Appendix D

Report of Geotechnical Investigations SR 202 Widening and Trestle Replacement and Addendum 1 – Neutral Plane Method

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AECOM

Report of Geotechnical Investigation SR 202 Widening and Trestle Replacement

NE 175th Street to NE 177th Street
Woodinville, Washington

City of Woodinville

Project number: 60683538

November 25, 2025

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Quality information

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Revision History

Revision	Revision date	Details	Authorized	Name	Position
0	N/A	Initial	June 16, 2025	M. Walbaum	Engineer of Record
1	Nov. 25, 2025	95% Design	Nov. 25, 2025	M. Walbaum	Engineer of Record

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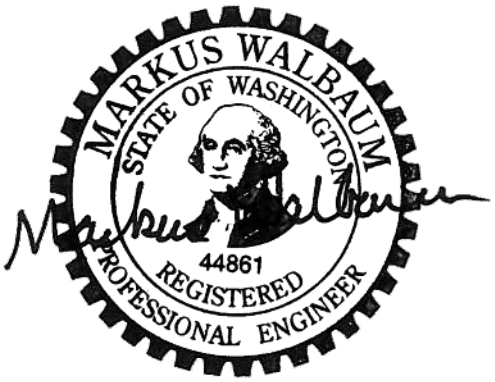
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November 25, 2025

Ms. Asha D'Souza
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Report of Geotechnical Investigation

SR 202 Widening and Trestle Replacement

Woodinville, Washington

Dear Ms. D'Souza:

AECOM Technical Services, Inc. (AECOM) is pleased to submit herewith our Report of Geotechnical Investigation for the “Phase B” elements of the SR 202 Widening and Trestle Replacement Project in Woodinville, Washington. Phase B includes widening and/or partial reconstruction of 131st Avenue NE between NE 175th Street and NE 177th Street, replacement of the existing railroad trestle over 131st Avenue NE with a new pedestrian bridge and retaining walls, and construction of a multi-user trail (Eastrail Trail) extending 1,256 feet east from the Sammamish River Trail to approximately 380 feet east of the bridge. Services provided for this project were performed in general accordance with the Professional Services Agreement No. 22-025 between AECOM and the City of Woodinville effective as of April 20, 2022.

AECOM thanks you for the opportunity to be of service on this project. Please call should questions arise or additional information is required.

Yours sincerely,

Markus Walbaum, PE
Senior Geotechnical Engineer
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TABLE OF CONTENTS

1. Introduction 1
2. Project Description 1
3. Scope of Services 2
4. Subsurface Investigation Program 3
4.1 Previous Borings and Laboratory Testing 3
4.2 Current Subsurface Explorations 4
4.3 Field and Laboratory Testing 5
5. Site And Subsurface Conditions 5
5.1 Site Description 5
5.2 Geologic Setting 6
5.3 Site Subsurface Conditions 6
5.3.1 Existing Pavement 6
5.3.2 Site Soils 7
5.3.3 Groundwater 8
5.3.4 Laboratory Testing Results 8
5.3.5 Environmental Investigation 9
5.4 Geologically Sensitive Areas (Critical/Hazard Areas) 9
6. Conclusion And Recommendations 10
6.1 Soil Engineering Parameters 10
6.2 Seismic Considerations 12
6.2.1 Regional Seismic Setting 12
6.2.2 Seismic Design Parameters 13
6.2.3 Liquefaction and Seismic-Induced Settlement 14
6.3 Bridge Abutments 15
6.3.1 Shallow Foundation Design Methodology 15
6.3.2 Shallow Foundation Design Parameters 16
6.3.3 Shallow Foundation General Data and Load Data 17
6.3.4 Shallow Foundation Bearing Resistance 17
6.3.5 Shallow Foundation Settlement 18
6.3.6 Shallow Foundation Lateral Load Resistance 20
6.3.7 Settlement of Embankment Fills Adjacent to Shallow Foundations 21
6.3.8 Deep Foundation Design Methodology 22
6.3.9 Deep Foundation General Data and Load Data 22
6.3.10 Deep Foundation Axial Load Resistance 22
6.3.11 Deep Foundation Axial Load Group Effects 23
6.3.12 Deep Foundation Lateral Load Resistance 24
6.3.13 Deep Foundation Downdrag Loading 24
6.3.14 Deep Foundation Static Settlement 25
6.3.15 Lateral Earth Pressures 25
6.3.16 Retaining Wall Subsurface Drainage 28
6.4 Permanent Retaining Walls 29
6.4.1 Concrete Retaining Wall Recommendations 29
6.4.2 Mechanically Stabilized Earth Walls 34
6.5 Bridge Wingwalls 38

6.5.1 Wall Type Evaluation 38
6.5.2 Recommended Wall Types 40
6.5.3 NW Wingwall Recommendations 40
6.5.4 SW, SE, and NE Wingwalls Recommendations: Two-Stage MSE Wall 42
6.5.5 SW, SE, and NE Wingwalls Alternative: Anchored Soldier Pile Wall 43
6.5.6 Wingwall Construction Considerations 47
6.5.7 Temporary Lagging 47
6.6 Global Stability Evaluation 48
6.7 Traffic Signal and Luminaire Foundations 49
6.8 Pavement Design 49
6.8.1 Proposed Pavement Rehabilitation and Construction 49
6.8.2 Existing Pavement Section 50
6.8.3 Subgrade Support 50
6.8.4 Design Standards 50
6.8.5 Traffic Loading Analysis 50
6.8.6 Pavement Design Parameter and Approach 51
6.8.7 Analysis Results 52
6.8.8 Pavement Design Recommendations 53
6.9 Earthwork and Construction Considerations 54
6.9.1 General 54
6.9.2 Subgrade Preparation 55
6.9.3 Bridge Approach and Embankment 55
6.9.4 Excavation and Temporary Slopes 57
6.9.5 Adjacent Utility Construction 58
6.9.6 Subsurface Drainage 58
6.9.7 Wet Weather Considerations 59
6.9.8 Dewatering Considerations 59
6.9.9 Pilot Hole Verification Test Borings for Drilled Shaft Installation 59
6.9.10 Settlement Monitoring 60
6.9.11 Monitoring Wells 61
7. Limitations 61
8. References 62

TABLES

Table 1. Recommended Geotechnical Engineering Design Parameters for In-Situ Soils 11
Table 2. Recommended Geotechnical Engineering Design Parameters 12
Table 3. Seismic Design Parameters 14
Table 4. Soil Parameters for Shallow Foundation Design – West Abutment 16
Table 5. Soil Parameters for Shallow Foundation Design – East Abutment 16
Table 6. Shallow Foundation General Data 17
Table 7. Shallow Foundation LRFD Loads for Controlling Load Combination 17
Table 8. Bridge Abutment Shallow Foundations Bearing Resistance Summary 18
Table 9. Summary of Staged Settlement Analysis for Bridge Abutment Footings 20
Table 10. Deep Foundation LRFD Loads for Controlling Load Combination 22
Table 11. Drilled Shaft Foundation Axial Resistance Summary 23

Table 12. Recommended P-Multipliers for Drilled Shaft Group Lateral Resistance....24
Table 13. Estimated Static Downdrag Loads for Drilled Shaft Foundations 25
Table 14. Lateral Earth Pressure Recommendations – Vertical Wall Backface....27
Table 15. Lateral Earth Pressure Recommendations – Battered Wall Backface 28
Table 16. Retaining Wall Shallow Foundations Recommendations – West Approach....31
Table 17. Retaining Wall Shallow Foundation Recommendations – East Approach 32
Table 18. Summary of Staged Settlement Analysis for Concrete Retaining Walls....33
Table 19. Summary of Analyzed Permanent MSE Wall Geometry....34
Table 20. Summary of Applicable Load and Resistance Factors for MSE Wall Analysis....35
Table 21. Permanent MSE Walls – Summary of MSEW+ Inputs and External Stability Results36
Table 22. Summary of Proposed Wingwalls 38
Table 23. MSE Wingwalls – Summary of MSEW+ Inputs and External Stability Results 42
Table 24. LRFD Design Bond Stress Recommendations for Ground Anchors....44
Table 25. Global Stability Analysis Results – Bridge Abutment Walls....48
Table 26. Global Stability Analysis Results – Permanent Concrete Walls 48
Table 27. Global Stability Analysis Results – Permanent MSE Walls....48
Table 28. Global Stability Analysis Results – Soldier Pile Walls....49
Table 29. Estimated Traffic Loading 51
Table 30. Pavement Design Input Parameters 51
Table 31. Widening Full Depth Construction Section....52
Table 32. Existing Pavement Rehabilitation - Asphalt Overlay Sections 53
Table 33. Select Borrow Grading and Quality....56

FIGURES

Figure 1. Site Location
Figure 2. Project Limits Plan
Figure 3. Site Plan
Figure 4. Estimated Subsurface Profile A-A’
Figure 5. Geologic Map
Figure 6. Groundwater Elevations in Monitoring Wells

APPENDICES

Appendix A Historical Exploration Logs and Laboratory Testing Results
Appendix B Subsurface Exploration Program
Appendix C Summary of Pavement Cores
Appendix D Geotechnical Laboratory Testing
Appendix E Environmental Investigation Report
Appendix F Retaining Wall Global Stability Analysis Results
Appendix G Drilled Shaft Axial Capacity For East And West Bridge Abutments

ACRONYMS AND ABBREVIATIONS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AECOM	AECOM Technical Services, Inc.
bgs	below ground surface
bpf	blows per foot
BDM	WSDOT 2022 Bridge Design Manual (LRFD) M 23-50.21
BDS	AASHTO 2020 LRFD Bridge Design Specifications
BNSF	Burlington Northern Santa Fe
BOF	Bottom of Footing
CAB	Crushed Aggregate Base
CBR	California Bearing Ratio
CIP	Cast-in-place
City	City of Woodinville, Washington
CMP	Corrugated Metal Pipe
CSZ	Cascadia Subduction Zone
DOT	Department of Transportation
ESAL	Equivalent Single Axle Load
FHWA	Federal Highway Administration
FOS	Factor of Safety
GDM	WSDOT 2022 Geotechnical Design Manual M 46-03.16
H:V	Horizontal to Vertical
LRFD	Load and Resistance Factor Design
MP	Milepost (WSDOT Roadways)
MSE	Mechanically Stabilized Earth
NAVD 88	North American Vertical Datum of 1988
OSHA	Occupational Safety and Health Administration
PGA	Peak Ground Acceleration
PID	Photoionization Detector/Organic Vapor Meter
PP	Pocket Penetrometer Test
SPT	Standard Penetration Test
SR	State Road
Site	SR 202 Widening and Trestle Replacement Project Site
TPH	Total Petroleum Hydrocarbons
USDOT	United States Department of Transportation
USGS	United States Geological Survey
VOCs	Volatile organic compounds
WADNR	Washington Department of Natural Resources
WISHA	Washington Industrial Safety and Health Act
Woodinville	City of Woodinville, Washington
WSDOT	Washington State Department of Transportation
WWD	Woodinville Water District
ZZA	Zipper Zeeman Associates, Inc.

EXECUTIVE SUMMARY

This geotechnical report has been prepared in support of design for the Woodinville SR202 Widening and Trestle Replacement project in Woodinville, WA for the City of Woodinville. Geotechnical investigations and analyses were performed in accordance with AASHTO LRFD Bridge Design Specifications and WSDOT Geotechnical Design Manual guidelines to evaluate proposed improvements including bridge replacement with shallow and deep foundation options, new retaining walls, new pavements, and related earthwork.

1. Characterization of subsurface stratigraphy and groundwater conditions was based on results of a project-specific geotechnical investigation conducted by AECOM consisting of geotechnical test borings with Standard Penetration Testing (SPT), piezometer installation and monitoring, and geotechnical laboratory testing. The investigation results were augmented by results of available previous historical geotechnical and environmental investigations by others.
2. Subsurface data indicate three generalized soil strata in descending order: Stratum 1 (Existing Fill consisting of loose to medium dense sands and isolated layers of medium stiff to stiff silt and lean clay); Stratum 2 (loose to dense sand with gravel and silt); and Stratum 3 (dense to very dense sand with gravel and silt). Groundwater readings over the monitoring period of October 2022 to early August 2023 ranged from Elevation +23 to +27 feet (approximately 5 to 8 feet below ground surface at the 131st Avenue underpass road grade), with Elevation +25 selected for design analyses.
3. The site lies within an area of high seismicity. Seismic design coefficients were provided for structural design. Liquefaction triggering analyses indicate isolated thin lenses of liquefaction-susceptible soils below the groundwater table in Stratum 2 and Stratum 3. Due to the lack of continuous thick liquefiable layers, the risk of liquefaction and its related effects of strength loss, reconsolidation settlement, and lateral spreading are low.
4. Results of global stability analyses for proposed bridge abutment, approach embankments, and retaining walls indicate the minimum required factors of safety are satisfied for static and seismic conditions.
5. For the bridge abutments, shallow foundations both with and without ground improvement were evaluated, but results indicate an unacceptable risk of excessive immediate settlement (greater than 1 inch) due imposed stresses associated with the proposed raised height of approach embankments. Accordingly, deep foundation support for the bridge abutments consisting of drilled shafts is recommended.
    - Drilled shaft axial capacity plots are provided for 3-, 4-, 5-, and 6-foot diameters at each abutment. Recommendations for lateral load response (LPILE) parameters and group effects are also provided.
    - Required lengths of drilled shaft foundations to meet axial loading demand extend below the exploration depths of the available borings. Accordingly, two additional verification pilot test borings (one per abutment) should be drilled either as part of design or as a pre-construction activity to verify assumed subsurface conditions.
    - Drilled shafts should be designed to resist estimated downdrag loads caused by relative settlement of surrounding soils in Stratum 2 caused by the imposed stresses associated with raised height of approach embankments. Note that downdrag loads are expected to occur immediately after construction of the approach embankment to finish grade. Downdrag recommendations are provided

according to the AASHTO and WSDOT explicit method, which tends to be conservative. If requested, recommendations according to the neutral plane method as allowed by AASHTO can be provided to reduce conservatism.

6. Design recommendations are provided for wingwalls adjacent to the abutments, which are expected to consist of cantilever soldier pile walls with ultra-lightweight backfill for the northwest (NW) wingwall, and mechanically stabilized earth (MSE) walls for the southeast (SE), southwest (SW), and northeast (NE) wingwalls. Staged MSE wall construction using temporary facing is needed to accommodate estimated foundation settlement of between 3 and 4 inches caused by construction of the new approach embankments, followed by installation of permanent concrete facing after settlement is complete. Ground improvement is not needed for MSE walls due to the inherent flexibility of the temporary facing which can accommodate settlements of up to 4 inches without special design measures per the WSDOT Geotechnical Design Manual. Additionally, final concrete facings for the soldier pile wingwalls may be installed after approach embankments have been built to final grade elevation to allow settlement/deflection to occur and reduce the potential for concrete cracking.

    - Recommendations for alternative wingwall design consisting of anchored soldier pile walls with prestressed ground anchors are also provided for the SE, SW, and NE walls. We understand this alternative will not be selected by the design team on the basis of cost and special construction sequencing needed to accommodate the proposed near-complete removal of the existing approach embankments near the abutment to construct the new foundation shaft caps. This construction sequence would require building-up fill behind the solider pile walls in stages to reconstruct the approach embankments, and the use of temporary berms in front of the soldier pile walls and/or limited-access drill rigs to install prestressed ground anchors.

7. Design recommendations are provided for approach embankment retaining walls, which are expected to consist of MSE walls. Similar to the wingwalls, staged MSE construction is needed to accommodate foundation settlement associated with construction of the new approach embankments, but ground improvement is not necessary.

    - Recommendations for alternative permanent concrete retaining walls are also provided for the approach embankments. We understand this alternative will not be selected by the design team on the basis of cost. Results for shallow spread-footing foundations both with and without ground improvement are presented. Results indicate that ground improvement consisting of stone columns is needed to limit settlement to tolerable levels due to imposed stresses associated with the proposed raised height of adjacent approach embankments. With stone column ground improvement, immediate settlements between 1.5 and 2 inches are expected, which may require special slip-jointing at the connection with wingwalls to accommodate differential settlement.

8. Lateral earth pressures for static and seismic conditions are provided for bridge abutment walls, wingwalls, and concrete retaining walls. High-quality imported gravel backfill is recommended behind each of the retaining walls, except at the NW wingwall where ultra-lightweight backfill consisting of foamed-glass aggregate is recommended to limit lateral earth pressures and deflections of the cantilever soldier piles to tolerable values.

9. Pavement design recommendations are provided, which include full-depth flexible pavement for road widening, and milling and asphalt overlay for existing pavements.

10. Recommendations for design and construction are provided, which address surface stripping, temporary excavations, control of surface and subsurface water, subgrade preparation, fill material specifications, fill placement and compaction criteria, wet weather construction, and construction materials testing and inspections.

1. INTRODUCTION

This report presents the results of the AECOM geotechnical investigation for the proposed State Route (SR) 202 roadway widening and railroad trestle bridge replacement in the City of Woodinville, Washington. The purpose of AECOM's services was to investigate subsurface conditions within the Project limits and provide geotechnical and foundation design and construction recommendations for replacement of the trestle bridge by a pedestrian bridge, approach embankments, traffic signals and luminaires, utility improvements, and roadway pavement. **Figure 1** illustrates the site location and **Figure 2** shows the overall limits of the Project.

Topographic feature and design element elevations discussed herein are referenced to the North American Vertical Datum of 1988 (NAVD88). Where an elevation in the report appears without a dimension, the unit of measurement is the foot, e.g., Elevation +36 denotes Elevation +36 feet.

2. PROJECT DESCRIPTION

The Project is located along 131st Avenue NE (SR 202) between NE 175th Street and NE 177th Place (named Little Bear Creek Parkway east of 131st Ave NE) in Woodinville, Washington; refer to **Figure 3** for the site plan. Per the AECOM 95% design drawings dated November 14, 2025, the project extends from approximately 131st Ave. NE Stations 102+50 (SR 202 MP 0.31) to 108+00 (SR 202 MP 0.23) with an approximate length of 550 feet from south to north. To accommodate increasing traffic, 131st Ave. NE will be widened, and traffic lanes reconfigured, to increase the roadway width from five lanes to eight lanes. The widening requires increasing the span length of the existing inactive Burlington Northern Santa Fe (BNSF) steel railroad trestle bridge over NE 131st Street. Instead of replacing the trestle bridge with a new railway bridge, the bridge will be replaced by a new pedestrian bridge for a new section of the Eastrail recreation and transportation trail. The trestle replacement includes removal of the existing bridge, abutments, wingwalls, and concrete pier in the median of 131st Ave. NE and constructing new abutments, wingwalls, and a clear-span pedestrian bridge. The new west abutment will be constructed about 5 feet west of the existing west abutment and the new east abutment will be constructed about 40 feet east of the existing east abutment. The replacement bridge will be constructed at the same general location as the existing trestle, but the bridge span will increase from 80 feet to 125 feet, and the bridge will be raised about 7 feet to increase the vertical clearance over the roadway below.

The Project also encompasses the construction of a new section of the Eastrail recreation and transportation trail along the former BNSF railway corridor. (Note: The BNSF right-of-way in the Project area now comprises City right-of-way.) This section of the Eastrail will be surfaced with asphalt concrete pavement and will extend about 740 feet west from the west abutment of the new bridge and connect to the Sammamish River Trail at Eastrail Station 30+30. On the east side of the bridge, the new Eastrail will extend about 380 feet east from the new east abutment and terminate on the existing railbed at Eastrail Station 42+86. The overall length of the Eastrail trail within the Project will be 1,256 feet and the trail width will range from 10 to 12 feet paved plus a 2-foot-wide gravel shoulder on either side. The existing railway bridge approach embankments will be raised up to about 7 to 8 feet to about Elevation +58 to match the new bridge elevation, and the sideslopes will be flattened to maximum inclinations of 3 Horizontal to 1 Vertical (3H:1V) versus the existing approximately 2H:1V sideslopes. Retaining walls up to approximately 15 to 30 feet tall will be required due to inadequate right of way to provide full-height 3H:1V embankment sideslopes. The bridge abutment backfill zones will be excavated for construction of the replacement bridge abutment and wingwall foundations. The backfill will

consist of recompacted excavated soils, if suitable, and/or replaced and supplemented with compacted imported fill.

Construction of the Eastrail trail will include removal of the BNSF track structure (completed in fall 2023), clearing and grubbing, regrading with soil cuts and fills of up to approximately 10 feet, but generally less than 5 feet, and placement of base course and asphalt pavement. On the east side of the proposed bridge, a new "177th Connection Trail" will connect the east sidewalk of 131st Ave. NE with the Eastrail trail northeast of the proposed bridge. The 320-foot-long connecting trail will rise 19 feet over its length and require soil cuts and fills of generally less than 2 feet. Retaining walls or similar structures may be used in this area depending on the landscaping design and slopes selected during design development.

Additional project elements include:

- Grinding existing pavement and overlaying with new pavement;
- Constructing new pavement where 131st Ave. NE will be widened on the east side of the roadway;
- Widening and improving sidewalks;
- Installing new traffic signals, pedestrian signals, and luminaires; and
- Relocating utilities and relocating/upsizing the Woodinville Water District (WWD) sewer main from 175th Street NE to 177th Street NE.

3. SCOPE OF SERVICES

The proposed scope of services for the Project is identified in the Professional Services Agreement No. 22-025 between the City of Woodinville (City) and AECOM dated April 20, 2022. The Agreement includes a Scope of Work for the Engineering and Traffic Study Services to widen SR 202 in the vicinity of the SR 522 / SR 202 Interchange and Trestle Replacement Project No. ST-13-002. The scope of the geotechnical investigation included:

Review of Existing Information: We compiled and reviewed available geologic and geotechnical information, including geologic maps, logs of borings and pavement cores, and geotechnical reports by others for projects at and near the Site. We also reviewed available as-built plans.

Test Borings and Laboratory Testing: We conducted additional subsurface explorations consisting of pavement cores and test borings at key locations near the planned facilities to confirm and supplement existing information by others. We performed geotechnical laboratory and corrosivity testing on selected soil samples from the explorations to evaluate soil physical and engineering characteristics and parameters.

Engineering Analyses: We evaluated the subsurface information, prepared a subsurface profile based on the borings, and performed engineering analyses to inform the design of the proposed roadway and bridge widening. The analyses included establishing soil engineering parameters for design, seismic design parameters, bridge foundation and approach embankment settlement, embankment stability, and potential liquefaction. Analyses were performed in general accordance with the current versions of the Washington State Department of Transportation (WSDOT) 2022 Geotechnical Design Manual (GDM), WSDOT 2022 Bridge Design Manual (BDM), and American Association of State Highway and Transportation Officials (AASHTO) 2020 LRFD Bridge Design Specifications (BDS), and AASHTO 2011 Guide Specifications for LRFD Seismic Bridge Design.

Geotechnical Report: This geotechnical report was prepared to describe and discuss the investigation program, field explorations, laboratory testing, technical analyses, and recommendations for design and construction of the proposed project elements.

Under this contract, as part of a separate environmental evaluation scope of work for the Woodinville Water District, we performed three shallow (15-20 feet deep) borings, installed a groundwater monitoring well in one of the borings, conducted laboratory analytical testing on soil and groundwater samples from the borings, and prepared a summary report of environmental conditions in the area of sewer main improvements within the overall roadway widening project. The summary report is included as an appendix herein. (Note: This scope of work was associated with the Jacksons Shell and Convenience Store, where a release of total petroleum hydrocarbons (TPH) had occurred in the past.)

4. SUBSURFACE INVESTIGATION PROGRAM

Subsurface soil and groundwater conditions at the project site were investigated by a combination of reviewing relevant reports of prior studies in the vicinity of the Project and conducting a site-specific subsurface exploration program consisting of pavement cores, test borings, and laboratory testing on selected samples collected from the borings.

4.1 Previous Borings and Laboratory Testing

Information on the subsurface conditions was obtained by reviewing the results of previous exploration programs that were conducted at and near the Site by others. We reviewed the following reports or design drawings/as-built drawings:

1. WSDOT (previously Washington State Highway Commission, Department of Highways), Design Drawings, State Highways Nos. 2 and 1-A, Woodinville Vicinity, King County, Contract No. 7234, April 15, 1963 (22 sheets). Also includes a separate 18-sheet drawing set with filename “BNSF Trestle Bridge over SR202 – As-Builts;” some of these drawings are similar to or identical to drawings in the 22-sheet set. One of the drawings contained logs for Borings H-1 and H-2.
2. WSDOT, Geotechnical Report, SR 202: SR 522 to NE 175th Street, C.S. 1710, L-8602, July 1, 1987.
3. WSDOT, Intra-Departmental Communication (WSDOT), SR 202: SR 522 to NE 175th Street, C.S. 1720, L-8602, Retaining Wall, January 25, 1988.
4. WSDOT, Intra-Departmental Communication (WSDOT), SR 202: SR 522 to NE 175th, Cantilever Signals, L-8602, February 29, 1988.
5. Zipper Zeman Associates, Inc., Geotechnical Engineering Exploration and Analysis, Four Signal Pole Foundations at SR202 & NE 177th Place, Intersection Improvement Design Recommendations, Woodinville, Washington, May 25, 2001.
6. Zipper Zeman Associates, Inc., Geotechnical Report, Signal Support Foundation, SR 202/SR 522 Interchange Improvements Perteet Project No. 23055, (Phase II - SR 202/177th PL. NE, North Leg), Woodinville, Washington, June 06, 2004.
7. Zipper Zeman Associates, Inc., Geotechnical Report, Proposed Retaining Walls and Pedestrian Bridge, SR 202/SR 522 Interchange Improvements Perteet Project No. 23055, (Phase II - SR 202/NE I 77th PL., North Leg), Woodinville, Washington, July 08, 2004.
8. Zipper Zeman Associates, Inc., Geotechnical Report, BNSF- SR 202/SR 522 Interchange Improvements, Woodinville, Washington, September 23, 2005.

9. Zipper Zeman Associates, Inc., Geotechnical Report, NE 175th Street and 131st Avenue NE Improvements, Woodinville, Washington, August 15, 2006.
10. Zipper Zeman Associates, Inc., Soil Sampling Results, NE 175th Street and 131st Avenue Northeast Improvements, Woodinville, Washington, ZZA Project No. 81051721, April 05, 2007.
11. ZZA-Terracon, Report of Geotechnical Services, New 36-Inch Storm Drain, SR 202 Southeast Leg Improvements, Woodinville, Washington, March 31, 2008.
12. Terracon, Report of Geotechnical Services, City of Woodinville – BNRP SE Leg Storm Phase 2, Woodinville, Washington, Terracon Project No. 81095801, September 16, 2009.

Although the documents listed were reviewed, only borings close to the Site and containing relevant data were considered for design and are shown in **Figure 3**. Logs of the borings and results of laboratory testing performed are provided in **Appendix A**.

4.2 Current Subsurface Explorations

To confirm and supplement existing subsurface conditions information, AECOM performed subsurface explorations at the Site including five test borings, designated BT-1 through BT-5, and four pavement cores, designated BT-6 through BT-9; refer to **Figure 3** for the exploration locations. **Appendix B** contains descriptions of the exploration methods, the exploration logs, and results of standard penetration test (SPT) energy measurements. Section 5 of the text and **Appendix C** provide a summary of findings from the pavement cores.

Prior to drilling Borings BT-1 through BT-5 with a drill rig, we excavated them to 5 feet below ground surface (bgs) using air knifing combined with vacuum excavation to check for the presence of utilities that may not have been marked by the required “one-call” (Washinton 811) utility clearance process. At the pavement core locations, before coring, we scanned the subsurface for buried utilities, storage tanks, or other potentially sensitive structures using ground penetrating radar instead of performing air knifing/vacuum excavation.

Borings BT-1 through BT-3 were drilled to depths of 16.5 to 21.1 feet bgs from the 131st Avenue NE roadway level; the ground surface at the borings varied from Elevations +32.4 to +35.5 feet. The borings were drilled using hollow-stem auger methods. A groundwater monitoring well was installed in Boring BT-2 to permit periodic groundwater level measurements, and if needed, facilitate collection of future groundwater samples, e.g., for analytical testing. The original primary purpose of these three borings was to characterize subsurface soil, groundwater, and environmental conditions for upsizing of the existing 18-inch-diameter sanitary sewer main across 131st Avenue NE. However, during design development, the project team developed an alternate and preferable route for the sewer main that does not require upsizing across the road at the original location.

Borings BT-4 and BT-5 were drilled to depths of 101.5 ft and 81.5 feet bgs, respectively, from the top of the railway embankment (approximately Elevation +50.5 feet) and as close to the proposed bridge abutments as was practically feasible. The borings were drilled using mud rotary methods. We used the results of these borings primarily to develop recommendations for design and construction of the bridge abutment and wingwall foundations, and to evaluate the stability and settlement of the bridge approach embankments that will be raised.

We advanced the four pavement cores, BT-6 through BT-9, to depths ranging from 1.1 to 4.2 feet below top of pavement of 131st Avenue NE. We used a 6-inch core drill to bottom of pavement followed by hand augering and used the information from the cores to develop design and construction recommendations for overlay of existing pavement and for new pavement where the roadway will be widened.

We backfilled the boreholes in non-pavement areas with bentonite chips followed by native soils within 1 foot of ground surface. In paved areas, we backfilled Borings BT-1 and BT-3 with bentonite chips and patched the surface with fast-setting concrete dyed black, and backfilled Boring BT-2 with materials for a 2-inch-diameter groundwater monitoring well (see boring log and well construction log for details). We backfilled the pavement cores with the excavated soils (tamped) followed by black-dyed fast-setting concrete.

4.3 Field and Laboratory Testing

During the 2023 test borings, and beginning at 5 feet bgs, we obtained soil samples at 2.5- to 5-foot depth intervals using standard penetration testing (SPT). A larger diameter Modified California sampler (MC) was used to collect two samples in Boring BT-5 where sample recovery using the SPT sampler was poor. Standard penetration testing consists of driving a 2-inch-diameter split-barrel sampler 18 inches into the ground with a 140-pound hammer while recording the number of hammer blows to drive the sampler for each 6-inch interval. The number of hammer blows to penetrate the last 12 inches is called the SPT "N-value." The results of SPT are empirically correlated with physical and engineering properties of soils. Our field testing also included screening soil samples for the potential presence of volatile organic compounds (VOCs) using an organic vapor meter/photoionization detector (PID). The SPT raw penetration resistance values (N-values) are shown at the sampling depths on the estimated soil profile depicted in **Figure 4** and on the boring logs in **Appendix B**. Results of PID screening are shown on the boring logs.

Selected soil samples collected during the 2023 field investigation were tested for moisture content, Atterberg Limits, gradation (sieve analysis), and corrosivity parameters. The moisture content and fines content from the gradation test results are shown on the boring logs in **Appendix B**. The laboratory testing results are included in **Appendix D**.

During previous geotechnical investigations conducted by WSDOT and Zipper Zeman Associates, selected samples from the borings were obtained at about 2- to 5-foot depth intervals using the SPT procedure (in borings) or by collecting a grab sample (in test pits). Raw penetration resistance values (N-values) are shown on the boring logs in **Appendix A**. The resistance N-values for the four borings by Zipper Zeman Associates (2005) along the trestle bridge approach embankments are shown in **Figure 4**. Results of laboratory testing on samples from the previous borings are included following the borings logs in the respective sub-appendices in **Appendix A**.

As part of the environmental investigation for the Woodinville Water District's sanitary sewer across 131st Avenue NE, we submitted soil and groundwater samples to our subcontracted laboratory Fremont Analytical for analytical testing. The testing program included analyses for gasoline, diesel and heavy oil-range petroleum hydrocarbons, and VOCs. The analytical results are included in the environmental investigation report in **Appendix E**.

5. SITE AND SUBSURFACE CONDITIONS

5.1 Site Description

The proposed project alignment along SR 202/131st Avenue NE is gently sloped with the paved ground surface ranging from Elevations +31 feet to +37 feet. The ground surface of the top of the existing trestle bridge approach embankments, within about 200 feet of the abutments, ranges from approximately Elevations +49 to +51 on the west side of the bridge, and from approximately Elevations +51.5 to +53 on the east side of the bridge. Beyond 200 feet from the abutments, the ground surface varies from about Elevations +32 to +49 west of the bridge, and from about

Elevations +54 to +60 east of the bridge. The embankment ground surface consists of railbed aggregate (rail tracks and ties were removed in fall 2023), lightly to heavily vegetated ground, and bare earth. Much of the former railroad right-of-way consists of embankment up to about 20 ft high. The embankment side slopes become flatter with distance from the bridge and generally have inclinations of 2H:1V or flatter, though there are localized steeper areas where the inclination ranges from 1.8H:1V to 2H:1V.

5.2 Geologic Setting

Published geologic mapping for the Site is presented in **Figure 5** at a 1:100,000-scale. According to the Geologic Map of the Bothell Quadrangle, Snohomish and King Counties, Washington (Minard 1985), the geologic setting of the Woodinville area is primarily influenced by its location within the Puget Lowland, a part of the larger Puget Sound Basin which is a north-south trending depression bounded on the east and west by Cascade and Olympic Mountain ranges. The Puget Lowland is a geologic province characterized by a complex history of tectonic activity and glaciation. The region has experienced multiple cycles of glaciation during the Pleistocene epoch, which has had a significant impact on shaping the landscape. The underlying geology of the Woodinville area primarily consists of sedimentary deposits, including glacial till, outwash, and alluvial fan deposits. Glacial till is a mixture of clay, silt, sand, gravel, and boulders that were deposited directly by the glaciers. Outwash deposits, which are typically coarser and more sorted, were transported and deposited by glacial meltwater streams. Alluvial fan deposits are often found in the valleys and consist of sediments deposited by rivers and streams. Additionally, the Woodinville area is underlain by older bedrock formations, primarily of volcanic and sedimentary origin. The bedrock in the region includes basalt flows from ancient volcanic activity, as well as sandstone, siltstone, and shale formations.

As summarized in ZZA (2005), in the vicinity of the project site, the glacial sediments are typically overlain by more recent alluvial sediment associated with the Sammamish River, and to a lesser extent the North Creek and Bear Creek tributaries. The recent sediment was deposited following glacial retreat as the Sammamish River valley infilled with river flood plain deposits. The alluvium consists mostly of sand and is underlain by glacially consolidated coarse sand and gravel.

The depth to bedrock map by Yount et al. (1985) for the Seattle area shows bedrock at the Site is located 200 meters (approximately 660 feet) below ground surface. Therefore, we do not anticipate that bedrock will be encountered during construction. However, this does not preclude the possibility of encountering boulders when excavating or drilling into native glacial deposits.

5.3 Site Subsurface Conditions

Interpreted subsurface conditions at the Site based on the results of AECOM's investigation and previous investigations by others noted previously are discussed in the following sections.

5.3.1 Existing Pavement

The results of the pavement cores along SR 202 are summarized in Table C-1 (**Appendix C**.) The results suggest there are two distinct pavement sections between NE 175th Street and NE 177th Street:

- Pavement section consisting of 6.5 to 9 inches asphalt concrete (AC) pavement over gravelly sand base course/subgrade soils. This section was encountered in the two pavement cores near the south end of 131st Avenue NE between NE 175th Street and NE 177th Place.
- Pavement consisting of 11 to 17 inches AC pavement (plus 8 inches asphalt treated base (ATB) at one location, Core HA-1(ZA05)) over 6 to 7 inches Portland cement (PC) pavement, overlying gravelly sand or crushed rock over gravelly sand base course/

subgrade soils. This section was encountered in the four pavement cores in the northern half of 131st Avenue NE between NE 175th Street and NE 177th Place. (Note: Core BT-9, located under the bridge, was cored through the AC pavement but then terminated once the presence of the underlying PC pavement was confirmed, i.e., the full thickness of PC pavement was not penetrated.)

The information from the six cores indicates the transition between the two pavement sections is located between Cores BT-7 and BT-8, i.e., between 131st Ave. NE Sta. 104+00 and Sta. 105+25. To refine the location of the transition, we reviewed WSDOT as-built drawings titled "State Highways Nos. 2 and 1-A, Woodinville Vicinity," dated April 15, 1963 (SR 202 Widening Improvements; several of the drawings contain the note "As-Built Changes in Red 3-10-65"). Our review showed that the pavement section transition is at approximately 131st Avenue NE Station 104+50.

The pavement cores encountered cold joints at varying depths in the AC pavement, indicating overlays had been placed since construction of the widening improvements.

During utility clearance activities before initiating coring, our subcontracted ground penetrating radar (GPR) survey detected multiple objects about 4 feet below top of pavement north and south of Core BT-7. In the GPR image in Figure C-1 of Appendix C, these objects can be seen as inverted parabolas spaced at approximately 7 to 8 feet; the top of the parabolas correspond to the approximate depths of the objects. Our review of the 1963 SR Widening Improvements' "Drainage & R/W Plan" shows that a network of east-west aligned 6-inch-diameter perforated metal drain pipes was installed below the roadway. The drawing shows the pipes at spacings of 14 feet on centers in the east and west sides of the roadway. The east and west pipes overlap by 8 feet near the middle of the roadway. These findings suggest that the objects detected by the GPR survey comprise the drain pipes where they overlap. The network of drain pipes extends from approximately NE 131st Avenue Stations 104+75 to 106+90, as shown in **Figure 3** (also see the figure in Table C-1). Thus, the drain pipes may comprise obstructions during construction that includes excavation in that area.

5.3.2 Site Soils

The subsurface soil profile, interpreted conditions, and SPT N-values along the replacement bridge alignment are illustrated in **Figure 4**. A summary of the soil deposits encountered from the ground surface at the top of the railroad embankment to the boring termination depths is presented below. The soil deposits have been numbered to simplify references to them throughout the report.

Stratum 1: Existing Fill

The current and historical borings encountered embankment fill to depths of about 16 to 20 feet bgs along the existing approach embankments, with corresponding range of approximate bottom Elevations (EL) +32 to +35 feet. Beneath a thin (6 to 8 inches) veneer of railroad ballast, the embankment fill generally consists of loose to medium dense, moist to wet, gray to brown, fine to medium grained sand with varying amounts of silt and gravel. In contrast, Boring BT-4 encountered medium to stiff, moist, gray silt to lean clay with sand, trace gravel from the bottom of the ballast to at least 6 feet bgs. The SPT N values ranged from 2 to 14 per foot.

Outside the footprint of the existing approach embankments (i.e., on 131st Avenue underpass and adjacent private properties), Stratum 1 was generally encountered to depths of 5 feet or less with similar bottom elevations as encountered within the existing approach embankments.

Stratum 2: Loose to Dense Sand with Gravel and Silt

Below the embankment fill, native alluvial soils are present sitewide to depths ranging from about 25 to 47 feet bgs, with corresponding ranges of approximate bottom EL +14 to +26 feet at the west approach embankment and EL +4 to +13 feet at the east approach embankment. This deposit

consists of loose to dense (predominantly medium dense to dense), moist to saturated, gray to brown sand with gravel and silt, or gravel with sand and silt. The sand gradation varies from fine to medium to medium to coarse. In Boring BT-4, gravel with sand and silt was encountered from about 20 to 33 feet bgs. Standard penetration test N-values ranged from 3 blows per foot (bpf) to 50 blows per 5 inches, but more typically from about 12 to 35 bpf.

Stratum 3: Dense to Very Dense Sand with Gravel and Silt

The borings along the embankment terminated in dense to very dense alluvial soils consisting of saturated, brown to gray, medium to coarse grained sand with varying amounts of silt and gravel. The SPT for the borings by ZZA (2005) typically indicated very dense soils with N-values of 50 blows for 2 to 8 inches (i.e., refusal). The SPT N-values for the current borings indicated dense to very dense soils, with N-values generally ranging from 30 to 65 bpf, but no refusal N-values. One atypical low N-value of 17 bpf was encountered in Boring BT-4 at 80 feet bgs, and one high N-value of 81 bpf was encountered in Boring BT-5 at 60 feet bgs.

5.3.3 Groundwater

During the current field investigation, groundwater was encountered in Borings BT-1, BT-2, and BT-3 at depths ranging from 7 to 10.7 feet bgs during drilling. At Borings BT-4 and BT-5, the mud rotary method of drilling used prevented the accurate detection and measurement of groundwater levels. We measured groundwater levels in the monitoring well installed in Boring BT-2 as well as in the monitoring well installed previously by ZZA (2005) in Boring B-4. The results of the monitoring from late October 2022 to early August 2023 are shown in **Figure 6**. The data show that groundwater levels varied from approximately Elevation +23 feet during the dry months of the year (late summer to middle of fall), to approximately Elevations +25 to +26 during the wetter winter months. The data also show that the groundwater level at Well B-4 was consistently lower (by about 0.5 feet) than the groundwater level in Well BT-2. Based on this limited data, groundwater appears to flow in a westerly direction, i.e., toward the Sammamish River.

In 2004, ZZA (2005) had also installed a monitoring well in Boring B-1 at the top of the railway embankment on the east side of the trestle bridge. We located the well at the Site in early 2023 but found the surface monument had been damaged such that it was not possible to measure the groundwater level in the well. Zipper Zeman Associates had measured the groundwater level at 27 feet bgs in this well during June and July 2004, corresponding to Elevation +24, which is similar to the groundwater elevations we observed in the 2023 borings and Well BT-2.

Groundwater levels and soil moisture conditions should be expected to vary throughout the year due to fluctuations in seasonal precipitation and temperature, construction activities, and other on- and off-site factors.

5.3.4 Laboratory Testing Results

The results of sieve analysis, water content, Atterberg limits, and corrosivity testing are included in **Appendix D**. The results indicate the following:

- Sieve analyses of the field-classified silt/clay with sand and silty sand in the uppermost approximately 12 feet of Boring BT-4 on the west side of the bridge show this material consists of 40 to 51 percent fines (i.e., material passing the No. 200 sieve) with moisture contents ranging from 16 to 40 percent. The silty sand and silty gravel soils below this depth, from 15 to 30 feet bgs, contain about 6 to 14 percent fines with a moisture content ranging from 8 to 15 percent. An Atterberg limits test on the cohesive sample from 5 feet bgs indicates the soil has medium plasticity (plasticity index, PI, of 25).

- At Boring BT-5 on the east side of the bridge, sieve analyses of the granular soils in the upper 12 feet show this material contains about 5 to 6 percent fines with moisture contents ranging from 5 to 17 percent. The sandy soils below this to about 30 feet bgs contain about 7 to 8 percent fines with moisture contents of 16 to 35 percent.
- The deeper Stratum 3 soils have a fines content ranging from 6 to 23 percent, with moisture content ranging from 10 to 35 percent.

We performed corrosivity testing on a near-surface cohesive soil sample in Boring BT-4 (5 to 6.5 ft bgs, clay fill) and two deeper granular soil samples in Borings BT-4 and BT-5 (25 to 26.5 bgs, sand with silt and gravel). The elevation of the deeper samples corresponds to approximately the depth of the proposed foundations for the replacement bridge. The test results indicate the following:

- pH: 7.1 to 8.5
- Chloride: 21 to 24 ppm
- Sulfate: 51 to 66 ppm (0.0051 to 0.0066 percent by mass)
- Electrical Resistivity: 1,725 ohm-cm for clay sample; 10,736 to 12,702 ohm-cm for granular soil samples.

The results show that when compared to generally accepted criteria for assessing ground corrosion potential, corrosivity generally does not appear to be a concern at the site for concrete and steel structures, given the approximately neutral pH levels, low chloride and sulfate levels, and, for the granular soil samples, high resistivity (i.e., greater than 5,000 ohm-cm). For the shallow clay fill, the low resistivity result (i.e., less than 2,000 ohm-cm) does indicate a strong corrosion potential. In accordance with American Concrete Institute (ACI) 318 Building Code Requirements for Structural Concrete, the sulfate exposure class is S0.

5.3.5 Environmental Investigation

The limited environmental investigation performed in early 2023 for the Woodinville Water District detected low concentrations of volatile organic compounds in soil (Perchloroethylene) and groundwater (Perchloroethylene and Toluene) samples from Boring BT-3, on the west side of 131st Avenue NE. Low concentrations of diesel-range hydrocarbons were also detected in a groundwater sample from BT-1, on the east side of 131st Avenue NE. The detections will impact construction activities occurring in these areas. Refer to the investigation report in **Appendix E** for further information.

5.4 Geologically Sensitive Areas (Critical/Hazard Areas)

The Woodinville Critical Areas Map Woodinville 2016) shows Geologically Sensitive Areas throughout the City, including Erosion, Landslide, and Seismic Hazard Areas. The map shows that the Site is in a Seismic Hazard Area, and that NE 177th Place is the southern limit of an area mapped as a Frequently Flooded Area, which is associated with Little Bear Creek. (Note: In June 2025, the City permit reviewer informed AECOM that the Critical Areas Map contains an inaccuracy in the limits of the Frequently Flooded Area. According to the current effective Federal Emergency Management Agency Flood Insurance Rate Map for King County, Washington, and Incorporated Areas (Map Number 53033C0068H, effective 8/19/2020), no portion of the project occurs within frequently flooded areas (FEMA 2020).

We also reviewed geologic hazard mapping by the WADNR and USGS, which shows the following:

- Landslide Hazard: The Site does not contain mapped landslides. The nearest mapped landslides are located approximately one-half mile south-southwest of the Site. Although the existing railroad embankments have sideslopes with inclinations as steep as about 1.8H:1V (55 percent), they are not generally prone to landslide-like movements.
- Earthquakes Hazard: The Site is in an area mapped as having moderate to high liquefaction susceptibility. The area coincides with the mapped Quaternary alluvium geologic unit (Qa); refer to Figure 5. Seismic considerations are further discussed in Section 6.2.
- Volcano Hazards: The Site is not in a volcanic hazard area. The mapped volcanic hazard areas closest to the Site are located approximately 30 miles north and south of the Site and are associated with the Glacier Peak and Mount Rainier volcanoes.

6. CONCLUSION AND RECOMMENDATIONS

AECOM recommendations for SR 202 widening and trestle replacement and construction considerations as provided below are based on the information summarized above, engineering analyses, and interpretations and engineering judgement.

6.1 Soil Engineering Parameters

The soil engineering parameters recommended herein were developed based on the current and previous subsurface explorations, laboratory data discussed above, and published empirical correlations using a combination of:

- Evaluation of boring logs and SPT N-values vs. depth;
- Published correlations between soil type, soil density/consistency, laboratory testing results and the parameters of interest, as highlighted in the literature excerpts included herein; and
- Engineering judgment based on experience at other sites with similar subsurface conditions.

Table 1 presents recommended soil engineering parameters for design. The recommended parameters assume that soil conditions between the borings and near the proposed structures are like those encountered in the borings.

Table 2 provides recommended soil engineering parameters for various types of fill materials based on presumptive design property ranges presented in Table 5-2 of the WSDOT GDM (2022).

Table 1. Recommended Geotechnical Engineering Design Parameters for In-Situ Soils

Parameter	Unit	Stratum 1	Stratum 2	Stratum 3
Description	(---)	Existing Fill	Medium Dense to Dense Sand with Gravel and Silt	Dense to Very Dense Sand with Gravel and Silt
USCS	(---)	SP, ML (Fill)	SP, ML, GP-GM	SP, SP-SM
Exist. Ground Surface Elevation, Top of Eastrail Embankment	(ft, NAVD88)	+49 to +54 (within 200±ft of bridge abutments)		
Exist. Ground Surface Elevation, 131st Ave. NE Roadway Level	(ft, NAVD88)	+31 to +36 (Sta. 103+50 to 107+00)		
Bottom Elevation of Stratum	(ft, NAVD88)	+35 to +32 West Design: +32 East Design: +31	+26 to +4 West Design: +16 East Design: +8	Below -26 to -52 West Design: -52 East Design: -52
Groundwater Elevation	(ft, NAVD88)	Existing: +23 to + 26.2 [3] For Design: +25		
SPT N_{field} [1]	(Blows/12")	2 – 14	12 – 50	30 – 50+
SPT N_{60} [1]	(Blows/12")	3 – 19 For Design[2]: 7	15 – 69 For Design[2]: 27	24 – 100 For Design[2]: 55
SPT $N_{1(60)}$ [1]	(Blows/12")	4 – 22 For Design[2]: 8	13 – 56 For Design[2]: 24	25 – 79 For Design[2]: 32
Total Unit Weight [2]	(pcf)	120	125	130
Saturated Unit Weight [2]	(pcf)	125	130	130
Friction Angle, ϕ' [2]	(º)	30	35	38
Elastic Modulus, E [2]	(ksf)	100	330	840
1D Constrained Modulus, E_s [2]	(ksf)	150	440	1,070
Poisson's Ratio, v [2]	(---)	0.35	0.30	0.28
P-y Curve Material Model (LPILE)	(---)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Horizontal Modulus of Subgrade Reaction, k (LPILE)	(pci)	40	110 (above GW) 80 (below GW)	120

Notes

1. SPT N-values based on the minimum and maximum for each stratum considering the six borings drilled from the top of the former railway embankments.
2. Recommended parameters based on the average (or 30th percentile values when the coefficient of variation is above 0.3) of the corresponding stratum considering the six borings drilled from the top of the former railway embankments, in combination with engineering judgment.
3. Groundwater Elevations +23 to +27 correspond to groundwater depths of approximately 5 to 9 feet below top of 131st Avenue NE pavement (EL +32) below the trestle bridge, and 24 to 28 feet below top of former railroad embankments near the trestle bridge (EL +51).

Table 2. Recommended Geotechnical Engineering Design Parameters for Different Fill Materials

Parameter	Unit	Common Fill for Embankments or General Grading (On-site)	Approach Embankment Select Fill (On-site)	Retaining Wall Fill (Imported)	Lightweight Fill (Imported)
Description	(---)	WSDOT Common Borrow	WSDOT Select Borrow	WSDOT Gravel Backfill for Walls (Note 2)	See Note 4
WSDOT 2025 Standard Specification Item No.	(---)	9-03.14(3)	9-03.14(2)	9-03.12(2) 9-03.14(4) (Note 2)	n/a
USCS	(---)	ML, SM, GM	GP, GP-GM, SP, SP-SM	GW, GP, SW, SP	See Note 4
Total Unit Weight	(pcf)	125	130	130	25
Friction Angle, ϕ'	(degrees)	32	36	38 (see Note 3)	41

Notes

1. Recommended parameters are for compacted fill.
2. For mechanically stabilized earth (MSE) walls, use WSDOT Gravel Borrow for Structural Earth Walls, WSDOT Standard Specification 9-03.14(4). The Gravel Borrow shall contain 5 percent maximum passing the No. 200 sieve and consist of angular to subangular particles.
3. A maximum value of 38 degrees can be used for non-MSE walls and MSE walls reinforced fill zone where no project-specific strength testing of the backfill source has been performed as allowed by WSDOT GDM Sections 15-5.3.4 and 15-3.7. Note that AASHTO BDS is more restrictive and states the maximum value without source-specific testing is 34 degrees (Section 11.10.6.2). Both WSDOT and AASHTO limit maximum value to 40 degrees when source-specific testing is performed; construction-phase testing by the Contractor may allow use of up to 40 degrees if supported by source-specific strength test data.
4. Ultra-lightweight Foamed Glass Aggregate (UL-FGA) by AeroAggregates of North America, LLC or approved equal by the Engineer.

6.2 Seismic Considerations

Seismic design considerations and recommendations for design are provided in the following paragraphs.

6.2.1 Regional Seismic Setting

The project area is influenced by the tectonic activity associated with the Cascadia Subduction Zone and nearby fault systems. Three significant seismic source zones in the broader region surrounding Woodinville, including the Cascadia Subduction Zone, Seattle Fault, and Regional Crustal Faults, have the potential to generate earthquakes (Bowman and Czajkowski 2019, 2014; WA DNR 2023; USGS 2023).

- Cascadia Subduction Zone: The closest part of the Cascadia Subduction Zone is located offshore, approximately 200 miles west of Woodinville. While this is a considerable distance, strong earthquakes originating from the subduction zone can still affect the region, albeit with reduced intensity compared to coastal areas.
- Seattle Fault: The Seattle Fault is located closer to Woodinville, aligned east-west beneath Puget Sound. The fault is located roughly 12 to 16 miles south to southwest of Woodinville. While the Seattle Fault's primary impact is in the Seattle metropolitan area, it can produce earthquakes that have the potential to affect Woodinville as well.

- Other Regional Crustal Faults: Woodinville is surrounded by several regional faults that can generate earthquakes. For example, the Site is located in approximately the middle of the approximately 4- to 7-mile-wide Southern Whidbey Island Fault Zone (SWIF), and the Darrington-Devils Mountain Fault Zone is located about 40 miles to the north. These faults, among others in the region, can contribute to seismic activity in the broader area.

Per the Liquefaction Susceptibility Map of King County, Washington by Palmer et al. (2004), the Site is located in an area delineated as having moderate to high susceptibility to liquefaction as a result of earthquake shaking. The area comprises alluvial plains associated with the Sammamish River and Littler Bear Creek, located west and north of the Site, respectively. Due to the relatively flat ground surface, the potential for seismically induced lateral spread as a result of liquefaction is negligible.

6.2.2 Seismic Design Parameters

The seismic design parameters in accordance with the General Procedure in the WSDOT GDM were evaluated as described in Chapter 6 of the GDM, which follows the AASHTO Guide Specifications for Load and Resistance Factor Design (LRFD) Seismic Bridge Design (2011) and the WSDOT BDM (2022).

AASHTO (2011) Section 3.4.2.1-Site Class Definition, and corresponding Table 3.4.2.1.1 Site Class Definition delineate an approach to specify the site class by incorporating the average shear wave velocity, average standard penetration test (SPT) N-value, or average undrained shear strength within the top 100 feet below the ground surface. Based on the existing 80-ft-deep subsurface explorations (ZZA, 2005) along the top of the Eastrail embankment, average N-values range from 2 bpf to refusal. According to AASHTO (2011) Table 3.4.2.1.1, and considering the analysis performed on data from the two 80- and 100-ft-deep borings (BT-4, BT-5) drilled from the top of the embankment in 2023, the Site is classified as Class D, stiff soil, with average blows/ft ranging between 15 and 37.

WSDOT GDM Section 6 and WSDOT BDM seismic requirements are based on AASHTO (2011) and define a Safety Evaluation Earthquake (SEE) and Functional Evaluation Earthquake (FEE). The SEE corresponds to a 7 percent probability of exceedance in 75 years with an approximate earthquake return period of 1,000 years, while the FEE corresponds to a 30 percent probability of exceedance in 75 years with an approximate earthquake return period of 210 years. Seismic design parameters were obtained from the USGS) Unified Hazard Tool using the Dynamic Conterminous U.S. 2014 seismic source model. Because the proposed bridge and retaining walls are classified as “Ordinary” (Not Lifeline) structures per AASHTO BDS Tables 4.1-1 and 4.5-1, only the SEE was evaluated for this project.

Section 6-3.2.1 of the GDM indicates that for seismic design based on the General Procedure, the site response spectrum shall be developed in accordance with the AASHTO (2011), except that the USGS 2014 deaggregation/ground motions shall be used to establish the acceleration coefficients (PGA, S_s, and S_1).

To provide a realistic ground motion scenario, AECOM evaluated the ground-motion hazard for the mean magnitude, M_w 7.11 event from the total ground-motion hazard using the deaggregation data available from the USGS website. **Table 3** provides the recommended seismic design parameters for the Site.

Table 3. Seismic Design Parameters

Parameter [1]	Identifier	Unit	Value for a Return Period ~1,000 Years
Site Class	(---)	(---)	D
Site Latitude	(---)	(---)	47.7553
Site Longitude	(---)	(---)	-122.1654
Peak Horizontal Ground Acceleration Coefficient [2]	PGA	(Fraction of g)	0.50
0.2 Sec Period Spectral Acceleration Coefficient [2]	S_S	(Fraction of g)	1.15
1 Sec Period Spectral Acceleration Coefficient [2]	S_1	(Fraction of g)	0.58
Site Coefficient for Peak Ground Acceleration [3]	F_{PGA}	(---)	1.10
Site Coefficient for 0.2 Sec Period Spectral Acceleration [3]	F_a	(---)	1.04
Site Coefficient for 1 Sec Period Spectral Acceleration [3]	F_v	(---)	1.72
Design Acceleration Coefficient	A_S	(Fraction of g)	0.55
Design Short-Period Acceleration Coefficient	S_{DS}	(Fraction of g)	1.20
Design 1-Second-Period Acceleration Coefficient	S_{D1}	(Fraction of g)	1.00

Notes
1. In accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design (2011).
2. In accordance with WSDOT GDM Maps (pages 6-18, 6-19, and 6-20) and USGS Unified Hazard Tool Website.
3. In accordance with WSDOT GDM Tables 6-4, 6-5, and 6-6.

6.2.3 Liquefaction and Seismic-Induced Settlement

Liquefaction is a phenomenon whereby saturated granular soils lose their inherent shear strength due to increased pore water pressures, which may be induced by cyclic loading such as that caused by an earthquake. A low relative density of the granular soils, shallow water, long duration and high acceleration of seismic shaking are some of the factors favorable for liquefaction to occur. Liquefaction usually does not manifest at the surface when it occurs at depths greater than 50 feet due to the larger overburden pressures at that depth.

The liquefaction potential analysis was performed by employing the computer program LiqSVs Version 2.3.1.5 produced by GeoLogismiki, which utilizes the SPT N-values or shear wave velocities along with fines content (percent passing the No. 200 sieve) and moist unit weight of the soil to evaluate the potential of liquefaction and post-liquefaction vertical cumulative displacement in susceptible soils.

The data from AECOM Borings BT-4 and BT-5, 2022 and the historical Borings B-1 to B-4 reported by Zipper Zeman Associates (2005) were used in our analyses. The Idriss and Boulanger (2014) method was selected for the liquefaction analysis. In this analysis, the potential for the soil to liquefy exists when the ratio of the Cyclic Resistance Ratio (CSR) to the Cyclic Stress Ratio (CRR) becomes less than one.

Our analysis indicates liquefaction-induced reconsolidation settlement of about 1.5 inches at Boring BT-4 (located about 40 ft west of the proposed west bridge abutment) within a 5-foot-thick interval at an approximate elevation of about -30 feet NAVD88. Ishihara (1985) in the Surface Manifestation method stated that the impact of a thin liquefiable layer can be negligible on the foundation located on an overlying non-liquefiable capping layer if the thickness of the capping layer exceeds the criteria presented in the author's "Liquefaction Induced Damage Chart". A comparison between the liquefiable layer in Boring BT-4 located at 55 ft below the bridge foundation level (bottom of footing approximately Elevation 25 ft), and peak ground acceleration according to the Liquefaction Induced Damage Chart (Ishihara 1985) indicates the available

thickness of the non-liquefiable capping layer is sufficient to minimize the impact of the liquefiable deep layer. Thus, it can be implied that a shallow foundation located at the top layer will not be impacted by the liquefiable layer in the event of an earthquake.

For the purposes of deep foundation design, we have interpreted the BT-4 thin liquefiable layer as a discontinuous, isolated pocket of lower density soil within Stratum 3 since the SPT blowcount was uncharacteristically low relative to other measured blowcounts within this stratum. Accordingly, we do not expect that liquefaction-induced settlement and associated downdrag will develop given the isolated nature of this lower-density zone and therefore were not considered in this evaluation. However, the potential presence of lower-density zones within two diameters of the tip elevation of deep foundations could reduce end bearing capacity for static and seismic loading and should be considered in deep foundation design and construction quality assurance / quality control (see subsequent sections of this report).

Due to the relatively flat ground surface and lack of horizontally continuous liquefiable layers, lateral spread as a result of seismic-induced liquefaction is not expected to occur at the Site.

6.3 Bridge Abutments

AECOM evaluated deep and shallow foundation alternatives for the proposed replacement bridge. The shallow foundation alternative evaluated for the east and west bridge abutments consisted of an integrated abutment retaining wall stem and buried slab-on-grade footing (i.e., reinforced concrete semi-gravity spread footing wall). Proposed bottom of footing (BOF) was at Elevation +25.5 and +24.9 feet for the west and east abutment footings, respectively, and both bearing in Stratum 2. Based on the design bearing pressures provided by the Structures team and estimated settlement under design loads discussed later, ground improvement of Stratum 2 would be needed to satisfy design criteria. Ground improvement was assumed to consist of stone columns for these analyses.

The deep foundation alternative evaluated for the east and west bridge abutments consisted of an integrated abutment retaining wall stem and shaft cap incorporating a two-row group of drilled shafts, with top of shafts / bottom of shaft cap at about Elevation +25 feet within Stratum 2. Based on the design axial and lateral shaft loads provided by the Structures team, the shaft tips would need to terminate within Stratum 3 as discussed later.

Based on the results of geotechnical analysis and anticipated settlement performance of the footings under static and seismic loads presented in the following sections, we recommend supporting the bridge abutments on deep foundations consisting of drilled shafts.

6.3.1 Shallow Foundation Design Methodology

The WSDOT GDM recommends utilizing the shallow foundation bearing capacity analysis method specified by the AASHTO Bridge Design Specifications (BDS). A resistance factor of 0.45 presented in AASHTO LRFD BDS, Table 10.5.5.2.2-1 was selected and applied to estimate the maximum factored bearing resistance of soil against the Strength Limit State. Section 8.11.2.4.1 of the GDM, in addition to the design considerations provided in the AASHTO LRFD BDS, recommends limiting the nominal bearing resistance for the strength and extreme event limit states to 120 ksf for foundation design in soil.

A permissible total settlement of 1.0 inch was considered for evaluating the performance of the bridge abutment shallow footings under service limit state loads in accordance with the structural design criteria, and consistent with WSDOT GDM Table 8-4, *Settlement Criteria for Bridges*.

6.3.2 Shallow Foundation Design Parameters

The soil parameters used to evaluate the bearing capacity of proposed shallow foundation are displayed in **Table 4** and **Table 5**. These parameters were developed based on the general soil profile at site, and in-situ and laboratory tests as discussed in Section 6.1. According to adjacent boring logs, the bearing stratum (Stratum 2) contains medium dense to dense sand with gravel and silt with SPT N_{60} values ranging from 15 to 69 blows-per-foot (bpf).

As mentioned previously, ground improvement assumed to consist of stone columns is needed to meet bearing and settlement criteria for the shallow foundation option. Analyses assumed 3-foot-diameter stone columns installed on a 5-foot x 5-foot rectangular grid. Assumed design parameters for the stone columns are summarized in the tables below.

Table 4. Soil Parameters for Shallow Foundation Design – West Abutment

Support Location	Design Ground Water Elevation (ft)	Stratum / Improvement	Approximate Elevation of Stratum / Improvement Zone (ft)	Total Unit Weight (pcf)	ϕ (deg)	C (psf)	E (ksf)	Es (ksf)	Poisson's Ratio
West Abutment (BOF Elev. 25.5)	+25	Stratum 2	+32 to +16	See Table 1					
		Stratum 3	+16 to -50	See Table 1					
		Stone Columns	+25.5 to +13	125 (total) / 135 (sat.)	40	0	2,400	3,860	0.35

Table 5. Soil Parameters for Shallow Foundation Design – East Abutment

Support Location	Design Ground Water Elevation (ft)	Stratum / Improvement	Approximate Elevation of Stratum / Improvement Zone (ft)	Total Unit Weight (pcf)	ϕ (deg)	C (psf)	E (ksf)	Es (ksf)	Poisson's Ratio
East Abutment (BOF Elev. 24.7)	+25	Stratum 2	+31 to +8	See Table 1					
		Stratum 3	+8 to -50	See Table 1					
		Stone Columns	+24.7 to +5	125 (total) / 135 (sat.)	40	0	2,400	3,860	0.35

6.3.3 Shallow Foundation General Data and Load Data

The general data and foundation loads for the proposed foundations provided by the Structural Engineers are summarized in **Table 6** and **Table 7**, respectively.

Table 6. Shallow Foundation General Data

Support Location	Finished Grade Elev. at Toe of Footing [1]	Finished Grade Elev. at Heel of Footing [2]	Bottom of Footing Elevation (ft NAVD88)	Footing Dimensions (ft)		Permissible Settlement under Service Load (inch)
				Width, B	Length, L	
West Abutment	+31	+57.9	+25.5	21.5	42.2	1.0
East Abutment	+32	+58.6	+24.7	27.5	42.2	1.0

Notes

1. Pavement surface on the 131st Ave NE underpass roadway level.
2. Top of Eastrail Trail embankment at bridge approach.

Table 7. Shallow Foundation LRFD Loads for Controlling Load Combination

Limit State	Total Factored Vertical Load (kips/LF)		Effective Footing Width (feet) Considering Eccentricity		Maximum Factored Bearing Pressure Demand (psf) [1]	
	West Abutment	East Abutment	West Abutment	East Abutment	West Abutment	East Abutment
Strength	131.2	168.8	16.4	22.5	8,000	7,500
Service	105.3	144.3	16.2	22.2	6,500	6,500
Extreme	205.7	196.4	18.7	18.7	11,000	10,500

Note

1. Based on effective footing width

6.3.4 Shallow Foundation Bearing Resistance

Foundation design recommendations for factored bearing resistance and estimated total settlements from foundation loading alone (excluding stress influence from surrounding abutment embankments) are summarized in **Table 8** considering cases both with and without stone columns ground improvement. Factored bearing resistances for cases without ground improvement were estimated according to the theoretical bearing capacity formula for general shear failure as presented in AASHTO BDM Section 10.6.3.1.2. Factored bearing resistance for cases with stone columns ground improvement was estimated using commercial software StoneC by GeoLogismiki, which uses the methods proposed by Priebe (1995). Settlements were also calculated using the StoneC software, which incorporates the semiempirical method developed by Schmertmann (1978) with adjustments by Priebe (1995) for cases with stone columns.

The factored bearing resistances for Strength and Extreme Limit States, which are based on general shear failure criterion, exceed the maximum factored loads and appear to meet design criteria. However, estimated settlements at the Service Limit State bearing pressure exceed the permissible maximum settlement of 1 inch both with and without ground improvement, and therefore the Service Limit State is the critical limit state. Accordingly, **Table 8** also presents settlement-limited bearing resistances for the Service Limit State corresponding to 1 inch of settlement. Based on the relatively high levels of estimated settlement and the anticipated

complexity of the construction sequence for this project, a more sophisticated staged settlement analysis was also performed (see Section 6.3.5). The foundation designer should compare the ratio of recommended nominal bearing resistance to applied factored loads (i.e., Capacity to Demand Ratio, CDR) to specify the governing design case against the proposed shallow foundation specification. A CDR less than 1.0 indicates the failure of the design case against applied loads.

Table 8. Bridge Abutment Shallow Foundations Bearing Resistance Summary

Support Location	Footing Size (ft)		BOF (ft NAVD88)	Min. Footing Embed. (ft bgs)	Ground Improve-ment	Limiting Failure Mode for Bearing Resistance	Est. Max. Total Settlement (inches)[1]	Factored Bearing Resistance (ksf) Per Each Limit State[3]		
	B	L						Service[2]	Strength[2]	Extreme[2]
West Abut.	21.5	42.2	+25.5	5.5	None	Shear	3.4	6.5	25.7	57.1
						Settlement	**1.0**	**1.8**	**25.7**	**57.1**
					Stone Columns	Shear	1.7	6.5	28.2	62.8
						Settlement	**1.0**	**5.8**	**28.2**	**62.8**
East Abut.	27.5	42.2	+24.7	7.3	None	Shear	4.2	6.5	33.2	73.9
						Settlement	**1.0**	**1.4**	**33.2**	**73.9**
					Stone Columns	Shear	2.1	6.5	32.9	73.1
						Settlement	**1.0**	**3.4**	**32.9**	**73.1**

Notes:
1. Considers settlement of the footing alone; does not account for load influence of adjacent embankments and associated settlements.
2. Resistance factors (ϕ_b) of 1.0, 0.45, and 1.0 were applied to the estimated nominal bearing resistance for service, strength, and extreme limit states, respectively.
3. Recommendations are based on the foundation geometry and the loads provided by Structure Design in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.

6.3.5 Shallow Foundation Settlement

Based on the results presented subsequently in this section, the performance of shallow foundations is marginal at the anticipated Service Limit state loading and may exceed permissible settlements. Excessive settlements could compromise bridge integrity and/or serviceability, and/or result in inadequate drainage of the bridge deck. Accordingly, support of the bridge will need to be accomplished using either 1) shallow foundations with reduced footing pressures and embankment loads (e.g., using lightweight fill), or 2) deep foundations. Considering the risks of excessive settlement and the high importance of this project, we recommend deep foundations for the bridge abutments.

Proposed Improvements, Anticipated Construction Sequence, and Criteria

The maximum grades along the trail embankment will be altered and raised by about 7 to 7.5 feet at the bridge approaches on either side. The sideslopes of the raised embankment will be flattened from 2H:1V to 3H:1V, requiring retaining walls near the right of way limits and associated raise of the embankment by up to about 10 feet at the wall locations. To construct the new bridge footings and retaining walls, it is anticipated that most of the existing embankment will be removed to surrounding road grade level (Elevation +31 to +32), with locally deeper excavations to BOF elevation at the footings. Based on discussions with the design team regarding anticipated construction sequence, the bridge footings and substructures would be installed and the approach embankments then constructed to interim grades of about Elevation +48 and +49 at the west and east abutments, respectively, to provide a crane working platform needed to construct the bridge superstructure. Following bridge superstructure installation, the embankments would be constructed to final grade.

Based on discussions with the structures team, the critical settlement criterion for this project is to limit total settlement of the bridge to 1 inch or less. Settlements of the granular soils are expected to occur immediately as embankment and bridge loads are applied, and therefore settlements occurring before the bridge superstructure is installed can be accommodated by compensatory grouting of the bridge bearings to set the bridge at the correct elevation.

Analysis Methodology

AECOM evaluated the potential settlement of shallow foundations associated with loads acting on the footings themselves, and the loading from proposed adjacent embankment fills for the abutments. The foundation total settlement is a function of both the bearing pressure on the footing, and the loading imposed by the surrounding embankment fill. The adjacent embankment fill will generally increase settlement as compared to settlement of the footing alone, as the widened footprint of surface loading produces additional stress increases at depth.

Subsurface explorations indicate the existing site soils are primarily granular, and we anticipate imported fill to construct the embankments will also be granular in accordance with WSDOT specifications. Therefore, settlement is anticipated to occur immediately after the load placement because of the elastic immediate settlement of granular soils. The AASHTO LRFD BDS, Section 10.6.2.4.2c specifies a semiempirical method developed by Schmertmann (1978) for sandy soils which was utilized to evaluate the immediate settlement for this project. In addition to the proposed shallow foundation dimensions provided in **Table 8**, the approach embankment footprint of approximately 120 feet wide (including retaining walls) by 250 feet long was assumed at each abutment for the purpose of settlement analysis. The settlement analysis was conducted at the service limit state maximum bearing pressure of 6,500 psf.

It is recognized that the existing embankments at the bridge abutments provide a degree of pre-loading of the bearing strata, which tends to decrease total settlement associated with the proposed loads. Accordingly, the beneficial pre-loading effect was considered in the settlement calculations by performing staged settlement analyses using commercial software Settle3 by Rocscience. The Settle3 analysis incorporated the Westergaard stress distribution method for immediate settlement as recommended by the software technical manual. Use of Settle3 software also allowed for evaluation of the anticipated complex construction sequence and incremental settlements that may occur during the various stages of construction.

Results and Discussion

The analysis stages and resulting incremental settlements considering the Service Limit state bearing pressure of 6,500 psf for cases both with and without stone columns ground improvement are summarized in **Table 9**.

For the west and east abutments, estimated cumulative settlements experienced by the bridge footings with stone columns range from 2.6 to 3.2 inches, respectively, and without ground improvement from 2.9 to 3.7 inches, respectively. As expected, the cumulative settlements are slightly higher than estimated using simplified analyses accounting only for footings loads presented previously (i.e., about 0.3 inches greater without ground improvement and about 1.5 inches greater with stone columns). The settlements occurring between Stages 5 and 7 of the analysis (which simulate installation of the bridge superstructure and embankment final grade construction) should be limited to 1 inch or less per input from the structures team. However, based on the results presented in the table below, estimated total settlements for bridge superstructure alone range from 1.0 to 1.1 inches (with stone columns) and 1.1 to 1.4 inches (with no ground improvement) and thus exceed the permissible settlement limit.

