DESCHUTES COUNTY

ROAD DEPARTMENT

ADDENDUM NO. 1

HAMEHOOK RD BRIDGE #17C32 REPLACEMENT

The Bidding Documents for the HAMEHOOK RD BRIDGE #17C32 REPLACEMENT project are amended as follows:

BIDDING PLANS

- ***Replace Plan Sheets C4.0 through C4.3 with the attached.***
  - Cut/Fill volumes are now included in the Profile View on each sheet.

BIDDING REFERENCE DOCUMENTS

- ***Include the attached Geotechnical Report as a Bid Reference Document.***

The Bidding Documents for the HAMEHOOK RD BRIDGE #17C32 REPLACEMENT project are amended as described above.

Cody Smith, PE
County Engineer/Assistant Director

Date

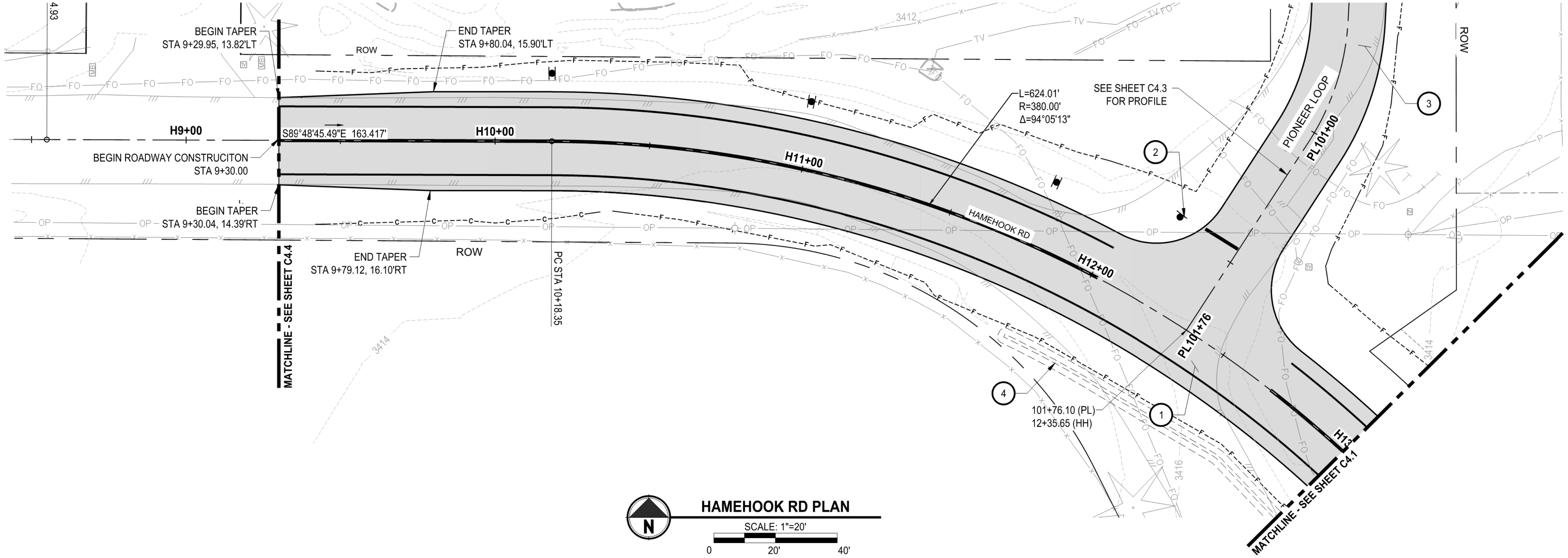
I acknowledge receipt of Addendum No. 1.

BIDDER NAME

SIGNATURE OF BIDDER

Date

THIS ADDENDUM, EXCLUDING ATTACHMENTS, SHALL BE SIGNED AND SUBMITTED WITH THE BID PROPOSAL BY THE BIDDER.



HAMEHOOK RD PLAN

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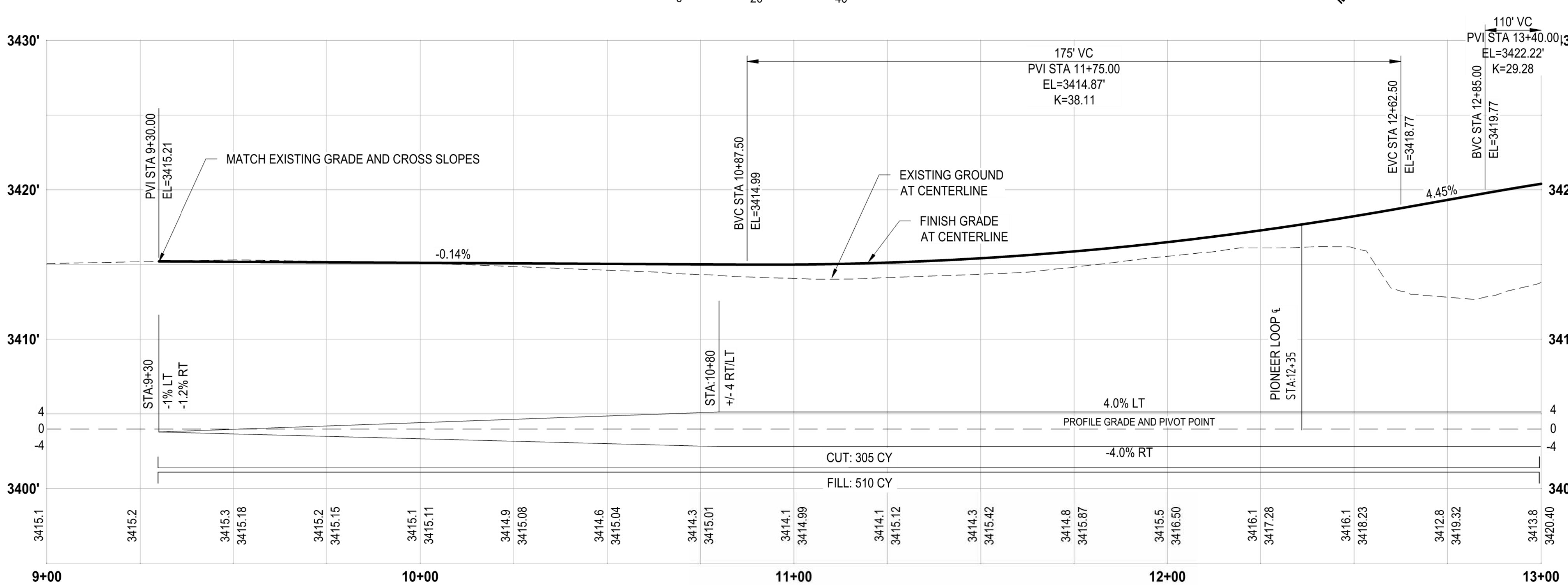


HAMEHOOK RD PROFILE

H: 1"=20' V: 1"=5'

CONSTRUCTION NOTES

1. CONSTRUCT ROAD PER TYPICAL SECTION 1/C1.0
2. INSTALL STOP SIGN. SEE SIGNING AND STRIPING PLAN
3. CONSTRUCT ROAD PER TYPICAL SECTION 3/C1.0
4. CONSTRUCT WATER QUALITY SWALE PER DETAIL 1/C5.0

BIDDING PLANS

REVISIONS	DATE	BY	
			DESIGNED DR
			DRAWN DR, CA, TVM
			CHECKED BCJ, DR
			APPROVED BCJ

ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY

FILE NAME BE2509010-C4-PP

JOB No. 297-2509-010

DATE AUGUST 2024

REGISTERED PROFESSIONAL ENGINEER 94066PE David Rico OREGON SEPT. 11, 2018 DAVID RICO EXPIRES: 12/31/2024

Digitally signed by David Rico Date: 2024.08.28 11:25:07-07'00'

Parametrix

PROJECT NAME

HAMEHOOK RC BRIDGE #17C32 REPLACEMENT

DESCHUTES COUNTY

PLAN & PROFILE-HAMEHOOK RD

DRAWING NO. 18 OF 28

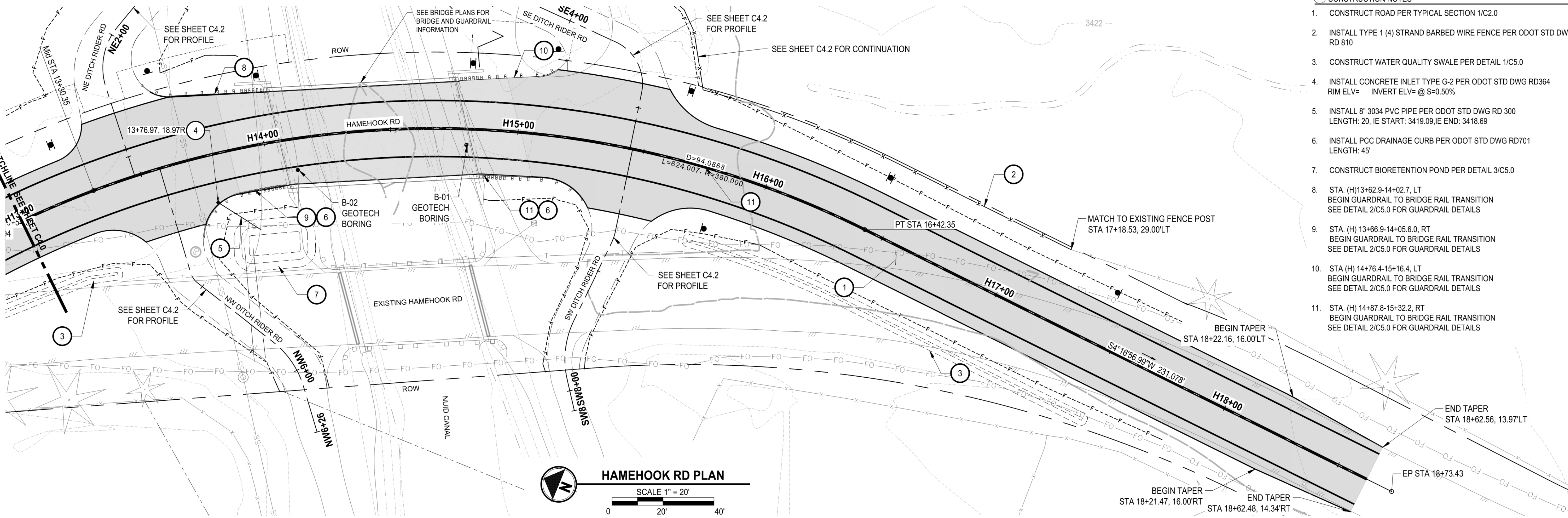
C4.0

PATH: U:\Bend\Projects\Clients\2509-Deschutes County\297-2509-010 Hamehook Rd Bridge\99Svcs\CADD\DWG\SHEETS\3-ROADWAY PLOTTED BY: ricodav DATE: Wednesday, August 28, 2024 3:03:28 PM

LAYOUT: C4.0

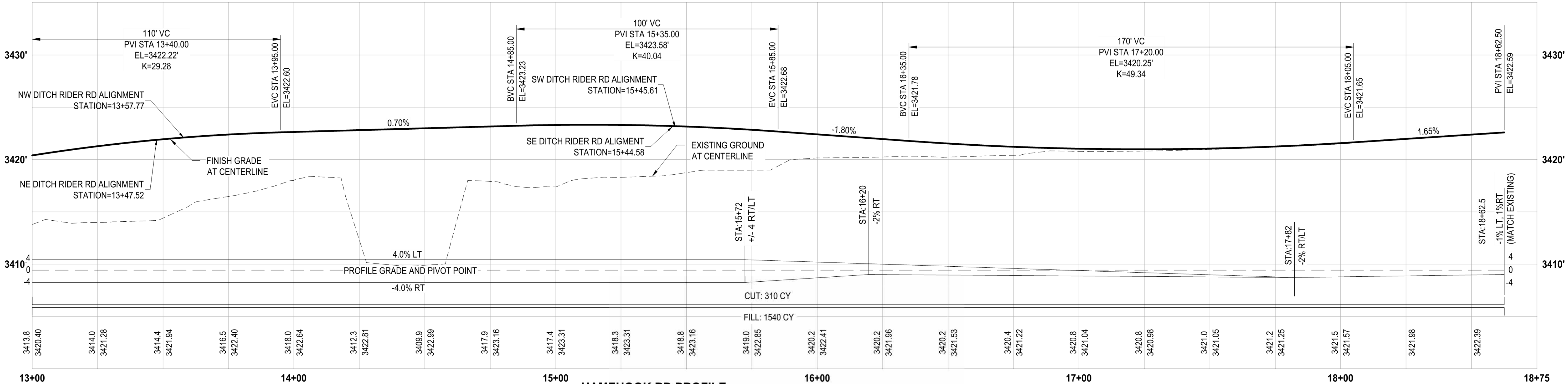
CONSTRUCTION NOTES

1. CONSTRUCT ROAD PER TYPICAL SECTION 1/C2.0
2. INSTALL TYPE 1 (4) STRAND BARBED WIRE FENCE PER ODOT STD DWG RD 810
3. CONSTRUCT WATER QUALITY SWALE PER DETAIL 1/C5.0
4. INSTALL CONCRETE INLET TYPE G-2 PER ODOT STD DWG RD364 RIM ELV= INVERT ELV= @ S=0.50%
5. INSTALL 8" 3034 PVC PIPE PER ODOT STD DWG RD 300 LENGTH: 20, IE START: 3419.09,IE END: 3418.69
6. INSTALL PCC DRAINAGE CURB PER ODOT STD DWG RD701 LENGTH: 45'
7. CONSTRUCT BIORETENTION POND PER DETAIL 3/C5.0
8. STA. (H)13+62.9-14+02.7, LT BEGIN GUARDRAIL TO BRIDGE RAIL TRANSITION SEE DETAIL 2/C5.0 FOR GUARDRAIL DETAILS
9. STA. (H) 13+66.9-14+05.6.0, RT BEGIN GUARDRAIL TO BRIDGE RAIL TRANSITION SEE DETAIL 2/C5.0 FOR GUARDRAIL DETAILS
10. STA (H) 14+76.4-15+16.4, LT BEGIN GUARDRAIL TO BRIDGE RAIL TRANSITION SEE DETAIL 2/C5.0 FOR GUARDRAIL DETAILS
11. STA. (H) 14+87.8-15+32.2, RT BEGIN GUARDRAIL TO BRIDGE RAIL TRANSITION SEE DETAIL 2/C5.0 FOR GUARDRAIL DETAILS



HAMEHOOK RD PLAN

SCALE 1" = 20'



HAMEHOOK RD PROFILE

H: 1"=20' V: 1"=5'

BIDDING PLANS

REVISIONS	DATE	BY	
			DESIGNED DR
			DRAWN DR, CA, TVM
			CHECKED BCJ, DR
			APPROVED BCJ

ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY

FILE NAME BE2509010-C4-PP

JOB No. 297-2509-010

DATE AUGUST 2024

REGISTERED PROFESSIONAL ENGINEER 94066PE David Rico OREGON SEPT. 11, 2018 DAVID RICO

Digitally signed by David Rico Date: 2024.08.28 17:25:07-07'00'

