon

CGT Project Number B2301527

August 10, 2023

Prepared For:

Chad McKinney, P.E.
Tetra Tech
19803 North Creek Parkway
Bothell, WA 98011

Prepared by
Carlson Geotechnical

Exploration Key .... Figure A1
Soil Classification .... Figure A2
Rock Classification .... Figure A3
Exploration Logs .... Figures A4 – A7

A.1.0 SUBSURFACE INVESTIGATION

Our field investigation consisted of four borings completed in July 2023. The exploration locations are shown on the Site Plan, attached to the geotechnical report as Figure 2. The exploration locations shown therein were determined based on measurements from existing site features (bridge abutments, etc.) and are approximate. Surface elevations indicated on the logs were estimated based on the topographic contours shown on the referenced Site Plan and are approximate. The attached figures detail the exploration methods (Figure A1), soil and rock classification criteria (Figures A2 and A3), and present detailed logs of the explorations (Figures A4 through A7), as discussed below.

A.1.1 Drilled Borings

CGT observed the advancement of four drilled borings (B-1 through B-4) at the site, using a CME 75 truck-mounted drill rig provided and operated by our subcontractor, Western States Soil Conservation of Hubbard, Oregon. The borings were advanced using the mud rotary drilling technique to depths ranging from approximately 12½ to 15 feet bgs, at which depths basalt bedrock was encountered. Upon encountering intact rock, the borings were further advanced using HQ3 rock coring assembly, consisting of a 61.1-millimeter (2.5-inch) inner diameter, triple-tube core barrel. The maximum core run length was 5 feet, and the borings were advanced to total depths of 26 to 31 feet bgs using the HQ3 rock coring methods. Upon completion, the borings were backfilled with granular bentonite. Drilling wastes (cuttings and drilling fluids) were left onsite.

A.1.2 In-Situ Testing: Standard Penetration Tests (SPTs)

SPTs were conducted within the borings using a split-spoon sampler in general accordance with ASTM D1586. The SPTs were conducted at 2½- to 5-foot intervals until intact bedrock was encountered in each of the borings. The SPT is described on the attached Exploration Key, Figure A1.

A.1.3 Material Classification & Sampling

Soil samples were obtained at selected intervals in the borings using the referenced split-spoon (SPT) sampler, detailed on Figure A1. A qualified member of CGT's geological staff collected the samples and logged the soils in general accordance with the Visual-Manual Procedure (ASTM D2488). Rock was classified in general accordance with procedures outlined in ASTM D5878. An explanation of these classification systems is attached as Figures A2 and A3. The SPT samples were stored in sealable plastic bags and transported to our soils laboratory for further examination and testing. Our geotechnical staff visually examined all samples in order to refine the initial field classifications.

For each rock core run, the Rock Quality Designation (RQD) and relative rock hardness were measured in the field. Explanations of these characteristics are provided on the Rock Classification criteria attached as Figure A3. Photographs of the rock cores are presented in Appendix B. Select intervals of the rock core samples were wrapped in plastic to preserve their moisture content and reduce atmospheric exposure, then were placed in treated cardboard core boxes and transported to our soils laboratory for further examination and testing. Our geological staff visually examined all samples in order to refine the initial field classifications.

A.1.4 Subsurface Conditions

Subsurface conditions are summarized in Section 2.3 of the geotechnical report. Detailed logs of the explorations are presented on the attached exploration logs, Figures A4 through A7.

A.2.0 LABORATORY TESTING

Laboratory testing was performed on samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing included the following:

- Six moisture content determinations (ASTM D2216).
- Five percentage passing the U.S. Standard No. 200 Sieve tests (ASTM D1140).
- One Atterberg limits (plasticity) test (ASTM D4318).
- Five compressive strength of rock core test (ASTM D4543/D7012 Method C).

Results of the laboratory tests are shown on the exploration logs.