Table 9. Summary of Staged Settlement Analysis for Bridge Abutment Footings

Stage No.	Stage Name	Stage Description	Model Cumulative Construction Settlement at Center of Bridge Footing at BOF Level (inch) [1]			
			West Abutment		East Abutment	
			No Ground Improve	Stone Columns	No Ground Improve	Stone Columns
1	Existing conditions	"Back-analysis" of existing embankment (ignores existing footings). This settlement has already occurred in the field.	1.1	1.1	1.3	1.3
2	Excavate bridge footing	Removes the existing embankment to El. +32 (east) or +31 (west), and creates a square excavation to BOF in new bridge footing footprint.	-0.3 (heave)	-0.3 (heave)	-0.4 (heave)	-0.4 (heave)
3	Install stone columns	Installs a 5'x5' square grid of 3' diam. stone columns, with top elev. at BOF and bottom elev. embedded 3' into Stratum 3.	n/a	-0.5	n/a	-0.4
4	Construct bridge footing / substructure	Installs bridge footing at BOF elev. Applied load is 68% of the 6,500 psf service load, simulating construction of substructure only (including soil above).	1.4	1.3	1.9	1.7
5	Interim embankment	Installs the interim grade embankment to max El. +48 adjacent to the footing. Footing loads include the weight of soil backfill directly above the footing.	1.8	1.6	2.3	2.1
Foundation settlement prior to bridge install (Stages 2 to 5)			**1.8**	**1.6**	**2.3**	**2.1**
6	Install bridge superstructure	Increases the applied footing load from 68% to 100% of the 6,500 psf total load, simulating construction of the bridge superstructure. Footing loads include the weight of soil backfill directly above the footing (to final grade).	2.6	2.3	3.4	2.9
7	Final embankment	Installs the adjacent final grade embankment and retaining walls with 3H:1V backslopes to max El. +58.5.	2.9	2.6	3.7	3.2
Foundation total settlement during/after bridge superstructure installation (Stages 5 to 7)			**1.1**	**1.0**	**1.4**	**1.1**
Differential settlement between abutments during/after bridge superstructure installation (Stages 5 to 7)			**No ground improve = 0.3 inch Stone columns = 0.1 inch**			

Note

1. Analysis conducted considering Service Limit State bearing pressure of 6,500 psf on bridge footing.

6.3.6 Shallow Foundation Lateral Load Resistance

Resistance to lateral loads may be developed by a combination of sliding friction acting at the base of the footings and passive earth pressure developed on the face of the footing as allowed by Section 10.6.3.4 of the AASHTO BDS. If frictional and passive resistances are combined, only the mobilized passive resistance corresponding to the permissible horizontal movement of the footing should be used in combination with the frictional resistance. The mobilized passive resistance for design is generally assumed as about 50 percent of the ultimate passive resistance for strain compatibility reasons, since the full passive resistance is mobilized only after

relatively large lateral movements have occurred as compared to the smaller movements required to mobilize the full frictional resistance.

Shallow Foundation Passive Resistance

Earth pressure coefficients for passive soil resistance of proposed shallow foundations were estimated using the log-spiral method as summarized in Naval Facilities Engineering Command (NAVFAC) Design Manual (DM) 7.02 (NAVFAC, 2025). The ultimate (nominal) passive resistance should be calculated as a triangular pressure distribution with an equivalent fluid weight equal to the buoyant unit weight of soil multiplied by the appropriate passive earth pressure coefficient. Recommended passive earth pressure coefficients for various soil units and geometric conditions are provided in **Tables 1** and **2**. The upper 1.5 feet of the passive earth pressure distribution should be ignored for passive resistance due to the potential for near-surface soil disturbance. The factored passive resistance should be obtained by multiplying the nominal passive resistance by a resistance factor of 0.50 per AASHTO BDS Table 10.5.5.2.2-1, which accounts for the mobilized passive resistance.

Shallow Foundation Base Shear Friction

If frictional resistance is used to resist lateral loads, we recommend using an ultimate coefficient of friction, μ, of 0.4 between the soil and foundation cast-in-place concrete. If a shear key is used for the shallow footings, the ultimate coefficient of friction can be increased to 0.67 to represent the internal soil-to-soil friction (tan ϕ, where ϕ is assumed to be 34^{o}). The ultimate (nominal) frictional resistance should be obtained by multiplying the ultimate coefficient of friction by the total vertical load imparted by the footing onto the soil; a reduction factor of 0.8 should also be employed for precast footings. The factored frictional resistance should be obtained by multiplying the nominal frictional resistance by a resistance factor of 0.80 per AASHTO BDS Table 10.5.5.2.2-1.

6.3.7 Settlement of Embankment Fills Adjacent to Shallow Foundations

We conducted a settlement analysis to estimate the potential settlement at the top of the bridge approach embankment due to the proposed embankment raise. The Settle3 software model used to estimate settlements for the bridge abutment shallow foundations was also used to estimate maximum settlement occurring within the subgrade below the proposed embankment fill. Additional self-weight compression of the new fill material during construction is not readily modeled in conventional settlement analyses but is generally assumed as about 1 to 2 percent of the height of the compacted fill based on published case histories.

Although the WSDOT GDM does not provide specific guidance for the allowable settlement within an embankment, the maximum immediate settlement in subgrades below the embankment fill during construction estimated from the Settle3 software was about 4 inches without ground improvement or about 2 inches with ground improvement. This settlement can be addressed by over-building and leveling the fill at the end of construction to match with the top of pedestrian bridge elevation. Additional self-weight compression of the new fill is estimated to be about 3 to 6 inches for the maximum embankment height of about 26.5 feet but is also expected to occur immediately as construction progresses and can be addressed by placing additional fill lifts during placement and compaction of the fill. Our analysis indicates that the differential settlement along the abutment footings and approach embankments under static loads for the shallow foundation option is less than 1 inch. The WSDOT GDM (2022), Table 8-4 provides various ranges of permissible settlement criteria for bridge structures.

Note that for the deep foundation option discussed in the next section, differential settlement between the bridge deck and adjacent embankment fill surface will likely be several inches but will occur during construction and can be accommodated by placing additional fill to design final grade.

6.3.8 Deep Foundation Design Methodology

The deep foundation type proposed by the structures team for support of the bridge is drilled shafts. Groups of drilled shafts connected by a shaft cap at each abutment are planned. The WSDOT GDM recommends designing drilled shafts according to the methods specified by the AASHTO BDS Section 10.8.

A permissible total settlement of 1.0 inch was considered for evaluating the performance of the bridge abutment deep foundations under service limit state loads in accordance with the structural design criteria, and consistent with WSDOT GDM Table 8-4, *Settlement Criteria for Bridges*.

6.3.9 Deep Foundation General Data and Load Data

Alternatives for design layouts of the drilled shaft foundations and associated limit state factored loads provided by the structures team are summarized in **Table 10**. The length of the shaft cap is 42.2 feet for each alternative, and the base of the shaft cap / top of drilled shafts is approximately Elevation +25 feet.

Table 10. Deep Foundation LRFD Loads for Controlling Load Combination

Drilled Shaft Diameter, D (feet)	Shaft Group Geometry	Shaft Cap Width (ft)	Center-to-Center Shaft Spacing (D)	Factored Axial Compression Loads (kips/Shaft)			Factored Axial Uplift Loads (kips/Shaft)		
				Service	Strength	Extreme	Service	Strength	Extreme
3	2 x 5	12.5	2.5D	650	800	750	n/a	-100	-200
4	2 x 4	16.0	2.5D	750	950	900	n/a	n/a	-50
5	2 x 3	19.5	2.5D	1,250	1,350	1,200	n/a	n/a	n/a
6	2 x 3	23.0	2.5D	1,450	1,600	1,400	n/a	n/a	n/a

6.3.10 Deep Foundation Axial Load Resistance

Axial load resistance for drilled shaft foundations for the bridge abutments was analyzed according to AASHTO BDS Sections 10.8.3.5.2 and 10.8.3.5.3, which for cohesionless soils specifies the use of the Beta method for side resistance and the SPT-based Brown et al. (2010) method for end bearing resistance. The commercial software SHAFT by Ensoft was used to estimate drilled shaft axial resistance for the west and east abutments considering several different shaft diameters. Recommended axial resistance curves for the various LRFD limit states are presented in **Appendix G**. A summary of recommendations for axial load resistance is provided in **Table 11**.

For design purposes, the axial resistance curves are presented to maximum depths corresponding to Elevations -89 and -88 feet for the west and east abutments, respectively. However, the available geotechnical borings data does not extend below Elevation -51 feet for the west abutment and Elevation -29.7 feet for the east abutment. Accordingly, drilled shafts which are designed to tip within two shaft diameters above Elevation -51 (west) / Elevation -29.7 (east) or deeper will require drilling pilot hole verification test borings to a minimum depth of two shaft diameters below the tip elevation prior to foundation installation. The purpose of the verification test borings is to confirm subsurface conditions are consistent with design assumptions. Depending on the results of the verification test borings, lengthening of drilled shafts may be required to bypass weak zones (if encountered). Recommendations for pilot hole verification test borings are provided in Section 6.9.9.

The mobilization of side resistance and end bearing resistance in drilled shafts is developed through settlement of the shaft in response to loading. Mobilization of the full end bearing resistance requires larger settlements than does mobilization of the full side resistance in cohesionless soils. Recommended axial resistance for drilled shafts herein were developed based on the following assumptions regarding load transfer and mobilization of axial resistance:

- Strength and Extreme Limit States: The nominal axial resistance was evaluated at a settlement to shaft base diameter (s/D) ratio of 5 percent per guidance in AASHTO BDS Section 10.8.3.5. Based on the normalized load transfer plots for cohesionless soils (AASHTO Figures 10.8.2.2.2-3 and 10.8.2.2.2-4), the mobilized side resistance and mobilized end bearing resistance for drilled shafts is approximately 100% of the ultimate values for s/D of 5 percent.
- Service Limit State: The nominal axial resistance is evaluated at the maximum permissible settlement of 1.0 inch, which corresponds to a s/D ratio of 1.7 percent for a 5-foot-diameter shaft. Based on the normalized load transfer curves for cohesionless soils in the AASHTO BDS, the drilled shaft mobilized side resistance is about 95% of ultimate and the mobilized end bearing resistance is about 50% of ultimate values for s/D of 1.7 percent.

Table 11. Drilled Shaft Foundation Axial Resistance Summary

Support Location	Shaft Diam. (feet)	Shaft Tip Elev. (ft NAVD88) [4]	Min. Shaft Length (ft) [1,4]	Shaft Settlement at Service Load (inch) [2]	Factored Axial Compression Resistance (kips) [3,4]			Factored Axial Uplift Resistance (kips) [3,4]	
					Service [4]	Strength [5]	Extreme [4]	Strength [6]	Extreme [7]
West Abutment	3	-44	69	1.0	653	827	1,541	503	894
	4	-30	55	1.0	755	1,016	1,916	523	929
	5	-56	81	1.0	1,261	1,683	3,150	976	1,734
	6	-54	79	1.0	1,455	1,988	3,722	1,145	2,035
East Abutment	3	-41	66	1.0	651	812	1,514	490	872
	4	-28	53	1.0	758	1,006	1,898	515	915
	5	-53	78	1.0	1,253	1,663	3,113	959	1,705
	6	-52	77	1.0	1,461	1,980	3,707	1,138	2,023

Notes
1. Assumes top of shaft / bottom of shaft cap at El. +25 feet (i.e., 6 ft bgs on west abutment and 7 ft bgs on east abutment).
2. Refer to Table 10 for corresponding service loads.
3. Axial resistance values presented herein do not include a disregard or exclusion zone for side resistance in the upper portion of the shaft because the bottom of the shaft cap is located greater than 5 feet below final grade, there is no anticipated scour, and the shafts bear entirely within cohesionless soils, consistent with AASHTO, WSDOT, and FHWA guidance.
4. Note that these values do not account for downdrag-imposed shaft loads and loss of skin friction in settling zone; see Section 6.3.13.
5. Resistance factor (ϕ_b) of 1.0 was applied to compression resistance for both service and extreme limit states. Service limit state resistance assumes 1.0 inch of settlement.
6. Resistance factors (ϕ_b) of 0.55 and 0.50 were applied to side resistance and end bearing resistance, respectively, for the strength limit state.
7. Resistance factor (ϕ_b) of 0.45 was applied to uplift resistance for the strength limit state.
8. Resistance factor (ϕ_b) of 0.80 was applied to uplift resistance for the extreme limit state.

6.3.11 Deep Foundation Axial Load Group Effects

Axial group effects on drilled shaft foundation groups generally reduce the maximum axial load that can be developed in an individual shaft. The bearing strata for the drilled shafts on this project will be entirely cohesionless soils, and thus the group reduction factors (η) in Table 10.8.3.6.3-1 of AASHTO BDS apply. For this project, we do not recommend a group reduction factor be incorporated into the drilled shaft design (i.e., η=1.0) based on the following provisions in the AASHTO BDS:

- The shaft center-to-center spacing exceeds twice the diameter (2D) for each of the proposed shaft group configurations.
- The bottom of the shaft caps will be embedded at least 6 to 7 feet below finished grade for both abutments.
- The shaft cap is expected to be in intimate contact with ground consisting of medium dense or denser cohesionless soil.
- The shafts are not located in an area of flowing water, and no scour below the shaft cap is expected.

6.3.12 Deep Foundation Lateral Load Resistance

We recommend lateral load resistance of deep foundations be evaluated according to the P-y method of soil-structure interaction using commercial software such as LPILE or GROUP by Ensoft, Inc. Recommended soil parameters for use in the P-y method of analysis are presented in **Table 1**. Total soil unit weights should be used above the design groundwater level and buoyant unit weights should be used below the design groundwater level. Based on the absence of identified continuous liquefiable strata near the bridge abutments, consideration of liquefied soil strength/stiffness is not necessary for lateral load resistance.

For closely spaced groups of drilled shafts, lateral group interaction effects reduce the available lateral resistance for individual drilled shafts. The use of “P-multipliers” is necessary to account for group effects, which serve to reduce both the ultimate lateral resistance and stiffness of the P-y curves. The AASHTO BDS Table 10.7.2.4-1 provides recommended P-multipliers according to the shaft center-to-center (CTC) spacing in the direction of loading, and the row number of the shafts. Since the AASHTO table does not provide P-multipliers for CTC spacings less than 3D and the proposed CTC spacing is 2.5D for this project, recommendations for P-multipliers were developed from the AASHTO BDS and the WSDOT GDM Section 8.12.32.3 and are presented below in **Table 12**.

Table 12. Recommended P-Multipliers for Drilled Shaft Group Lateral Resistance

Shaft Center-to-Center Spacing (in the direction of loading) as Ratio of Diameter, D	P-Multiplier, P_m		
	Row 1	Row 2	Row 3 and higher
2D	0.45	0.33	0.25
2.5D	0.63	0.37	0.27
3D	0.8	0.4	0.3
4D	0.9	0.63	0.5
5D	1.0	0.85	0.7

6.3.13 Deep Foundation Downdrag Loading

Downdrag loading (“negative skin friction”) develops when soils surrounding deep foundations settle differentially relative to the deep foundations. The AASHTO BDS recommends downdrag be assumed to fully develop if settlement in the soil layer is 0.4 inches or greater relative to the shaft. Based on approach embankment settlement analysis results presented in Section 6.3.7, subgrade settlements below the shaft cap are expected to exceed 0.4 inches. Accordingly, downdrag loading should be considered.

Downdrag only occurs when the settlements of surrounding soils exceeds the settlement of the drilled shafts. Given the estimated service limit settlements of about 1 inch for drilled shafts, settlements in Stratum 3 are expected to be less than those required to develop downdrag based on the estimated settlement versus elevation profiles developed in the approach embankment settlement calculations. Accordingly, downdrag loading is expected to occur only in Stratum 2.

Estimated unfactored downdrag loads are based on the peak skin friction developed within Stratum 2 and are summarized below in **Table 13**.

For the downdrag case, AASHTO BDS and WSDOT GDM conservatively specify the downdrag loads be treated as an external applied load for the structural Strength limit state, and the axial resistance in the settling stratum (Stratum 2) be ignored. Load factors for downdrag acting on drilled shafts are provided in Table 3.4.1-2 of the AASHTO BDS. Modified axial capacity curves accounting for downdrag and loss of support in the settling Stratum 2 are provided in **Appendix G**.

Note that the anticipated downdrag is associated with immediate settlement of the cohesionless subsurface soils caused by imposed stresses from the proposed raise of the adjacent approach embankment. Accordingly, the full downdrag loads will occur immediately during embankment construction.

If incorporation of downdrag loads is problematic for drilled shaft design, supplemental analyses and recommendations incorporating the neutral plane method can be performed upon request to reduce conservatism as allowed by AASHTO BDS Section 3.11.8. The neutral plane method (Briaud and Tucker 1993, Siegel et al. 2014) locates the intersection of two curves along the shaft profile – the sustained permanent dead load on the shaft plus the negative skin friction, and the mobilized tip resistance plus the positive side friction – to estimate the downdrag force and settlement of the shaft due to the surrounding soil. The neutral plane differs from the AASHTO and WSDOT explicit method of downdrag analysis in two key ways: 1) the downdrag load is treated as an internal load on the shaft for structural limit state analysis only; 2) the downdrag load is excluded in geotechnical limit state analysis, and the result of the method is an estimate of shaft settlement at the service limit state. Consequently, the neutral plane method generally produces a less conservative design than the explicit method cited in AASHTO and WSDOT.

Table 13. Estimated Static Downdrag Loads for Drilled Shaft Foundations

Support Location	Assumed Bottom Elev. of Downdrag Zone (ft NAVD88)	Unfactored Downdrag Loads (kips) for Varying Drilled Shaft Diameter (D) [1]			
		D = 3 foot	D = 4 foot	D = 5 foot	D = 6 foot
West Abutment	+16	92	122	153	183
East Abutment	+8	214	285	356	428

Notes

1. Assumes downdrag loading only occurs for portions of drilled shafts penetrating through Stratum 2.

6.3.14 Deep Foundation Static Settlement

Top settlement of drilled shaft foundations was estimated according to the load-transfer relationships and elastic compression of the shaft as presented in the AASHTO BDS based on estimated service loads provided by the structures team. Settlement results corresponding to the minimum shaft lengths needed to satisfy the various limit state axial loads are presented in **Table 11**. Results indicate drilled shaft settlement under axial loads should be about 1 inch or less.

6.3.15 Lateral Earth Pressures

The bridge abutment walls and other permanent retaining walls should be designed to resist the pressure exerted by retained soils, plus groundwater and/or any additional lateral forces due to loads placed adjacent to or near the wall.

Earth Pressures

Earth pressure recommendations in this section only pertain to non-gravity cantilever, semi-gravity, and gravity retaining wall types without anchors or internal bracing. Apparent earth

pressure diagrams should be used for anchored or braced retaining walls. Apparent earth pressure recommendations are excluded from this section and are addressed later in this report.

Flexible retaining walls in which the top of the wall is free to rotate or translate away from the soil mass by at least 0.2 percent of the wall height (assuming medium dense granular backfill) should be designed using active earth pressures per AASHTO BDS Section C3.11.1. At-rest earth pressures should be used for walls which have more stringent movement tolerances and/or cannot deflect/rotate sufficiently to develop active earth pressures.

Recommended geotechnical parameters for calculating lateral earth pressures for walls with vertical and battered backfaces are provided in **Table 14** and **Table 15**, respectively**.** The recommended lateral earth pressures are based upon the assumption that the retained native soils and/or backfill meet the material requirements listed, and adequate internal drainage is provided to prevent buildup of hydrostatic pressure behind the walls.

For static loading conditions, lateral earth pressures for retaining walls should be estimated as a triangular distribution with an equivalent fluid pressure equal to the appropriate static earth pressure coefficient (at-rest or active) multiplied by the effective unit weight of the soil. The effective unit weight of the soil should be taken as either the total unit weight above the groundwater table, or as the buoyant unit weight below the groundwater table. The resultant of the static lateral earth pressure distribution should be applied at one-third of the wall height (H/3) as measured from the bottom of the wall.

For seismic loading conditions, the lateral earth pressures should be estimated as a triangular distribution with an equivalent fluid pressure equal to the appropriate seismic earth pressure coefficient multiplied by the effective unit weight of the soil. The resultant of the seismic lateral earth pressure distribution should be applied at 0.4 of the wall height (0.4H) as measured from the bottom of the wall per AASHTO guidance[1]. The seismic earth pressure coefficients represents the total earth pressure (i.e., static plus seismic). The seismic active earth pressure coefficients (K_{ae}) were developed using the Mononobe-Okabe (M-O) pseudostatic method according to the provisions of AASHTO BDS Sections 11.6.5.2 and 11.6.5.3 as follows:

- For flexible walls which are capable of displacements of 1.0 to 2.0 inches during the design seismic event and movements of this magnitude are tolerable, the seismic horizontal acceleration coefficient, k_h should be estimated as ½ the site-adjusted peak horizontal ground acceleration, A_s (i.e., $k_h = 0.5 \times A_s$).
- For rigid walls which are capable of displacements less than 1.0 inch during the design seismic event and/or strict displacement tolerances are required, the k_h is equal to A_s (i.e., $k_h = 1.0 \times A_s$).

It is noted in AASHTO BDS Section C11.6.5.3 that for some combinations of wall backslope, soil friction angle, and seismic acceleration, the M-O equation does not converge to a real solution for the K_{ae} and/or K_{pe} parameters. This issue was encountered for several soil units when evaluating the 3H:1V retained backslope condition. Accordingly, the Generalized Limit Equilibrium (GLE) method was used to estimate appropriate values of K_{ae} in these circumstances according to the methodology presented in the NHRCP *Report 611* (Anderson et al. 2008) using the software program GeoStudio Slope/W by Seequent.

The seismic passive earth pressure coefficients (K_{pe}) were estimated from AASHTO BDS Figure A11.4-2 in accordance with the guidance of AASHTO Section 11.6.5.5 which recommends the M-

[1] Past practice considered the seismic component separately as an inverted trapezoid with seismic resultant at 0.6H above the wall base. More recent research cited in AASHTO indicates the combined static and seismic resultant force acts at H/3 above the wall base for seismic active pressures, although some limited studies indicate it may be as high as 0.4H to 0.5H for both flexible and rigid non-yielding wall types. The approach recommended herein is consistent with current AASHTO guidance.

O method not be used for estimating K_{pe} values. Final recommended values are presented in **Table 14** and **Table 15**.

Hydrostatic Pressures

If the retained earth is not allowed to drain, and/or where water is expected to pond behind a wall, the effect of hydrostatic pressure should be added to that of earth pressure unless the wall is designed for full hydrostatic pressure. The hydrostatic pressure should be analyzed as a triangular distribution with the unit weight of water and should be added to the lateral earth pressures per Figure C3.11.3-1 of AASHTO BDS. The resultant of the hydrostatic pressure distribution should be applied at one-third of the submerged wall height as measured from the bottom of the wall.

External Loads

Uniform vertical surcharge pressures should be added to the above lateral earth pressures when surcharge loads may be located above or adjacent to the wall. Surcharge pressures should be applied as a uniform rectangular lateral pressure distribution equal to the vertical pressure multiplied by the appropriate earth pressure coefficient. The resultant of the lateral surcharge pressure distribution should be applied at one-half of the wall height measured from the bottom of the wall. Effects of surcharges set back behind the wall a horizontal distance greater than the wall height need not be included in the design pressure. For this project, we recommend considering traffic loading as a live load surcharge with uniform vertical surcharge pressure equal to 250 psf. The designer is responsible for determining whether additional surcharge loads are applicable.

Concentrated vertical loads acting behind the retaining wall (i.e., point, line, and strip loads), if applicable, should be estimated according to the equations and diagrams presented in Section 3.11.6.2 of the AASTHO BDS.

Table 14. Lateral Earth Pressure Recommendations – Vertical Wall Backface

Retained Material [1]	Total / Buoyant Unit Weight (pcf)	Lateral Earth Pressure Coefficients Horizontal Backslope [2]						Lateral Earth Pressure Coefficients 3H:1V Backslope [2,3]					
		Ko	Ka	Kp[6,7,8]	Kae[4] Flexible	Kae[5] Rigid	Kpe[6,7,9] Flexible	Ko	Ka	Kp[6,7,8]	Kae[4] Flexible	Kae[5] Rigid	Kpe[6,7,9] FLexible
Stratum 1	120 / 57.6	0.50	0.30	5.7	0.54	1.30	4.0	0.66	0.40	2.3	0.91	1.74	---
Stratum 2	125 / 62.6	0.43	0.24	8.4	0.46	0.96	5.7	0.56	0.31	3.3	0.72	1.69	---
Stratum 3	130 / 67.6	0.38	0.22	10.8	0.42	0.85	7.4	0.51	0.27	4.1	0.62	1.47	---
WSDOT Common Borrow	125 / 62.6	0.47	0.28	6.6 (4.9)[7]	0.50	1.11	4.6 (2.8)[7]	0.62	0.36	2.7	0.81	1.78	---
WSDOT Select Borrow	130 / 67.6	0.41	0.23	9.1 (6.7)[7]	0.44	0.92	6.3 (3.8)[7]	0.54	0.30	3.5	0.69	1.62	---
WSDOT Gravel Backfill	130 / 67.6	0.38	0.22	10.8 (7.9)[7]	0.42	0.85	7.4 (4.5)[7]	0.51	0.27	4.1	0.62	1.47	---
Lightweight UL-FGA Backfill	25 / -37.4	0.29	0.19	11.0 (7.8)	0.38	0.77	8.7 (2.6)	0.45	0.19	5.3	0.52	1.30	---

Notes
1. See Tables 1 and 2 for material descriptions, specifications, and soil internal friction angles used to calculate earth pressure coefficients. Soil-wall interface friction angle assumed as 2/3 of the soil internal friction angle.
2. Assumes vertical backface of retaining wall. Abbreviations: Ko = static at-rest earth pressure coefficient, Ka = static active earth pressure coefficient, Kp = static earth pressure coefficient, Kae = seismic active earth pressure coefficient, Kpe = seismic passive earth pressure coefficient

3. Assumes backslope grades down towards the back of the wall on the retained side for active and at-rest earth pressures (positive angle), and foreslope grades down away from the front of the wall on the resisting side for passive earth pressure (negative angle).
4. Assumes k_h = 0.5 x As which is applicable to walls which are capable of 1.0 to 2.0 inches of deflection under seismic loading.
5. Assumes k_h = 1.0 x As which is applicable to walls which exhibit and/or can tolerate less than 1.0 inches of deflection under seismic loading.
6. A factor of safety of 1.5 should be applied to Kp and Kpe in passive pressure (resistance) calculations.
7. Value in parentheses () accounts for reduction due to 16 degree skew of proposed bridge.
8. Coulomb method Kp values per AASHTO BDS Figures 3.11.5.4.1-1 and 3.11.5.4-2.
9. K_{pe} values obtained from AASHTO BDS Figure A11.4-2 for horizontal foreslope only. Supplemental recommendations for sloping foreslope can be provided upon request.

Table 15. Lateral Earth Pressure Recommendations – Battered Wall Backface

Retained Material [1]	Total / Buoyant Unit Weight (pcf)	Lateral Earth Pressure Coefficients – Horizontal Backslope [2]						Lateral Earth Pressure Coefficients – 3H:1V Backslope [2,3]					
		Ko	Ka	Kp [6,7,8]	Kae [4] Flexible	Kae [5] Rigid	Kpe [6,7,9] Flexible	Ko	Ka	Kp [6,7,8]	Kae [4] Flexible	Kae [5] Rigid	Kpe [6,7,9] Flexible
WSDOT Select Borrow	130 / 67.6	0.41	0.28	7.4	0.51	1.05	6.5	0.54	0.36	2.9	0.94	2.23	---
WSDOT Gravel Backfill	130 / 67.6	0.38	0.26	10.2	0.48	0.98	8.02	0.51	0.33	3.3	0.82	2.31	---
Lightweight UL-FGA Backfill	25 / -37.4	0.34	0.24	14.4	0.44	0.90	11.4	0.45	0.29	4.1	0.69	2.45	---

Notes
1. See Tables 1 and 2 for material descriptions, specifications, and soil internal friction angles used to calculate earth pressure coefficients. Soil-wall interface friction angle assumed as 2/3 of the soil internal friction angle.
2. Assumes backface of retaining wall is battered at 1.25 inches horizontal per 1 foot vertical (5.95 degrees from vertical). Abbreviations: Ko = static at-rest earth pressure coefficient, Ka = static active earth pressure coefficient, Kp = static earth pressure coefficient, Kae = seismic active earth pressure coefficient, Kpe = seismic passive earth pressure coefficient
3. Assumes backslope grades down towards the back of the wall on the retained side for active and at-rest earth pressures (positive angle), and foreslope grades down away from the front of the wall on the resisting side for passive earth pressure (negative angle).
4. Assumes k_h = 0.5 x As which is applicable to walls which are capable of 1.0 to 2.0 inches of deflection under seismic loading.
5. Assumes k_h = 1.0 x As which is applicable to walls which exhibit and/or can tolerate less than 1.0 inches of deflection under seismic loading.
6. A factor of safety of 1.5 should be applied to Kp and Kpe in passive pressure (resistance) calculations.
7. Value in parentheses () accounts for reduction due to 16 degree skew of proposed bridge.
8. Coulomb method Kp values per AASHTO BDS Figures 3.11.5.4.1-1 and 3.11.5.4-2.
9. K_{pe} values obtained from AASHTO BDS Figure A11.4-2 for horizontal foreslope only. Supplemental recommendations for sloping foreslope can be provided upon request.

6.3.16 Retaining Wall Subsurface Drainage

Retaining walls should have adequate drainage behind the wall to avoid developing hydrostatic pressure from irrigation, leaking underground utilities, and/or surface runoff. Recommendations in this section pertain specifically to concrete retaining walls; drainage for other wall types are addressed in respective subsequent sections of this report.

Concrete retaining walls that are less than five (5) feet tall should have a perimeter drain at the base of the wall. The base drain should be a minimum six-inch-diameter perforated pipe. The pipe should be surrounded with at least one cubic foot per lineal foot of pipe of Gravel Backfill for Drains conforming with WSDOT Standard Specification 9-03.12(4) wrapped with geotextile conforming with WSDOT Standard Specification 9-33.2(1) Table 2.

Retaining walls greater than five (5) feet tall should have free draining material along the back of the wall in addition to the base drain. The free draining material should be at least 12 inches wide, as measured horizontally immediately behind the wall, and should consist of Gravel Backfill for Walls conforming with WSDOT Standard Specification 9-03.12(2).

Panel drainage systems, conforming with WSDOT Standard Specification 9-33.2(3), *Prefabricated Drainage Mat* may be used as an alternative to a pipe and filter drainage system. Panel drainage systems consisting of minimum 24-inch-wide strips placed every 10 feet on center are recommended.

Adequate weep holes or collector pipes need to be incorporated into the system to provide an outlet for the drained water. Drainage from collector pipes should be directed to a suitable outlet.

Existing or proposed water bearing utilities, surface conditions that could promote infiltration (e.g., irrigated landscaping) behind walls, and seeps encountered during construction may require additional subsurface drainage. An inclined drainage system (chimney drain) along the wall backcut should be considered in such circumstances.

6.4 Permanent Retaining Walls

Permanent retaining walls are planned along the north and south sides of the approach embankments parallel to the Eastrail Trail. The retained height of the walls ranges from approximately 15 to 25 feet. The walls will retain embankment backfill sloping at an inclination of 3H:1V. The walls have specific architectural / aesthetic criteria requiring cast-in-place concrete final facings with only vertical joint lines.

AECOM evaluated both concrete retaining walls and Mechanically Stabilized Earth (MSE) walls. The concrete walls considered are cast-in-place (CIP) reinforced concrete, semi-gravity cantilever spread footing retaining walls similar to WSDOT standard plan designs. Concrete walls both with and without stone columns ground improvement were evaluated.

The MSE wall type evaluated for this project is based on the WSDOT standard for “Geosynthetic Walls” which utilizes a two-stage construction procedure. In the first stage, a flexible geosynthetic-wrapped facing is used for initial wall erection. In the second stage, a permanent CIP concrete facing anchored into the reinforced soil mass is installed after settlement of the wall is complete. The flexible initial facing allows the wall to settle and move prior to installation of the rigid permanent facing to reduce risk of aesthetic or structural damage. The two-stage MSE wall construction using temporary facing was selected to accommodate estimated foundation settlement of between 3 and 4 inches caused by construction of the new bridge abutment, although lesser amounts of settlement are expected for the permanent retaining walls. According to the WSDOT Geotechnical Design Manual Table 15-2, flexible-facing MSE walls can accommodate total settlements of up to 4 inches and differential settlements of up to 3 inches over 50 feet without special design measures. Accordingly, ground improvement was not considered necessary if two-stage MSE walls are used for this project based on estimated settlements.

6.4.1 Concrete Retaining Wall Recommendations

Recommendations for the CIP concrete retaining walls are provided in the sections below.

Lateral Earth Pressures

The lateral earth pressure recommendations provided in **Section 6.3.15** are applicable for concrete retaining walls.

Internal Drainage

The internal drainage recommendations provided in **Section 6.3.16** are applicable for concrete retaining walls.

Lateral Load Resistance

The lateral load resistance recommendations provided in **Section 6.3.6** are applicable for the shallow foundations supporting concrete retaining walls.

Foundation Bearing Resistance

The bearing resistance of shallow foundations for concrete retaining walls was analyzed using procedures similar to those discussed in **Section 6.3.4** for the bridge abutment shallow foundation option. The factored bearing resistance and estimated total settlements from foundation loading alone (excluding stress influence from surrounding abutment embankments) are summarized in **Table 16** and **Table 17** considering cases both with and without stone columns ground improvement.

The factored bearing resistances for Strength and Extreme Limit States, which are based on the general shear failure criterion, are in excess of the maximum factored loads and appear to meet design criteria. However, estimated settlements at the Service Limit State bearing pressure exceed the permissible maximum settlement of 1 inch both with and without ground improvement, and therefore the Service Limit State is the critical limit state. Accordingly, **Table 16** and **Table 17** also present settlement-limited bearing resistances for the Service Limit State corresponding to 1 inch of settlement. Based on the relatively high levels of estimated settlement and the anticipated complexity of the construction sequence for this project, a more sophisticated staged settlement analysis was also performed (see **Section 6.3.5**). The foundation designer should compare the ratio of recommended nominal bearing resistance to applied factored loads (i.e., Capacity to Demand Ratio, CDR) to specify the governing design case against the proposed shallow foundation specification. A CDR less than 1.0 indicates the failure of the design case against applied loads.

Table 16. Retaining Wall Shallow Foundations Recommendations – West Approach

Support Location	Design Wall Height(ft)	BOF Elev. (ft NAVD88) [1]	Min. Footing Embed. (ft bgs)	Footing Size (ft)		Ground Improve	Limiting Failure Mode for Bearing Resistance	Est. Max. Total Settle-ment (inches) [2]	Factored Bearing Resistance (ksf) Per Each Limit state [1]		
				B	L				Service [3,4]	Strength [3,4]	Extreme [3,4]
West Approach	25	+28.0	8	16	60	None	Shear	2.0	3.2	22.0	48.8
							Settlement	**1.0**	**1.6**	**22.0**	**48.8**
						Stone Cols.	Shear	1.0	3.2	21.7	48.2
							Settlement	---[5]	---[5]	---[5]	---[5]
	20A	+36.0	6	16	30	None	Shear	2.3	3.5	13.3	29.7
							Settlement	**1.0**	**1.6**	**13.3**	**29.7**
						Stone Cols.	**Shear**	**1.0**	**3.5**	**12.5**	**25.1**
							Settlement	---[5]	---[5]	---[5]	---[5]
	15	+39.0	3.75	10.5	60	None	Shear	1.7	2.3	9.7	21.5
							Settlement	**1.0**	**1.4**	**9.7**	**21.5**
						Stone Cols.	**Shear**	**0.5**	**2.3**	**8.1**	**17.9**
							Settlement	---[5]	---[5]	---[5]	---[5]

Notes:

1. Recommendations are based on the foundation geometry and the loads provided by Structure Design in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.
2. Considers settlement of the footing alone; does not account for load influence of adjacent embankments and associated settlements. Shear-limited bearing pressures is based on bearing capacity theory for general shear failure, whereas settlement-limited bearing resistance is based on the bearing pressure which produces the maximum permissible settlement of 1.0 inches.
3. Resistance factors (ϕ_b) of 1.0, 0.45, and 1.0 were applied to the estimated nominal bearing resistance for service, strength, and extreme limit states, respectively.
4. Factored bearing resistance assumes wall footings bear in Stratum 2 for West Approach wall height of 25 feet and East Approach wall heights of 20 and 25 feet, and wall footings bear in Stratum 1 for West approach wall heights of 15 and 20 feet and East Approach wall height of 15 feet.
5. Shear-controlled bearing resistance produces settlement equal to or less than the maximum permissible settlement of 1.0 inch.

Table 17. Retaining Wall Shallow Foundation Recommendations – East Approach

Support Location	Design Wall Height(ft)	BOF Elev. (ft NAVD88) [1]	Min. Footing Embed. (ft bgs)	Footing Size (ft)		Ground Improve	Limiting Failure Mode for Bearing Resistance	Est. Max. Total Settle-ment (inches) [2]	Factored Bearing Resistance (ksf) Per Each Limit state [1]		
				B	L				Service [3,4]	Strength [3,4]	Extreme [3,4]
East Approach	25	+28.0	5	16	60	None	Shear	2.0	3.2	22.0	48.8
							Settlement	**1.0**	**1.6**	**22.0**	**48.8**
						Stone Cols.	Shear	1.0	3.2	21.7	48.2
							Settlement	---[5]	---[5]	---[5]	---[5]
	20B	+30.8	5	12.5	60	None	Shear	1.9	2.7	20.8	46.3
							Settlement	1.0	2.0	20.8	46.3
						Stone Cols.	Shear	**1.0**	**2.7**	**21.7**	**48.2**
							Settlement	---[5]	---[5]	---[5]	---[5]
	15	+39.0	3.75	10.5	60	None	Shear	1.7	2.3	9.7	21.5
							Settlement	**1.0**	**1.4**	**9.7**	**21.5**
						Stone Cols.	Shear	**0.5**	**2.3**	**8.1**	**17.9**
							Settlement	---[5]	---[5]	---[5]	---[5]

Notes:

1. Recommendations are based on the foundation geometry and the loads provided by Structure Design in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.
2. Considers settlement of the footing alone; does not account for load influence of adjacent embankments and associated settlements. Shear-limited bearing pressures is based on bearing capacity theory for general shear failure, whereas settlement-limited bearing resistance is based on the bearing pressure which produces the maximum permissible settlement of 1.0 inches.
3. Resistance factors (ϕ_b) of 1.0, 0.45, and 1.0 were applied to the estimated nominal bearing resistance for service, strength, and extreme limit states, respectively.
4. Factored bearing resistance assumes wall footings bear in Stratum 2 for West Approach wall height of 25 feet and East Approach wall heights of 20 and 25 feet, and wall footings bear in Stratum 1 for West approach wall heights of 15 and 20 feet and East Approach wall height of 15 feet.
5. Shear-controlled bearing resistance produces settlement equal to or less than the maximum permissible settlement of 1.0 inch

Foundation Settlement

Settlement evaluation of shallow foundations for concrete retaining walls – considering added loading from adjacent embankment fills – was analyzed using similar procedures discussed in **Section 6.3.5** for the bridge abutment shallow foundation option.

Analysis was conducted for the east approach only, which has less favorable soil conditions than the west approach. The results of the staged analysis are presented in **Table 18**.

The results indicate cumulative foundation settlements from the staged analysis are expected to range from about 1.8 to 2.0 inches for the east approach concrete walls when stone columns ground improvement is incorporated below the footings. Without ground improvement, the estimated cumulative settlements range from 2.1 to 2.3 inches. Settlement for the west approach concrete walls would be slightly less. As expected, the settlement of concrete retaining wall footings is slightly higher when accounting for imposed subgrade stresses from the adjacent raise of the approach embankments, as compared to settlement from to the applied footing loads alone.

Based on the magnitude of estimated settlement, special slip-jointing may be required at the interface with wingwalls to accommodate differential settlement. Further, final concrete facings for the soldier pile wingwalls may be installed after approach embankments have been built to final grade elevation to allow settlement to occur and reduce potential for concrete cracking.

Table 18. Summary of Staged Settlement Analysis for Concrete Retaining Walls

Stage No.	Stage Name	Stage Description	Model Cumulative Construction Settlement at Center of Bridge Footing at BOF Level (inch) [1]					
			25-ft-tall wall		20-ft-tall wall		15-ft-tall wall	
			No Ground Improve	Stone Columns	No Ground Improve	Stone Columns	No Ground Improve	Stone Columns
1	Existing conditions	"Back-analysis" of existing embankment (ignores existing footings). This settlement has already occurred in the field.	0.7	0.7	0.8	0.8	0.8	0.8
2	Excavate for retaining wall footing	Removes the existing embankment to El. +32 (east) or +31 (west), and creates a square excavation to BOF under retaining wall footing.	-0.1 (heave)	-0.1 (heave)	-0.1 (heave)	-0.1 (heave)	-0.1 (heave)	-0.1 (heave)
3	Install stone columns	Installs a 5'x5' square grid of 3' diam. stone columns, with top elev. at BOF and bottom elev. embedded 3' into Stratum 3.	n/a	-0.1 (heave)	n/a	-0.1 (heave)	n/a	-0.1 (heave)
4	Construct retaining wall footing and initial backfill	Installs concrete wall footing at BOF elev. Applied load is 65% of the service load, simulating partial backfilling beyond / over wall footing.	0.8	0.7	0.8	0.7	0.6	0.6
5	Interim embankment	Installs the interim grade approach embankment to max El. +48 adjacent to the footing.	1.4	1.2	1.4	1.2	1.2	1.1
Concrete wall settlement prior to bridge install (Stages 2 to 5)			**1.4**	**1.2**	**1.4**	**1.2**	**1.2**	**1.1**
6	Final wall backfill	Increases the applied load on concrete wall footing from 65% to 100% of the service load, simulating final	1.9	1.6	1.9	1.6	1.6	1.4

		backfilling beyond / over wall footing.						
7	Final embankment	Installs the adjacent final grade embankment and retaining walls with 3H:1V backslopes to max El. +58.5.	2.3	2.0	2.3	2.0	2.1	1.8
Concrete wall settlement during/after bridge superstructure install (Stages 5 to 7)			**0.9**	**0.8**	**0.9**	**0.8**	**0.9**	**0.7**

Note
1. Analysis conducted considering Service Limit State bearing pressure for concrete walls corresponding to wall height (see Table 7)

6.4.2 Mechanically Stabilized Earth Walls

Mechanically stabilized earth (MSE) walls are fill-type walls which use steel or geosynthetic reinforcements between lifts of compacted earthfill to provide stable wall structure, and a variety of facing types are available.

Design layouts for the permanent walls shown in the 95 percent drawings indicate maximum total heights (i.e., exposed wall height plus embedded depth below finish grade) for MSE walls ranging from about 17 to 24 feet for the NW, SW, SE, and NE walls; refer to **Table 19**.

Based on the settlement analysis conducted for the concrete walls and approach embankments presented in **Table 18**, maximum settlement for MSE walls without ground improvement is expected to be on the order of about 2 to 2.5 inches. The WSDOT BDM Table 15-4 indicates that MSE walls with two-stage construction (i.e., walls constructed with an initial flexible geosynthetic facing followed by installation of the structural facing as a second stage after wall settlement is complete) can accommodate total settlements of up to 4 inches and differential settlements over a 50-foot distance of up to 3 inches with no special design measures. Accordingly, ground improvement is not required to meet settlement criteria for the permanent walls if designed and constructed as two-stage MSE walls.

Geotechnical analysis of MSE walls was performed based on guidance in the WSDOT GDM, AASHTO BDS, and FHWA design guidance for MSE walls (Berg et al. 2009). A summary of the analyzed geometry for permanent walls and relevant soils information is presented in **Table 19**.

Table 19. Summary of Analyzed Permanent MSE Wall Geometry

Wall ID	STA.	Max. Final Exposed Height (feet)	Max. Interim Retained Height (feet) [1]	Backslope [2]		Foreslope [2]	
				Angle	Height (feet)	Angle	Height (feet)
NW	1+54	15.6	17.8	3H:1V	2.1	Horiz.	~0
SW	1+28	15.1	17.1	3H:1V	7.6	Horiz.	~0
SE	1+20	19.5	23.9	3H:1V	2.5	Horiz.	~0
	1+68	8.7	10.7	3H:1V	11.9	Horiz.	~0
NE	1+31	15.2	17.2	3H:1V	8.5	Horiz.	~0

Notes:
1. Height to bottom of embedded facing. Facing embedment ranges from about 2 to 3 feet typically. Adjacent utility trench excavations are not considered in this value.
2. Positive slope indicates slope upward away from wall, negative slope indicates slope downward away from wall.

Geotechnical Limit States

The analyzed geotechnical limit states and corresponding load and resistance factors used in the analysis were based on AASHTO BDS and WSDOT GDM and are summarized in **Table 20**.

Table 20. Summary of Applicable Load and Resistance Factors for MSE Wall Analysis

Parameter	Value for Geotechnical Limit State		
	Service Limit	Strength Limit	Extreme Limit
Load Factor			
Vertical Earth Pressure, EV	1.0	1.0	1.0
Active Lateral Earth Pressure, EH	1.0	1.5	1.0
Earthquake Load, EQ	n/a	n/a	1.0
Live load surcharge, LS	1.0	1.75	0.5
Dead load surcharge, ES	1.0	1.5	1.0
Resistance Factor			
Bearing Resistance	1.0	0.65	0.90
Sliding	1.0	1.00	1.00
Tensile resistance (geosynthetic, geogrid)	1.0	0.80	1.00
Pullout resistance (geosynthetic, geogrid)	1.0	0.70	1.00

Lateral Earth Pressures

The lateral earth pressure recommendations provided in **Section 6.3.15** are applicable for MSE walls. Assumptions regarding wall geometry, groundwater conditions, foundation soil, retained soil, and reinforced soil are stated in **Table 21** and its footnotes. Groundwater was conservatively assumed at the base of each analyzed wall section due to limitations of the analysis software, which can only consider cases of no groundwater or groundwater at/above the wall base.

For external stability, the analysis considers the earth pressure coefficients corresponding to the retained soil (see **Table 21**). For internal stability, the earth pressure coefficients for the reinforced zone backfill are used which are calculated internally by the analysis software used (see discussion in *External Stability* section below).

Lateral Load Resistance

The lateral load resistance for MSE walls is derived from frictional sliding resistance along the base of the reinforced soil mass. The analysis for sliding stability must consider the potential for 1) sliding along the interface of the reinforced soil and the foundation soil (external failure); and 2) sliding along the interface of the geosynthetic reinforcements and surrounding soil (internal failure). External sliding resistance is based on the lower friction angle of the foundation soil or the reinforced zone soil; for this project the foundation soil is expected to be weaker. The internal sliding resistance is a function of the geosynthetic properties and the surrounding soil and is generally less than the friction angle of the surrounding soil. For this reason, AECOM recommends the lower level of geosynthetic reinforcement be placed upon at least 6 to 12 inches of reinforced zone soil to prevent a sliding failure along the interface of the foundation soil and the geosynthetic reinforcement.

External Stability

The software program MSEW+ by ADAMA Engineering, Inc. (Update #2025.01) was used to establish minimum reinforcement lengths to satisfy external stability requirements (bearing, sliding, overturning, eccentricity). The MSEW+ software was executed using the built-in AASHTO 2017-2020 Simplified AASHTO Method for geosynthetics reinforcement. The software was executed in “Design Mode” to automatically calculate the minimum reinforcement length to satisfy stability for the controlling failure mode, and then was converted to “Analysis Mode” to allow printing of the output.

The MSEW+ software was initially executed with the user-input values of K_a and K_{ae} as presented in **Table 21**, where K_{ae} for seismic loading was estimated using either the M-O or GLE methods as stated previously. However, the initial results for cases with a sloping backfill indicated excessive reinforcement lengths would be required to meet stability criteria for seismic loading. Those results are based on seismic earth pressure methods that assume an infinitely-long backslope, which is not representative of the actual geometry of the analyzed wall section. Accordingly, the MSEW+ software was re-run using the built-in Coulomb's Constrained Wedge Method, which considers a broken-back equivalent slope in the retained zone, and produced more reasonable results. Where the Coulomb's Constrained Wedge Method was used, the input seismic acceleration in the MSEW+ software was adjusted to 50 percent of the site-adjusted ground acceleration ($kh = 0.5 \times A_s$) to account for the expectation that the MSE walls can permanently deform 1 to 2 inches during seismic loading, which is consistent with the formulation of the K_{ae} values for "flexible walls" developed using M-O or GLE methods.

Results using the Coulomb's Constrained Wedge Method are summarized in **Table 21** and demonstrate the minimum Capacity to Demand Ratios (CDR) are satisfied for each external failure mode considered. As can be seen, longer reinforcements are required for walls founded on Stratum 1 as compared to Stratum 2. Based on these results, we recommend a design minimum reinforcement length to total wall height ratio (L/H) of 1.0 for permanent MSE walls.

Table 21. Permanent MSE Walls – Summary of MSEW+ Inputs and External Stability Results

Parameter	Results by Analysis Section				
	NW Wall	SW Wall	SE Wall		NE Wall
Geometry and Soils Information					
Analysis Station	1+54	1+28	1+20	1+68	1+31
Exposed Height, H' (ft)	15.6	15.1	19.5	8.7	15.2
Embedment (ft)	2.2	2.0	4.4	2.0	2.0
Total Wall Height, H (ft)	17.8	17.1	23.9	10.7	17.2
Bottom of Facing Elev. (ft NAVD88)	El. +38.1	El. +33.2	El. +31.2	El. +36.0	El. +33.0
Unfactored Live Load Surcharge (psf)	250	250	250	250	250
Foundation Soil Unit	Stratum 1	Stratum 2	Stratum 2	Stratum 1	Stratum 2
Reinforced Zone Soil[1]	GB	GB	GB	GB	GB
Retained Soil [1]	CB	CB	CB	CB	CB
Retained Soil Ka / Kae [2]	0.36 / 0.81	0.36 / 0.81	0.36 / 0.81	0.36 / 0.81	0.36 / 0.81
Groundwater	Wall base	Wall base	Wall base	Wall base	Wall base
External Stability Results [3,4]					
Factored Max. Bearing Stress (psf) – Strength	4,186	4,062	5,766	2,637	4,431
Factored Max. Bearing Stress (psf) – Extreme	4,868	5,547	7,700	3,185	5,568
Factored Bearing Resistance (psf) – Strength	5,790	10,245	14,063	4,234	11,790
Factored Bearing Resistance (psf) – Extreme	4,952	7,502	10,495	3,698	9,381
Bearing CDR – Strength (1.0 min.)	1.38	2.52	2.44	1.61	2.66
Bearing CDR – Extreme (1.0 min.)	1.02	1.35	1.36	1.16	1.68
Sliding Soil/Soil CDR – Strength (1.0 min.)	2.15	2.57	2.41	1.72	2.02
Sliding Soi/Soil CDR – Extreme (1.0 min.)	1.43	1.50	1.50	1.05	1.36
Sliding Soi/Reinf. CDR – Strength (1.0 min.)	2.01	2.00	1.85	1.64	1.56
Sliding Soil/Reinf. CDR – Extreme (1.0 min.)	1.35	1.17	1.16	1.00	1.06
Eccentricity – Strength (0.25 max)	0.07	0.08	0.09	0.09	0.10
Eccentricity – Extreme (0.40 max)	0.22	0.29	0.29	0.24	0.28
Controlling Failure Mode	Sliding	Sliding	Sliding	Sliding	Sliding
Design Recommendations					
Minimum Reinforcement Length, L (ft)	15	12	17	11	14
Minimum L/H Ratio	0.84	0.70	0.71	1.03	0.81

Notes:

1. GB – Gravel Borrow for Retaining Walls, CB – Common Borrow, SB – Select Borrow
2. Obtained from Tables 14 and 15. Kae assumes "flexible" retaining wall which may deflect at least 1 to 2 inches laterally during earthquake loading.
3. Reported results are based on earth pressures calculated internally in the MSEW+ software using the Coulomb's Constrained Wedge Method. The input kh value for seismic analysis was taken as 0.5 x As ("flexible wall") for consistency with the M-O approach used to calculate Kae.
4. Internal stability results were calculated but not reported; internal stability is the responsibility of the Contractor's wall vendor.

Internal Stability

Internal and compound stability design and accompanying calculations are the responsibility of the Contractor's wall vendor/Engineer per WSDOT standard practice. The contract documents should require the Contractor to submit for review shop drawings and internal/compound stability calculations signed and sealed by a Washington-licensed Professional Engineer.

The MSEW+ software requires input parameters for geosynthetic reinforcements to perform the analysis. Accordingly, for preliminary analysis purposes, geosynthetic reinforcements were modeled in both MSEW+ to provide a preliminary indication of internal stability. A maximum vertical spacing of 2 feet was used in the analysis to accommodate anticipated overlap of reinforcements from adjacent wall sections. The geosynthetic was modelled as a high-strength geogrid (Tensar UX1800HS) to prevent internal stability from controlling the minimum length of the reinforcements in the MSEW+ analysis; input parameters include an ultimate tensile strength (T_{ult}) of 14,390 lbs/ft, durability reduction factor of 1.0, installation damage reduction factor of 1.05, and creep reduction factor of 2.6, producing a long-term allowable tensile strength (T_{al}) of 5,271 lbs/ft. Default values were used for the geogrid-soil interface friction angle (ρ), interaction coefficient (Ci) for the pullout resistance factor, and scale effect correction factor (α). Internal stability results are included in the MSEW+ software output.

Global Stability

Global slope stability for MSE walls was checked using software program GeoStudio Slope/W by Seequent to confirm minimum reinforcements lengths from MSEW+ were satisfactory to meet minimum factors of safety. Spencer's method of slices was used for the analysis. Similar to the MSEW+ analysis, geosynthetic reinforcements were modeled in Slope/W. Results are presented later in Section 6.6.

Settlement

Settlement of MSE walls for this project is largely due to the imposed stresses associated with the increased height of the proposed bridge approach embankment. Accordingly, the settlement of the MSE walls are expected to be similar to the results presented previously for the concrete permanent walls without ground improvement (**Table 18**), and will likely be on the order of 2 to 2.5 inches. Settlement will be primarily elastic (immediate) settlement due to the predominantly granular foundation soils, and is expected to occur during construction as the fill is placed. Long-term post-construction settlement is expected to be minor to negligible.

The permanent CIP facing for MSE walls should be installed only after the walls and approach embankment have been constructed to full height (finish grade) over the full approach length, and survey monitoring has confirmed settlement is complete.

Internal Drainage

Internal drainage for MSE walls should consist of free-draining aggregates (less than 5 percent passing the No. 200 sieve by weight) used in the wall reinforced soil zone, and weepholes to drain collected water. Weepholes should be installed through the permanent wall facing at a height of 1 foot above finished grade at the toe of the wall according to WSDOT standard drawings for permanent geosynthetic walls.

If free-draining aggregates cannot be provided for the reinforced zone, a wall underdrain should be included at the back of the reinforced zone at the bottom of the wall according to WSDOT standard drawings for “structural earth walls” to provide internal drainage. The undrain should consist of a slotted drain pipe embedded in free-draining aggregates, and the outside of the aggregates should be wrapped with geotextile separator fabric. Solid drain pipe laterals from the undrain should be provided to drain collected water by gravity to an approved discharge location, such as daylighting into an open channel or discharging into an adjacent storm sewer.

6.5 Bridge Wingwalls

Wingwalls are planned at the new bridge abutments and they provide transition from the abutment walls/foundations to the permanent retaining walls. The maximum final exposed height of the wingwalls is approximately 27 feet, but interim excavations near the toe of the wingwalls for utility replacement will produce maximum design exposed wall heights of about 34 feet.

A wingwall is proposed at each corner of the bridge abutments, as summarized in **Table** 22 below.

Design criteria provided by the Structures team were to 1) limit static lateral wall deflections to 1.0 inch or less, and 3) not introduce lightweight backfill materials in the passive wedge directly behind the bridge abutment wall/foundation due to reliance on passive earth pressures to resist lateral bridge loads.

Table 22. Summary of Proposed Wingwalls

Wall ID	STA.	Max. Final Exposed Height (feet)	Max. Interim Retained Height (feet) [1]	Backslope [2]		Foreslope [2]	
				Angle	Height (feet)	Angle	Height (feet)
NW	1+00 (East)	26.0	28.2	~Horiz.	~0	~Horiz.	~0
	1+20 (North)	22.0	24.2	3H:1V	4.2	-6H:1V	2
	1+54 (West)	18.8	21.0	~Horiz.	~0	~Horiz.	~0
SW	1+00	26.5	29.5	~Horiz.	~0	~Horiz.	~0
SE	1+00	24.1	27.7	~Horiz.	~0	~Horiz.	~0
NE	1+00	26.4	29.5	~Horiz.	~0	~Horiz.	~0

Notes:
1. Height to bottom of embedded facing. Adjacent utility trench excavations are not considered in this value.
2. Positive slope indicates slope upward away from wall, negative slope indicates slope downward away from wall.

6.5.1 Wall Type Evaluation

Several wall types were considered for each of the four wingwalls based on site and design constraints. Because the existing embankment will mostly be excavated down to BOF level for the bridge abutment in the vicinity of the wingwalls, the wingwalls will be primarily fill-type walls. A primary design criterion is to physically isolate the wingwall structures from the bridge abutment structures. Additionally, there are specific architectural / aesthetic requirements for the wingwalls and concrete retaining walls requiring cast-in-place concrete final facings with only vertical joint lines. An additional constraint is the unusual geometry of the northwest (NW) wingwall, which is “U-shaped” and limits many conventional wall types.

Several wall types were initially considered, and are discussed below:

1. Concrete semi-gravity spread footing walls (like those planned for the embankment retaining walls) are not practical because the spread footings conflict with the adjacent footings of the

bridge abutment. This is due to the acute angles in the wall alignments where the wingwalls connect to both the bridge abutment walls and the concrete retaining walls for the approach embankment. Similar issues would be encountered with pile-supported concrete "T-walls."

2. MSE walls in general are well-suited to the engineering performance criteria and constructability requirements for this project due to their inherent flexibility which can accommodate larger total and differential movements and seismic loading than many other wall types. However, MSE walls which use precast segmental wall panels ("Structure Earth Walls" per WSDOT standards) do not meet the aesthetic requirements. MSE walls which are constructed with full-height facing consisting of CIP concrete or tilt up precast concrete wall panels may meet aesthetic criteria, but would not be able to accommodate the heights required for the wingwalls on this project and associated settlement if constructed in a single stage. Accordingly, only two-stage MSE walls, constructed to full height using flexible geosynthetic-wrap or gabion basket temporary facing followed by installation of a permanent CIP concrete facing after settlement is complete ("Geosynthetic Walls" per WSDOT standards), were deemed feasible for wingwalls on this project.
3. Non-gravity cantilever walls consisting of soldier piles or closely-spaced drilled shafts were considered for the wingwalls. However, typical cantilevered walls are generally limited to heights of about 15 feet with standard wall backfill, and preliminary analysis found that cantilever soldier pile deflections at the design wall heights were excessive even with large steel sections (e.g., W44x335). Preliminary analysis found that large-diameter drilled shafts (60 inch) with minimal clear spacing and lengths on the order of 75+ feet would be needed to limit deflections to tolerable levels, which was deemed cost prohibitive for this project. Alternatively, the use of lightweight backfill behind cantilever walls may be able to limit earth pressures to levels which can be accommodated by routine-size soldier piles to maintain deflections to tolerable levels. Additionally, soldier piles with prestressed ground anchors are routinely used where the wall height exceeds the limits of cantilevered walls.
4. Deadman anchored walls were initially evaluated for the wingwalls. Deadman anchored walls are primarily fill-type walls consisting of cantilever sheetpile or soldier pile wall elements with one of more levels of "deadman" anchors (e.g., near-horizontal tensioned tie-rods connected to concrete blocks or a second row of piles) which serve to limit wall deflections and provide additional resistance to lateral loads. The deadman anchors rely on passive soil resistance and must be set a sufficient distance behind the wall elements so that the theoretical passive wedge of soil in front of the deadman does not overlap with the unstable active wedge of soil behind the wall elements. Soldier piles were considered in lieu of sheetpiles due to questionable drivability of sheetpiles into the dense Stratum 3 soils. Preliminary analysis found the proposed abutment geometry provided insufficient clear distance between the theoretical passive wedge of the deadman anchor and the active wedge of the wall, resulting in overlapping of the stress zones and inefficient design. The acute angle of the wingwalls and relatively large deadman blocks required for lateral load resistance also produced physical conflicts with deadman anchors overlapping proposed grade lines, adjacent concrete wall and bridge abutments footings, and utilities. The deadman anchors would significantly complicate staged construction of the bridge and embankments, and provide potential issues with future excavations for utilities/etc.
5. Soil nail walls were deemed not applicable since these are a cut-type wall, and adapting to a fill-type wall would be difficult and inefficient.

Recommended wall types considering the constraints listed above are provided in the next section.

6.5.2 Recommended Wall Types

Based on project constraints, preliminary evaluations, and discussion with the design team, the following wall types are recommended for design:

- NW Wingwall: Cantilevered soldier pile wall with ultra-lightweight backfill
- SW, SE, and NE Wingwalls: MSE walls with two-stage facing installation similar to WSDOT standard for “Permanent Geosynthetic Wall with CIP Permanent Fascia”

Note that a previous version of this report recommended the following (recommendations for this wall type have been maintained in this report as a potential design alternative to the MSE wall recommendation):

- SW, SE, and NE Wingwalls Alternative Design: Soldier pile wall with two rows of prestressed ground anchors

Details and recommendations are provided in the following sections.

6.5.3 NW Wingwall Recommendations

The “U-shape” alignment of the NW Wingwall limits potential options for anchored wall types due to geometric constraints. Accordingly, we recommend the NW Wingwall consist of a cantilevered soldier pile wall with ultra-lightweight backfill to reduce wall earth pressures, bending moment, and deflections. Foamed Glass Aggregate (FGA) is recommended for the ultra-lightweight (UL) backfill due to its low unit weight (25 to 30 pcf), high friction angle (41-50 degrees), and free-draining gradation. We recommend timber lagging be placed between adjacent soldier piles to provide temporary facing for retaining excavated soils and backfill. Permanent facing consisting of CIP reinforced concrete anchored to the soldier piles via shear studs is recommended to resist permanent loads and satisfy wall aesthetic requirements.

Lateral Earth Pressures

The lateral earth pressure recommendations provided in **Section 6.3.15** are applicable for the NW Wingwall design.

Lateral Load Resistance

Lateral loads acting on the soldier pile can be resisted by the passive earth pressure of the soil on the embedded portion of the soldier pile. The passive resistance and loads on the embedded portion of the soldier pile can be evaluated according to the simplified earth pressure distribution in AASHTO BDS Figure 3.11.5.6-1, and the earth pressure recommendations provided in **Section 6.3.15**.

Alternatively, the lateral load resistance and required pile embedment can be evaluated by the P-y method using computer software such as LPILE or PYWALL by Ensoft. Recommended P-y method analysis parameters are provided in **Table 1**. P-multipliers should be employed as reductions to the available lateral load resistance when the center-to-center soldier pile spacing is less than 5D according to recommendations in **Table 12**.

Wall Internal Drainage

Internal drainage should consist of 4-foot-wide prefabricated geocomposite strip drains placed between each set of adjacent soldier piles. The strip drains should consist of a polymer internal drainage net or core with external non-woven filter fabric on both sides. The strip drains should be attached to the front face of the timber lagging and behind the permanent concrete wall facing.

Refer to WSDOT Standard Plans for Soldier Pile/Tieback Walls, and WSDOT Standard Specifications Item 6-16 for additional details and requirements.

Preliminary Wall Design Layout

Preliminary design evaluations were conducted for both the north-facing and east-facing wall sections using PYWALL software by Ensoft. Active earth pressures (**Table 14**) and P-y method parameters (**Tables 1 and 2**) were used in the analysis. A uniform traffic surcharge pressure of 250 psf was considered. The retained soil was modeled as UL-FGA backfill with a 1-foot-thick cap of standard-weight backfill soil.

Preliminary analysis results indicate that the east-facing wall section is the critical section, and soldier piles with the following criteria can resist anticipated static lateral earth loads and limiting lateral deflection to about 1.3 inch or less for static loading:

- Soldier piles consisting of W18x211 steel sections placed into 30-inch-diameter (D) drill holes.
- Soldier pile drill holes backfilled with structural concrete (4,000 psi) below the excavation line in front of the wall. Low-strength lean concrete should be used to backfill drill holes above the excavation line, and this material was ignored in the analysis.
- Soldier pile spacing of 5 feet on-center (2D), with appropriate P-multipliers for closely-spaced piles applied internally in the software.
- Soldier pile embedment of at least 21 feet below final ground surface at toe of wall.

Deflections less than 1.0 inch for static loading can be achieved by either decreasing the soldier pile spacing to 4 feet on-center, or by increasing the size of the soldier pile steel to W24x176 which requires a larger drill hole of 36-inch-diameter.

For seismic loading, we expect that this wall type will be capable of deflecting at least 1 to 2 inches under design seismic loads. Thus, the seismic earth pressures should be based on k_h = 0.5 x A_s for final design analyses to verify soldier pile sizing. Seismic deflections of up to 1 percent of the wall height are likely to be tolerable, which is a common criterion set for static deflections by other state DOT's around the country.

Lightweight Backfill Recommendations

The limits of the UL-FGA should extend a sufficient distance beyond the back of the wall to ensure the active failure wedge is confined to the UL-FGA backfill zone. Additionally, to prevent the standard-weight soil backfill from transferring earth pressures through the UL-FGA backfill onto the wall, the interface of standard-weight soil backfill and UL-FGA backfill must be inclined at a slope at which the standard-weight backfill is inherently stable. Accordingly, AECOM recommends the following for the UL-FGA backfill zone:

1. Set the base of the UL-FGA backfill at or below the lowest excavation elevation at the toe of the wall.
2. Extend the base of the UL-FGA backfill at least 5 feet horizontally beyond the back of the soldier piles.
3. The interface of the UL-FGA and the standard-weight backfill behind the UL-FGA backfill should slope upward away from the excavation base at 1.5H:1V or flatter (2H:1V if possible based on-site constraints).
4. A 12- to 18-inch-thick cap of topsoil or soil backfill should be placed above the top of the UL-FGA to provide erosion protection per manufacturer recommendations.
5. Placement and compaction of the UL-FGA should be in accordance with manufacturer recommendations to achieve maximum density while preventing grain crushing.
6. Provide nonwoven geotextile as a separator between the subgrade and the initial lift of UL-FGA as well as above the final lift and on side slopes as a separator between the UL-

FGA and capping layer. A geotextile meeting WSDOT Standard Specification Section 9-33.2(1) Table 3 requirements for nonwoven separation geotextile is recommended.

6.5.4 SW, SE, and NE Wingwalls Recommendations: Two-Stage MSE Wall

The analysis procedures and design recommendations discussed in **Section 6.4.2** for permanent MSE walls are applicable to the wingwalls if designed as MSE walls. Only two-stage MSE walls should be considered for wingwalls based on the estimated 3 to 4 inches of settlement predicted near the bridge abutments (see **Table 9**). Ground improvement is not necessary for the wingwalls if two-stage MSE walls are implemented and provided that the permanent CIP concrete facing is installed only after the walls and embankment are constructed to full height (finish grade) for the full approach and monitoring indicates settlement is complete.

Specific to the wingwalls, the following exceptions to the conclusions and recommendations of **Section 6.4.2** apply:

1. Estimated total settlement of the wingwalls is about 3 to 4 inches due to the taller wall heights as compared to the permanent walls. According to Table 15-4 of the WSDOT GDM, two-stage MSE walls should be able to tolerate settlement of this magnitude without special design measures.

2. The adjacent bridge abutment walls are expected to be supported on drilled shaft foundations and will experience significantly less settlement than the MSE wingwalls (about 1 inch maximum). Since the permanent CIP concrete facing for two-stage MSE wingwalls will not be installed until after settlement is complete, there is limited concern for differential settlement between the bridge abutment walls and MSE wingwalls.

3. The MSEW+ software analysis for MSE wingwalls incorporated the user-defined K_a and K_{ae} parameters and produced reasonable results, precluding the need to perform the Coulomb Constrained Wedge Method for seismic analysis. These results are primarily due to the horizontal backslope present for each of the analyzed wingwall sections, which avoids issues of sloping backfill inherent to the M-O method used to estimate K_{ae}.

Results of the MSEW+ analysis for the MSE wingwalls are summarized in Table 23, and demonstrate the minimum CDR are satisfied for each external failure mode considered. Based on these results, we recommend a minimum reinforcement length to total wall height ratio (L/H) of 1.1 for permanent MSE walls at stations which are not explicitly defined in the table below.

Table 23. MSE Wingwalls – Summary of MSEW+ Inputs and External Stability Results

Parameter	Results by Analysis Section		
	SW Wingwall	SE Wingwall	NE Wingwall
Geometry and Soils Information			
Analysis Station	1+00	1+00	1+00
Exposed Height, H' (ft)	26.5	24.1	26.4
Embedment (ft)	3.0	3.6	3.1
Total Wall Height, H (ft)	29.5	27.8	29.5
Bottom of Facing Elev. (ft NAVD88)	El. +28.6	El. +31.2	El. +29.0
Unfactored Live Load Surcharge (psf)	250	250	250
Subgrade Soil Unit	Stratum 2	Stratum 2	Stratum 2
Reinforced Zone Soil	Gravel Borrow	Gravel Borrow	Gravel Borrow
Retained Soil	Common Borrow	Common Borrow	Common Borrow
Retained Soil Ka / Kae [1]	0.22 / 0.42	0.22 / 0.42	0.22 / 0.42
External Stability Results [2,3]			
Factored Max. Bearing Stress (psf) – Strength	5,961	5,661	5,966
Factored Max. Bearing Stress (psf) – Extreme	7,319	6,731	7,354

Parameter	Results by Analysis Section		
	SW Wingwall	SE Wingwall	NE Wingwall
Factored Bearing Resistance (psf) – Strength	28,730	27,590	28,474
Factored Bearing Resistance (psf) – Extreme	23,398	23,206	23,100
Bearing CDR – Strength (1.0 min.)	3.20	4.87	4.77
Bearing CDR – Extreme (1.0 min.)	4.82	3.45	3.14
Sliding Soil/Soil CDR – Strength (1.0 min.)	3.42	3.07	3.42
Sliding Soi/Soil CDR – Extreme (1.0 min.)	1.30	1.30	1.30
Sliding Soi/Reinf. CDR – Strength (1.0 min.)	2.64	2.36	2.64
Sliding Soil/Reinf. CDR – Extreme (1.0 min.)	1.00	1.00	1.00
Eccentricity – Strength (0.25 max)	0.06	0.07	0.06
Eccentricity – Extreme (0.40 max)	0.19	0.18	0.19
Controlling Failure Mode	Sliding (extreme)	Sliding (extreme)	Sliding (extreme)
Design Recommendations			
Minimum Reinforcement Length, L (ft)	32	31	32
Minimum L/H Ratio	1.08	1.11	1.08

Notes:
1. Obtained from Tables 14 and 15. Kae assumes “flexible” retaining wall which may deflect at least 1 to 2 inches laterally during earthquake loading.
2. Internal stability results were calculated but not reported; internal stability is the responsibility of the Contractor’s wall vendor.

6.5.5 SW, SE, and NE Wingwalls Alternative: Anchored Soldier Pile Wall

If the recommended two-stage MSE walls are not preferred, we recommend an alternative design for the SW, SE, and NE Wingwalls consisting of anchored soldier pile walls. Anchored soldier pile walls are needed here due to the relatively taller heights as compared to the NW Wingwall, and inability to limit wall deflections to less than the 1-inch maximum permissible value with cantilevered soldier piles even when UL-FGA backfill is used. We recommend soldier piles with two levels of pressure-grouted, prestressed ground anchors. Similar to the NW Wingwall, temporary facing consisting of timber lagging and permanent facing consisting of CIP concrete anchored to soldier piles via shear studs are recommended.

Prestressed Ground Anchor Recommendations

We anticipate that the bond zones for ground anchors will be located primarily within proposed backfill materials behind the walls, with portions of the bond zones extending into Stratum 1 and/or Stratum 2 depending on location and anchor orientation. Recommended design values of bond stress for both pressure-grouted and gravity-grouted anchors are presented in **Table 24**. For this project, we recommend pressure-grouted ground anchors achieving a minimum 50 psi grout pressure over a 5-minute hold time during installation.

The use of steeply inclined ground anchors that bond into deeper zones of Stratum 2 and/or Stratum 3 to take advantage of higher bond stresses at depth are likely not feasible. This is based on the relatively narrow width of the approach embankments, and physical conflicts associated with the proximity of the drilled shaft cap for the adjacent bridge abutment and spread footings for adjacent concrete retaining walls. Accordingly, we recommend ground anchors with relatively flat inclinations of 20 degrees below horizontal to avoid conflicts with adjacent foundations.

The factored pullout resistance for prestressed ground anchors should be calculated according to Equation 11.9.4.2-1 of AASHTO BDS. A minimum drillhole and bond zone diameter of 6 inches should be assumed for design. In general, the bond length of ground anchors in soil should be limited to 50 feet or less for constructability, with typical anchor loads of 25 to 100 kips in soil. The following minimum lengths should be used in design:

- Unbonded length: Minimum 15 feet or longer to extend beyond the critical failure surface of the wall. The critical failure surface is assumed to extend a horizontal distance at the wall base equal to the maximum of either H/3 or 5 feet, and project upwards away from the wall at an angle of 45° + ϕ'/2 above horizontal (see WSDOT Standard Plan 8.1-A3-3).
- Bonded length: Minimum 15 feet or longer to develop minimum required anchor resistance.