EXPIRES: 12/31/2024

Parametrix

PROJECT NAME

HAMEHOOK RC BRIDGE #17C32 REPLACEMENT

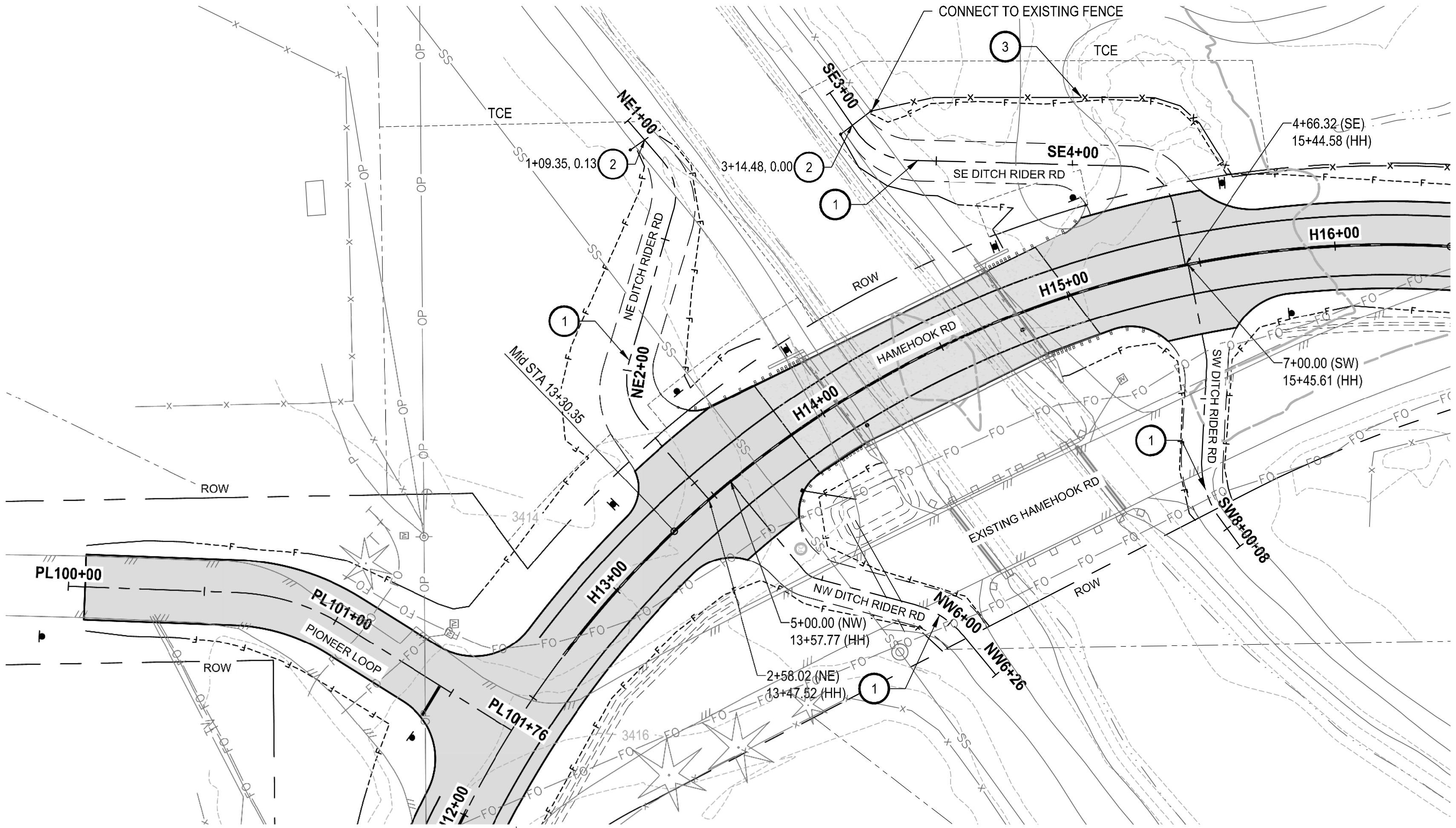
DESCHUTES COUNTY

PLAN & PROFILE-HAMEHOOK RD

DRAWING NO. 19 OF 28

C4.1

LAYOUT: C4.1 PATH: U:\Bend\Projects\Clients\2509-Deschutes County\297-2509-010 Hamehook Rd Bridge\99Svcs\CADD\DWG\SHEETS\3-ROADWAY PLOTTED BY: ricodav DATE: Wednesday, August 28, 2024 3:05:28 PM



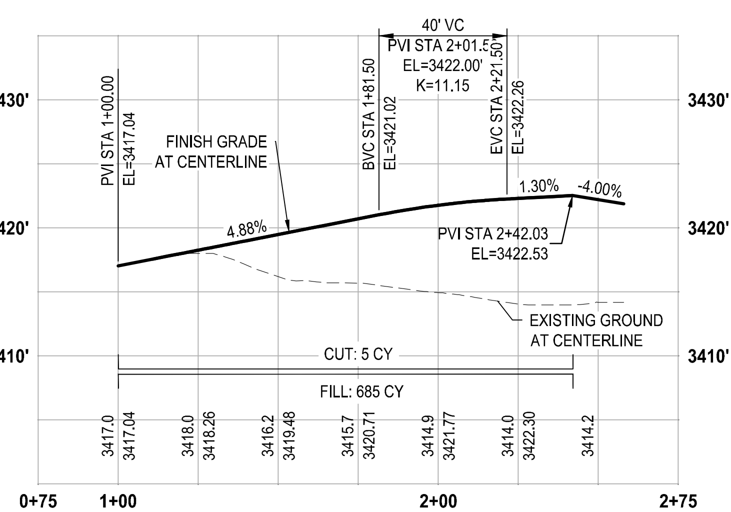
PLAN & PROFILE-DITCH RIDER RD

SCALE: 1"=30'

0 30' 60'

CONSTRUCTION NOTES

1. CONSTRUCT DITCH RIDER ROADS PER DETAIL 2/C2.0
2. REINSTALL GATE
3. INSTALL TYPE 1 (4) STRAND BARBED WIRE FENCE PER ODOT STD DWG RD 810



NE DITCH RIDER RD PROFILE

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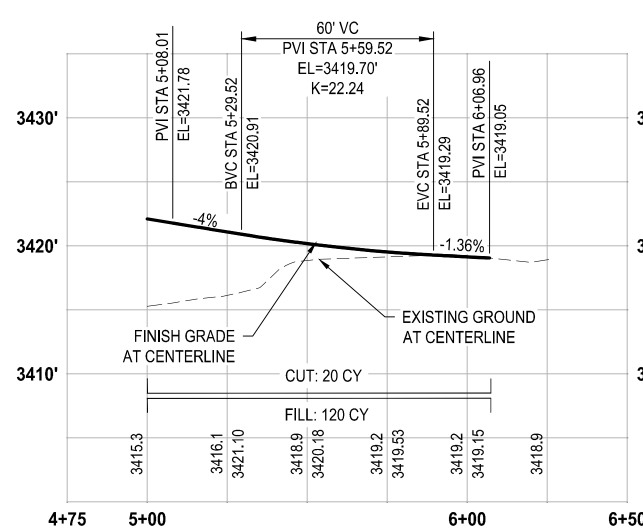

NW DITCH RIDER RD PROFILE

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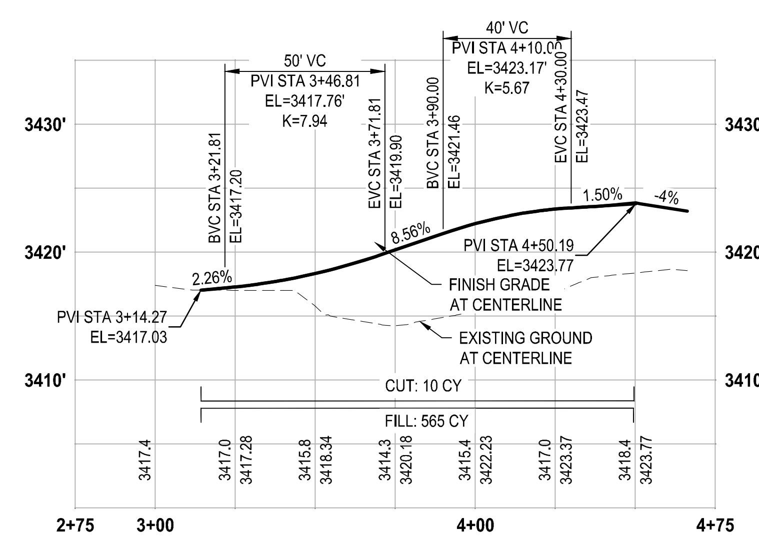

SE DITCH RIDER RD PROFILE

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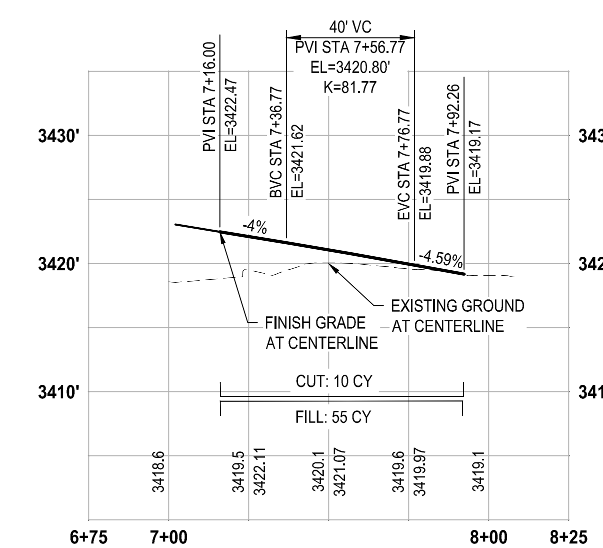


SW DITCH RIDER RD PROFILE

H: 1"=30' V: 1"=8'

BIDDING PLANS

REVISIONS	DATE	BY	
			DESIGNED DR
			DRAWN DR, CA, TVM
			CHECKED BCJ, DR
			APPROVED BCJ

ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY

FILE NAME BE2509010-C4-PP

JOB No. 297-2509-010

DATE AUGUST 2024

REGISTERED PROFESSIONAL ENGINEER 94066PE David Rico OREGON SEPT. 11, 2018 DAVID RICO

Digitally signed by David Rico Date: 2024.08.28 11:25:07-07'00'

EXPIRES: 12/31/2024

Parametrix

PROJECT NAME

HAMEHOOK RC BRIDGE #17C32 REPLACEMENT

DESCHUTES COUNTY

PLAN & PROFILE-DITCH RIDER RD

DRAWING NO. 20 OF 28

C4.2

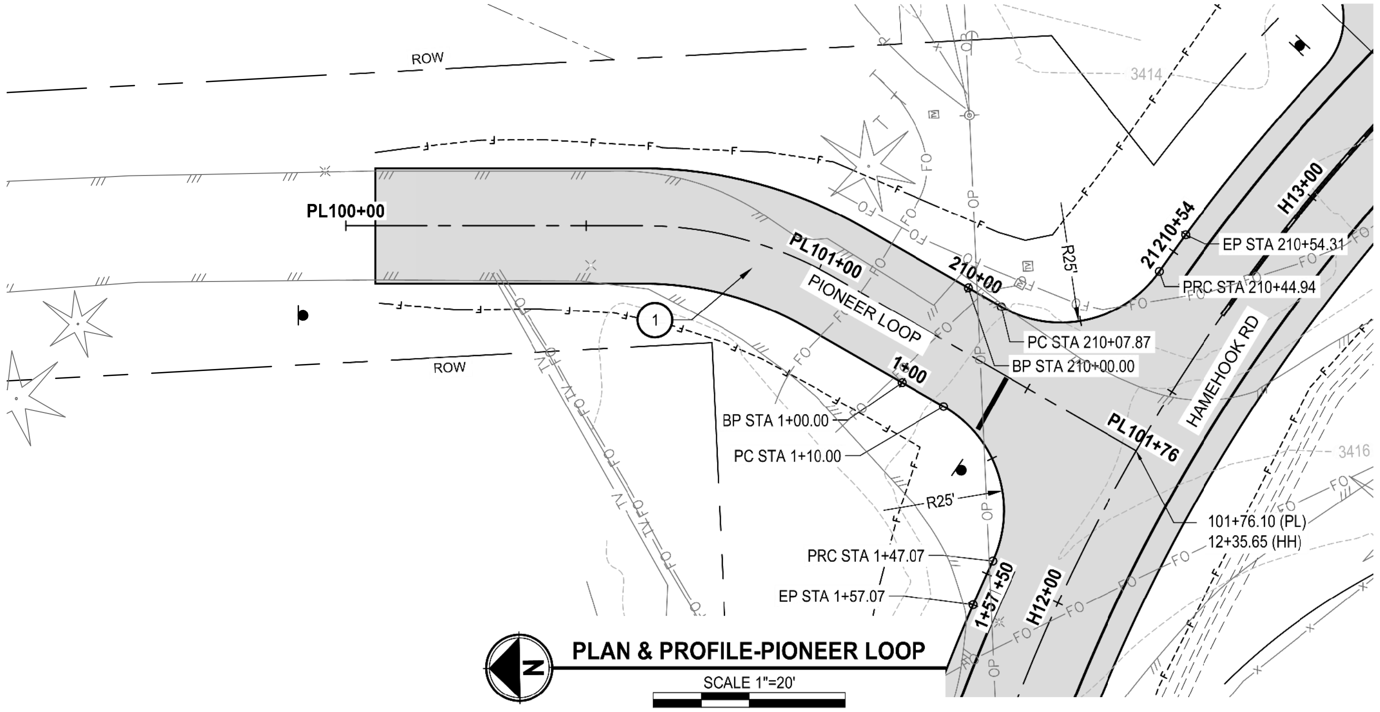
PLOTTED BY: ricodav DATE: Wednesday, August 28, 2024 11:20:08 AM

PATH: U:\Bend\Projects\Clients\2509-Deschutes County\297-2509-010 Hamehook Rd Bridge\99Svcs\CADD\DWG\SHEETS\3-ROADWAY

LAYOUT: C4.2

CONSTRUCTION NOTES

1. CONSTRUCT PIONEER LOOP ROAD PER TYPICAL SECTION 3/C1.0



PLAN & PROFILE-PIONEER LOOP

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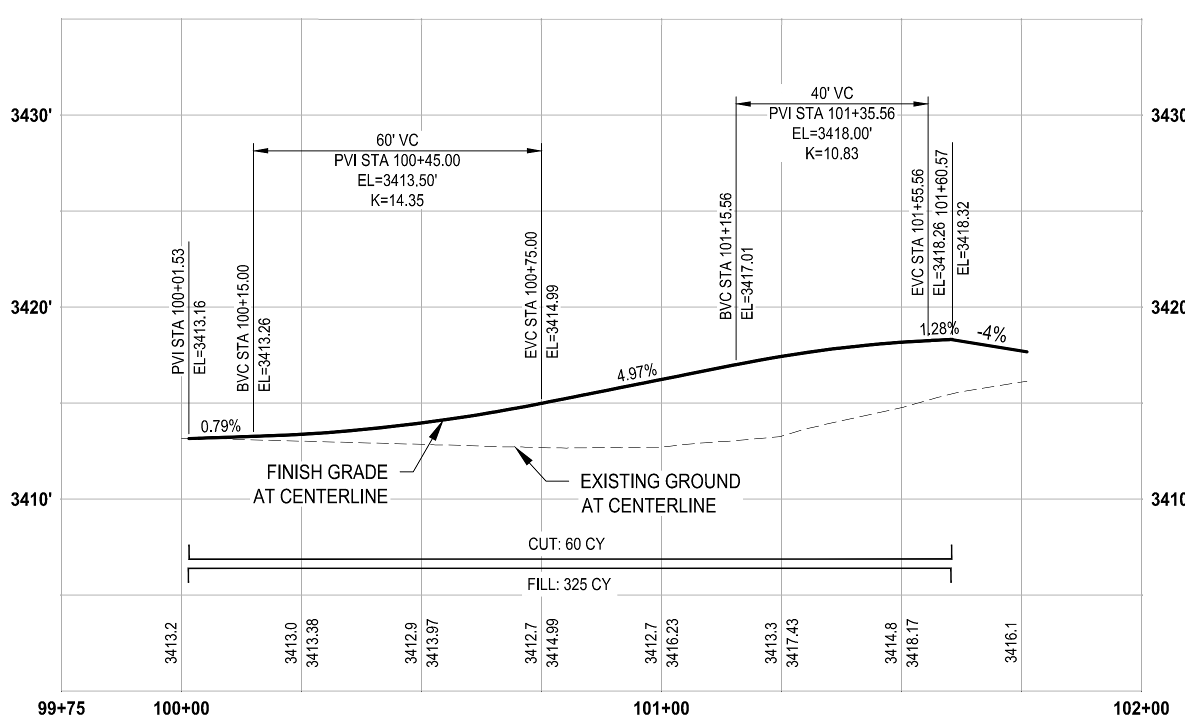
0 20' 40'



PIONEER LOOP PROFILE

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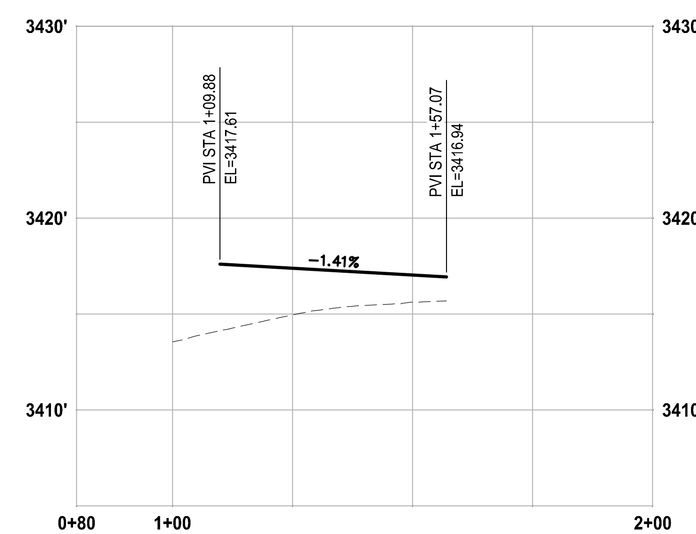

PIONEER LOOP NORTH CR PROFILE

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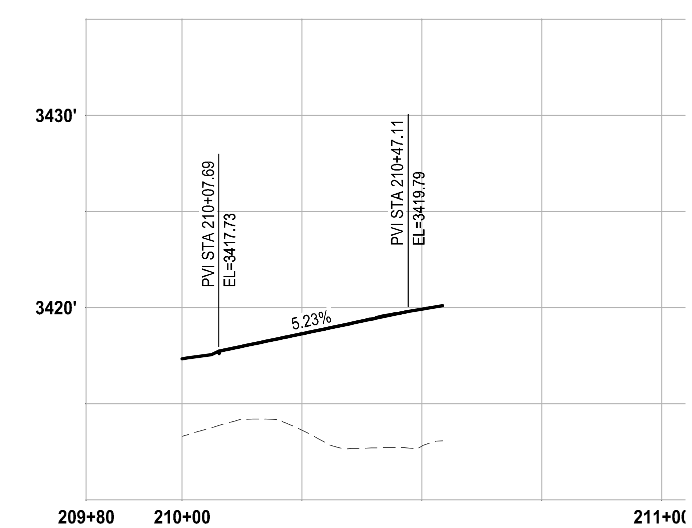

PIONEER LOOP SOUTH CR PROFILE

H: 1"=30' V: 1"=5'

BIDDING PLANS

REVISIONS	DATE	BY	
			DESIGNED DR
			DRAWN DR, CA, TVM
			CHECKED BCJ, DR
			APPROVED BCJ

ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY

FILE NAME BE2509010-C4-PP

JOB No. 297-2509-010

DATE AUGUST 2024

REGISTERED PROFESSIONAL ENGINEER 94066PE David Rico OREGON SEPT. 11, 2018 DAVID RICO

Digitally signed by David Rico Date: 2024.08.28 11:25:32-07'00'

EXPIRES: 12/31/2024

Parametrix

PROJECT NAME

HAMEHOOK RC BRIDGE #17C32 REPLACEMENT

DESCHUTES COUNTY

PLAN & PROFILE-PIONEER LOOP

DRAWING NO. 21 OF 28

C4.3

LAYOUT: C4.3 PATH: U:\Bend\Projects\Clients\2509-Deschutes County\297-2509-010 Hamehook Rd Bridge\99Svcs\CADD\DWG\SHEETS\3-ROADWAY PLOTTED BY: ricodav DATE: Wednesday, August 28, 2024 11:22:19 AM

WALLACE GROUP
NORTHWEST GEOSYSTEM EXPERTS

GEOTECHNICAL EXPLORATION REPORT
HAMEHOOK ROAD BRIDGE
BEND, OREGON

January 19, 2024

Wallace Group Project No. 23106-1

A Report Prepared For:

Parametrix
c/o Mr. Barry Johnson
150 NW Pacific Park Lane, Suite 110
Bend, OR 97701

GEOTECHNICAL EXPLORATION REPORT
HAMEHOOK ROAD BRIDGE
BEND, OREGON

Wallace Group Project Number 23106-1

Prepared By:

Adam Larson, P.E.
Project Geotechnical Engineer

Lisa M. Splitter, P.E., G.E.
Senior Geotechnical Engineer

The Wallace Group, Inc.
62915 NE 18th Street, Suite 1
Bend, OR 97701

EXECUTIVE SUMMARY

The Wallace Group, Inc., (Wallace Group) completed a geotechnical engineering exploration for the Hamehook Road Bridge located in Bend, Oregon (**Figure 1**, *Vicinity Map*). A figure with exploration locations is shown on **Figure 2**, *Exploration Location Map*.