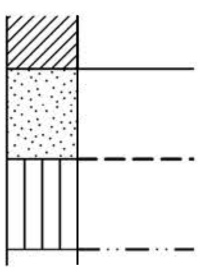
TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON Project Number B2301527	FIGURE A1 Exploration Key

PL — MC — LL	Atterberg limits (plasticity) test results (ASTM D4318): PL = Plastic Limit, LL = Liquid Limit, and MC= Moisture Content (ASTM D2216)
☐ FINES CONTENT (%)	Percentage passing the U.S. Standard No. 200 Sieve (ASTM D1140)

SAMPLING

GRAB	Grab sample
BULK	Bulk sample
SPT	**Standard Penetration Test** (SPT) consists of driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation with repeated blows of a 140-pound, hammer falling a vertical distance of 30 inches (ASTM D1586). The number of blows (N-value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. The drill rig was equipped with an cat-head or automatic hammer to conduct the SPTs. The observed N-values, hammer efficiency, and N_{60} are noted on the boring logs.
MC	**Modified California** sampling consists of 3-inch, outside-diameter, split-spoon sampler (ASTM G3550) driven similarly to the SPT sampling method described above. A sampler diameter correction factor of 0.44 is applied to calculate the equivalent SPT N_{60} value per Lacroix and Horn, 1973.
CORE	**Rock Coring** interval
SH	**Shelby Tube** is a 3-inch, inner-diameter, thin-walled, steel tube push sampler (ASTM D1587) used to collect relatively undisturbed samples of fine-grained soils.
WDCP	**Wildcat Dynamic Cone Penetrometer** (WDCP) test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding SPT N_{60} values.
DCP	**Dynamic Cone Penetrometer** (DCP) test consists of driving a 20-millimeter diameter, hardened steel cone on 16-millimeter diameter steel rods into the ground using a 10-kilogram drop hammer with a 460-millimeter free-fall height. The depth of penetration in millimeters is recorded for each drop of the hammer.
POCKET PEN. (tsf)	**Pocket Penetrometer** test is a hand-held instrument that provides an approximation of the unconfined compressive strength in tons per square foot (tsf) of cohesive, fine-grained soils.

CONTACTS



Observed (measured) contact between soil or rock units.

Inferred (approximate) contact between soil or rock units.

Transitional (gradational) contact between soil or rock units.

ADDITIONAL NOTATIONS

Italics	Notes drilling action or digging effort
{ Braces }	Interpretation of material origin/geologic formation (e.g. { Base Rock } or { Columbia River Basalt })

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All measurements are approximate.

TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON
Project Number B2301527

FIGURE A2
Soil Classification

Classification of Terms and Content

NAME: Group Name and Symbol
Relative Density or Consistency
Color
Moisture Content
Plasticity
Other Constituents
Other: Grain Shape, Approximate Gradation
Organics, Cement, Structure, Odor, etc.
Geologic Name or Formation

Grain Size

		U.S. Standard Sieve
Fines		<#200 (0.075 mm)
Sand	Fine	#200 - #40 (0.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75 mm)
Gravel	Fine	#4 - 0.75 inch
	Coarse	0.75 inch - 3 inches
Cobbles		3 to 12 inches
Boulders		> 12 inches

Coarse-Grained (Granular) Soils

Relative Density

SPT N_{60}-Value	Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
>50	Very Dense

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace silt"
5 - 15%	"With" as part of group name	"**POORLY GRADED SAND WITH SILT**"
15 - 49%	Modifier to group name	"**SILTY SAND**"

Fine-Grained (Cohesive) Soils

SPT N_{60}-Value	Torvane tsf Shear Strength	Pocket Pen tsf Unconfined	Consistency	Manual Penetration Test
<2	<0.13	<0.25	Very Soft	Thumb penetrates more than 1 inch
2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about ¼ inch
8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ inch
15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace fine-grained sand"
5 - 15%	"Some" as part of soil description	"some fine-grained sand"
15 - 30%	"With" as part of group name	"**SILT WITH SAND**"
30 - 49%	Modifier to group name	"**SANDY SILT**"

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

	Plasticity	Dry Strength	Dilatancy	Toughness
ML	Non to Low	Non to Low	Slow to Rapid	Low, can't roll
CL	Low to Medium	Medium to High	None to Slow	Medium
MH	Medium to High	Low to Medium	None to Slow	Low to Medium
CH	Medium to High	High to Very High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite fracture planes