Estimated ground anchors loads (T_1, T_2, T_n) are based on the total lateral pressures in the retained zone (earth pressure plus groundwater and external loads) and the position of the anchors relative to the top of the wall. The tributary area method should be used to calculate anchor loads for walls with a single level of anchors and walls with multiple levels of anchors according to AASHTO Figure C11.9.5.1-1 and Figure C11.9.5.1-2, respectively. The resultant lateral load acting on the embedded portion of the soldier pile (R) should be calculated from these figures.

Table 24. LRFD Design Bond Stress Recommendations for Ground Anchors

Material [1]	AASHTO Soil Type Bond Stress Material Assumption [2]	Soil-Grout Bond Stress (psf) – Pressure-grouted Anchors [2,3]		Soil-Grout Bond Stress (psf) – Gravity-grouted Anchors [2]	
		Nominal	Factored [4]	Nominal	Factored [4]
Stratum 1	Sand: Fine-medium grained, medium dense	2,000	1,950	1,000	650
Stratum 2	Sand: Fine-medium grained, medium dense-dense	3,000	1,950	2,000	1,300
Stratum 3	Sand: Medium-coarse grained, dense-very dense	7,000	4,550	3,000	1,950
WSDOT Common Borrow	Sand: Fine-medium grained, medium dense-dense	2,000	1,300	1,000	650
WSDOT Select Borrow	Sand: Fine-medium grained, medium dense-dense	3,000	1,950	2,000	1,300
WSDOT Gravel Backfill	Sandy gravel, medium dense-dense	--- [5]	--- [5]	3,000	1,950
UL-FGA	Sand: Medium-coarse grained, medium dense-dense	--- [5]	--- [5]	2,500	1,625

Notes:

1. See Tables 1 and 2 for material descriptions, specifications, and other soil design properties.
2. Corresponding to presumptive bond stress values for various materials as presented in Table C11.9.4.2-2 of AASHTO BDS.
3. Assumes pressure-grouted anchors with recommended minimum 50 psi grout pressure achieved over a 5-minute hold time during installation.
4. Considers LRFD resistance factor of 0.65 for cohesionless soils per AASHTO BDS Table 11.5.7-1. If the Allowable Stress Design (ASD) method is used, a factor of safety equal to 2.5 should be applied to the nominal values to obtain allowable values.
5. High permeability of these materials may not permit development of required grout pressures during pressure grouting. Post-grouting may allow the design bond stress to be increased by 25% or more.

Lateral Earth Pressures

Trapezoidal Apparent Earth Pressure (AEP) diagrams should be used for the design of the anchored SW, SE, and NE Wingwalls. The AEP distributions for cohesionless soils shown in AASHTO BDS Figure 3.11.5.7.1-1 should be used for design for walls with either one level of anchors (a) or multiple levels of anchors (b).

The maximum ordinate of the AEP diagram is computed differently for walls with one level of ground anchors versus walls with multiple levels of anchors as reflected by AASHTO BDS equations 3.11.5.7.1-1 and 3.11.5.7.1-2, respectively. The equations incorporate the soil active earth pressure coefficient (Ka), effective soil unit weight (γ'_s), wall height (H), and location of anchors along the wall height (H_1, H_2, H_n, H_{n+1}). Refer to **Tables 1** and **2** for recommended

parameters of the various retained soil materials. For the seismic load case, the seismic earth pressure coefficient Kae can be substituted for Ka in these equations.

Groundwater and external surcharge loads should be considered separately and added to the AEP as per recommendations in **Section 6.3.15**.

For embedded portions of the soldier piles below the lowest excavation level in front of the wall, the lateral earth pressure recommendations provided in **Section 6.3.15** apply.

Soldier Pile Lateral Load Resistance

The resultant lateral load from the AEP distribution acting on the embedded portion of the soldier pile (R per AASHTO BDS Figures C11.9.5.1-1 and C11.9.5.1-2) can be resisted by the passive earth pressure of the soil. The passive resistance and loads on the embedded portion of the soldier pile can be evaluated according to the simplified earth pressure distribution in AASHTO BDS Figure 3.11.5.6-1 with the additional R load, and the earth pressure recommendations provided in **Section 6.3.15**.

Alternatively, the lateral load resistance and required pile embedment can be evaluated by the P-y method using software such as LPILE or PYWALL by Ensoft. Recommended P-y method analysis parameters are provided in **Table 1**. P-multipliers should be employed as reductions to the available lateral load resistance when the center-to-center soldier pile spacing is less than 5D according to recommendations in **Table 12**.

Soldier Pile Axial Load Resistance

The tension force in the angled ground anchors will impart a vertical load component onto the soldier pile that must be resisted by the embedded portion of the soldier pile below the excavation line as an axial load. Resistance to axial loads can be developed by a combination of side friction and end bearing resistance on the embedded portion of the soldier pile with structural concrete annular backfill.

Axial resistance recommendations were developed based on the Beta method for side resistance and the SPT-based method (Brown et al. 2010) for end bearing resistance as per AASHTO BDS. AECOM recommends soldier piles bearing in Stratum 2 be designed using a nominal unit end bearing resistance of 44 ksf, and nominal unit side resistance of 0.9 ksf. Soldier piles should extend a minimum of three diameters into the bearing stratum below the lowest surrounding excavation depth, and side resistance above the lowest surrounding excavation depth should be ignored.

Resistance factors of 0.55 and 0.50 should be applied to side resistance and end bearing resistance, respectively, for the strength limit state per AASHTO BDS. Resistance factors of 1.0 should be applied to the service and extreme limit states. Additionally, for the service load case, AECOM recommends the factored side resistance be limited to 90% of the nominal value and the factored end bearing resistance be limited to 30% of the nominal value to limit settlement of the soldier pile to 1 inch or less based on the normalized load transfer curves presented in AASHTO BDS.

Punching of the soldier pile steel section through the concrete should be checked as a potential failure mode. Resistance to punching failure is provided by friction along the steel-concrete interface.

Wall Internal Drainage

Internal drainage should consist of 4-foot-wide prefabricated geocomposite strip drains placed between each set of adjacent soldier piles. The strips drains should consist of a polymer internal drainage net or core with external non-woven filter fabric on both sides. The strip drains should be attached to the front face of the timber lagging and behind the permanent concrete wall

facing. The strip drains should empty into either weepholes or wall underdrain pipe system and drain by gravity.

Refer to WSDOT Standard Drawings for Soldier Pile/Tieback Walls, and WSDOT Standard Specifications Item 6-16 for additional details and requirements.

Preliminary Wall Design Layout

Preliminary design evaluations were conducted for the maximum height section for each of the SW, SE, and NE Wingwalls using PYWALL software by Ensoft. Apparent earth pressures and P-y method parameters (Tables 1 and 2) were used in the analysis. A uniform traffic surcharge pressure of 250 psf was considered. The retained soil was modeled as WSDOT Gravel Backfill.

Preliminary analysis results indicate that the SE wall section is the critical section, and soldier piles with the following criteria are capable of resisting anticipated static lateral earth pressures and limiting lateral deflection to 1.0 inch or less for static loading:

1. Soldier piles consisting of double C15x33.9 steel channels placed into 30-inch-diameter (D) drill holes.
2. Soldier pile drill holes backfilled with structural concrete (4,000 psi) below the excavation line in front of the wall. Low-strength lean concrete should be used to backfill drill holes above the excavation line, and this material was ignored in the analysis.
3. Soldier pile spacing of 5 feet on-center (2D), with appropriate p-multipliers for closely-spaced piles applied internally in the software.
4. Soldier pile embedment of at least 20 feet below final ground surface at toe of wall.
5. Two levels of prestressed ground anchors for each soldier pile, each installed in minimum 6-inch-diameter drill holes and inclined at 20 degrees below horizontal as follows:
    a. Upper anchor located at 8 feet below the top of the wall, with minimum unbonded length of 20 feet, minimum bonded length of 36 feet, and unfactored design load of 67 kips per anchor.
    b. Lower anchor located at 17.5 feet below the top of the wall, with minimum unbonded length of 15 feet, minimum bonded length of 18 feet, and unfactored design load of 33 kips per anchor.
6. Prestressed ground anchors were evaluated as #9 steel all-thread bars with 150 ksi yield strength and elastic modulus 29,000 ksi. Equivalent 7-wire steel strand anchors with 270 ksi yield strength may also be considered for prestressed ground anchors.
7. Wall backfill consisting of WSDOT Gravel Backfill for Walls within a zone extending at least 5 feet horizontally beyond the back of the soldier piles at the base of the excavation and extending upwards away from excavation base at 1.5H:1V or flatter.

Corrosion Protection

Post-tensioning institute (PTI) Class I double corrosion protection (encapsulated anchor tendon) is recommended for prestressed ground anchors since the proposed walls are permanent.

Load Testing

Proof testing should be required for every production prestressed ground anchor. Pre-production performance tests should be performed in at least 4 sacrificial prestressed ground anchors (2 per abutment). Recommended loading schedules for proof and performance tests are provided in WSDOT Standard Specification 6-17.

6.5.6 Wingwall Design and Construction Considerations

Design and construction considerations for the recommended and alternative wingwall types are provided in the following sections.

MSE Walls

Special attention to design of the reinforcements will be needed to address the following issues: 1) plan view overlap of MSE reinforcements near obtuse external corners in the wall alignment; and 2) potential overlap with the adjacent spread footings of the bridge abutment walls and/or permanent concrete walls (if used). These issues are routinely encountered in MSE walls for transportation projects and are usually accommodated by skewing the reinforcements up to 15 degrees from perpendicular with the wall facing and maintaining at least 4 inches of compacted earthfill above and below each soil reinforcement where overlap is necessary.

Anchored Soldier Pile Alternative

Typically soldier pile walls are used in cut-type wall applications, but some degree of fill-type wall construction is anticipated for the wingwalls. Construction sequencing means and methods are the responsibility of the construction Contractor. However, for planning purposes, construction of the wingwalls may be accomplished entirely as a fill-type wall, in which excavation of the existing embankment is performed down to surrounding road grade, the cantilever soldiers piles are installed (before or after construction of the bridge abutment) with “stick up” above surrounding grade to the final top of wall elevation, soldier piles are braced from excessive deflections during backfilling, and backfill to final grade is completed in lifts with installation of ground anchors as applicable. Bracing could consist of temporary structural members and/or earthen berms. Timber lagging placed in between the flanges of adjacent soldier piles could serve as earth forms to accomplish the backfill. Soldier piles splices could be considered for taller wall sections, if allowed by structural design. One advantage of this approach is that the soldier piles could potentially serve as temporary shoring for the bridge abutment footing excavation, in which case the temporary loading direction is opposite of final wall loading direction.

Alternatively, the wall could be constructed as a partial cut-type and fill-type by building up the embankment fill to an intermediate level prior to soldier pile installation. This approach would require a temporary slope in front of the wall face that would be excavated following completion of wall construction, with temporary timber lagging placed between soldier pile flanges concurrently during excavation. The advantage of this approach is that the interim stick-up of the soldier piles is minimized, and the temporary berm will aid in reducing wall deflections during backfilling. This approach is likely preferable for wingwalls requiring ground anchors, since a minimum 15-foot-wide work bench is typically needed in front of the wall for drilling anchors with conventional drill rigs. However, there may be insufficient right-of-way in all areas of the wingwalls for this approach to be feasible. In that case, limiting the height of the intermediate embankment and using “reach-over” type drill rigs (i.e., drilling unit attached to the boom of an excavator) operating from the interim backfill behind the wall could be used to drill the ground anchors.

6.5.7 Temporary Lagging

Temporary timber lagging is anticipated for the NW wingwall, and each of the wingwalls which adopt the anchored soldier pile alternative. Temporary timber lagging should comply with WSDOT Standard Specification 6-16 for “Soil Type 1” (i.e., “coarse-grained soils either GW, GP, GM, GC, SW, SP, or SM that are medium dense to dense”) assuming WSDOT Gravel Borrow for Walls is used in the retained zone.

6.6 Global Stability Evaluation

Global stability evaluations were performed for the embankment and retaining walls using GeoStudio Slope/W software by Seequent. The software uses two-dimensional limit equilibrium methods and search algorithm to locate the critical slip surface with the lowest factor of safety, defined as the ratio of resisting forces to the driving forces. Spencer's method was used for interslice force assumptions. For the seismic pseudostatic case, a horizontal seismic coefficient (k_h) equal to 50% of the PGA was employed (0.25g).

Graphical output of the analysis results is provided in **Appendix F** and summarized below in **Table 26** through **28**. The results indicate that the proposed wall configurations for both the west and east approaches meet the minimum required factors of safety. A minimal embedment of about 6 feet is satisfactory for soldier pile walls to satisfy global stability, indicating that external stability failure modes (i.e., overturning and lateral translation) govern embedment depth for soldier pile walls.

Table 25. Global Stability Analysis Results – Bridge Abutment Walls

Analysis Case	Minimum Factor of Safety	Minimum Calculated Factor of Safety	
		West Abutment Wall (H=26.3')	East Abutment Wall (H=26.8')
Static [2]	1.5	1.9	1.7
Seismic Pseudostatic	1.1	1.2	1.2

Notes:
1. Per AASHTO BDS Section 11.6.3.7.
2. Only drained strengths were evaluated, and undrained strengths do not apply due to predominantly cohesionless nature of existing native soils and proposed fill materials.

Table 26. Global Stability Analysis Results – Permanent Concrete Walls

Analysis Case	Minimum Factor of Safety [1]	Minimum Calculated Factor of Safety			
		West Approach – Concrete Walls		East Approach – Concrete Walls	
		SW (H=32')	NW (H=20.5')	SE (H=32')	NE (H=15')
Static [2]	1.5	2.0	1.9	2.1	1.8
Seismic Pseudostatic	1.1	1.5	1.3	1.4	1.1

Notes:
1. Per AASHTO BDS Section 11.6.3.7.
2. Only drained strengths were evaluated, and undrained strengths do not apply due to predominantly cohesionless nature of existing native soils and proposed fill materials.

Table 27. Global Stability Analysis Results – Permanent MSE Walls

Analysis Case	Minimum Factor of Safety [1]	Minimum Calculated Factor of Safety			
		West Approach – MSE Walls		East Approach – MSE Walls	
		SW (H=29.5')	NW (H=17.8')	SE (H27.7')	NE (H=29.5')
Static [2]	1.5	2.1	2.1	2.4	2.2
Seismic Pseudostatic	1.1	1.2	1.1	1.3	1.2

Notes:
1. Per AASHTO BDS Section 11.6.3.7.
2. Only drained strengths were evaluated, and undrained strengths do not apply due to predominantly cohesionless nature of existing native soils and proposed fill materials.

Table 28. Global Stability Analysis Results – Soldier Pile Walls

Analysis Case	Minimum Factor of Safety [1]	Minimum Calculated Factor of Safety			
		West Approach – Soldier Pile Walls		East Approach – Soldier Pile Walls	
		SW – Anchored (H=32')	NW - Cantilever (H=20.5')	SE – Anchored (H=??)	NE - Anchored (H=??)
Static [2]	1.5	1.9	2.3		
Seismic Pseudostatic	1.1	1.2	1.3		

Notes:
1. Per AASHTO BDS Section 11.6.3.7.
2. Only drained strengths were evaluated, and undrained strengths do not apply due to predominantly cohesionless nature of existing native soils and proposed fill materials.

6.7 Traffic Signal and Luminaire Foundations

The design of traffic signal and luminaire foundations should be performed in accordance with WSDOT Standards Plans J-21.10-04, J-26.10-03, and J-28.30-03 and the BDM. The standard plan designs are per WSDOT GDM Section 17.2, including Table 17-2, which provides a correlation between SPT N values (blows/foot) and allowable lateral bearing pressure (ALBP). The design recommendations provided in the GDM are applicable to relatively flat ground conditions. Note that the recommended WSDOT standards disregard the effect of the groundwater table on the proposed foundation size.

A review of the boring logs located in the vicinity of the proposed luminaires and traffic signal poles indicate that the near-surface subgrade soils are comprised of loose to medium dense sand with silt and gravel with SPT N-values ranging between 8 and 16 blow counts per foot. The field observation and site survey topography indicated that the ground surface at the proposed signals or luminaire poles are generally level to gently sloped. Based on the anticipated subsurface conditions, AECOM recommends an ALBP of 1,500 psf for pedestrian signals and luminaire poles and an ALBP of 1,000 psf for traffic signals. A friction angle of 30 degrees may be used to calculate the torsional capacity of the structure.

Where the foundations require excavation to depths no greater than about 5 ft, open excavations may be used subject to site constraints. Foundations should be backfilled with Gravel Borow compacted in 6-inch-thickness lifts. Foundation excavations below approximately Elevations +23 to +26 feet (depending on time of year) are expected to encounter groundwater. We recommend installing these foundations as drilled piers using temporary casing, with drilling slurry if required, to maintain a stable excavation. If the piers are installed with permanent casing, steel casing should be used in lieu of Sonotubes®.

6.8 Pavement Design

Pavement design recommendations are presented in the following sections.

6.8.1 Proposed Pavement Rehabilitation and Construction

The proposed work on the SR 202 roadway includes rehabilitation of the existing pavement by means of a mill and overlay as well as widening at 131st Avenue NE (SR 202) between NE 175th Street and NE 177th Place. The project extends from approximately 131st Ave. NE Stations 102+70 (SR 202 MP 0.31) to 108+00 (SR 202 MP 0.23) with an approximate length of 530 feet from south to north. To accommodate increasing traffic, 131st Ave. NE will be widened, and traffic lanes will be reconfigured to increase the roadway width from five lanes to eight lanes.

The project elements as it relates to the pavement include:

- Milling the existing pavement and overlaying with new asphalt pavement,

- Constructing new pavement where 131st Ave. NE will be widened on the east side of the roadway.

The pavement engineering analysis includes:

- Determining the required milling and asphalt overlay thickness on the existing pavement to accommodate future traffic
- Determining a full depth construction section for the planned widening work

6.8.2 Existing Pavement Section

Based on the summary of the historical and current pavement cores (discussed in detail in Section 5.3), the existing pavement consists of the following two main sections:

- **Section 1**: 6.5 to 9 inches of asphalt concrete (AC) pavement over gravelly sand base course/subgrade soils (two cores were taken in this location)
- **Section 2**: 11 to 17 inches of asphalt pavement on top of 8 inches of ATB in one location and over 6 to 7 inches of Portland Cement Concrete (PCC) pavement on top of gravelly sand base course/subgrade soil (three cores)

The pavement section transition is estimated to be at approximately 131st Avenue NE Station 104+50, with Section 1 located south of this transition.

6.8.3 Subgrade Support

Based on the boring logs, the subgrade soil is composed of poorly graded sand (SP-SM), which is uniform across the project site. A CBR test performed previously by others on a soil sample from one location resulted in a value of 28.

Considering the limited number of CBR tests and the typical CBR range of 15-25 for this type of soil encountered on the site, a CBR value of 15 (equating to approximately 14,500 psi subgrade resilient modulus) was used for design purposes. This is consistent with previous design assumptions for this roadway.

6.8.4 Design Standards

The pavement design analysis was conducted following the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures and the Washington State Department of Transportation (WSDOT) Pavement Policy (2018). The WSDOT Pavement Policy recommends using the AASHTO Guide for the design of both Hot-Mix-Asphalt (HMA) and Portland Cement Concrete Pavements (PCCP). Accordingly, the pavement sections were determined using PAVEXpress (for the widening full depth construction and asphalt overlay on the existing asphalt pavement) and Darwin pavement design programs (for the asphalt overlay on the existing composite pavement), which both employ empirical equations from the AASHTO design guide considering estimated traffic loading and other relevant pavement input parameters.

The subsequent subsections outline the methodology and inputs used for flexible and rigid/composite pavement designs.

6.8.5 Traffic Loading Analysis

For design purposes, traffic was converted to equivalent 18-kip single axle loads (ESALs) using the approach outlined in the 1993 AASHTO pavement design guide. Traffic estimates, including Average Annual daily traffic (AADT) and traffic growth factor, used for the design were based on information obtained from TENW's ICE report and confirmed by AECOM's Traffic Engineer:

- AADT (2022): 39,500 vehicles per day,

- Estimated percent trucks: 3.54%,
- Expected growth rate: 1% annual.

The estimated equivalent ESAL values were calculated for the full depth widening asphalt section for 20, 30 and 50-year design life. For the composite pavement designs, a 15-year ESAL was calculated as required by WSDOT. To convert the AADT to ESALs, truck factors shown in **Table 29** (recommended values by PaveXpress) were used. A directional distribution of 50% and a lane distribution factor of 75% were assumed. Based on these assumptions, the resulting 18-kip ESALs corresponding to the full depth widening asphalt section and asphalt overlay on the existing asphalt and composite pavements are presented in **Table 29**.

Table 29. Estimated Traffic Loading

Pavement Type	AADT (2022)	Vehicle Type	% of AADT	Truck Factor	Design Life, Years	Growth Factor	Directional Distribution	Design Lane Distribution	Total ESALs
Widening Full Depth Pavement Construction	39,500	Passenger Cars	96.46	0.0001	**50**	1.0%	50%	75%	**19,294,000**
		Trucks	3.54	1.5					
		Passenger Cars	96.46	0.0001	**30**	1.0%	50%	75%	**10,411,000**
		Trucks	3.54	1.5					
		Passenger Cars	96.46	0.0001	**20**	1.0%	50%	75%	**6,590,000**
		Trucks	3.54	1.5					
Existing Pavement Milling and Overlay	39,500	Passenger Cars	96.46	0.0001	**15**	1.0%	50%	75%	**4,818,000**
		Trucks	3.54	1.5					

6.8.6 Pavement Design Parameter and Approach

Design analyses were conducted for the full depth widening section and pavement rehabilitation sections (asphalt overlay on existing asphalt pavement, Section 1, and asphalt overlay on existing composite pavement, Section 2), following the 1993 AASHTO design guide and WSDOT Pavement Policy. The design input parameters are summarized below in **Table 30**.

For the full depth widening section, a flexible pavement design was developed based on the anticipated future traffic and design input parameters. For the existing pavement design, the required milling and asphalt overlay thickness to accommodate the design traffic was determined based on the necessary structural capacity to meet future traffic demands and the existing pavement's structural capacity.

Table 30. Pavement Design Input Parameters

Parameter	Asphalt Pavement Design (Full Depth/Overlay Design)	Composite Pavement Overlay Design
CBR	15	15
Reliability	95	95

Parameter	Asphalt Pavement Design (Full Depth/Overlay Design)	Composite Pavement Overlay Design
Initial serviceability	4.2	4.5
Terminal serviceability	2.5	2.5
Standard Deviation	0.49	0.35
PCC Compressive Strength, psi	---	4,000
Layer Coefficient for Asphalt Layer	0.44	--
Layer Coefficient for Aggregate Base Layer	0.14	--

6.8.7 Analysis Results

Designs were developed for the full-depth widening section (20-, 30-, and 50-year design) and for the mill-and-overlay of the existing pavement (15-year design).

Widening Full Depth Construction Section: **Table 31** shows the minimum required pavement structural number (SN) based on total design traffic, subgrade condition, and other design inputs. Additionally, the table provides recommended pavement sections to meet this minimum SN requirement for a 20, 30 and 50-year design life.

- **20-Year Design Life**
  - ***Construct 7.5 inches of HMA layer,***
  - Construct 8 inches of graded aggregate base course,
  - Grade and compact the in-situ subgrade to achieve 95% compaction using modified Proctor method.
- **30-Year Design Life**
  - ***Construct 8 inches of HMA layer,***
  - Construct 8 inches of graded aggregate base course,
  - Grade and compact the in-situ subgrade to achieve 95% compaction using modified Proctor method.
- **50-Year Design Life**
  - ***Construct 9 inches of HMA layer,***
  - Construct 8 inches of graded aggregate base course,
  - Grade and compact the in-situ subgrade to achieve 95% compaction using modified Proctor method.

Table 31. Widening Full Depth Construction Section

Design	Design Life	Required SN	HMAC Surface (inch)	PCC Layer (inch)	HMA Base (inch)	Aggregate Base (inch)	Design SN
Widening Full Depth Construction Asphalt Section	20	4.00	2	--	5.5	8.0	4.42
	30	4.20	2	--	6.0	8.0	4.64

Design	Design Life	Required SN	HMAC Surface (inch)	PCC Layer (inch)	HMA Base (inch)	Aggregate Base (inch)	Design SN
	50	4.60	2	--	7.0	8.0	5.08

Existing Pavement Rehabilitation – Asphalt Overlay Section: **Table 32** presents the required minimum overlay thickness based the effective existing pavement thickness, underlying unbound layers, subgrade support, traffic, and other relevant design inputs.

Given the overall good condition of the existing pavement section and the non-structural nature of the cracking (longitudinal and transverse), milling 2 to 3 inches and crack sealing is recommended prior to overlay placement. With the recommended 2 to 3 inches milling and 2 to 3 inches overlay, the existing profile will remain as is.

- Mill the existing asphalt pavement up to 3 inches and crack seal,
- Place 3 inches of asphalt overlay.

The exception to this is a portion of the existing pavement near the north end of the project where a 3-inch reduction in grade is required. This involves milling 5 inches of the existing pavement below the existing grade and replacing it with only 2 inches, resulting in a 3 inches reduction in the overall asphalt section of the composite section (14 inches AC over 6 inches PCC). The structural analysis for this section indicates that the remaining structure is structurally adequate for a 15-year design life.

- Mill the existing asphalt pavement up to 5 inches and crack seal,
- Place 2 inches of asphalt overlay.

Table 32. Existing Pavement Rehabilitation - Asphalt Overlay Sections

Design Section	Milling Depth (inch)	Existing AC Thickness (inch)	Existing PCC Thickness (inch)	Effective Existing Pavement Thickness (inch)	Required HMA Overlay Thickness (inch)	Recommended Overlay Thickness (inch)
Asphalt Overlay on Existing Asphalt Pavement – Section 1	3	7.75	---	4.75	2.6	**3.0**
Asphalt Overlay on Existing Composite Pavement – Section 2	3	14	6.0	10.35	0.0	**3.0**
Asphalt Overlay on Existing Composite Pavement – Section 2 (Where grade reduction required)	5	14	6.0	9.45	1.28	**2.0**

6.8.8 Pavement Design Recommendations

This section outlines the minimum recommended rehabilitation (involving milling and placing asphalt overlay) and full depth asphalt pavement construction sections.

Widening – Full-Depth Construction Section (50-year Design)

- Construct 9 inches of HMA layer
  - 2 inches HMA Class ½ inch Surface Course with PG 58H-22 Binder
  - 3.5 inches HMA Class ½ inch Base Course with PG 58H-22 Binder (two lifts)
- Construct 8 inches of graded aggregate base course
- Grade and compact the in-situ subgrade to achieve 95% compaction using modified Proctor method.

Existing Pavement Rehabilitation – Asphalt Overlay Section (15-year Design)

- Mill the existing pavement up to 3 inches and crack seal
- Construct 3 inches of HMA overlay layer
  - 1.5 inches HMA Class ½ inch Surface Course with PG 58H-22 Binder
  - 1.5 inches HMA Class ½ inch Base Course with PG 58H-22 Binder

Existing Pavement Rehabilitation – Asphalt Overlay Section (15-year Design)

This applies only to the existing composite pavement section where a 3-inch grade reduction is required.

- Mill the existing pavement up to 5 inches and crack seal
- Construct 2 inches of HMA overlay layer
  - 2 inches HMA Class ½ inch Surface Course with PG 58H-22 Binder

6.9 Earthwork and Construction Considerations

6.9.1 General

The WSDOT/American Public Works Association Standard Specifications for Road, Bridge, and Municipal Construction (WSDOT/APWA 2026) should be used for construction of the Project.

Final Project Plans and Specifications should be reviewed by an AECOM geotechnical representative prior to construction to confirm that the full intent of the recommendations presented in this report have been applied to the design and that the recommendations presented are applicable to the final scope of the project. Following review of Plans and Specifications, sufficient and timely observations during construction should be performed to correlate findings of the investigation with actual subsurface conditions exposed during the construction. Observation and testing by a qualified geotechnical professional should be performed during construction.

As described in Section 5.3.5, a limited environmental investigation performed in 2023 for the Woodinville Water District detected low concentrations of contaminants in soil and/or groundwater on the west and east sides of 131st Avenue NE. During construction, the excavated soils and groundwater from possible dewatering should be removed and disposed of properly in accordance with appropriate environmental protocols based on the findings and guidance provided in the environmental investigation report in Appendix E. The Contractor shall perform additional environmental investigation as required to facilitate proper disposal.

All utility lines within the project limits should be identified and located by the Contractor with prior one-call notification to the Washington Utility Notification Center and confirmed by the as-built plans and/or potholing prior to any proposed earthwork to determine proximity to adjacent existing utility lines. These utility lines may be non-yielding and their tolerance to soil movement during excavation may be low. All active or inactive utilities within the construction limits should be identified for relocation, abandonment, or protection in place prior to grading.

Prior to any site work and excavations, conditions of existing structures and improvements should be surveyed and photo documented.

6.9.2 Subgrade Preparation

Weeds, grass, trees, shrubs and construction debris within areas that are to be graded should be cleared and properly disposed of off-site. Roots and other vegetative matter should be removed and disposed either offsite or stockpiled for reuse in landscape areas. Existing infrastructure should be properly demolished and disposed in accordance with WSDOT Standard Specifications Section 3-01.2. Existing bridge foundations, if being demolished, should be removed to a depth of at least 2 feet below finished ground elevation or subgrade elevation, whichever is lower, in accordance with WSDOT Standard Specifications Section 3-02.3(2).

Removal of existing unsuitable materials may be required to expose suitable material on which to place fill. Removals should generally extend into competent materials as observed and approved by a qualified Geotechnical Engineer.

Following the clearing of weeds, grass and debris, and upon completion of removals, the surface at subgrade should first be proof compacted and then proofrolled to densify zones of loose granular soil or to help identify very soft zones that may require mitigation. The initial proof compaction should be performed over the entire footprint of the excavated and replaced embankment and should consist of a vibratory drum roller having a minimum static drum weight of 10,000 pounds and capable of at least 20,000 pounds of dynamic force. Static compaction may be necessary if the natural soils are wet, which would make them more susceptible to disturbance during proof compaction. Proofrolling should consist of one pass with a loaded dump truck. If proofrolling results in more than a few inches of deformation where the embankment will be greater than 5 feet high, the ruts should be filled with Common Borrow compacted to at least 90 percent of its maximum dry density as per ASTM D 1557 (Modified Proctor), and with Select Borrow to pavement subgrade elevation. Where the embankment will be 5 feet high or less, the ruts should be filled with compacted Common Borrow. Filling should then continue with placement of Common Borrow to pavement subgrade elevation. The Contractor should consult WSDOT Standard Specifications Sections 3-02.3 regarding the details and considerations of removing the existing foundation, bridge, and other buried structures to eliminate the damage to existing structures and stability of cuts.

The existing embankment and ground surface behind the trestle will be excavated to the target elevation and replaced by compacted Select Borrow material with an achieved top layer length of about 60 feet from the edge of the proposed bridge. The excavation of existing embankment will be long enough to allow for a remaining embankment front slope of 1.5H:1V until the new fill is placed. The Contractor should pay close attention to the location of buried structures or utilities to avoid damage to the existing facilities during the excavation.

6.9.3 Bridge Approach and Embankment

Bridge approach embankment is defined as fill within 100 feet of the bridge abutments and must be constructed per WSDOT Standard Specifications Section 3-03.3(14)B.

Embankment Fill Materials

For construction of the embankments greater than 5 feet high, AECOM recommends that Select Borrow be used per Section 9-03.14(2) of the WSDOT Standard Specifications. The Select Borrow should consist of granular material, either naturally occurring or processed, and should meet the WSDOT requirements for grading and quality displayed in **Table 33**. We expect that some suitable material for Select Borrow may be obtained on-site from excavations into Stratum 1 and/or Stratum 2, provided that silty/clayey intervals are screened and removed from stockpiles of material proposed for Select Borrow.

Table 33. Select Borrow Grading and Quality

U.S. Standard Sieve Size	Percent Passing
4 inches (1)	99-100
3 inches	75-100
No. 40	50 max.
No. 200	10.0 max.
Sand Equivalent	30 min.

All percentages are by weight.
(1) 99 to 100 percent shall pass 4-inch-square sieve and 75 to 100 percent shall pass 2-inch sieve when select borrow is used in the top 2 feet of embankments.

For construction of the embankments 5 feet or less which will be used to raise the height of existing embankment to comply with proposed bridge grading, AECOM recommends that Common Borrow be used per Section 9-03.14(3) of the WSDOT Standard Specifications. The Common Borrow should consist of granular or nongranular material, which is free of deleterious material. Deleterious material includes wood, organic waste, coal, charcoal, or any other extraneous or objectionable material. The quality of Common Borrow can be visually specified by the project engineer or in accordance with AASHTO T 194 or AASHTO T 267 and should meet the WSDOT Standard Specifications soil plasticity Option 1, with passing sieve No. 200 between 0 to 12 percentage and no plasticity. On-site excavated granular soils are generally considered suitable for reuse as common borrow.

Retaining Wall Backfills

For construction of retaining wall backfill behind the bridge abutment walls, concrete retaining walls, and the SW, SE, NE Wingwalls, AECOM recommends use of Gravel Backfill for Walls meeting the requirements of Section 9-03.12(2) of the WSDOT Standard Specifications. The Gravel Backfill for Walls should consist of an imported crushed-stone material.

For the NW Wingwall, ultra-lightweight foamed glass aggregate (UL-FGA) backfill should be used. The UL-FGA should be supplied by AeroAggregates® of North America, LLC or Engineer-approved equal. See **Section 6.5.3** for additional recommendations for the UL-FGA backfill.

For all retaining wall backfill, the wall backfill should be located within a zone extending at least 5 feet horizontally beyond the back of the wall at the base of the excavation and extend upwards away from excavation base at 1.5H:1V or flatter (2H:1V if possible contingent upon site constraints).

Placement of Fill and Compaction

Select and Common Borrows that will support structures and roadways should be placed and compacted in accordance with WSDOT Standard Specification Section 3-03.3(14)C, Method B, unless otherwise approved by the geotechnical engineer. Bridge approach embankments shall be compacted to at least 95 percent of the maximum density as determined by the tests described in WSDOT Standard Specifications Section 3-03.3(14)D. The Contractor should not

backfill the abutments prior to placing the bridge superstructure, unless working drawings and calculations confirming the abutment is stable for overturning and sliding without the superstructure in place. After the superstructure is in place, use of small compactors may be required to compact the backfill around the structure.

The compaction moisture content should be within 2 percent of the optimum value per ASTM D 1557. We recommend that compaction tests be performed at regular intervals to determine the effectiveness of the compaction operations. To reduce the potential for embankment slope face instability, we recommend overbuilding the embankments laterally at the slope face approximately 2 feet, compacting the fill, and then trimming off the outer 2 feet of the embankment to produce a well compacted slope face.

For the widening of the existing embankment, terracing of the existing slope should be performed to key the new structural fill material into the existing embankment. Keying helps stabilize steep slopes and reduces erosion potential of a steep slope during construction. Terracing should be performed as specified in WSDOT Standard Specifications Section 3-03.3(14). The face of each terrace should be 1 to 2 feet high and vertical or near vertical as required to remain stable.

The upper six (6) inches of pavement subgrade should be compacted to at least 95 percent of the maximum density as determined by the tests described in WSDOT Standard Specifications Section 3-03.3(14)D.

6.9.4 Excavation and Temporary Slopes

Temporary excavations up to 5 to 10 feet deep for the proposed foundation supports may be required for construction. Temporary excavations should be sloped or will require shoring. Excavation should be avoided during the wet season, or construction dewatering is required. The stability of temporary excavations is a function of several factors, including the total time the excavation is exposed, moisture conditions, weather, soil type and consistency, and Contractor's operations. The Contractor is responsible for excavation safety.

The support of temporary excavations is the responsibility of the Contractor. Shoring is usually designed as either cantilever (unbraced) or internally braced/anchored. Cantilevered shoring is commonly constructed by either using soldier piles with lagging placed between piles or using sheet piles. If soldier piles and lagging are used, continuous lagging may be required. Difficulties in driving soldier piles into dense native soils with gravel and cobbles and installing the lagging due to caving cohesionless soils should be anticipated. If the soldier pile and lagging system is used, earth pressure behind the retained earth needs to be considered.

Surcharge loads from vehicles and stockpiled material should be kept away from the top of temporary excavations at a distance equal to at least one half of the excavation depth. During wet weather, runoff water should be prevented from entering the excavation and collected and disposed of outside the construction limits. To prevent runoff from adjacent areas from entering the excavation, a perimeter berm should be constructed at the top of the slope.

Design of temporary shoring is the responsibility of the Contractor and must be prepared by a Washington-licensed Professional Engineer. Cantilever temporary shoring should be designed for a triangular distribution of active earth pressure as recommended in **Section 6.3.15**. Internally braced or anchored shoring should be designed using a trapezoidal apparent earth pressure distribution with maximum pressure ordinate equal to 0.65 x unit weight x K_a x shoring retained height, with applicable values of unit weight and K_a as presented in **Section 6.3.15**. Shoring should be monitored for lateral and vertical movement. If large deflections (greater than 0.5 percent of the shoring height) are noted, the bracing systems should be checked and strengthened as needed. If tension cracks appear in the ground surface adjacent to the shoring,

the cracks should be monitored and sealed to prevent infiltration of water, and the significance of the cracks should be immediately evaluated.

In addition, the Contractor should strictly adhere to any requirements of WSDOT and applicable federal and state health and safety regulations such as those of the Occupational Safety and Health Administration (OSHA). In accordance with OSHA regulations, the near-surface onsite soils are predominately classified as Type C. Raveling of gravel and cobbles should be expected in sloped or shored excavations and could pose a potential safety concern to the construction personnel. The Contractor should have a competent person evaluate the soil conditions encountered during excavation to determine permissible temporary slope inclinations and other excavation safety measures as required by Washington Industrial Safety and Health Act (WISHA) regulations.

6.9.5 Adjacent Utility Construction

The design drawings indicate new storm drain (SD) utility lines will be installed in front of the permanent wingwalls, abutment walls, and/or permanent walls at both the east and west approach embankments. A sanitary sewer (SS) utility line is also proposed in in front of the walls at the east approach embankment. The centerlines of the SD are a minimum of about 12.5 to 15.5 feet from the wall faces, and the SS centerline is a minimum of about 8 to 10 feet from the wall faces. The invert elevation of the SS ranges from about El. +25.8 to +26.5 feet corresponding to depths of about 3.2 to 5.3 feet below the adjacent wall footings. The invert elevation of the SD ranges from about El. +27.7 to +27.9 feet corresponding to depths of about 1 to 1.5 feet and 1 to 3.5 feet below the wall footings on the west and east approaches, respectively. Several SD manholes with invert elevations of about El. +25.3 to +25.6 feet will also be located near the wall alignments. We expect trench excavations will be on the order of 4 to 5 feet wide, and manhole excavations will be on the order of 6 to 8 feet in diameter.

The utility Contractor will be responsible for stability of the utility trenches and manhole excavations. If the utilities are installed after the permanent walls have been constructed, the temporary shoring for these excavations must be designed to account for the additional surcharge loading from the adjacent permanent walls. The selected shoring system(s) should be rigid enough to minimize ground deflections to prevent excessive deflections and/or instability of the adjacent permanent walls. Installation methods of the selected shoring system should minimize the unsupported excavation height of the retained soil during utility construction to maintain global stability and prevent raveling of retaining soil between the shoring and the permanent retaining wall. In addition, temporary shoring design and installation should follow the recommendations of **Section 6.9.4**. The shoring design must account for loading from the adjacent permanent retaining walls and may require global stability evaluations specific to the selected shoring system.

6.9.6 Subsurface Drainage

Surface drainage should be directed away from the top of slopes and road surfaces. Ponded water at the top of slope and sheet flow over slope surfaces should not be allowed. During construction, project areas should be graded such that surface water can drain readily away from improvement areas. Given the encountered soil conditions, it is important to avoid ponding of water in or near excavations. Water that accumulates in excavations should be promptly pumped out or otherwise removed and these areas should be allowed to dry out before resuming construction. Berms, ditches, and similar means should be used to decrease storm water entering work areas and to efficiently convey it offsite.

6.9.7 Wet Weather Considerations

During the traditional wet weather period from November 1 through April 30, the workability of the clayey and silty sand soils will be adversely affected if these soils are exposed and subjected to direct rainfall and flow of surface water onto the site. In wet conditions, the fill soils are expected to soften and deteriorate quickly to a point where they are unsuitable for use in the pavement subgrade, embankment, or foundations of structures. Work during wet periods should be limited, and subgrades and fill stockpiles should be protected from rainfall and water collection.

6.9.8 Dewatering Considerations

Static groundwater measurements in the site monitoring wells B-4 (ZA05) and BT-2 between October 2022 and March 2024 ranged between Elevation +23 and +26.2 feet. The recommended design groundwater elevation is Elevation +25 feet. We understand the bottom of the existing bridge foundations is about 10.5 feet bgs (approximate Elevation +21.5 feet), which includes a 3-foot-thick concrete seal. The bottoms of the new abutment footings are expected to bear at about Elevation +25 feet. Excavation for removal of the existing bridge foundation on the west side and construction of the new bridge foundations may require construction dewatering.

Construction dewatering design is the responsibility of the Contractor. The Contractor's selected method of dewatering will depend on the excavation slopes and/or type of shoring selected, as well as the permeability of soils and the required height of groundwater drawdown. We expect that dewatering may include installing well-point dewatering wells around the excavation perimeter, supplemented with sumping and pumping. The soil below the groundwater level consists of primarily granular material with relatively high permeability and therefore high flow rates are expected. The estimated permeability of Stratum 2 is expected to range from $1x10^{-4}$ to $1x10^{-3}$ cm/s. Dewatering systems should maintain groundwater levels at least 2 feet below the lowest elevation of excavation subgrades which will receive fill and/or support structure foundations. Construction dewatering activities generally require permits through the Washington Department of Ecology and must comply with all applicable rules and regulations.

Based on prior site usage and suspected soil/groundwater contamination at adjacent properties, treatment of collected groundwater may be required before discharging to City sewers and/or bodies of water. In addition to or in lieu of on-site groundwater treatment systems and nearby discharge, containing and hauling groundwater off-site for treatment may be considered. We recommend the contract documents include performance-based specifications for construction dewatering and treatment which require the Contractor to submit a detailed work plan for review and approval by the Engineer.

6.9.9 Pilot Hole Verification Test Borings for Drilled Shaft Installation

Required depths of proposed drilled shaft foundations for the bridge abutment may extend below the lowest available soils information from the 2023 AECOM borings and previous borings by others. To confirm assumed subsurface conditions and available axial resistance for drilled shafts, AECOM recommends conducting two (2) pilot hole verification test borings (one per abutment). The test borings should be drilled at least 20 feet deeper than the design shaft tip elevation to allow for potential adjustments in tip elevation based on the results; anticipated boring depths range from approximately 120 to 130 feet deep if drilled from the top of the existing bridge approach embankments). Test borings should consist of mud rotary wash borings with SPT (ASTM D1586) sampling and testing at maximum 5-foot depth intervals. A recent (within 1 year) SPT hammer energy calibration report should be provided by the driller for the specific drill rig used to drill the borings. Borings should be logged by a qualified geologist or geotechnical engineer, and detailed boring logs should be prepared. Standard penetration test samples should be retained for potential laboratory testing, if necessary.

The pilot hole verification testing borings should be performed either by AECOM under supplemental authorization or incorporated into the construction contract as a pre-construction submittal by the Contractor submitted to AECOM for review and analysis. Potential adjustments in shaft length may be required depending upon the results of the borings.

6.9.10 Settlement Monitoring

Settlement monitoring is recommended for the MSE walls to confirm settlement is complete prior to installation of the final CIP concrete facing. Settlement monitoring is intended to confirm the design assumption that anticipated settlement due to fill placement is primarily elastic (immediate), or identify if there is a time-dependent component of the settlement due to isolated fine-grained silt layers identified in some borings. A settlement monitoring program reduces the risk of the permanent CIP facing being installed before settlement is complete, which could otherwise result in aesthetic damage such as cracking or differential settlement.

Settlement monitoring points should be established as early in wall construction as possible and be installed at or near the base of the wall (within 5 feet of the wall bottom) at accessible and secure locations that accurately reflect settlement of the wall and retained embankment as construction proceeds. Potential settlement monitoring point types/locations may include one or more of the following:

- Settlement plates placed within the MSE reinforced zone fill and/or just behind the reinforced zone within the embankment fill. Settlement plates typically consist of a flat steel plate with threaded connection for flush-threaded rods within a steel or polyvinyl chloride (PVC) riser pipe that penetrate up through the fill. Additional sections of threaded rod and sleeve are placed as the fill height increases to allow for continued surveying. This option is expected to provide the highest accuracy but requires the Contractor to work around and protect the rod and sleeve and may slow progress. Decommissioning of the settlement plate should include cutting and removing the riser pipe to 2 feet below finish grade once settlement is confirmed substantially complete, followed by filling of the remaining riser with lean grout.

- Survey benchmarks installed on the MSE wall face. These points would consist of steel rebar driven horizontally into the geosynthetic-wrapped face with a survey monument or prism affixed to the exposed end of the rebar. This option largely does not interfere with wall construction but may not capture some of the initial settlement since it needs to be installed on the wall face, and the accuracy may be inconsistent due to the potential deflection of the “cantilever” monument.

- Survey points established on the concrete “footing” at the base of wall, e.g., steel bolt or rebar either cast vertically into the footing concrete or post-drilled after the footing concrete cures. This option largely does not interface with wall construction. However, it requires the footing to be placed around the same time as the initial lift of MSE reinforced filll to capture initial settlement data and also requires the concrete footing at each survey point to remain exposed for the duration of monitoring, unless the bolt or rebar are extended up to the top of backfill at the toe of the wall.

Settlement monitoring points should be established by a licensed Professional Surveyor at spacing no greater than 25 linear feet along the MSE wall alignment; however, settlement plates can be limited to one plate per wall, with the plate installed at a location corresponding to 90 percent of the maximum MSE wall height, i.e., about 10 to 20 feet from the end of the MSE wall. We recommend survey monitoring be conducted weekly while fill placement is being conducted, and monthly for a period of 6 months after the wall backfill and retained embankment have been constructed to finish grade elevation over the full length of the approach. The monitoring duration

may be shortened at the discretion of the Geotechnical Engineer if the survey data indicates no trend of increasing settlement with time.

6.9.11 Monitoring Wells

The construction drawings should show the location of existing monitoring wells, as they may obstruct excavation and other construction operations. The known wells include the following (refer to **Figure 3**):

- B-1(ZA05)(MW): About 60 feet east of the existing trestle east abutment, near the south crest of the embankment. This well is damaged.
- B-4(ZA05)(MW): About 110 feet west of the existing trestle west abutment, in the southern half of the embankment crest.
- BT-2(MW): Near the northwest portion of the Jacksons Shell parking lot, about 20 feet east of the east curb of 131st Avenue NE.
- B-2(ZA06)(MW): In the sidewalk at the southeast corner of the intersection of 131st Avenue NE and NE 175th Street. The presence of this well has not been confirmed by AECOM.

The Contractor should also be verbally advised about the presence of these wells, and that other wells unknown to AECOM may also be present. The wells typically include 5 vertical feet of steel casing and road box near ground surface, and polyvinyl chloride (PVC) riser pipe from the bottom of the well to about 6 inches below ground surface.

7. LIMITATIONS

This Geotechnical Report has been prepared by AECOM for the exclusive use of the City of Woodinville and the SR 202 Widening and Trestle Replacement Project design team members. The report was prepared using the degree of care and skill ordinarily exercised under similar circumstances by responsible engineers and geologists practicing in the same general location. No other warranty or representation, either expressed or implied, is made as to the findings and professional advice in this report.

The analyses and recommendations contained in this report are based on the data obtained from previous and current investigations of field exploration, laboratory testing, soil design parameters, and provided plans. The boring logs indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. They do not necessarily reflect variations that may exist between locations or possible changes that may take place with time and depth. If variations in subsurface conditions from those described in this report are noted during construction, the recommendations presented in this report should be re-evaluated.

In the event that any change in the nature, design, or location of the proposed structures occurs, the conclusions and recommendations of this report should not be considered valid unless such changes are reviewed, and the conclusions of this report are modified or verified in writing by the AECOM engineer of record. AECOM is not responsible for any claims, damages, or liability associated with the reinterpretation or reuse of the subsurface data or engineering analyses in this report by others.

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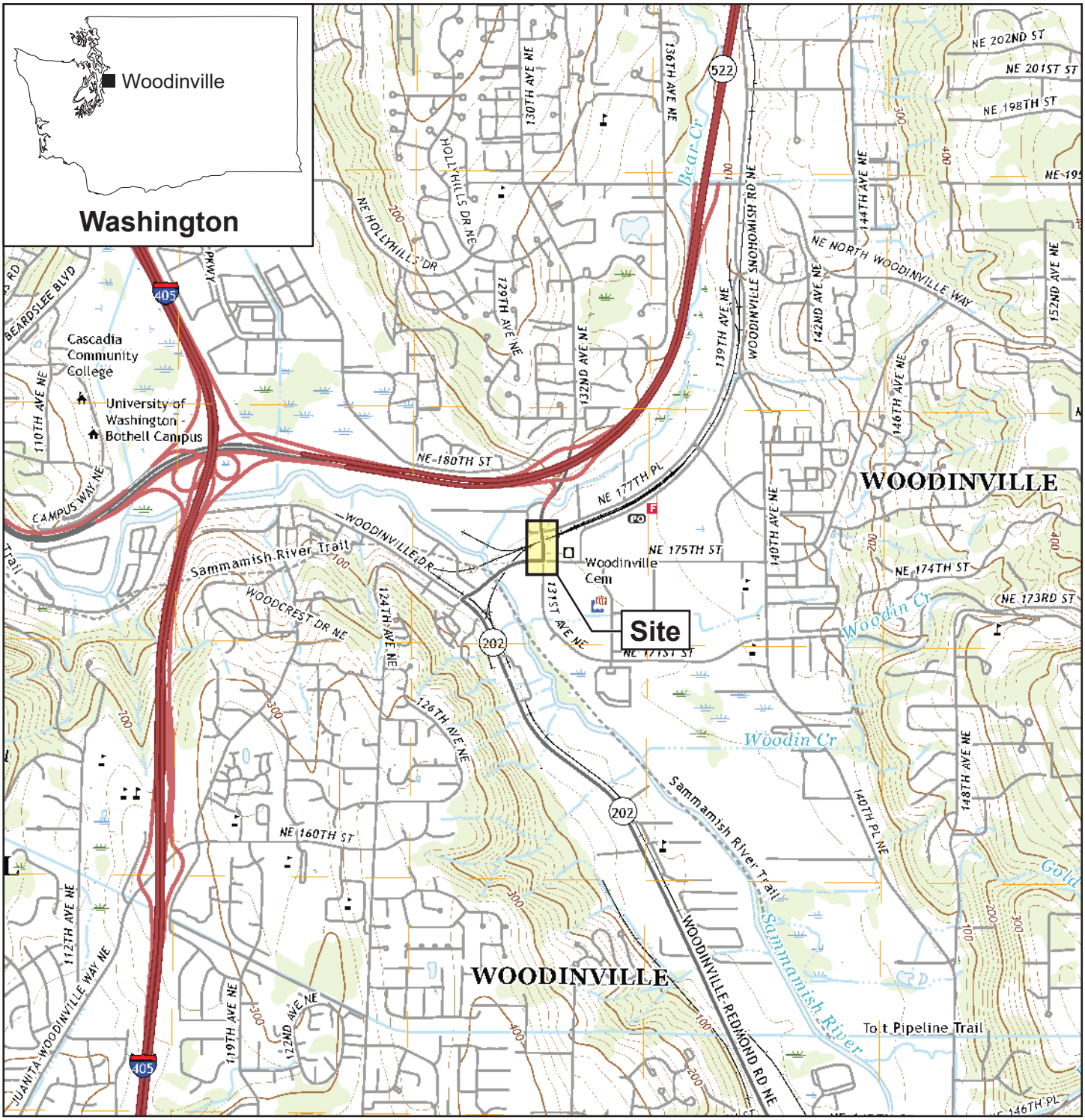
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FIGURES

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Source: USGS 7.5-minute topographic quadrangles:
Woodinville, Washington, 2020; and Kirkland, Washington, 2020

0 2,000 4,000

Scale in Feet

Figure 1

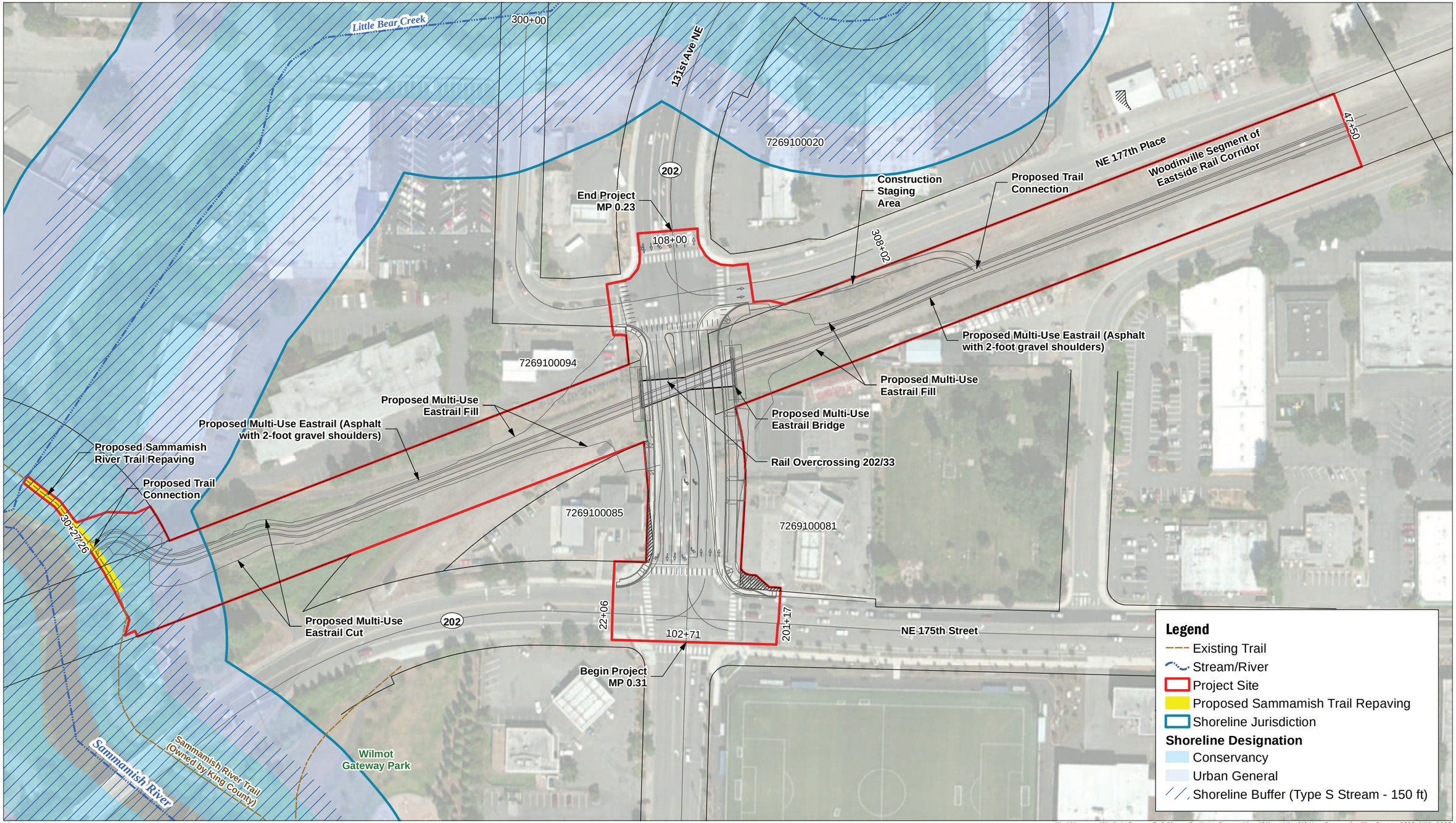
Site Location

Job No. 60683538

AECOM

SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Place
Woodinville, Washington

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World Imagery (Clarity): Source: Esri, Maxar, Earthstar Geographics, IGN, and the GIS User Community; King County, 2023; NHD, 2023.

Figure 2

Project Limits Plan

Job No. 60683538

AECOM

SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Place
Woodinville, Washington

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Legend

Exploration Designation and Approximate Location, Author, Report Date

- Boring by AECOM, 2023
- Pavement Core by AECOM, 2023
- (CR11) by Conestoga-Rovers & Associates, 2011
- (ZA06) by Zipper Zeman Associates, Inc., 2006
  (ZA05) by Zipper Zeman Associates, Inc., 2005
  (ZA01) by Zipper Zeman Associates, Inc., 2001
- (SE90) by Sweet-Edwards/EMCON, Inc., 1990
- (WS88) by State of Washington Dept. of Highways, 1988
  (WS61) by State of Washington Dept. of Highways, 1961

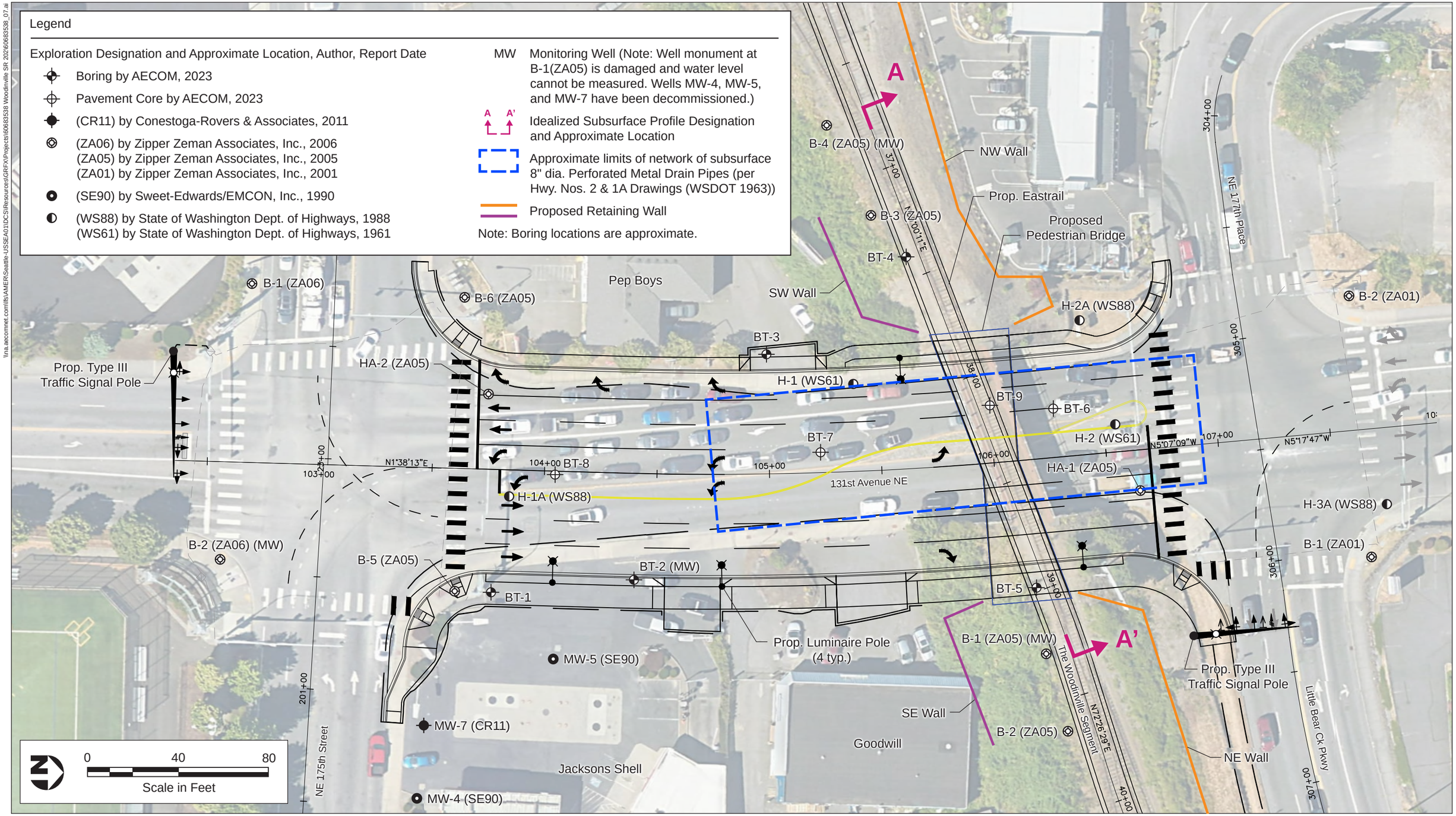
MW Monitoring Well (Note: Well monument at B-1(ZA05) is damaged and water level cannot be measured. Wells MW-4, MW-5, and MW-7 have been decommissioned.)

A A' Idealized Subsurface Profile Designation and Approximate Location

Approximate limits of network of subsurface 8" dia. Perforated Metal Drain Pipes (per Hwy. Nos. 2 & 1A Drawings (WSDOT 1963))

Proposed Retaining Wall

Note: Boring locations are approximate.



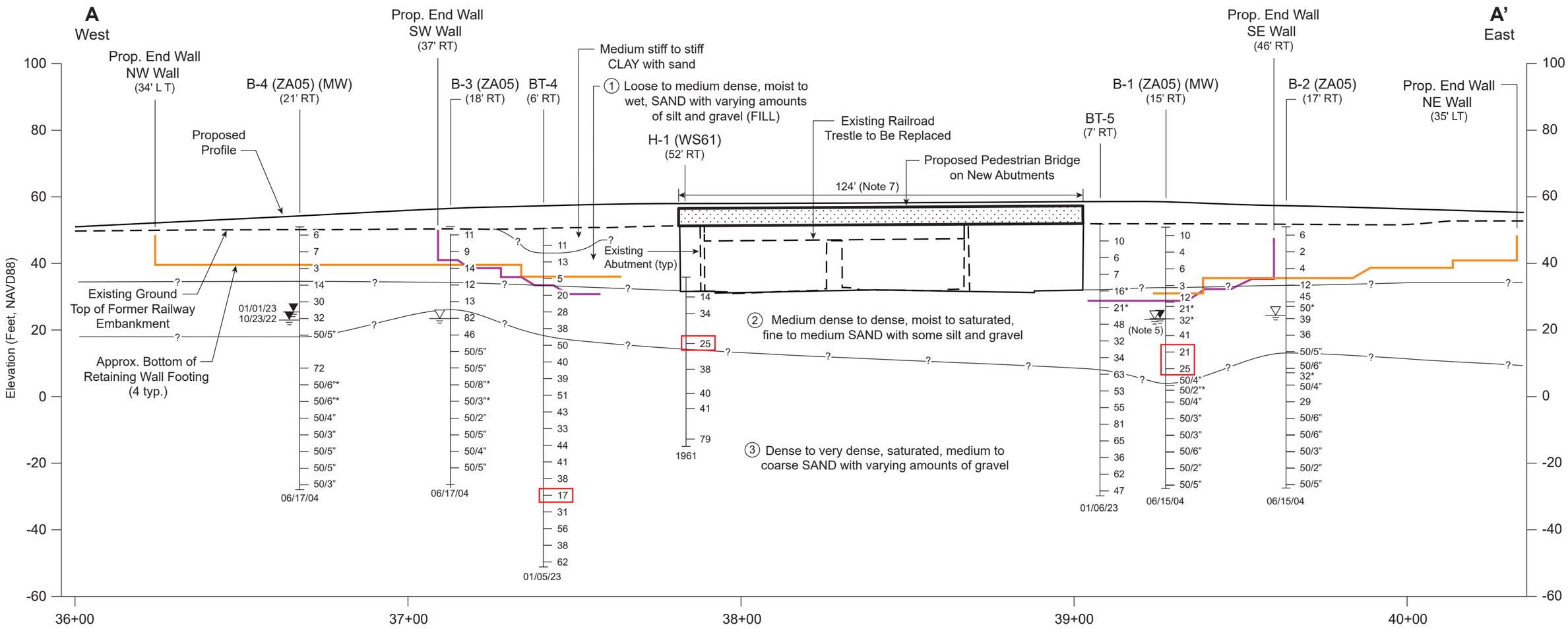
Sources: Google Earth Pro, image dated 8/23/22; Proposed roadway design per AECOM 30% design drawings dated 2/3/23.
Proposed signal pole and luminaire locations per TENW 90% design drawings dated 6/11/24.

Job No. 60683538

Figure 3
Site Plan

AECOM

SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Place
Woodinville, Washington



EASTRAIL TRAIL STATION

Legend

- BT-4 (6' RT) — Boring Designation, (MW) Indicates Monitoring Well Installed; Offset from Profile Line (LT = Left, RT = Right)
- Top of Boring
- Water Level at Time of Drilling
- Water Level Observed in Well
- ? — Approximate Geologic Contact
- 16* — Standard Penetration Test, Blows/Foot (N-Value) * See Note 6
- 25 — Potentially Liquefiable During SEE Before Ground Improvement (Note 8)
- 50/5" — Standard Penetration Test, Blows/Inches Driven
- Bottom of Boring
- 01/05/23 — Date of Completion
- (1) — Soil Stratum Number (See Report Text)

Notes

1. Subsurface profile taken along centerline of proposed EastRail Trail.
2. The subsurface conditions shown are based on interpolation of widely spaced borings and should be considered approximate.
3. Borings BT-4 and BT-5 drilled by AECOM from Jan. 4 through 6, 2023 using mud rotary drilling methods.
4. Borings B-1 (ZA05) through B-4 (ZA05) drilled by Zipper Zeman Assoc., Inc. in June 2004 using hollow-stem auger drilling methods.
5. Groundwater level in Boring/Well B-1 (ZA05) at 27 ft below ground surface at the time of drilling and in monitoring well on 6/22/04 and 7/13/04, respectively.
6. For Boring BT-5 samples (2) with an asterisk (*), SPT N-values shown on the profile reflect a factor of 0.8 applied to blow counts obtained using a Modified California Sampler (2.5" I.D., 3" O.D.). For the Zipper Zeman Assoc. borings on the profile, the asterisk denotes samples reportedly collected with a 3" I.D. Shelby tube (refer to boring logs in the respective appendix of this report), though a California Sampler may have been used.
7. 124-ft bridge length is bearing to bearing and for 16deg skew.
8. SEE = Safety Evaluation Earthquake has a seismic hazard level corresponding to an earthquake with an approximate return period of 1,000 years.

0 30 60

Approximate Horizontal and Vertical Scale in Feet

Job No. 60683538

Figure 4

Estimated Subsurface Profile A-A'

AECOM

SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Place
Woodinville, Washington

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LEGEND

Project Site

Active Faults, Known or Suspected
Inferred fault trace

1:100,000-scale Geology
Attitude Measurements
Radiometric Age Date
Fossil Age Date
Shoreline

Folds - Anticline
(A) Anticline, certain accurate
(A) Anticline, certain inferred
(A) Anticline, certain concealed

Folds - Syncline
(B) Syncline, certain accurate
(B) Syncline, certain inferred
(B) Syncline, certain concealed
Contact - Identity and existence certain, location accurate

Faults - Dip-Slip Movement
(F) Thrust, certain accurate
(F) Thrust, certain concealed

Faults - Movement Unknown
(A) Fault, certain accurate
(A) Fault, certain concealed
(A) Fault, questionable concealed

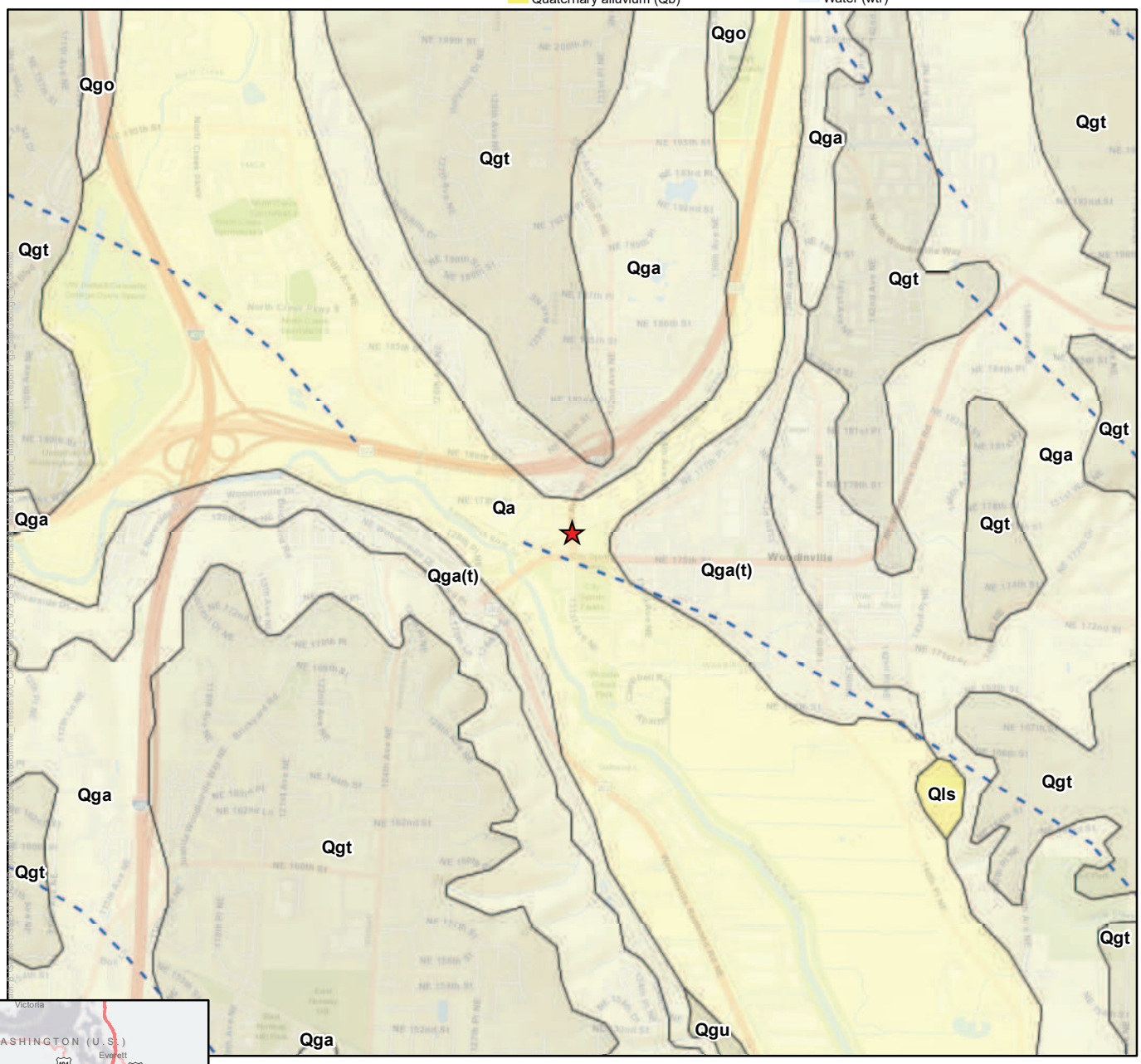
Quaternary Rocks and Deposits
Quaternary bog, marsh, swamp, or lake deposits (Qp)
Holocene artificial fill and modified land (Qf)
Quaternary alluvium (Qa)
Quaternary alluvium (Qb)
Quaternary mass-wasting deposits (Qls)
Pleistocene continental glacial drift (Qga)
Pleistocene continental glacial till (Qgt)
Pleistocene glacial and nonglacial deposits (Qgu)

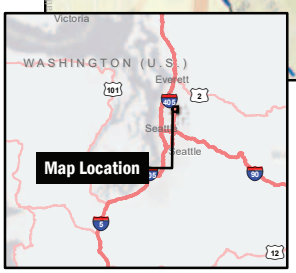
Tertiary Rocks
Tertiary sedimentary rocks and deposits (Ec(cp))

Mesozoic Rocks
Mesozoic sedimentary rocks and deposits (Kn(ne))
Mesozoic intrusive rocks (Ji(f))
Mesozoic heterogeneous metamorphic rocks (Jmt(l))

Paleozoic Rocks
Paleozoic volcanic and sedimentary rocks (PMDvs(e))

Water
Water (wtr)





Source: Bowman and Czajkowski (2019); Washington Division of Geology and Earth Resources (November 2016)

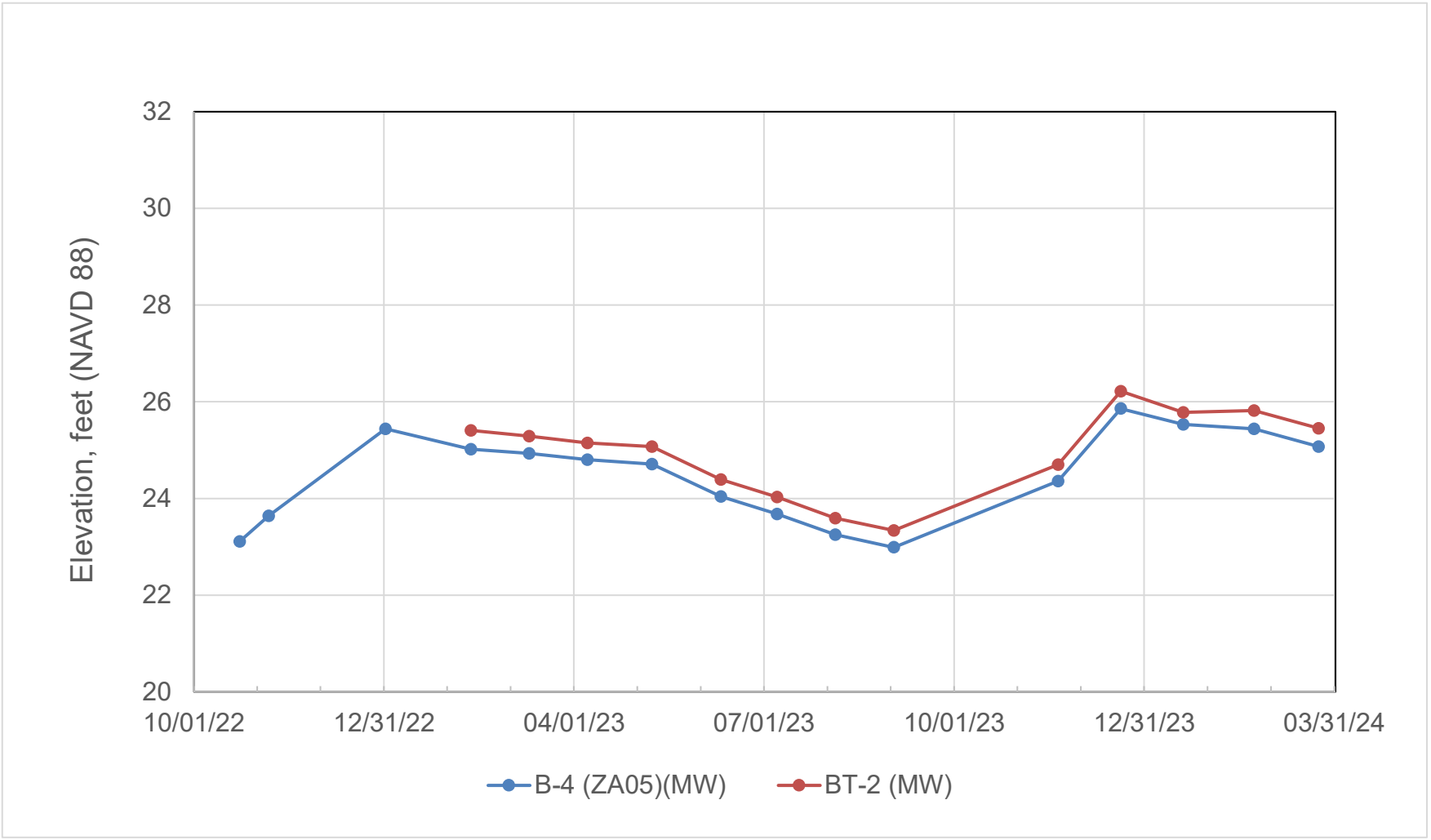
0 0.2 0.4 0.6
Scale in Miles

Figure 5
Geologic Map

Job No. 60683538

AECOM

SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Place
Woodinville, Washington



Note: Well B-4 (ZA05) installed 6/17/04; data prior to 2022 not shown. Water level in that well was approximately Elevation +24 feet in June July 2004. Well BT-2 installed Feb. 8, 2023; first stabilized water level measured Feb. 11, 2023.