We understand that the existing Hamehook Road Bridge is a two-span timber bridge that was constructed in 1977. The bridge is located in Deschutes County near northeast Bend and spans the North Unit Irrigation District (NUID) canal which is owned by the U.S. Bureau of Reclamation. Presently, the bridge has a sufficiency rating of 73.9 and has notable deficiencies including: cracking of the timber deck and girders, delamination of the concrete bents, cracking and potholes in the asphalt wearing course, and generally does not meet current standards for vehicle and pedestrian safety. Based on review of preliminary conceptual plans prepared by Parametrix (project civil and structural engineer), undated, we anticipate that a new bridge will be constructed approximately 37-feet downstream of the existing bridge. The new bridge will also have an updated horizontal curve in the road alignment that will accommodate better sight distance for increased user safety. Based on the conceptual plans, we anticipate the bridge deck will be approximately 40-feet wide and will include two 11-foot-wide travel lanes with minimum 5-foot-wide shoulders. The bridge will also have a 4-percent superelevation and will be slightly skewed to accommodate the horizontal curvature of the road alignment. We anticipate that each bridge abutment (north and south) will consist of seven (7) HP piles embedded in basalt bedrock with a pile cap.

Subsurface explorations generally encountered existing fill and native silty-sand underlain by basalt bedrock. The undocumented fill materials extended from the surface to a depth of approximately 5-feet below ground surface (bgs). Undocumented fill materials consisted of medium dense aggregate base course (ABC) and silty-sand. Native silty-sand was encountered below the undocumented fill materials in each boring location. The native silty sand extended to depths ranging between approximately 12- to 13-feet bgs and was very loose to medium dense. Basalt bedrock was encountered in each exploration location and extended the full depth of our exploration of approximately 42-feet bgs. The basalt was generally unweathered to highly weathered, gray, and vesicular to massive. Open voids were encountered in B-01 at a depth of 35-feet bgs and 6-inches thick, and in B-02 at a depth of 29.5-feet bgs and 2.5-feet thick. The basalt bedrock was generally very broken and rubblized in each boring at depths ranging between 31- to 36-feet bgs. Due to the depths of the voids, we do not anticipate significant impacts to the construction of the new bridge.

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Based on this exploration, the sites appear suitable for the expected development from a geotechnical perspective, provided the recommendations contained in this report are incorporated into design and construction.

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62915 NE 18th St, Ste 1, Bend, OR 97701 **p |** 541.382.4707 **f |** 541.383.8118 wallacegroup-inc.com

TABLES

FIGURES

APPENDICES

62915 NE 18th St, Ste 1, Bend, OR 97701 **p |** 541.382.4707 **f |** 541.383.8118 wallacegroup-inc.com

1.0 INTRODUCTION

1.1 GENERAL

The Wallace Group, Inc., (Wallace Group) completed a geotechnical engineering exploration for the Hamehook Road Bridge project, located in Bend, Oregon (**Figure 1**, *Vicinity Map*). A figure with exploration locations is shown on **Figure 2**, *Exploration Location Map*.

1.2 PROJECT DESCRIPTION

We understand that the existing Hamehook Road Bridge is a two-span timber bridge that was constructed in 1977. The bridge is located in Deschutes County near northeast Bend and spans the North Unit Irrigation District (NUID) canal which is owned by the U.S. Bureau of Reclamation. Presently, the bridge has a sufficiency rating of 73.9 and has notable deficiencies including: cracking of the timber deck and girders, delamination of the concrete bents, cracking and potholes in the asphalt wearing course, and generally does not meet current standards for vehicle and pedestrian safety. Based on review of preliminary conceptual plans prepared by Parametrix (project civil and structural engineer), undated, we anticipate that a new bridge will be constructed approximately 37-feet downstream of the existing bridge. The new bridge will also have an updated horizontal curve in the road alignment that will accommodate better sight distance for increased user safety. Based on the conceptual plans, we anticipate the bridge deck will be approximately 40-feet wide and will include two 11-foot-wide travel lanes with minimum 5-foot-wide shoulders. The bridge will also have a 4-percent superelevation and will be slightly skewed to accommodate the horizontal curvature of the road alignment. We anticipate that each bridge abutment (north and south) will consist of seven (7) HP piles embedded in basalt bedrock with a pile cap.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this exploration was to evaluate general subsurface conditions for the Hamehook Road Bridge development area to provide recommendations for foundation design and earthwork considerations. Our scope of services included the following tasks:

- Review regional and project area geology;
- Explore subsurface conditions using hollow stem auger drilling and HQ-wireline coring techniques to collect in-situ samples and perform Standard Penetration Tests (SPT);
- Perform geotechnical laboratory testing;
- Perform engineering evaluations for the feasibility of geotechnical-related aspects of development; and

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- Prepare this design-level geotechnical report summarizing our findings, conclusions, and recommendations.

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2.0 FIELD EXPLORATION

2.1 SUBSURFACE EXPLORATION

Subsurface conditions were explored on October 23 and 24, 2023. Two (2) geotechnical borings, designated B-01 and B-02, were advanced to depths of approximately 41.5- to 42-feet below ground surface (bgs). Borings were performed using a CME 850 truck-mounted drilling rig operated by Western States Soil Conservation, Inc. of Hubbard, Oregon. The boring logs, located in **Appendix A**, describe the materials encountered at each location explored. The soil and bedrock types between explorations are anticipated to be similar; however, variation should be expected. The stratigraphic contacts indicated at each point of exploration represent the approximate boundaries between soil and bedrock types. The approximate locations of the borings are shown on **Figure 2**.

A more complete description of the sampling techniques and soil-classification terminology is presented in **Appendix A**.

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3.0 LABORATORY TESTING

Soil samples were transported to our Bend geotechnical laboratory where they were visually classified and prepared for laboratory analyses. Laboratory tests were conducted in general accordance with standard procedures and included the following:

- Moisture Content, ASTM D2216;
- Rock Core Compressive Strength, ASTM D7012;
- Consolidated Undrained Direct Shear Testing, ASTM D3080;
- Gradation Analyses, ASTM C117 / C136; and
- Classification of Soil for Engineering Purposes, ASTM D2487.

Laboratory test results are included in **Appendix B**.

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4.0 EXISTING SITE CONDITIONS

4.1 SURFACE

The project site has previously been developed with the construction of the existing bridge, NUID canal, and adjacent ditch-rider roads. Based on our understanding of the project plans, the existing Hamehook Road bridge was constructed in 1977. We anticipate the NUID canal pre-dates the bridge by at least several decades. The NUID canal is approximately 48-feet-wide and 5-feet deep. The canal in this stretch is concrete lined and has a floor elevation of approximately 3,413 feet mean sea level (msl). The concrete-lined side slopes of the canal are nearly 1 to 1 horizontal to vertical (H to V). The ditch-rider road is surfaced with 6-inches of ABC on the north side of the canal and is exposed soil on the south side. Within the project development area, the groundcover generally consists of sagebrush, rabbitbrush, and western junipers.

4.2 SUBSURFACE CONDITIONS

Subsurface explorations generally encountered undocumented fill and native silty-sand underlain by basalt bedrock. The undocumented fill materials extended from the surface to depths of approximately 5-feet bgs. Undocumented fill materials consisted of medium dense aggregate base course (ABC) and silty-sand. Native silty-sand was encountered below the undocumented fill materials in both boring locations. The native silty sand extended to depths ranging between approximately 12- to 13-feet bgs. The silty-sand contained varying amounts of gravel and cobble that generally increased in quantity near the contact with the bedrock surface. Basalt bedrock was encountered in each exploration location and extended the full depth of our exploration of approximately 42-feet bgs.

4.2.1 Undocumented Fill

Where explored, undocumented fill materials were encountered extending from the surface to depths of approximately 5-feet bgs. The undocumented fill materials consisted of ABC and silty-sand that is likely sourced from the project area. Due to the sampling nature of drilled borings, it is difficult to discern the actual depth of fill materials when the fill materials are similar to the native soil. However, we anticipate there is approximately 5-feet of undocumented fill materials based on review of existing and historical ground surface elevations. Based on Standard Penetration Testing (SPT's), the undocumented fill material is generally medium dense. Gradation tests indicate the existing fill generally classifies as silty-sand (SM), according to the Unified Soil Classification System (USCS). Where tested, the fill soil samples contained between 22 to 26 percent clay- and silt-sized particles, had moisture contents ranging between

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5.8 to 8.5 percent, and contained varying amounts of cobble and gravel-sized particles. The silty-sand was slightly moist, brown, and fine- to coarse-grained.

4.2.2 Silty-Sand

Native, silty-sand was encountered extending below the existing fill up to depths of between 12- to 13-feet bgs in the borings. Gradation tests indicate the native soil generally classifies as silty-sand, according to the USCS. Based on relative density recordings using SPT's, the native silty-sand was generally very loose to dense. Where tested, the soil samples contained between 19 to 21 percent clay- and silt-sized particles, had moisture contents ranging between 16.2 to 24.7 percent, and contained varying amounts of gravel-sized particles. Based on direct shear testing, the native soil has an undrained angle of internal friction (Phi) of 38.9° and a cohesion of 50 pounds per square foot (psf). The silty-sand was moist, brown, and fine- to coarse-grained.

4.2.3 Basalt Bedrock

Basalt bedrock was encountered below the native soil at depths ranging between 12- to 13-feet bgs (corresponding to elevation 3,406 to 3,405 feet msl). The basalt was generally unweathered to highly weathered, gray, and vesicular to massive. Based on laboratory testing results, the core compressive strength of the basalt generally ranged from moderate to medium high strength (R3 to R4) and had compressive strengths that ranged from approximately 4,880 to 8,140 pounds per square inch (psi). Open voids were encountered in B-01 at a depth of 35-feet bgs and 6-inches thick, and in B-02 at a depth of 29.5-feet bgs and 2.5-feet thick. The basalt bedrock was generally very broken and rubblized in each boring at depths ranging between 31- to 36-feet bgs.

4.3 GROUNDWATER

Groundwater was not encountered in the explorations and should not influence site development. Seasonal perched water may occur at the soil and basalt bedrock interface during periods of precipitation or when the canal is carrying water. A review of well logs obtained from the Oregon Water Resource Department indicates groundwater is at least 500 feet bgs in the project area.

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5.0 SEISMIC CONSIDERATIONS

5.1 REGIONAL AND SITE GEOLOGY

Bend is located within the High Lava Plains Physiographic Province of central Oregon. This relatively young volcanic region is characterized by a multitude of volcanic cones and buttes, lava flows, and lava tubes. The region exhibits moderate topographic relief and a poorly developed drainage network with few canyons and gullies because of central Oregon's location in the rain shadow formed by the western and high Cascade Mountains. Annual precipitation averages between 10 and 15 inches. With exception of talus depositions, isolated lake-bottom sediments, and fluvial debris, most of the rocks in the province are volcanic and thick accumulations of basaltic lava are common.

Shallow, on-site native soil consists of silty-sand and gravel reflecting the deposition and weathering of volcanic ash and pumice. The volcanic soil is underlain at relatively shallow depths by basalt and Pleistocene Age basalt flows. The basalt flows originated from the north and west flanks of Newberry Volcano located approximately 26 miles south of the site (Sherrod, et al., 2004). These coalescing flows consist of gray, porphyritic, olivine basalt and basaltic andesite. The flows are partially vesicular and exhibit columnar jointing in some exposures, in addition to flow features such as pressure ridges, tumuli, pressure plateaus, and residual depressions. These flows are responsible for most of the current landscape and topography in the Bend area.

Seismically, the project site is in a northwest-trending zone of en echelon faults that parallel the High Cascade Mountain Range. These structural features extend northward from a convergence zone at Newberry Volcano and are generally referred to as the Metolius and Sisters Fault Zone. This zone includes the Rimrock-Tumalo, Sisters, and Green Ridge Faults and may represent a structural transition between the Brothers and Metolius Fault zones (Geomatrix, 1995), or mark the eastern boundary of the High Cascades Graben (Taylor, 1981). The nearest fault of the Sisters Fault Zone is located approximately 1-mile southwest of the project area. Generally, this portion of the Sisters Fault Zone is thought to be last active during the middle and late Quaternary, and less than 750,000 years old (USGS, 2024). Therefore, the Sisters fault zone is geologically considered 'inactive' due to the age of the latest displacement. Based on our geologic research and local experience, the fault is reasonably mapped and risk of fault rupture at the project site is negligible.

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5.2 SEISMIC HAZARDS

Central Oregon is in an area of low to moderate seismic risk. The 2022 Oregon Structural Specialty Code (OSSC), based on the 2021 International Building Code (IBC), requires that the structure be designed to sustain the maximum considered earthquake. Based on subsurface explorations on the project site and the anticipated design elevations for the structure we anticipate the structure will be underlain by less than approximately 13 feet of fill and/or native soil. Fill and native, silty-sand soil underlain by basalt bedrock are not liquefiable during earthquakes. Other seismically related hazards, including lateral spreading, landslides, and fault rupture are not applicable for this project. The average subsurface profile for the upper 100 feet most closely resembles the OSSC description for "Rock." Therefore, we recommend ASCE 7-16 Seismic Site Class B – Estimated (see **Appendix C**).

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6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of field exploration, laboratory testing, engineering analyses, and our local experience, it is our opinion that the site is suitable for the anticipated development from a geotechnical perspective.

6.1 EXCAVATIONS AND GRADING

Cuts and fills will generally be less than approximately 5-feet to remove undocumented fill and native soil. Cuts to remove the fill and native silty-sand soil can generally be excavated with conventional earth-moving equipment such as excavators and small dozers.

Excavations made in fill and native soil should be classified as “Type C” material for Occupational Safety and Health Administration (OSHA) excavation purposes. If sloping excavations are used in fill material or native soil, the temporary slopes should not be steeper than 1.5 to 1, horizontal to vertical (H to V). Permanent slopes should be at grades no steeper than 2 to 1 (H to V).

6.1.1 Site Preparation

Given the local soil types and inconsistent nature of undocumented fill, we recommend removal and replacement with new structural fill meeting the project specifications. The undocumented fill soil encountered during the exploration is granular and is not expected to consolidate under surcharge loads. However, because undocumented fill consistency is unknown, differential settlement would be expected if left in place.

We recommend that development areas are stripped of all undocumented fill materials and organic soil to expose native, inorganic soil. The development area should be considered to extend five feet beyond the limits of the roadway prism. We anticipate deeper stripping depth will be required where mature trees are removed or were previously removed. All roots larger than 0.5-inch diameter should be removed within the development areas.