Slickensided: Striated, polished, or glossy fracture planes

Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown

Lenses: Has small pockets of different soils, note thickness

Homogeneous: Same color and appearance throughout

Visual-Manual Classification

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: 50% or more *retained* on the No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel/sand mixtures, little or no fines
			GP	Poorly-graded gravels and gravel/sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel/sand/silt mixtures
			GC	Clayey gravels, gravel/sand/clay mixtures
	Sands: More than 50% *passing* the No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand/silt mixtures
			SC	Clayey sands, sand/clay mixtures
Fine-Grained Soils: 50% or more Passes No. 200 Sieve	Silt and Clays Low Plasticity Fines		ML	Inorganic silts, rock flour, clayey silts
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			OL	Organic soil of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic soil of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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References:

ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
Terzaghi, K., and Peck, R.B., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons.

Scale of Relative Rock Hardness

Term	Field Identification	Hardness Designation	Approximate Unconfined Compressive Strength
Extremely Weak	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	R0	<150 psi
Very Weak	Crumbles under firm blows with point of geology pick. Can be peeled by pocket knife. Scratched with finger nail.	R1	150-725 psi
Weak	Can be peeled by pocket knife with difficulty. Cannot be scratched with finger nail. Shallow indention made by firm blow of geology pick.	R2	725-3,500 psi
Medium Strong	Can be scratched by knife or pick. specimen can be fractured with a single firm blow of hammer/geology pick.	R3	3,500-7,250 psi
Strong	Can be scratched with knife or pick only with difficulty. Several hard blows required to fracture specimen.	R4	7,250-14,500 psi
Very Strong	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	R5	14,500-36,250 psi
Extremely Strong	Can only be chipped with firm blows of hammer.	R6	>36,250 psi

Scale of Relative Rock Weathering

Designation	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. Decomposition extends up to 1 inch into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. 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	Rock mass is more than 50% 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. May be reduced to soil with hand pressure.

Spacing Terms

Bedding/Foliation	Spacing	Joints
Very thin	< 2 in.	Very close
Thin	2 in. - 1 ft.	Close
Medium	1 ft. - 3 ft.	Moderately Close
Thick	3 ft. - 10 ft.	Wide
Massive	> 10 ft.	Very Wide

Discontinuity/Joint Condition

Condition	Description
Very Good Condition	Very rough, fresh surfaces, no separation
Good Condition	Slightly rough, slightly weathered surfaces, separation less than 1 mm (0.05 in.)
Fair Condition	Smooth to slightly rough, moderately weathered and altered surfaces, separation greater than 1 mm (0.05 in.)
Poor Condition	Slickensided, highly weathered surfaces, or soft gouge less than 5 mm (0.2 in.) thick, or open discontinuities 1 to 5 mm (0.05 to 0.2 in.)
Very Poor Condition	Highly weathered surfaces with soft gouge greater than 5 mm (0.2 in.) thick, or open discontinuities greater than 5 mm (0.2 in.)

Rock Quality Designation

Rock Quality Designation (RQD) is the percent of a core run with intact lengths greater than 4 inches excluding mechanical breaks caused by drilling.

$$RQD = \frac{\sum \text{length of sound pieces >4 inches}}{\text{total length of core run in inches}}$$

RQD (%)	Designation
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Fracture Frequency

$$FF = \frac{\text{number of natural fractures}}{\text{total length of core recovered}}$$

Degree of Vesicularity

Term	Volume (%)
Trace Vesicles	<5
Some Vesicles	5 - 25
Vesicular	25 - 50
Scoriaceous	> 50

Explanation of Common Terms Used in Rock Descriptions, adapted from ASTM International D5878, 1987 Oregon Department of Transportation Soil and Rock Classification Manual, 2019 Washington State Department of Transportation Geotechnical Design Manual, and 2017 Federal Highway Administration Geotechnical Engineering Circular No. 5.