Figure 6

Job No.: 60683538

Groundwater Elevations in Monitoring Wells

AECOM

SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Place
Woodinville, Washington

APPENDICES

APPENDIX A HISTORICAL EXPLORATION LOGS AND LABORATORY TESTING RESULTS

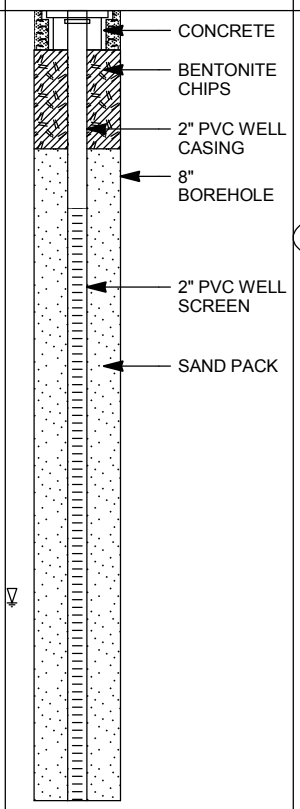
A.1 Boring and Monitoring Well Log from Conestoga-Rovers & Associates (2011)

STRATIGRAPHIC AND INSTRUMENTATION LOG (OVERBURDEN)

PROJECT NAME: 13106 WOOD
PROJECT NUMBER: 060566
CLIENT: SHELL OIL PRODUCTS US
LOCATION: 13106 NE 175TH STREET WOODINVILLE, WA

HOLE DESIGNATION: MW-7
DATE COMPLETED: September 21, 2011
DRILLING METHOD: HOLLOW STEM AUGER
FIELD PERSONNEL: J. SONG

DEPTH ft BGS	STRATIGRAPHIC DESCRIPTION & REMARKS	DEPTH ft BGS
	ASPHALT	0.50
2	SW-SAND, with up to 1" subrounded gravel, fine to coarse grained, well graded, medium dense, brown, dry, no odor	
4		
6		
8		
10		10.00
12	SW-SAND, 5% silt, fine to coarse grained, well graded, medium dense, brown, dry, no odor	
14		
16		15.00
18	SW-SAND, 10% silt, 10% gravel up to 0.5", fine to coarse grained, well graded, dense, moist, no odor	
20		20.00
22	SW-SAND, 5% silt, trace gravel, fine to coarse grained, well graded, medium dense, wet, no odor	21.50
24	END OF BOREHOLE @ 21.5ft BGS	
26		
28		
30		
32		
34		



WELL DETAILS
Screened interval:
5.00 to 20.00ft BGS
Length: 15ft
Diameter: 2in
Slot Size: 0.010
Material: PVC
Seal:
1.00 to 3.50ft BGS
Material: BENTONITE CHIPS
Sand Pack:
3.50 to 20.00ft BGS
Material: SAND

SAMPLE NUMBER	INTERVAL	REC (%)	BLOW COUNTS	PID (ppm)
MW-7-5		50	14 16 18	30.0
SS2		100	6 10 14	25.6
SS3		100	18 12 30	15.1
SS4		100	12 18 22	19.5

NOTES: MEASURING POINT ELEVATIONS MAY CHANGE; REFER TO CURRENT ELEVATION TABLE
WATER FOUND ▽
CHEMICAL ANALYSIS ⬭

A.2 Boring Logs from Zipper Zeman Associates (2006)

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS

Our field exploration for this project included 8 borings advanced on July 20 and 21, 2006. In addition, three falling head infiltration tests were completed on July 28, 2006. The approximate exploration and infiltration test locations are shown on the Site and Exploration Plan, Figure 1. Figure 1 also shows the locations of two previous borings (ZZA B-5 and ZZA B-6) completed in 2004 at the NE 175th Street and 131st Avenue NE intersection.

Exploration locations were determined by measuring distances from existing site features with a fiberglass tape relative to a site plan provided by Perteet Engineering. As such, the exploration locations should be considered accurate to the degree implied by the measurement method. The following sections describe our procedures associated with the exploration. Descriptive logs of the explorations are presented in this appendix.

Soil Boring Procedures

Our exploratory borings were advanced with a hollow stem auger, using a trailer - mounted drill rig operated by an independent drilling company working under subcontract to our firm. A geotechnical engineer from our firm continuously observed the borings, logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further visual classification and testing. After each boring was completed, the borehole was backfilled with soil cuttings, and the surface was patched with either bentonite clay or concrete.

Throughout the drilling operation, soil samples were obtained at 2.5- to 5-foot depth intervals by means of the Standard Penetration Test (ASTM: D-1586). This testing and sampling procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows are struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed

between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring, as well as any laboratory tests performed on these soil samples. If groundwater was encountered in a borehole, the approximate groundwater depth, and date of observation, are depicted on the log. Groundwater depth estimates are typically based on the moisture content of soil samples, the wetted portion of the drilling rods, the water level measured in the borehole after the auger has been extracted, or through the use of an observation well.

Groundwater observation wells were installed at three of the boring locations (Borings B-2, B-5 and B-6). Each well consisted of a length of slotted 2-inch PVC pipe placed in the bottom of the borehole. A blank PVC riser extended from the lower slotted section to the ground surface. Washed silica sand was utilized to backfill the annular space between the slotted interval and the borehole to allow entry of water into the well, while a mixture of bentonite clay and soil cuttings was used to backfill around the blank riser. A concrete surface seal and locking metal monument cover were placed at the surface. The groundwater level measured within each observation well subsequent to completion of drilling is indicated by a triangular symbol on the logs, along with the date of measurement. This information is also discussed in the text.

The boring logs presented in this appendix are based upon the drilling action, observation of the samples secured, laboratory test results, and field logs. The various types of soils are indicated as well as the depth where the soils or characteristics of the soils changed. It should be noted that these changes may have been gradual, and if the changes occurred between sample intervals, they were inferred.

Infiltration Test Procedures

Three infiltration tests were performed using the EPA falling head percolation test procedure (Design Manual – Onsite Wastewater Treatment and Disposal Systems, EPA, 1980) at the locations shown on Figure 1. Infiltration test depths of 3.5 to 3.7 feet below existing grade were selected based on preliminary infiltration depths. The bottom of the infiltration test borings were cleaned and the pipes were seated into the undisturbed soil to the appropriate test depth. Infiltration rates were measured in accordance with the methods outlined in the falling head test procedure EPA, 1980. The results of the infiltration tests are presented in the report text.

PROJECT: NE 175th St. & 131st Ave. NE Improvements **JOB NO.:** 81051721 **BORING:** B-1 **PAGE 1 OF 2**

Location: Woodinville, WA **Approximate Surface Elevation:**

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0 10 20 30 40 50)	N-values	Testing
0	Loose to medium dense, damp to moist, brown-gray, silty SAND with some gravel (Fill?)		S-1		▲	14	
5	Very loose, moist, brown-gray, silty SAND with some gravel, organics and charcoal (Relic Topsoil?)		S-2		▲	3	
	Medium dense, moist, brown-gray, gravelly SAND with some silt		S-3		▲	17	
10	Grades to fine to medium SAND with some coarse sand, silt and gravel		S-4		▲	11	
	Medium dense, wet, brown-gray, fine to coarse SAND with some gravel and trace silt						
15	Becomes saturated		S-5	▼ ATD	▲	20	
20			S-6		▲	27	
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

ZZA **Zipper Zeman Associates, Inc.**
Geotechnical and Environmental Consulting

BORING LOG | **Figure A-1**
Date Drilled: 7/20/2006 | **Logged By: CRT**

PROJECT: NE 175th St. & 131st Ave. NE Improvements **JOB NO.:** 81051721 **BORING:** B-1 **PAGE 2 OF 2**

Location: Woodinville, WA **Approximate Surface Elevation:**

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
25	Medium dense, saturated, brown-gray, fine to coarse SAND with some gravel and trace silt	I	S-7		▲ 30	30	
	Boring completed at 26.5 feet on 07/20/06. Groundwater observed at 15 feet at time of drilling.						
30							
35							
40							
45							
50							

Explanation

- I — 2-inch O.D. split spoon sample
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

ZZA **Zipper Zeman Associates, Inc.**
Geotechnical and Environmental Consulting

BORING LOG	Figure A-1
Date Drilled: 7/20/2006	Logged By: CRT

PROJECT: NE 175th St.& 131st Ave. NE Improvements **JOB NO.:** 81051721 **BORING:** B-2 **PAGE 1 OF 2**

Location: Woodinville, WA **Approximate Surface Elevation:**

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0 10 20 30 40 50)	N-values	Testing
0	2" Asphalt over Medium dense, damp, brown-gray, silty, gravelly SAND and SAND with some gravel (Fill)						
	3" loose, damp, dark brown, silty SAND (Relic topsoil)		S-1		▲	16	
5	Medium dense, moist, brown-gray, fine to medium SAND with some coarse sand and trace gravel and silt		S-2		▲	11	
			S-3		▲	15	
10	Medium dense, moist, brown-gray, fine to medium SAND with some coarse sand and trace silt and gravel		S-4		▲	12	
	Becomes saturated		▼ 8/3/06				
15			S-5	ATD	▲	15	
20	Interbedded sandy silt (1")		S-6		▲	24	
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

ZZA **Zipper Zeman Associates, Inc.**
Geotechnical and Environmental Consulting

BORING LOG	Figure A-2
Date Drilled: 7/20/2006	Logged By: CRT

PROJECT: NE 175th St.& 131st Ave. NE Improvements	JOB NO.: 81051721	BORING: B-2	PAGE 2 OF 2
Location: Woodinville, WA	Approximate Surface Elevation:		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
25	Medium dense, saturated, brown-gray, fine to medium SAND with some coarse sand and trace silt and gravel	I	S-7		▲ at 28	28	
	Boring completed at 26.5 feet on 07/20/06. Groundwater observed at 15 feet at time of drilling. Groundwater measured at 12.8 feet on 8/3/06.						
30							
35							
40							
45							
50							

Explanation

- I — 2-inch O.D. split spoon sample
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-2
	Date Drilled: 7/20/2006	Logged By: CRT

A.3 Exploration Logs and Laboratory Testing Results from Zipper Zeman Associates (2005)

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS

Our field exploration for this project included 6 borings advanced between June 15 and June 17, 2004, and 3 hand auger test holes completed on June 30, 2004. Approximate exploration locations are shown on the Site and Exploration Plan, Figure 1. Exploration locations were determined by measuring distances from existing site features with a fiberglass tape relative to a topographic map of the site prepared by Perteet Engineering dated May 28, 2004. As such, the exploration locations should be considered accurate to the degree implied by the measurement method. The approximate ground surface elevation at each exploration location was determined by interpolating the information provided on the topographic plan. The following sections describe our procedures associated with the exploration. Descriptive logs of the explorations are presented in this appendix.

Soil Boring Procedures

Our exploratory borings were advanced with a hollow stem auger, using a truck-mounted drill rig operated by an independent drilling company working under subcontract to our firm. A geotechnical engineer from our firm continuously observed the borings, logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further visual classification and testing. After each boring was completed, the borehole was backfilled with soil cuttings, and the surface was patched with either bentonite clay or concrete.

Throughout the drilling operation, soil samples were obtained at 2.5- to 5-foot depth intervals by means of the Standard Penetration Test (ASTM: D-1586). This testing and sampling procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows are struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth

of each soil sample obtained from the boring, as well as any laboratory tests performed on these soil samples. If groundwater was encountered in a borehole, the approximate groundwater depth, and date of observation, are depicted on the log. Groundwater depth estimates are typically based on the moisture content of soil samples, the wetted portion of the drilling rods, the water level measured in the borehole after the auger has been extracted, or through the use of an observation well.

Groundwater observation wells were installed at two of the boring locations (Borings B-1 and B-4). Each well consisted of a length of slotted 2-inch PVC pipe placed in the bottom of the borehole. A blank PVC riser extended from the lower slotted section to the ground surface. Washed silica sand was utilized to backfill the annular space between the slotted interval and the borehole to allow entry of water into the well, while a mixture of bentonite clay and soil cuttings was used to backfill around the blank riser. A concrete surface seal and locking metal monument cover were placed at the surface. The groundwater level measured within each observation well subsequent to completion of drilling is indicated by a triangular symbol on the logs, along with the date of measurement. This information is also discussed in the text.

The boring logs presented in this appendix are based upon the drilling action, observation of the samples secured, laboratory test results, and field logs. The various types of soils are indicated as well as the depth where the soils or characteristics of the soils changed. It should be noted that these changes may have been gradual, and if the changes occurred between sample intervals, they were inferred.

Hand Auger Boring Procedures

Three hand auger borings (HA-1 through HA-3) were advanced at the approximate locations shown on the Site and Exploration Plan, Figure 1. A geotechnical engineer from our firm advanced a 3.5-inch diameter auger by hand, continuously observing the soil cuttings as they were retrieved. Representative portions of the soils retrieved were placed in moisture tight containers and returned to our laboratory for further visual classification and testing.

Granular soil density and cohesive soil consistency were determined through the use of the Dynamic Cone Penetrometer in general accordance with ASTM Special Technical Publication No. 399. This procedure entails driving a steel rod equipped with a specifically sized conical tip into the ground with a 15-pound drop hammer free falling 20 inches. After seating the penetrometer into the soil a depth of 2 inches, the number of blows required to drive the penetrometer over three successive 1.75-inch intervals is recorded and averaged. This numerical value, "N_C", has been shown to correlate with the Standard Penetration Test (ASTM: D-1586) "N" value. The N_C values for specific depths are indicated on the logs in this appendix.

Pavement Coring Procedures

Three pavement section cores were completed during the field exploration. A local contractor working under subcontract to ZZA completed the pavement cores. The cores were 14 inches in diameter, and were advanced with an electric coring machine. Once the cores were removed, a ZZA representative measured the pavement thickness with a hand-held tape, and measured the depth of granular pavement section aggregates by excavating below the core subgrade with a hand shovel. We then advanced a hand auger (HA-1 through HA-3) down to the depths indicated on the attached logs. Upon completion of section measurement and classification, we backfilled each core location with concrete. Details regarding the pavement cores are presented in the report text.

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-1

Location: Woodinville, WA **Approximate Elevation:** 51 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
0	Medium dense, black, moist, sandy GRAVEL, with some silt (FILL)						
	Medium dense, brown, wet, silty SAND, some gravel (FILL)						
			S-1		▲ ●	10	GS
5							
	Grades to loose		S-2		▲	4	
10							
	Loose, moist, brown, medium SAND, with some silt and gravel						
			S-3		▲ ●	6	
15							
	grades to medium SAND with trace to some gravel, trace silt		S-4		▲	3	
20							
	Medium dense, saturated, brown, medium SAND with some gravel and trace silt		S-5		● ▲	12	
			S-6		▲	21	
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

ZZA Zipper Zeman Associates, Inc.
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BORING LOG	Figure A-1
Date Drilled: 6/15/04	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-1 **PAGE 2 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 51 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
25	Medium dense to dense, saturated, brown, medium SAND with some gravel and trace silt			ATD; 6-22-04 and 7-13-04			
			S-7		▲	32	
30			S-8		● ▲	41	
35			S-9		▲	21	
40			S-10		● ▲	25	
45	Very dense, saturated, brown, coarse to medium SAND, with some gravel to gravelly, trace silt		S-11		▲	50/4	
50			S-12		▲	50/2	

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

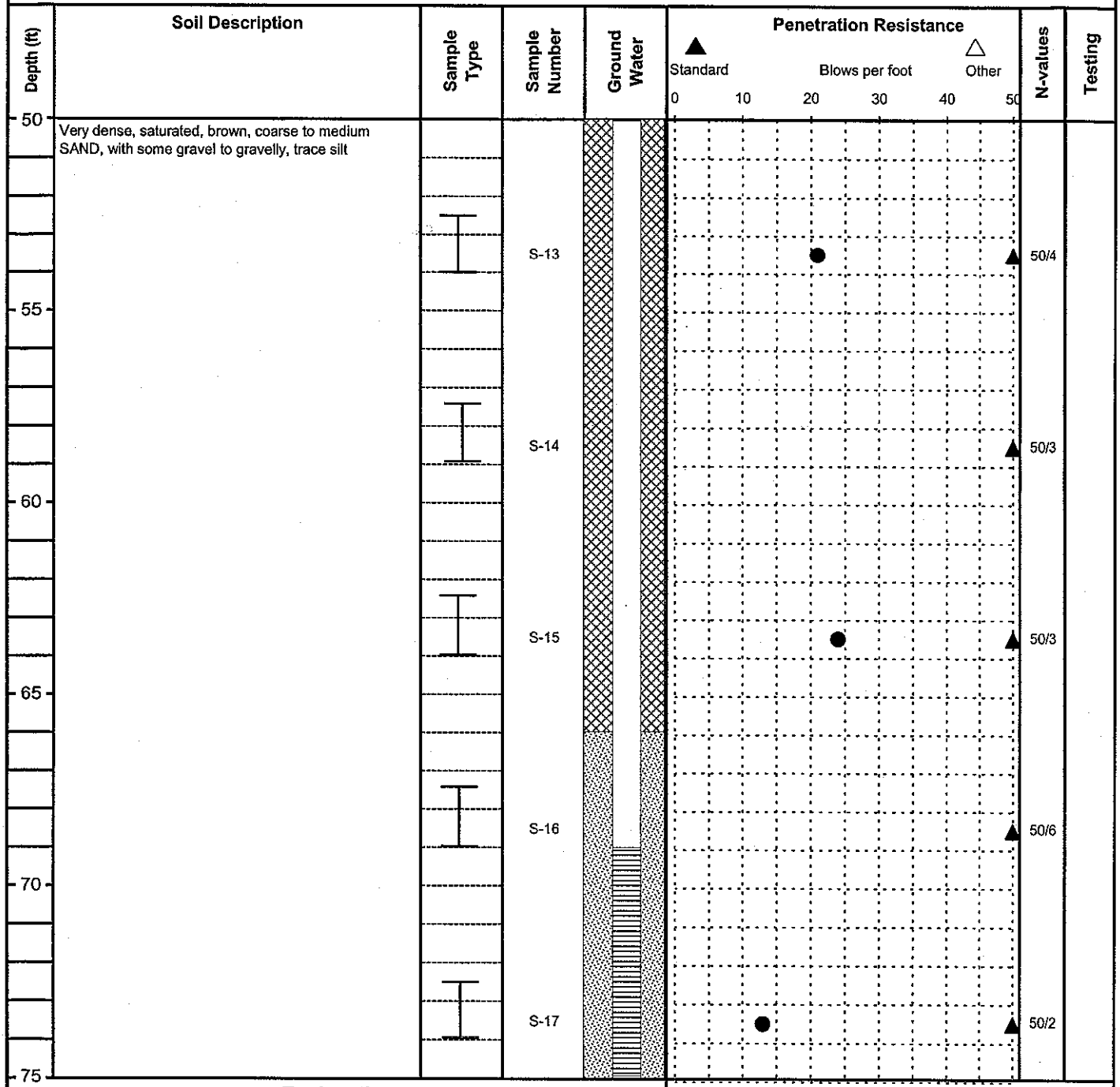
Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

PROJECT: BNRP - SR 202/SR 522 I/C Improvements	JOB NO.: J-1721	BORING: B-1	PAGE 3 OF 4
Location: Woodinville, WA	Approximate Elevation: 51 Feet		



Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
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Consol. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-1
	Date Drilled: 6/15/2004	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-1 **PAGE 4 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 51 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0 10 20 30 40 50)	N-values	Testing
75	Very dense, saturated, brown, coarse to medium SAND, with some gravel to gravelly, trace silt						
			S-18		▲ 50/5		
	Boring completed at 78.5 feet on 6/15/04 Groundwater observed at 27 feet at time of drilling.						
80							
85							
90							
95							
100							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
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Zipper Zeman Associates, Inc.
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BORING LOG	Figure A-1
Date Drilled: 6/15/2004	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-2 **PAGE 1 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 51 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
0	Medium dense, moist, black, sandy GRAVEL, with some silt (FILL)						
	Loose, wet, gray, gravelly, silty SAND (FILL)		S-1		▲	6	
5							
	Loose, moist, brown, medium SAND, with some gravel and trace silt (probable FILL)		S-2		▲ ●	2	
10							
			S-3		▲	4	
15	Medium dense, moist, brown, fine to medium SAND, with some gravel and trace silt						
	grades to medium SAND with trace to some gravel, trace silt		S-4		● ▲	12	
20							
	grades to dense and from some gravel to gravelly, with interbedded sandy gravel		S-5		● △ ▲	45	GS
25			S-6		▲	50	

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
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Consol. = Consolidation Test

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BORING LOG | **Figure A-2**

Date Drilled: 6/15/04 | **Logged By: EG**

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-2 **PAGE 2 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 51 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
25	Dense, saturated, brown, gravelly, fine to medium SAND with trace silt			▼ ATD			
			S-7		● ▲	39	
30			S-8		▲	36	
35			S-9		● ▲	50/5	
	Very dense, saturated, brown, gravelly, coarse to medium SAND with trace silt		S-10		▲	50/6	
40			S-11		▲	32	
45			S-12		● ▲	50/4	
50							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

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Zipper Zeman Associates, Inc.
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BORING LOG	Figure A-2
Date Drilled: 6/15/2004	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-2 **PAGE 3 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 51 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
50	Very dense, saturated, brown, coarse to medium SAND with some gravel to gravelly, trace silt						
			S-13		▲	29	
55							
			S-14		● ▲	50/6	
60							
			S-15		▲	50/6	
65							
			S-16		● ▲	50/3	GS
70							
			S-17		▲	50/2	
75							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

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BORING LOG	Figure A-2
Date Drilled: 6/15/04	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-2 **PAGE 4 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 51 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
75	Very dense, saturated, brown, coarse to medium SAND, with some gravel to gravelly, trace silt						
		I	S-18		● ▲	50/5	
	Boring completed at 78.5 feet on 6/15/04 Groundwater observed at 26.5 feet at time of drilling.						
80							
85							
90							
95							
100							

Explanation

- I — 2-inch O.D. split spoon sample
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

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BORING LOG | Figure A-2

Date Drilled: 6/15/2004 | Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-3 **PAGE 1 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 49 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
0	Medium dense, moist, black, sandy GRAVEL, with some silt (FILL)						
	Medium dense, moist, brown, SAND with some silt and gravel (FILL)	I	S-1		▲	11	
5		I	S-2		▲●	9	
10		⊗	S-3		▲	14	
15	Medium dense, wet, brown, medium SAND, with some silt and gravel	I	S-4		▲●	12	
20	grades to trace silt	I	S-5		▲	13	
25							

Explanation

- I — 2-inch O.D. split spoon sample
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

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BORING LOG | **Figure A-3**
Date Drilled: 6/17/04 | **Logged By: EG**

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-3 **PAGE 2 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 49 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
25	Very dense, brown, saturated, coarse to medium SAND, with some gravel and trace silt			▼ ATD			
			S-6			82	
30			S-7			46	
35			S-8			50/5	
40			S-9			50/5	
45			S-10			50/8	
50							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

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BORING LOG	Figure A-3
Date Drilled: 6/17/04	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-3 **PAGE 3 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 49 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
50	Very dense, saturated, brown, coarse to medium SAND with some gravel to gravelly, trace silt						
			S-11		▲ 50/3		
55							
			S-12		● ▲ 50/2		
60							
			S-13		▲ 50/5		
65							
			S-14		● ▲ 50/4		
70							
			S-15		▲ 50/5		
75							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

Zipper Zeman Associates, Inc.
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BORING LOG | **Figure A-3**
Date Drilled: 6/17/04 | **Logged By: EG**

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-3 **PAGE 4 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 49 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
75	Very dense, saturated, brown, coarse to medium SAND with trace to some gravel and trace silt						
	Boring completed at 77.5 feet on 6/17/04 Groundwater observed at 27 feet at time of drilling.						
80							
85							
90							
95							
100							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
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Consol. = Consolidation Test

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BORING LOG | **Figure A-3**

Date Drilled: 6/17/04 | **Logged By: EG**

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-4 **PAGE 1 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 50 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
0	Medium dense, moist, black, sandy GRAVEL, with some silt (FILL)						
	Loose, wet, brown, silty SAND, with some gravel (FILL)						
			S-1		▲ ●	6	
5							
			S-2		▲	7	
10							
			S-3		▲ ●	3	
15	Medium dense, wet, brown, medium SAND, with some silt and gravel						
			S-4		▲	14	
20							
	grades to trace silt		S-5		● ▲	30	
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

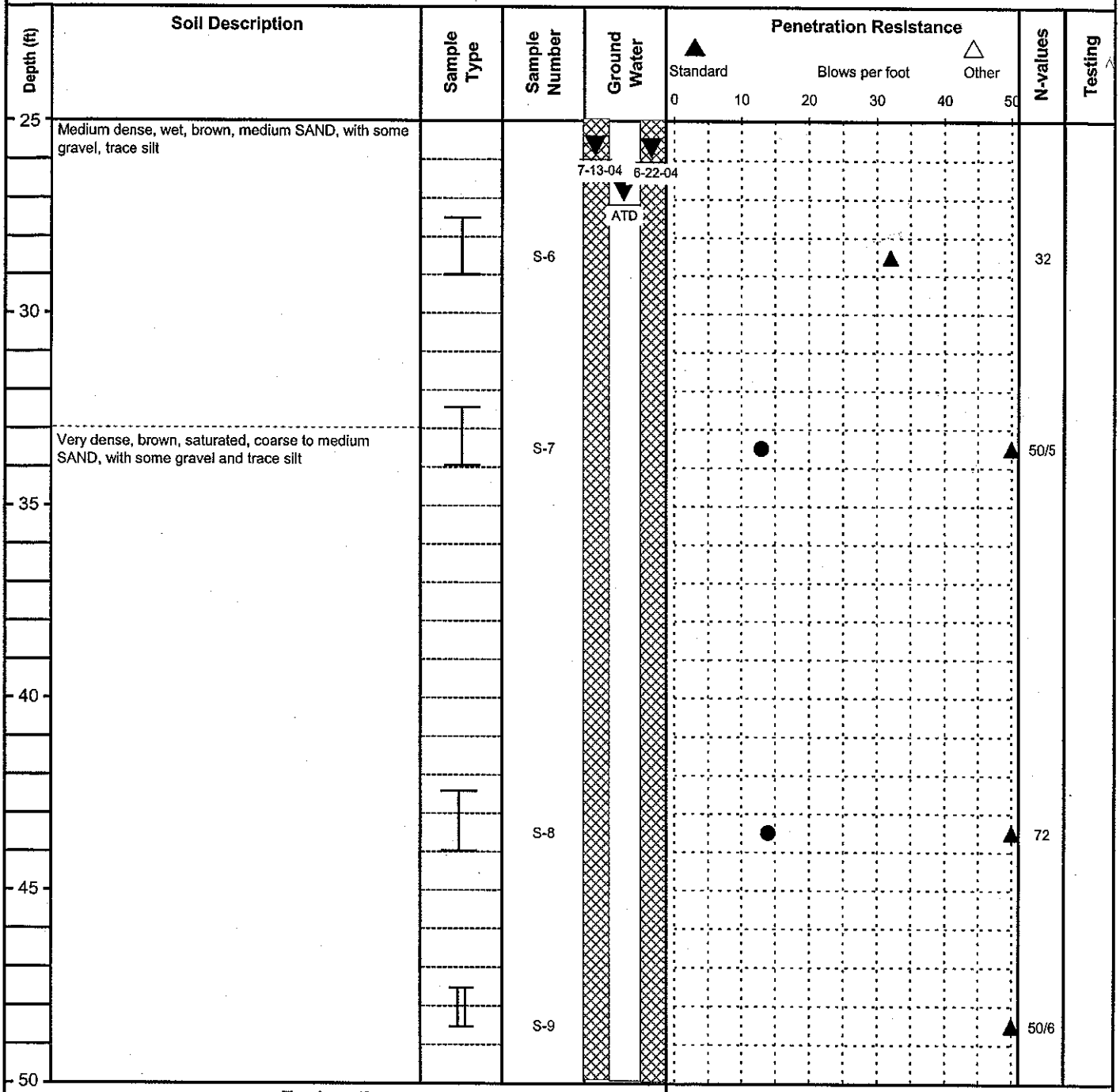
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BORING LOG | **Figure A-4**

Date Drilled: 6/17/04 | **Logged By: EG**

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-4 **PAGE 2 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 50 Feet



Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-4
	Date Drilled: 6/17/2004	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-4

Location: Woodinville, WA **Approximate Elevation:** 50 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
50	Very dense, saturated, brown, coarse to medium SAND with some gravel to gravelly, trace silt						
			S-10			50/6	
55			S-11			50/4	
60			S-12			50/3	
65			S-13			50/5	
70			S-14			50/5	
75							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-4 **PAGE 4 OF 4**

Location: Woodinville, WA **Approximate Elevation:** 50 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0 10 20 30 40 50)	N-values	Testing
75	Very dense, saturated, brown, coarse to medium SAND with trace to some gravel and trace silt		S-15		● ▲	50/3	
	Boring completed at 79 feet on 6/17/04 Groundwater observed at 27 feet at time of drilling.						
80							
85							
90							
95							
100							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

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Att. = Atterberg Limits
Consol. = Consolidation Test

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Geotechnical and Environmental Consulting

BORING LOG | **Figure A-4**

Date Drilled: 6/17/2004 | **Logged By: EG**

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-5 **PAGE 1 OF 1**

Location: Woodinville, WA **Approximate Elevation:** 36 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
0	4" of AC over 6" gravel base						
	Medium dense, moist, brown, gravelly, fine to medium SAND, with some silt						
			S-1		● ▲	10	GS
5	grades to trace silt						
			S-2		▲	13	
10				ATD			
	Medium dense to very dense, saturated, brown, medium SAND, with some gravel and trace silt						
			S-3		▲ ●	15	
15							
			S-4		▲	71	
20							
			S-5		● ▲	50/6	
	Boring completed at 24 feet on 6/16/04 Groundwater observed at 11 feet at time of drilling.						
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG	Figure A-5
Date Drilled: 6/16/04	Logged By: EG

PROJECT: BNRP - SR 202/SR 522 I/C Improvements **JOB NO.:** J-1721 **BORING:** B-6 **PAGE 1 OF 1**

Location: Woodinville, WA **Approximate Elevation:** 36 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50)	N-values	Testing
0	4" of AC over 6" gravel base						
	Medium dense, moist, brown, gravelly, fine to medium SAND, with some silt		S-1		▲	11	
5	grades to trace silt						
	grades to loose		S-2		▲●	7	
10				▼ ATD			
	Medium dense, saturated, brown, medium sand, with some gravel and trace silt		S-3		▲	16	
15							
			S-4		● ▲	24	GS
20							
	grades to dense		S-5		▲	37	
	Boring completed at 24 feet on 6/16/04 Groundwater observed at 11 feet ATD						
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG	Figure A-6
Date Drilled: 6/16/04	Logged By: EG

PROJECT:	BNRP-SR 202/SR 522 I/C Improvements	JOB NO. J-1721	BORING: HB-1
Location:	Woodinville, Washington	Approximate Elevation:	33 Feet

Depth (ft)	Material Description	N_c	Moisture	Sample	Testing
	Two layers of 2" asphalt overlay				
	7" asphalt concrete				
1					
	ATB consisting of sandy gravel			S-1	
2	7" concrete slab (at 19 inches)				
	Crushed rock and/or sandy gravel	50/.5"			
	Boring completed at 27 inches on 6/30/04 No ground water observed at time of excavation				
3					
4					
5					
6					
7					

Explanation

N_c Relative Penetration Resistance
(ASTM STP #399)

Zipper Zeman Associates, Inc. Geotechnical & Environmental Consulting	HAND-AUGER BORING LOG Date Logged: 6/30/04	FIGURE A-1 Logged by: EG

PROJECT:	BMRP-SR 202/SR 522 I/C Improvements	JOB NO. J-1721	BORING: HB-2
Location:	Woodinville, Washington	Approximate Elevation:	35 Feet

Depth (ft)	Material Description	N_c	Moisture	Sample	Testing
	6 1/2" asphalt concrete				
1	Medium dense, moist, brown, gravelly, fine to medium SAND			S-1	CBR
		20			
2					
3		34			
	Boring completed at 34 inches on 6/30/04 No ground water observed at time of excavation				
4					
5					
6					
7					

Explanation

N_c Relative Penetration Resistance
(ASTM STP #399)

Zipper Zeman Associates, Inc. Geotechnical & Environmental Consulting	HAND-AUGER BORING LOG Date Logged: 6/30/04	FIGURE A-2 Logged by: EG

PROJECT:	BMRP-SR 202/SR 522 I/C Improvements	JOB NO. J-1721	BORING: HB-3
Location:	Woodinville, Washington	Approximate Elevation:	37 Feet

Depth (ft)	Material Description	N_c	Moisture	Sample	Testing
	7" asphalt concrete				
1	Medium dense to dense, moist, brown, gravelly, fine to medium SAND			S-1	CBR
2		18			
3	Loose, wet, brown, silty, fine to medium SAND with some gravel	6			
4	Boring completed at 42 inches on 6/30/04 No ground water observed at time of excavation				
5					
6					
7					

Explanation

N_c Relative Penetration Resistance
(ASTM STP #399)

Zipper Zeman Associates, Inc. Geotechnical & Environmental Consulting	HAND-AUGER BORING LOG Date Logged: 6/30/04	FIGURE A-3 Logged by: EG

APPENDIX B
LABORATORY TESTING PROCEDURES AND RESULTS

A series of laboratory tests were performed during the course of this study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of tests performed are given below.

Visual Classification

Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with the Unified Soil Classification system. Visual soil classification includes evaluation of color, relative moisture content, soil type based upon grain size, and accessory soil types included in the sample. Soil classifications are presented on the exploration logs in Appendix A.

Moisture Content Determinations

Moisture content determinations were performed on representative samples obtained from the exploration in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM: D-2216. The results are shown on the exploration logs in Appendix A.

Grain Size Analyses

A grain size analysis indicates the range in diameter of soil particles included in a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM: D-2487. The results of the grain size determinations for the samples were used in classification of the soils, and are presented in this appendix.

California Bearing Ratio Test

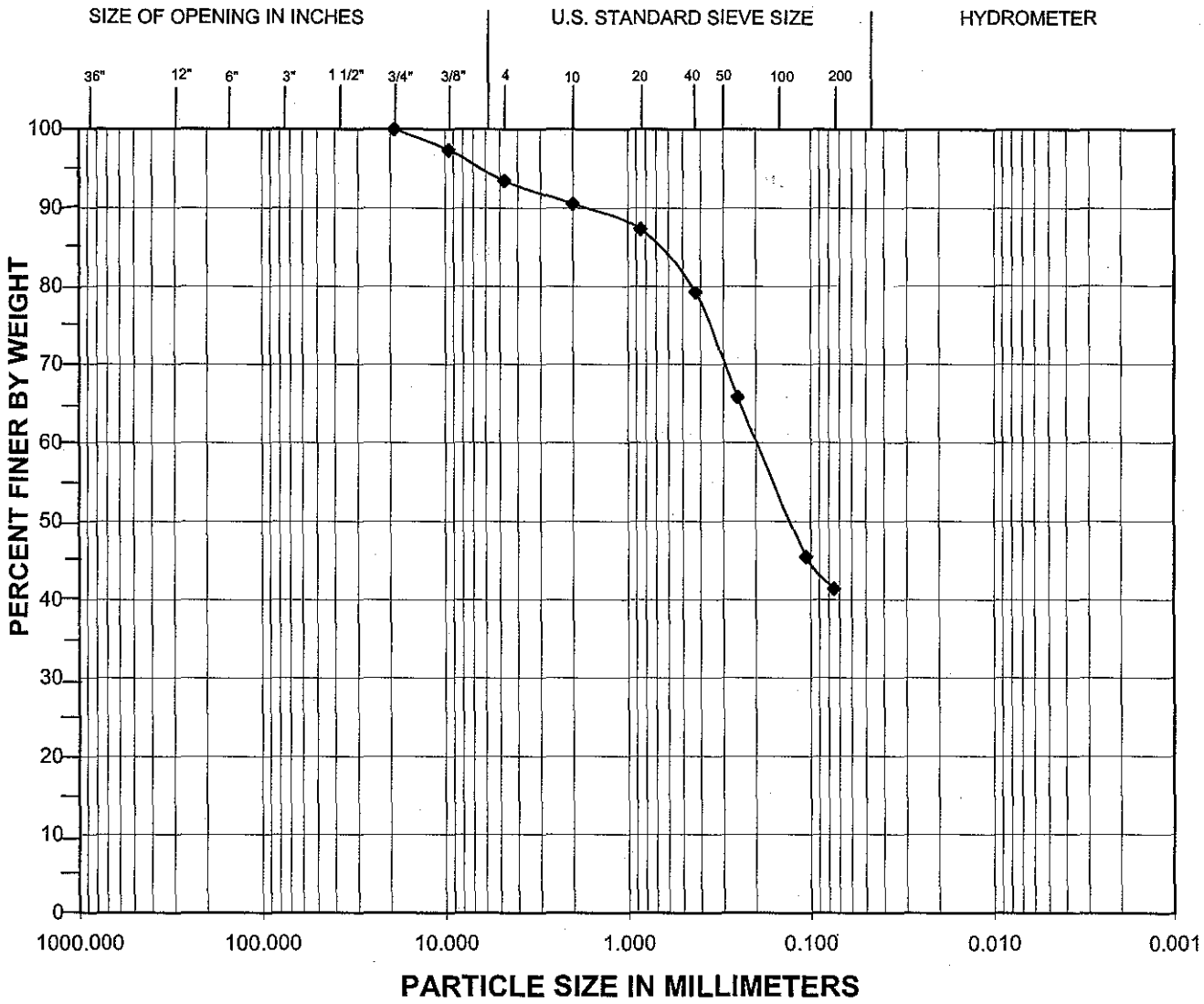
A California Bearing Ratio (CBR) test was performed on a representative sample in general accordance with ASTM: D-1883-73 to provide an evaluation of the relative quality and support characteristics of subgrade soils. Representative portions from the sample were compacted in a mold, generally in accordance with ASTM: D-1557, in order to obtain a moisture-density relationship curve. Following compaction, a 15 pound surcharge was applied to each sample which was then immersed in water and allowed to soak for a period of 72 to 96 hours, during which time it was monitored for swell. At the end of this period, the sample was removed, drained, and a vertical load applied to the surcharged soil with a penetration piston at a constant rate of strain. Measurements of

the applied vertical load were obtained as selected penetration depths. CBR test results and moisture-density relationships plotted in terms of percent water content versus percent corrected CBR and dry density, respectively, are presented in this appendix.

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM



BOULDERS	COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		GRAVEL		SAND			FINE GRAINED	

Comments:

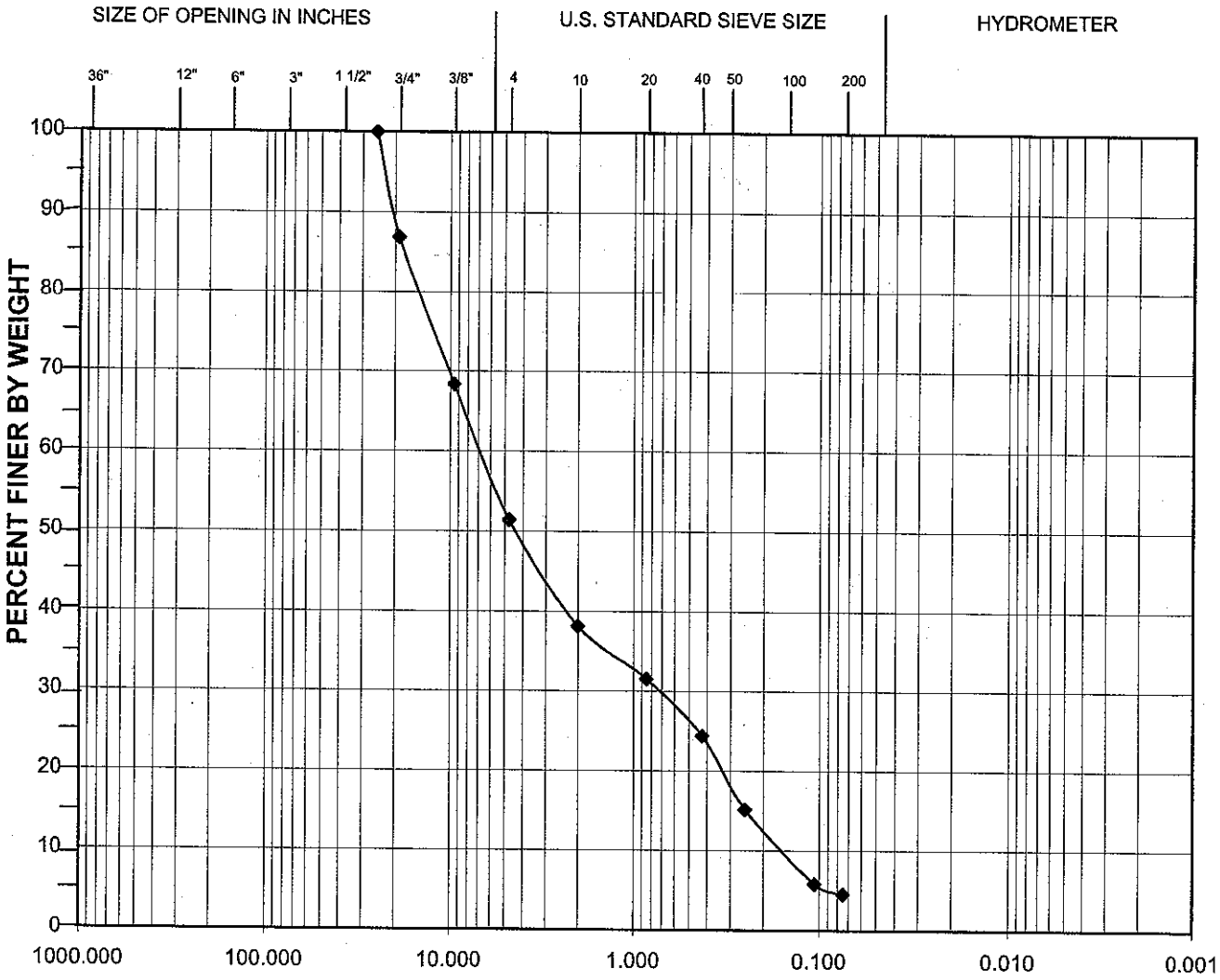
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-1	2.5'	16	41.4	silty SAND with some gravel

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1721 DATE OF TESTING: 6/17/2004	PROJECT NAME: Woodinville

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

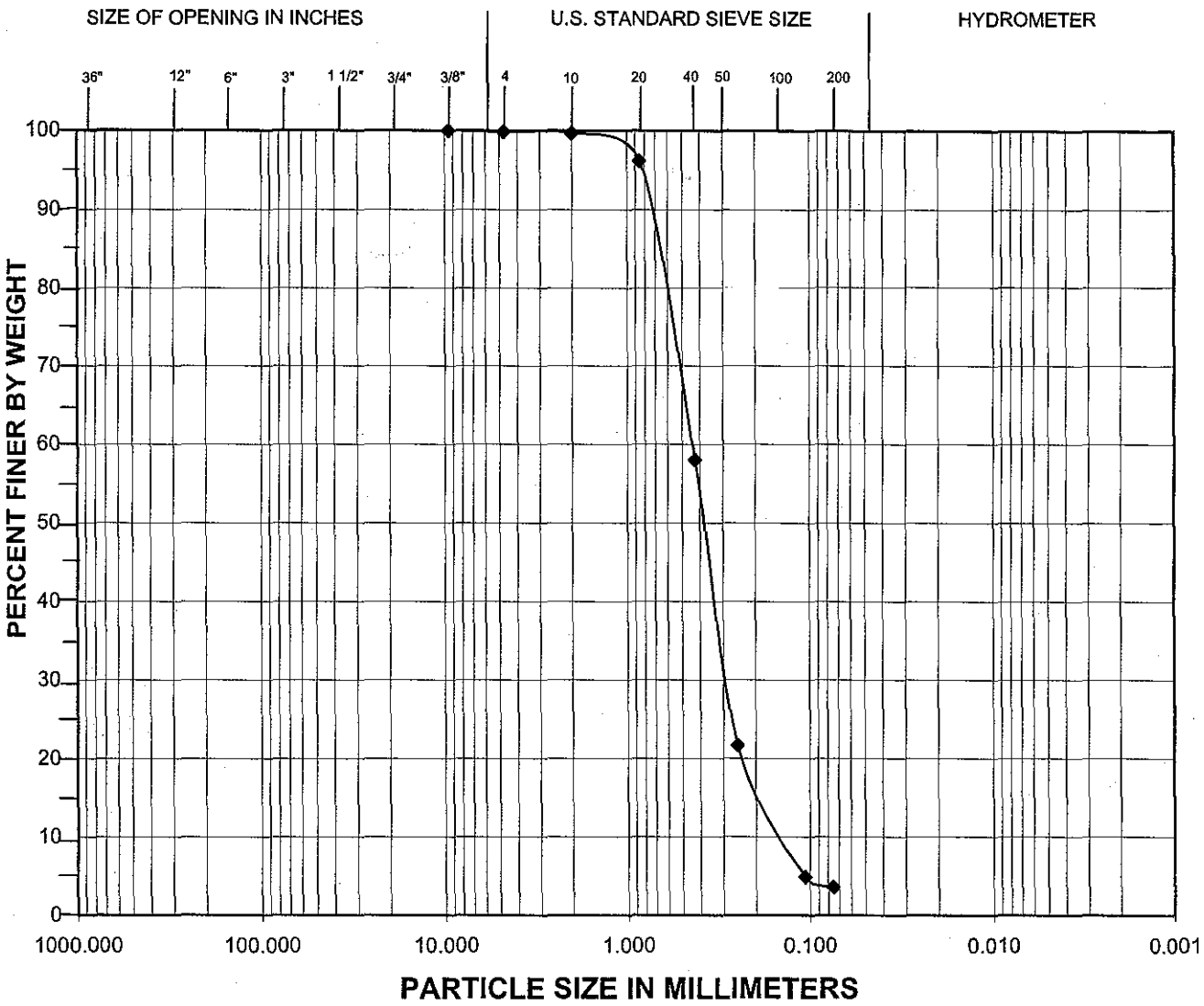
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-5	22.5'	3	4.6	sandy GRAVEL with a trace of silt

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1721 DATE OF TESTING: 6/17/2004	PROJECT NAME: Woodinville

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

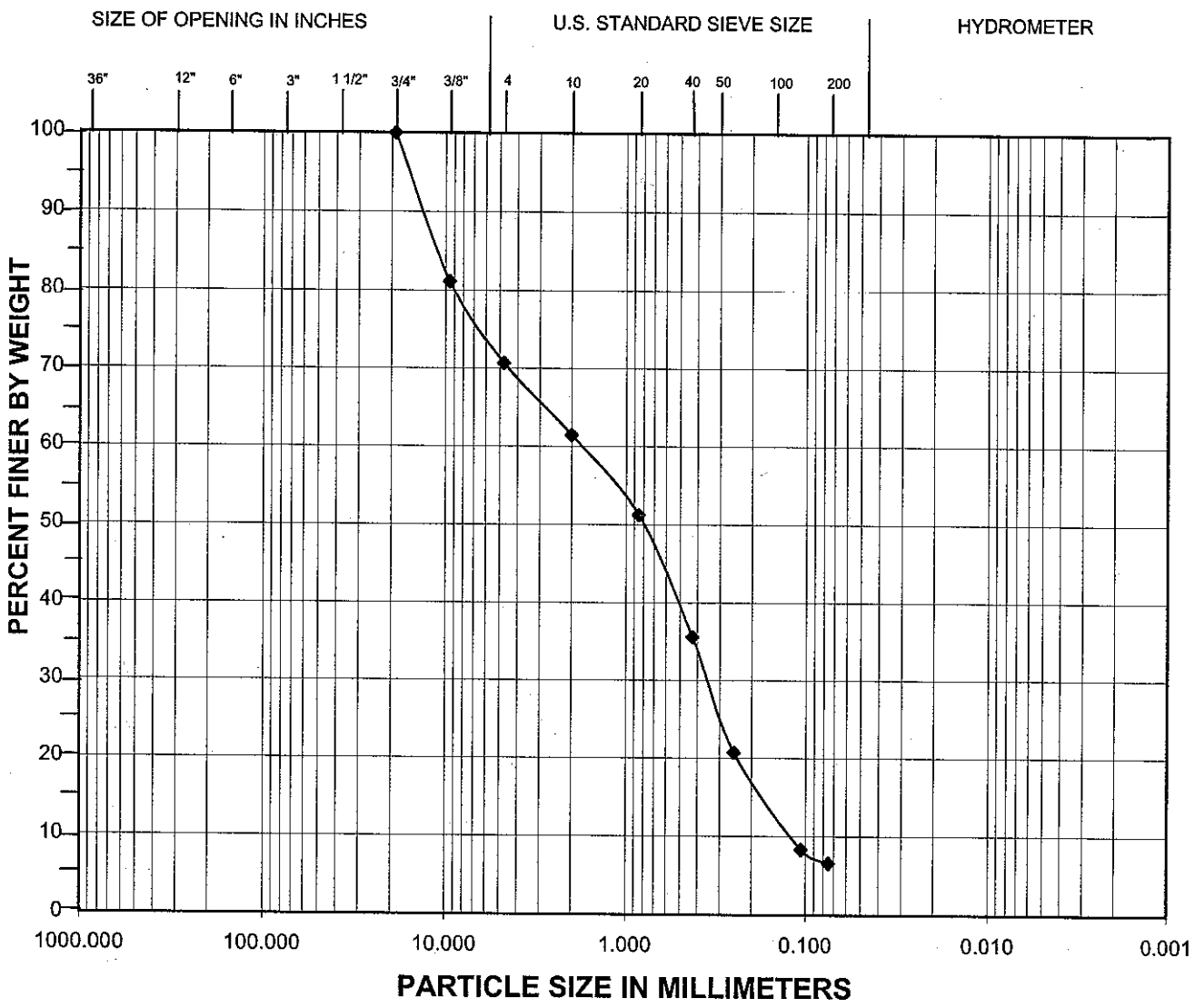
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-16	67.5'	20	3.6	SAND with a trace of SILT and gravel

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1721 DATE OF TESTING: 6/17/2004	PROJECT NAME: Woodinville

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

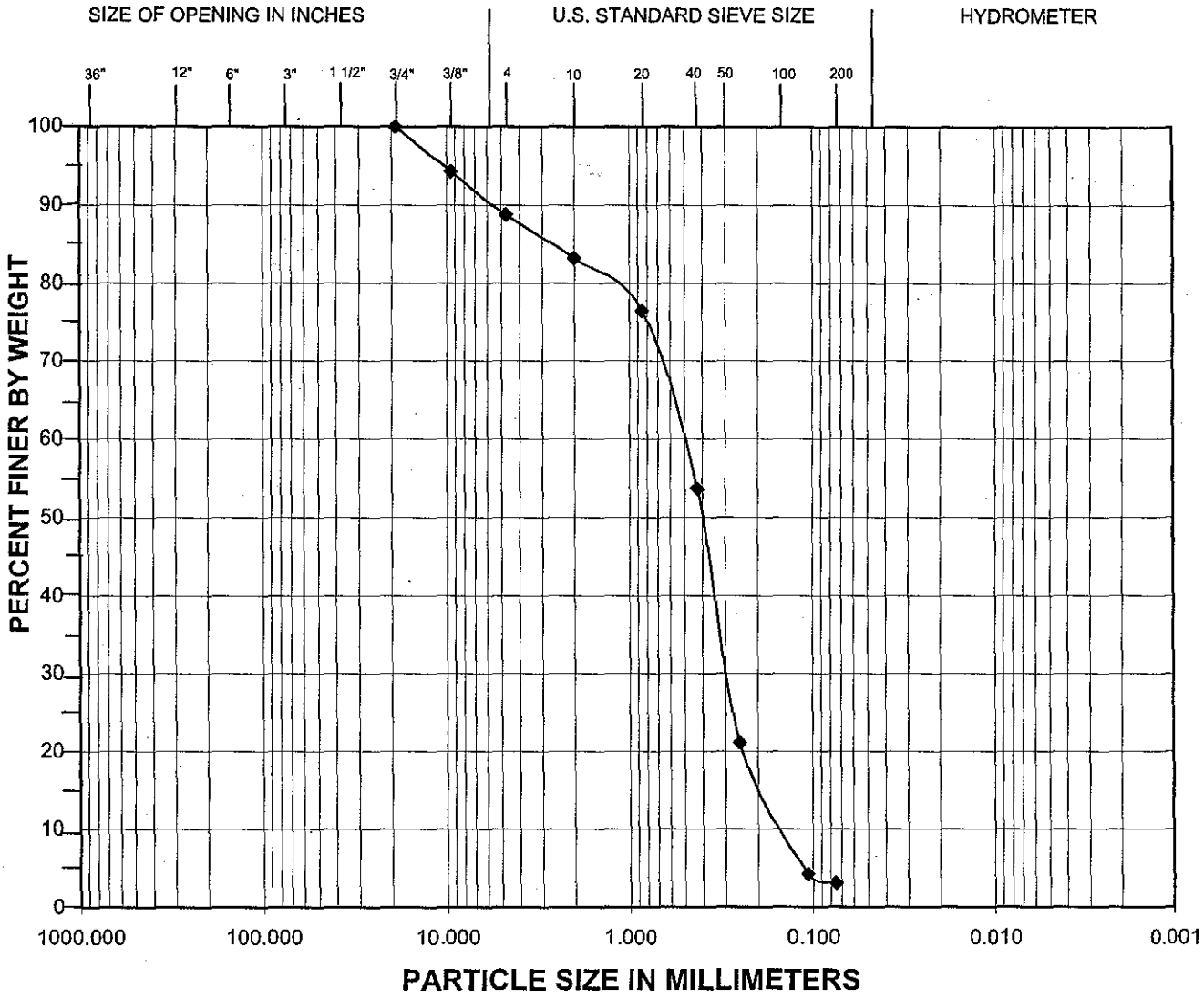
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-5	S-1	2.5'	8	6.6	gravelly SAND with some silt

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1721 DATE OF TESTING: 6/17/2004	PROJECT NAME: Woodinville

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-6	S-4	17.5'	19	3.1	SAND with some gravel, trace of silt

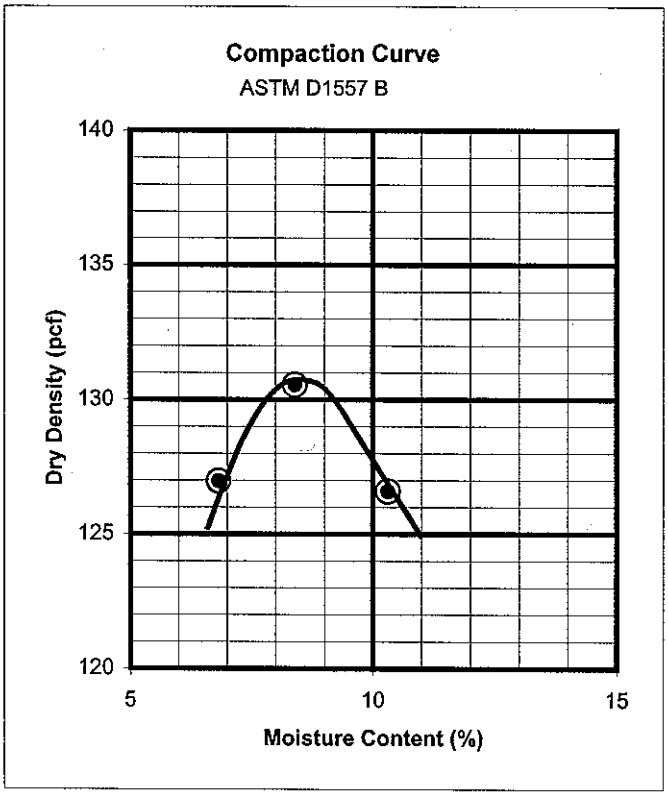
Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1721 DATE OF TESTING: 6/17/2004	PROJECT NAME: Woodinville

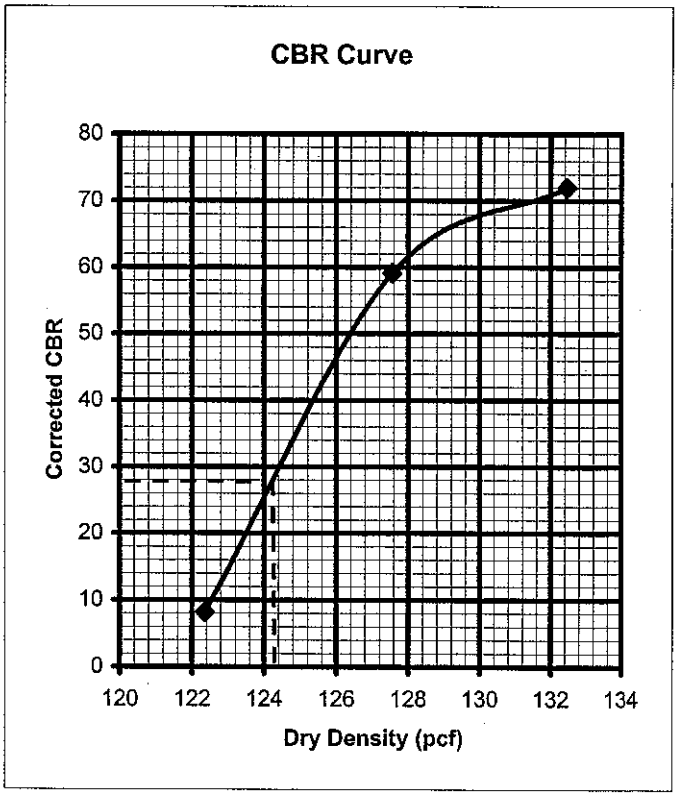
CALIFORNIA BEARING RATIO ASTM D 1883

Exploration: HB-2 and HB-3
Sample No.: S-1 (composite)
Depth: 7-18 inches

Soil Description: gravelly SAND with some silt
Tested By: DVH/KH
Comments: moisture content as received 6.8%

	10 Blows/Lift		25 Blows/Lift		56 Blows/Lift	
Condition of Sample:	soaked		soaked		soaked	
Dry Density Before Soaking:	122	pcf	128	pcf	132	pcf
Dry Density After Soaking:	123	pcf	129	pcf	133	pcf
Moisture Content:						
Before Compaction:	8.3	%	8.4	%	7.8	%
After Compaction:	8.6	%	7.6	%	8.5	%
Top 1-in Layer After Soaking:	11.3	%	8.8	%	8.3	%
Average After Soaking:	9.5	%				
Swell:	0.2	%	0.1	%	0.0	%
Surcharge Amount:	64.8	psf	64.8	psf	64.8	psf





Max. Dry Density = 131 pcf
Optimum Moisture = 8.5 %

CBR at 95% of Max. Dry Density = **28**
95% of Max. Dry Density = **124.5**

ZIPPER ZEMAN ASSOCIATES, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTING	PROJECT NO: J-1721 DATE OF TESTING: 7/15/2004	PROJECT NAME: BNRP-SR202/SR522 I/C Improvements Woodinville, WA

A.4 Boring Logs and Laboratory Testing Results from Zipper Zeman Associates (2001)

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS
LABORATORY TESTING PROCEDURES AND RESULTS

Our field exploration for this project included borings advanced with trailer-mounted drilling equipment and hand auger borings. Approximate exploration locations are shown on the Site and Exploration Plan, Figure 1. Exploration locations were determined by measuring distances from existing site features with a fiberglass tape relative to a site plan, referenced in this report. As such, the exploration locations should be considered accurate to the degree implied by the measurement method. The following sections describe our procedures associated with the explorations. Descriptive logs of the explorations are enclosed in this appendix.

Soil Boring Procedures

Exploratory borings were advanced with a hollow stem auger, using a trailer-mounted drill rig operated by an independent drilling firm working under subcontract to our firm. An experienced geotechnical engineer from our firm continuously observed the borings logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further visual classification and testing. After each boring was completed, the borehole was backfilled with bentonite and soil cuttings.

Throughout the drilling operation, soil samples were obtained at 2.5- to 5-foot depth intervals by means of the Standard Penetration Test (ASTM: D-1586). This testing and sampling procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows is struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

Hand auger borings were advanced and logged by an engineer from our firm. Disturbed soil samples were collected and stored in moisture-tight containers and transported to our laboratory for further visual classification and testing. After each boring was completed, the borehole was backfilled with soil cuttings. Granular soil density and cohesive soil consistency were assessed through the use of the Dynamic Cone Penetrometer (DCP) in general accordance with ASTM Special Technical Publication No. 399.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between

sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring, as well as any laboratory tests performed on these soil samples. If any groundwater was encountered in a borehole, the approximate groundwater depth, and date of observation, is depicted on the log. Groundwater depth estimates are typically based on the moisture content of soil samples, the wetted portion of the drilling rods, the water level measured in the borehole after the auger has been extracted, or through the use of an observation well.

The boring logs presented in this appendix are based upon the drilling action, observation of the samples secured, laboratory test results, and field logs. The various types of soils are indicated as well as the depth where the soils or characteristics of the soils changed. It should be noted that these changes may have been gradual, and if the changes occurred between sample intervals, they were inferred.

LABORATORY TESTING PROCEDURES

A series of laboratory tests were performed during the course of this study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of tests performed are given below.

Visual Classification

Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with the Unified Soil Classification system. Visual soil classification includes evaluation of color, relative moisture content, soil type based upon grain size, and accessory soil types included in the sample. Soil classifications are presented on the exploration logs in Appendix A.

Moisture Content Determinations

Moisture content determinations were performed on representative samples obtained from the exploration in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM: D-2216. The results are shown on the exploration logs in Appendix A.

Grain Size Analysis

A grain size analysis indicates the range in diameter of soil particles included in a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM: D-422. The results of the grain size determinations for the samples were used in classification of the soils, and are presented in this appendix.

PROJECT: Woodinville Signal Poles | **JOB NO.** J-889 | **BORING** B-1 | **PAGE 1 OF 2**

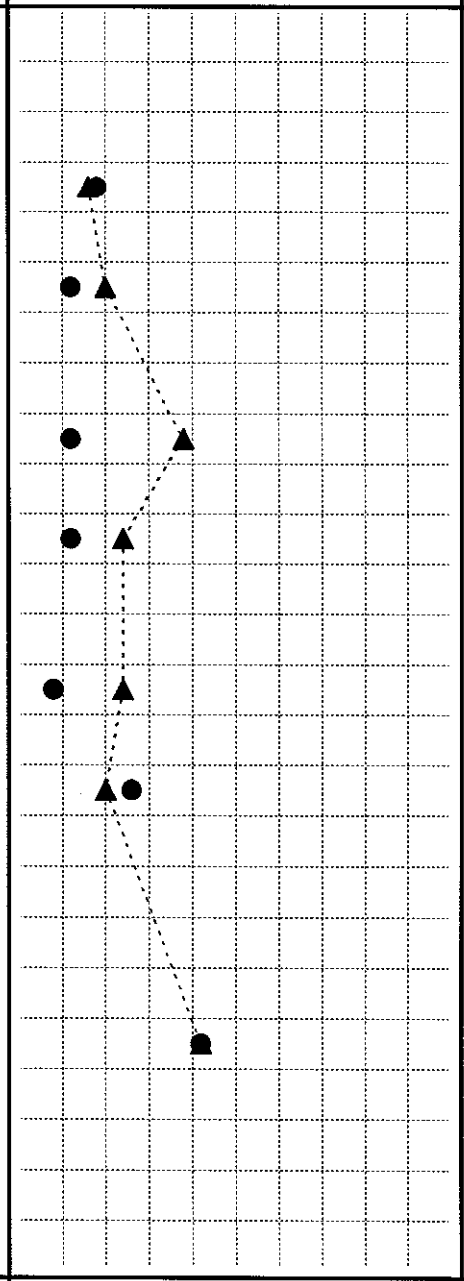
Location: See Site and Exploration Plan, Figure 1 | **Approximate Elevation:** Coincident with adjacent sidewalk

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
	2" Grass sod.					
	Loose, moist, mottled gray-brown, silty SAND with trace gravel. (Fill)					
	Loose, moist, dark brown, fine to medium SAND with some silt. (Relic Topsoil)		S-1		8	
5	Loose, moist, light gray-brown, fine to medium SAND with some silt.		S-2		10	
	Medium dense, moist, light gray-brown, SAND with some gravel and silt. (Blowcount overstated rock in sampler)		S-3		19	
10	Medium dense, moist to wet, light gray-brown, SAND with some gravel amd trace to some silt.		S-4		12	GS
	Medium dense, moist, light gray-brown, SAND with some gravel and trace to some silt.		S-5		12	
15	Loose, moist grading to saturated, gray-brown, fine to medium SAND with trace silt.		S-6	▼ ATD	10	
20	Medium dense, saturated, gray, well graded SAND with some gravel and trace silt.		S-7		21	
25						

Penetration Resistance

▲ Standard — Blows per foot — △ Other

0 10 20 30 40 50



0 10 20 30 40 50

Moisture Content

Plastic Limit — Natural — Liquid Limit

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Cuttings
- Bentonite
- Grout
- Screened Casing

Zipper Zeman Associates, Inc. Geotechnical & Environmental Consultants	BORING LOG	Figure A-1
	Date Drilled: 3/21/01	Logged By: JPG

PROJECT: Woodinville Signal Poles | **JOB NO.** J-889 | **BORING** B-1 | **PAGE 2 OF 2**

Location: See Site an Exploration Plan, Figure 1 | **Approximate Elevation:** Coincident with existing adjacent sidewalk

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot: 0 10 20 30 40 50)	N-values	Testing
	Dense, saturated, gray, well graded SAND with some gravel and trace silt.	I	S-8		● ▲	48	
	Boring terminated at 26.5 feet on 3/21/01. Groundwater seepage observed at 16 feet ATD.						
30							
35							
40							
45							
50							

Explanation

- I — 2-inch O.D. split spoon sample
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Cuttings
- Bentonite
- Grout
- Screened Casing

0 10 20 30 40 50

Moisture Content

Plastic Limit — Natural — Liquid Limit

Zipper Zeman Associates, Inc. Geotechnical & Environmental Consultants	BORING LOG	Figure A-1
	Date Drilled: 2/21/01	Logged By: JPG

PROJECT: Woodinville Signal Poles | **JOB NO.** J-889 | **BORING** B-2 | **PAGE 1 OF 1**

Location: See Site and Exploration Plan, Figure 1 | **Approximate Elevation:** 4 1/2 feet higher than existing adjacent sidewalk

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
	Loose, moist, gray-brown, silty SAND with trace gravel. (Fill)					
	Medium dense, moist, gray-brown, SAND with some gravel and silt.		S-1		13	
5	Loose, moist, gray-brown, well graded SAND with trace gravel and silt.		S-2		9	
	Loose, moist, gray-brown, gravelly, well graded SAND with trace gravel and silt.		S-3		9	GS
10	Loose, moist grading to wet, gray-brown, fine to medium SAND with trace to some silt.		S-4		9	
	Medium dense, saturated, gray-brown, well graded SAND with some silt.		S-5	ATD	21	
15	Loose, saturated, gray-brown, fine to medium SAND with trace to some silt.		S-6		10	
20	Medium dense, saturated, gray, well graded SAND with trace gravel.		S-7		23	
	Boring terminated at 21.5 feet on 3/21/01. Groundwater seepage observed at 13.5 feet ATD.					
25						

Penetration Resistance

▲ Standard — Blows per foot — △ Other

0 10 20 30 40 50

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

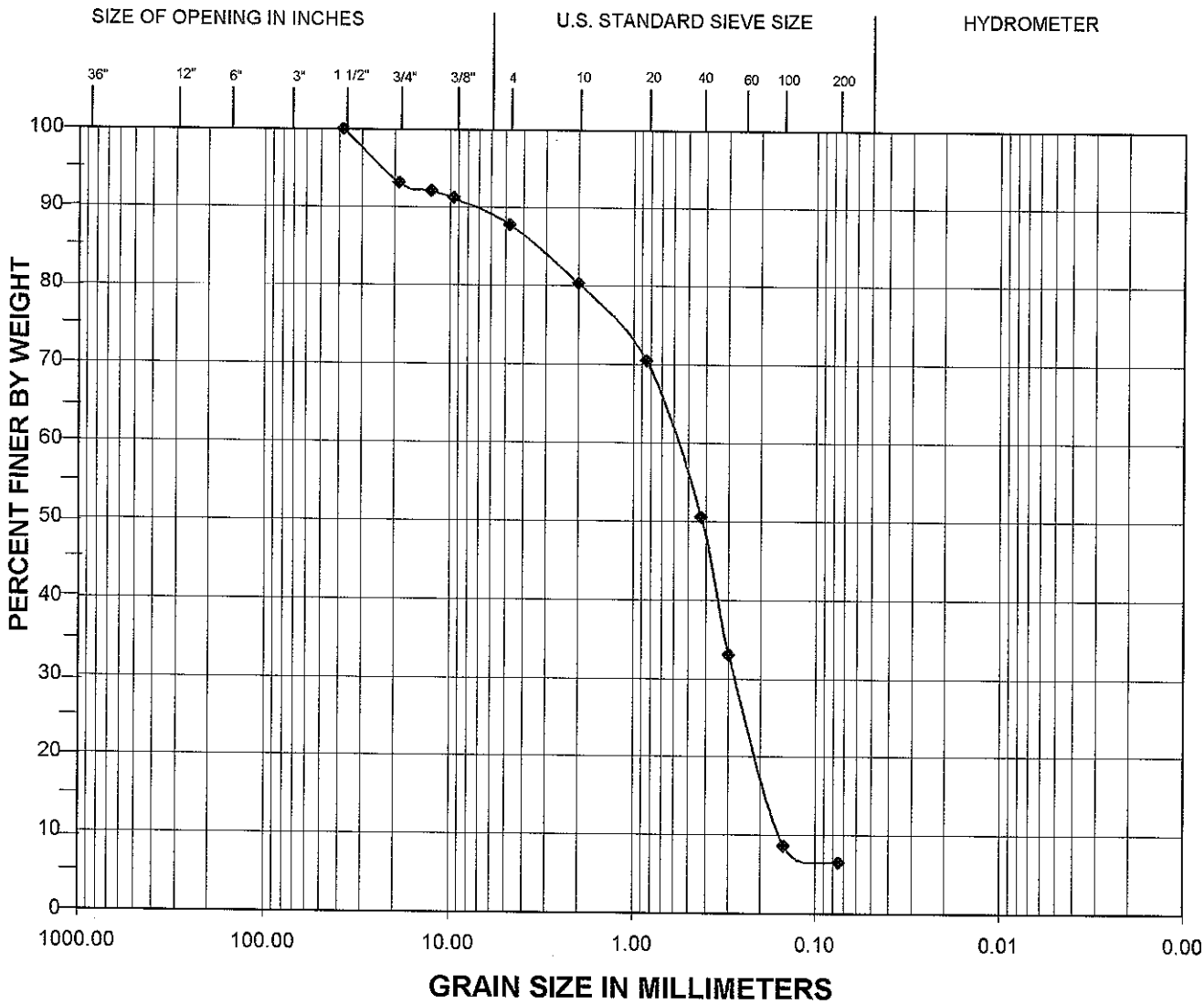
- Clean Sand
- Cuttings
- Bentonite
- Grout
- Screened Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Zipper Zeman Associates, Inc. Geotechnical & Environmental Consultants	BORING LOG	Figure A-1
	Date Drilled: 3/21/01	Logged By: JPG

GRAIN SIZE DISTRIBUTION ASTM D422

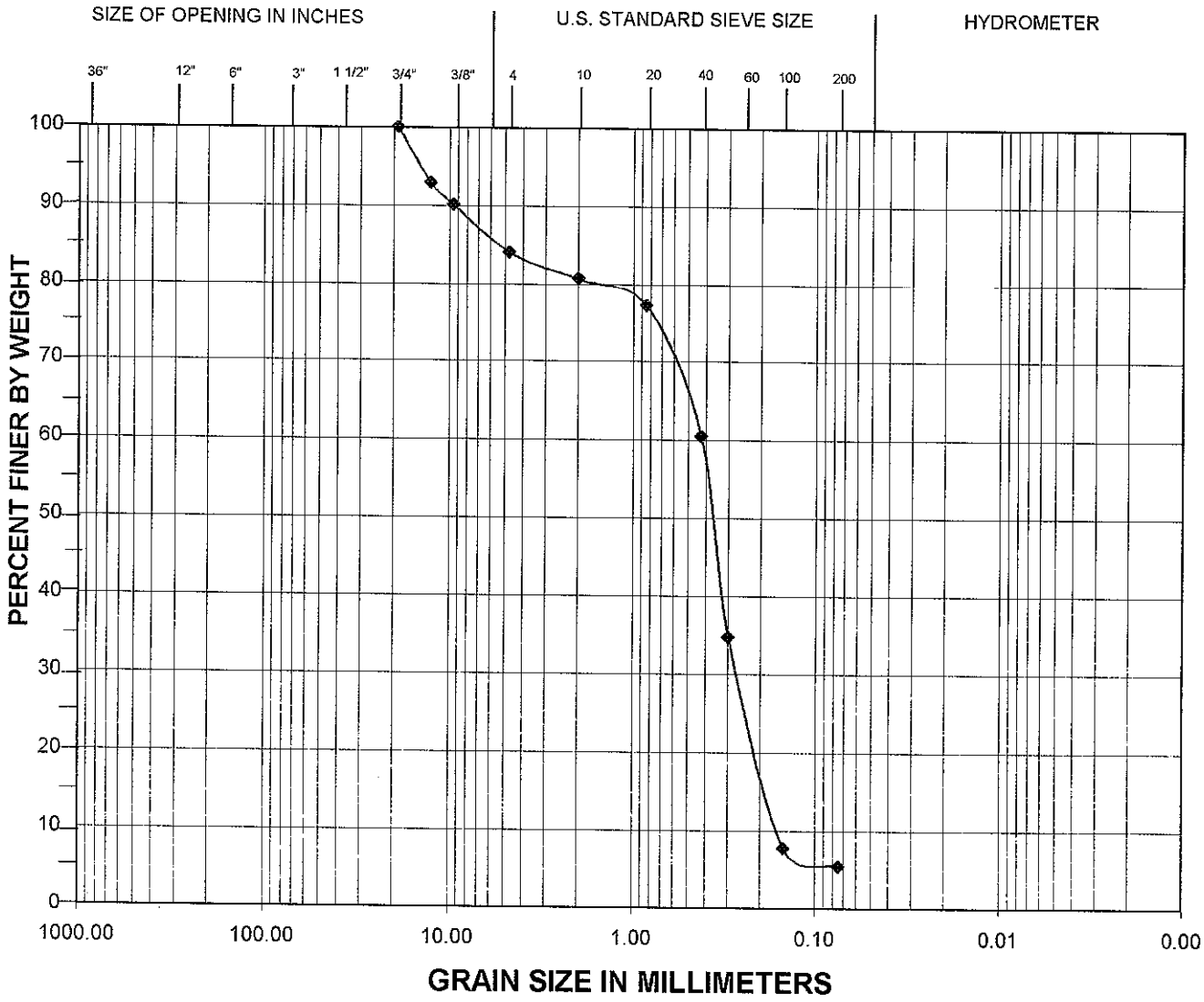


		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-4	10'	6	6	SAND with some silt and gravel.