After removal of unsuitable materials as noted above, the subgrade in pavement areas should be proof-rolled with a loaded 10-cubic-yard-dump truck, or full 4,000-gallon water truck, to confirm subgrade stability prior to placing new fill, where accessible. Any deflection observed during proof-rolls should be addressed. If unstable (“pumping”) soil is observed in isolated areas, remedial measures may consist of further compaction, including moisture-conditioning (aeration), or over-excavation and replacement with granular, structural fill. Pumping-soil

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conditions are more common when site preparation occurs during spring and after periods of prolonged precipitation. Wallace Group should be consulted to confirm stripping depths and observe proof-rolls and subgrade-bearing conditions prior to placing new structural fill.

6.1.2 Structural Fill Materials and Placement

Where structural fill is needed to achieve design subgrade elevations, we recommend the structural fill in paved areas consist of density-testable 2-inch minus processed on-site native soil or imported sand and gravel meeting the requirements of ODOT Section 02630.10 and 00510.13.

6.2 FOUNDATIONS

Based on conversations with Parametrix, we anticipate the bridge foundation will consist of pre-drilled HP piles embedded into basalt bedrock. Once the pile locations are drilled and the steel HP piles are installed, we anticipate the annular space will be filled with non-shrink grout. We understand that seven (7) HP 14x89 piles with a pile cap are anticipated for each bridge abutment. Expected axial loads range from a Strength 1 Load of 204 kips per pile, to a Service Load of 140 kips per pile. Lateral loads are anticipated to range from approximately 16 to 23.3 kips per pile at the pile head. Based on our understanding of the structural design, we expect that the maximum allowable pile head deflection is 3/8 inch. We assume that the piles will be designed with a free-head condition, with no moment at the pile head.

6.2.1 Deep Foundations

We anticipate that pre-drilled HP-pile foundations are the most appropriate foundation system based on the expected loads and subsurface conditions. The primary consideration related to the construction of predrilled piles is the overlying soil which contains boulders and cobbles and the variability in basalt bedrock strength, fractured zones, and open voids. Embedment depth of the basalt rock socket is an iterative process based on the diameter of the shaft. Open voids, cinder layers, and rubblized zones were encountered during our exploration at depths below approximately 25-feet bgs; however, given the depth to these zones and our anticipated rock embedment length, we do not anticipate significant impacts to development. We expect that the overburden soil may require casing to prevent sloughing into the drilled shaft. Because the NUID canal is concrete-lined, we do not anticipate significant effects to the bridge foundation from scouring.

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6.2.2 Axial Capacity

Due to the inability to easily clean and inspect the bottoms of drilled shafts and the large deformation necessary to mobilize end bearing, the pre-drilled piles should be designed to derive their axial capacity from the skin friction in competent basalt bedrock, assumed below elevation 3,403 feet. Side friction in the overlying soil should be ignored. Based on the AASHTO LRFD Bridge Design Specification, 9th Edition, we recommend an axial capacity using side friction with a Service Limit resistance of 11 kips per square foot (ksf) in basalt bedrock, this includes a resistance factor equal to 1. We recommend a Strength Limit resistance of 6.3 ksf in basalt bedrock, this includes a resistance factor equal to 0.55. For temporary uplift loads, we recommend a nominal factored side friction of 4.4 ksf in basalt bedrock.

We recommend the predrilled pile shaft be the minimum diameter larger than the pile to allow for non-shrink grout flow and achieve the recommended cover clearance. We recommend that predrilled piles are constructed with a rock socket of at least 8-feet into competent basalt. The depth to and competency of the basalt should be confirmed during drilling by a Wallace Group representative.

The settlement of properly constructed predrilled piles designed based on the recommendations above is expected to be less than ½ inch. We understand that the piles will be spaced approximately 6.5-feet apart, center-to-center. At this time, the actual diameter of the predrilled hole is not known; however, we recommend that the center-to-center spacing remain at least six pile shaft diameters to avoid service reductions due to group effects. Wallace Group should confirm the axial and lateral capacities and settlement estimates after final loads, dimensions, final pile locations, and depths are selected.

6.2.3 Lateral Resistance

HP piles will provide lateral resistance from passive pressure acting on the upper portion of the piles and from their structural rigidity. Lateral resistance of piles will depend on the pile diameter, pile head condition (restrained or unrestrained), allowable deflection of the pile top, and the bending moment resistance of the piles. Values used in our LPile analysis are presented in **Table 6.2.3**, below.

Table 6.2.3
HP Pile Lateral Design Values

Soil or Bedrock Type	Lpile Soil or Rock Type	Assumed Depth (Top /Bottom of Layer)	Elev. (feet, msl)	Friction Angle (degree)	Soil Modulus (k) (pci)	Uniaxial Compressive Strength (psi)	Hoek-Brown Material Index (m_i)	Intact Rock Modulus (psi)	Geologic Strength Index (GSI)	Poisson Ratio
Fill & Silty-Sand	Sand	0	3,418	35	90	---	---	---	---	---
		13	3,405	39	90	---	---	---	---	---
Basalt Bedrock	Massive Rock	13	3,405	---	---	4,880	25	1×10^6	0.55	0.2
		42	3,376	---	---	8,140	25	1×10^6	0.55	0.2

The lateral resistances tabulated in **Tables 6.2.3** are for isolated piles and piles in a group with a spacing of at least six pile shaft diameters. If piles are installed in a group of two with a spacing of three pile shaft diameters, we recommend reducing the lateral capacities by 15 percent. However, the design bending moments should not be reduced; they should be the same as those for single piles. If larger pile groups are needed to support the design loads, we should provide the reduction factors for these groups. The LPile model assumes level ground near the pile head. If sloping conditions are present, Wallace Group should revise the LPile model. Lateral pile deflection and moment curves are presented on **Figures 3, 4, and 5**. Structural capacity of predrilled piles should be confirmed by a structural engineer.

6.2.4 Construction Considerations

Predrilled piles should be installed by a qualified contractor with demonstrated experience in this type of foundation. Boulders, cobbles, and potentially caving soil and fractured rock may be encountered during drilling. Therefore, casing and/or drilling fluid may be required. Wallace Group should be present on a full-time basis during drilling to confirm the quality of the basalt bedrock and that sufficient embedment is achieved. To achieve optimum skin friction, we recommend that the basalt bedrock is washed with water to remove rock flour accumulated on the side walls from the drilling activities.

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We understand that a non-metallic, non-shrink, grout with a compressive strength of 4,000 pounds per square inch (psi) will be specified in the project plans. Non-shrink grout placement should start upon completion of the drilling and clean out. Non-shrink grout should be placed from the bottom up in a single operation using a tremie and/or a pumper pipe. The tremie pipe should be maintained at least 5 feet below the upper surface of the grout during casting of the piles. As the grout is placed, casing used to stabilize the hole can be withdrawn. The bottom of the casing should be maintained at least 3 feet below the surface of the grout.

6.2.5 Lateral Earth Pressure

We understand that earth retaining walls will be constructed for the bridge abutment wing walls. Lateral pressures on earth-retaining walls depend upon the type of wall, hydrostatic pressure behind the wall, type of backfill material and method, and allowable wall movement. Where allowable wall movement is less than 1/2-percent of the wall height or where wall movement is constrained, lateral earth pressures should be estimated for an “at rest” condition. Where allowable wall movement is greater than 1/2-percent of the wall height, lateral earth pressures may be estimated for an “active” condition. In general, walls that are attached to a structure or braced should be designed for the “at-rest” condition, and unattached, un-braced walls may be designed for the “active” condition. Where walls are backfilled and compacted, the compaction effort can induce somewhat higher lateral earth pressures than “at rest.” Backfill compaction within three feet of the wall face should be conducted with manually operated “jumping jack” and vibratory plate compactors to reduce the potential for high lateral earth pressures.

We recommend an allowable coefficient of sliding resistance between the retaining wall foundation and underlying soil of 0.33. We recommend that a factor of safety against sliding of at least 1.5 is used for design. Sliding friction can be increased by 1/3 for temporary loads, such as wind and seismic. We recommend all active lateral loads on retaining walls be resisted by friction only, where possible, because of the relatively large deformations required to mobilize full passive earth pressure.

Retaining walls backfilled and compacted with structural fill should be designed for an equivalent fluid lateral earth pressure of 58 pcf for “at-rest” conditions. If earth retaining walls are free to rotate at the top, we recommend an “active” earth pressure of 37 pcf be used for design. These lateral pressures are for level backfill and do not include surcharge loads or hydrostatic pressure. Surcharge loads should be included in design where vehicle loads or foundations will be present within a lateral distance equal to the height of the wall. For seismic

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design, we recommend a triangular pressure distribution of 20 pcf be applied to walls greater than 8-feet-tall.

Any additional lateral earth pressures from surcharges or slopes need to be adequately resisted. Wallace Group should provide supplemental recommendations if surcharge loading, such as adjacent foundations or vehicles, will be present within a lateral distance equal to the height of the wall.

6.3 PAVEMENT RECOMMENDATIONS

6.3.1 Hamehook Road

We anticipate that traffic will likely consist of light automotive and occasional moderate duty trucks. Traffic during construction will consist of heavier vehicles with higher wheel loads and precautions should be taken to prevent damage to any newly constructed pavement.

The proof-rolled, inorganic-native-granular soil, and properly compacted new structural fill will provide, in our opinion, adequate subgrade support for asphalt-paved-roadways associated with the development. Proper roadway section drainage, including site drainage to avoid ponding of water adjacent to roadway areas, will aid in reducing the potential for pavement distress. Structural fill in paved areas should consist of density-testable 2-inch minus processed on-site native soil or imported sand and gravel meeting the requirements of ODOT Section 02630.10 and 00510.13.

Based on the project soil conditions, Deschutes County Road Department Specifications, and assumed traffic loads for the Hamehook Road improvements, we recommend a pavement section of 3 inches of Asphalt Concrete (AC) underlain by 8 inches of 'Class B' backfill for the public Right-of-Way (ROW) improvements. AC in the public roadways should meet the ODOT 2021 Standard Specifications for Construction.

6.3.2 Pavement Material Specifications

The AC should be dense-graded, hot mix asphalt concrete (HMAC) as specified in ODOT Section 00745 plus the following supplemental specifications for density testing:

- The HMAC mix design for the roadways should be Level 3.
- The asphalt binder should be PG 64-28, or as specified by the Civil Engineer.
- The ABC should be ½-inch minus, dense graded aggregate as specified in ODOT Sections 00641 and 02630.10.

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- Road-mixed ABC is permitted per Section 00641. Road-mixed ABC allows water to be added on-site for compaction vs. pug-milled materials processing.
- The HMAC should be compacted to a minimum of 92 percent of the Rice theoretical maximum density. The ABC should be compacted to a minimum of 92 percent of ASTM D1557.

Supplemental Specifications for Density Testing: The roadway AC and ABC should be field tested for in-place density. Density test frequency should be based on a “roll-pattern” or standard Deschutes County procedures.

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7.0 ADDITIONAL SERVICES

7.1 DESIGN AND CONSTRUCTION PERIOD ENGINEERING SERVICES

Wallace Group should perform a review the geotechnical and civil aspects of the project design plans and specifications, when completed, to confirm that our recommendations are incorporated into the project documents, and to make appropriate modifications, if necessary. We currently anticipate performing design review of foundation and grading plans to document that our recommendations are incorporated into design. This review will reduce misinterpretation of our recommendations and reduce the potential for costly design changes and construction delays.

7.2 CONSTRUCTION INSPECTION AND TESTING

To maintain our role as the geotechnical engineer of record, Wallace Group will also observe and monitor earthwork construction including site preparation, placement and compaction of engineered fill. The purpose of these services will be to help document that site grading and development-area-subgrade preparation complies with the recommendations of this geotechnical report and the approved project plans and specifications. If subsurface conditions are encountered during construction that differ from the conditions described herein, we will review our recommendations considering these different conditions and recommend changes in design or construction procedures.

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8.0 LIMITATIONS

Exploratory borings performed for this study were placed to obtain an understanding of underground conditions for evaluation and design purposes. The exploration was performed using a mutually-agreed-upon scope of services. Variations from these conditions, not indicated by the borings, are possible. These variations are sometimes enough to necessitate design modifications. The Client must recognize that it is impossible to predict every physical condition that will be encountered. If unexpected conditions are observed during construction, or if the size, type, elevation, or location of the proposed structure should differ significantly from the conceptual design, we should be notified to review the recommendations contained in this report. The professional judgments expressed in this report meet the standard of care of our profession; however, no warranty is expressed or implied.

This report may be used only by the Client and only for the intended site and for the purposes stated within a reasonable time from its issuance, but in no event, later than three (3) years from the date of the report. Land or facility use, on- and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Any party other than the Client or their design team who wishes to use this report shall notify the Wallace Group of such intended use. Based on the intended use of the report, the Wallace Group may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Wallace Group from any liability resulting from the use of this report by any unauthorized party.

The contractor selected for this project is responsible for supervision and direction of the actual work performed by his employees, subcontractors, and agents. Wallace Group will use accepted geotechnical engineering and testing procedures; however, our testing and observations will not relieve the contractor of his primary responsibility to produce a completed project conforming to the project plans and specifications.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor’s operations, and we cannot be responsible for the safety of personnel other than our own on the site. The safety of others is the responsibility of the contractor. The contractor should notify the owner if they consider any of the recommended actions presented herein unsafe.

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10.0 PROFESSIONAL AUTHENTICITY

This report has been authored and reviewed by the undersigned, respectively. This report is void if the original seal(s) and signature(s) are not included.

Digitally signed by Adam Larson
Date: 2024.01.19 09:34:51-08'00'

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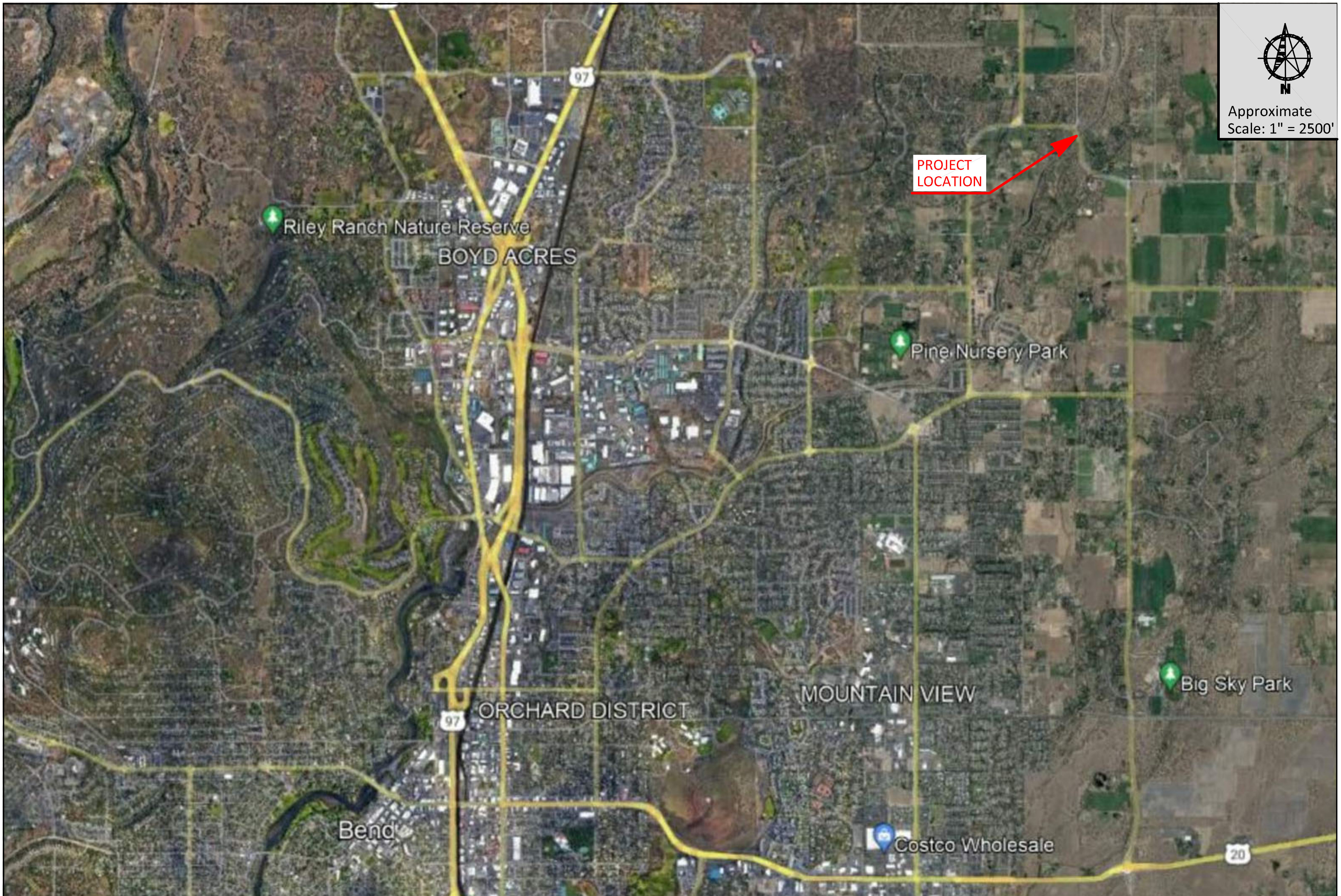
FIGURES