FIGURE A4
Boring B-1

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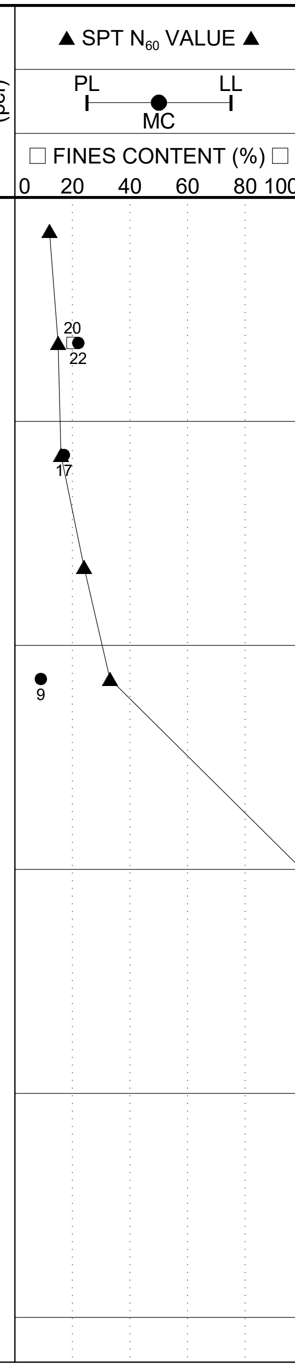
CLIENT Tetra Tech
PROJECT NAME Taylor Lane Bridge Replacement
PROJECT NUMBER B2301527
PROJECT LOCATION Taylor Lane - Umatilla County, OR
DATE STARTED 7/10/23 **GROUND ELEVATION** 970.1 ft
ELEVATION DATUM See Figure 2
WEATHER Sunny 80°F **SURFACE** Gravel
LOGGED BY GS **REVIEWED BY** SJK
DRILLING CONTRACTOR Western State Soil Conservation, Inc.
SEEPAGE ---
EQUIPMENT CME 75 Truck #5
GROUNDWATER DURING DRILLING ---
DRILLING METHOD 4-7/8" Tri Cone - Mud Rotary & HQ Rock Core
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 85.5%	DRY UNIT WT. (pcf)
	GP-GM FILL	**POORLY GRADED GRAVEL WITH SILT FILL:** Medium dense, gray, dry, subangular to angular, up to ¾-inch in diameter, nonplastic fines. {Base Rock}		0	SPT 1	17	7-6-5 (11)	12	
	SM	**SILTY SAND:** Medium dense, brown, moist, subangular to subrounded, fine- to medium-grained, nonplastic to low plasticity fines. Trace subrounded to rounded gravel up to ½-inch in diameter below 2½ feet bgs.			SPT 2	44	5-7-7 (14)	15	
965	ML	**SANDY SILT WITH GRAVEL:** >Stiff to very stiff, brown, moist to wet, nonplastic fines, subangular to subrounded, fine- to medium-grained sand (30% by volume), subangular to subrounded gravel (15% by volume) up to ¾-inch in diameter.		5	SPT 3	39	5-7-8 (15)	16	
	GP	**POORLY GRADED GRAVEL WITH SAND:** Medium dense, gray to red, moist, subangular to angular up to ¾-inch in diameter, subangular to subrounded, fine- to medium-grained sand (15% by volume).			SPT 4	67	7-11-11 (22)	24	
960		Dense below 10 feet bgs.		10	SPT 5	61	9-10-19 (29)	33	
955	RX	**BASALT:** Medium strong to strong (R3-R4), red to brown, slightly weathered, highly fractured between 15 and 17 feet bgs.		15	SPT 6	100	50/1"	100	
					CORE 7	100 (0)			
		Fresh, strong (R4), fewer fractures below 17 feet bgs.			CORE 8	100 (8)			
950		Rock compressive strength (ASTM 7012): 12,810 psi		20					
945		Highly fractured between 23½ and 24 feet bgs.		25	CORE 9	100 (40)			



- Boring terminated at 26 feet bgs.
- No groundwater encountered during drilling.
- No caving of borehole observed.
- Borehole backfilled with granular bentonite upon completion.