ZIPPER ZEMAN ASSOCIATES, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTING	PROJECT NO: J-889 DATE OF TESTING: 3/23/01	PROJECT NAME: Woodinville Signal Pole.

GRAIN SIZE DISTRIBUTION ASTM D422



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-3	7.5'	7	5	Gravelly SAND with trace silt.

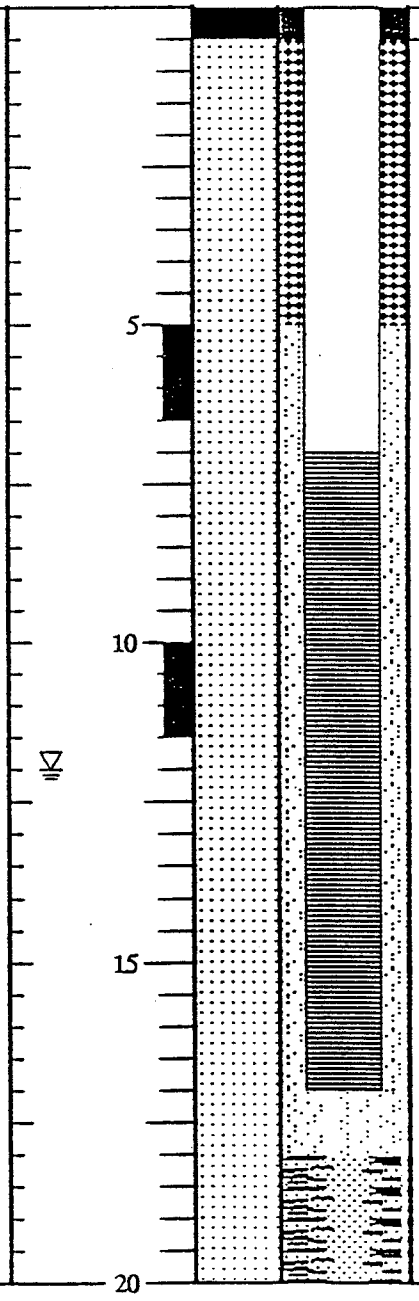
ZIPPER ZEMAN ASSOCIATES, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTING	PROJECT NO: J-889 DATE OF TESTING: 3/22/01	PROJECT NAME: Woodinville Signal Pole.

A.5 Monitoring Well Logs from Sweet-Edwards/EMCON (1990)

LOG OF EXPLORATORY BORING

PROJECT NAME	Shell/Texaco	BORING NO.	SB-4
LOCATION	Woodinville, Washington	PAGE	1 OF 1
DRILLED BY	Pacific Testing Lab	REFERENCE ELEV.	99.54'
DRILL METHOD	H.S. Auger	TOTAL DEPTH	20.00'
LOGGED BY	Mike Noll	DATE COMPLETED	7/16/90

SAMPLING METHOD and NUMBER	PID (ppm)	BLOWS per 6"
SB-1	4.7	3 3 5
SB-2	1.6	3 5 7



LITHOLOGIC DESCRIPTION

0 - 0.5 foot: ASPHALT (AS).

0.5 - 20 feet: SAND with gravel (SP), dark brown, becoming olive at 5 feet, fine to medium and some coarse, some fine to coarse gravel (subrounded to 3-inch diameter), few to some low plasticity fines, loose, dry to moist, becoming wet at 12 feet. (ALLUVIUM)

Well Completion Details - MW-4:

0.2 - 7 feet: 4-inch diameter schedule 40 PVC riser pipe

7 - 17 feet: 4-inch diameter schedule 40 PVC screen with 0.020-inch slots

0 - 0.5 feet: concrete

0.5 - 5 feet: bentonite chips

5 - 18 feet: 8x12 Colorado silica sand

18 - 20 feet: native caved material

Bottom of boring at 20 feet.

REMARKS

1) SB = split barrel samples collected using a modified Dames & Moore 2.5" ID core barrel driven with a 300# hammer. 2) REFERENCE ELEVATION represents top of PVC casing relative to a site datum. 3) Blow counts do not represent Standard Penetration Test results. 4) MW-4 installed in SB-4. 5) PID = Model 580B photoionization detector.

LOG OF EXPLORATORY BORING

PROJECT NAME	Shell/Texaco	BORING NO.	SB-5
LOCATION	Woodinville, Washington	PAGE	1 OF 1
DRILLED BY	Pacific Testing Lab	REFERENCE ELEV.	99.53'
DRILL METHOD	H.S. Auger	TOTAL DEPTH	20.00'
LOGGED BY	Mike Noll	DATE COMPLETED	7/17/90

SAMPLING METHOD and NUMBER	PID (ppm)	BLOWS per 6"	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHO-LOGIC COLUMN	WELL DETAILS	LITHOLOGIC DESCRIPTION
								0 - 0.5 foot: ASPHALT (AS).
	3.4							0.5 - 4 feet: SILTY SAND with gravel (SM), very dark gray, fine to coarse, few to some low plasticity fines; few fine to coarse gravel (subrounded, to 3-inch diameter), loose to medium dense, dry to moist. (TOPSOIL/ALLUVIUM)
SB-1	1.5	3 7 9		5				4 - 6 feet: SAND with gravel (SP), dark yellowish brown, fine to coarse, some fine to coarse gravel (subrounded, to 3-inch diameter), trace fines, loose to medium dense, dry to moist. (ALLUVIUM)
				10				6 - 20 feet: SAND with gravel (SW), olive !6 - 20 feet: SAN to coarse gravel (subrounded, to 3-inch diameter), loose, moist, becoming wet at 10 feet. (ALLUVIUM)
			▽					Well Completion Details - MW-5: 0.2 - 9 feet: 4-inch diameter schedule 40 PVC riser pipe 9 - 19 feet: 4-inch diameter schedule 40 PVC screen with 0.020-inch slots 0 - 0.5 feet: concrete 0.5 - 3.5 feet: bentonite chips 3.5 - 5.5 feet: native caved material 5.5 - 7 feet: bentonite chips 7 - 19 feet: 8x12 Colorado silica sand 19 - 20 feet: native caved material
				15				
				20				Bottom of boring at 20 feet.

REMARKS

1) SB = split barrel samples collected using a modified Dames & Moore 2.5" ID core barrel driven with a 300# hammer. 2) REFERENCE ELEVATION represents top of PVC casing relative to a site datum. 3) Blow counts do not represent Standard Penetration Test results. 4) MW-5 installed in SB-5. 5) PID = Model 580B photoionization detector.

A.6 Boring Logs from Washington State Department of Highways (1988)

H. F. 26.66 (Rev. 5-67)

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Engineer
Copy to

LOG OF TEST BORING

S.H. ____ S.R. 202 Section SR 522 TO N.E. 175th SIGNALS Job No. L 8602
Hole No. H-1A Sub Section ____ Cont. Sec. ____
Station 7+74 SR202 Offset ℄ Ground El. ℄
Type of Boring Mobile B-61 8" augers Casing ____ W.S. El. -10.5' below ground surface
Inspector VFW Date 2-3-88 Sheet 1 of 1

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				HOLE IN Median.
0.36'				ACP
0.76'				MOIST brown medium dense SAND & FINE TO COARSE GRAVEL
1.41'	P-1 44		rec. 0.8 / 10	ACP MOIST. mottled dense slightly gravelly FINE TO COARSE
			24 / 20	SAND
	P-2 16		9 / 8 rec. 1.0'	MOIST mottled medium dense SLIGHTLY GRAVELLY SLIGHTLY SILTY
5.0		0.5'	8 / 10	FINE TO COARSE SAND & Pieces of ASPHALT
	P-3 24		7 / 11 rec. 0.6	MOIST mottled medium dense SLIGHTLY SILTY GRAVELLY FINE TO
			13 / 13	COARSE SAND
	P-4 15		rec. 0.6 / 7	MOIST mottled medium dense SLIGHTLY SILTY SLIGHTLY GRAVELLY
			8 / 7	FINE TO COARSE SAND
10.0			7	WET from -10.5'
	P-5 20		6 / 9 rec. 1.0'	WET mottled medium dense SLIGHTLY SILTY SLIGHTLY GRAVELLY
			11 / 14	FINE TO COARSE SAND
15.0				
	P-6 21		6 / 8 rec. 1.2'	WET mottled medium dense SLIGHTLY SILTY SLIGHTLY
17.0			13 / 15	GRAVELLY FINE TO COARSE SAND
				End of boring 2-3-88 @ -17.0' below ground surface
				water elevation -10.5' below ground surface. Hole
				drilled easy

H. F. 26.66 (Rev. 5-67)

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Engineer
Copy to

LOG OF TEST BORING

S.H. ____ S.R. 202 Section SR522 TO N.E. 175th SIGNALS Job No. L8602
Hole No. H-2A Sub Section SYSTEM I Signal 2 Cont. Sec. ____
Station 10+64 SR202 Offset 47.0' LT. ± Ground El. -0.4' ± (6" below gr. surface)
Type of Boring Mobile B-61 8" Augers Casing ____ W.T. El. -9.0
Inspector VFW Date 2-3-88 Sheet 1 of 1

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.		DESCRIPTION OF MATERIAL
0.0					
	P-1		3	rec,	
	13		5	0.8'	moist mottled medium dense sandy slightly silty fine to
			8		
3.0'			10		coarse GRAVEL
	P-2	O.B.	12	rec,	
	21		10	1.0'	moist mottled medium dense sandy slightly silty fine to
			11		
5.0'			10		coarse GRAVEL
	P-3		6	rec,	
	17		9	1.0'	moist mottled medium dense gravelly slightly silty
			8		
			9		fine to medium SAND
			7		
	P-4		7	rec,	
	14		7	1.0'	moist mottled medium dense gravelly slightly silty fine to
			11		
10.0'					medium SAND wet from -9.0'
	P-5		8	rec,	
	25		12	0.7'	wet mottled dense gravelly slightly silty fine to
			13		
			17		coarse SAND
15.0'					
	P-6		12	rec.	
	34		15	1.4'	wet mottled dense gravelly slightly silty fine to coarse
			19		
17.0'			20		SAND
					End of boring 2-3-88 @ 17.0' below ground surface
					water elevation -9.0' below ground surface
					Hole drilled dry.

J

H. F. 26.66 (Rev. 5-67)

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

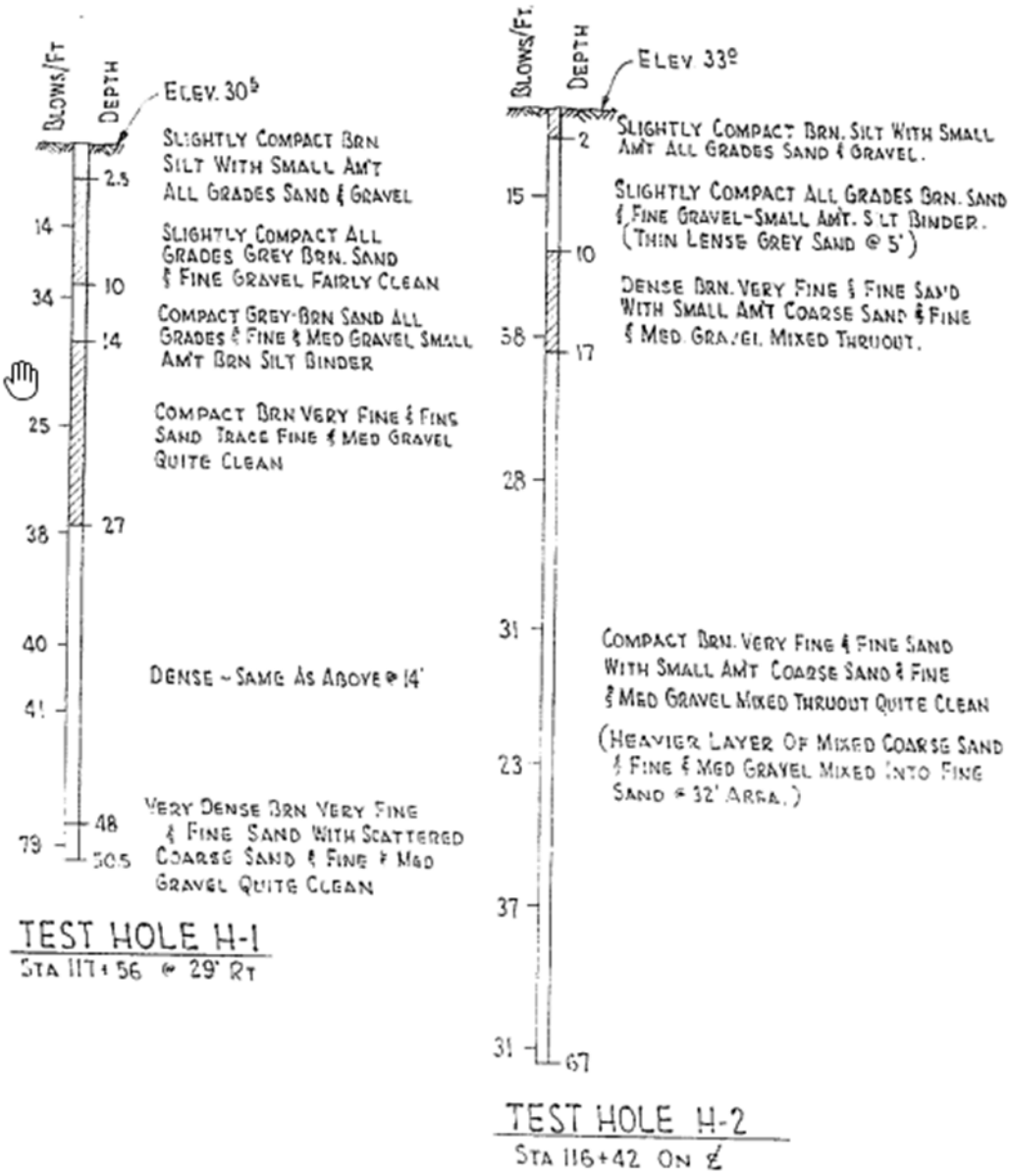
Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Engineer
Copy to .....

LOG OF TEST BORING

S.H. ..... S.R. 202 Section SR522 TO N.E. 175th SIGNALS Job No. L 8602
Hole No. H-3A Sub Section SYSTEM I SIGNAL 4 Cont. Sec. .....
Station 11+66 SR202 Offset 29.0' RT. ℄ Ground El. -1.8' ℄
Type of Boring Mobile B-61 8" Augers Casing ..... W.S. El. -10.0' below ground surface
Inspector KW Date 2-3-88 Sheet 1 of 1

DEPTH 0.0	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	P-1 14		10 rec. 9 0.14'	MOIST MOTTLED MEDIUM DENSE SANDY SLIGHTLY SILTY FINE TO COARSE
3.0'			5 4	GRAVEL
	P-2 14	0.5'	4 rec. 6 0.5'	MOIST MOTTLED MEDIUM DENSE GRAVELLY SLIGHTLY SILTY FINE
5.0'			8 7	TO COARSE SAND
	P-3 14		6 rec. 6 0.5	MOIST MOTTLED MEDIUM DENSE GRAVELLY SLIGHTLY SILTY
			8 6	FINE TO COARSE SAND
	P-4 14		rec. 6 0.3'	MOIST MOTTLED MEDIUM DENSE SLIGHTLY SILTY FINE TO
			7 7	MEDIUM SAND
10.0'			10	WET FROM -9.5'
	P-5 12		5 rec. 6 0.2'	WET MOTTLED MEDIUM DENSE SLIGHTLY GRAVELLY SLIGHTLY
			6 8	SILTY FINE TO COARSE SAND
			12	
	P-6 24		11 rec. 13 1.2'	WET MOTTLED MEDIUM DENSE SLIGHTLY SILTY GRAVELLY FINE TO
15.0'			16	COARSE SAND
	P-7 27		7 rec. 13 1.6'	WET MOTTLED DENSE SLIGHTLY SILTY GRAVELLY FINE TO COARSE
17.0'			14 19	SAND
				End of boring 2-3-88 @ 17.0' below ground
				SURFACE WATER ELEVATION -10.0' below ground
				SURFACE hole drilled easy

A.7 1961 Boring Logs from Washington State Department of Highways (1965)



TEST HOLE REFERENCE : STATE OF WASHINGTON DEPT. OF HIGHWAYS LAYOUT D FOR PRIMARY STATE HIGHWAY Nº 2-B0 DATED JUNE 15, 1961.

ELEVATIONS ARE PER STATE DATUM.
STATE DATUM 10.90 ABOVE RY. DATUM.

APPROVED

BRIDGE ENGINEER N.P.RY.

CHIEF ENGINEER N.P.RY.

NORTHERN PACIFIC RAILWAY CO.
TACOMA DIV. SUMAS BR. 5TH SUB-DIV.
BRIDGE Nº 24!
P.S.H. Nº 2-B0 UNDERCROSSING
WOODINVILLE, WASHINGTON
OFFICE OF BRIDGE ENGINEER
ST. PAUL, MINN. NOV. 30, 1961

D	T	C	N.P. INDEX Nº	SHT
LFC	LFC	JAG	112128	1

STATE HIGHWAYS NOS. 2 AND 1-A
WOODINVILLE VICINITY
KING COUNTY
N. P. RY. UNDERCROSSING

GENERAL PLAN

WASHINGTON STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
OLYMPIA, WASHINGTON

E. A. COWELL, Chairman
E. J. KETCHAM
R. L. MIKALSON
G. S. ZAHN
J. M. MAYE, SR.

APPROVED April 15, 1963

STATE HIGHWAY ENGINEER

SHEET 12 OF 22 SHEETS

CONTRACT NO. 7234

2005-2

5/1014 139/242

APPENDIX B SUBSURFACE EXPLORATION PROGRAM

B.1 2023 Field Exploration Procedures and Logs

Project: SR 202 Widening and Trestle Replacement
Project Location: Woodinville, WA
Project Number: 60683538

Key to Log of Boring and Descriptive Terms for Soil

Unified Soil Classification System (ASTM D2487 & D2488)

Major Divisions			Symbols: Graph	Symbols: Letter	Typical Descriptions
Coarse Grained Soils More than 50% of No. 200 Sieve Size	**Gravels** More than 50% of Coarse Fraction Retained in No. 4 Sieve	**Clean Gravels** (less than 5% fines)		GW	Well-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines
				GP	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines
		Gravels with Fines (more than 12 % fines)		GM	Silty Gravels, Gravel-Sand-Silt Mixtures
				GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
	Sands More than 50% of Coarse Fraction Passing through No. 4 Sieve	**Clean Sand** (less than 5% fines)		SW	Well-Graded Sands, Gravelly Sands, Little or no Fines
				SP	Poorly Graded Sands, Gravelly Sands, Little or no Fines
		Sands with Fines (more than 12 % fines)		SM	Silty Sands, Sand-Clay Mixtures
				SC	Clayey Sands, Sand-Clay Mixtures
Fine Grained Soils More than 50% of Material is Smaller than No. 200 Sieve Size	**Silts and Clays** Liquid Limit Less than 50%			ML	Inorganic Silts and very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
				CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
				OL	Organic Silts and Organic Silty Clays of Low Plasticity
	Silts and Clays Liquid Limit Greater than 50%			MH	Inorganic Silts, Micaceous or Diatomaceous Fine Sand or Silty Soils
				CH	Inorganic Clays of High Plasticity, Fat Clays
				OH	Organic Clays of Medium to High Plasticity, Organic Silts
Highly Organic Soils				PT	Peat, Humus, Swamp Soils with High Organic Contents (see ASTM D4427-92)

Abbreviations

SA	Sieve Analysis
M	Moisture
DD	Dry Density
AL	Atterberg Limits
HA	Hydrometer Analysis
C	Consolidation
Pc	Constant Head Permeability
Pf	Falling Head Permeability
DS	Direct Shear
TX	Triaxial
TV	Torvane Shear
LV	Laboratory Vane Shear
PP	Pocket Penetrometer
OVA	Organic Vapor Analyzer
OC	Organic Content
N	Number of hammer blows for last 12 inches sampled

Field Test Symbols

Pocket Penetrometer Test

Hand Vane Shear Test

Relative Density or Consistency

Coarse-Grained Soils		Fine-Grained Soils	
Relative Density	**N, SPT Blows / ft**	**Relative Consistency**	**N, SPT Blows / ft**
Very loose sand	0 - 4	Very soft	< 2
Loose	4 - 10	Soft	2 - 4
Medium dense	10 - 30	Medium stiff	4 - 8
Dense	30 - 50	Stiff	8 - 15
Very dense	Over 50	Very stiff	15 - 30
		Hard	Over 30

Sampler Symbols

3" O.D. Split Spoon Sample with brass rings

3" O.D. Shelby Tube Sample

Core

Piston Sample

Non-standard penetration test

Grab Sample

2" O.D. Split Spoon with 140lb Hammer and 30-inch drop (SPT)

Minor Descriptors

Trace	0 - 5%
Slightly (clayey, silty, sandy, gravelly)	5 - 12%
Clayey, silty, sandy, gravelly	12 - 30%
Very (clayey, silty, sandy, gravelly)	30 - 50%

Moisture Content

Dry	Absence of moisture, dusty
Moist	Damp but no visible water
Wet	Visible free water, from below the water table

NOTES:

1. Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.
2. Dual Symbols are used to indicate borderline soil classifications

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/8/23	**LOG NO: BT-1**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 35.5	PAGE 1 OF 1 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 16.5	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Kevin B.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-60 Truck Rig	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** SW side of Shell Parking Lot
DRILL METHOD: Hollow Stem Auger	**GROUNDWATER (ft bgs):** 10.67	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE:	**COMPLETION (ft):**	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					ASPHALT, Pavement 4"									
					Moist, light tan to dark gray, fine to medium grained, silty SAND, with GRAVEL, little fine gravel, 8", (FILL)									
					Moist, light brown to dark brown, medium grained, SAND, trace coarse sand, (ALLUVIUM)									
		GB BT-1-HA2.5		100	Medium grained, BT-1-HA2.5: PID Reading 0.0ppm									
5														
		SS BT-1-5.5	3-4-4 (8)	100	Top 5.25-feet hydro-excavated/hand-augered on 2/6/2023. Backilled with import sand and finished with cold-patch. Stop at 11:05 PM. Loose, moist, light tan to dark gray, fine to medium grained, SAND, BT-1-5.5: PID Reading 0.2ppm, (ALLUVIUM)									
		SS BT-1-8	3-6-7 (13)	83	Medium dense, moist, light tan to dark gray, fine to medium grained, SAND, trace fine gravel, BT-1-8: PID Reading 0.4ppm, (ALLUVIUM)									
10														
		SS BT-1-10	3-5-8 (13)	67	▽ Medium dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-1-10: PID Reading 0.5ppm, (ALLUVIUM) Groundwater 10'8" bgs at 12:22 AM on 2/8/23.									
		SS BT-1-12 H2O BT-1-GW	5-6-7 (13)	78	Medium dense, wet, light tan to dark gray, fine to medium grained, SAND, trace coarse gravel, BT-1-12: PID Reading 0.4ppm, (ALLUVIUM) BT-1-GW									
15														
		SS BT-1-15	14-24-29 (53)	89	Very dense, wet, light tan to dark gray, fine to medium grained, SAND, trace fine gravel, BT-1-15: PID Reading 0.5ppm, (ALLUVIUM)									
					Finish Drilling 11:51 PM on 2/7/23. Bottom of borehole is 16.5 ft bgs. Backfilled with bentonite chips and patched with Quikrete FastSet DOT Mix (dyed black). Bottom of hole at 16.5 feet.									

SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 4/21/23 12:48 - \\NA.AECOMNET.COM\LFS\AMER\SEATTLE-USSEA01\DCS\PROJECTS\TRN\60683538_WOODINVILLE_TRESTLE\400_TECHNICAL\433_GEOTECH\BORING LOGS\GINT FILE\60683538_WOODINVI

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/8/23	**LOG NO: BT-2 (MW)**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 35.0	PAGE 1 OF 2 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 21.083	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Kevin B.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-60 Truck Rig	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** West side of Shell Parking Lot
DRILL METHOD: Hollow Stem Auger	**GROUNDWATER (ft bgs):** 10.5	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE:	**COMPLETION (ft):**	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					ASPHALT, Pavement 2"									
					Light tan to dark gray, fine to medium grained, SAND, 7", (FILL)									
					Light brown, fine grained, SAND, (ALLUVIUM)									
		GB BT-2-HA2.5		100	Light brown, fine grained, SAND, BT-2-HA2.5: PID Reading 0.0ppm, (ALLUVIUM)									
5														
		SS BT-2-5.5	5-7-8 (15)	33	Top 5.25-feet hydro-excavated/hand-augered on 2/6/2023. Backilled with import sand and finished with cold-patch. Stop at 10:20 PM. Medium dense, moist, light tan to dark gray, fine to medium grained, SAND, trace fine gravel, BT-2-5.5: PID Reading 0.1ppm, (ALLUVIUM)									
		SS BT-2-7	5-7-9 (16)	44	Medium dense, moist, light tan to dark gray, fine to medium grained, SAND, BT-2-7: PID Reading 0.3ppm, (ALLUVIUM)									
10														
		SS BT-2-10	5-7-8 (15)	44	▽ Medium dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-2-10: PID Reading 0.3ppm, (ALLUVIUM) Groundwater 10'6" bgs at 10:35 PM on 2/8/23									
		SS BT-2-12.5	8-23-50/3"/3"	100	Very dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-2-12.5: PID Reading 0.3ppm, (ALLUVIUM)									
15														
		SS BT-2-15	16-20-50/4"/4"	100	Very dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-2-15: PID Reading 0.3ppm, (ALLUVIUM)									
		SS BT-2-17.5	18-38-50/2"/2"	100	Very dense, wet, light tan to dark gray, fine to medium grained, SAND, little fine gravel, BT-2-17.5: PID Reading 0.4ppm, (ALLUVIUM)									
20														

(Continued Next Page)

SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 4/21/23 12:48 - \\NA.AECOMNET.COM\LFS\AMER\SEATTLE-USSEA01\DCS\PROJECTS\TRN\60683538_WOODINVILLE_TRESTLE\400_TECHNICAL\433_GEOTECH\BORING LOGS\GINT FILE\60683538_WOODINVI

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/8/23	**LOG NO: BT-2 (MW)**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 35.0	PAGE 2 OF 2 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 21.083	

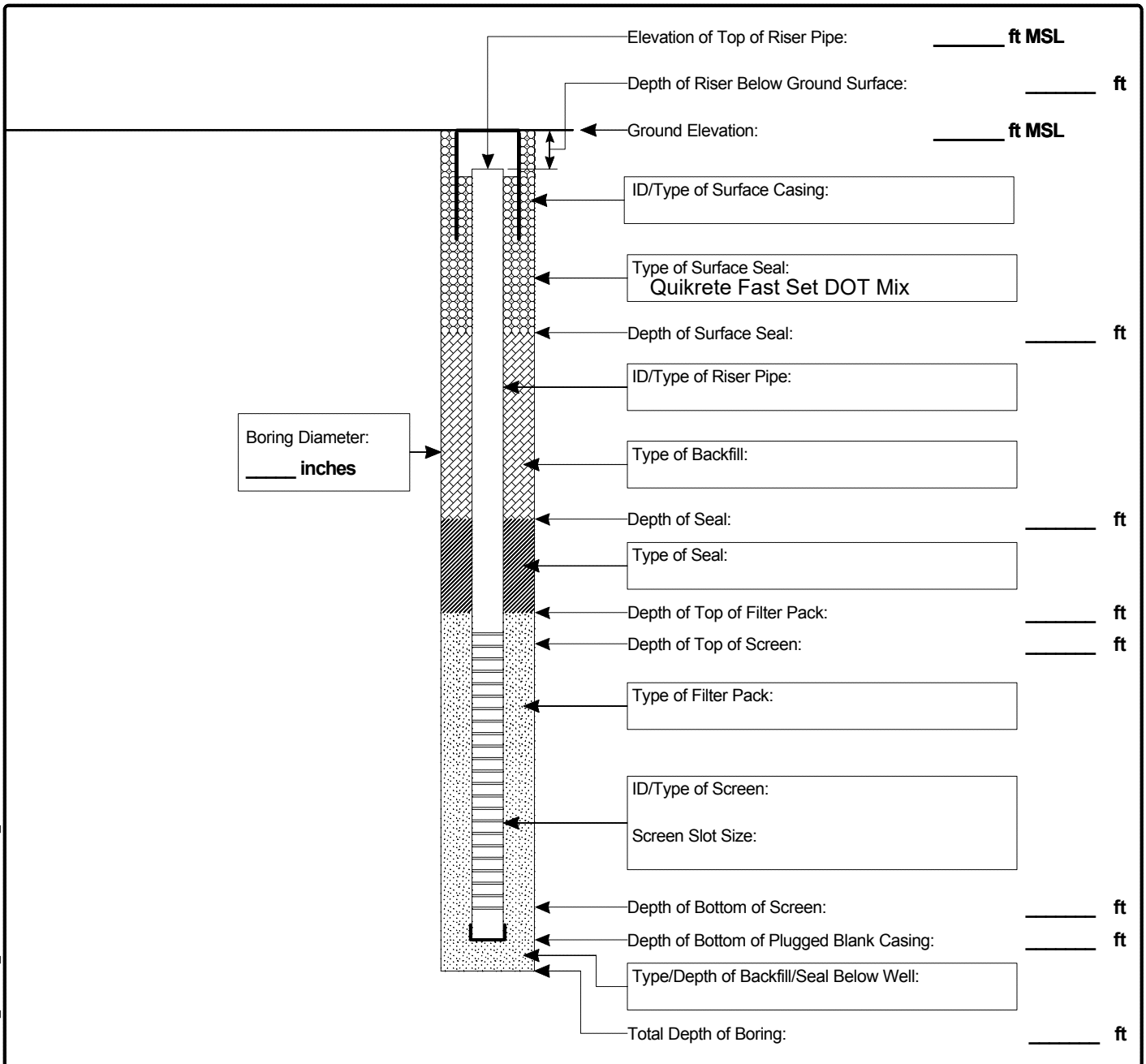
DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
20														
	X	SS BT-2-20	15-37-50//1"	100	Light brown, fine grained, SAND, (ALLUVIUM) *(continued)* Very dense, wet, light tan to dark gray, fine to medium grained, SAND, trace fine to coarse gravel, BT-2-20: PID Reading 0.3ppm, (ALLUVIUM)									
					Finish Drilling 2/8/23 at 1:21 AM. Installed 2" PVC monitoring well to 20' bgs (see well installation log). Well developed on 2/8/2023 at 10:35 PM . Screened interval 10.83' to 9.83'. Filter pack 20' - 9'. 3/8" Bentonite chips seal/backfill 9' to 2'. 10" diameter steel flush-mount locking monumnet 2' - 0'. Top of PVC riser 0.32' below the monument (two 9/16" bolts for locking cover) Bottom of hole at 21.1 feet.									

SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 4/21/23 12:48 - \\NA.AECOMNET.COM\LFS\AMER\SEATTLE-USSEA01\DCS\PROJECTS\TRN\60683538_WOODINVILLE_TRESTLE\400_TECHNICAL\433_GEOTECH\BORING LOGS\GINT FILE\60683538_WOODINVI

Project: SR 202 Widening and Trestle Replacement
Project Location: Woodinville, WA 98072
Project Number: 60683538

MONITORING WELL CONSTRUCTION LOG FOR WELL BT-2 (MW)

Well Location	Date(s) Installed	Time
Installed By	Observed By	Total Depth
Method of Installation		
Screened Interval	Completion Zone	
Remarks		



NOTE: DIAGRAM IS NOT TO SCALE

Report: ENV_WELL_CONSTR_BELOW_GROUND; File: WC_CORP2.GPJ; 8/20/2002 -

URS

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/7/23	**LOG NO: BT-3**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 32.4	PAGE 1 OF 1 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 16.5	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Kevin B.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-60 Truck Rig	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** NEside of Pep Boys Parking Lot
DRILL METHOD: Hollow Stem Auger	**GROUNDWATER (ft bgs):** 7	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE:	**COMPLETION (ft):**	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					ASPHALT, Pavement 2"									
					Fine to medium grained, SAND, 4", (FILL)									
					ASPHALT, Pavement 1"									
					Moist, light tan to dark gray, fine to medium grained, SAND, (ALLUVIUM)									
		GB BT-3-HA2.5		100	Fine to medium grained, SAND, BT-3-HA2.5: PID Reading 0.0ppm, (ALLUVIUM)									
5														
		SS BT-3-5.5	5-3-5 (8)	61	Begin drilling 3/7/34 at 9:12 PM. Top 5.5-feet hydro-excavated/hand-augered on 2/6/2023. Backfilled with import sand and finished with cold-patch. Stop at 9:28 PM. BT-3-5.5: PID Reading 0.1ppm									
					Loose, moist, light tan to dark gray, fine to medium grained, SAND, BT-3-5.5: PID Reading 0.1ppm, (ALLUVIUM)									
		SS BT-3-7	5-7-10 (17)	78	Medium dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-3-7: PID Reading 0.1ppm									
					Groundwater 7.0' bgs at 10:00 PM, (ALLUVIUM)									
10														
		SS BT-3-10 H2O BT-3-GW	5-10-27 (37)	83	Dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-3-10: PID Reading 0.3ppm, (ALLUVIUM)									
					BT-3-GW									
		SS BT-3-12.5	16-29-50/4"/4"	100	Very dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-3-12.5: PID Reading 0.2ppm, (ALLUVIUM)									
15														
		SS BT-3-15	17-17-23 (40)	100	Dense, wet, light tan to dark gray, fine to medium grained, SAND, BT-3-15: PID Reading 0.2ppm, (ALLUVIUM)									
					Finish Drilling on 2/7/23 at 9:50 PM. Bottom of borehole 16.5 ft bgs. Backfilled with bentonite chips and patched with Quikrete Fast Set DOT Mix (dyed black). Bottom of hole at 16.5 feet.									

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PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/4/23 - 1/5/23	**LOG NO: BT-4**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 50.5	PAGE 1 OF 5 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 101.5	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Justin Juggert
DRILLER: Mitch M.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-57 Track Rig	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** RR West side of 131st Ave NE
DRILL METHOD: Mud Rotary	**GROUNDWATER (ft bgs):** 25	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE:	**COMPLETION (ft):** Bentonite chips	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					Railroad Ballast Fill, angular gravel, (FILL)									
					Medium stiff to stiff, moist, light gray to dark gray, SILT, with sand, trace to little gravel, minor (1" +-) clay, (ML), (FILL)									
					Top 5-feet hydro-excavated on 1/4/2023									
		GRAB HA-1		100	HA-1: PID Reading = 0.1ppm, (SM)		16.2						40.3	
5		GRAB HA-2		100	HA-2: PID Reading = 0.1ppm, (CL)		39.5						50.8	
		SS SS-3	5-6-5 (11)	44	Medium stiff to stiff, moist, light gray to dark gray, SILT, with sand, trace to little gravel, (CL), (FILL) SS-3: PID Reading = 0.1ppm				43	25				
10														
		SS SS-4	3-7-6 (13)	33	Medium dense, wet, dark gray to light brown, SAND, with silt, few gravel, (SM), (ALLUVIUM) SS-4: PID Reading = 0.3ppm								39.6	
15														
		SS SS-5	2-3-2 (5)	33	Similar to above. SS-5: PID Reading = 0.1ppm								13.5	
20														

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PROJECT NAME: SR-202 Widening and Trestle Replacement	DATES DRILLED: 1/4/23 - 1/5/23	**LOG NO: BT-4**
PROJECT NO: 60683538	SURFACE ELEVATION (ft): 50.5	PAGE 2 OF 5 AECOM
LOCATION: Woodinville, WA	TOTAL DEPTH (ft): 101.5	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
20														
		SS SS-6	8-9-11 (20)	33	Medium dense, wet, dark brown to dark gray, GRAVEL, with sand, and silt, medium to coarse grained sand, (SP-SM), (ALLUVIUM) SS-6: PID Reading = 0.1ppm		15.1						16	
25					▽									
		SS SS-7	16-17-11 (28)	44	Medium dense, dark gray, GRAVEL, with silt, and sand, (GP-GM), (ALLUVIUM)		7.9				58.1	36.2	5.7	
30														
		SS SS-8	17-22-16 (38)	61	Dense, wet, dark brown to dark gray, GRAVEL, with sand, and silt, medium to coarse grained sand, (GP), (ALLUVIUM) SS-8: PID Reading = 0.0ppm									
35														
		SS SS-9	17-26-24 (50)	61	Dense, wet, dark brown to dark gray, medium to coarse grained, SAND, with trace gravel, trace silt, SS-9: PID Reading = 0.1ppm, (SP), (ALLUVIUM)		10.3							
40														
		SS SS-10	19-22-18 (40)	56	Dense, wet, dark brown to dark olive gray, SAND, with silt, and gravel, Similar to above. SS-10: PID Reading = 0.0ppm, (SP-SM), (ALLUVIUM)		14.2				15.9	73.4	10.7	

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PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/4/23 - 1/5/23	**LOG NO: BT-4**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 50.5	PAGE 3 OF 5 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 101.5	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
45					Dense, wet, dark brown to dark gray, medium to coarse grained, SAND, with trace gravel, trace silt, SS-9: PID Reading = 0.1ppm, (SP), (ALLUVIUM) *(continued)*									
	X	SS SS-11	20-20-19 (39)	72	SS-11: PID Reading = 0.1ppm									
50														
	X	SS SS-12	21-26-25 (51)	72	Very dense, wet, dark brown to dark gray, medium to coarse grained, silty SAND, with trace gravel, (SM), (ALLUVIUM) SS-12: PID Reading = 0.0ppm		17						12.2	
55														
	X	SS SS-13	26-22-21 (43)	61	Dense, wet, dark brown to dark gray, medium to coarse grained, SAND, with SILT, trace gravel, (SP-SM), (ALLUVIUM) SS-13: PID Reading = 0.1ppm								8.9	
60														
	X	SS SS-14	16-16-17 (33)	67	Dense, wet, dark brown to dark gray, medium to coarse grained, SAND, few gravel, (SP-SM), (ALLUVIUM) SS-14: PID Reading = 0.0ppm		18						11.9	
65														
	X	SS SS-15	16-21-23 (44)	67	Dense, wet, dark brown to light brown, medium to coarse grained, SAND, with silt, (SP-SM), (ALLUVIUM) SS-15: PID Reading = 0.0ppm								8.1	

(Continued Next Page)

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/4/23 - 1/5/23	**LOG NO: BT-4**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 50.5	PAGE 4 OF 5 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 101.5	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
70					Dense, wet, dark brown to dark gray, medium to coarse grained, SAND, with trace gravel, trace silt, SS-9: PID Reading = 0.1ppm, (SP), (ALLUVIUM) *(continued)*									
		SS SS-16	14-20-21 (41)	67	Dense, wet, dark brown to gray, medium to coarse grained, SAND, with silt, (SP-SM), (ALLUVIUM) SS-16: PID Reading = 0.1ppm		17						8.4	
75														
		SS SS-17	20-21-17 (38)	67	Dense, wet, dark brown to olive gray, with silt, and gravel, Similar to above SS-17: PID Reading = 0.0ppm, (SP-SM), (ALLUVIUM)		18.8				0	89.1	10.9	
80														
		SS SS-18	11-9-8 (17)	44	Medium dense, wet, dark brown to gray, medium to coarse grained, SAND, with silt, trace to little gravel, (SP-SM), (ALLUVIUM) SS-18: PID Reading = 0.0ppm End drilling on 1/5/2023 at 4:00 PM Resume drilling on 1/6/2023 at 8:30 AM		21.4						6.5	
85					At 83-feet: Driller noted drill rig chatter and slower drilling. Driller noted gravelly drilling conditions									
		SS SS-19	19-16-15 (31)	22	Dense, wet, dark brown to gray, fine to medium grained, silty SAND, (SM), (ALLUVIUM) SS-19: PID Reading = 0.0ppm		35.3						22.8	
90														
		SS		56										

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PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/4/23 - 1/5/23	**LOG NO: BT-4**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 50.5	PAGE 5 OF 5 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 101.5	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
		SS-20 SS SS-20	16-26-30 (56)	56	Very dense, wet, dark brown to dark gray, fine to medium grained, SAND, with silt, (SP-SM), (ALLUVIUM) *(continued)* SS-20: PID Reading = 0.2ppm									
95		SS SS-21	19-18-20 (38)	67	Dense, wet, dark brown, fine to medium grained, silty SAND, with silt, (SM), (ALLUVIUM) SS-21: PID Reading = 0.0ppm		21.3						14.6	
100		SS SS-22	28-30-32 (62)	56	Very dense, wet, dark brown, fine to medium grained, silty SAND, with silt, (SM), (ALLUVIUM) SS-22: PID Reading = 0.0ppm									

PID readings obtained with MiniRAE 3000, lamp voltage 10.6 eV.
Groundwater depth approx. 25 ft bgs based on data from monitoring well B-4 approximately 75 ft west of boing BT-4.
Bottom of hole at 101.5 feet. Borehole terminated at 10:10 AM.
Backfilled with Bentonite chips (borehole termination note)

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PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/5/23 - 1/6/23	**LOG NO: BT-5**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 51.8	PAGE 1 OF 4 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 81.5	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Justin Juggert
DRILLER: Mitch M.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-57 Track Rig	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** RR East side of 131st Ave NE
DRILL METHOD: Mud Rotary	**GROUNDWATER (ft bgs):** 25	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE:	**COMPLETION (ft):** Bentonite chips	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					Ballast Fill, angular gravel, (FILL)									
					Medium dense, moist, dark gray, angular, GRAVEL, with sand, (GP), (FILL)									
					Top 5-feet hydro-excavated on 1/4/2023									
		GRAB HA-1		100	HA-1: PID Reading = 0.1ppm								4.5	
5		GRAB HA-2		100	Loose, moist, light brown to dark brown, fine to medium grained, SAND, with trace gravel, and silt, (SP), (FILL) HA-2: PID Reading = 0.1ppm No sample recovery. Drilling mud in sampler								4.5	
		SS SS-3	5-5-5 (10)	0			4.5							
10														
		SS SS-4	4-3-3 (6)	22	Loose, moist, light brown to dark brown, fine to medium grained, SAND, with silt, trace gravel, (SP-SM), (ALLUVIUM) SS-4: PID Reading = 0.0ppm		17.2						6.2	
15														
		SS SS-5	2-4-3 (7)	11	Similar to above SS-5: PID Reading = 0.0ppm		36.3							
20														

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SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 4/21/23 12:48 - \\NA.AECOMNET.COM\LFS\AMER\SEATTLE-USSEA01\DCS\PROJECTS\TRN\60683538_WOODINVILLE_TRESTLE\400_TECHNICAL\433_GEOTECH\BORING LOGS\GINT FILE\60683538_WOODINVI

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/5/23 - 1/6/23	**LOG NO: BT-5**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 51.8	PAGE 2 OF 4 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 81.5	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
20														
		MC MC-6	7-9-11 (20)	56	Loose, moist, light brown to dark brown, fine to medium grained, SAND, with silt, trace gravel, (SP-SM), (ALLUVIUM) *(continued)* Medium dense, moist, light brown to dark olive brown, poorly graded, SAND, with silt, and gravel, MC-6: PID Reading = 0.0ppm, (SP-SM)		15.5				33	60.1	6.9	
25					▼ Similar to above									
		MC MC-7	10-11-15 (26)	56										
30														
		SS SS-8	17-23-25 (48)	11	Dense, moist, light brown to dark brown, fine to medium grained, SAND, with silt, trace gravel, (SP-SM), (ALLUVIUM) SS-8: Gravel stuck in sampler shoe. N-value not representative of in-situ soil conditions		34.9						7.6	
35														
		SS SS-9	15-15-17 (32)	61	Dense, wet, light brown to dark olive brown, poorly graded, fine to medium grained, SAND, with silt, Becomes wet, (SP-SM), (ALLUVIUM)						7	83.4	9.6	
40														
		SS SS-10	17-18-16 (34)	56	Dense, wet, dark brown, medium to coarse grained, SAND, with silt, (SP-SM), (ALLUVIUM)		25.5						10	

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PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/5/23 - 1/6/23	**LOG NO: BT-5**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 51.8	PAGE 3 OF 4 AECOM
LOCATION: Woodinville, WA	**TOTAL DEPTH (ft):** 81.5	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
45					Loose, moist, light brown to dark brown, fine to medium grained, SAND, with silt, trace gravel, (SP-SM), (ALLUVIUM) *(continued)*									
		SS SS-11	26-32-31 (63)	50	Very dense, wet, dark brown, medium grained, SAND, with some gravel, and silt, (SP), (ALLUVIUM)									
50					Driller noted drill rig chatter and slower driller, continued for about 10ft									
		SS SS-12	22-27-26 (53)	33	Very dense, wet, dark gray to brown, medium to coarse grained, SAND, with silt, and gravel, (GP-GM), (ALLUVIUM)		9.2						7.2	
55														
		SS SS-13	27-28-27 (55)	61	Dense, wet, dark gray to brown, medium grained, SAND, trace silt, (SP-SM), (ALLUVIUM)								9.4	
60														
		SS SS-14	31-47-34 (81)	50			10.9						9.9	
65														
		SS SS-15	23-32-33 (65)	67	Very dense, wet, light brown to dark brown, medium to coarse grained, SAND, with silt, trace gravel, (SP), (ALLUVIUM) End drilling on 1/4/2023 at 4:00 PM Resume Drilling on 1/5/2023 at 9:00 AM								11.7	

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PROJECT NAME: SR-202 Widening and Trestle Replacement	DATES DRILLED: 1/5/23 - 1/6/23	**LOG NO: BT-5**
PROJECT NO: 60683538	SURFACE ELEVATION (ft): 51.8	PAGE 4 OF 4 AECOM
LOCATION: Woodinville, WA	TOTAL DEPTH (ft): 81.5	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
					Dense, wet, dark gray to brown, medium grained, SAND, trace silt, (SP-SM), (ALLUVIUM) *(continued)*									
70														
		SS SS-16	17-18-18 (36)	61	Dense, wet, light brown to dark olive brown, poorly graded, SAND, with silt, (SP-SM), (ALLUVIUM)		19.9				1.5	88	10.5	
75														
		SS SS-17	22-30-32 (62)	39	Very dense, wet, dark brown, medium grained, SAND, with silt, trace gravel, (SP-SM), (ALLUVIUM)		11						9.4	
80														
		SS SS-18	23-21-26 (47)	61	Similar to above but dense		17.5							

PID readings obtained with MiniRAE 3000, lamp voltage 10.6 eV.
Groundwater depth approximately 25 ft below ground surface based on June/July 2006 data from monitoring well B-1 approximately 20' east of boring BT-5, as well as data from other borings nearby., (SP-SM)

Bottom of hole at 81.5 feet. Borehole terminated at 10:10 AM.
Backfilled with Bentonite chips (borehole termination note)

SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 4/21/23 12:48 - \\NA.AECOMNET.COM\LFS\AMER\SEATTLE-USSEA01\DCS\PROJECTS\TRN\60683538_WOODINVILLE_TRESTLE\400_TECHNICAL\433_GEOTECH\BORING LOGS\GINT FILE\60683538_WOODINVI

PROJECT NAME: Woodinville Trestle SR-202 Widening	**DATES DRILLED:** 2/8/23 - 2/9/23	**LOG NO: BT-6**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):**	PAGE 1 OF 1 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 3	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Cody	**AZIMUTH (deg):** N/A	**CHECKED BY:**
DRILL EQUIP: Core Drill	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** S of 131st Ave and Little Bear Ck Pk
DRILL METHOD: Single-tube Coring	**GROUNDWATER (ft bgs):**	**LATITUDE (deg)** Not Surveyed **or NORTHING (ft):**
BIT SIZE/TYPE: 6"	**COMPLETION (ft):** Grouted	**LONGITUDE (deg)** Not Surveyed **or EASTING (ft):**

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					Cold joint at 3" bgs									
					Light tan to dark gray, fine to medium grained, SAND, with SILT, Subrounded fine to coarse gravel, (SP-SM), (ALLUVIUM)									

Finish core/hand-auger at 12:50 AM. Hole backilled with cuttings and patched with black concrete.

Bottom of hole at 3.0 feet. Hole terminated at 12:50 AM. Backfilled with black concrete

SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 2/24/23 09:49 - C:\USERS\MICHAEL.BLACK1\DESKTOP\FIELDWORK\MARKUS WALBAUM\60683538\60683538_WOODINVILLE TRESTLE_2023.01.23_MB.GPJ

PROJECT NAME: Woodinville Trestle SR-202 Widening	**DATES DRILLED:** 2/9/23 - 2/9/23	**LOG NO: BT-7**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):**	PAGE 1 OF 1 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 4.17	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Cody	**AZIMUTH (deg):** N/A	**CHECKED BY:**
DRILL EQUIP: Core Drill	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** Between 175th Ave and RR Trestle
DRILL METHOD: Single-tube Coring	**GROUNDWATER (ft bgs):**	**LATITUDE (deg)** Not Surveyed **or NORTHING (ft):**
BIT SIZE/TYPE: 6"	**COMPLETION (ft):** Grouted	**LONGITUDE (deg)** Not Surveyed **or EASTING (ft):**

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					Cold joint at 14" bgs									
					Light tan to dark gray, fine to medium grained, SAND, little fine to coarse gravel, trace cobbles, Subrounded gravel and cobbles, (SP), (ALLUVIUM)									

Finish core/vacuum at 9:00 PM. Hole backilled with cuttings and patched with black concrete.
Bottom of hole at 4.2 feet. Hole terminated at 9:00 PM. Backfilled with black concrete

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PROJECT NAME: Woodinville Trestle SR-202 Widening	**DATES DRILLED:** 2/9/23 - 2/9/23	**LOG NO: BT-8**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):**	PAGE 1 OF 1 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 3.17	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Cody	**AZIMUTH (deg):** N/A	**CHECKED BY:**
DRILL EQUIP: Core Drill	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** N of 131st Ave and 175th Ave
DRILL METHOD: Single-tube Coring	**GROUNDWATER (ft bgs):**	**LATITUDE (deg)** Not Surveyed **or NORTHING (ft):**
BIT SIZE/TYPE: 6"	**COMPLETION (ft):** Grouted	**LONGITUDE (deg)** Not Surveyed **or EASTING (ft):**

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					Cold joint at 6" bgs Light tan to dark gray, SAND, little fine to coarse gravel, trace cobbles, Subrounded gravel and cobbles, (SP), (ALLUVIUM)									

Finish core/hand-auger at 2:20 AM. Hole backilled with cuttings and patched with black concrete.
Bottom of hole at 3.2 feet. Hole terminated at 2:20 AM. Backfilled with black concrete

SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 2/24/23 09:49 - C:\USERS\MICHAEL.BLACK1\DESKTOP\FIELDWORK\MARKUS WALBAUM\60683538\60683538_WOODINVILLE TRESTLE_2023.01.23_MB.GPJ

PROJECT NAME: Woodinville Trestle SR-202 Widening	**DATES DRILLED:** 2/9/23 - 2/9/23	**LOG NO: BT-9**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):**	PAGE 1 OF 1 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 1.125	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Cody	**AZIMUTH (deg):** N/A	**CHECKED BY:**
DRILL EQUIP: Core Drill	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** Below RR Trestle
DRILL METHOD: Single-tube Coring	**GROUNDWATER (ft bgs):**	**LATITUDE (deg)** Not Surveyed **or NORTHING (ft):**
BIT SIZE/TYPE: 6"	**COMPLETION (ft):** Grouted	**LONGITUDE (deg)** Not Surveyed **or EASTING (ft):**

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
0					Cold joint at 8" bgs									
					Finish core at 10:00 PM. Hole patched with black concrete.									
					Bottom of hole at 1.1 feet. Hole terminated at 10:00 PM. Backfilled with black concrete									

SMART-SOIL-LAB - URS_DENVER STD US LAB.GDT - 2/24/23 09:49 - C:\USERS\MICHAEL.BLACK1\DESKTOP\FIELDWORK\MARKUS WALBAUM\60683538\60683538_WOODINVILLE TRESTLE_2023.01.23_MB.GPJ

B.2 SPT Energy Measurement Report for Rig #215 (Borings BT-1, BT-2, BT-3)

Robert Miner Dynamic Testing, Inc.
Dynamic Measurements and Analyses for Deep Foundations

November 15, 2018

Mr. Travis Stephens
Holt Services, Inc.
10621 Todd Rd. East
Edgewood, WA 98372

Re: Penetration Test Energy Measurements
Mobile B-60 Rig 215, Mobile Auto Hammer
Bore Hole: B1, November 14, 2018
POR Underwood, Camas, Washington

RMDT Job No. 18F39

Dear Mr. Stephens:

This letter presents energy transfer measurements made during Standard Penetration Tests for the drill hole and drill rig referenced above. Robert Miner Dynamic Testing, Inc. (RMDT) made dynamic measurements with a Pile Driving Analyzer® as a hammer advanced the NW rod during sampling with a split spoon sampler.

The purpose of RMDT's testing was the measurement of energy transferred to the drill rods. Measurements were made on a section of NW gauge rod at the top of the drill string. Strain gages and accelerometers on the rod were connected to a Pile Driving Analyzer® (PDA) which generally processed acceleration and strain measurements from each hammer blow and stored both the measurements and computed results. Measurements and data processing generally followed the ASTM D 4633-16 standard. Energy transfer past the gage location, EFV, was computed by the PDA using force and velocity records as follows:

$$EFV = \int_a^b F(t)\, v(t)\, dt$$

The value "a" corresponds to the start of the record which is when the energy transfer begins and *"b"* is the time at which energy transferred to the rod reaches a maximum value. Appendix A contains more information on our measurement equipment and methods of analysis. The EFV energy calculation is identical to the EMX energy result discussed in Appendix A. The EFV and EMX values apply to the sensor location near the top of the rod.

TEST DETAILS

On November 14, 2018, a single boring was advanced at the POR Underwood project site in Camas, Washington. The drill rig used during sampling was a truck mounted Mobile B-60 unit manufactured by Mobile Drilling International and referred to as Rig 215 by the operator. The B-60 unit drilled to predetermined depth intervals ranging from 5 to 45 ft below ground surface.

The rod used to advance the spoon at each sample depth had a diameter matching that of NW rod. The automatic hammer in use during our testing was manufactured by Mobile Drilling International and appeared to use a chain drive powered by a hydraulic motor, with the ram and chain drive enclosed within an outer casing.

RESULTS

A summary of testing and monitoring results is given in Table 1. The tabulated results include the starting sample depth, the penetration resistance, the number of hammers blows in our data set, measured energy transfer, EFV, the computed transfer efficiency, ETR, and the hammer blow rate, BPM. Appendix B contains detailed numeric results for each individual test.

Energy measurements must be divided by the theoretical free fall energy of the hammer to obtain an efficiency. A 140 lb ram raised 30 inches above an impact surface has 350 lb-ft of potential energy. Thus, the transfer energy results for sampling with the 140 lb ram may be divided by 350 lb-ft to yield the ratio of the delivered energy to the nominal potential energy. This efficiency ratio, ETR, is given for each sample interval as a percent efficiency.

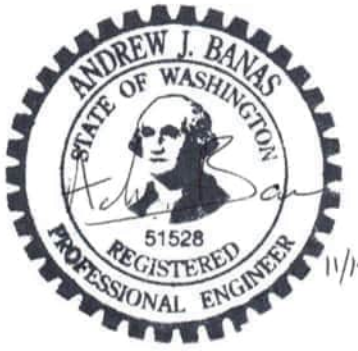
Table 1. Summary of Test Details and Results for the 140-lb ram and Split Spoon Sampler

Sample Name and Sample Depth	Penetration Resistance (Blow/Set)	Number of Blows in Data Set	Average Transfer Energy EFV (lb-ft)	Average Transfer Efficiency ETR (percent)	Average Hammer Blow Rate BPM (blow/min)
20 ft Sample	52/3 in	51	302	86	44
25 ft Sample	51/1 in	50	304	87	44
30 ft Sample	52/2 in	50	311	89	45
35 ft Sample	50/2 in	50	308	88	43
40 ft Sample	50/4 in	45	312	89	44
Average for Split Spoon samples:			**307**	**88**	**44**

Five sample returns were monitored while the 140 lb ram and standard split spoon sampler were in use. The overall average ETR and hammer blow rate was 88 percent and 44 blows per minute, respectively.

It was a pleasure to assist you and to participate on this project with the staff of Holt Services Inc. Please do not hesitate to contact us if you or other project participants have any questions about this report.

Sincerely,



Andrew J. Banas, P.E.

Robert Miner Dynamic Testing, Inc.

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

The following has been written by Goble Rausche Likins and Associates, Inc. and may only be copied with its written permission.

BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during preconstruction test programs and also production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (such as that of a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the Case Method or "High Strain Test" Method of pile testing, however, for the sake of completeness, the "Low Strain Test" performed with the Pile Integrity Test™ (PIT), mainly for pile integrity evaluation, will also be described.

RESULTS FROM DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing*.

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both cast *insitu* piles or drilled shafts and impact driven piles during restrike.

Dynamic Pile Monitoring

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- Bearing capacity at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- Dynamic pile stresses, axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- Pile integrity assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- Hammer performance parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

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Dynamic Pile Load Testing

Bearing capacity testing of either driven piles or drilled shafts applies the same basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it most important that the test is conducted after a sufficient waiting time following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- Bearing capacity i.e. the mobilized capacity present at the time of testing
- Resistance distribution including shaft resistance and end bearing components
- Stresses in pile or shaft calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.
- Shaft impedance vs depth; this is an estimate of the shaft shape if it differs substantially from the planned profile
- Dynamic soil parameters for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

MEASUREMENTS

PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

Saximeter™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

PIT

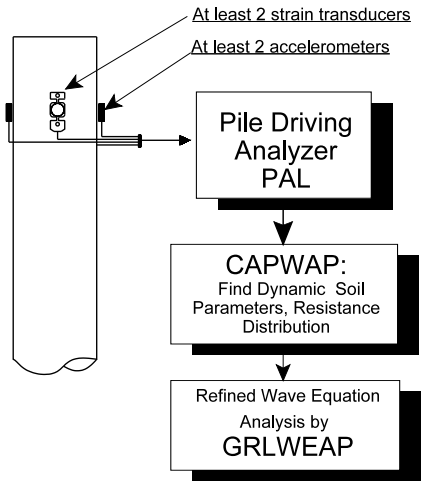
The Pile Integrity Tester™ (PIT) can be used to evaluate defects in concrete piles or shafts which may have occurred during driving or casting. Also timber piles of limited length can be tested in that manner. This so-called "Low Strain Method" or “Pulse-Echo Method” of integrity testing requires only the measurement of acceleration at the pile top. The stress wave producing impact is then generated by a small hand-held hammer and the records interpreted in the time domain. PIT also supports the so-called “Transient Response Method” which requires the additional measurement of the hammer force and an analysis in the frequency domain. This method may also be used to evaluate the unknown length of deep foundations under existing structures.

ANALYTICAL SOLUTIONS
BEARING CAPACITY

Wave Equation

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements and therefore can be performed during the design stage of a project, for example for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (see schematic below) is often performed by inputting the PDA and CAPWAP calculated parameters. Then the bearing graph from the RWEA is the basis for a safe and sufficient driving criteria.



Case Method

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force F(t) and pile top velocity v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \quad (1)$$

where

t = a point in time after impact
t_2 = time t + 2L/c
L = pile length below gages
c = $(E/\rho)^{1/2}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile ($\rho\ c^2$)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \quad (2)$$

The dynamic component may be computed from a soil damping factor, J, and a pile toe velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \quad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 can be evaluated. Most commonly, t_2 is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2

therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed.

The static resistance calculated by Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDAPLOT program.

CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffnesses. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters.

STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance R(t) minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = ½[F(t) - Zv(t)] \quad (4)$$

$$W_d = ½[F(t) + Zv(t)] \quad (5)$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

PILE INTEGRITY

High Strain Tests (PDA)

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{(E\rho)}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The

magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β_i (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta_i = (1 - \alpha_i)/(1 + \alpha_i) \quad (6)$$

with

$$\alpha_i = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \quad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the reflected wave. It is caused by resistance.

W_{UD} is the upwards traveling wave due to the damage reflection.

W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

Low Strain Tests (PIT)

The pile top is struck with a held hand hammer and the resulting pile top velocity is measured, displayed and interpreted for signs of wave reflections. In general, a comparison of the reflected acceleration leads to a relative measure of extent of damage, again the location of the problem is indicated by the arrival time of the reflection. PIT records can also be interpreted by the β-Method. However, low strain tests do not activate much resistance which simplifies Eq. 7 since W_{UR} is then equal to zero.

For drilled shafts and PIT records that clearly show a toe reflection, an approximate shaft profile can be calculated from low strain records using the PITSTOP program's PROFILE routine.

HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = {}_o\int^t F(t)v(t)\,dt \quad (8a)$$

The maximum of the E(t) curve is the most important information for an overall evaluation of the performance of a hammer and driving system. This **EMX** value allows for a classification of the hammer's performance when presented as the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency

$$e_T = EMX/E_R \quad (8b)$$

where

E_R is the manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^{\,2} - h_L \quad (9)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

DETERMINATION OF WAVE SPEED

An important facet of dynamic pile testing is an assessment of pile material properties. Since in general force is determined from strain by multiplication with elastic modulus, E, and cross sectional area, A, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \quad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \qquad (11)$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c is slower than that at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

Proportionality

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \qquad (12a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \qquad (12b)$$

or strain

$$\epsilon = v / c \qquad (12c)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

Measurements are always taken at opposite sides of the pile as a means of calculating the average force and velocity in the pile. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. The averaging of the two strain signals does then not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile

diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

LIMITATIONS, ADDITIONAL CONSIDERATIONS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

(A) Soil setup

Because excess positive pore pressures often develop during pile driving in fine grained soil (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze.

(*B) Relaxation*

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. Pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically (with particular emphasis than on the first few blows). Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. Again, restrike tests should be used, with great emphasis on early blows.