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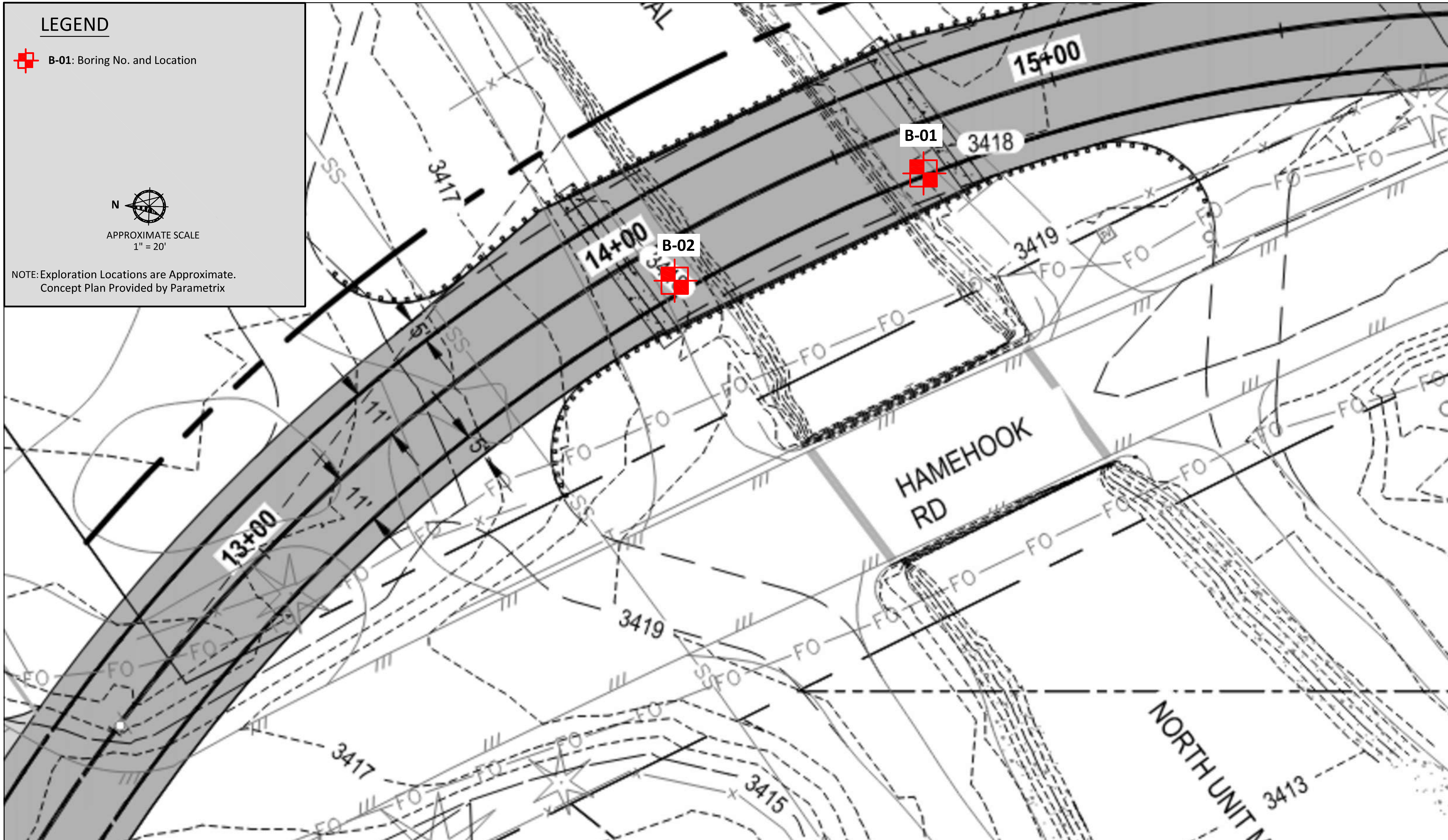
VICINITY MAP
HAMEHOOK ROAD BRIDGE
BEND, OREGON

PROJECT No:	23106 (1)
DRAWN:	October 25, 2023
DRAWN BY:	KAK
CHECKED BY:	LMS
FILE NAME:	23106 (1) Figure 1

FIGURE
1



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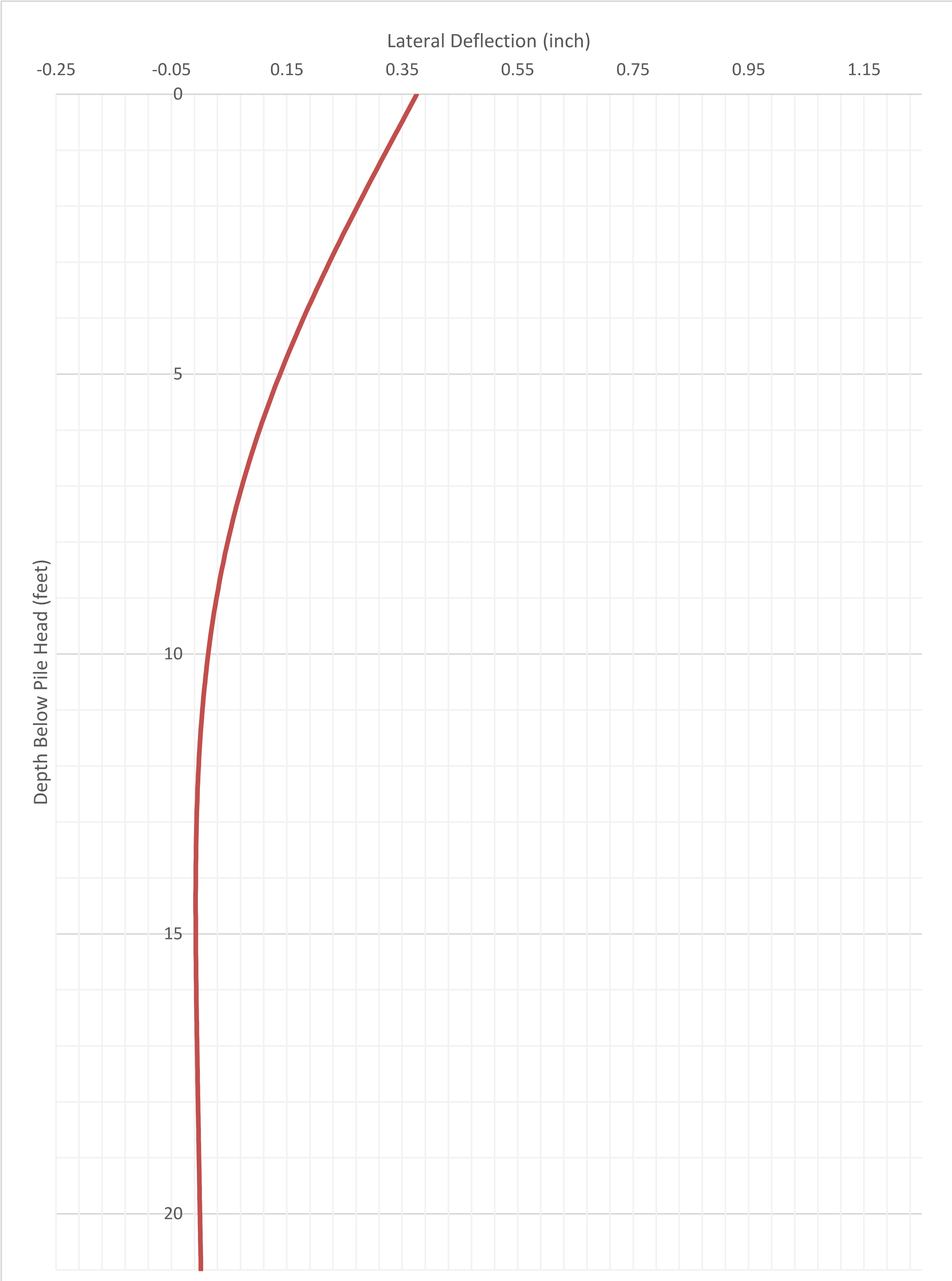
EXPLORATION LOCATION MAP
HAMEHOOK ROAD BRIDGE
BEND, OREGON

PROJECT No:	23106 (1)
DRAWN:	November 28, 2023
DRAWN BY:	KAK
CHECKED BY:	AML
FILE NAME:	23106 (1) Figure 2

FIGURE: 2



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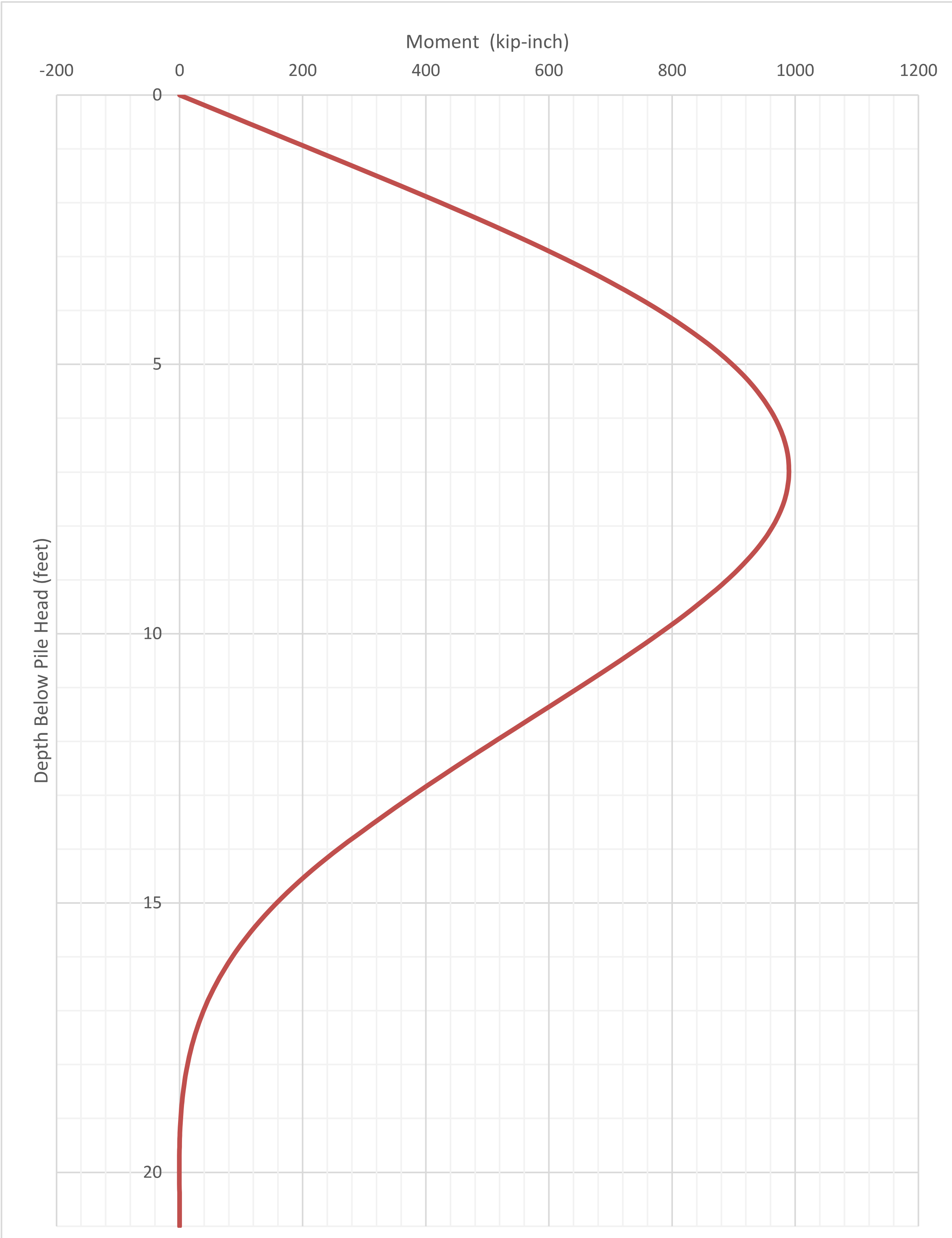
Depth vs. Lateral Deflection
Hamehook Road Bridge
Bend, OR

PROJECT NO. 23106-1

FIGURE 3



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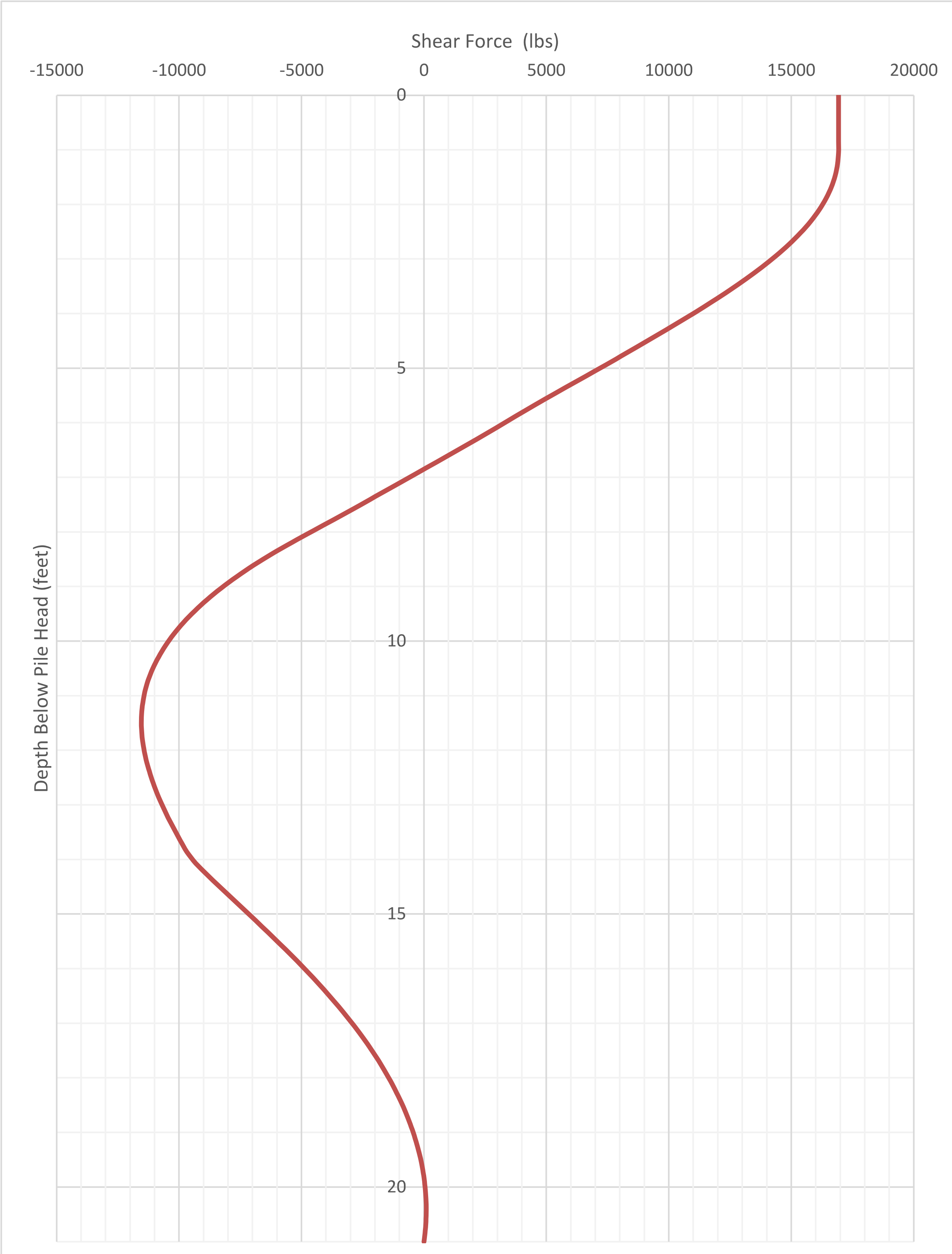
Depth vs. Moment
Hamehook Road Bridge
Bend, OR

PROJECT NO. 23106-1	**FIGURE 4**



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Depth vs. Shear Force
Hamehook Road Bridge
Bend, OR

PROJECT NO. 23106-1	**FIGURE 5**

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APPENDIX A

APPENDIX A
FIELD EXPLORATION SUMMARY

GENERAL

Subsurface conditions for the Hamehook Road Bridge project, located in Bend, Oregon were explored by drilling two (2) borings (designated B-01 and B-02), at locations shown on **Figure 2**, *Exploration Location Map*. Geotechnical boring logs are included in this appendix. The exploration program was completed from October 23 to 24, 2023. The procedures used to drill the borings and collect soil samples, and other field techniques are described in detail in this appendix. Unless otherwise noted, all soil sampling and classification procedures followed local engineering practices, which are in general conformance with relevant ASTM procedures and the Unified Soil Classification System (USCS). “General conformance” means that certain local and common drilling and descriptive practices and methodologies have been followed.

BORINGS

Two (2) geotechnical borings were drilled near the proposed bridge foundations by Western States Soil Conservation of Hubbard, Oregon. The borings were drilled to depths of up to approximately 42-feet below ground surface (bgs) with a Central Mining Equipment (CME) 850 truck-mounted drilling rig. Borings were advanced using hollow stem auger and HQ-wireline coring drilling techniques. Borings were abandoned in accordance with Bureau of Reclamation requirements.