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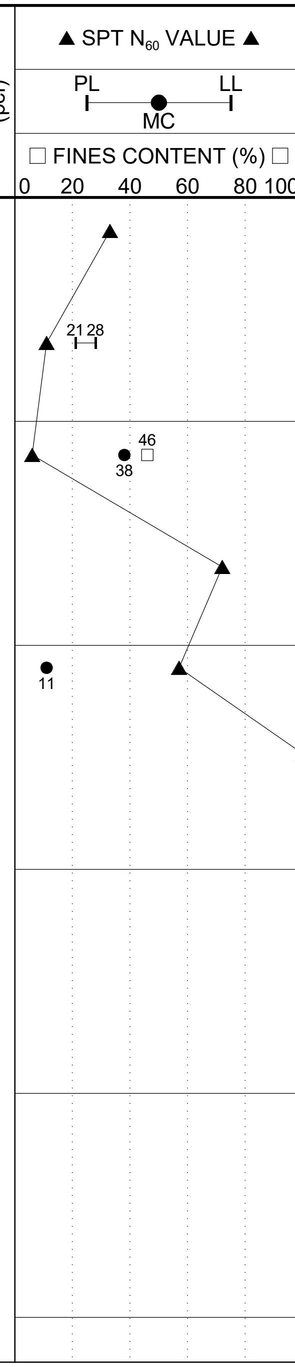
FIGURE A5

Boring B-2

CLIENT Tetra Tech
PROJECT NAME Taylor Lane Bridge Replacement
PROJECT NUMBER B2301527
PROJECT LOCATION Taylor Lane - Umatilla County, OR
DATE STARTED 7/10/23
GROUND ELEVATION 969.0 ft
ELEVATION DATUM See Figure 2
WEATHER Sunny 90°F
SURFACE Gravel
LOGGED BY GS
REVIEWED BY SJK
DRILLING CONTRACTOR Western State Soil Conservation, Inc.
SEEPAGE ---
EQUIPMENT CME 75 Truck #5
GROUNDWATER DURING DRILLING ---
DRILLING METHOD 4-7/8" Tri Cone - Mud Rotary & HQ Rock Core
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 85.5%	DRY UNIT WT. (pcf)
	GP-GM FILL	**POORLY GRADED GRAVEL WITH SILT FILL:** Dense, gray, dry, subangular to angular, up to ¾-inch in diameter, nonplastic fines. {Base Rock}	0	SPT 1	72	10-15-16 (31)	33	
965	SC-SM	**SILTY, CLAYEY SAND:** Medium dense, brown, dry, subangular to subrounded, fine- to medium-grained, nonplastic fines, trace subangular to subrounded gravel up to ½-inch in diameter.		SPT 2	89	3-5-5 (10)	11	
		Loose below 5 feet bgs. Moist to wet at 5 feet bgs.	5	SPT 3	100	1-1-5 (6)	6	
960	GP	**POORLY GRADED GRAVEL WITH SAND:** Very dense, brown to gray to red, subangular to angular up to ¾-inch in diameter, subangular to subrounded, fine- to medium-grained sand (20% by volume), trace orange mottling.		SPT 4	100	19-29-38 (67)	72	
			10	SPT 5	100	32-50	57	
955	RX	**BASALT:** Medium strong to strong (R3-R4), gray to red to brown, slightly weathered.		SPT 6	100	50/1"	100	
		Highly fractured between 14 and 17 feet bgs.	15	CORE 7	100 (0)			
950		Very strong (R5), fresh below 17 feet bgs.		CORE 8	100 (63)			
		Rock compressive strength (ASTM 7012): 18,740 psi	20					
945		Silty sand interbed between rock fracture at 22 feet bgs.	25	CORE 9	100 (7)			
940								



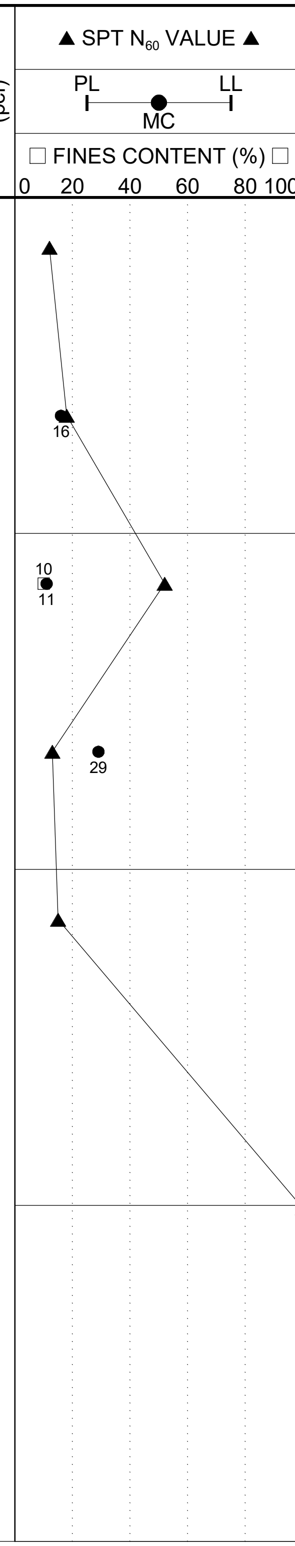
•Boring terminated at 26 feet bgs.
•No groundwater encountered during drilling.
•No caving of borehole observed.
•Borehole backfilled with granular bentonite upon completion.