Capacity results for open pile profiles

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90% of yield strength for steel piles

85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression

100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Wave equation analysis results

The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

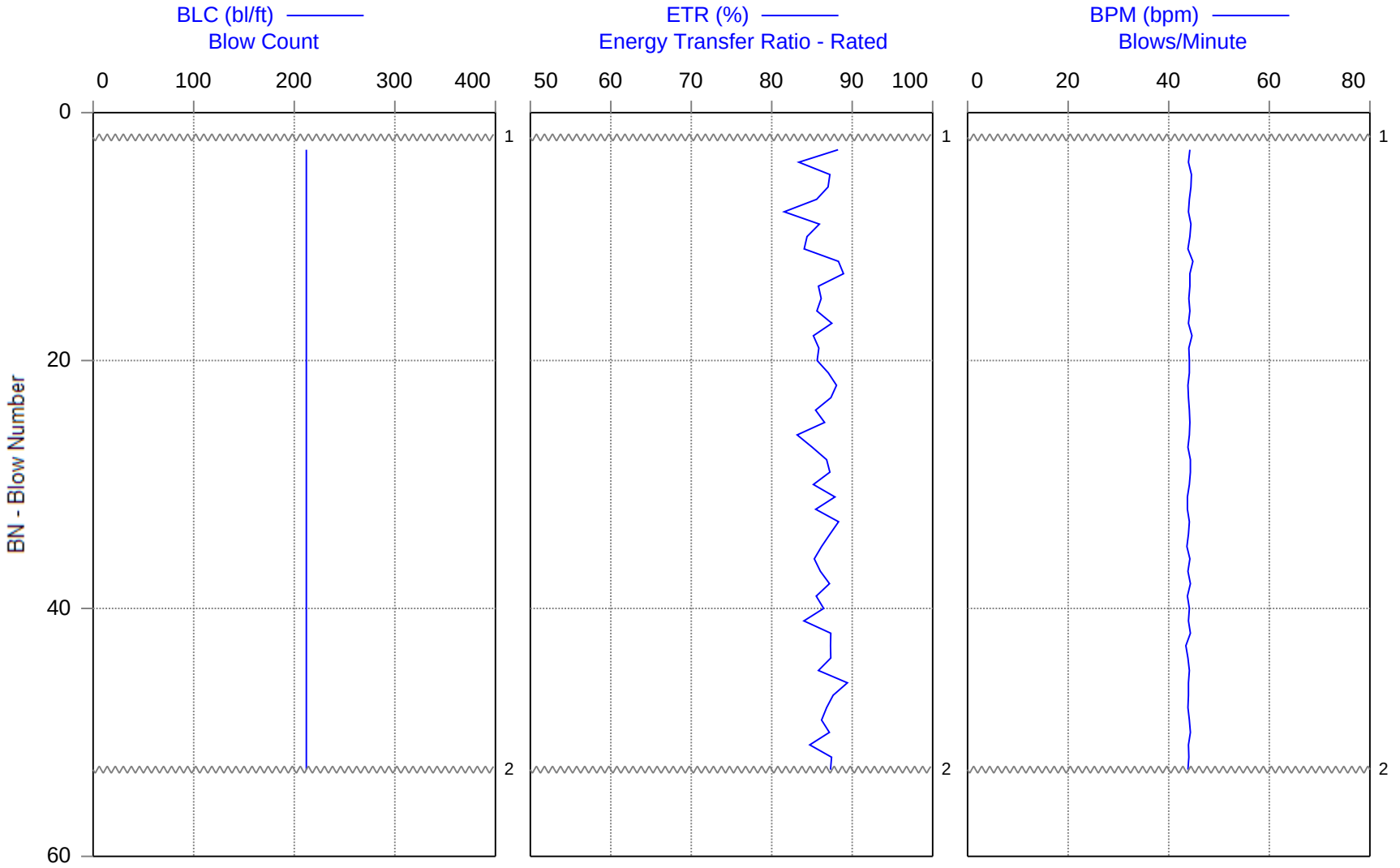
Appendix B

Summary of Case Method Results

Pile Dynamics, Inc. - PDIPLOT2 Ver 2017.2.58.5 - Case Method & iCAP® Results

Printed: 14-November-2018 Test started: 14-November-2018

HOLT DRILLING, RIG 215POR UNDERWOOD - 20 FT SAMPLE



1 - Start of test on 11/14/2018 at 10:40 AM

2 - End of test on 11/14/2018 at 10:41 AM

HOLT DRILLING, RIG 215POR UNDERWOOD - 20 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

AR: 1.43 in² SP: 0.492 k/ft³
LE: 26.00 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.35

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated
BPM: Blows/Minute
CSX: Compression Stress Maximum
CSI: Compression Stress Maximum - Individual Sensor
FMX: Maximum Force
VMX: Maximum Velocity
RAT: Length Ratio for SPT
DMX: Maximum Displacement

BL#	Depth ft	BLC bl/ft	EFV k-ft	ETR (%)	BPM bpm	CSX ksi	CSI ksi	FMX kips	VMX f/s	RAT	DMX in
3	25.01	212	308.8	88.2	44.2	27.8	28.4	40	14.3	1.0	0.31
4	25.02	212	291.8	83.4	43.9	27.4	27.7	39	13.3	1.0	0.30
5	25.02	212	305.3	87.2	44.5	29.0	30.0	41	13.9	1.0	0.28
6	25.03	212	304.5	87.0	44.4	28.5	29.6	41	13.9	1.0	0.29
7	25.03	212	299.5	85.6	44.1	28.6	29.7	41	14.3	1.0	0.25
8	25.04	212	285.4	81.5	43.9	27.1	27.8	39	13.2	1.0	0.24
9	25.04	212	300.7	85.9	44.4	27.3	27.8	39	13.9	1.0	0.24
10	25.05	212	295.3	84.4	44.2	27.2	28.0	39	13.7	1.0	0.24
11	25.05	212	294.1	84.0	43.8	27.8	28.5	40	13.4	1.0	0.23
12	25.06	212	309.0	88.3	44.8	28.7	29.7	41	14.0	1.0	0.24
13	25.06	212	311.2	88.9	44.2	27.7	28.3	40	14.2	1.0	0.24
14	25.07	212	300.3	85.8	44.2	28.1	28.9	40	13.4	1.0	0.24
15	25.07	212	301.5	86.1	44.0	28.4	28.8	41	13.5	1.0	0.23
16	25.08	212	299.6	85.6	44.2	28.3	28.6	41	13.5	1.0	0.23
17	25.08	212	306.1	87.5	43.9	29.1	30.3	42	13.7	1.0	0.24
18	25.08	212	298.1	85.2	44.6	28.5	28.9	41	13.4	1.0	0.23
19	25.09	212	300.5	85.8	44.0	28.0	28.4	40	13.6	1.0	0.24
20	25.09	212	299.8	85.6	44.1	28.4	29.1	41	13.6	1.0	0.23
21	25.10	212	304.6	87.0	44.1	27.9	29.2	40	13.5	1.0	0.23
22	25.10	212	308.1	88.0	43.8	28.9	29.3	41	13.6	1.0	0.24
23	25.11	212	305.7	87.4	43.9	28.9	29.4	41	13.5	1.0	0.23
24	25.11	212	299.1	85.4	44.1	28.3	29.1	41	13.3	1.0	0.23
25	25.12	212	302.9	86.6	44.2	28.7	29.4	41	13.4	1.0	0.24
26	25.12	212	291.1	83.2	44.1	27.9	28.5	40	13.4	1.1	0.24
27	25.13	212	297.6	85.0	43.8	28.0	28.7	40	13.4	1.0	0.23
28	25.13	212	303.9	86.8	44.3	28.1	28.9	40	13.5	1.0	0.23
29	25.14	212	305.2	87.2	44.3	27.8	28.7	40	13.5	1.0	0.23
30	25.14	212	298.1	85.2	44.1	28.1	28.9	40	13.5	1.0	0.22
31	25.15	212	307.5	87.8	43.7	28.3	29.1	41	13.7	1.0	0.23
32	25.15	212	299.1	85.5	43.7	27.6	27.8	39	13.5	1.0	0.22
33	25.16	212	309.0	88.3	44.1	28.8	29.1	41	13.6	1.0	0.23
34	25.16	212	305.4	87.2	43.9	27.5	27.9	39	13.6	1.0	0.22
35	25.17	212	301.7	86.2	43.6	28.8	29.7	41	13.4	1.0	0.23
36	25.17	212	298.5	85.3	44.2	28.5	29.3	41	13.3	1.0	0.22
37	25.17	212	301.1	86.0	43.8	27.9	28.3	40	13.6	1.0	0.23
38	25.18	212	305.1	87.2	44.3	27.7	28.7	40	13.4	1.0	0.22
39	25.18	212	299.4	85.5	43.7	28.4	29.0	41	13.4	1.0	0.23
40	25.19	212	302.5	86.4	44.1	27.2	27.4	39	13.4	1.0	0.21
41	25.19	212	294.0	84.0	43.9	28.2	29.2	40	13.2	1.0	0.23
42	25.20	212	305.7	87.3	44.3	28.0	29.0	40	13.4	1.0	0.22
43	25.20	212	305.5	87.3	43.4	28.9	29.1	41	13.4	1.0	0.23
44	25.21	212	305.7	87.3	43.8	28.4	29.5	41	13.3	1.0	0.22
45	25.21	212	300.3	85.8	44.1	27.8	29.0	40	13.4	1.0	0.23
46	25.22	212	312.9	89.4	43.9	28.8	28.9	41	13.4	1.0	0.22
47	25.22	212	306.7	87.6	43.9	28.1	29.2	40	13.6	1.0	0.24

HOLT DRILLING, RIG 215POR UNDERWOOD - 20 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

BL#	Depth	BLC	EFV	ETR	BPM	CSX	CSI	FMX	VMX	RAT	DMX
	ft	bl/ft	k-ft	(%)	bpm	ksi	ksi	kips	f/s		in
48	25.23	212	303.9	86.8	43.8	28.1	29.2	40	13.3	1.0	0.22
49	25.23	212	301.7	86.2	44.1	27.8	28.5	40	13.4	1.0	0.23
50	25.24	212	305.0	87.1	44.3	27.6	28.7	40	13.5	1.0	0.22
51	25.24	212	296.5	84.7	43.9	26.8	27.1	38	13.5	1.0	0.22
52	25.25	212	306.0	87.4	44.0	27.6	28.1	40	13.5	1.0	0.22
53	25.25	212	305.5	87.3	43.8	28.0	29.0	40	13.4	1.0	0.22
		Average	302.1	86.3	44.0	28.1	28.8	40	13.5	1.0	0.24
		Std. Dev.	5.3	1.5	0.3	0.5	0.7	1	0.2	0.0	0.02
		Maximum	312.9	89.4	44.8	29.1	30.3	42	14.3	1.1	0.31
		Minimum	285.4	81.5	43.4	26.8	27.1	38	13.2	1.0	0.21

Total number of blows analyzed: 51

BL# Sensors

3-53 F1: [62NWJ-1] 215.0 (1.00); F2: [62NWJ-2] 216.9 (1.00); A1: [K2449] 340.0 (1.00); A2: [K3258] 337.0 (1.00)

BL# Comments

2 Start of test on 11/14/2018 at 10:40 AM
53 End of test on 11/14/2018 at 10:41 AM

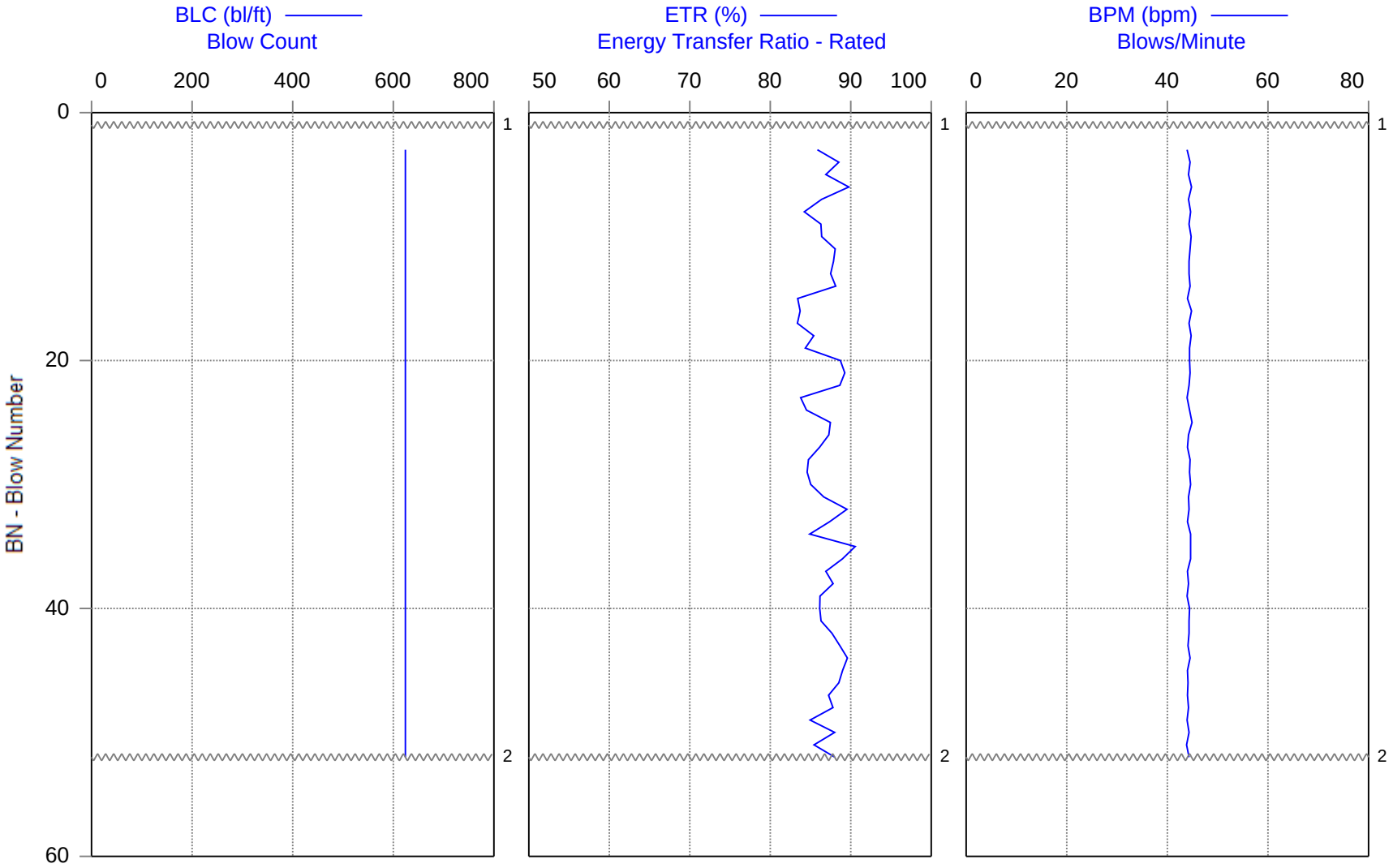
Time Summary

Drive 1 minute 9 seconds 10:40 AM - 10:41 AM BN 2 - 53

Pile Dynamics, Inc. - PDIPLOT2 Ver 2017.2.58.5 - Case Method & iCAP® Results

Printed: 14-November-2018 Test started: 14-November-2018

HOLT DRILLING, RIG 215 POR UNDERWOOD - 25 FT SAMPLE



1 - Start of test on 11/14/2018 at 11:09 AM

2 - End of test on 11/14/2018 at 11:10 AM

HOLT DRILLING, RIG 215 POR UNDERWOOD - 25 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

AR: 1.43 in² SP: 0.492 k/ft³
LE: 30.00 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.35

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated
BPM: Blows/Minute
CSX: Compression Stress Maximum
CSI: Compression Stress Maximum - Individual Sensor
FMX: Maximum Force
VMX: Maximum Velocity
RAT: Length Ratio for SPT
DMX: Maximum Displacement

BL#	Depth ft	BLC bl/ft	EFV k-ft	ETR (%)	BPM bpm	CSX ksi	CSI ksi	FMX kips	VMX f/s	RAT	DMX in
3	25.00	624	300.5	85.8	43.9	29.3	30.4	42	12.7	1.1	0.25
4	25.01	624	309.8	88.5	44.5	29.9	31.7	43	12.9	1.1	0.28
5	25.01	624	304.2	86.9	44.2	30.4	31.6	43	13.0	1.1	0.27
6	25.01	624	314.1	89.8	44.8	30.5	31.8	44	13.1	1.1	0.25
7	25.01	624	302.4	86.4	44.2	29.6	31.3	42	12.8	1.1	0.24
8	25.01	624	294.8	84.2	44.6	29.3	30.5	42	13.1	1.1	0.23
9	25.01	624	302.0	86.3	44.3	30.0	31.2	43	12.7	1.1	0.22
10	25.02	624	302.4	86.4	44.7	30.4	31.3	44	13.1	1.1	0.21
11	25.02	624	308.2	88.1	44.5	29.8	30.8	43	13.4	1.1	0.21
12	25.02	624	307.5	87.9	44.3	30.1	31.4	43	13.6	1.1	0.21
13	25.02	624	306.3	87.5	44.3	29.9	30.6	43	12.6	1.1	0.21
14	25.02	624	308.4	88.1	44.5	29.5	30.3	42	12.9	1.1	0.21
15	25.02	624	291.9	83.4	44.0	28.6	29.9	41	12.9	1.1	0.20
16	25.03	624	293.0	83.7	44.8	30.2	30.7	43	11.8	1.1	0.20
17	25.03	624	291.8	83.4	44.3	28.8	29.3	41	12.0	1.6	0.20
18	25.03	624	298.9	85.4	44.7	30.6	30.6	44	11.8	1.1	0.20
19	25.03	624	295.2	84.3	44.4	30.4	31.0	43	12.4	1.1	0.20
20	25.03	624	310.5	88.7	44.4	30.7	32.7	44	12.8	1.1	0.21
21	25.03	624	312.4	89.3	44.5	31.3	32.6	45	12.8	1.6	0.21
22	25.04	624	310.2	88.6	44.3	30.9	32.4	44	12.9	1.6	0.21
23	25.04	624	293.2	83.8	43.9	30.8	32.0	44	12.5	1.1	0.20
24	25.04	624	295.7	84.5	44.4	30.6	31.4	44	12.4	1.1	0.20
25	25.04	624	306.1	87.5	44.9	31.4	32.6	45	12.7	1.1	0.20
26	25.04	624	305.4	87.3	44.2	30.8	31.6	44	12.6	1.6	0.21
27	25.04	624	301.4	86.1	44.0	31.8	32.2	45	13.1	1.6	0.20
28	25.04	624	296.6	84.7	44.5	30.6	31.4	44	12.6	1.6	0.20
29	25.05	624	296.0	84.6	44.4	30.5	31.2	44	12.5	1.1	0.20
30	25.05	624	297.6	85.0	44.6	30.4	31.6	44	12.5	1.1	0.20
31	25.05	624	303.2	86.6	44.2	31.0	32.2	44	12.6	1.1	0.20
32	25.05	624	313.4	89.5	44.3	31.0	32.9	44	12.8	1.6	0.21
33	25.05	624	305.8	87.4	44.0	30.7	32.2	44	12.7	1.1	0.21
34	25.05	624	297.1	84.9	44.6	30.8	31.8	44	12.8	1.1	0.20
35	25.06	624	316.9	90.6	44.6	31.1	33.2	44	12.8	1.6	0.21
36	25.06	624	311.4	89.0	44.6	31.1	32.2	44	12.6	1.6	0.21
37	25.06	624	304.1	86.9	44.0	31.5	32.8	45	12.8	1.1	0.20
38	25.06	624	307.3	87.8	44.2	31.4	33.1	45	12.8	1.6	0.21
39	25.06	624	301.6	86.2	43.9	31.4	32.9	45	12.8	1.1	0.21
40	25.06	624	301.5	86.1	44.4	31.0	32.5	44	12.9	1.1	0.21
41	25.07	624	302.0	86.3	44.3	31.3	32.8	45	12.9	1.1	0.21
42	25.07	624	306.8	87.7	44.3	31.5	32.8	45	12.7	1.1	0.21
43	25.07	624	310.3	88.6	44.1	31.1	32.6	44	12.9	1.1	0.21
44	25.07	624	313.5	89.6	44.5	31.8	33.5	45	12.7	1.1	0.21
45	25.07	624	311.4	89.0	44.0	31.4	33.4	45	12.7	1.1	0.21
46	25.07	624	309.8	88.5	44.1	31.5	32.7	45	12.7	1.6	0.21
47	25.08	624	305.3	87.2	44.0	30.9	32.4	44	12.6	1.1	0.21

HOLT DRILLING, RIG 215 POR UNDERWOOD - 25 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

BL#	Depth	BLC	EFV	ETR	BPM	CSX	CSI	FMX	VMX	RAT	DMX
	ft	bl/ft	k-ft	(%)	bpm	ksi	ksi	kips	f/s		in
48	25.08	624	307.2	87.8	44.2	31.0	32.4	44	12.7	1.1	0.21
49	25.08	624	297.3	84.9	43.9	30.9	32.6	44	12.9	1.1	0.20
50	25.08	624	308.0	88.0	44.3	31.4	32.8	45	12.7	1.1	0.21
51	25.08	624	299.0	85.4	43.8	31.1	33.1	44	12.7	1.1	0.20
52	25.08	624	308.0	88.0	44.3	30.6	31.7	44	12.5	1.1	0.21
		Average	303.9	86.8	44.3	30.7	31.9	44	12.7	1.2	0.21
		Std. Dev.	6.4	1.8	0.3	0.7	1.0	1	0.3	0.2	0.02
		Maximum	316.9	90.6	44.9	31.8	33.5	45	13.6	1.6	0.28
		Minimum	291.8	83.4	43.8	28.6	29.3	41	11.8	1.1	0.20

Total number of blows analyzed: 50

BL# Sensors

3-52 F1: [62NWJ-1] 215.0 (1.00); F2: [62NWJ-2] 216.9 (1.00); A1: [K2449] 340.0 (1.00); A2: [K3258] 337.0 (1.00)

BL# Comments

1 Start of test on 11/14/2018 at 11:09 AM
52 End of test on 11/14/2018 at 11:10 AM

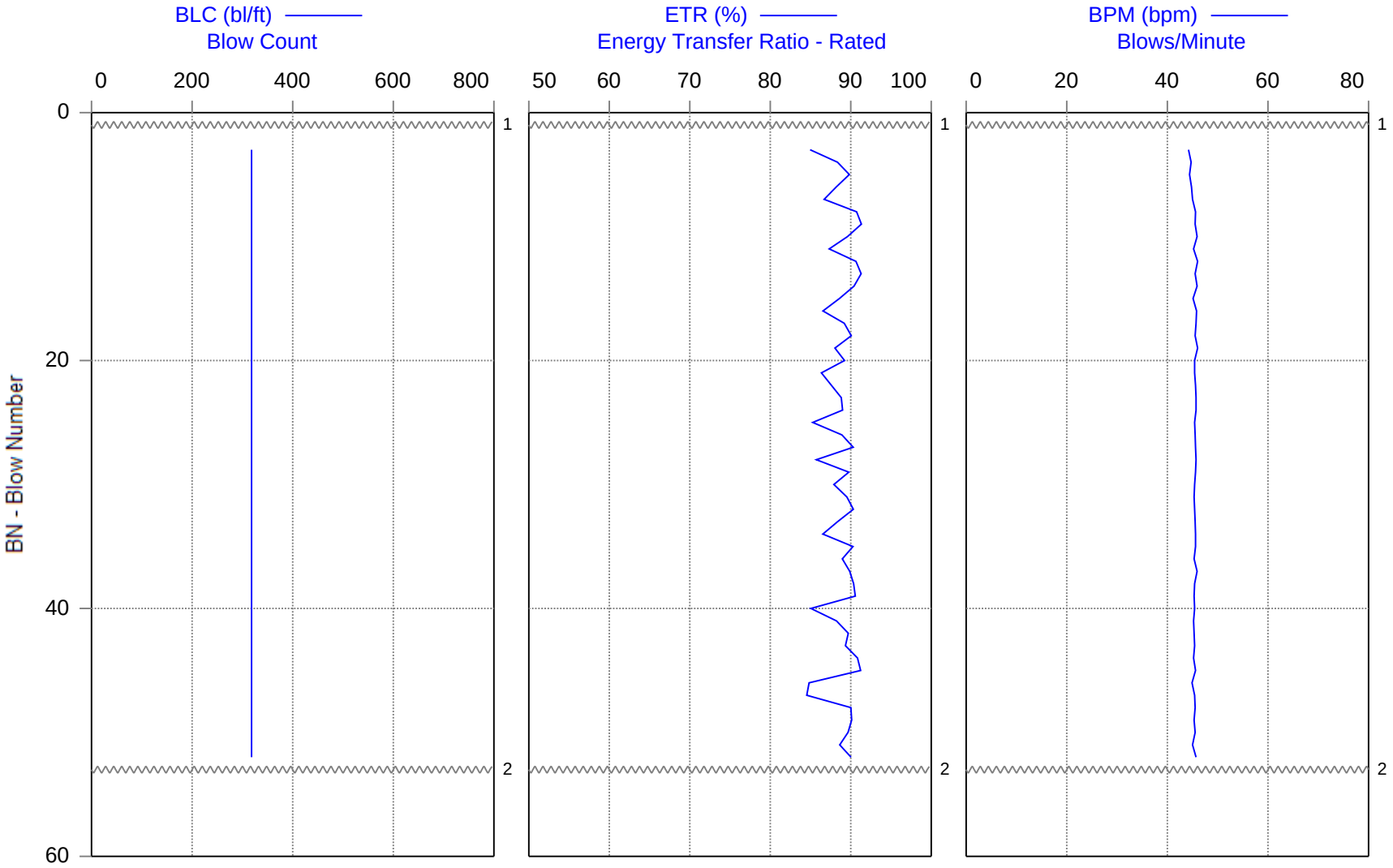
Time Summary

Drive 1 minute 7 seconds 11:09 AM - 11:10 AM BN 1 - 52

Pile Dynamics, Inc. - PDIPLOT2 Ver 2017.2.58.5 - Case Method & iCAP® Results

Printed: 14-November-2018 Test started: 14-November-2018

HOLT DRILLING, RIG 215 POR UNDERWOOD - 30 FT SAMPLE



1 - Start of test on 11/14/2018 at 11:39 AM

2 - End of test on 11/14/2018 at 11:40 AM

HOLT DRILLING, RIG 215 POR UNDERWOOD - 30 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

AR: 1.43 in² SP: 0.492 k/ft³
LE: 35.00 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.35

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated
BPM: Blows/Minute
CSX: Compression Stress Maximum
CSI: Compression Stress Maximum - Individual Sensor
FMX: Maximum Force
VMX: Maximum Velocity
RAT: Length Ratio for SPT
DMX: Maximum Displacement

BL#	Depth ft	BLC bl/ft	EFV k-ft	ETR (%)	BPM bpm	CSX ksi	CSI ksi	FMX kips	VMX f/s	RAT	DMX in
3	30.01	318	297.3	84.9	44.2	28.6	30.1	41	12.7	1.1	0.26
4	30.01	318	309.2	88.4	44.7	30.3	32.4	43	13.0	1.1	0.28
5	30.02	318	314.3	89.8	44.4	29.7	31.1	42	13.2	1.1	0.32
6	30.02	318	308.7	88.2	44.8	29.6	31.1	42	13.0	0.7	0.29
7	30.02	318	303.4	86.7	45.0	29.6	31.5	42	12.9	1.1	0.26
8	30.03	318	317.5	90.7	45.6	30.1	31.6	43	13.1	0.7	0.26
9	30.03	318	319.6	91.3	45.5	30.5	32.5	44	13.2	1.1	0.27
10	30.03	318	313.5	89.6	45.9	30.1	31.9	43	13.3	1.1	0.26
11	30.03	318	305.6	87.3	45.2	29.2	30.7	42	13.4	1.1	0.26
12	30.04	318	317.2	90.6	46.0	30.5	32.3	44	13.1	1.1	0.26
13	30.04	318	319.5	91.3	45.5	30.0	31.8	43	13.4	1.1	0.26
14	30.04	318	316.4	90.4	45.9	30.2	31.8	43	13.2	0.7	0.24
15	30.05	318	310.1	88.6	45.1	28.7	29.5	41	13.0	1.1	0.22
16	30.05	318	302.9	86.5	45.8	28.9	30.9	41	12.8	1.1	0.25
17	30.05	318	312.2	89.2	45.7	29.4	31.2	42	13.2	1.1	0.23
18	30.06	318	315.2	90.1	45.5	29.8	32.0	43	13.2	1.1	0.28
19	30.06	318	308.1	88.0	46.0	28.8	30.0	41	12.8	1.1	0.29
20	30.06	318	312.2	89.2	45.4	29.6	31.6	42	13.1	1.1	0.25
21	30.07	318	302.2	86.3	45.4	28.8	30.7	41	12.6	1.1	0.22
22	30.07	318	306.6	87.6	45.6	28.4	29.8	41	12.5	1.1	0.33
23	30.07	318	310.9	88.8	45.7	30.0	32.2	43	13.0	1.1	0.24
24	30.08	318	311.4	89.0	45.7	29.6	32.1	42	13.0	1.1	0.23
25	30.08	318	298.4	85.3	45.4	28.3	30.3	40	12.3	1.1	0.22
26	30.08	318	311.2	88.9	45.5	29.4	31.7	42	13.1	1.1	0.23
27	30.08	318	316.1	90.3	45.6	29.6	31.8	42	13.3	1.1	0.22
28	30.09	318	300.0	85.7	45.7	28.3	29.3	40	11.9	1.1	0.23
29	30.09	318	314.2	89.8	45.6	29.4	31.7	42	13.1	1.1	0.23
30	30.09	318	307.7	87.9	45.4	28.7	29.9	41	12.1	1.1	0.22
31	30.10	318	313.3	89.5	45.3	29.3	31.4	42	13.1	1.1	0.24
32	30.10	318	316.1	90.3	45.4	29.8	32.0	43	13.2	1.1	0.30
33	30.10	318	309.2	88.3	45.5	29.5	32.2	42	13.0	1.1	0.24
34	30.11	318	302.8	86.5	45.6	28.3	30.3	40	12.0	1.1	0.28
35	30.11	318	315.9	90.3	45.6	29.6	31.9	42	12.9	1.1	0.27
36	30.11	318	311.3	88.9	45.3	29.3	31.9	42	12.8	1.1	0.25
37	30.12	318	314.5	89.9	45.9	29.1	31.5	42	13.0	1.1	0.27
38	30.12	318	316.2	90.4	45.4	29.7	31.9	42	13.0	1.1	0.28
39	30.12	318	317.0	90.6	45.3	29.6	31.6	42	13.1	1.1	0.31
40	30.13	318	297.6	85.0	45.4	28.6	29.5	41	11.7	1.1	0.25
41	30.13	318	308.8	88.2	45.2	28.7	30.7	41	12.5	1.1	0.24
42	30.13	318	313.9	89.7	45.3	29.0	31.4	41	13.2	1.1	0.32
43	30.14	318	312.7	89.3	45.4	29.1	31.3	42	13.3	1.1	0.27
44	30.14	318	317.9	90.8	45.2	29.1	31.5	42	13.6	1.1	0.34
45	30.14	318	319.3	91.2	45.6	29.5	31.4	42	13.6	1.1	0.30
46	30.14	318	296.9	84.8	44.9	27.9	29.9	40	13.1	1.0	0.23
47	30.15	318	295.9	84.5	45.4	27.8	29.6	40	12.6	1.1	0.23

HOLT DRILLING, RIG 215 POR UNDERWOOD - 30 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

BL#	Depth	BLC	EFV	ETR	BPM	CSX	CSI	FMX	VMX	RAT	DMX
	ft	bl/ft	k-ft	(%)	bpm	ksi	ksi	kips	f/s		in
48	30.15	318	315.0	90.0	45.5	29.2	30.9	42	13.3	1.1	0.30
49	30.15	318	315.4	90.1	45.3	29.2	31.3	42	13.5	1.1	0.29
50	30.16	318	313.8	89.7	45.5	29.0	31.0	41	13.2	1.1	0.26
51	30.16	318	310.2	88.6	45.0	28.8	31.0	41	13.7	1.0	0.26
52	30.16	318	315.1	90.0	45.7	29.1	31.1	42	13.7	1.1	0.27
		Average	310.6	88.7	45.4	29.3	31.2	42	13.0	1.0	0.26
		Std. Dev.	6.4	1.8	0.4	0.6	0.8	1	0.4	0.1	0.03
		Maximum	319.6	91.3	46.0	30.5	32.5	44	13.7	1.1	0.34
		Minimum	295.9	84.5	44.2	27.8	29.3	40	11.7	0.7	0.22

Total number of blows analyzed: 50

BL# Sensors

3-52 F1: [62NWJ-1] 215.0 (1.00); F2: [62NWJ-2] 216.9 (1.00); A1: [K2449] 340.0 (1.00); A2: [K3258] 337.0 (1.00)

BL# Comments

1 Start of test on 11/14/2018 at 11:39 AM
53 End of test on 11/14/2018 at 11:40 AM

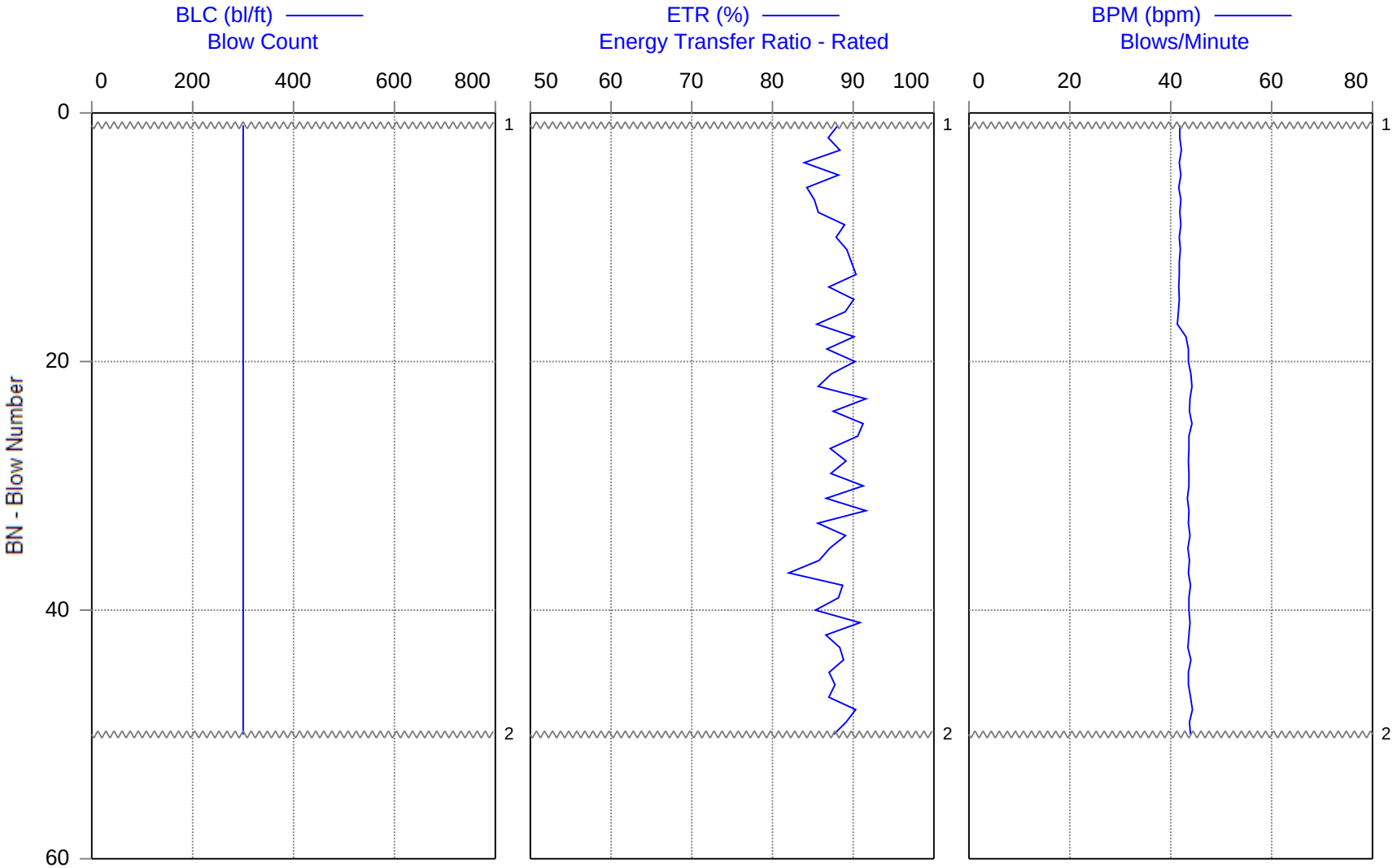
Time Summary

Drive 1 minute 7 seconds 11:39 AM - 11:40 AM BN 1 - 53

Pile Dynamics, Inc. - PDIPLOT2 Ver 2017.2.58.5 - Case Method & iCAP® Results

Printed: 14-November-2018 Test started: 14-November-2018

HOLT DRILLING, RIG 215 POR UNDERWOOD - 35 FT SAMPLE



1 - Start of test on 11/14/2018 at 12:04 PM

2 - End of test on 11/14/2018 at 12:05 PM

HOLT DRILLING, RIG 215 POR UNDERWOOD - 35 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

AR: 1.43 in² SP: 0.492 k/ft³
LE: 40.00 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.35

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated
BPM: Blows/Minute
CSX: Compression Stress Maximum
CSI: Compression Stress Maximum - Individual Sensor
FMX: Maximum Force
VMX: Maximum Velocity
RAT: Length Ratio for SPT
DMX: Maximum Displacement

BL#	Depth	BLC	EFV	ETR	BPM	CSX	CSI	FMX	VMX	RAT	DMX
	ft	bl/ft	k-ft	(%)	bpm	ksi	ksi	kips	f/s		in
1	50.00	300	308.4	88.1	41.8	29.8	30.8	43	12.1	0.7	0.34
2	50.01	300	304.2	86.9	41.8	29.7	30.5	42	12.1	0.6	0.36
3	50.01	300	309.2	88.4	42.1	30.5	31.0	44	12.2	0.6	0.42
4	50.01	300	293.8	83.9	41.7	30.0	30.4	43	12.1	0.7	0.23
5	50.02	300	308.6	88.2	42.0	29.9	30.9	43	12.1	0.6	0.40
6	50.02	300	294.9	84.2	41.6	30.5	30.8	44	12.1	0.7	0.24
7	50.02	300	298.1	85.2	42.0	31.0	31.4	44	12.2	0.7	0.25
8	50.03	300	299.9	85.7	41.8	30.8	31.5	44	12.2	0.7	0.25
9	50.03	300	311.3	88.9	42.0	30.4	31.0	44	12.4	0.7	0.26
10	50.03	300	307.6	87.9	41.7	30.1	30.3	43	11.7	0.7	0.31
11	50.04	300	312.2	89.2	41.9	31.3	32.1	45	12.1	0.6	0.34
12	50.04	300	314.2	89.8	41.7	30.8	31.7	44	12.4	0.6	0.31
13	50.04	300	316.2	90.4	41.7	31.2	32.1	45	12.5	0.6	0.30
14	50.05	300	304.4	87.0	41.6	30.5	31.3	44	12.2	0.6	0.29
15	50.05	300	315.3	90.1	41.7	31.0	31.6	44	12.4	0.6	0.27
16	50.05	300	311.5	89.0	41.5	30.7	31.4	44	12.4	0.6	0.25
17	50.06	300	299.2	85.5	41.3	31.2	31.4	45	12.7	0.7	0.21
18	50.06	300	315.3	90.1	43.0	32.1	32.5	46	12.4	0.6	0.32
19	50.06	300	303.6	86.7	43.5	31.3	31.9	45	12.1	0.6	0.34
20	50.07	300	315.7	90.2	43.5	31.3	32.5	45	12.5	0.6	0.33
21	50.07	300	305.5	87.3	44.0	31.1	32.7	44	12.1	0.6	0.29
22	50.07	300	299.8	85.7	44.2	30.7	32.0	44	12.0	0.8	0.22
23	50.08	300	320.5	91.6	43.8	30.7	32.5	44	12.5	0.5	0.41
24	50.08	300	306.4	87.5	43.7	31.3	32.9	45	12.3	0.6	0.35
25	50.08	300	319.3	91.2	44.2	30.5	30.9	44	12.5	0.5	0.39
26	50.09	300	316.9	90.6	43.6	30.4	31.4	44	12.9	0.7	0.37
27	50.09	300	305.0	87.2	43.6	31.5	32.6	45	12.2	0.6	0.30
28	50.09	300	311.9	89.1	43.5	31.8	32.4	45	12.4	0.6	0.37
29	50.10	300	305.3	87.2	43.6	32.0	33.3	46	12.1	0.6	0.29
30	50.10	300	319.3	91.2	43.6	32.4	33.3	46	12.7	0.6	0.29
31	50.10	300	303.3	86.7	43.3	32.6	34.1	47	12.2	0.7	0.22
32	50.11	300	320.5	91.6	43.6	31.6	32.7	45	12.5	0.6	0.23
33	50.11	300	299.6	85.6	43.5	32.4	34.2	46	12.0	0.7	0.22
34	50.11	300	311.6	89.0	43.8	31.8	33.0	45	12.5	0.6	0.22
35	50.12	300	304.9	87.1	43.4	32.2	33.5	46	12.3	0.7	0.27
36	50.12	300	300.2	85.8	43.7	31.9	33.5	46	12.1	0.7	0.22
37	50.12	300	287.0	82.0	43.5	31.9	34.2	46	11.3	0.8	0.21
38	50.13	300	310.4	88.7	43.9	31.1	31.9	44	12.2	0.7	0.27
39	50.13	300	308.6	88.2	43.6	31.2	32.1	45	12.2	0.6	0.28
40	50.13	300	298.6	85.3	43.6	32.0	33.5	46	11.6	0.7	0.21
41	50.14	300	317.9	90.8	43.8	32.5	34.0	47	12.4	0.6	0.24
42	50.14	300	303.1	86.6	43.6	32.4	34.2	46	11.1	0.8	0.22
43	50.14	300	309.2	88.3	43.4	31.4	33.2	45	12.5	0.7	0.28
44	50.15	300	310.8	88.8	44.0	31.9	34.8	46	12.3	0.7	0.30
45	50.15	300	304.5	87.0	43.5	31.0	32.7	44	12.0	0.7	0.38

HOLT DRILLING, RIG 215 POR UNDERWOOD - 35 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

BL#	Depth	BLC	EFV	ETR	BPM	CSX	CSI	FMX	VMX	RAT	DMX
	ft	bl/ft	k-ft	(%)	bpm	ksi	ksi	kips	f/s		in
46	50.15	300	307.1	87.7	43.5	30.7	32.5	44	12.2	0.7	0.36
47	50.16	300	304.4	87.0	43.9	32.5	35.0	46	12.3	0.7	0.25
48	50.16	300	316.1	90.3	44.3	30.8	33.3	44	12.5	0.7	0.22
49	50.16	300	311.9	89.1	43.7	31.7	32.6	45	12.4	0.7	0.24
50	50.17	300	306.5	87.6	43.9	30.6	33.0	44	12.5	0.7	0.21
		Average	307.8	87.9	43.0	31.2	32.4	45	12.2	0.7	0.29
		Std. Dev.	7.3	2.1	0.9	0.8	1.2	1	0.3	0.1	0.06
		Maximum	320.5	91.6	44.3	32.6	35.0	47	12.9	0.8	0.42
		Minimum	287.0	82.0	41.3	29.7	30.3	42	11.1	0.5	0.21

Total number of blows analyzed: 50

BL# Sensors

1-50 F1: [62NWJ-1] 215.0 (1.00); F2: [62NWJ-2] 216.9 (1.00); A1: [K2449] 340.0 (1.00); A2: [K3258] 337.0 (1.00)

BL# Comments

1 Start of test on 11/14/2018 at 12:04 PM
50 End of test on 11/14/2018 at 12:05 PM

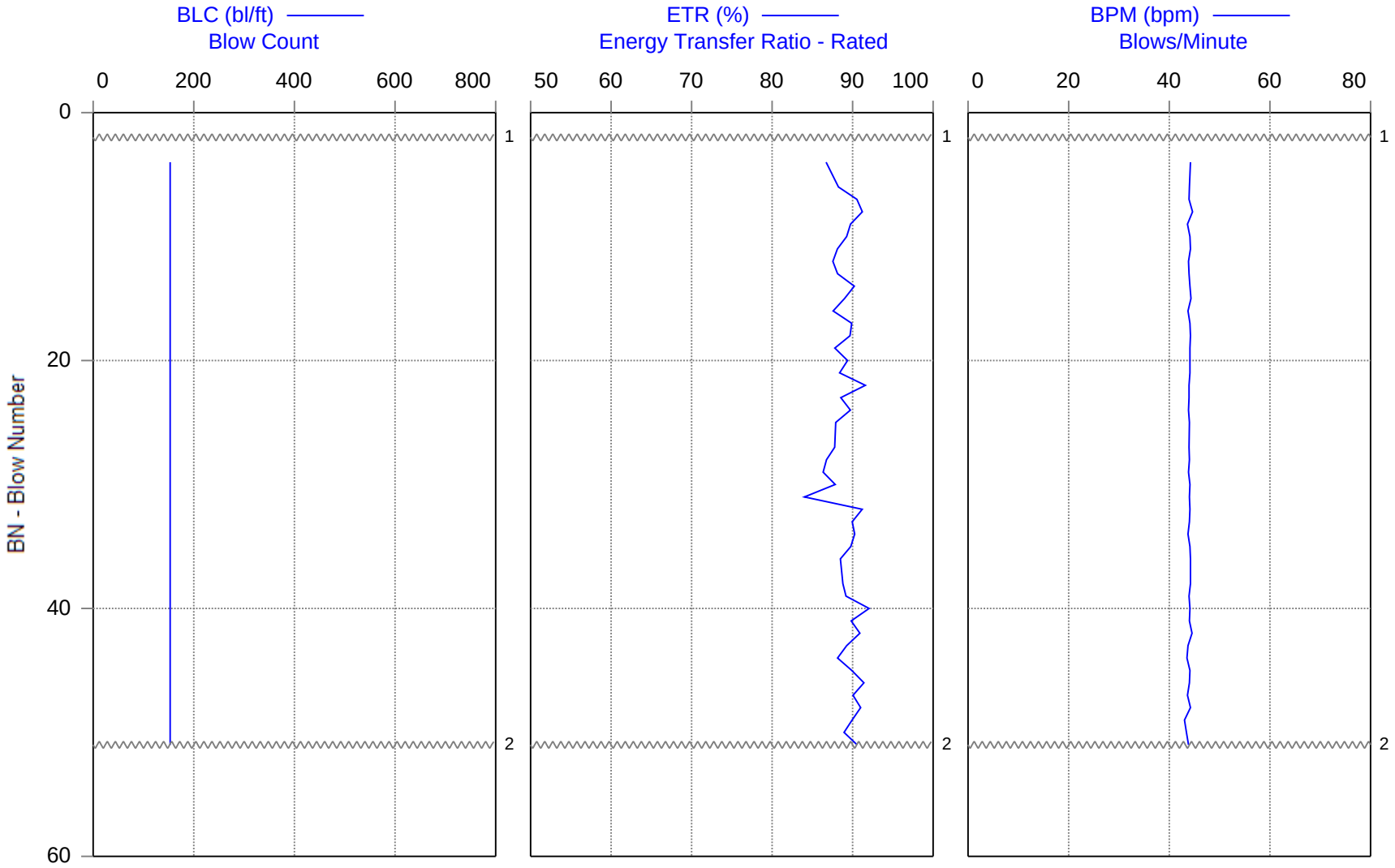
Time Summary

Drive 1 minute 8 seconds 12:04 PM - 12:05 PM BN 1 - 50

Pile Dynamics, Inc. - PDIPLOT2 Ver 2017.2.58.5 - Case Method & iCAP® Results

Printed: 14-November-2018 Test started: 14-November-2018

HOLT DRILLING, RIG 215 POR UNDERWOOD - 40 FT SAMPLE



1 - Start of test on 11/14/2018 at 12:33 PM

2 - End of test on 11/14/2018 at 12:35 PM

HOLT DRILLING, RIG 215 POR UNDERWOOD - 40 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

AR: 1.43 in² SP: 0.492 ksi
LE: 45.00 in² EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.35

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated
BPM: Blows/Minute
CSX: Compression Stress Maximum
CSI: Compression Stress Maximum - Individual Sensor
FMX: Maximum Force
VMX: Maximum Velocity
RAT: Length Ratio for SPT
DMX: Maximum Displacement

BL#	Depth	BLC	EFV	ETR	BPM	CSX	CSI	FMX	VMX	RAT	DMX
	ft	bl/ft	k-ft	(%)	bpm	ksi	ksi	kips	f/s		in
4	40.03	153	303.5	86.7	44.2	27.2	28.1	39	12.3	0.6	0.32
6	40.04	153	308.8	88.2	44.0	28.4	30.1	41	13.4	0.6	0.25
7	40.05	153	316.9	90.5	43.9	28.9	30.7	41	13.6	0.6	0.28
8	40.05	153	319.2	91.2	44.6	29.4	31.1	42	13.7	0.7	0.30
9	40.06	153	314.1	89.7	43.6	29.2	31.0	42	13.5	0.6	0.26
10	40.07	153	312.4	89.3	44.1	28.4	30.0	41	13.2	0.7	0.32
11	40.07	153	308.4	88.1	44.2	29.1	30.3	42	13.2	0.7	0.24
12	40.08	153	306.4	87.5	43.8	28.7	30.1	41	13.2	0.7	0.24
13	40.08	153	308.4	88.1	43.9	29.3	30.3	42	12.9	0.7	0.26
14	40.09	153	315.7	90.2	44.1	27.4	28.1	39	13.3	0.6	0.41
15	40.10	153	311.5	89.0	44.3	28.7	29.5	41	13.6	0.6	0.22
16	40.10	153	306.5	87.6	43.7	29.0	29.9	41	13.0	0.7	0.24
17	40.11	153	314.6	89.9	44.1	27.9	28.5	40	13.2	0.6	0.29
18	40.12	153	313.8	89.7	44.2	28.3	29.0	40	13.2	0.7	0.41
19	40.12	153	307.3	87.8	44.1	29.0	30.3	41	13.2	0.6	0.25
20	40.13	153	312.8	89.4	44.1	28.7	30.0	41	13.2	0.8	0.37
21	40.14	153	309.3	88.4	44.1	29.0	30.4	41	13.3	0.6	0.23
22	40.14	153	320.5	91.6	43.9	28.2	29.3	40	13.6	0.6	0.29
23	40.15	153	309.8	88.5	43.9	28.6	29.6	41	13.3	0.6	0.26
24	40.16	153	314.0	89.7	43.8	29.4	30.8	42	13.5	0.6	0.26
25	40.16	153	307.7	87.9	44.0	28.7	29.7	41	13.3	0.7	0.26
27	40.18	153	307.2	87.8	43.9	28.7	29.4	41	13.2	0.6	0.25
28	40.18	153	303.6	86.7	44.0	28.0	28.7	40	13.3	0.6	0.21
29	40.19	153	302.2	86.3	43.8	27.1	27.4	39	13.3	0.6	0.23
30	40.20	153	307.4	87.8	44.1	27.7	28.8	40	13.2	0.6	0.24
31	40.20	153	294.0	84.0	44.0	28.2	29.6	40	12.9	0.7	0.23
32	40.21	153	319.1	91.2	44.1	28.8	30.1	41	13.4	0.6	0.24
33	40.22	153	314.8	89.9	44.0	27.8	28.8	40	13.4	0.6	0.25
34	40.22	153	315.9	90.2	43.7	29.3	30.6	42	13.4	0.6	0.22
35	40.23	153	314.2	89.8	44.1	28.5	29.6	41	13.6	0.6	0.19
36	40.24	153	309.7	88.5	44.2	28.4	29.2	41	13.6	0.6	0.19
38	40.25	153	310.7	88.8	44.2	29.0	29.9	41	13.7	0.6	0.21
39	40.25	153	312.1	89.2	43.9	27.4	28.1	39	13.4	0.6	0.20
40	40.26	153	322.2	92.0	44.1	29.2	30.2	42	13.6	0.6	0.21
41	40.27	153	314.3	89.8	44.0	29.7	31.2	43	13.5	0.6	0.21
42	40.27	153	318.1	90.9	44.5	29.8	31.4	43	13.5	0.6	0.21
43	40.28	153	312.4	89.3	43.7	29.0	30.5	41	13.6	0.6	0.19
44	40.29	153	308.5	88.1	43.5	28.7	29.6	41	13.8	0.8	0.21
45	40.29	153	314.7	89.9	44.1	27.1	27.9	39	13.6	0.6	0.21
46	40.30	153	319.9	91.4	44.0	28.8	29.5	41	13.9	0.6	0.20
47	40.31	153	315.2	90.0	43.6	28.1	29.4	40	13.7	0.6	0.19
48	40.31	153	318.5	91.0	44.2	29.4	30.9	42	13.6	0.6	0.20
49	40.32	153	314.7	89.9	43.0	29.1	30.4	42	13.8	0.6	0.19
50	40.33	153	311.3	88.9	43.4	28.1	28.9	40	13.7	0.6	0.19
51	40.33	153	317.0	90.6	43.8	28.7	30.3	41	13.7	0.6	0.21

HOLT DRILLING, RIG 215 POR UNDERWOOD - 40 FT SAMPLE RIG 215, MOBILE B-60, 140LB, SPT
OP: RMDT Date: 14-November-2018

BL#	Depth ft	BLC bl/ft	EFV k-ft	ETR (%)	BPM bpm	CSX ksi	CSI ksi	FMX kips	VMX f/s	RAT	DMX in
		Average	312.0	89.1	44.0	28.6	29.7	41	13.4	0.7	0.25
		Std. Dev.	5.5	1.6	0.3	0.7	0.9	1	0.3	0.0	0.05
		Maximum	322.2	92.0	44.6	29.8	31.4	43	13.9	0.8	0.41
		Minimum	294.0	84.0	43.0	27.1	27.4	39	12.3	0.6	0.19

Total number of blows analyzed: 45

BL# Sensors

4-51 F1: [62NWJ-1] 215.0 (1.00); F2: [62NWJ-2] 216.9 (1.00); A1: [K2449] 340.0 (1.00); A2: [K3258] 337.0 (1.00)

BL# Comments

2 Start of test on 11/14/2018 at 12:33 PM
51 End of test on 11/14/2018 at 12:35 PM

Time Summary

Drive 1 minute 6 seconds 12:33 PM - 12:35 PM BN 2 - 51

B.3 SPT Energy Measurement Report for Rig #10 (Borings BT-4 and BT-5)



GRL engineers, inc.

Dynamic Measurements and Analyses

Job No. 226010-1

Report on:
Dynamic Energy Measurements
SPT Rigs #10 and #17
Fife and Federal Way, Washington

Prepared for Holt Services, Inc.
By Marty Bixler and Gabriela Wong

April 28, 2022

www.GRLengineers.com info@GRLengineers.com

April 28, 2022

Travis Stephens
Holt Services, Inc.
10621 Todd Road E
Puyallup, WA 98372

Re: Energy Measurement for Dynamic Penetrometers
Standard Penetration Tests (SPT) – Rigs #10 and #17
Fife and Federal Way, Washington GRL Job No. 226010-1

Dear Mr. Stephens:

This report transmits our findings from energy measurements and related data analysis conducted by GRL Engineers, Inc. (GRL) for two of your drill rigs; one operating in Fife, Washington and one operating in Federal Way, Washington. One automatic hammer and penetrometer system for each drill rig was monitored during Standard Penetration Tests. Dynamic testing summarized in this report was conducted on April 27, 2022.

A Pile Driving Analyzer® Model 8G recorded, processed and displayed the dynamic data to meet the objectives of the hammer system calibration. Discussions on the test methods, limitations and implementation are provided in Appendix A. The energy measurement results are summarized in the appended table with the average and standard deviation provided in Appendix B together with representative plots of force and normalized velocity.

EQUIPMENT

Hammer and Penetrometer System

Energy measurements were recorded during standard penetration tests conducted for two automatic hammers and the following drill rig types.

Drill Rig Number and Type	
Rig #10 Mobile B-57 Track-rig	Rig #17 Mobile B-58 Truck-rig
SN 2019010	SN 2017004

Measurements were recorded for three boring locations; reported for the respective rigs as H-355-22 and Y1-13-22 and located at the southwest corner vicinity of Valley Avenue and Freeman Road in Fife and near 33645 20th Ave S, Federal Way, WA 98003, respectively. Holt Services, Inc. advanced the penetrometers to testing depths of approximately $131\frac{1}{2}$ feet and $30\frac{1}{3}$ feet,

respectively. The instrumented subassembly was connected to the top of the drill rod string and measurements recorded at intervals for each depth of data. Selected data presented in this report is from depths of approximately 105, 125, and 130 feet for Rig #10 (boring H-355-22), and 5, 7.5, 10, 12.5, 15, 17.5, 20, 25, and 30 feet for Rig #17 (boring Y1-13-22).

Measurements were recorded for each blow required to advance the sampler. For Rig #10, results are provided for 12 inches of the sampler advancement (i.e., excluding the initial 6 inches of advancement) for the averaging of results. For Rig #17, results are averaged for all data, as refusal driving (i.e., 50 blows for 6-inches advancement, or greater) was typically encountered. Please refer to ASTM D4633 regarding recommendations on blow counts and instrumented drill rod lengths, as well as other details of the test method.

The following drill rod dimensions, of rod size NWJ, were employed during testing.

Drill Rod Area sq. inch	Outside Diameter Inch	Inside Diameter inch
1.45	2.63	2.25

Note that these are the rod size dimensions for the instrumented section used for the testing, as well as each drill rod string. A split spoon sampler was used as the penetrometer.

Instrumentation

A Pile Driving Analyzer was employed for recording, processing, and displaying the dynamic data. An instrumented subassembly, inserted at the top of the drill rod string below the hammer and anvil system and above the drill rods, was used to record force and acceleration data. The subassembly was instrumented with two foil strain gages in a full bridge circuit and two piezoresistive accelerometers attached on diametrically opposite sides of the subassembly. Data sampling frequency was 50.0 kHz.

The 8G utilizes a digital system, and with the employed sampling frequency of 50.0 kHz, the signal conditioning conforms to ASTM D4633. Results for the maximum hammer operating rate, rod top force and velocity, and transferred energy are provided in Appendix B and summarized in the appended table. Discussions on the test method and its limitations can be found in Appendix A.

MEASUREMENTS AND CALCULATIONS

The primary objective of testing was the measurement of the energy transmitted from the hammer impact through the anvil into the instrumented subassembly and drill rods. Strain transducers and

accelerometers were employed for the calculation of the transferred energy using force, F(t) and velocity v(t), records as follows:

$$EFV = \int_{b}^{a} F(t)v(t)dt$$

where time "b" is to the beginning of the energy transfer and time "a" is to the time at which the energy transfer reaches a maximum. Force is calculated as the product of the measured strain, elastic modulus and cross-sectional area, and measured acceleration is integrated to velocity.

Integrated over the complete impact event and calculated from measured force and velocity, the energy transferred to the top of the drill rod was calculated as a function of time. The maximum transferred energy (i.e., EFV) is used as an indicator of the energy content of the event. The described method is the only theoretically correct method of measuring energy transfer and automatically corrects for rod non-uniformities such as connector masses or loose joints.

TEST RESULTS

Result Discussion

Dynamic data was evaluated for the hammer operating rate, rod top force and velocity, and transferred energy. Appendix B provides the evaluated quantities for blows making up the SPT N-value, with their averages and standard deviation, plotted and printed as a function of depth for the selected sequences of the standard penetration tests. Measurements collected for relevant samples are presented herein.

The tables in Appendix B include:

- FMX – the maximum measured rod top force
- VMX – the maximum measured rod top velocity
- BPM – the hammer operating rate in blows per minute
- EFV – the maximum calculated energy (EMX) transferred to the rod top
- ETR – ratio of transferred energy (EFV) to the maximum theoretical potential energy

The maximum theoretical potential energy is the product of the standard 140 lb hammer impact mass dropped the standard 30 inches.

A representative plot of force and normalized velocity versus time for a typical blow from each data set is provided in Appendix B to demonstrate the data quality.

Summary of Results

I. The automatic hammers were monitored during standard penetration tests conducted on April 27, 2022. The average energy transfer ratio calculated with the EFV method for the sequence for each drill rig is tabulated below together with the corresponding, average hammer operating rate.

Drill Rig	Energy Transfer Ratio percent	Operating Rate bpm
Rig #10 Mobile B-57	**86.3**	39.4
Rig #17 Mobile B-58	**82.5**	30.0

.

II. For Rig #10, the uncorrected N-values encountered during the sequences ranged from 20 to 85 blows per foot, and the corrected values ranged from 28 to 122 blows per foot. For Rig #17, the N-values are typically not able to be calculated, due to the refusal conditions encountered (i.e., the N-values shown in Appendix B for Rig #17 may be disregarded, as they were not able to be calculated correctly). Note that at the depth of 5 feet, the uncorrected N-value is 65 blows per foot, and the corrected value is 89 blows per foot.

III. To convert the uncorrected N-values for the employed hammer and penetrometer system and operators, the Schmertman correction for adjustment to 60 percent transfer efficiency is

$$N_{60} = \left(\frac{e_m}{60}\right) N_m$$

where N_{60} is the corrected hammer N-value, e_m is the percent energy transfer efficiency (i.e., e_m = 100*ETR) and N_m is the measured SPT N-value. N_{60} values for the measurements and monitored depths are presented in the appended tables. The measured overall energy transfer ratios tabulated above for the respective drill rigs produce respective N_{60} equivalent values of roughly $1.44N_m$ and $1.38N_m$ for each rig. Further corrections due to overburden stresses in the soil may be made prior to use of the N-values for design purposes.

We appreciate the opportunity to be of assistance to you on this project. Please contact our offices should you have any questions regarding the contents of this report, or if we may be of further service.

Respectfully,
GRL ENGINEERS, INC.



4-28-2022

Marty G. Bixler, P.E. (Washington #51806)
Senior Engineer

Gabriela Wong, E.I.T.
Staff Engineer

APPENDIX A
AN INTRODUCTION INTO SPT DYNAMIC PILE TESTING

The following has been written by GRL Engineers, Inc. and may only be copied with its written permission.

1. BACKGROUND

The Standard Penetration Test is frequently conducted as an in-situ assessment of soil strength. This test requires that a 140 lb weight is dropped 30 inches onto a drive rod at whose bottom a sampler is usually installed. The sampler is driven for 18 inches; the number of blows required for the last 12 inches of driving is the so-called N-value. The N-value may be used as a strength indicator for foundation design or as a means of assessing the liquefaction potential of soils.

Obviously, the SPT hammer efficiency is an important consideration when using the N-values for design purposes. Measurements have indicated that the energy in the drive rod is sometimes only 30% and and may reach 90% of the potential or rated energy of the SPT hammer (E-rated = 0.35 kip-ft or 0.475 kJ). The type of hammer used to drive the rod is the main reason for these variations. On the average, the energy in the drive rod is 60% of the standard rated energy.

Because of the variability of energy, methods based on N-values are considered unreliable. However, measurements during SPT testing using the Case Method can be done on a routine basis and these measurements yield the transferred energy values. With measured energy, EMX, known, an adjustment of the measured N-value, N_m, can be made as follows.

$$N_{60} = N_m [E_m / (0.6E_r)] \quad (1)$$

Thus, if the measured energy value is equal to the normally expected transferred energy of 60% of E-rated then the adjusted and measured N-values are identical. On the other hand, if the measured energy is only 30% then the adjusted blow count will be reduced by 50%.

2. DYNAMIC TESTING AND ANALYSIS METHODS APPLIED TO SPT

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (e.g. a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. Thus, the method is also referred to as a “High Strain Method”. The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then a calculation of various quantities. Conveniently, for SPT applications, the measurements and analyses are done by a single piece of equipment: the SPT Analyzer. The Pile Driving Analyzer® (PDA) is also suitable to perform these measurements and data processing.

A related analysis method is the “Wave Equation Analysis” which calculates a relationship between bearing capacity, pile stresses, transferred energy and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data. This program can be used very effectively to simulate the SPT driving process.

3. MEASUREMENTS

GRL uses equipment manufactured by Pile Dynamics, Inc. The system includes either an SPT-Analyzer™ (SPTA) or a Pile Driving Analyzer® (PDA), an instrumented rod section and two accelerometers. SPT energy testing is very closely related to and borrows procedures from dynamic pile testing. Those interested in the basis of the SPT energy testing method may obtain extensive literature on dynamic pile testing from GRL Engineers, Inc.

3.1 SPT Analyzer or Pile Driving Analyzer

The basis for the results calculated by the SPTA or PDA are strain and acceleration measured in an instrumented rod section. These signals are converted to rod top force, F(t), and rod top velocity, v(t). The SPTA or PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. The product of these two

measurements is then integrated over time which yields the energy transferred to the instrumented section as a function of time (see Section 4.1).

For convenience and accuracy, strain measurements are usually taken on an instrumented section of SPT drive rod. Ideally, the section properties of the instrumented rod and those of the drive rod are the same, however, using subs, other sections can also be utilized.

For the instrumented section, PDI provides a force calibration in such a way that the output of the instrumented rod is directly calculated without the need for an accurate elastic modulus or cross sectional area of the rod section.

The acceleration measurements are often demanding in the SPT environment, because of high frequency and high acceleration motion components. An experienced measurement engineer, therefore, has to evaluate the quality of this data before final conclusions are drawn from the numerical results calculated by SPTA or PDA.

SPTA or PDA records are taken while the standard N-value is acquired in the conventional manner. This then allows a direct correlation between N-value and average transferred energy.

3.2 HPA

The SPT hammer's ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. The impact velocity results can be automatically processed with a PC or recorded on a strip chart. HPA measurements yield a hammer kinetic energy, but not the energy transferred to the drive rod.

4 RECORD EVALUATION BY SPTA OR PDA

4.1 HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = {}_0\int^t F(\tau)v(\tau)\, d\tau \qquad (2)$$

The maximum of the E(t) curve is often called **ENTHRU or EMX;** it is the most important quantity for an overall evaluation of the performance of a hammer and driving system. **EMX** allows for a classification of the hammer's performance when presented as, e_T, the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency.

$$e_T = EMX/E_R \qquad (3)$$

where E_R is the hammer manufacturer's rated energy value or 0.35 kip-ft (0.475 kJ) in the case of the SPT hammer.

Often in the SPT literature one finds also reference to the EF2 energy. This evaluation is based on assumed proportionality between force and velocity (see also Section 5):

$$v(t) = F(t) / Z \qquad (4)$$

where Z = EA/c is the pile impedance, E is the elastic modulus, A is the cross sectional area and c is the speed of the stress wave in the pile material..

Combining equations 2 and 4 leads to

$$EF(t) = {}_0\int^t F(\tau)^2 / Z\, d\tau \qquad (5)$$

The EF2 transferred energy value is the EF-value at the time t = 2L/c, where L is the drive rod length and c is the stress wave speed in steel (16,800 ft/s or 5,124 m/s). Since the force is easier to measure than both force and velocity, Equation 5 is preferred by some test engineers. However, the EF method is fraught with errors and certain correction factors have to be applied to make it approximately correct. Among the error sources are the following:

- Proportionality is often violated prior to time 2L/c. The proportionality between force and velocity in a downward traveling wave only holds if the wave does not encounter a disturbance prior to reflecting off the pile toe. Such disturbances include a change in cross sectional area, an open or loose splice or joint, or resistance along the shaft.

- Using only one force measurement precludes a data quality check based on the proportionality between force and velocity. Thus, a force measurement that is for some reason in error may not be detectable, which will lead to errors in the EF2 value. Data quality checks will be discussed further in Section 5.

The use if EF2 is therefore not recommended but it is often included in result presentations for the sake of completeness.

4.2 STRESSES

During SPT monitoring, it is also of interest to monitor compressive stresses at both the top of the drive rod and at its bottom.

At the pile top (location of sensors) the maximum compression stress averaged over the rod's cross section, **CSX**, is directly obtained from the measurements. Note that this stress value refers to the instrumented section. If the rod has a different cross sectional area then the stress in the rod will be different from CSX.

The SPTA or PDA can also calculate, in an approximate manner, the force at the rod bottom, **CFB**. To obtain the corresponding stress, this force value should be divided by the appropriate cross sectional area, e.g. by the rod area just above the sampler or by the sampler area itself. Of course, non-uniform stress components as they might occur at the sampler tip due to a sloping rock are not considered in this calculation.

5. DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing SPTA or PDA tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to the so-called proportionality relationship.

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the rod, force and velocity measured at its top are proportional

$$F = v\,Z \qquad (5)$$

where Z is again the pile impedance, Z = EA/c. This relationship can also be expressed in terms of stress

$$\sigma = F/A = v\,(E/c) \qquad (6)$$

or strain

$$\epsilon = \sigma/E = v / c \qquad (7)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements but is only truly meaningful for perfectly uniform rods. Open or loose splices, for example, will lead to a non-proportionality. For SPT rods it is fortunate that usually no soil resistance acts along the shaft and for that reason, proportionality can exist until the stress wave returns from sampler top or rod bottom unless connectors are not sufficiently tightened or have a significant mass.

Velocity data quality can also be checked by looking at the final displacement, DFN, which is calculated from the acceleration by double integration. If the calculated final displacement is much higher or lower than indicated by the N-value, the accelerometer attachment may be loose or the sensor may be faulty. If major drift in the velocity is observed, the EMX value may be in error, even though proportionality from impact to time 2L/c exists. In this case, it may be useful to evaluate the energy transferred to the drill rod at time 2L/c, which is calculated by the PDA or SPTA as the E2E quantity.

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Appendix B

Results of SPT Rig Calibration

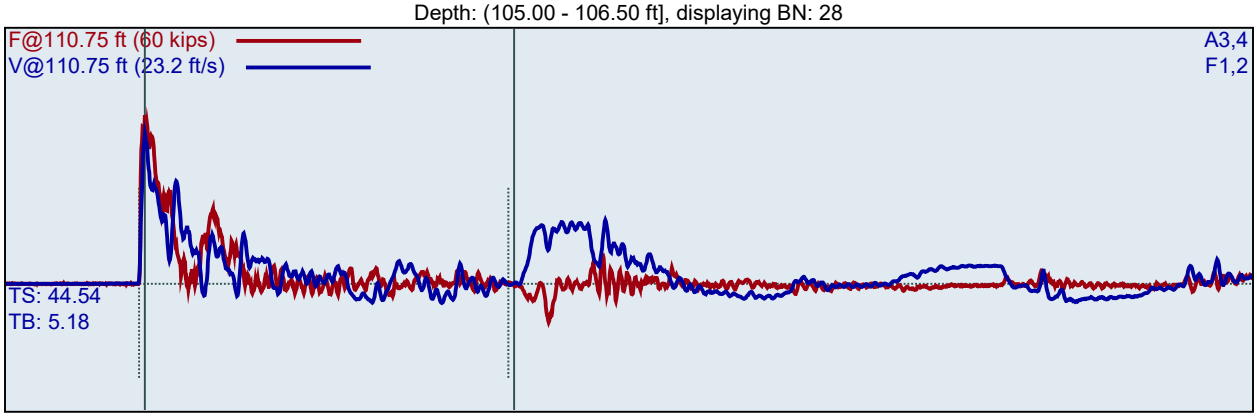
Rig 10
mgb-grl
H-355-22

105
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 110.75 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (105.00 - 106.50 ft], displaying BN: 28



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
1	10	105.05	39	13.8	7.9	305	87.2
2	10	105.10	38	13.1	39.4	303	86.5
3	10	105.15	37	13.2	39.6	297	84.9
4	10	105.20	39	14.3	39.6	316	90.4
5	10	105.25	39	13.5	39.6	309	88.2
6	10	105.30	38	13.2	39.6	306	87.5
7	10	105.35	40	13.5	39.6	307	87.6
8	10	105.40	39	13.4	39.6	309	88.4
9	10	105.45	39	13.7	39.6	311	89.0
10	10	105.50	39	13.7	39.6	312	89.1
11	8	105.56	39	13.6	39.6	310	88.6
12	8	105.63	38	13.5	39.6	305	87.2
13	8	105.69	41	14.1	39.7	309	88.2
14	8	105.75	38	13.3	39.6	302	86.4
15	8	105.81	40	13.5	39.6	305	87.1
16	8	105.88	40	13.6	39.6	308	88.0
17	8	105.94	41	13.3	39.7	307	87.6
18	8	106.00	40	14.0	39.6	311	88.9
19	12	106.04	39	14.0	39.7	307	87.7
20	12	106.08	38	13.7	39.6	311	88.8
21	12	106.13	38	13.2	39.7	298	85.0
22	12	106.17	40	13.1	39.5	302	86.2
23	12	106.21	40	13.5	39.7	309	88.2
24	12	106.25	38	13.4	39.6	308	88.1
25	12	106.29	40	13.7	39.6	312	89.1
26	12	106.33	40	13.5	39.7	311	88.9
27	12	106.38	38	12.8	39.6	296	84.6
28	12	106.42	40	13.6	39.7	311	88.8

29	12	106.46	41	12.9	39.6	302	86.3
30	12	106.50	39	13.3	39.7	305	87.3
		Average	39	13.5	39.6	306	87.5
		Std Dev	1	0.3	0.1	4	1.3
		Maximum	41	14.1	39.7	312	89.1
		Minimum	38	12.8	39.5	296	84.6
			N-value: 20				

BN: 30 10-8-12

Sample Interval Time: 43.90 seconds.