SAMPLING

Disturbed soil samples were retrieved from the borings using Standard Penetration Testing (SPT). The samples were classified and sealed in plastic bags for further examination and physical testing in our laboratory for gradation and moisture content. Samples of rock were collected using rock coring techniques.

MATERIAL DESCRIPTIONS

Soil samples were visually classified in the field as they were collected. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the samples were noted. Afterwards, the samples were re-examined in the laboratory, various standard classification tests were conducted, and the field classifications were modified where necessary. The terminology used in the soil classifications and rock descriptions are defined beginning on Page 3 and are included under the material description on each log.

BORING LOGS

Figure A is a Legend explaining the information and symbols presented on the boring logs. The logs of the borings are presented on Figures A-1 through A-2.2. The logs describe the materials encountered and the depths where materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. On each boring log, the types of samples collected (including their identification number) are reported, including laboratory test results, and SPT blow counts. Corrected SPT blowcounts, presented on the boring logs, have been corrected for sampling type and hammer energy.

GROUNDWATER

Groundwater was not encountered during drilling activities and the adjacent canal was empty. It is unknown if there is potential for water when the canal is full.

TERMINOLOGY USED TO DESCRIBE SOIL AND ROCK

Soils exist in mixtures with varying proportions of components. The predominant soil, i.e., greater than 50 percent based upon total dry weight, is the primary soil type and is capitalized in our log descriptions, e.g., SAND, GRAVEL, SILT or CLAY. Lesser percentages of other constituents in the soil mixture are indicated by use of modifier words in general accordance with the Visual-Manual Procedure (ASTM D2488-93). "General Accordance" means that certain local and common descriptive practices have been followed. In accordance with ASTM D2488, group symbols (such as GP or CH) are applied on that portion of the soil passing the 3-inch (75mm) sieve based upon visual examination. The following describes the use of soil names and modifying terms used to describe fine- and coarse-grained soils.

Fine - Grained SOILS (More than 50% fines passing 0.074 mm, #200 sieve)

The primary soil type i.e. SILT or CLAY is designated through visual – manual procedures to evaluate soil toughness, dilatancy, dry strength, and plasticity. The following describes the terminology used to describe fine - grained soils and varies from ASTM 2488 terminology in the use of some common terms.

Primary soil NAME, adjective and symbols			**Plasticity Description**	**Plasticity Index (PI)**
SILT ML & MH	CLAY CL & CH	ORGANIC SILT & CLAY OL & OH		
SILT		Organic SILT	Non-plastic	0 - 3
SILT		Organic SILT	Low plasticity	4 - 10
Clayey SILT	Silty CLAY	Organic clayey SILT	Medium Plasticity	>10 – 20
Clayey SILT	CLAY	Organic silty CLAY	High Plasticity	>20 – 40
Clayey SILT	CLAY	Organic CLAY	Very Plastic	>40

Modifying terms describing secondary constituents, estimated to 5 percent increments, are applied as follows:

Description	**% Composition**
Trace sand, trace gravel	5% - 10%
With sand; with gravel	15% - 25%
Sandy, or gravelly	30% - 45%

Borderline Symbols, for example CH/MH, are used where soils are not distinctly in one category or where variable soil units contain more than one soil type. Dual Symbols, for example CL-ML, are used where two symbols are required in accordance with ASTM D2488.

Soil Consistency. Consistency terms are applied to fine-grained, plastic soils (i.e., PI > 4). Descriptive terms are based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84, as follows.

Consistency Term	SPT N-value	Unconfined Compressive Strength Tons/sq.ft.	kPa
Very soft	Less than 2	Less than 0.25	Less than 24
Soft	2 - 4	0.25 - 0.5	24 - 48
Medium stiff	5 - 8	0.5 - 1.0	48 – 96
Stiff	9 - 15	1.0 - 2.0	96 – 192
Very stiff	16 - 30	2.0 - 4.0	192 – 383
Hard	Over 30	Over 4.0	Over 383

Note: For SILT with low to non-plastic behavior, (i.e., PI < 4) a relative density description is applied.

Coarse-Grained Soils (less than 50% fines)

Coarse-grained soil descriptions, i.e., SAND or GRAVEL, are based on that portion of materials passing a 3-inch (75mm) sieve. Coarse-grained soil group symbols are applied in accordance with ASTM D2488 based upon the degree of grading, or distribution of grain sizes of the soil. For example, well graded sand containing a wide range of grain sizes is designated SW; poorly graded gravel, GP, contains high percentages of only certain grain sizes. Terms applied to grain sizes follow.

	Particle Diameter Inches	Millimeters
Sand (S)	0.003 - 0.19	0.075 - 4.8
Gravel (G)	0.19 - 3.0	4.8 - 75
	Additional Constituents	
Cobble	3.0 - 12	75 - 300
Boulder	12 - 120	300 - 3050
Rock Block	>120	>3050

The primary soil type is capitalized, and the amount of 'fines' in the soil are described as indicated by the following examples. Other soil mixtures will provide similar descriptive names.

Example: Coarse-Grained Soil Descriptions with Fines

5% fines	10% fines (Dual Symbols)	15% to 45% fines
GRAVEL with trace silt: GW or GP	GRAVEL with silt, GW-GM	Silty GRAVEL: GM
SAND with trace clay: SW or SP	SAND with clay, SP-SC	Silty SAND: SM

Additional descriptive terminology applied to coarse-grained soils follow.

Coarse-Grained Soil Containing Secondary Constituents

Clean	< 5% fines
With sand or with gravel	15% - 25% sand or gravel
Sandy or gravelly	30% - 45% sand or gravel
With cobbles; with boulders	Any amount cobbles or boulders. Additional terms may be used to describe amount including abundant, scattered.

Cobble and boulder deposits may include a description of the matrix soils, as defined above.

Relative Density terms are applied to granular, non-plastic soils based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586.

Relative Density Term	***SPT N-value***
Very loose	0 - 4
Loose	4 - 10
Medium dense	10 - 30
Dense	30 - 50
Very dense	> 50

Terminology Used to Describe Rock

Scale of Rock Strength

Description	Designation	Unconfined Compressive Strength, psi	Unconfined Compressive Strength, MP	Field Identification
Very low strength	R1	100 – 1000	0.7 – 7	Crumbles under firm blows with point of geology pick; can be peeled by a pocketknife.
Low strength	R2	1,000 – 4,000	7 – 28	Can be peeled by a pocketknife with difficulty; shallow indentation made by firm blows of geology pick.
Moderate strength	R3	4,000 – 8,000	28 – 55	Cannot by scraped or peeled with a pocketknife; specimen can be fractured with a single firm blow of geology hammer.
Medium high strength	R4	8,000 – 16,000	55 – 110	Specimen requires more than one blow with a geology hammer to fracture it.
High strength	R5	16,000 – 32,000	110 – 120	Specimen requires many blows of geology hammer to fracture it.
Very high strength	R6	> 32,000	> 220	Specimen can only be chipped with geology pick.

Descriptive Terminology for Joint Spacing or Bedding

Descriptive Term	*Spacing of Joints*	
Very close	Less than 2 inches	< 50 mm
Close	2 inches - 1 foot	50 mm – 300 mm
Moderately close	1 foot - 3 feet	300 mm – 1 m
Wide	3 feet -10 feet	1 m – 3 m
Very wide	Greater than 10 feet	> 3 m

Descriptive Terminology for Vesicularity

Descriptive Term	*Percent voids by volume*
Dense	< 1%
Slightly vesicular	1 – 10%
Moderately vesicular	10 – 30%
Highly vesicular	30 – 50%
Scoriaceous	> 50%

Correlation of RQD and Rock Quality

Rock Quality Descriptor	RQD Value
Very poor	0 – 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90

SCALE OF ROCK WEATHERING

Stage	*Description*	*Quality Distinction*
Fresh	Rock is fresh, crystals are bright, a few joints may show slight staining because of ground water.	Discoloration
Very Slight	Rock is generally fresh, joints are stained, some joints may have thin clay coatings, crystals in broken faces show bright.	Discoloration only on major discontinuity surfaces [i]
Slight	Rock is generally fresh, joints are stained, and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some feldspar crystals are dull and discolored. Rocks ring under hammer if crystalline.	Discoloration on all discontinuity surfaces and on rock
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.	Decomposition and/or disintegration < 50% of rock [ii]
Moderately Severe	All rock, except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.	Decomposition and/or disintegration > 50%, but not complete
Severe	All rock, except quartz, discolored or stained. Rock "fabric" is clear and evident but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of harder rock usually left, such as corestones in basalt.	Decomposition and/or disintegration > 75%, nearly complete
Very Severe	All rock, except quartz, discolored or stained. Rock "fabric" is discernible, but mass effectively reduced to "soil" with only fragments of harder rock remaining.	Decomposition and/or disintegration 100% with structure/fabric intact
Complete	Rock is reduced to "soil". Rock "fabric" is not discernible, or only in small scattered locations. Quartz may be present as dikes or stringers.	Decomposition and/or disintegration 100% with structure/fabric destroyed

NOTES: [i] Discontinuities consist of any natural break (joint, fracture or fault) or plane of weakness (shear or gouge zone, bedding plane) in a rock mass

[ii] Decomposition refers to chemical alteration of mineral grains; disintegration refers to mechanical breakdown

[iii] Stage and description from ASCE Manual No. 56 (1976), quality distinction from Murray (1981)

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NORTHWEST GEOSYSTEM EXPERTS

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KEY TO SYMBOLS

CLIENT Parametrix
PROJECT NAME Hamehook Road Bridge
PROJECT NUMBER 23106-1
PROJECT LOCATION Bend, OR

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

BASALT: Basalt

FILL: Fill (artificial fill)

SM: USCS Silty Sand

SAMPLER SYMBOLS

Rock Core (RC)

Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
MC - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
FINES - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)
OC - ORGANIC CONTENT (%)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UCCS- UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
Water Level at Time of Drilling, or as Shown
Water Level at End of Drilling, or as Shown
Water Level After 24 Hours, or as Shown

Figure: A

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Wallace Group Inc.
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Bend, OR 97701

BORING NUMBER B-01

CLIENT Parametrix
PROJECT NAME Hamehook Road Bridge
PROJECT NUMBER 23106-1
PROJECT LOCATION Bend, OR
DATE STARTED 10/24/23 **COMPLETED** 10/24/23
GROUND ELEVATION 3418 ft
DRILLING CONTRACTOR Western States Soil Conservation Inc.
GROUND WATER LEVELS:
DRILLING METHOD CME 850
AT TIME OF DRILLING ---
LOGGED BY KAK **CHECKED BY** LMS
24HRS AFTER DRILLING ---
NOTES

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS	Corrected N-Value	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Approximate Elevation
0	SPT S-1		7 12 13	30	FC: 26% MC: 5.8%	SM		SILTY SAND, medium dense, slightly moist, brown, fine to coarse grained, subangular to subrounded, [FILL]	
	SPT S-2		5 5 4	11	FC: 22% MC: 8.5%				
5								5.0	3413.0
	SPT S-3		3 3 3	7	FC: 19% MC: 16.2%	SM		SILTY SAND, gravel, very loose, very moist, brown, fine to coarse grained, subangular to subrounded	
	SPT S-4		4 8 32	48					
10	SPT S-5		3 1 2	3	FC: 21% MC: 24.7%				
								12.0	3406.0
	SPT S-6		50/1"	60/1"				BASALT, unweathered to highly weathered, moderately fractured, flow banded, gray, very low strength to medium high strength, vesicular to massive, very poor to good rock quality	
15	RC C-1	100 (57)			UCCS: 4880 psi				
20	RC C-2	95 (58)							
25	RC C-3	80 (17)			25 - 26' soil interbed reddish brown fluid returns				
30	RC C-4	20 (0)							

Figure: A - 1.1

Wallace Group Inc.
62915 NE 18th Street, Suite 1
Bend, OR 97701

BORING NUMBER B-01

CLIENT Parametrix **PROJECT NAME** Hamehook Road Bridge

PROJECT NUMBER 23106-1 **PROJECT LOCATION** Bend, OR

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS	Corrected N-Value	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Approximate Elevation
30					31.5 - 34' fast advance rubblized basalt			BASALT, unweathered to highly weathered, moderately fractured, flow banded, gray, very low strength to medium high strength, vesicular to massive, very poor to good rock quality *(continued)*	
35	RC C-5	40 (8)			35 - 35.5' void very fast advance				
40	RC C-6	97 (78)							
							41.5		3376.5

Bottom of borehole at 41.5 feet.

Figure: A - 1.2

BORING NUMBER B-02

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CLIENT Parametrix
PROJECT NAME Hamehook Road Bridge
PROJECT NUMBER 23106-1
PROJECT LOCATION Bend, OR
DATE STARTED 10/23/23 **COMPLETED** 10/23/23
GROUND ELEVATION 3418 ft
DRILLING CONTRACTOR Western States Soil Conservation Inc.
GROUND WATER LEVELS:
DRILLING METHOD CME 850
AT TIME OF DRILLING ---
LOGGED BY KAK **CHECKED BY** LMS
24HRS AFTER DRILLING ---
NOTES

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS	Corrected N-Value	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Approximate Elevation
0	SPT S-1		10 14 13	32	-	GM		0.5 SILTY GRAVEL WITH SAND, dry, gray, fine to coarse grained, angular to well rounded, [FILL]	3417.5
	SPT S-2		7 9 9	22		SM		SILTY SAND, medium dense, moist to slightly moist, brown, fine to coarse grained, subangular to subrounded, [FILL]	
5	SPT S-3		7 6 7	16	Phi: 38.9° C: 50 psf	SM		5.0 SILTY SAND, gravel, medium dense, moist, brown, fine to coarse grained, subangular to subrounded	3413.0
	SPT S-4		9 13 9	26					
10	SPT S-5		3 3 10	16					
								13.0 BASALT, unweathered to moderately weathered, moderately fractured, flow banded, gray, low strength to high strength, vesicular to massive, very poor to good rock quality	3405.0
15	SPT S-6		50/1"	60/1"					
	RC C-1	71 (58)			UCCS: 8140 psi				
20	RC C-2	100 (67)							
25	RC C-3	82 (70)							
	RC C-4	100 (87)							
30									

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BORING NUMBER B-02

CLIENT Parametrix

PROJECT NAME Hamehook Road Bridge

PROJECT NUMBER 23106-1

PROJECT LOCATION Bend, OR

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS	Corrected N-Value	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Approximate Elevation
30	RC C-5	0 (0)			29.5 - 32' void no recovery very fast advancement			BASALT, unweathered to moderately weathered, moderately fractured, flow banded, gray, low strength to high strength, vesicular to massive, very poor to good rock quality *(continued)*	
35	RC C-6	20 (0)			32 - 36' very rubblized basalt low recovery				
40	RC C-7	100 (78)							
								42.0	3376.0

Bottom of borehole at 42.0 feet.