FIGURE A6

Boring B-3

CLIENT Tetra Tech
PROJECT NAME Taylor Lane Bridge Replacement
PROJECT NUMBER B2301527
PROJECT LOCATION Taylor Lane - Umatilla County, OR
DATE STARTED 7/11/23 **GROUND ELEVATION** 970.1 ft
ELEVATION DATUM See Figure 2
WEATHER Sunny 90°F **SURFACE** Gravel
LOGGED BY GS **REVIEWED BY** SJK
DRILLING CONTRACTOR Western State Soil Conservation, Inc.
SEEPAGE ---
EQUIPMENT CME 75 Truck #5
GROUNDWATER DURING DRILLING ---
DRILLING METHOD 4-7/8" Tri Cone - Mud Rotary & HQ Rock Core
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 85.5%	DRY UNIT WT. (pcf)
		GP-GM FILL	**POORLY GRADED GRAVEL WITH SILT FILL:** Medium dense, gray, dry, subangular to angular, up to ¾-inch in diameter, nonplastic fines. {Base Rock}		0	SPT 1	33	6-5-6 (11)	12	
		ML	**SILT WITH SAND:** Stiff, brown, dry to moist, non to low plasticity fines, subangular to subrounded, fine- to medium-grained sand (20% by volume), trace subrounded to rounded gravel up to ¾-inch in diameter.							
		ML	**SANDY SILT WITH GRAVEL:** Very stiff, brown, dry to moist, nonplastic fines, subangular to subrounded, fine- to medium-grained sand (30% by volume), subrounded to round gravel (15% by volume) up to ¾-inch in diameter.			SPT 2	67	7-6-11 (17)	18	
965		GP-GM	**POORLY GRADED GRAVEL WITH SILT AND SAND:** Very dense, gray, wet, subangular to angular, up to 1 inch in diameter, subangular to subrounded, fine- to medium-grained sand (20% by volume).		5	SPT 3	44	7-28-21 (49)	52	
		SM	**SILTY SAND WITH GRAVEL:** Medium dense, gray, moist to wet, subangular to subrounded, fine- to medium-grained, nonplastic fines, subrounded to rounded gravel (15% by volume) up to ¾-inch in diameter, trace orange mottling.			SPT 4	61	10-5-7 (12)	13	
960		GP	**POORLY GRADED GRAVEL:** Medium dense, gray, wet, subangular to angular, up to ¼-inch in diameter, trace subangular to subrounded, fine- to medium-grained sand (>2% by volume).		10	SPT 5	17	17-8-5 (13)	15	
955		RX	**BASALT:** Medium strong to strong (R3-R4), red to gray, slightly weathered, fair condition, closely spaced subhorizontal jointing, trace vesicles. Silt with sand interbed, 5 inches thick at 15 feet bgs.		15	SPT 6	100	50/1"	100	
						CORE 7	100 (0)			
						CORE 8	100 (0)			
					20					



(Continued Next Page)

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FIGURE A6

Boring B-3

CLIENT Tetra Tech

PROJECT NAME Taylor Lane Bridge Replacement

PROJECT NUMBER B2301527

PROJECT LOCATION Taylor Lane - Umatilla County, OR

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 85.5%	DRY UNIT WT. (pcf)	▲ SPT N_{60} VALUE ▲ / PL —●— LL (MC) / ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
			BASALT: Medium strong to strong (R3-R4), red to gray, slightly weathered, fair condition, closely spaced subhorizontal jointing, trace vesicles. *(continued)*		20						
			Very strong (R5) below 21 feet bgs. Rock compressive strength (ASTM 7012): 15,350 psi								
						CORE 9	85 (43)				
			Yellow to tan, low plasticity clay interbed at 24 feet bgs, 3 inches thick.								
945		RX			25						
			Rock compressive strength (ASTM 7012): 22,660 psi			CORE 10	100 (22)				
940					30						
935			•Boring terminated at 31 feet bgs. •No groundwater encountered during drilling. •No caving of borehole observed. •Borehole backfilled with granular bentonite upon completion.								
930											