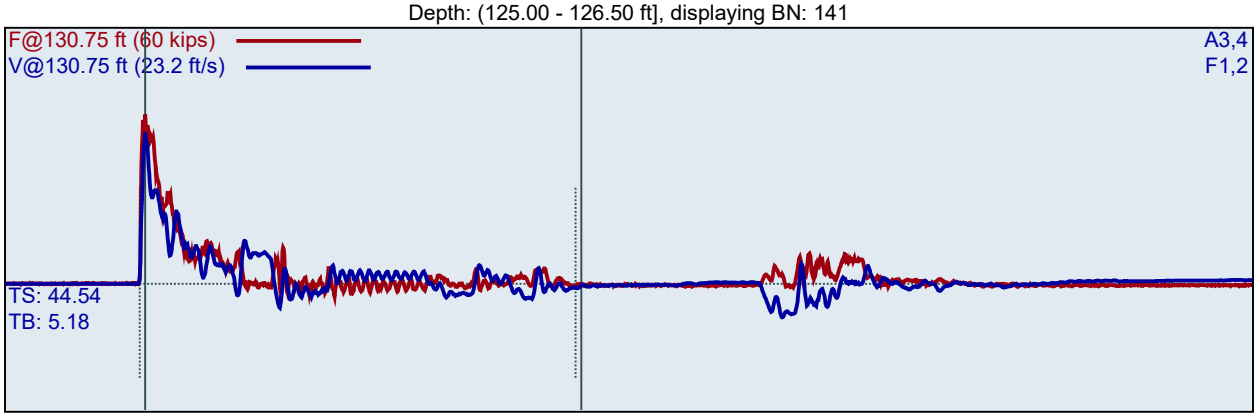
Rig 10
mgb-grl
H-355-22

105
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 130.75 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (125.00 - 126.50 ft], displaying BN: 141



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
31	28	125.02	40	13.4	1.9	305	87.2
32	28	125.04	39	12.8	38.6	299	85.5
33	28	125.05	38	12.8	38.8	290	82.9
34	28	125.07	36	13.2	38.8	297	84.8
35	28	125.09	37	13.1	38.7	301	85.9
36	28	125.11	38	13.0	38.8	298	85.0
37	28	125.13	36	13.5	38.8	294	84.0
38	28	125.14	35	13.5	38.8	302	86.4
39	28	125.16	35	13.3	38.9	299	85.4
40	28	125.18	37	13.7	38.8	299	85.4
41	28	125.20	37	13.6	38.8	300	85.7
42	28	125.21	36	13.4	38.8	300	85.7
43	28	125.23	36	13.6	38.8	301	85.9
44	28	125.25	38	13.8	38.9	302	86.2
45	28	125.27	37	14.0	38.9	298	85.0
46	28	125.29	36	13.3	38.8	296	84.6
47	28	125.30	39	13.5	38.8	304	86.9
48	28	125.32	36	13.6	38.8	300	85.7
49	28	125.34	39	14.1	38.8	300	85.9
50	28	125.36	39	13.6	38.9	296	84.5
51	28	125.38	38	14.1	38.8	298	85.2
52	28	125.39	36	13.9	38.9	301	86.0
53	28	125.41	37	13.5	38.9	301	86.1
54	28	125.43	37	13.3	38.9	295	84.3
55	28	125.45	35	13.7	38.9	294	84.0
56	28	125.46	35	13.9	38.8	302	86.2
57	28	125.48	37	13.4	38.8	301	85.9
58	28	125.50	38	12.6	38.8	292	83.3
59	42	125.51	36	13.7	38.9	294	83.9
60	42	125.52	37	13.0	38.8	293	83.7
61	42	125.54	36	13.6	38.9	294	84.0

62	42	125.55	35	13.5	38.8	291	83.2
63	42	125.56	36	13.8	38.8	293	83.6
64	42	125.57	36	13.8	38.8	298	85.3
65	42	125.58	36	13.1	38.9	287	82.0
66	42	125.60	36	13.0	38.8	295	84.2
67	42	125.61	37	12.9	38.9	293	83.8
68	42	125.62	36	13.6	38.8	295	84.2
69	42	125.63	37	13.3	38.8	295	84.3
70	42	125.64	37	13.4	38.9	292	83.5
71	42	125.65	37	13.1	38.8	296	84.5
72	42	125.67	39	13.4	38.9	301	85.9
73	42	125.68	39	12.7	38.8	302	86.2
74	42	125.69	36	12.7	38.9	295	84.4
75	42	125.70	38	13.5	38.9	297	84.8
76	42	125.71	38	13.3	38.8	300	85.6
77	42	125.73	38	13.7	38.9	299	85.5
78	42	125.74	38	13.3	38.8	302	86.4
79	42	125.75	38	12.7	38.9	298	85.1
80	42	125.76	38	12.6	38.8	299	85.4
81	42	125.77	38	12.4	38.9	297	84.9
82	42	125.79	38	11.9	38.8	291	83.1
83	42	125.80	38	12.2	38.9	296	84.6
84	42	125.81	39	12.1	38.9	298	85.3
85	42	125.82	39	13.0	38.8	300	85.8
86	42	125.83	37	11.7	38.9	294	84.0
87	42	125.85	38	12.6	38.8	298	85.2
88	42	125.86	38	12.6	38.9	297	85.0
89	42	125.87	37	14.6	38.9	299	85.5
90	42	125.88	38	14.4	38.8	300	85.6
91	42	125.89	37	13.7	38.9	296	84.5
92	42	125.90	38	13.4	38.8	291	83.1
93	42	125.92	39	12.0	38.9	287	82.0
94	42	125.93	35	14.2	38.9	296	84.6
95	42	125.94	35	13.7	38.8	294	84.1
96	42	125.95	38	13.0	38.9	288	82.3
97	42	125.96	37	13.6	38.8	295	84.2
98	42	125.98	38	13.3	38.9	295	84.2
99	42	125.99	38	12.9	38.8	294	84.0
100	42	126.00	37	14.3	38.9	296	84.5
101	43	126.01	36	13.6	38.8	294	83.9
102	43	126.02	37	13.3	38.9	293	83.7
103	43	126.03	38	14.2	38.8	291	83.3
104	43	126.05	37	13.4	38.9	292	83.3
105	43	126.06	36	14.0	38.8	294	83.9
106	43	126.07	37	14.5	38.8	301	86.0
107	43	126.08	38	13.1	38.9	297	84.9
108	43	126.09	39	13.6	38.8	298	85.2
109	43	126.10	38	13.9	38.8	302	86.3
110	43	126.12	38	13.0	38.9	297	84.9
111	43	126.13	38	14.2	38.8	297	84.8
112	43	126.14	39	12.9	38.9	296	84.6
113	43	126.15	39	13.4	38.8	298	85.1
114	43	126.16	39	13.3	38.8	300	85.8
115	43	126.17	39	13.5	38.9	307	87.6
116	43	126.19	39	13.4	38.9	298	85.3
117	43	126.20	39	13.4	38.9	306	87.3
118	43	126.21	38	13.5	38.9	305	87.2
119	43	126.22	39	13.6	38.8	303	86.7
120	43	126.23	39	13.8	38.9	307	87.8
121	43	126.24	39	13.7	38.8	307	87.7
122	43	126.26	40	13.7	38.9	300	85.7

123	43	126.27	39	13.8	38.9	308	88.1
124	43	126.28	40	13.8	38.8	308	87.9
125	43	126.29	39	13.8	38.9	302	86.3
126	43	126.30	40	13.9	38.9	303	86.7
127	43	126.31	39	13.6	38.8	302	86.2
128	43	126.33	40	13.1	38.9	294	83.9
129	43	126.34	39	13.7	38.8	305	87.2
130	43	126.35	39	13.8	38.9	306	87.5
131	43	126.36	39	13.9	38.8	305	87.0
132	43	126.37	40	13.8	38.8	300	85.6
133	43	126.38	39	13.7	38.9	301	86.0
134	43	126.40	39	13.7	38.9	306	87.3
135	43	126.41	40	13.5	38.9	299	85.6
136	43	126.42	39	13.7	38.9	301	85.9
137	43	126.43	40	13.7	39.0	303	86.6
138	43	126.44	40	13.9	38.7	305	87.1
139	43	126.45	40	13.3	38.9	299	85.5
140	43	126.47	40	13.8	38.9	306	87.5
141	43	126.48	40	13.7	38.8	306	87.3
142	43	126.49	39	13.6	38.8	305	87.1
143	43	126.50	39	13.3	38.9	300	85.7
		Average	38	13.4	38.9	298	85.2
		Std Dev	1	0.6	0.0	5	1.4
		Maximum	40	14.6	39.0	308	88.1
		Minimum	35	11.7	38.7	287	82.0
			N-value: 85				

BN: 143 28-42-43

Sample Interval Time: 174.53 seconds.

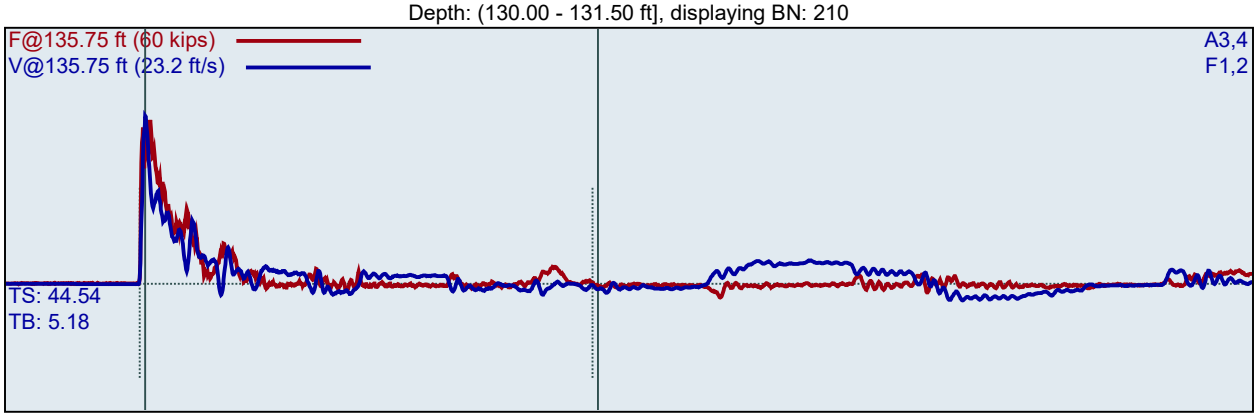
Rig 10
mgb-grl
H-355-22

105
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 135.75 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (130.00 - 131.50 ft], displaying BN: 210



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
144	20	130.03	39	13.4	2.0	306	87.4
145	20	130.05	36	13.5	39.8	297	84.9
146	20	130.08	38	14.7	40.1	305	87.1
147	20	130.10	38	14.7	40.0	306	87.5
148	20	130.13	38	12.9	40.1	298	85.2
149	20	130.15	38	13.1	38.6	301	85.9
150	20	130.18	39	13.5	40.0	304	86.7
151	20	130.20	39	14.6	40.1	308	88.1
152	20	130.23	39	13.5	40.0	306	87.3
153	20	130.25	39	14.5	40.0	306	87.4
154	20	130.28	39	13.0	40.1	303	86.7
155	20	130.30	39	14.1	40.1	307	87.7
156	20	130.33	39	12.9	40.0	293	83.7
157	20	130.35	39	14.1	40.1	306	87.3
158	20	130.38	39	13.9	40.1	305	87.3
159	20	130.40	39	13.3	40.2	299	85.5
160	20	130.43	38	13.8	40.1	306	87.5
161	20	130.45	39	15.2	40.1	302	86.3
162	20	130.48	38	13.9	40.0	301	85.9
163	20	130.50	37	14.5	40.1	303	86.6
164	26	130.52	38	14.3	40.1	304	86.8
165	26	130.54	39	13.7	40.1	299	85.3
166	26	130.56	38	14.6	40.1	307	87.8
167	26	130.58	38	13.8	40.1	302	86.2
168	26	130.60	40	13.6	40.0	305	87.2
169	26	130.62	37	15.0	40.2	305	87.3
170	26	130.63	39	13.6	40.0	304	87.0
171	26	130.65	39	14.1	40.1	302	86.3
172	26	130.67	39	14.2	40.2	301	86.0
173	26	130.69	39	14.1	40.1	306	87.3
174	26	130.71	38	14.2	40.1	304	86.9

175	26	130.73	39	13.9	40.2	307	87.7
176	26	130.75	39	14.1	40.1	302	86.3
177	26	130.77	39	13.7	40.1	304	86.9
178	26	130.79	39	15.3	40.2	313	89.6
179	26	130.81	39	14.3	40.1	305	87.3
180	26	130.83	38	13.8	40.1	304	86.8
181	26	130.85	38	15.7	40.1	308	88.1
182	26	130.87	40	14.1	40.1	307	87.8
183	26	130.88	39	16.0	40.1	313	89.4
184	26	130.90	38	16.3	40.1	313	89.5
185	26	130.92	38	14.7	40.1	309	88.4
186	26	130.94	39	16.0	40.1	313	89.5
187	26	130.96	38	15.4	40.1	306	87.3
188	26	130.98	38	14.1	40.1	302	86.2
189	26	131.00	39	14.6	40.2	304	87.0
190	23	131.02	40	14.3	40.2	303	86.7
191	23	131.04	39	15.2	40.1	309	88.4
192	23	131.07	38	15.0	40.1	308	88.0
193	23	131.09	40	14.1	40.2	307	87.8
194	23	131.11	39	16.1	40.1	312	89.0
195	23	131.13	40	14.8	40.2	308	88.1
196	23	131.15	38	15.3	40.1	308	88.0
197	23	131.17	39	16.0	40.1	309	88.3
198	23	131.20	39	15.3	40.2	305	87.2
199	23	131.22	40	14.9	40.1	305	87.0
200	23	131.24	39	14.6	40.1	307	87.6
201	23	131.26	39	14.9	40.1	304	86.9
202	23	131.28	37	15.0	40.1	308	88.0
203	23	131.30	39	15.4	40.1	306	87.3
204	23	131.33	38	15.1	40.1	306	87.4
205	23	131.35	39	15.0	40.2	304	86.9
206	23	131.37	37	14.4	40.1	305	87.1
207	23	131.39	39	14.9	40.1	304	86.8
208	23	131.41	37	15.2	40.1	302	86.3
209	23	131.43	39	15.2	40.2	310	88.4
210	23	131.46	38	15.2	40.2	311	88.9
211	23	131.48	38	15.5	40.0	310	88.7
212	23	131.50	38	15.5	40.2	309	88.3
		Average	39	14.8	40.1	306	87.5
		Std Dev	1	0.7	0.0	3	1.0
		Maximum	40	16.3	40.2	313	89.6
		Minimum	37	13.6	40.0	299	85.3
			N-value: 49				

BN: 212 20-26-23

Sample Interval Time: 101.77 seconds.

Summary of SPT Test Results

Project: Rig 10, Test Date: 4/27/2022

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	Start Depth ft	Final Depth ft	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
110.75	**10-8-12**	105.00	106.50	**20**	**28**	39	13.5	39.6	306	87.5
130.75	**28-42-43**	125.00	126.50	**85**	**122**	38	13.4	38.9	298	85.2
135.75	**20-26-23**	130.00	131.50	**49**	**70**	39	14.8	40.1	306	87.5
				Overall Average Values:		38	13.9	39.4	302	86.3
				Standard Deviation:		1	0.9	0.6	6	1.7
				Overall Maximum Value:		41	16.3	40.2	313	89.6
				Overall Minimum Value:		35	11.7	38.7	287	82.0

GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

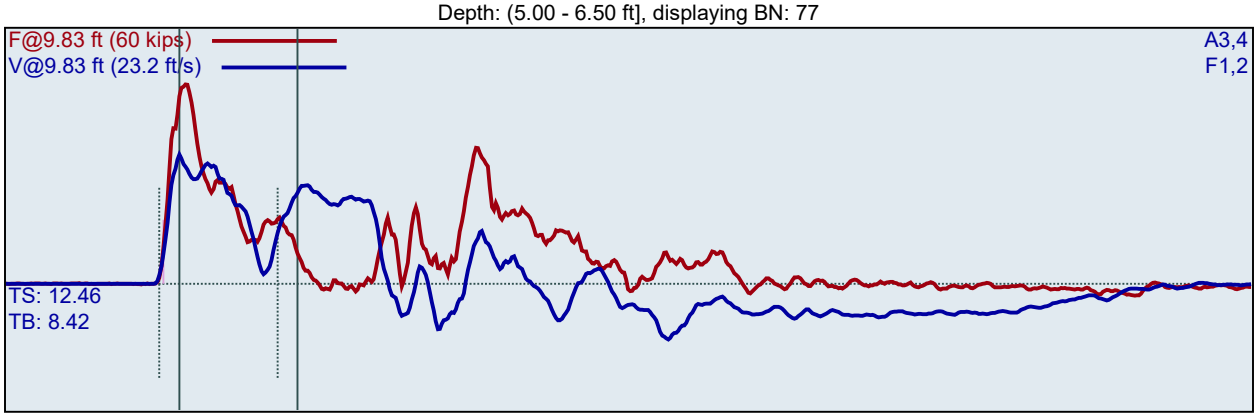
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 9.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (5.00 - 6.50 ft], displaying BN: 77



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

LP	BL#	BC	FMX	VMX	BPM	EFV	ETR
ft		/6"	kips	ft/s	bpm	ft-lb	%
5.04	1	14	37	14.6	1.9	259	74.1
5.07	2	14	40	12.6	30.8	279	79.6
5.11	3	14	39	14.1	30.8	294	84.1
5.14	4	14	37	14.1	30.9	284	81.2
5.18	5	14	37	13.5	30.9	277	79.2
5.21	6	14	41	14.3	30.8	293	83.8
5.25	7	14	42	14.5	30.9	293	83.8
5.29	8	14	41	13.6	30.9	287	82.0
5.32	9	14	41	14.2	30.8	290	82.9
5.36	10	14	42	14.0	31.0	291	83.0
5.39	11	14	43	13.9	30.9	291	83.2
5.43	12	14	41	14.0	30.7	287	81.9
5.46	13	14	42	13.5	30.9	290	82.8
5.50	14	14	44	13.0	30.8	289	82.7
5.52	15	27	42	14.2	30.8	286	81.6
5.54	16	27	36	14.0	31.0	282	80.7
5.56	17	27	40	14.1	30.9	277	79.3
5.57	18	27	41	14.1	30.9	278	79.5
5.59	19	27	42	14.3	30.7	283	80.8
5.61	20	27	43	14.5	30.9	289	82.5
5.63	21	27	43	14.1	30.8	285	81.4
5.65	22	27	42	14.0	30.9	280	80.0
5.67	23	27	42	14.1	30.8	284	81.2
5.69	24	27	47	15.2	30.8	307	87.8
5.70	25	27	44	16.0	30.7	310	88.5
5.72	26	27	31	13.2	30.7	252	72.0
5.74	27	27	33	12.6	30.9	262	74.8
5.76	28	27	37	13.1	30.7	271	77.6

5.78	29	27	37	13.3	30.8	272	77.8
5.80	30	27	36	13.8	30.6	278	79.3
5.81	31	27	38	14.0	30.8	278	79.6
5.83	32	27	35	13.7	30.8	281	80.2
5.85	33	27	41	11.3	30.8	277	79.1
5.87	34	27	44	11.1	30.8	280	80.1
5.89	35	27	40	11.2	30.8	283	81.0
5.91	36	27	43	11.1	30.8	282	80.7
5.93	37	27	43	11.1	30.7	280	80.0
5.94	38	27	45	11.2	30.9	288	82.3
5.96	39	27	44	11.3	30.7	291	83.2
5.98	40	27	43	11.1	30.7	286	81.8
6.00	41	27	44	11.3	30.7	280	79.9
6.01	42	38	43	11.2	30.5	280	79.9
6.03	43	38	42	11.2	30.8	281	80.2
6.04	44	38	43	11.1	30.8	282	80.5
6.05	45	38	44	11.3	30.6	285	81.5
6.07	46	38	44	11.4	30.8	292	83.4
6.08	47	38	45	11.5	30.8	291	83.2
6.09	48	38	46	11.6	30.7	284	81.2
6.11	49	38	47	11.9	30.7	289	82.7
6.12	50	38	47	12.2	30.7	295	84.2
6.13	51	38	46	11.7	30.7	285	81.5
6.14	52	38	46	11.8	30.6	288	82.2
6.16	53	38	47	12.1	30.7	288	82.4
6.17	54	38	47	11.8	30.7	291	83.0
6.18	55	38	47	12.1	30.7	291	83.0
6.20	56	38	47	11.9	30.5	288	82.4
6.21	57	38	47	11.8	30.5	286	81.8
6.22	58	38	47	12.1	30.5	293	83.7
6.24	59	38	46	11.8	30.5	287	82.0
6.25	60	38	46	11.7	30.8	283	81.0
6.26	61	38	48	11.8	30.8	293	83.6
6.28	62	38	47	12.1	30.5	292	83.6
6.29	63	38	47	12.2	30.6	295	84.3
6.30	64	38	47	12.0	30.8	287	82.0
6.32	65	38	47	12.2	30.5	294	84.0
6.33	66	38	48	12.3	30.6	293	83.7
6.34	67	38	47	12.0	30.4	291	83.3
6.36	68	38	47	11.4	30.7	284	81.3
6.37	69	38	47	12.1	30.5	286	81.7
6.38	70	38	47	12.0	30.6	289	82.5
6.39	71	38	48	11.3	30.5	282	80.5
6.41	72	38	47	11.6	30.5	286	81.7
6.42	73	38	47	11.7	30.5	290	82.9
6.43	74	38	47	11.7	30.6	289	82.7
6.45	75	38	48	11.4	30.4	284	81.1
6.46	76	38	47	11.5	30.6	284	81.1
6.47	77	38	47	11.7	30.7	282	80.5
6.49	78	38	47	11.5	30.5	288	82.2
6.50	79	38	47	11.5	30.7	285	81.3
		Average	43	12.6	30.3	285	81.5
		Std Dev	4	1.2	3.2	8	2.4
		Maximum	48	16.0	31.0	310	88.5
		Minimum	31	11.1	1.9	252	72.0
			N-value: 79				

BN: 79 14-27-38

Sample Interval Time: 152.35 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

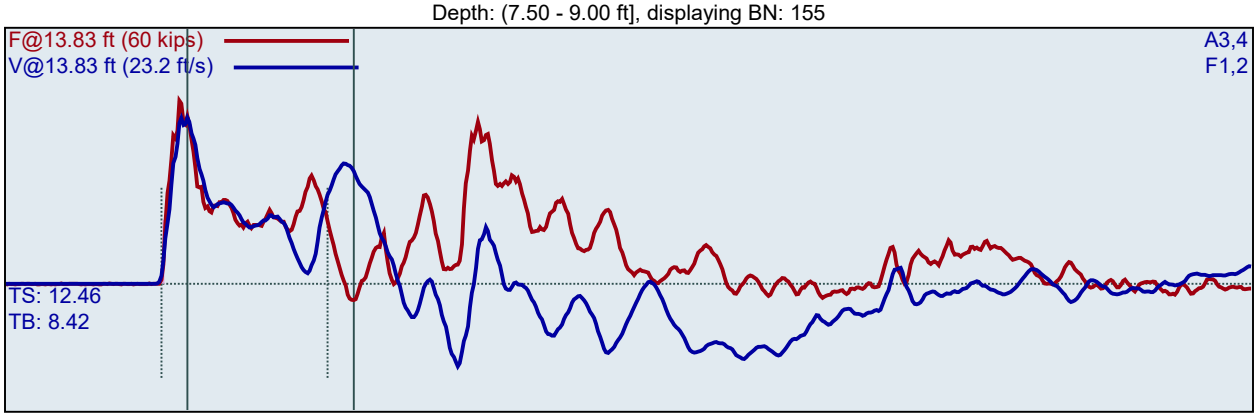
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 13.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (7.50 - 9.00 ft], displaying BN: 155



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
7.52	80	28	39	14.8	1.9	247	70.6
7.54	81	28	41	15.4	30.0	282	80.6
7.55	82	28	42	15.8	30.0	292	83.5
7.57	83	28	42	15.3	30.0	288	82.3
7.59	84	28	39	14.5	29.9	268	76.7
7.61	85	28	43	15.8	30.0	302	86.2
7.63	86	28	42	15.4	30.0	289	82.5
7.64	87	28	43	15.4	30.1	296	84.6
7.66	88	28	40	14.3	29.9	267	76.2
7.68	89	28	42	15.2	30.1	285	81.5
7.70	90	28	42	15.2	30.0	283	80.8
7.71	91	28	42	15.7	29.9	288	82.4
7.73	92	28	39	14.4	29.8	275	78.5
7.75	93	28	42	15.5	29.9	300	85.7
7.77	94	28	41	14.7	30.0	279	79.7
7.79	95	28	42	14.9	29.8	296	84.5
7.80	96	28	42	15.3	29.8	297	85.0
7.82	97	28	44	15.8	30.0	302	86.3
7.84	98	28	41	14.7	29.9	285	81.5
7.86	99	28	41	15.3	29.9	290	82.9
7.88	100	28	41	14.9	29.9	289	82.6
7.89	101	28	42	15.1	29.8	290	82.8
7.91	102	28	43	15.5	29.8	291	83.1
7.93	103	28	40	14.4	29.8	278	79.5
7.95	104	28	41	14.8	29.9	280	80.0
7.96	105	28	40	14.4	29.7	273	78.0
7.98	106	28	43	15.3	29.8	294	84.1
8.00	107	28	42	15.3	30.0	291	83.2
8.01	108	50	42	15.4	29.9	288	82.2
8.01	109	50	42	15.5	29.9	285	81.6
8.02	110	50	42	15.1	29.8	295	84.2

8.03	111	50	42	15.4	30.0	289	82.7
8.03	112	50	44	15.7	29.7	296	84.5
8.04	113	50	41	14.9	29.9	277	79.1
8.05	114	50	41	14.9	29.8	276	78.9
8.05	115	50	42	15.2	29.8	284	81.2
8.06	116	50	42	15.3	29.9	290	82.7
8.07	117	50	42	15.4	29.8	290	82.8
8.07	118	50	42	15.4	29.8	286	81.6
8.08	119	50	42	15.0	29.7	285	81.3
8.09	120	50	43	15.2	29.7	280	80.0
8.09	121	50	42	15.2	29.8	285	81.5
8.10	122	50	41	14.7	29.8	280	80.1
8.11	123	50	43	15.3	29.8	297	84.9
8.11	124	50	41	14.6	29.6	289	82.6
8.12	125	50	41	14.8	29.7	289	82.5
8.13	126	50	41	14.5	29.8	269	76.9
8.13	127	50	43	15.1	29.8	282	80.5
8.14	128	50	43	15.3	29.7	290	82.9
8.15	129	50	43	15.3	29.8	295	84.2
8.15	130	50	42	14.9	29.8	278	79.5
8.16	131	50	43	15.3	29.9	288	82.4
8.17	132	50	42	14.7	29.6	280	80.0
8.17	133	50	43	14.9	29.8	283	80.8
8.18	134	50	44	15.3	29.8	298	85.2
8.19	135	50	41	14.5	29.7	274	78.3
8.19	136	50	44	15.3	30.0	292	83.4
8.20	137	50	43	15.4	29.6	296	84.5
8.21	138	50	43	15.0	29.8	295	84.4
8.21	139	50	43	15.0	29.8	287	81.9
8.22	140	50	43	15.4	29.8	290	82.8
8.23	141	50	44	15.2	29.8	296	84.7
8.23	142	50	42	14.8	29.8	289	82.7
8.24	143	50	42	15.0	29.9	282	80.6
8.25	144	50	42	15.1	29.7	283	81.0
8.25	145	50	42	15.0	29.8	278	79.4
8.26	146	50	43	15.5	29.7	296	84.7
8.27	147	50	44	15.3	29.7	294	84.1
8.27	148	50	43	15.1	29.5	283	80.9
8.28	149	50	41	14.7	29.7	269	77.0
8.29	150	50	44	15.2	29.7	293	83.6
8.29	151	50	42	14.8	29.7	268	76.5
8.30	152	50	45	15.1	29.7	291	83.2
8.31	153	50	43	15.0	29.7	286	81.7
8.31	154	50	44	15.2	29.7	286	81.7
8.32	155	50	43	15.0	29.7	281	80.3
8.33	156	50	43	15.0	29.8	294	84.0
8.33	157	50	44	15.1	29.8	289	82.4
		Average	42	15.1	29.5	286	81.7
		Std Dev	1	0.3	3.1	9	2.7
		Maximum	45	15.8	30.1	302	86.3
		Minimum	39	14.3	1.9	247	70.6
			N-value: 78				

BN: 157 28-50/4"

Sample Interval Time: 156.97 seconds.

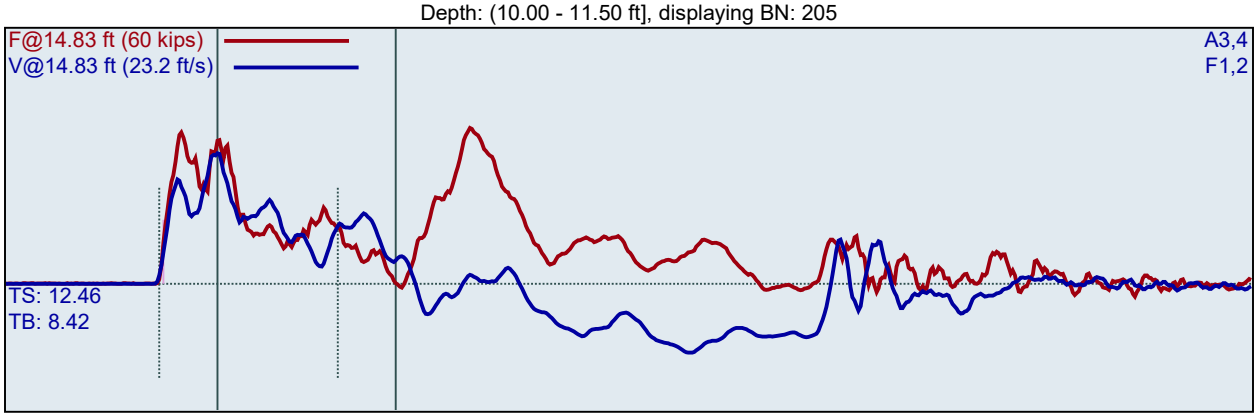
GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

Rig 17 5
mgb-grl Interval start: 4/27/2022
Y1-13-22

AR: 1.45 in^2 SP: 0.492 k/ft3
LE: 14.83 ft EM: 30000 ksi
WS: 16807.9 ft/s

Depth: (10.00 - 11.50 ft], displaying BN: 205



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
10.01	158	50	38	14.7	1.9	244	69.8
10.02	159	50	39	12.7	21.5	263	75.0
10.03	160	50	40	11.5	21.4	243	69.4
10.04	161	50	41	11.3	22.4	251	71.7
10.05	162	50	43	11.1	22.4	258	73.7
10.06	163	50	43	11.0	22.4	277	79.2
10.07	164	50	43	10.8	22.5	254	72.6
10.08	165	50	43	10.9	22.6	250	71.5
10.09	166	50	43	10.9	24.3	263	75.3
10.10	167	50	41	11.5	29.5	253	72.3
10.11	168	50	43	10.6	29.4	264	75.5
10.12	169	50	43	10.6	29.4	267	76.4
10.13	170	50	42	10.4	29.4	262	74.9
10.14	171	50	42	10.7	29.5	259	73.9
10.15	172	50	41	10.3	29.4	259	74.0
10.16	173	50	41	10.2	29.4	253	72.4
10.17	174	50	40	10.4	29.4	252	72.0
10.18	175	50	41	10.6	29.4	253	72.4
10.19	176	50	39	10.4	29.4	251	71.7
10.20	177	50	40	10.5	29.4	252	72.0
10.21	178	50	39	10.6	29.5	257	73.4
10.22	179	50	38	10.3	29.4	256	73.1
10.23	180	50	38	10.0	29.4	249	71.2
10.24	181	50	39	10.0	29.3	246	70.3
10.25	182	50	38	9.9	29.5	251	71.6
10.26	183	50	37	10.2	29.4	251	71.6
10.27	184	50	41	10.1	29.4	248	70.9
10.28	185	50	40	9.9	29.5	244	69.6
10.29	186	50	40	10.4	29.3	247	70.5
10.30	187	50	37	10.7	29.3	249	71.2
10.31	188	50	37	10.1	29.4	245	70.1

10.32	189	50	35	10.3	29.5	245	69.9
10.33	190	50	35	10.0	29.4	236	67.5
10.34	191	50	35	9.9	29.2	236	67.5
10.35	192	50	35	10.1	29.4	236	67.5
10.36	193	50	37	10.2	29.4	238	68.1
10.37	194	50	37	10.5	29.4	240	68.5
10.38	195	50	38	11.4	29.4	251	71.7
10.39	196	50	37	10.1	29.2	238	68.1
10.40	197	50	37	10.9	29.4	243	69.4
10.41	198	50	36	11.0	29.4	245	70.0
10.42	199	50	35	10.2	29.4	234	66.8
10.43	200	50	36	11.1	29.2	246	70.2
10.44	201	50	37	10.9	29.4	244	69.7
10.45	202	50	37	10.8	29.3	246	70.3
10.46	203	50	34	10.5	29.2	234	67.0
10.47	204	50	34	10.2	29.4	233	66.7
10.48	205	50	37	11.8	29.3	254	72.7
10.49	206	50	37	11.3	29.3	247	70.6
10.50	207	50	36	11.5	29.3	245	70.1
		Average	39	10.7	27.7	249	71.2
		Std Dev	3	0.8	4.5	9	2.6
		Maximum	43	14.7	29.5	277	79.2
		Minimum	34	9.9	1.9	233	66.7

N-value: 50

BN: 207 50/6"

Sample Interval Time: 105.07 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

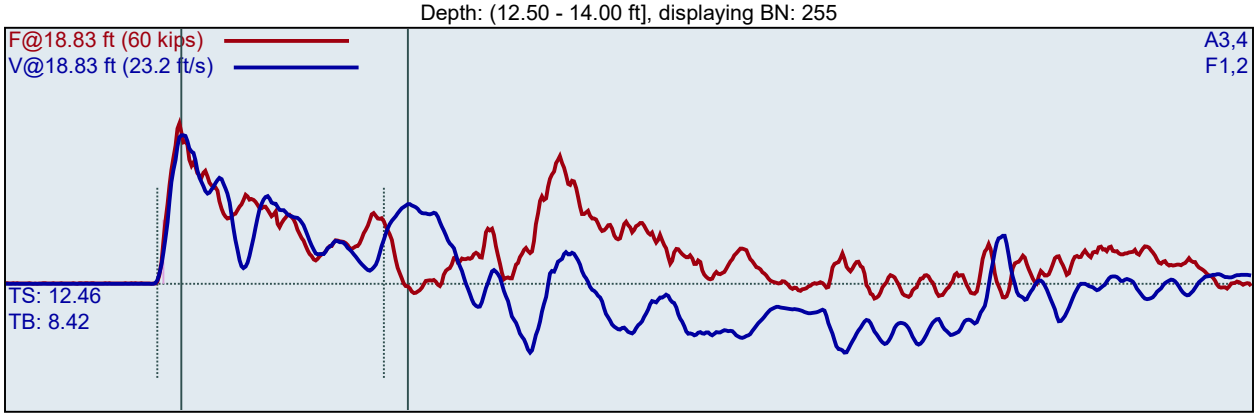
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 18.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (12.50 - 14.00 ft], displaying BN: 255



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
12.51	208	50	43	15.7	1.9	289	82.6
12.52	209	50	40	14.8	3.1	279	79.7
12.53	210	50	39	14.4	30.4	277	79.2
12.54	211	50	41	15.1	30.3	300	85.6
12.55	212	50	38	14.1	30.3	269	76.9
12.56	213	50	40	14.6	30.4	290	83.0
12.57	214	50	36	14.2	30.4	275	78.4
12.58	215	50	36	13.8	30.4	266	75.9
12.59	216	50	36	13.6	30.4	275	78.5
12.60	217	50	35	13.4	30.3	270	77.0
12.61	218	50	35	13.4	30.5	271	77.3
12.62	219	50	36	13.4	30.4	263	75.1
12.63	220	50	36	13.3	30.3	270	77.0
12.64	221	50	36	13.3	30.3	277	79.1
12.65	222	50	36	13.1	30.5	284	81.1
12.66	223	50	34	12.2	30.2	297	84.9
12.67	224	50	36	12.7	30.5	264	75.5
12.68	225	50	33	12.0	30.4	292	83.5
12.69	226	50	35	12.4	30.4	266	75.9
12.70	227	50	35	12.7	30.4	267	76.3
12.71	228	50	35	12.5	30.4	272	77.8
12.72	229	50	36	12.8	30.3	270	77.0
12.73	230	50	36	13.0	30.4	269	76.8
12.74	231	50	35	12.9	30.4	273	78.1
12.75	232	50	36	12.8	30.2	267	76.2
12.76	233	50	38	12.9	30.5	285	81.5
12.77	234	50	35	12.7	30.2	274	78.3
12.78	235	50	35	12.6	30.2	260	74.2
12.79	236	50	35	12.6	30.1	261	74.5
12.80	237	50	35	12.6	30.4	261	74.6
12.81	238	50	38	13.6	30.2	280	80.1

12.82	239	50	37	13.5	30.1	271	77.5
12.83	240	50	38	13.8	30.2	276	78.8
12.84	241	50	41	14.7	30.2	293	83.6
12.85	242	50	37	13.9	30.2	270	77.1
12.86	243	50	40	14.2	30.4	287	82.0
12.87	244	50	38	14.0	30.2	275	78.5
12.88	245	50	40	14.3	30.3	278	79.4
12.89	246	50	39	14.0	30.0	278	79.5
12.90	247	50	39	14.1	30.3	279	79.8
12.91	248	50	40	14.4	30.3	284	81.2
12.92	249	50	41	14.8	30.1	289	82.5
12.93	250	50	40	14.4	30.3	285	81.3
12.94	251	50	40	14.5	30.1	278	79.4
12.95	252	50	39	14.2	30.2	270	77.2
12.96	253	50	39	13.6	30.2	281	80.4
12.97	254	50	38	14.0	30.3	278	79.5
12.98	255	50	38	13.5	30.2	271	77.5
12.99	256	50	37	13.9	30.0	281	80.2
13.00	257	50	38	14.5	30.2	294	83.9
		Average	37	13.6	29.2	277	79.0
		Std Dev	2	0.8	5.4	10	2.8
		Maximum	43	15.7	30.5	300	85.6
		Minimum	33	12.0	1.9	260	74.2
			N-value: 50				

BN: 257 50/6"

Sample Interval Time: 114.30 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

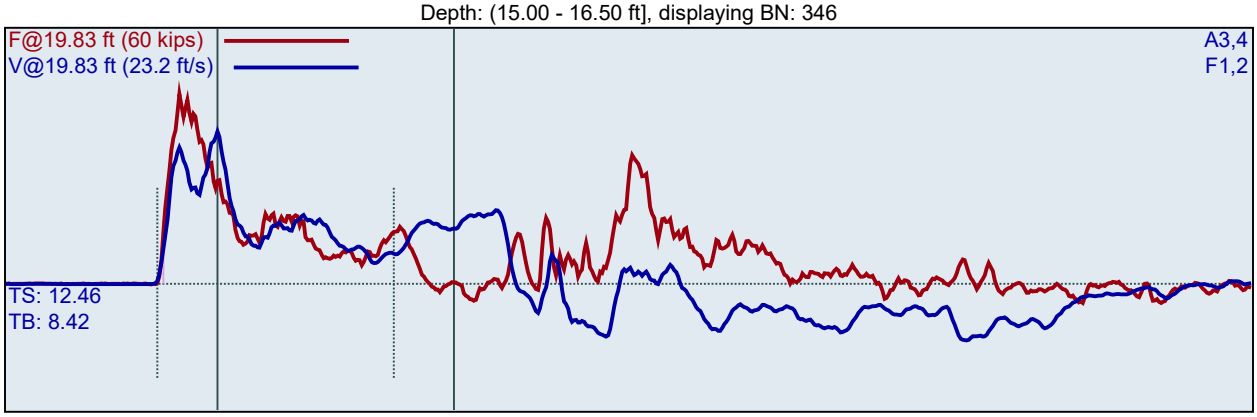
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 19.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (15.00 - 16.50 ft], displaying BN: 346



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
15.01	258	41	48	14.7	1.9	292	83.4
15.02	259	41	46	14.8	30.7	287	82.0
15.04	260	41	47	13.1	30.7	298	85.0
15.05	261	41	47	13.1	30.7	297	84.9
15.06	262	41	45	12.0	30.7	282	80.5
15.07	263	41	47	13.1	30.8	296	84.4
15.09	264	41	49	13.1	30.7	306	87.4
15.10	265	41	47	13.1	30.7	295	84.4
15.11	266	41	47	12.6	30.7	293	83.8
15.12	267	41	47	13.2	30.7	299	85.4
15.13	268	41	46	12.3	30.5	283	80.7
15.15	269	41	47	12.8	30.7	293	83.7
15.16	270	41	47	12.8	30.7	289	82.4
15.17	271	41	47	12.7	30.7	294	83.9
15.18	272	41	45	12.2	30.7	280	80.0
15.20	273	41	48	13.0	30.7	307	87.6
15.21	274	41	48	12.9	30.5	296	84.5
15.22	275	41	45	12.1	30.5	293	83.6
15.23	276	41	45	12.0	30.7	287	82.0
15.24	277	41	48	13.0	30.8	305	87.2
15.26	278	41	48	13.0	30.5	303	86.7
15.27	279	41	48	13.1	30.6	309	88.3
15.28	280	41	48	12.8	30.3	302	86.2
15.29	281	41	48	13.2	30.6	310	88.6
15.30	282	41	46	12.6	30.6	294	84.1
15.32	283	41	45	11.7	30.5	284	81.0
15.33	284	41	46	12.3	30.5	295	84.3
15.34	285	41	46	12.1	30.5	292	83.4
15.35	286	41	45	11.8	30.4	288	82.1
15.37	287	41	46	12.5	30.6	295	84.4
15.38	288	41	49	13.0	30.6	310	88.6

15.39	289	41	48	13.0	30.4	309	88.2
15.40	290	41	48	12.9	30.6	306	87.5
15.41	291	41	47	12.4	30.5	294	84.1
15.43	292	41	45	11.9	30.4	289	82.5
15.44	293	41	45	12.1	30.6	285	81.5
15.45	294	41	46	12.6	30.6	300	85.8
15.46	295	41	47	12.3	30.3	295	84.2
15.48	296	41	46	12.1	30.5	289	82.6
15.49	297	41	46	11.9	30.5	277	79.0
15.50	298	41	47	12.5	30.5	296	84.7
15.51	299	50	47	12.7	30.4	302	86.3
15.52	300	50	48	12.5	30.5	296	84.7
15.53	301	50	47	12.5	30.5	293	83.7
15.53	302	50	46	12.1	30.5	275	78.6
15.54	303	50	46	12.7	30.5	296	84.5
15.55	304	50	47	12.4	30.4	291	83.1
15.56	305	50	47	12.7	30.4	296	84.6
15.57	306	50	47	12.7	30.4	303	86.5
15.58	307	50	47	12.7	30.4	295	84.2
15.58	308	50	47	12.3	30.6	287	82.0
15.59	309	50	47	12.2	30.3	286	81.8
15.60	310	50	47	12.6	30.5	298	85.1
15.61	311	50	46	12.3	30.3	286	81.8
15.62	312	50	47	12.7	30.3	298	85.1
15.63	313	50	46	12.0	30.4	274	78.2
15.63	314	50	47	12.0	30.3	277	79.2
15.64	315	50	47	12.3	30.3	290	82.8
15.65	316	50	45	12.5	30.4	283	80.9
15.66	317	50	43	11.2	30.4	258	73.7
15.67	318	50	42	12.1	30.4	275	78.5
15.68	319	50	43	12.5	30.4	285	81.3
15.68	320	50	41	12.4	30.2	274	78.2
15.69	321	50	41	11.3	30.6	264	75.6
15.70	322	50	42	12.4	30.3	283	80.9
15.71	323	50	42	12.1	30.3	304	86.7
15.72	324	50	43	12.7	30.3	282	80.5
15.73	325	50	42	13.2	30.3	283	80.7
15.73	326	50	43	12.7	30.5	283	80.8
15.74	327	50	42	12.0	30.3	271	77.6
15.75	328	50	43	11.7	30.3	274	78.3
15.76	329	50	43	13.7	30.2	290	82.8
15.77	330	50	43	13.6	30.4	289	82.6
15.78	331	50	43	13.5	30.3	292	83.5
15.78	332	50	43	13.9	30.3	297	84.7
15.79	333	50	42	12.0	30.2	277	79.1
15.80	334	50	44	11.9	30.3	280	80.1
15.81	335	50	44	13.3	30.3	292	83.4
15.82	336	50	44	13.2	30.3	293	83.7
15.83	337	50	44	13.1	30.2	290	82.9
15.83	338	50	44	12.5	30.5	287	82.0
15.84	339	50	44	12.2	30.2	279	79.6
15.85	340	50	44	12.9	30.2	289	82.5
15.86	341	50	44	12.7	30.3	288	82.4
15.87	342	50	44	13.0	30.5	290	82.9
15.88	343	50	43	11.7	30.3	274	78.2
15.88	344	50	42	12.4	30.5	279	79.7
15.89	345	50	42	12.4	30.5	280	79.9
15.90	346	50	44	13.8	30.4	294	83.9
15.91	347	50	43	13.4	30.3	288	82.3
15.92	348	50	44	14.1	30.3	291	83.1

Average	45	12.6	30.1	290	82.9
Std Dev	2	0.6	3.0	10	2.9
Maximum	49	14.8	30.8	310	88.6
Minimum	41	11.2	1.9	258	73.7

N-value: 91

BN: 348 41-50/5"

Sample Interval Time: 183.21 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

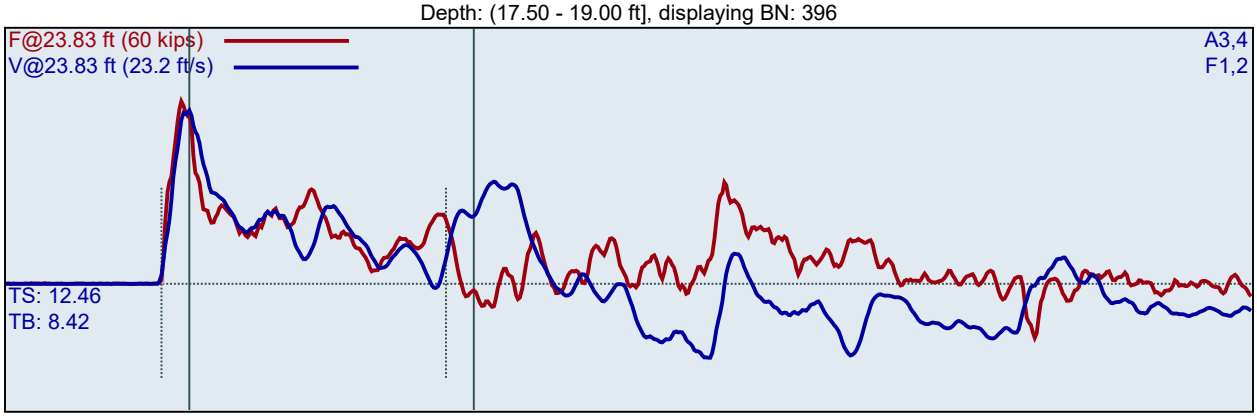
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 23.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (17.50 - 19.00 ft], displaying BN: 396



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
17.51	349	50	38	14.9	1.9	280	80.0
17.52	350	50	42	15.9	32.0	301	85.9
17.53	351	50	41	15.7	32.1	292	83.5
17.53	352	50	42	16.2	32.3	301	86.1
17.54	353	50	42	16.2	31.6	303	86.7
17.55	354	50	43	16.2	31.9	301	86.0
17.56	355	50	42	15.9	31.8	301	85.9
17.57	356	50	43	16.3	31.9	304	86.8
17.58	357	50	42	16.1	31.6	299	85.4
17.58	358	50	42	15.9	31.6	300	85.7
17.59	359	50	41	15.6	31.6	293	83.6
17.60	360	50	43	16.6	31.7	312	89.1
17.61	361	50	42	15.8	31.9	304	86.9
17.62	362	50	42	15.9	32.0	303	86.6
17.63	363	50	43	16.3	32.0	308	88.0
17.63	364	50	43	16.2	31.7	306	87.6
17.64	365	50	42	15.6	31.8	296	84.5
17.65	366	50	41	15.6	31.3	300	85.8
17.66	367	50	41	15.7	31.6	294	83.9
17.67	368	50	42	16.0	31.7	299	85.5
17.68	369	50	41	15.6	31.5	293	83.7
17.68	370	50	43	15.8	31.2	304	87.0
17.69	371	50	43	16.1	31.5	309	88.2
17.70	372	50	41	15.5	31.3	290	82.8
17.71	373	50	43	16.0	31.5	311	88.7
17.72	374	50	43	15.8	31.2	298	85.1
17.73	375	50	43	15.9	31.7	306	87.4
17.73	376	50	43	16.0	31.2	307	87.7
17.74	377	50	41	15.6	31.1	296	84.7
17.75	378	50	40	15.4	31.7	286	81.8
17.76	379	50	43	16.3	31.2	313	89.5

17.77	380	50	42	15.8	31.4	297	84.8
17.78	381	50	42	15.9	31.4	301	86.0
17.78	382	50	43	15.9	31.0	309	88.4
17.79	383	50	43	16.2	31.4	310	88.6
17.80	384	50	43	16.3	30.7	309	88.3
17.81	385	50	42	16.2	31.1	307	87.7
17.82	386	50	42	15.9	31.3	300	85.6
17.83	387	50	43	16.2	31.3	307	87.6
17.83	388	50	43	15.9	30.7	309	88.2
17.84	389	50	43	16.0	31.3	310	88.6
17.85	390	50	43	16.5	31.1	314	89.8
17.86	391	50	42	15.7	31.1	293	83.6
17.87	392	50	41	15.6	31.3	297	84.8
17.88	393	50	42	15.7	31.0	300	85.7
17.88	394	50	41	15.6	30.9	290	82.9
17.89	395	50	43	15.8	31.3	303	86.5
17.90	396	50	43	15.7	30.6	304	86.8
17.91	397	50	42	15.9	31.1	300	85.7
17.92	398	50	43	16.0	31.2	310	88.5
		Average	42	15.9	30.8	302	86.2
		Std Dev	1	0.3	4.1	7	2.1
		Maximum	43	16.6	32.3	314	89.8
		Minimum	38	14.9	1.9	280	80.0
			N-value: 50				

BN: 398 50/5"

Sample Interval Time: 97.38 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

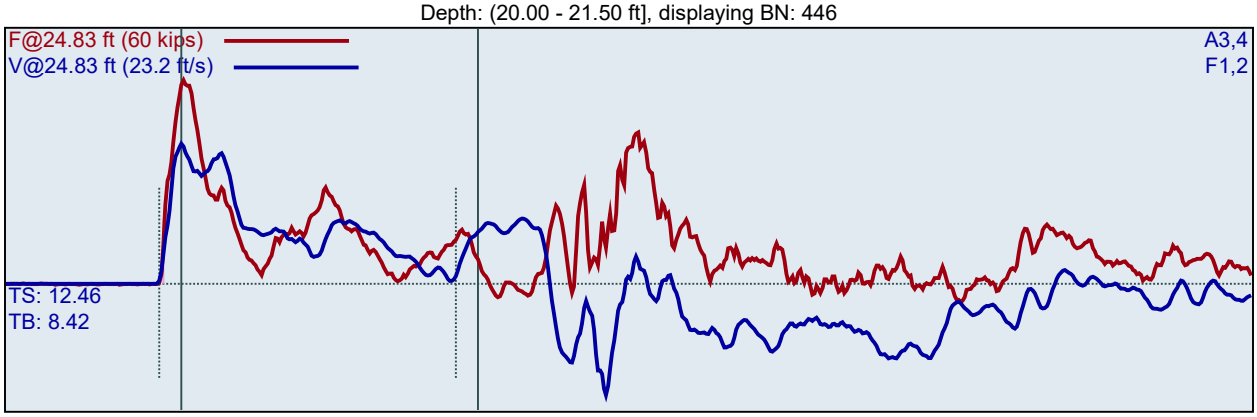
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 24.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (20.00 - 21.50 ft], displaying BN: 446



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
20.01	399	50	49	13.3	1.9	315	90.0
20.01	400	50	48	12.6	31.4	309	88.2
20.02	401	50	46	12.4	31.6	304	86.9
20.02	402	50	46	12.1	31.6	299	85.4
20.03	403	50	47	12.5	31.6	305	87.1
20.03	404	50	47	12.1	31.4	297	84.9
20.04	405	50	47	12.4	31.6	302	86.3
20.04	406	50	48	12.7	31.8	304	86.8
20.05	407	50	47	12.4	31.6	304	86.8
20.05	408	50	49	13.1	31.5	311	88.7
20.06	409	50	48	12.5	31.6	302	86.2
20.06	410	50	49	12.4	31.7	301	86.0
20.07	411	50	48	12.9	31.5	314	89.6
20.07	412	50	49	12.9	31.6	310	88.6
20.08	413	50	48	12.4	31.5	300	85.8
20.08	414	50	48	12.6	31.5	304	86.8
20.09	415	50	48	13.2	31.7	312	89.0
20.09	416	50	47	12.3	31.4	304	86.9
20.10	417	50	48	12.7	31.5	310	88.7
20.10	418	50	47	12.2	31.5	307	87.8
20.11	419	50	48	12.7	31.6	309	88.3
20.11	420	50	48	12.4	31.3	302	86.2
20.12	421	50	47	12.2	31.6	302	86.2
20.12	422	50	48	12.4	31.5	306	87.4
20.13	423	50	49	13.2	31.5	318	90.8
20.13	424	50	49	12.9	31.4	315	90.0
20.14	425	50	47	11.6	31.3	293	83.8
20.14	426	50	48	12.8	31.4	315	89.9
20.15	427	50	47	12.2	31.4	300	85.6
20.15	428	50	47	12.1	31.5	297	84.8
20.16	429	50	49	12.7	31.4	311	88.9

20.16	430	50	48	12.3	31.5	303	86.6
20.17	431	50	49	12.6	31.4	309	88.4
20.17	432	50	49	12.7	31.4	310	88.7
20.18	433	50	47	11.9	31.4	296	84.5
20.18	434	50	49	13.2	31.5	318	90.9
20.19	435	50	46	11.7	31.4	290	83.0
20.19	436	50	48	12.8	31.3	313	89.5
20.20	437	50	47	12.1	31.5	301	86.0
20.20	438	50	49	12.6	31.3	308	88.1
20.21	439	50	46	11.9	31.5	293	83.7
20.21	440	50	48	12.7	31.4	312	89.1
20.22	441	50	47	11.9	31.4	295	84.4
20.22	442	50	47	11.9	31.4	297	84.9
20.23	443	50	47	12.1	31.4	302	86.2
20.23	444	50	47	12.3	31.6	305	87.3
20.24	445	50	48	11.9	31.5	297	84.9
20.24	446	50	48	12.7	31.5	309	88.4
20.25	447	50	48	12.8	31.3	312	89.3
20.25	448	50	49	13.2	31.5	313	89.3
		Average	48	12.5	30.9	305	87.2
		Std Dev	1	0.4	4.1	7	1.9
		Maximum	49	13.3	31.8	318	90.9
		Minimum	46	11.6	1.9	290	83.0
			N-value: 50				

BN: 448 50/3"

Sample Interval Time: 93.44 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

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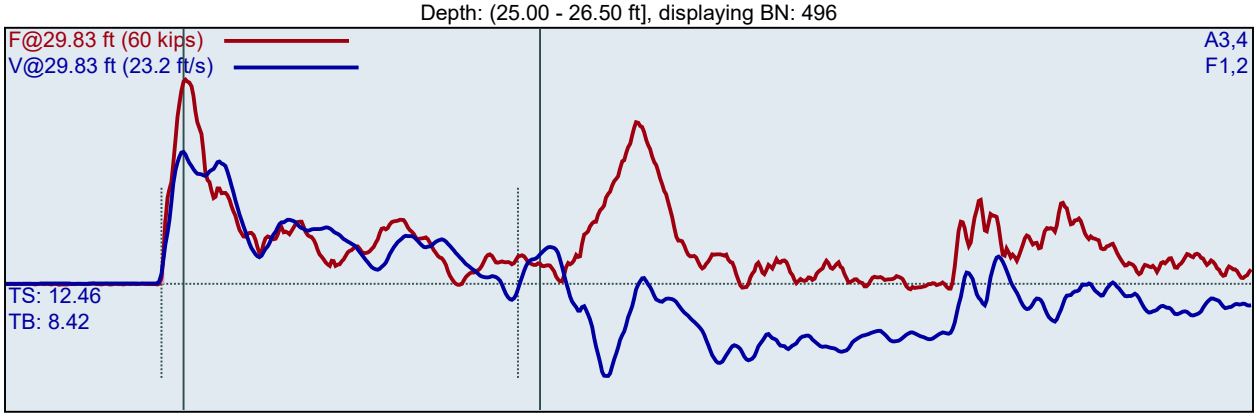
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 29.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (25.00 - 26.50 ft], displaying BN: 496



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
25.01	449	50	45	11.7	1.9	292	83.6
25.01	450	50	45	11.8	32.1	294	83.9
25.02	451	50	45	12.2	31.9	307	87.7
25.02	452	50	48	12.8	31.9	315	90.1
25.03	453	50	45	11.8	31.7	301	86.1
25.03	454	50	49	13.1	31.8	319	91.3
25.04	455	50	49	12.6	32.0	311	89.0
25.04	456	50	45	12.0	31.8	294	84.0
25.05	457	50	47	12.5	32.1	310	88.5
25.05	458	50	46	12.2	31.8	309	88.2
25.06	459	50	47	12.7	31.9	315	90.1
25.06	460	50	48	12.5	31.8	312	89.2
25.07	461	50	47	12.5	31.9	317	90.5
25.07	462	50	48	13.0	31.8	314	89.8
25.08	463	50	46	11.7	31.9	302	86.3
25.08	464	50	47	12.0	31.7	310	88.5
25.09	465	50	48	12.1	31.7	312	89.0
25.09	466	50	48	12.3	31.7	317	90.6
25.10	467	50	49	12.1	31.7	308	88.1
25.10	468	50	49	13.0	31.8	314	89.7
25.11	469	50	49	11.9	31.8	311	89.0
25.11	470	50	46	11.5	31.7	300	85.6
25.12	471	50	49	12.1	31.8	319	91.3
25.12	472	50	47	12.0	31.6	312	89.3
25.13	473	50	49	12.0	31.8	316	90.2
25.13	474	50	47	11.5	31.7	300	85.8
25.14	475	50	49	12.1	31.7	315	89.9
25.14	476	50	49	12.4	31.5	314	89.8
25.15	477	50	48	12.9	31.9	319	91.0
25.15	478	50	50	12.3	31.8	315	90.0
25.16	479	50	49	12.3	31.7	314	89.7

25.16	480	50	48	12.0	31.5	302	86.4
25.17	481	50	48	12.0	31.6	311	88.9
25.17	482	50	49	12.0	31.7	306	87.5
25.18	483	50	48	11.7	31.7	298	85.1
25.18	484	50	48	13.0	31.7	316	90.3
25.19	485	50	48	13.1	31.7	322	91.9
25.19	486	50	49	12.2	31.5	308	88.0
25.20	487	50	49	12.4	31.7	310	88.4
25.20	488	50	49	12.6	31.5	314	89.7
25.21	489	50	48	11.7	31.6	296	84.6
25.21	490	50	49	12.3	31.5	310	88.5
25.22	491	50	47	12.5	31.6	314	89.7
25.22	492	50	47	11.9	31.7	299	85.4
25.23	493	50	47	12.0	31.5	295	84.3
25.23	494	50	50	12.5	31.6	315	90.1
25.24	495	50	48	13.0	31.4	319	91.3
25.24	496	50	48	11.9	31.6	306	87.4
25.25	497	50	49	12.3	31.7	313	89.4
25.25	498	50	47	11.4	31.6	299	85.4
		Average	48	12.2	31.1	309	88.4
		Std Dev	1	0.4	4.2	8	2.2
		Maximum	50	13.1	32.1	322	91.9
		Minimum	45	11.4	1.9	292	83.6
			N-value: 50				

BN: 498 50/3"

Sample Interval Time: 92.68 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

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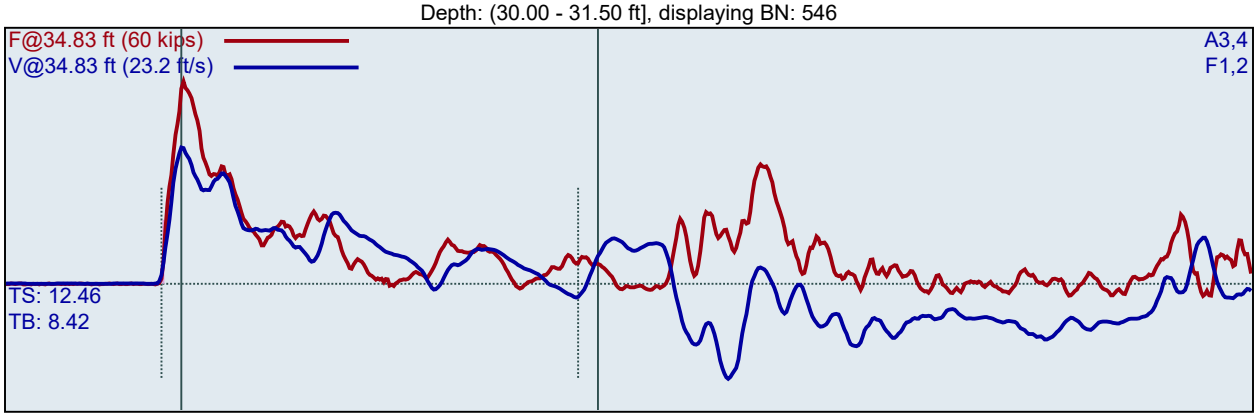
Rig 17
mgb-grl
Y1-13-22

5
Interval start: 4/27/2022

AR: 1.45 in^2
LE: 34.83 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (30.00 - 31.50 ft], displaying BN: 546



F1 : [292NWJ1] 213 PDICAL (1) FF1
F2 : [292NWJ2] 213.89 PDICAL (1) FF1

A3 (PR): [K4981] 319.416 mv/6.4v/5000g (1) VF1
A4 (PR): [K11432] 388.882 mv/6.4v/5000g (1) VF1

LP ft	BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
30.01	499	50	47	12.6	16.3	301	86.0
30.01	500	50	45	12.4	30.7	297	85.0
30.02	501	50	46	13.0	30.3	312	89.2
30.03	502	50	47	12.9	30.7	307	87.7
30.03	503	50	45	12.1	30.3	291	83.3
30.04	504	50	45	12.3	30.3	290	82.7
30.05	505	50	47	12.9	30.4	310	88.6
30.05	506	50	46	12.6	30.2	304	86.8
30.06	507	50	46	12.4	30.5	299	85.4
30.07	508	50	47	13.0	29.9	304	86.8
30.07	509	50	47	12.6	30.5	297	85.0
30.08	510	50	47	12.8	30.0	306	87.5
30.09	511	50	47	12.9	30.5	311	88.9
30.09	512	50	45	12.4	30.1	295	84.2
30.10	513	50	47	12.7	30.4	305	87.1
30.11	514	50	45	12.6	30.3	295	84.4
30.11	515	50	46	12.8	30.4	304	86.8
30.12	516	50	47	13.0	30.3	312	89.2
30.13	517	50	45	12.3	29.8	296	84.4
30.13	518	50	46	11.8	30.4	289	82.6
30.14	519	50	47	12.5	30.2	304	86.8
30.15	520	50	46	12.0	29.9	290	82.9
30.15	521	50	46	11.8	30.3	292	83.4
30.16	522	50	46	12.3	30.3	300	85.6
30.17	523	50	46	11.9	30.1	288	82.2
30.17	524	50	46	12.5	30.3	299	85.5
30.18	525	50	47	12.3	30.2	297	84.8
30.19	526	50	46	12.2	29.8	295	84.3
30.19	527	50	45	12.2	30.1	292	83.4
30.20	528	50	45	11.8	30.0	287	81.9
30.21	529	50	47	12.5	30.0	302	86.4

30.21	530	50	45	12.1	30.0	292	83.4
30.22	531	50	47	12.6	29.8	308	88.1
30.23	532	50	47	12.5	30.0	304	86.7
30.23	533	50	47	12.2	29.7	303	86.5
30.24	534	50	46	12.3	29.6	305	87.2
30.25	535	50	46	12.2	30.2	294	84.0
30.25	536	50	45	11.9	29.9	285	81.5
30.26	537	50	46	12.0	29.6	294	84.1
30.27	538	50	47	12.3	29.9	303	86.6
30.27	539	50	47	12.0	30.0	297	84.9
30.28	540	50	48	12.7	29.9	310	88.6
30.29	541	50	49	12.6	29.9	307	87.8
30.29	542	50	48	12.8	29.7	308	88.0
30.30	543	50	47	12.5	29.9	301	85.9
30.31	544	50	47	12.4	29.7	299	85.3
30.31	545	50	46	12.0	29.8	288	82.2
30.32	546	50	47	12.3	29.9	300	85.7
30.33	547	50	47	12.8	30.1	308	87.9
30.33	548	50	47	12.2	29.8	295	84.3
		Average	46	12.4	29.8	299	85.6
		Std Dev	1	0.3	1.9	7	2.1
		Maximum	49	13.0	30.7	312	89.2
		Minimum	45	11.8	16.3	285	81.5
			N-value: 50				

BN: 548 50/4"

Sample Interval Time: 97.69 seconds.

GRL Engineers, Inc.
SPT Analyzer Results

PDA-S Ver. 2021.34 - Printed: 4/28/2022

Summary of SPT Test Results

Project: Rig 17, Test Date: 4/27/2022

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Start Depth ft	Final Depth ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
9.83	5.00	6.50	**14-27-38**	**79**	**108**	43	12.6	30.3	285	81.5
13.83	7.50	9.00	**28-50-0**	**78**	**107**	42	15.1	29.5	286	81.7
14.83	10.00	11.50	**50-0-0**	**50**	**68**	39	10.7	27.7	249	71.2
18.83	12.50	14.00	**50-0-0**	**50**	**68**	37	13.6	29.2	277	79.0
19.83	15.00	16.50	**41-50-0**	**91**	**125**	45	12.6	30.1	290	82.9
23.83	17.50	19.00	**50-0-0**	**50**	**68**	42	15.9	30.8	302	86.2
24.83	20.00	21.50	**50-0-0**	**50**	**68**	48	12.5	30.9	305	87.2
29.83	25.00	26.50	**50-0-0**	**50**	**68**	48	12.2	31.1	309	88.4
34.83	30.00	31.50	**50-0-0**	**50**	**68**	46	12.4	29.8	299	85.6
				Overall Average Values:		44	13.1	30.0	289	82.5
				Standard Deviation:		4	1.6	3.9	18	5.1
				Overall Maximum Value:		50	16.6	32.3	322	91.9
				Overall Minimum Value:		31	9.9	1.9	233	66.7

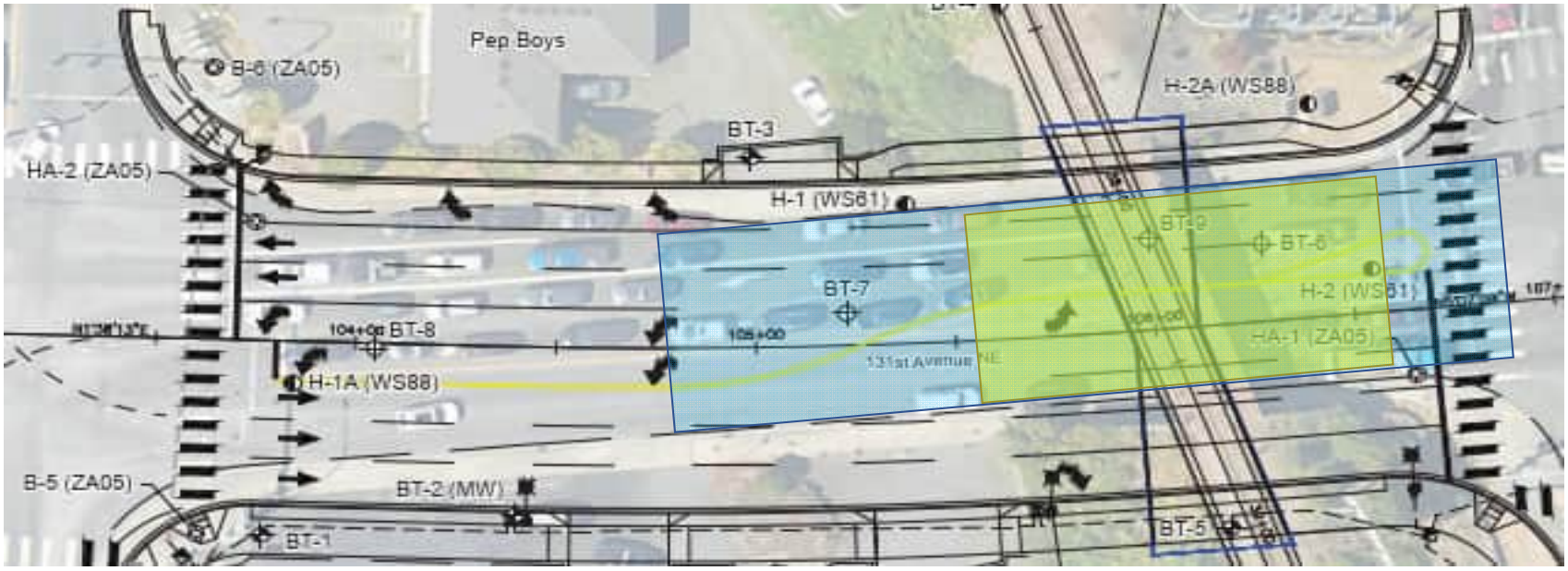
APPENDIX C SUMMARY OF PAVEMENT CORES

Table C-1

Summary of Historical and Current Pavement Cores

SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Place
Woodinville, Washington

Layer	HA-2 (HB-2) (ZZA Core 2004)	BT-8	BT-7	BT-9 (under bridge)	BT-6	HA-1 (HB-1) (ZZA Core 2004)
Total Exploration Depth	2.8 ft	3.2 ft	4.2 ft	1.1 ft	3.0 ft	2.25 ft
Asphalt Pavement	6.5" AC	9" AC, cold joint at 6"	17" AC, cold joint at 14"	13.5" AC, cold joint at 8"	15" AC, cold joint at 3"	19" AC+ATB, including: Two 2" asphalt overlays over 7" AC (total 11" AC) over 8" ATB (sandy gravel)
Portland Cement Pavement	0" PC	0" PC	7" PC (17" - 24")	Unknown (not fully penetrated), 6" assumed as at adjacent cores	6" PC (15" - 21")	7" PC (19" to 26")
Total Pavement Thickness	6.5"	9"	24"	19.5"+/- (est'd)	21"	26"
Base Course/ Subgrade	6.5" - 34" Medium dense gravelly f-m Sand (DCP Nc= 20, 34. CBR=28)	9" - 38" Poorly graded Sand (SP), w/ little f-c subrounded Gravel, trace Cobbles	24" - 50" Poorly graded Sand (SP) f-m Gravel, little f-c Gravel, trace subrounded Cobbles	Unknow (not penetrated)	21" - 24" Crushed rock (1/2" minus); 24"- 36" Poorly graded Sand (SP-SM) w/f-c subrounded gravel	21" - 27" Crushed rock and/or sandy Gravel (DCP Nc= 50/5" @ 26")



Note: Light blue shaded area (Sta. 104+75 to 106+90) is approximate area of 6" Peforated Metal Drain Pipes shown on as-built drawings for the 1963 Hwy 2 & 1-A widening improvements. Yellow shaded area (Sta. 105+54 to 106+58) is approximate area on 1963 drawings where additional 12" base course was specified for Roadway Section B.