TWG-BORING LOGS - WALLACE GROUP DATA TEMPLATE.GDT - 1/12/24 10:47 - W:\GINT PRO - FILES\BENTLEY\GINT\PROJECTS\23106-1 HAMEHOOK BRIDGE REPLACEMENT.GPJ

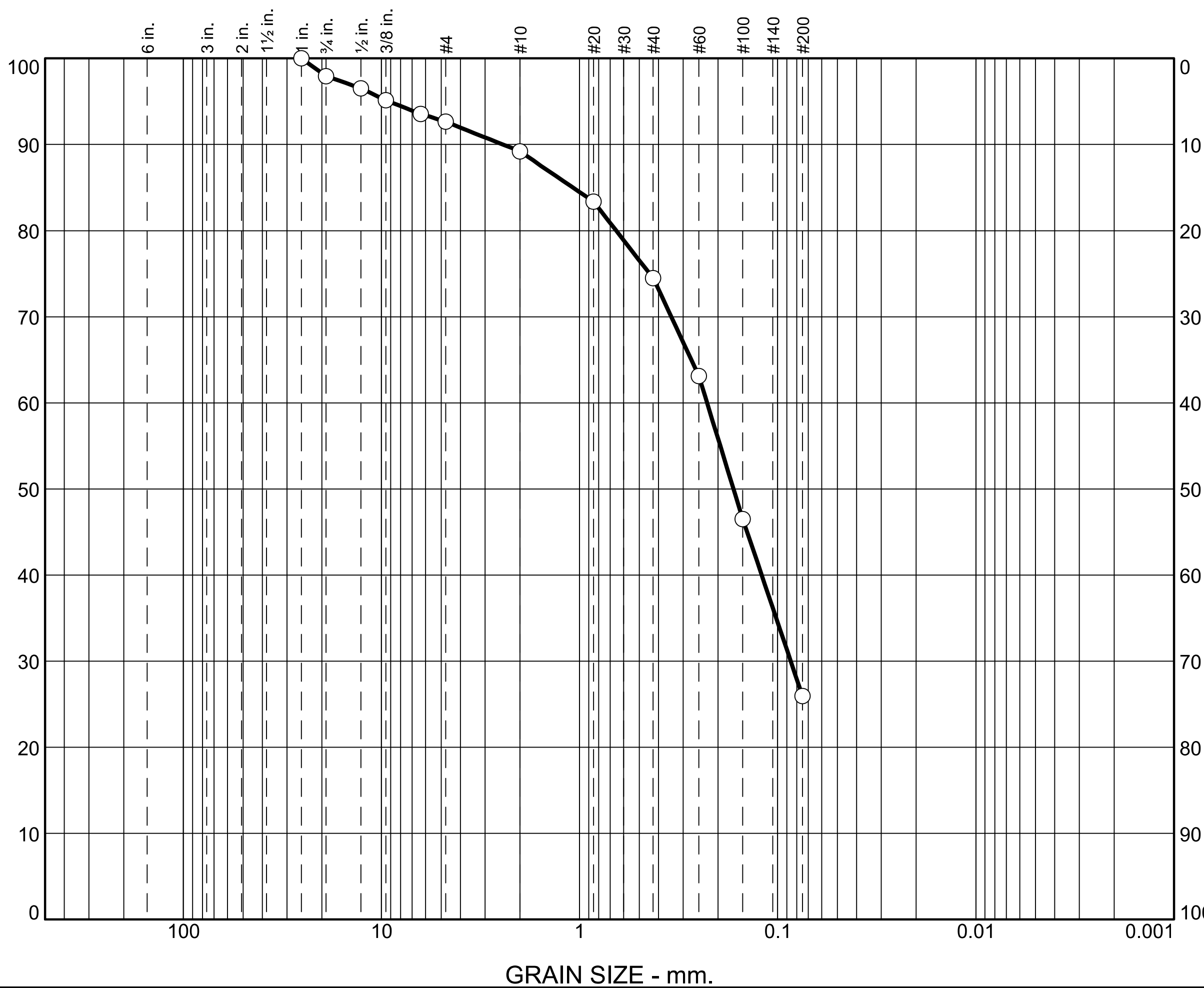
Figure: A - 2.2

WALLACE GROUP

NORTHWEST GEOSYSTEM EXPERTS

APPENDIX B

Sieve Analysis (C117/C136)



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	2	5	4	15	48	26	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	Not Tested	Not Tested	1.0804	0.2273	0.1671	0.0860				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ Silty Sand	10-30-23	SM	5.8%

Project No. 23106-1 **Client:** Parametrix

Project: Hamehook Bridge Replacement

○ **Source of Sample:** B-01 **Depth:** 0' **Sample Number:** S-1

Remarks:
○ Lab No.: WGG0336
Sampled By: KAK on 10-23-23
C117 Wash Procedure A

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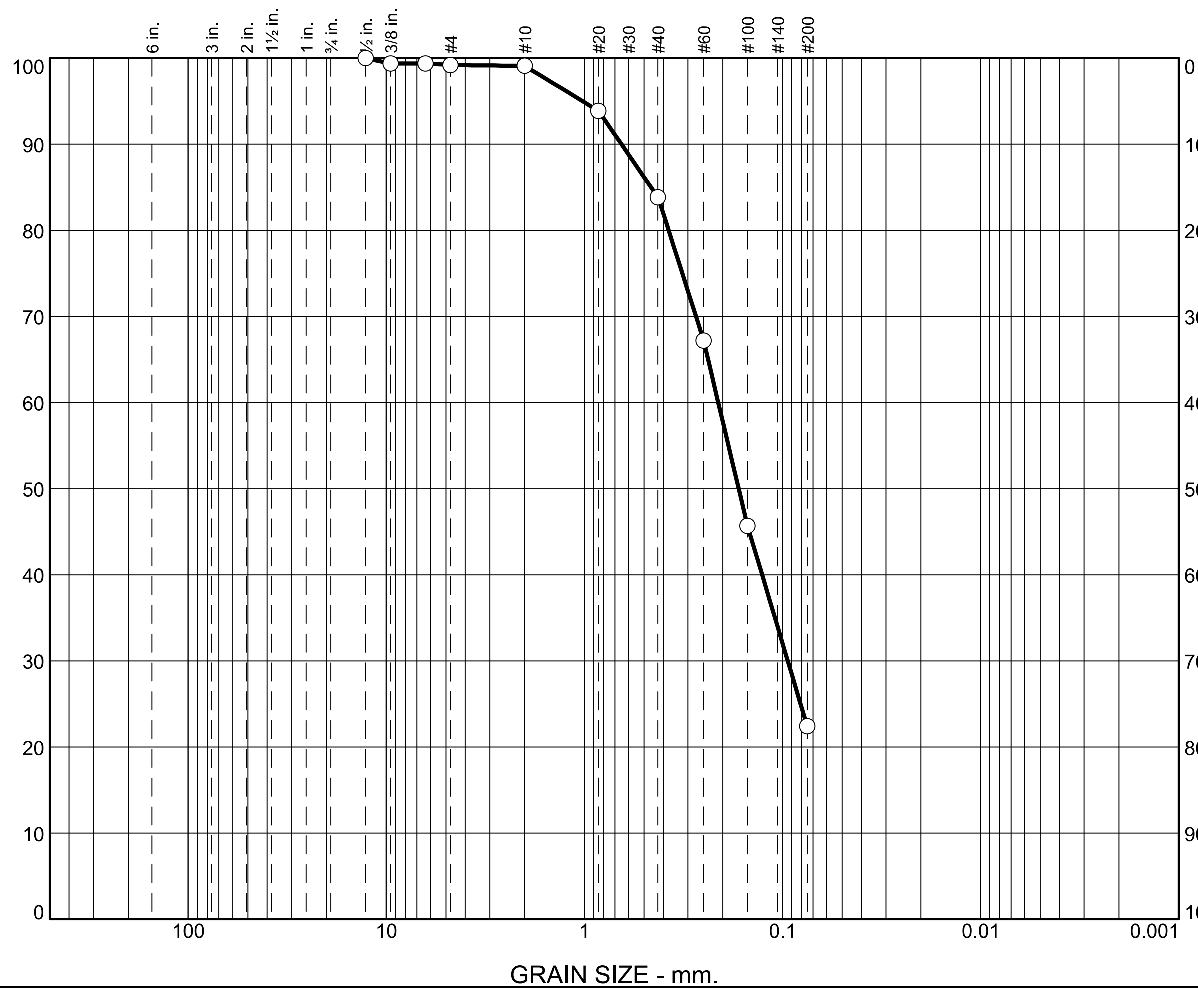
Figure B-1

Tested By: PJH **Checked By:** AML

Sieve Analysis (C117/C136)



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	1	0	15	62	22	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	Not Tested	Not Tested	0.4600	0.2108	0.1662	0.0940				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ Silty Sand	10-30-23	SM	8.5%

Project No. 23106-1 **Client:** Parametrix

Project: Hamehook Bridge Replacement

○ **Source of Sample:** B-01 **Depth:** 2.5' **Sample Number:** S-2

Remarks:

○ Lab No.: WGG0336
Sampled By: KAK on 10-23-23
C117 Wash Procedure A

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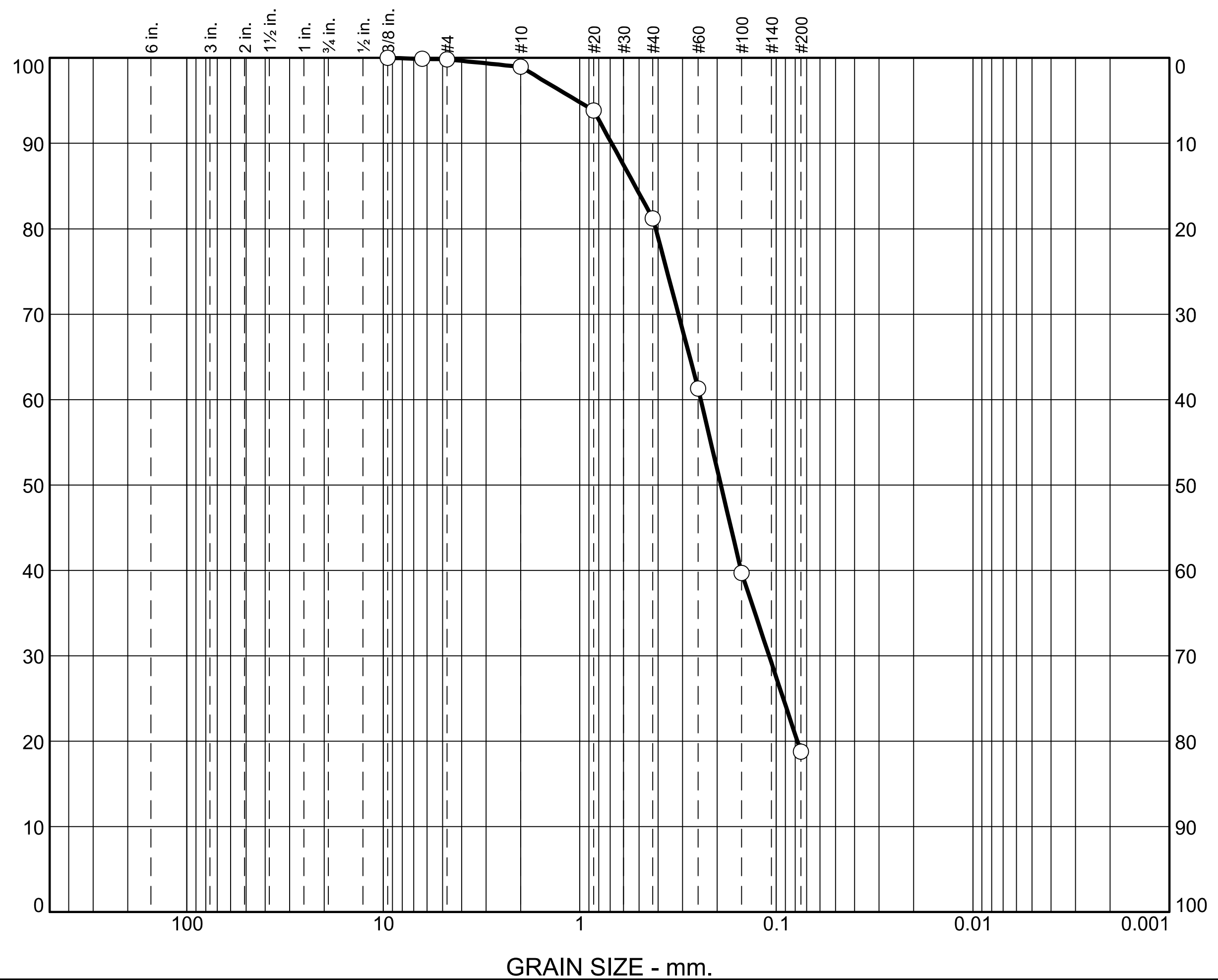
Figure B-2

Tested By: PJH **Checked By:** AML

Sieve Analysis (C117/C136)



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	0	1	18	62	19	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	Not Tested	Not Tested	0.5231	0.2425	0.1914	0.1088				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ Silty Sand	10-30-23	SM	16.2%

Project No. 23106-1 **Client:** Parametrix

Project: Hamehook Bridge Replacement

○ **Source of Sample:** B-01 **Depth:** 5' **Sample Number:** S-3

Remarks:

○ Lab No.: WGG0336
Sampled By: KAK on 10-23-23
C117 Wash Procedure A

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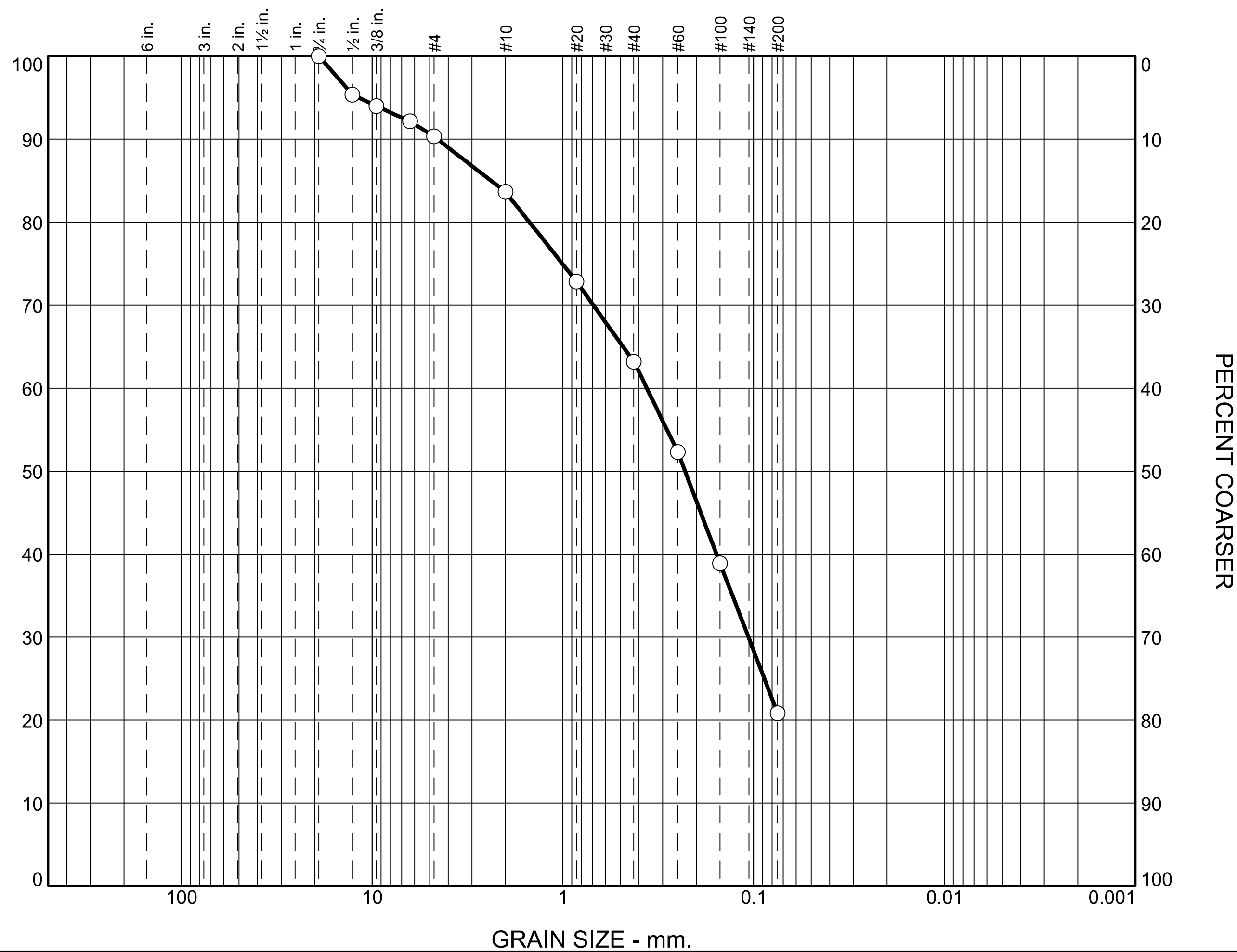
Figure B-3

Tested By: PJH **Checked By:** AML

Sieve Analysis (C117/C136)



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	10	6	21	42	21	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	Not Tested	Not Tested	2.3831	0.3640	0.2292	0.1067				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ Silty Sand	10-30-23	SM	24.7%

Project No. 23106-1 **Client:** Parametrix
Project: Hamehook Bridge Replacement

○ **Source of Sample:** B-01 **Depth:** 10' **Sample Number:** S-5

Remarks:
○ Lab No.: WGG0336
Sampled By: KAK on 10-23-23
C117 Wash Procedure A

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Figure B-4

Tested By: PJH **Checked By:** AML

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CORE COMPRESSIVE STRENGTH REPORT

ASTM D-7012

Client: Parametrix
Project Name: Hamehook Bridge Replace
Technician: PJH
Reviewed By: AML
Date Sampled: 10/23/2023
Project No. : 23106 - 1
Lab No. : WGG0336
Date Tested: 10/30/2023

Boring	B-01	B-02
Sample	C-1	C-1
Depth (ft.)	14.5'	15'
Date Prepared	10/26/2023	10/26/2023
Length (in.)	5.04	5.24
Diameter (in.)	2.34	2.34
Area (sq in.)	4.30	4.30
Ultimate Load (lbs.)	20990	35000
Compressive Strength (psi)	4880	8140
Cap Thickness (in.)	0.12	0.11

Note: Data and results shown above include ASTM Test Method D-7012. This report pertains only to the material tested and/or inspected and is not to be reproduced without prior authorization of Wallace Group. If part of a larger document, this report is not to be removed or reproduced separately. This report is the property of the Client and shall not be distributed to other parties without Client's permission.