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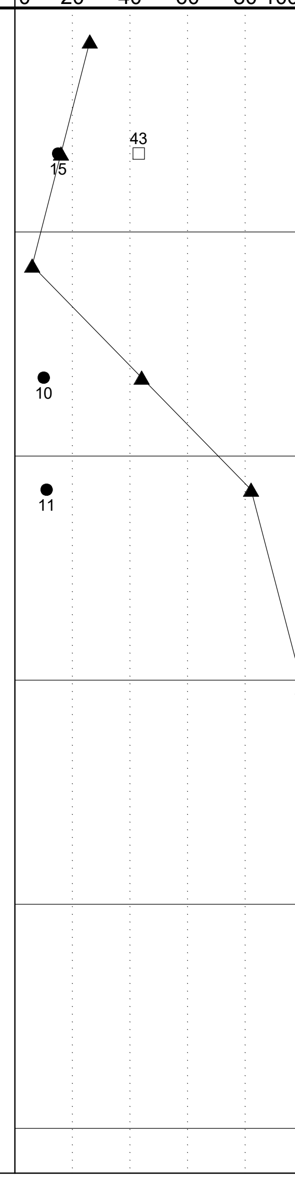
FIGURE A7

Boring B-4

CLIENT Tetra Tech
PROJECT NAME Taylor Lane Bridge Replacement
PROJECT NUMBER B2301527
PROJECT LOCATION Taylor Lane - Umatilla County, OR
DATE STARTED 7/11/23
GROUND ELEVATION 968.8 ft
ELEVATION DATUM See Figure 2
WEATHER Sunny 90°F
SURFACE Gravel
LOGGED BY GS
REVIEWED BY SJK
DRILLING CONTRACTOR Western State Soil Conservation, Inc.
SEEPAGE ---
EQUIPMENT CME 75 Truck #5
GROUNDWATER DURING DRILLING ---
DRILLING METHOD 4-7/8" Tri Cone - Mud Rotary & HQ Rock Core
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION
	GP-GM FILL	**POORLY GRADED GRAVEL WITH SILT FILL:** Medium dense, gray, dry, subangular to angular, up to ¾-inch in diameter, nonplastic fines. {Base Rock}
965	SM	**SILTY SAND WITH GRAVEL:** Medium dense, brown to gray, dry to moist, subangular to subrounded, fine- to medium-grained, nonplastic to low plasticity fines, subrounded to rounded gravel up to ¾-inch in diameter (15% by volume). Loose below 5 feet bgs.
960, 955	GP	**POORLY GRADED GRAVEL WITH SAND:** Dense, gray to red, moist, subangular to subrounded, up to ½-inch in diameter, subangular to subrounded, fine- to medium-grained sand (25% by volume). Very dense below 10 feet bgs.
950, 945	RX	**BASALT:** Weak to medium strong (R2-R3), red to brown, moderately weathered. Very strong (R5) below 16 feet bgs, red to gray, fresh, trace vesicles. Rock compressive strength (ASTM 7012): 23,160psi

SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 85.5%	DRY UNIT WT. (pcf)	MC	FINES CONTENT (%)
SPT 1	17	17-13-11 (24)	26			
SPT 2	44	5-7-8 (15)	16		15	43
SPT 3	0	2-3-3 (6)	6			
SPT 4	61	19-21-20 (41)	44		10	
SPT 5	67	15-22-50 (72)	82		11	
SPT 6	100	50/4"	100			
CORE 7	100 (62)					
CORE 8	100 (63)					



•Boring terminated at 26 feet bgs.
•No groundwater encountered during drilling.
•No caving of borehole observed.
•Borehole backfilled with granular bentonite upon completion.

Appendix B: Rock Core Photographs

Taylor Lane Bridge Replacement
Taylor Lane
Umatilla County, Oregon

CGT Project Number B2301527

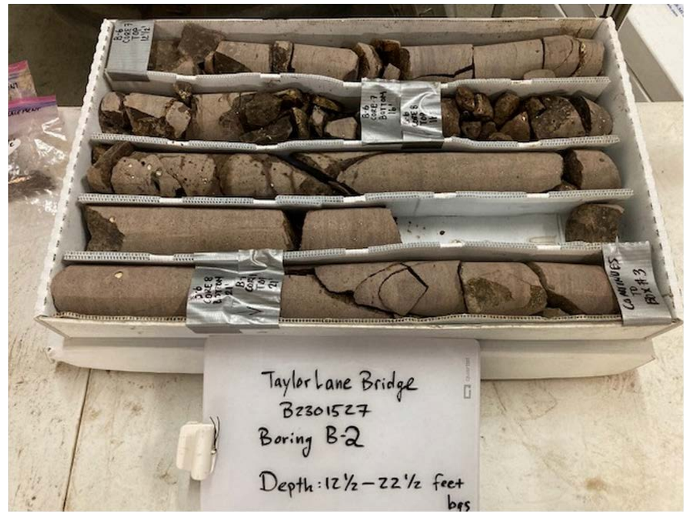
August 10, 2023

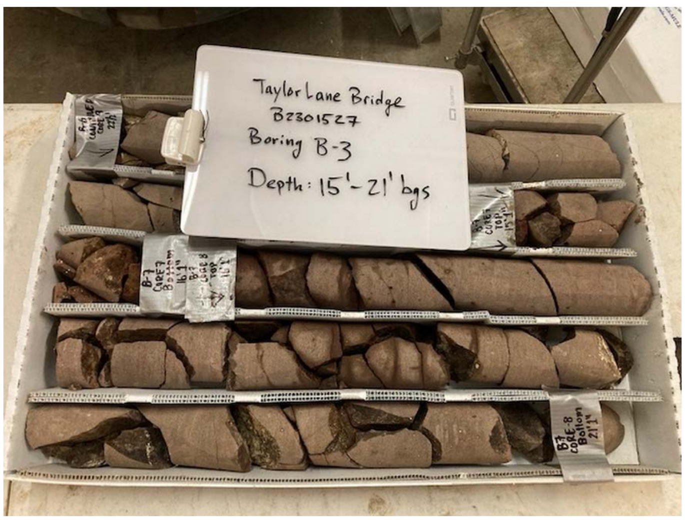
Prepared For:

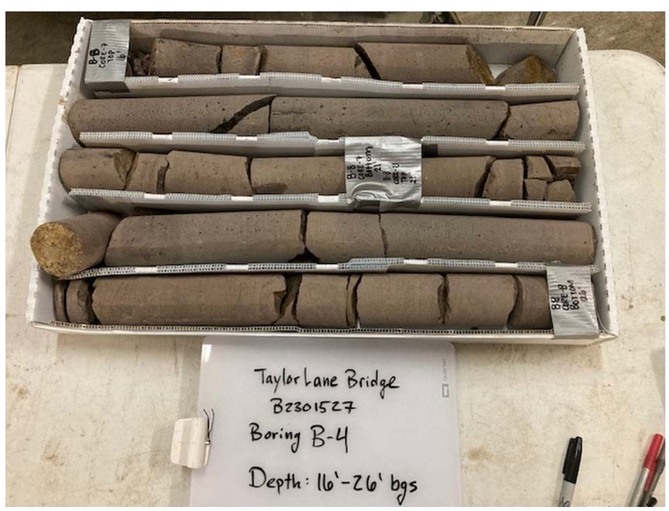
Chad McKinney, P.E.
Tetra Tech
19803 North Creek Parkway
Bothell, WA 98011

Prepared by
Carlson Geotechnical

TAYLOR LANE BRIDGE REPLACEMENT - UMATILLA COUNTY, OREGON Project Number B2301527	FIGURE B1 Rock Cores







CARLSON CGT GEOTECHNICAL 503-601-8250

Drafted by: PBR

See Figure 2 for boring locations. Photographs were taken upon return to the CGT Bend Soils Laboratory.