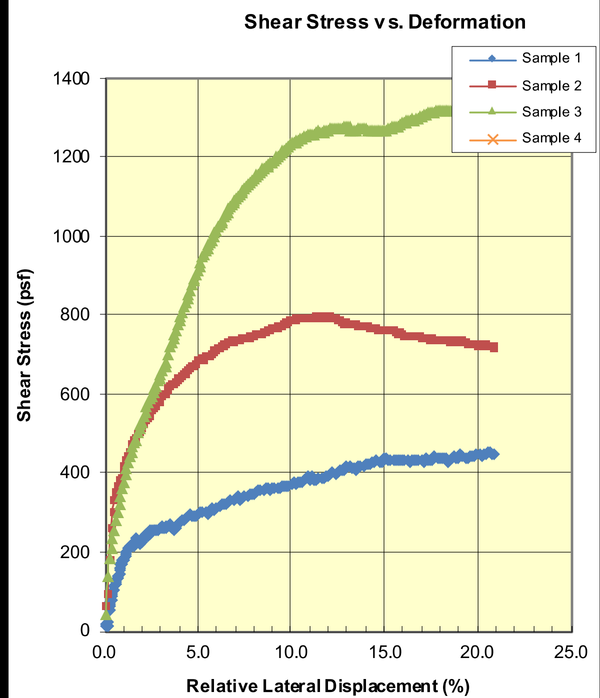
Consolidated Undrained Direct Shear (ASTM D3080M)

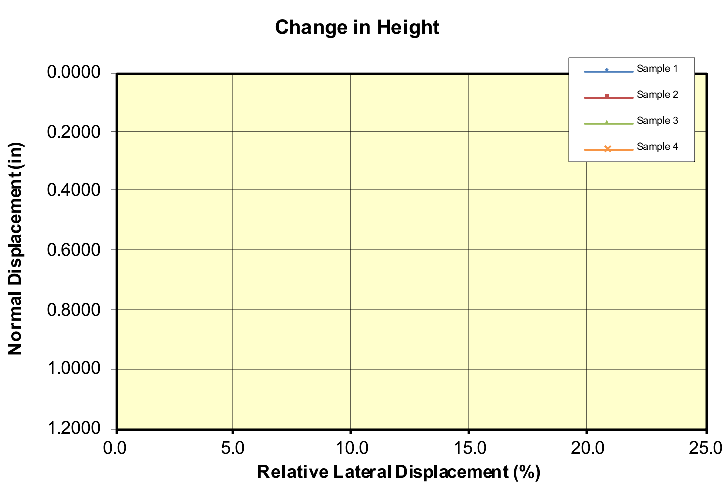
COOPER TESTING LABORATORY

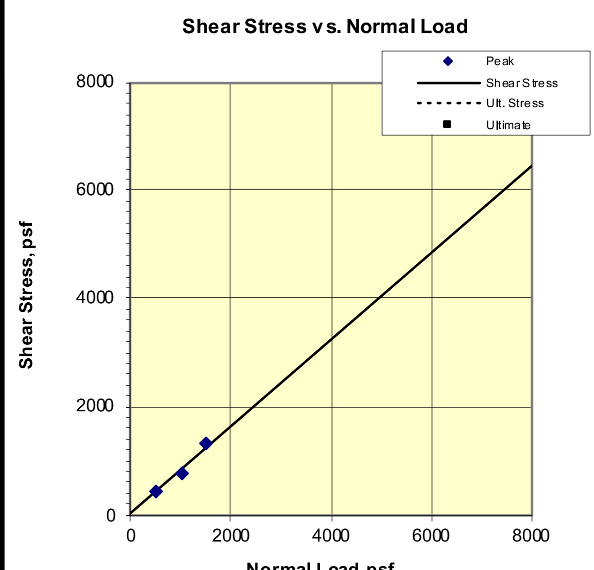
CTL Job #:	1152-007	Project #:	23106-1	By:	MD
Client:	Wallace Group, Inc.	Date:	11/10/2023	Checked:	PJ
Project Name:	Hamehook Bridge Replacement	Remolding Info:	80pcf @ 15%.		

Phi (deg)	38.9	Ult. Phi (deg)	
Cohesion (psf)	50	Ult. Cohesion (psf)	

Specimen Data	1	2	3	4
Boring:	B-02	B-02	B-02	
Sample:	S-3	S-3	S-3	
Depth (ft):	5	5	5	
Visual Description:	Dark Brown Silty SAND	Dark Brown Silty SAND	Dark Brown Silty SAND	
Normal Load (psf)	500	1000	1500	
Dry Mass of Specimen (g)	94.2	94.4	94.5	
Initial Height (in)	1.00	1.00	1.00	
Initial Diameter (in)	2.41	2.40	2.41	
Initial Void Ratio	1.142	1.120	1.135	
Initial Moisture (%)	16.8	16.6	16.6	
Initial Wet Density (pcf)	91.9	92.7	92.0	
Initial Dry Density (pcf)	78.7	79.5	78.9	
Initial Saturation (%)	39.8	40.1	39.4	
ΔHeight Consol (in)	0.0108	0.0270	0.0523	
At Test Void Ratio	1.119	1.063	1.023	
At Test Moisture (%)	34.3	32.0	30.5	
At Test Wet Density (pcf)	106.9	107.8	108.7	
At Test Dry Density (pcf)	79.6	81.7	83.3	
At Test Saturation (%)	82.9	81.3	80.6	
Strain Rate (%/min)	1.0	1.0	1.0	
Strengths Picked at	Peak	Peak	Peak	
Shear Stress (psf)	451	794	1329	
ΔHeight (in) at Peak				
Ultimate Stress (psf)				







Remarks: *DS-CU* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests.

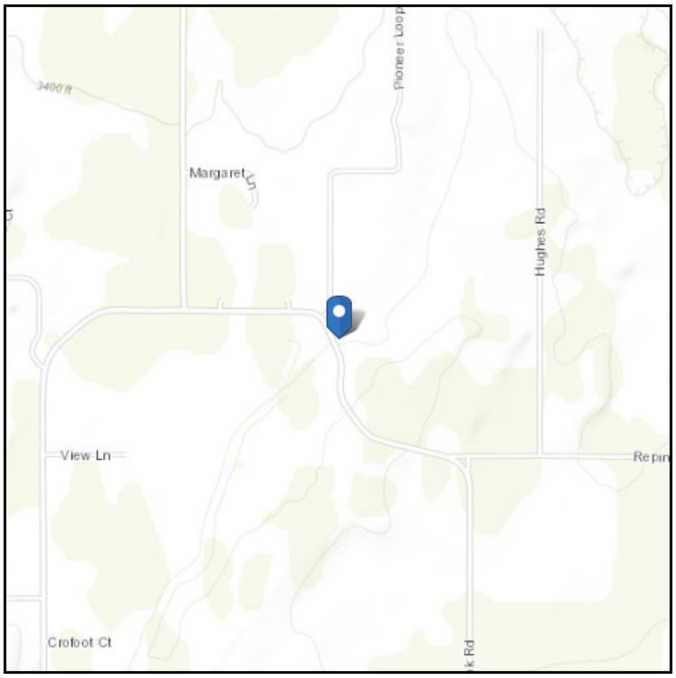
WALLACE GROUP
NORTHWEST GEOSYSTEM EXPERTS

APPENDIX C

ASCE 7 Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-16	**Latitude:**	44.10499
Risk Category:	II	**Longitude:**	-121.24817
Soil Class:	B - Estimated (see Section 11.4.3)	**Elevation:**	3416.370491157842 ft (NAVD 88)



Seismic

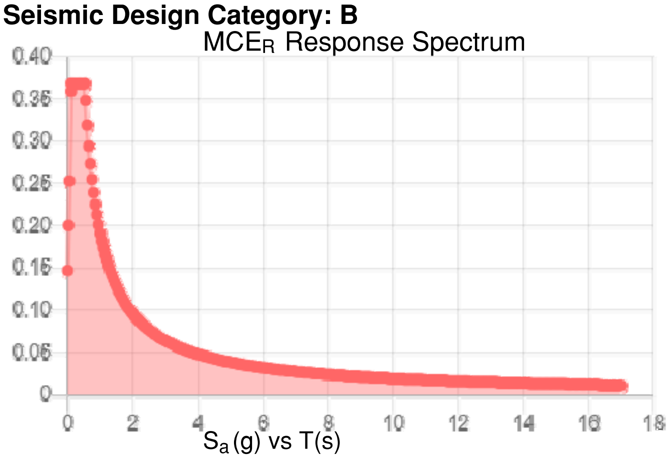
B - Estimated (see Section 11.4.3)

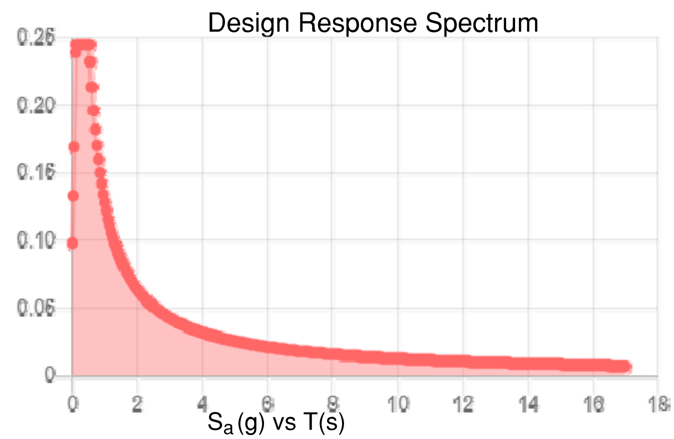
Site Soil Class:

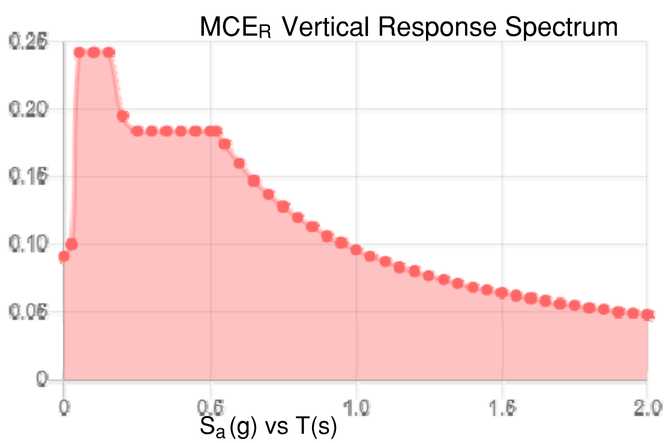
Results:

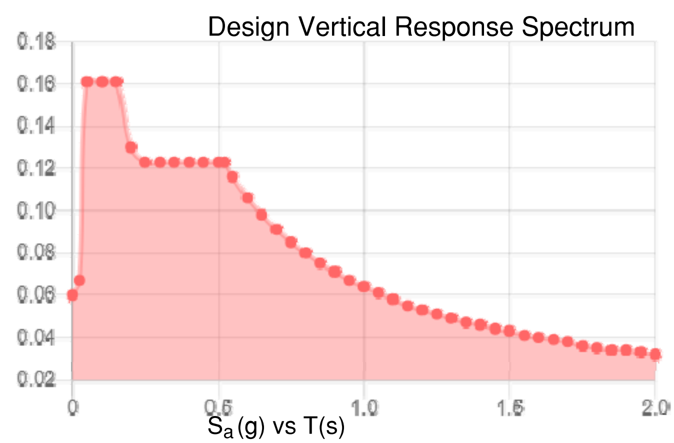
S_S :	0.368	S_{D1} :	0.128
S_1 :	0.191	T_L :	16
F_a :	1	PGA :	0.167
F_v :	1	PGA_M :	0.167
S_{MS} :	0.368	F_{PGA} :	1
S_{M1} :	0.191	I_e :	1
S_{DS} :	0.245	C_v :	0.823

Seismic Design Category: B









Data Accessed: **Fri Oct 27 2023**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

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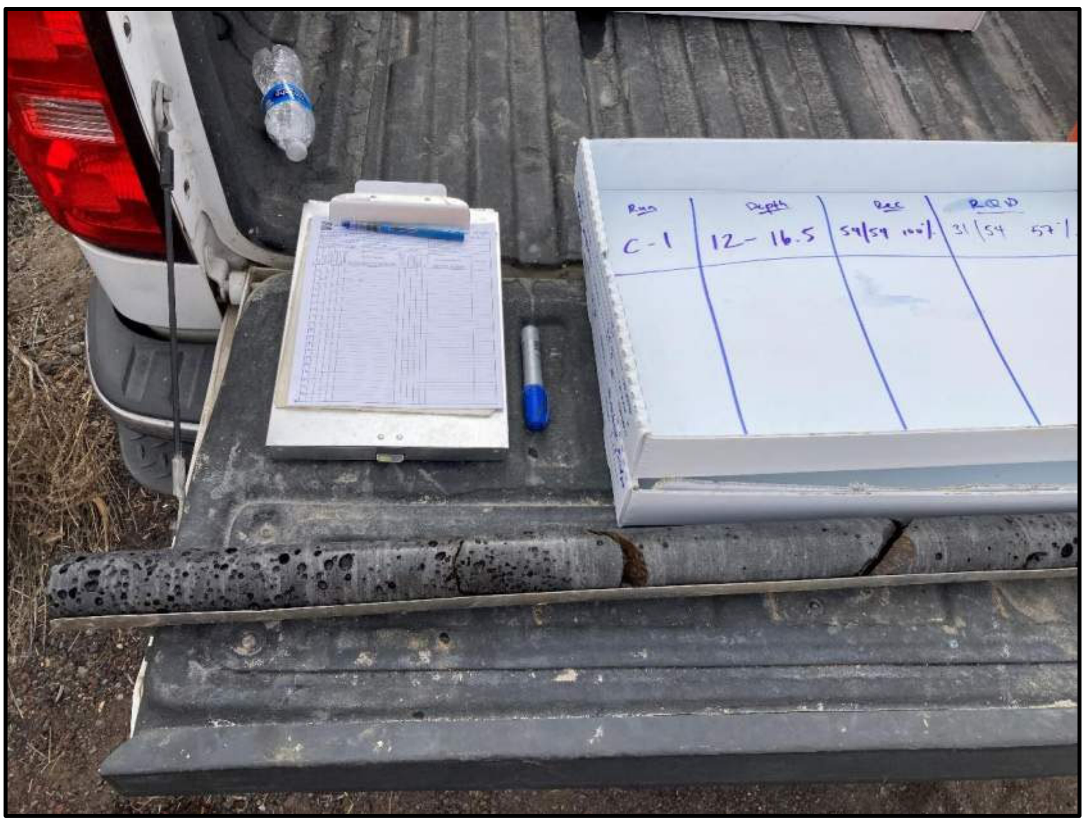
APPENDIX D

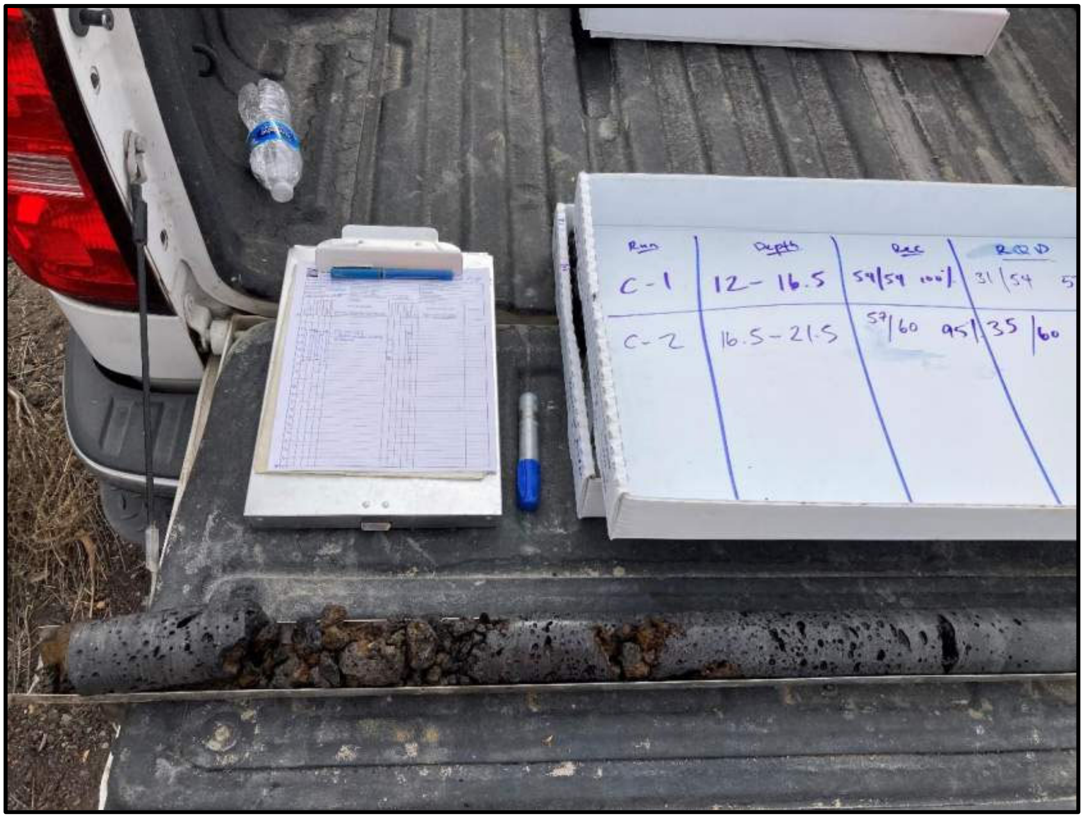
Existing conditions during exploration.

Geotechnical borings performed with truck-mounted CME 850 drilling rig.

Drilling methods included hollow stem auger and HQ-wireline coring.



B-01 core run C-1 from 12 to 16.5-feet bgs.



B-01 core run C-2 from 16.5 to 21.5-feet bgs. Note broken zones.

B-01 core run C-3 from 21.5 to 26.5-feet bgs. Note very poor rock quality.

B-02 core run C-1 from 15 to 17-feet bgs. Note abundant and large vesicles.

B-02 core run C-2 from 17 to 22-feet bgs. Note transition to massive rock with very few vehicles.