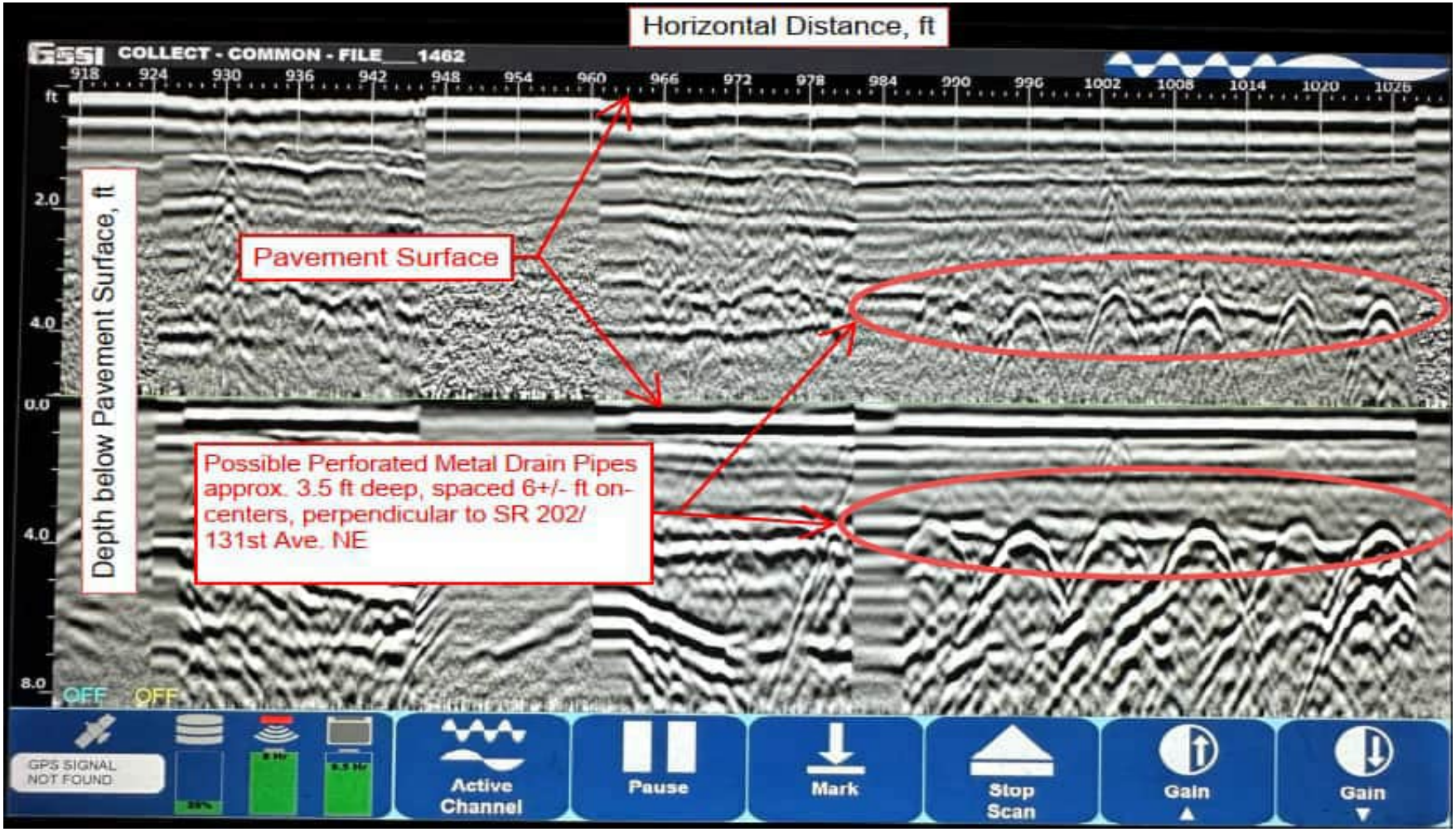


Figure C-1

GPR Image of Subsurface Near Pavement Core BT-7

APPENDIX D GEOTECHNICAL LABORATORY TESTING

Anaheim Geotechnical Laboratory Testing Summary

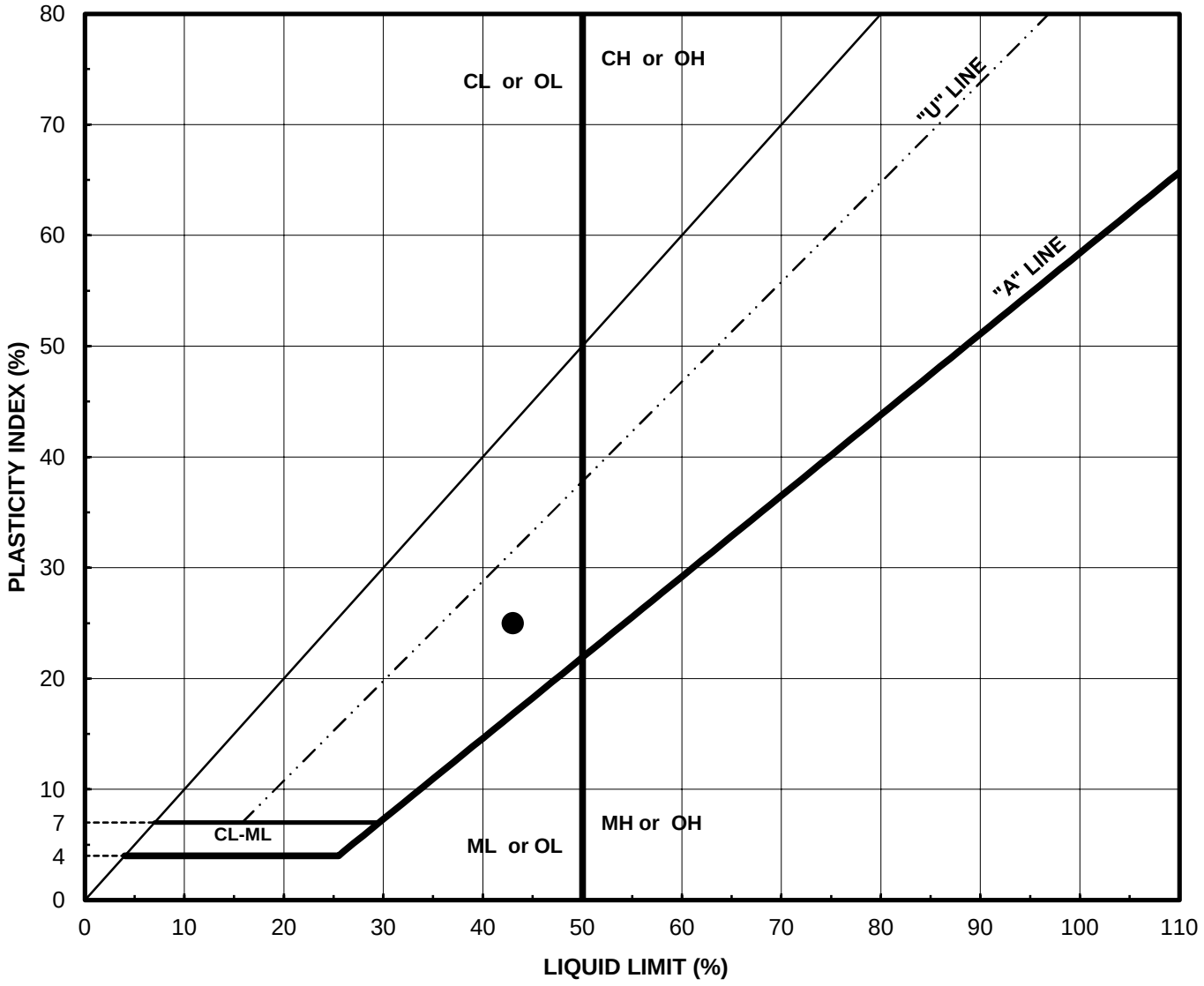
Project Name: Woodinville Trestle
Project Number: 60683538
Project Engineer: M. Walbaum

Location				Initial Condition			Limits			Gradation		
Boring Number	Sample Number	Depth (ft)	USCS Symbol	Water Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Liquid Limit	Plasticity Index	Liquidity Index	Gravel (%)	Sand (%)	Fines (%)
BT-4	HA-1	2.5	SM	16.2								40.3
BT-4	HA-2	4.0	CL	39.5								50.8
BT-4	SS-3	5.0	CL				43	25	---			
BT-4	SS-4	10.0	SM									39.6
BT-4	SS-5	15.0	SM									13.5
BT-4	SS-6	20.0	GP-GM	15.1								11.0
BT-4	SS-7	25.0	GP-GM	7.9						58.1	36.2	5.7
BT-4	SS-9	35.0	SP	10.3								
BT-4	SS-10	40.0	SP-SM	14.2						15.9	73.4	10.7
BT-4	SS-12	50.0	SM	17.0								12.2
BT-4	SS-13	55.0	SP-SM									8.9
BT-4	SS-14	60.0	SP-SM	18.0								11.9
BT-4	SS-15	65.0	SP-SM									8.1
BT-4	SS-16	70.0	SP-SM	17.0								8.4
BT-4	SS-17	75.0	SP-SM	18.8						0.0	89.1	10.9
BT-4	SS-18	80.0	SP-SM	21.5								6.5
BT-4	SS-19	85.0	SM	35.3								22.8
BT-4	SS-21	95.0	SM	21.3								14.6

Anaheim Geotechnical Laboratory Testing Summary

Project Name: Woodinville Trestle
Project Number: 60683538
Project Engineer: M. Walbaum

Location				Initial Condition			Limits			Gradation		
Boring Number	Sample Number	Depth (ft)	USCS Symbol	Water Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Liquid Limit	Plasticity Index	Liquidity Index	Gravel (%)	Sand (%)	Fines (%)
BT-5	HA-1	2.5	GP									4.5
BT-5	HA-2	4.0	SP	4.5								4.5
BT-5	SS-4	10.0	SP-SM	17.2								6.2
BT-5	SS-5	15.0	SP	31.3								
BT-5	MC-6	20.0	SP-SM	15.4						33.0	60.1	6.9
BT-5	SS-8	30.0	SP-SM	34.9								7.6
BT-5	SS-9	35.0	SP-SM							7.0	83.4	9.6
BT-5	SS-10	40.0	SP-SM	25.5								10.0
BT-5	SS-12	50.0	GP-GM	9.2								7.2
BT-5	SS-13	55.0	SP-SM									9.4
BT-5	SS-14	60.0	SP-SM	10.9								9.9
BT-5	SS-15	65.0	SP-SM									11.7
BT-5	SS-16	70.0	SP-SM	19.9						1.5	88.0	10.5
BT-5	SS-17	75.0	SP-SM	11.0								9.4
BT-5	SS-18	80.0	SP	17.5								



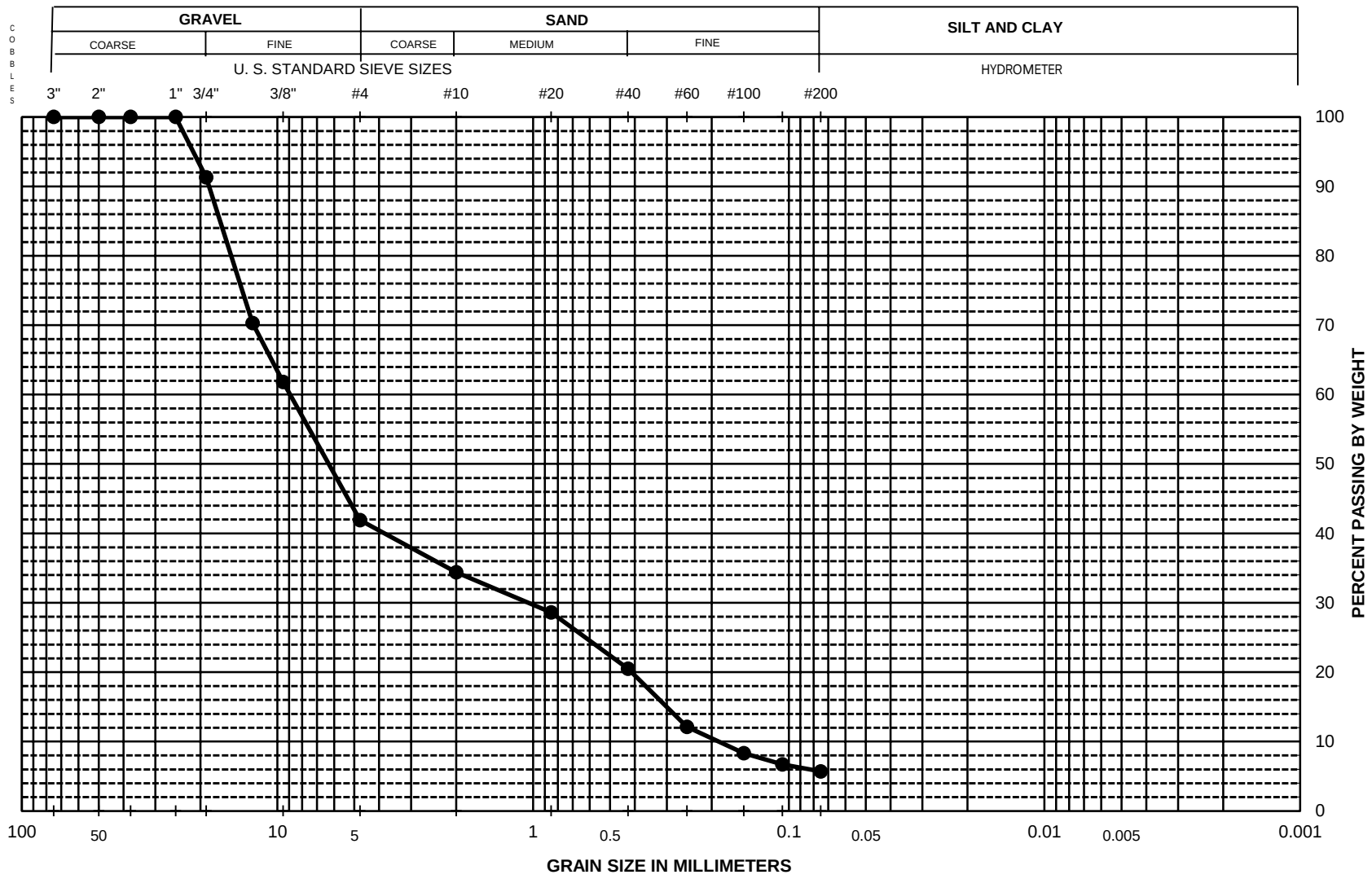
Boring Number	Sample Number	Depth (ft)	Water Content (%)	LL	PI	DESCRIPTION / CLASSIFICATION
BT-4	SS-3	5.0	NA	43	25	Gray Lean CLAY with Sand (CL)

Project Name: **Woodinville Trestle**

Project Number: **60683538**

PLASTICITY CHART

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	91.3
1/2"	12.50	70.3
3/8"	9.50	61.8
#4	4.75	41.9
#10	2.00	34.4
#20	0.850	28.6
#40	0.425	20.5
#60	0.250	12.1
#100	0.150	8.3
#140	0.106	6.7
#200	0.075	5.7
Hydrometer Analysis		

% Cobbles	---
% Gravel	58.1
% Sand	36.2
% Fines	5.7

D_{85}	16.757
D_{60}	8.923
D_{50}	6.298
D_{30}	1.045
D_{15}	0.300
D_{10}	0.189
C_u	47.3
C_c	0.6

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
BT-4	SS-7	25.0	●	7.9	---	---	---	**Dark gray Poorly Graded GRAVEL with Silt and Sand (GP-GM)**

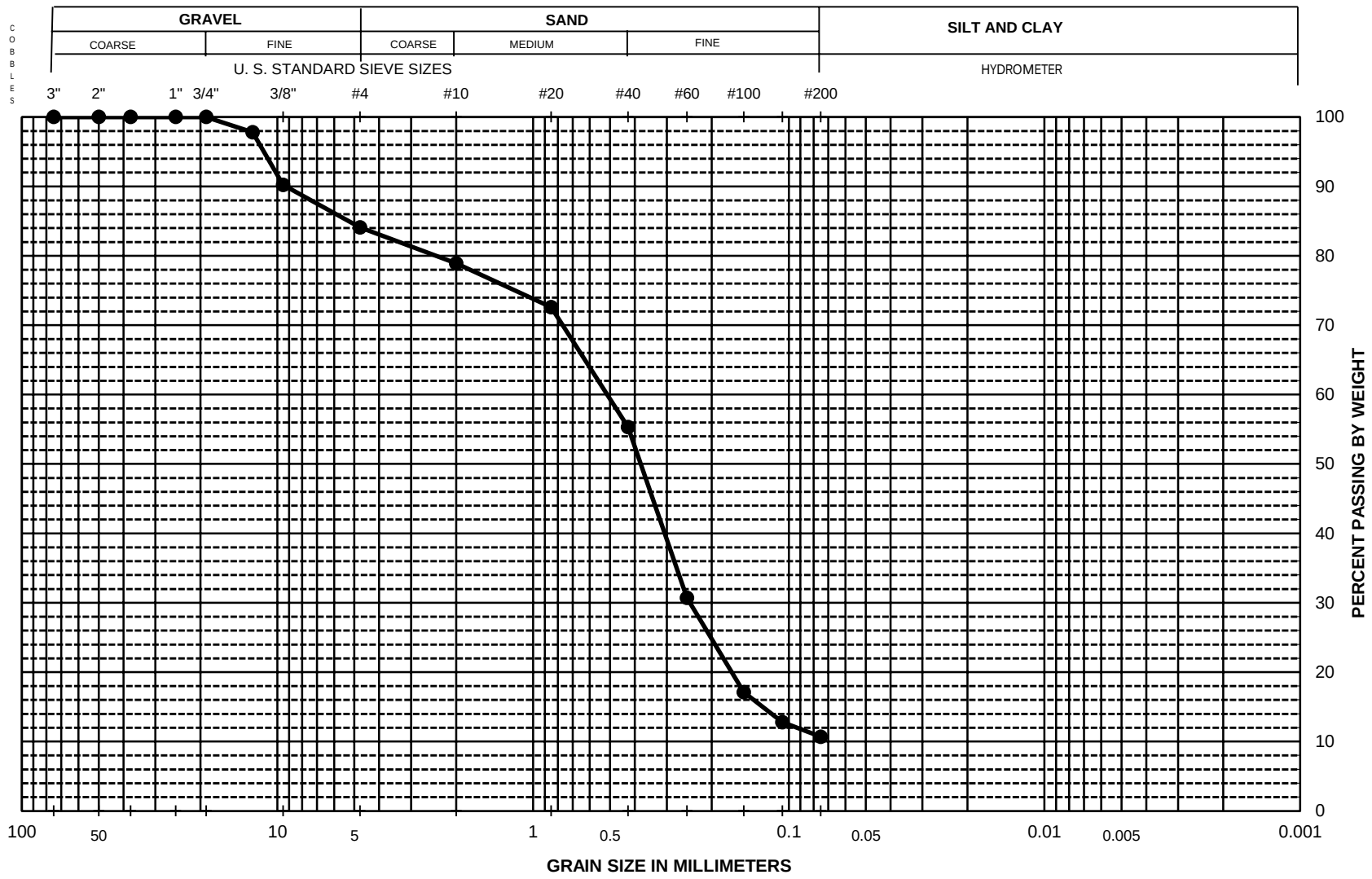
PROJECT NAME: **Woodinville Trestle**

PROJECT NUMBER: **60683538**

PARTICLE-SIZE DISTRIBUTION CURVES

AECOM

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	97.8
3/8"	9.50	90.2
#4	4.75	84.1
#10	2.00	78.9
#20	0.850	72.6
#40	0.425	55.3
#60	0.250	30.7
#100	0.150	17.1
#140	0.106	12.8
#200	0.075	10.7

Hydrometer Analysis

% Cobbles	---
% Gravel	15.9
% Sand	73.4
% Fines	10.7

D_{85}	5.261
D_{60}	0.513
D_{50}	0.379
D_{30}	0.244
D_{15}	0.127
D_{10}	----
C_u	----
C_c	-----

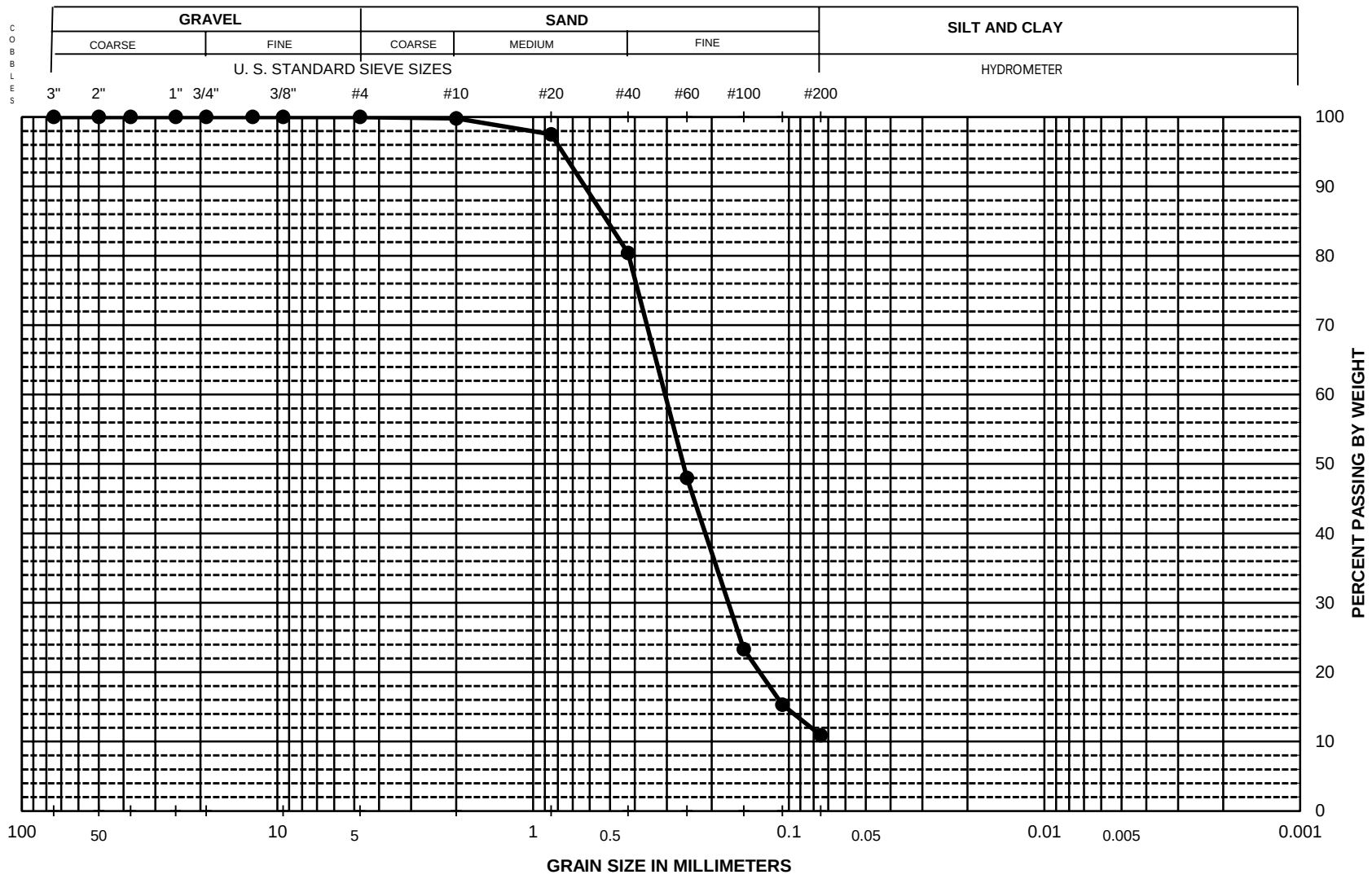
Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
BT-4	SS-10	40.0	●	14.2	---	---	---	**Olive Poorly Graded SAND with Silt and Gravel (SP-SM)**

PROJECT NAME: **Woodinville Trestle**

PROJECT NUMBER: **60683538**

PARTICLE-SIZE DISTRIBUTION CURVES

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	97.5
#40	0.425	80.4
#60	0.250	48.0
#100	0.150	23.3
#140	0.106	15.3
#200	0.075	10.9
Hydrometer Analysis		

% Cobbles	---
% Gravel	0.0
% Sand	89.1
% Fines	10.9

D_{85}	0.512
D_{60}	0.304
D_{50}	0.258
D_{30}	0.172
D_{15}	0.104
D_{10}	----
C_u	----
C_c	-----

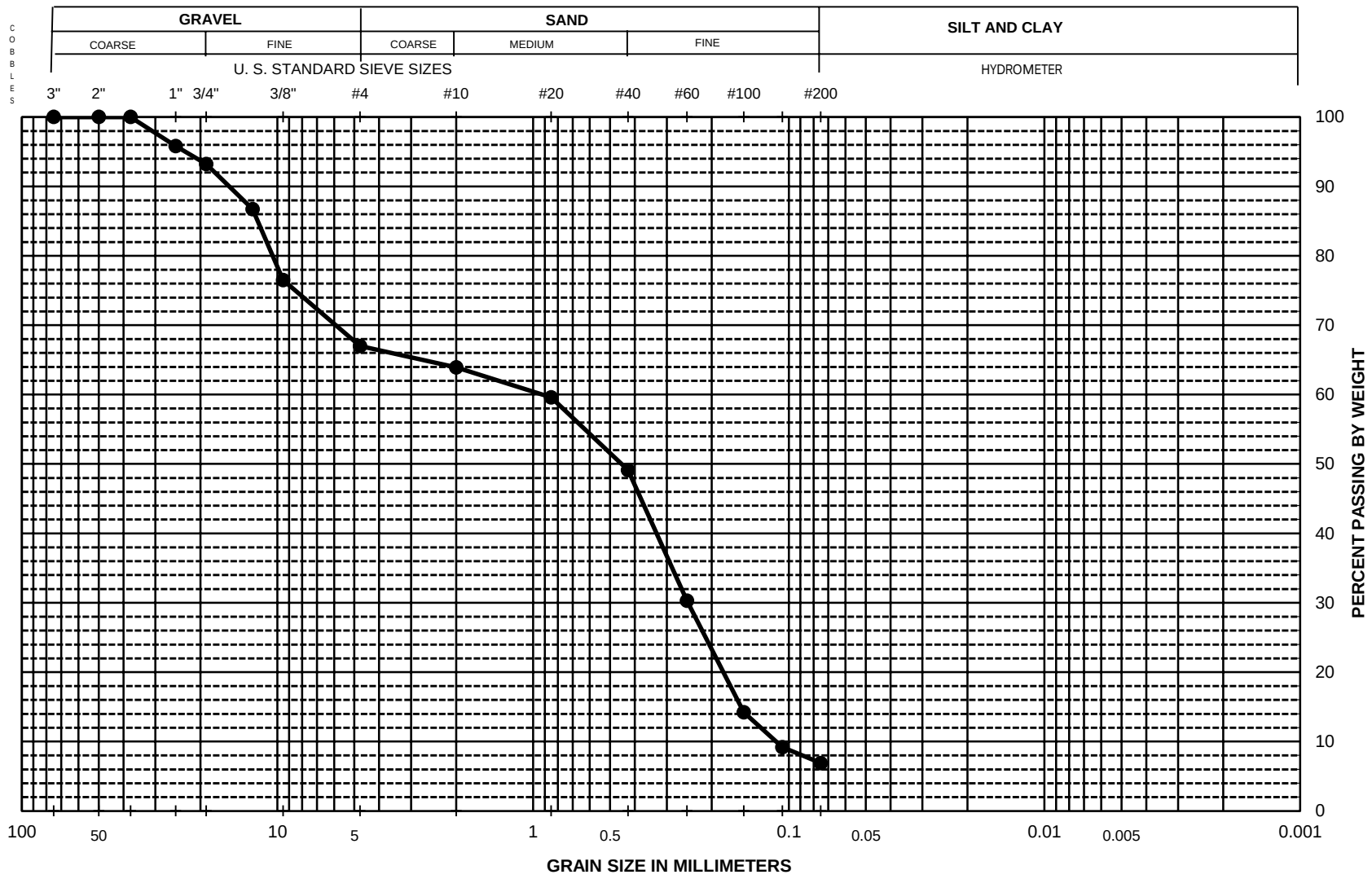
Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
BT-4	SS-17	75.0	●	18.8	---	---	---	**Olive Poorly Graded SAND with Silt and Gravel (SP-SM)**

PROJECT NAME: **Woodinville Trestle**

PROJECT NUMBER: **60683538**

PARTICLE-SIZE DISTRIBUTION CURVES

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	95.8
3/4"	19.00	93.2
1/2"	12.50	86.7
3/8"	9.50	76.5
#4	4.75	67.0
#10	2.00	63.9
#20	0.850	59.6
#40	0.425	49.1
#60	0.250	30.3
#100	0.150	14.2
#140	0.106	9.2
#200	0.075	6.9
Hydrometer Analysis		

% Cobbles	---
% Gravel	33.0
% Sand	60.1
% Fines	6.9

D_{85}	11.941
D_{60}	0.920
D_{50}	0.451
D_{30}	0.248
D_{15}	0.154
D_{10}	0.112
C_u	8.2
C_c	0.6

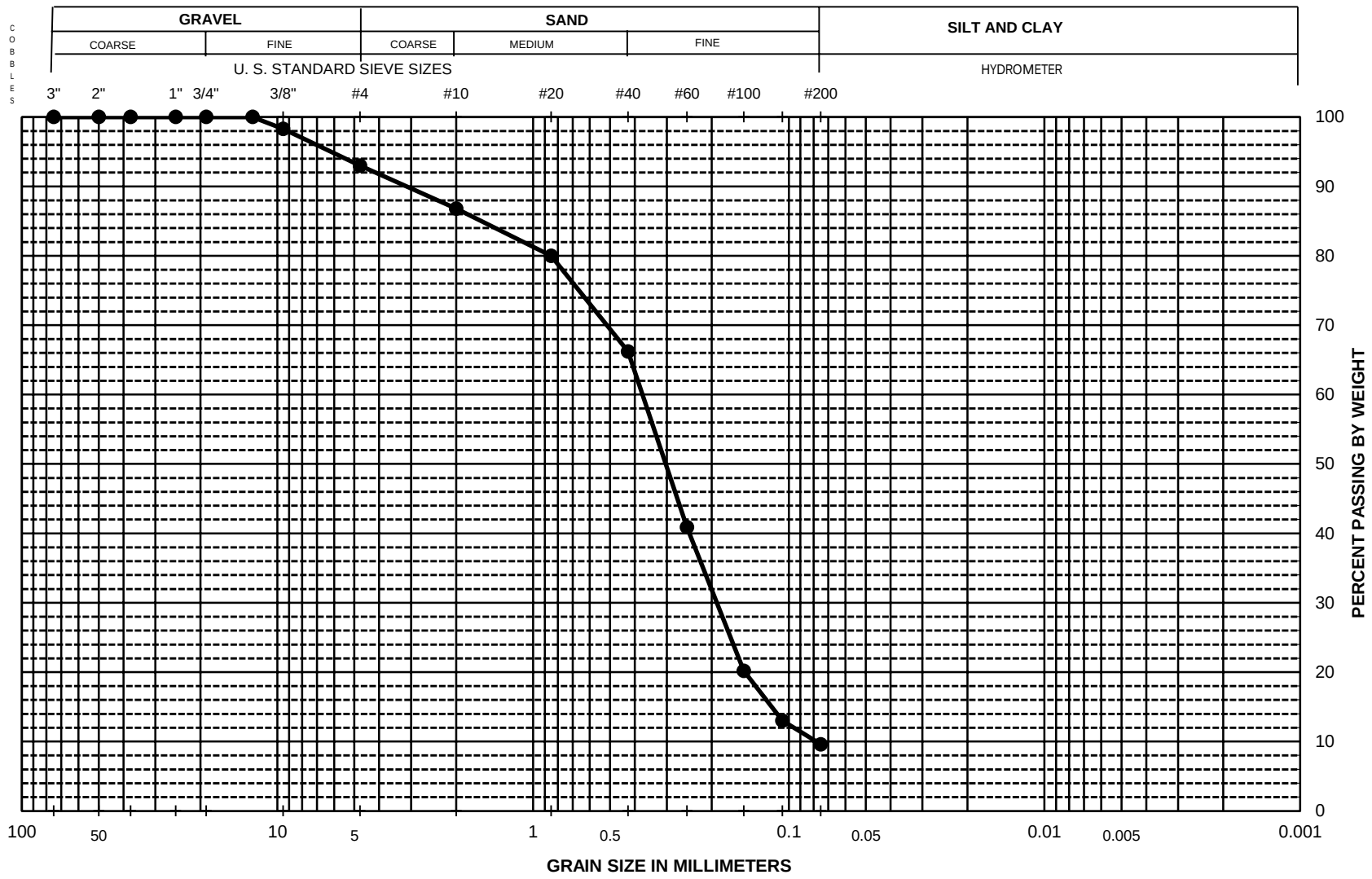
Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
BT-5	MC-6	20.0	●	15.4	---	---	---	**Olive Poorly Graded SAND with Silt and Gravel (SP-SM)**

PROJECT NAME: **Woodinville Trestle**

PROJECT NUMBER: **60683538**

PARTICLE-SIZE DISTRIBUTION CURVES

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	98.3
#4	4.75	93.0
#10	2.00	86.8
#20	0.850	80.0
#40	0.425	66.2
#60	0.250	40.9
#100	0.150	20.2
#140	0.106	13.0
#200	0.075	9.6
Hydrometer Analysis		

% Cobbles	---
% Gravel	7.0
% Sand	83.4
% Fines	9.6

D_{85}	1.595
D_{60}	0.373
D_{50}	0.303
D_{30}	0.191
D_{15}	0.117
D_{10}	0.078
C_u	4.8
C_c	1.3

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
BT-5	SS-9	35.0	●	---	---	---	---	**Olive Poorly Graded SAND with Silt (SP-SM)**

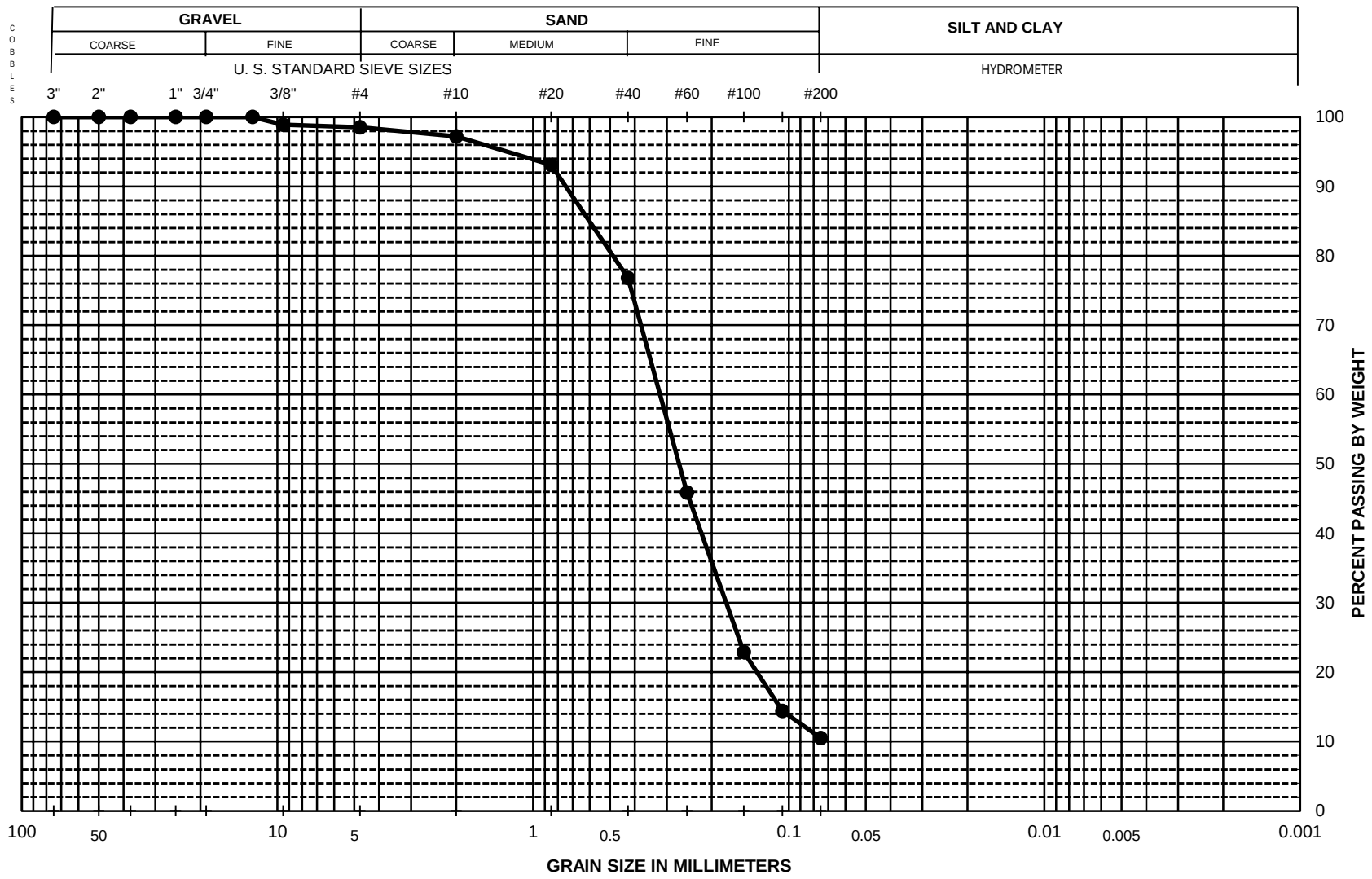
PROJECT NAME: **Woodinville Trestle**

PROJECT NUMBER: **60683538**

PARTICLE-SIZE DISTRIBUTION CURVES

AECOM

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	98.9
#4	4.75	98.5
#10	2.00	97.2
#20	0.850	93.1
#40	0.425	76.8
#60	0.250	45.9
#100	0.150	22.9
#140	0.106	14.4
#200	0.075	10.5
Hydrometer Analysis		

% Cobbles	---
% Gravel	1.5
% Sand	88.0
% Fines	10.5

D_{85}	0.602
D_{60}	0.318
D_{50}	0.268
D_{30}	0.176
D_{15}	0.109
D_{10}	----
C_u	----
C_c	-----

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
BT-5	SS-16	70.0	●	19.9	---	---	---	**Olive Poorly Graded SAND with Silt (SP-SM)**

PROJECT NAME: **Woodinville Trestle**

PROJECT NUMBER: **60683538**

PARTICLE-SIZE DISTRIBUTION CURVES

AECOM

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

Client Name: AECOM
Project Name: Woodinville Trestle
Project No.: 60683538

AP Job No.: 23-0263
Date: 03/01/23

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
BT-4	SS-3	5.0-6.5	Clay	1,725	7.3	66	24
BT-4	SS-7	25.0-26.5	Sand w/silt & gravel	12,702	8.5	51	22
BT-5	MC-7	25.0-26.5	Sand w/silt & gravel	10,736	7.1	53	21

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

APPENDIX E ENVIRONMENTAL INVESTIGATION REPORT

Environmental Investigation Report SR 202 Widening and Trestle Replacement

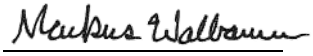
NE 175th Street to NE 177th Street Woodinville Washington

Woodinville Water District

Project number: 60683538

April 17, 2025

Quality information

Prepared by	Checked by	Verified by	Approved by
			
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Revision History

Revision	Revision date	Details	Authorized	Name	Position

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Table of Contents

1. Introduction 1
1.1 Description of Investigation Area 1
2. Objectives and Scope 1
3. Methods of Investigation 2
3.1 Sample Location Rationale 2
3.2 Drilling and Soil Sampling Procedures 2
3.3 Analytical Methods and Applicable Cleanup Levels 2
4. Investigation Findings 3
4.1 Subsurface Conditions 3
4.2 Analytical Results 3
4.2.1 Soil 3
4.2.2 Groundwater 3
4.2.3 Conclusions and Recommendations 4
5. References 5

Tables

Table 1 – Summary of Soil Analytical Results
Table 2 – Summary of Groundwater Analytical Results

Figures

Figure 1 – Site Location
Figure 2 – Site Vicinity

Appendices

Appendix A – Utility Locate Field Report
Appendix B – Photographic Log
Appendix C – Boring Logs
Appendix D – Laboratory Analytical Reports
Appendix E – Laboratory Data Quality Review
Appendix F – Sanitary Sewer Plan and Profile - Area of PCE Media Management

1. Introduction

AECOM Technical Services, Inc. (AECOM) was retained by Woodinville Water District (WWD) to perform a limited environmental investigation in the City of Woodinville, Washington, to assess for the presence of soil and groundwater contamination that could be encountered during proposed construction activities, conducted along 131st Avenue NE and near NE 175th Street, an auto repair and service station, and a gas station (Figure 1). This investigation was conducted in conjunction with a geotechnical exploration and groundwater level investigation, as part of engineering design services for the SR 202 Widening and Trestle Replacement project. This project includes upsizing and replacement of an existing sanitary sewer trunk line with a new line along 131st Avenue NE from NE 175th Street to NE 177th Place. This report presents background information regarding the area of investigation and proposed construction, the purpose and scope of the subsurface investigation, and our findings and conclusions. The findings of this report apply to sanitary sewer and other construction associated with the SR 202 Widening and Trestle Replacement project.

1.1 Description of Investigation Area

The area of investigation was along 131st Avenue NE from NE 175th Street to NE 177th Place in Woodinville, King County, Washington and was located in the city right-of-way, adjacent to an auto repair and service center (Pep Boys; 12978 NE 175th Street), and gas station (Jacksons Shell; 13106 NE 175th Street), as shown on Figure 2.

Known and suspected contaminated sites adjacent to the area of proposed sanitary sewer upsizing and replacement were evaluated in advance of this investigation. The Pep Boys auto service center is not a known or suspected contaminated site but is located hydraulically downgradient from the Jacksons Shell site. The Jacksons Shell site had historical documented releases of gasoline, which impacted soil and groundwater in excess of Washington State Department of Ecology (Ecology) cleanup levels for total petroleum hydrocarbons as gasoline (TPH-G), benzene, toluene, ethylbenzene, and xylenes (BTEX). These historic releases were reportedly remediated by excavation and Ecology subsequently issued a no-further action determination for the property in 2012 (Ecology, 2012). Based upon the known historic releases of gasoline at the Jacksons Shell property, further evaluation for the presence of gasoline in soils and groundwater in proposed adjacent areas of excavation and construction was conducted.

The regional geology in this area consists of Quaternary alluvial deposits that include unconsolidated or semi-consolidated alluvial clay, silt, sand, gravel, and (or) cobble deposits and locally may include peat and muck. Groundwater in this area is present at shallow depths within the unconsolidated Sammamish River floodplain deposits and has been encountered between approximately 9 and 12 feet below ground surface (bgs). Regional groundwater flow is to the west and northwest towards the Sammamish River (Figure 1).

2. Objectives and Scope

AECOM was retained by WWD to conduct a geotechnical and environmental investigation. The objectives of the environmental investigation were to assess the soil and groundwater conditions within geotechnical borings. To accomplish these objectives, AECOM implemented the following scope of work:

- Conducted field screening of three borings (BT-1, BT-2, and BT-3) and collected soil and groundwater samples for chemical analysis. Soil samples were field screened using a photoionization detector (PID) and for evidence of contamination (e.g., sheen, odor and/or discoloration). Soil samples were collected from all three borings and groundwater samples were collected from temporary wells established in Boring BT-1 and BT-3.
- Analyzed select soil and groundwater samples for gasoline-range hydrocarbons by NWTPH-Gasoline extended (Gx), diesel- and oil-range petroleum hydrocarbons by NWTPH-Diesel extended (Dx), and volatile organic compounds (VOCs) by EPA Method 8260D.

- Prepared this report presenting our findings and conclusions regarding the environmental testing conducted.

3. Methods of Investigation

3.1 Sample Location Rationale

Two borings (BT-1 and BT-2) were advanced on the east side of 131st Avenue NE, off the street and in the accessway to the Jacksons Shell (Figure 2). The locations of Borings BT-1 and BT-2 are within the City of Woodinville right-of-way. Boring BT-3 was advanced on the west side of 131st Avenue NE, in a driveway to the Pep Boys service center. Boring BT-3 was advanced near the edge of the City of Woodinville right-of-way.

3.2 Drilling and Soil Sampling Procedures

AECOM retained Applied Professional Services, Inc. (APS) to perform GPR and EM surveys to mark out anomalies and utilities identified in the vicinity of the proposed boring locations in advance of the drilling operations. The survey was performed on January 4, 2023 and a copy of APS's field report is provided in Appendix A.

Holt Services, Inc. (Holt) provided the drilling and sampling-related services from February 6 through 8, 2023. Prior to initiating drilling activities, each boring location was excavated to a minimum depth of 5 ft below ground surface (bgs) with air knifing/vacuum excavation to clear for utilities. Borings BT-1, BT-2 and BT-3 were advanced to 16.5, 21.1, and 16.5 feet bgs, respectively, using hollow stem auger drilling methods. Soil samples were collected every 2.5 feet using a 2-inch-diameter split spoon to the total depth of the boring. Photographs of the drilling operation are provided in Appendix B. Two soil samples were collected from each boring for laboratory chemical analysis. Soils were logged by an AECOM geologist in accordance with the Unified Soil Classification System (USCS). Particular attention was given to noting visible evidence of contamination such as staining, odors, or other relevant factors indicative of the presence of contaminants of concern. A field log of each boring was prepared noting soil type, soil color, moisture (density, stiffness, and plasticity as applicable), and PID readings. Headspace measurements for organic vapors in soil samples were made using a PID with a standard 10.6 electron volt lamp. The highest PID readings were recorded for each headspace sample. The boring logs are provided in Appendix C.

Volatile organics samples were collected using a 5-gram (g) EnCore® sampler (EPA Method 5035). Soil samples were transferred to appropriate laboratory-provided sample containers. Each sample was properly labeled with a unique sample identification number, placed in a cooler with ice, and submitted to the laboratory for analysis. Chain-of-custody forms were completed and signed by the field representative and accompanied the samples to the laboratory.

As part of the investigation, two of the borings (BT-1 and BT-3) were converted into temporary 2-inch-diameter monitoring wells that were screened from 5 to 15 feet bgs, to obtain static groundwater levels and collect groundwater samples. The two temporary wells were purged by pumping up to one hour to allow turbidity readings to stabilize prior to sample collection. The two temporary monitoring wells were then sampled using a peristaltic pump and new polyethylene tubing. Following sample collection, Borings BT-1 and BT-3 were backfilled per state regulations and patched to match surrounding conditions.

Boring BT-2 was converted to an observation well with a flush mounted monument for subsequent collection of groundwater elevations, which were conducted as part of the geotechnical evaluation.

Drilling spoils (soil and water) were drummed, profiled, transported, and disposed of at an approved disposal facility as non-hazardous investigative derived waste.

3.3 Analytical Methods and Applicable Cleanup Levels

Soil and groundwater samples were submitted for analysis to Fremont Analytical, an Ecology-accredited laboratory located in Seattle, Washington. The analytical program included analyses for gasoline, diesel,

and heavy oil-range petroleum hydrocarbons by NWTPH-Gx and Dx, respectively, and VOCs by Method 8260D. The soil analytical results are summarized in Table 1, along with their respective Ecology Model Toxic Control Act (MTCA) Method A or B cleanup levels. The lowest cleanup level for each compound was used for comparison. The MTCA Method A or B values used were obtained from Ecology's CLARC Database (Ecology, 2023). The laboratory analytical reports are provided in Appendix D. An AECOM project chemist performed a data quality review of all analytical data, and no data usability issues were identified. A summary of the laboratory data quality review is provided in Appendix E.

4. Investigation Findings

4.1 Subsurface Conditions

The borings encountered asphalt and fill material, characterized by dark grey to dark brown silty sand with gravel, from ground surface to about 1 foot bgs. The fill was underlain by fine to medium grained, loose to very dense sands with trace gravel to boring termination depth in each of the borings (16.5 to 21.1 feet bgs).

During field screening, no petroleum odors or staining were noted in the soils from any boring and PID readings were all below 0.5 ppm, which was considered representative of ambient air (background) concentrations.

Groundwater was noted during drilling at approximately 10.7, 10.5, and 7 feet bgs in boring BT-1, BT-2, and BT-3, respectively. Following drilling, soil boring BT-2 (Figure 2) was converted into a monitoring well, which was screened from 10 to 20 feet bgs (Appendix C). Static groundwater levels were taken by AECOM from Well BT-2 five times from February 8 through May 9, 2023. The static groundwater levels during these four monitoring events ranged from 9.26 to 10.50 feet below the top of well casing, with the top of casing set 0.3 ft bgs.

4.2 Analytical Results

4.2.1 Soil

A total of six soil samples (two samples per boring) were submitted to the laboratory for chemical analysis. The results are summarized in Table 1. Soils were analyzed from depths of 8 and 12 feet bgs in BT-1, 7 and 10 feet bgs in BT-2, and 5.5 and 10 feet bgs in BT-3. The shallower depth samples from each boring were collected from soils in the unsaturated zone, above the groundwater interface. The deeper samples from each boring were collected just below the groundwater interface, in the saturated zone.

There were no detections of gasoline-range, diesel-range, or oil-range petroleum hydrocarbons in any of the six soil samples analyzed and the reporting limits for these analytes were below their respective Ecology Model Toxic Control Act (MTCA) Method A or B cleanup levels. While most VOCs were not detected above their associated reporting limits, one VOC, tetrachloroethene (PCE) was detected in Boring BT-3, at a depth of 10 feet BGS, in the saturated zone. PCE was detected at a concentration of 0.0326 mg/kg, which is below the associated MTCA Method A cleanup level of 0.05 mg/kg.

4.2.2 Groundwater

The groundwater sampling results from temporary monitoring wells BT-1 and BT-3 are summarized in Table 2.

Gasoline-range petroleum hydrocarbons were not detected in groundwater samples collected from the two wells. Diesel-range petroleum hydrocarbons were detected in BT-1, at a concentration of 147 µg/L, which is below the associated MTCA Method A cleanup level of 500 µg/L. The laboratory noted this detection to be an unresolved complex mixture, which may be weathered and/or organic material and may not be from a diesel source. Oil-range petroleum hydrocarbons were detected in BT-3 at a concentration of 123 µg/L, which is below the associated MTCA Method A cleanup level of 500 µg/L. This

detection was noted by the laboratory as not being consistent with a petroleum standard and may not be from a heavy oil source.

Two VOCs, PCE and toluene, were detected in BT-3. PCE was detected in BT-3 at a concentration of 3.53 µg/L, which is below the associated MTCA Method A cleanup level of 5 µg/L. Toluene was detected in BT-3 at a concentration of 4.82 µg/L, which is below the associated MTCA Method A cleanup level of 1,000 µg/L.

4.2.3 Conclusions and Recommendations

This environmental investigation was conducted to assess for the potential presence of soil and groundwater contamination that could be encountered during proposed SR 202 Widening and Trestle Replacement project activities, including proposed sanitary sewer construction near NE 175th Street, the Pep Boy's auto repair and service station, and the Jacksons Shell gas station in Woodinville, Washington (Figure 2 and Appendix F). AECOM has drawn the following conclusions based on the results of this environmental investigation:

- Fill material was encountered in the three soil borings to about 1 foot bgs, with native sands with limited gravel below.
- Field screening showed no indication of soil contamination to boring completion depths of 16.5 (BT-1 and BT-3) and 21.1 (BT-2) feet bgs.
- There were no laboratory detections of VOCs, gasoline-range, diesel-range, or oil-range petroleum hydrocarbons in soil in the two borings (BT-1 and BT-2) completed adjacent to the Jacksons Shell gas station and in shallow (5.5 feet bgs), unsaturated soils in the boring (BT-3) completed adjacent to Pep Boy's
- A single VOC, PCE, was detected in deeper (10 feet bgs), saturated, soils in Boring BT-3, at a concentration of 0.0326 mg/kg, below the associated MTCA Method A cleanup level of 0.05 mg/kg.
- Groundwater was encountered during drilling between 7 to 10.7 feet bgs and static groundwater levels in a boring converted into a monitoring well (BT-2) ranged from approximately 9.26 to 10.50 feet bgs over the next three months (February through May 2023). The inferred groundwater gradient is west to northwesterly, towards the Sammamish River.
- Groundwater samples collected from BT-1 and BT-3 did not detect petroleum-range hydrocarbons above MTCA Method A cleanup levels. A low-level detection of diesel-range hydrocarbons (147 µg/L) in Boring BT-1 was noted as being weathered/or organic in nature. A low-level detection of oil-range petroleum hydrocarbons (123 µg/L) in BT-3 was noted as being inconsistent with heavy oil. Any groundwater generated during construction activities, such as for dewatering, should be evaluated for concentrations of diesel and heavy oil and accounted for in transportation and disposal.
- Two low level detections of VOCs in groundwater (PCE at 3.53 µg/L and Toluene at 4.82) were identified in BT-3, located adjacent to the Pep Boy's service center, at concentrations below applicable MTCA cleanup levels. While the toluene detection was well below the associated MTCA Method A cleanup level of 1,000 µg/L, the 3.53 µg/L detection of PCE in BT-3 was near the associated MTCA Method A cleanup level of 5 µg/L. Toluene and PCE are both solvents, commonly used in industrial, automotive, and dry-cleaning industry.
- Evidence of contamination was not detected in shallow soils above the groundwater table in the vicinity of the three borings. Evidence of contamination was also not detected in groundwater or in soils within the saturated zone on the east side of 131st Ave NE and these media are not suspected to be contaminated with PCE or petroleum hydrocarbons. However, based on the proximity of the project area to the documented former release of gasoline to soil at the Jacksons Shell property, contingencies should be evaluated in case petroleum impacted soils are encountered during the project. Soils to be excavated during construction from the unsaturated zone, and saturated zone on the east side of 131st Ave NE, appear to fall within Ecology's Soil Category 1 for best management practices. These soils would not require special handling or disposal at a landfill and would be suitable for reuse as backfill within the project area, assuming the soils meet with other project

requirements (e.g., geotechnical). Soils within the saturated zone may be contaminated with PCE along the west side of 131st Ave NE, as depicted in the two sheets included in Appendix F.

- Soils excavated from within the saturated zone and/or construction dewatering occurring near Boring BT-3 and on the west side of 131st Ave NE should be evaluated for appropriate media management (i.e., laboratory testing and waste profiling/disposal) due to the potential presence of PCE contamination. The lateral extent of where removed media (soil and groundwater) should be evaluated for PCE contamination is estimated in Attachment F.
- If staining or odors are observed from soils excavated during construction, it is recommended that soils be stockpiled, sampled, and laboratory analyses performed prior to proceeding with soil disposal.

5. References

Washington State Department of Ecology (Ecology), 2012. Re: No Further Action at the following Site: Site Name Jackson. VCP Project No.: NW2518. December 17, 2012.

Ecology, 2023. CLARC tables. January, 2023. (https://ecology.wa.gov/Regulations-Permits/Guidance-technical-assistance/Contamination-clean-up-tools/CLARC/Data-tables)

Tables

Table 1. Summary of Soil Analytical Results
Environmental Investigation Report
SR 202 Widening and Trestle Replacement

Sample Location:	MTCA Soil	BT-1		BT-2		BT-3	
Sample Depth (feet bgs):	Screening Levels	8	12	7	10	5.5	10
Sample Date:		2/8/2023	2/8/2023	2/8/2023	2/8/2023	2/7/2023	2/7/2023
VOCs (mg/kg)							
1,1,1,2-Tetrachloroethane	38 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
1,1,1-Trichloroethane (TCA)	2 (A)	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
1,1,2,2-Tetrachloroethane	5 (B)	0.185 U	0.179 U	0.194 U	0.200 U	0.194 U	0.177 U
1,1,2-Trichloroethane	18 (B)	0.0116 U	0.0112 U	0.0121 U	0.0125 U	0.0121 U	0.0111 U
1,1-Dichloroethane	180 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
1,1-Dichloroethene	4,000 (B)	0.0927 U	0.0897 U	0.0971 U	0.0999 U	0.0969 U	0.0886 U
1,1-Dichloropropene	NE	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
1,2,3-Trichlorobenzene	64 (B)	0.0556 U	0.0538 U	0.0582 U	0.0600 U	0.0581 U	0.0531 U
1,2,3-Trichloropropane	0.0063 (B)	0.0278 U	0.0269 U	0.0291 U	0.0300 U	0.0291 U	0.0266 U
1,2,4-Trichlorobenzene	34 (B)	0.0556 U	0.0538 U	0.0582 U	0.0600 U	0.0581 U	0.0531 U
1,2,4-Trimethylbenzene	800 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
1,2-Dibromo-3-Chloropropane	0.23 (B)	0.0278 U	0.0269 U	0.0291 U	0.0300 U	0.0291 U	0.0266 U
1,2-Dibromoethane (EDB)	0.005 (A)	0.00927 U	0.00897 U	0.00971 U	0.00999 U	0.00969 U	0.00886 U
1,2-Dichlorobenzene	7,200 (B)	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
1,2-Dichloroethane (EDC)	11 (B)	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
1,2-Dichloropropane	27 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
1,3,5-Trimethylbenzene	800 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
1,3-Dichlorobenzene	NE	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
1,3-Dichloropropane	1,600 (B)	0.00927 U	0.00897 U	0.00971 U	0.00999 U	0.00969 U	0.00886 U
1,4-Dichlorobenzene	190 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
2-Butanone (MEK)	48,000 (B)	0.278 U	0.269 U	0.291 U	0.300 U	0.291 U	0.266 U
2-Chlorotoluene	1,600 (B)	0.0153 U	0.0148 U	0.0160 U	0.0165 U	0.0160 U	0.0146 U
2-Hexanone (MBK)	400 (B)	0.0579 U	0.0561 U	0.0607 U	0.0624 U	0.0606 U	0.0554 U
4-Chlorotoluene	1,600 (B)	0.0153 U	0.0148 U	0.0160 U	0.0165 U	0.0160 U	0.0146 U
4-Isopropyltoluene	NE	0.185 U	0.179 U	0.194 U	0.200 U	0.194 U	0.177 U
Acetone	72,000 (B)	0.232 U	0.224 U	0.243 U	0.250 U	0.242 U	0.221 U
Benzene	0.03 (A)	0.0162 U	0.0157 U	0.0170 U	0.0175 U	0.0170 U	0.0155 U
Bromobenzene	640 (B)	0.0116 U	0.0112 U	0.0121 U	0.0125 U	0.0121 U	0.0111 U
Bromodichloromethane	16 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
Bromoform	130 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
Bromomethane	110 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
Carbon Tetrachloride	14 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
Chlorobenzene	1,600 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
Chloroethane	NE	0.0695 UJ	0.0673 UJ	0.0728 UJ	0.0749 UJ	0.0727 UJ	0.0664 UJ
Chloroform	32 (B)	0.0162 U	0.0157 U	0.0170 U	0.0175 U	0.0170 U	0.0155 U
Chloromethane	NE	0.0464 U	0.0449 U	0.0485 U	0.0500 U	0.0484 U	0.0443 U
cis-1,2-Dichloroethene	160 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
cis-1,3-Dichloropropene	10 (B)[1]	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
Dibromochloromethane	12 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
Dibromomethane	800 (B)	0.0116 U	0.0112 U	0.0121 U	0.0125 U	0.0121 U	0.0111 U
Dichlorodifluoromethane (CFC-12)	16,000 (B)	0.0139 UJ	0.0135 UJ	0.0146 UJ	0.0150 UJ	0.0145 UJ	0.0133 UJ
Ethylbenzene	6 (A)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
Hexachlorobutadiene	13 (B)	0.0371 U	0.0359 U	0.0388 U	0.0400 U	0.0388 U	0.0354 U
Isopropylbenzene	8,000 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
m,p-Xylene	16,000 (B)	0.0464 U	0.0449 U	0.0485 U	0.0500 U	0.0484 U	0.0443 U
Methyl isobutyl ketone (MIBK)	6,400 (B)	0.0556 U	0.0538 U	0.0582 U	0.0600 U	0.0581 U	0.0531 U
Methyl tert-butyl ether (MTBE)	0.1 (A)	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
Methylene Chloride	0.02 (A)	0.0324 U	0.0314 U	0.034 U	0.0350 U	0.0339 U	0.0310 U
Naphthalene	5 (A)	0.0927 U	0.0897 U	0.0971 U	0.0999 U	0.0969 U	0.0886 U
n-Butylbenzene	4,000 (B)	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
n-Propylbenzene	8,000 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
o-Xylene	16,000 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
sec-Butylbenzene	8,000 (B)	0.139 U	0.135 U	0.146 U	0.150 U	0.145 U	0.133 U
Styrene	16,000 (B)	0.00927 U	0.00897 U	0.00971 U	0.00999 U	0.00969 U	0.00886 U
tert-Butylbenzene	8,000 (B)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
Tetrachloroethene (PCE)	0.05 (A)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	**0.0326**
Toluene	7 (A)	0.0278 U	0.0269 U	0.0291 U	0.0300 U	0.0291 U	0.0266 U
trans-1,2-Dichloroethene	1,600 (B)	0.00927 U	0.00897 U	0.00971 U	0.00999 U	0.00969 U	0.00886 U
trans-1,3-Dichloropropylene	10 (B)[1]	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
Trichloroethene (TCE)	0.03 (A)	0.0139 U	0.0135 U	0.0146 U	0.0150 U	0.0145 U	0.0133 U
Trichlorofluoromethane (CFC-11)	24,000 (B)	0.0185 U	0.0179 U	0.0194 U	0.0200 U	0.0194 U	0.0177 U
Vinyl Chloride	0.67 (B)	0.0232 U	0.0224 U	0.0243 U	0.0250 U	0.0242 U	0.0221 U
TPHs (mg/kg)							
Gasoline-Range	30 / 100 (A)[2]	4.64 U	4.49 U	4.85 U	5.00 U	4.84 U	4.43 U
Diesel-Range	2,000 (A)	57.3 U	60.8 U	51.1 U	59.2 U	54.0 U	59.6 U
Heavy Oil-Range	2,000 (A)	115 U	122 U	102 U	118 U	108 U	119 U

Notes:

Values in **bold** font indicate that the analyte was detected.

Model Toxics Control Act (MTCA) Cleanup Regulation, chapter 173-340 WAC; MTCA Method A and B from Ecology website CLARC tables posted January 2023 (https://ecology.wa.gov/Regulations-Permits/Guidance-technical-assistance/Contamination-clean-up-tools/CLARC/Data-tables).

(A) - MTCA Method A

(B) - MTCA Method B

MTCA Method A screening levels are presented when available. MTCA Method B values are presented when MTCA Method A values are not established.

[1]The screening level is for 1,3-dichloropropene.

[2]The MTCA Method A screening level is 30 mg/kg if benzene is present and 100 mg/kg if benzene is not present.

bgs - below ground surface

mg/kg - milligram per kilogram

NE - not established

U - Compound was analyzed for but not detected above the reporting limit shown.

UJ - Compound was qualified as estimated at the reporting limit shown.

Table 2. Summary of Groundwater Analytical Results
Environmental Investigation Report
SR 202 Widening and Trestle Replacement

Sample Location: / Sample Date:	MTCA Groundwater Screening Levels	BT-1-GW 2/8/2023	BT-3-GW 2/7/2023
VOCs (ug/L)			
1,1,1,2-Tetrachloroethane	1.7 (B)	0.300 U	0.300 U
1,1,1-Trichloroethane (TCA)	200 (A)	0.300 U	0.300 U
1,1,2,2-Tetrachloroethane	0.22 (B)	0.200 U	0.200 U
1,1,2-Trichloroethane	0.77 (B)	0.250 U	0.250 U
1,1-Dichloroethane	7.7 (B)	0.500 U	0.500 U
1,1-Dichloroethene	400 (B)	0.500 U	0.500 U
1,1-Dichloropropene	NE	0.500 U	0.500 U
1,2,3-Trichlorobenzene	6.4 (B)	0.700 U	0.700 U
1,2,3-Trichloropropane	0.00038 (B)	0.400 U	0.400 U
1,2,4-Trichlorobenzene	1.5 (B)	0.750 U	0.750 U
1,2,4-Trimethylbenzene	80 (B)	0.500 U	0.500 U
1,2-Dibromo-3-Chloropropane	0.014 (B)	1.00 U	1.00 U
1,2-Dibromoethane (EDB)	0.01 (A)	0.200 U	0.200 U
1,2-Dichlorobenzene	720 (B)	0.500 U	0.500 U
1,2-Dichloroethane (EDC)	5 (A)	0.500 U	0.500 U
1,2-Dichloropropane	1.2 (B)	0.300 U	0.300 U
1,3,5-Trimethylbenzene	80 (B)	0.500 U	0.500 U
1,3-Dichlorobenzene	NE	0.500 U	0.500 U
1,3-Dichloropropane	160 (B)	0.300 U	0.300 U
1,4-Dichlorobenzene	8.1 (B)	0.500 U	0.500 U
2-Butanone (MEK)	4,800 (B)	1.50 U	1.50 U
2-Chlorotoluene	160 (B)	0.500 U	0.500 U
2-Hexanone (MBK)	40 (B)	1.25 U	1.25 U
4-Chlorotoluene	160 (B)	0.500 U	0.500 U
4-Isopropyltoluene	NE	0.500 U	0.500 U
Acetone	7,200 (B)	5.00 U	5.00 U
Benzene	5 (A)	0.440 U	0.440 U
Bromobenzene	64 (B)	0.500 U	0.500 U
Bromodichloromethane	0.71 (B)	0.250 U	0.250 U
Bromoform	5.5 (B)	0.300 U	0.300 U
Bromomethane	11 (B)	3.00 U	3.00 U
Carbon Tetrachloride	0.63 (B)	0.300 U	0.300 U
Chlorobenzene	160 (B)	0.500 U	0.500 U
Chloroethane	NE	1.00 U	1.00 U
Chloroform	1.4 (B)	0.500 U	0.500 U
Chloromethane	NE	0.750 U	0.750 U
cis-1,2-Dichloroethene	16 (B)	0.500 U	0.500 U
cis-1,3-Dichloropropene	0.44 (B)[a]	0.350 U	0.350 U
Dibromochloromethane	0.52 (B)	0.300 U	0.300 U
Dibromomethane	80 (B)	0.250 U	0.250 U
Dichlorodifluoromethane (CFC-12)	1,600 (B)	0.500 U	0.500 U
Ethylbenzene	700 (A)	0.400 U	0.400 U
Hexachlorobutadiene	0.56 (B)	0.500 U	0.500 U
Isopropylbenzene	800 (B)	0.500 U	0.500 U
m,p-Xylene	1,600 (B)	1.00 U	1.00 U
Methyl isobutyl ketone (MIBK)	640 (B)	1.00 U	1.00 U
Methyl tert-butyl ether (MTBE)	20 (A)	0.350 U	0.350 U
Methylene Chloride	5 (A)	0.750 U	0.750 U
Naphthalene	160 (A)	1.25 U	1.25 U
n-Butylbenzene	400 (B)	0.500 U	0.500 U
n-Propylbenzene	800 (B)	0.500 U	0.500 U
o-Xylene	1,600 (B)	0.500 U	0.500 U
sec-Butylbenzene	800 (B)	0.500 U	0.500 U
Styrene	1,600 (B)	0.500 U	0.500 U
tert-Butylbenzene	800 (B)	0.500 U	0.500 U
Tetrachloroethene (PCE)	5 (A)	0.350 U	**3.53**
Toluene	1,000 (A)	1.00 U	**4.82**
trans-1,2-Dichloroethene	160 (B)	0.350 U	0.350 U
trans-1,3-Dichloropropylene	0.44 (B)[a]	0.500 U	0.500 U
Trichloroethene (TCE)	5 (A)	0.400 U	0.400 U
Trichlorofluoromethane (CFC-11)	2,400 (B)	0.300 U	0.300 U
Vinyl Chloride	0.2 (A)	0.200 U	0.200 U
TPHs (ug/L)			
Gasoline Range Organics	800 / 1,000 (A)[b]	50.0 U	50.0 U
Diesel Range Organics	500 (A)	**147**[c]	97.7 U
Heavy Oil-Range	500 (A)	95.3 U	**123**[d]

Notes:

Values in **bold** font indicate that the analyte was detected.

Model Toxics Control Act (MTCA) Cleanup Regulation, chapter 173-340 WAC; MTCA Method A and B from Ecology website CLARC tables posted January 2023 (https://ecology.wa.gov/Regulations-Permits/Guidance-technical-assistance/Contamination-clean-up-tools/CLARC/Data-tables).

(A) - MTCA Method A

(B) - MTCA Method B

MTCA Method A screening levels are presented when available. MTCA Method B values are presented when MTCA Method A values are not established.

[a]The screening level is for 1,3-dichloropropene.

[b]The MTCA Method A screening level is 800 ug/L if benzene is present and 1,000 ug/L if benzene is not present.

[c]Chromatographic pattern indicates an unresolved complex mixture, which may be weathered and/or organic material.

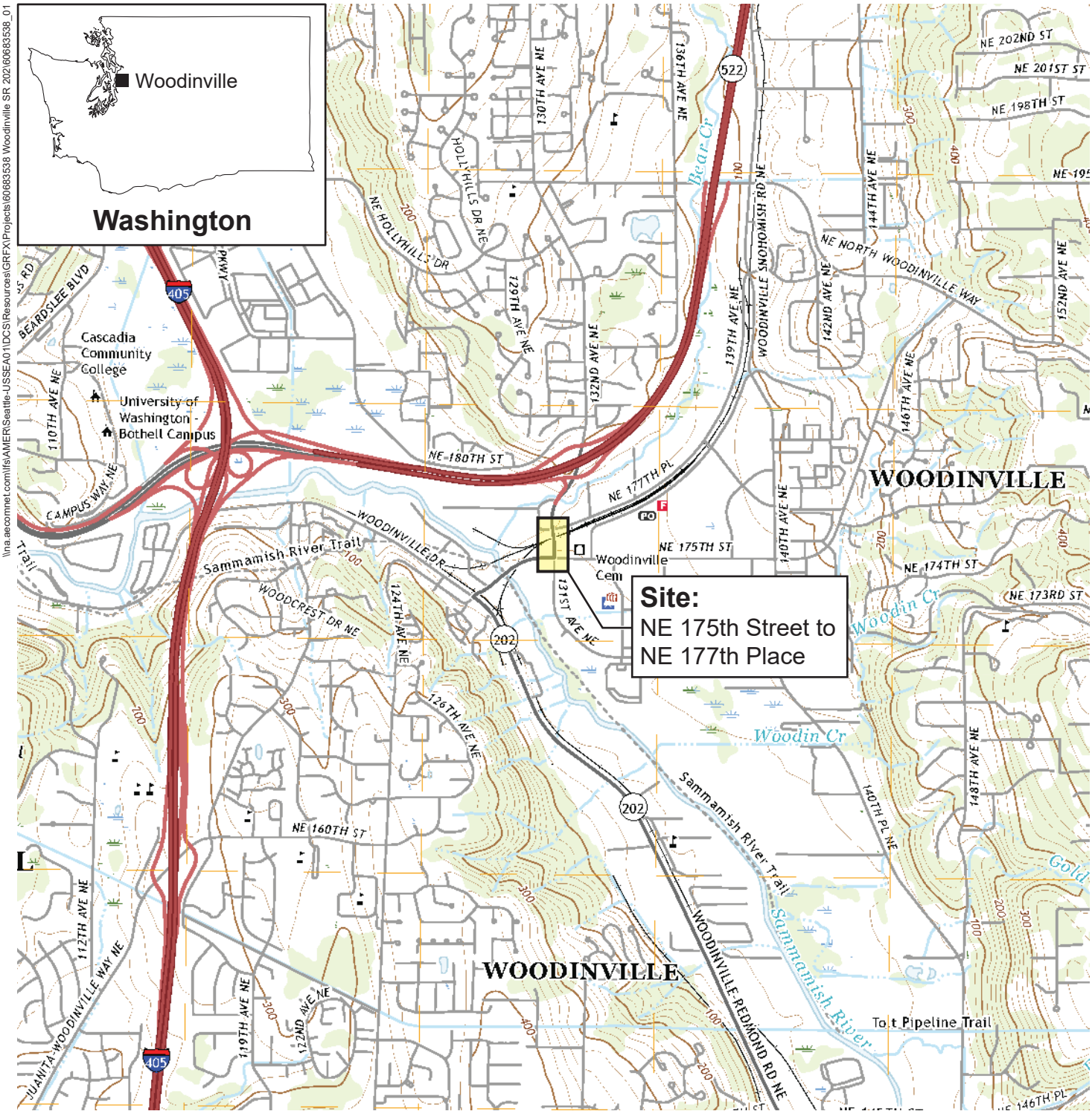
[d]Chromatographic pattern is not consistent with a petroleum standard.

NE - not established

ug/l - microgram per liter

U - Compound was analyzed for but not detected above the reporting limit shown.

Figures



Source: USGS 7.5-minute topographic quadrangles:
Woodinville, Washington, 2020; and Kirkland, Washington, 2020

0 2,000 4,000

Scale in Feet

Figure 1

Site Location

Job No. 60683538

AECOM

SR 202 Widening and Trestle Replacement
NE 175th Street to NE 177th Place, Woodinville, Washington

\\na.aecomnet.com\lfs\AMER\Seattle-USSEA01\DCS\Resources\GRFX\Projects\60683538 Woodinville SR 202\60683538_03



Source: Google Earth Pro, image dated 8/23/22

Legend

- Approximate parcel boundary
- Boring location
- Monitoring well

0 80 160

Approximate Scale in Feet

Job No. 60683538

Figure 2

Site Vicinity

SR 202 Widening and Trestle Replacement
NE 175th Street to NE 177th Place, Woodinville, Washington

Appendix A – Utility Locate Field Report

71195

LOCATE REQUEST
This is not an invoice

APPLIED PROFESSIONAL SERVICES INC.
Solutions That Exceed Expectations

43530 SE North Bend Way • North Bend, WA 98045
Phone: (425) 888-2590

RR + tracks

DATE: 1/4/23

Job#
PURCHASE ORDER #: 60683538 Task 3.4
Task 6.5.2 Pep Boys & Jacksons Shell

CUSTOMER: AECOM

OFFICE PHONE:

BILLING ADDRESS:

eMAIL:

CITY: ST: ZIP:

CONTACT PERSON: MARKUS WALBAUM

PHONE: 206-910-9099

BILLING SUMMARY

SERVICES REQUESTED:

☑ **CONDUCTIBLES** ☐ **NON-CONDUCTIBLES** ☐ **GPR** ☐ **POLY WATER** ☐ **RM LOCATING™**

LOCATOR'S NAME: Jeff B

LOCATE ADDRESS: 13002 NE 177TH PL Woodinville

DESCRIPTION OF LOCATE: Loc f/o Along R/R TRACKS IN SILVER
BOTH ENDS OF R/R BRIDGE. SWEPT LOCATIONS AT Pep Boys
N.- CONDUCTABLES FOUND. Pwr IN SIDEWALK MARKED By 1-CALL
2 LOCATIONS AT SHELL N.i CONDUCTABLES FOUND
MARKED SEWER fm M.H.

Power

TOTAL TIME INCLUDING TRAVEL: 3HRS

This locate is being performed on private property or as a design locate in the public ROW at the customer's request. APS has no knowledge of, and customer has not furnished as installed plans, drawings or other information of other utilities in the area of the requested locate. It is not guaranteed or implied that APS will find all underground utilities within the area. APS shall not be liable for damage to any type of facility, utility, or any loss of injury caused by such damage.

PAYMENT TERMS: NET 30 DAYS

CUSTOMER SIGNATURE:

LOCATOR SIGNATURE: DATE: 1/4/23

CUSTOMER SHOULD CALL ONE-CALL FOR PUBLICLY OWNED UTILITIES 1-800-424-5555

CC CARD INFO: VISA/MC/AMEX: EXP DATE:

ZIP: 3 DIGIT CODE ON BACK: 4 DIGIT CODE ON FRONT(AMEX ONLY):

72501

LOCATE REQUEST
This is not an invoice

APPLIED PROFESSIONAL SERVICES INC.
Solutions That Exceed Expectations

43530 SE North Bend Way • North Bend, WA 98045
Phone: (425) 888-2590

DATE: 1-4-23

PURCHASE ORDER #: Task 6.52 Dep Boxes # Jackson Shell

CUSTOMER: ACCOM

OFFICE PHONE: _____

BILLING ADDRESS: On File

EMAIL: _____

CITY: _____ ST: _____ ZIP: _____

CONTACT PERSON: Mark

PHONE: 206 910-9099

BILLING SUMMARY

SERVICES REQUESTED:

☐ **CONDUCTIBLES** ☐ **NON-CONDUCTIBLES** ☒ **GPR** ☐ **POLY WATER** ☐ **RM LOCATING™**

LOCATOR'S NAME: Andrew R

LOCATE ADDRESS: 13002 NE 177th Pl Woodinville WA

DESCRIPTION OF LOCATE: designated Storm line & Sewer near both work areas

TOTAL TIME INCLUDING TRAVEL: 1-Shot

This locate is being performed on private property or as a design locate in the public ROW at the customer's request. APS has no knowledge of, and customer has not furnished as-installed plans, drawings or other information of other utilities in the area of the requested locate. It is not guaranteed or implied that APS will find all underground utilities within the area. APS shall not be liable for damage to any type of facility, utility, or any loss of injury caused by such damage.

PAYMENT TERMS: NET 30 DAYS

CUSTOMER SIGNATURE: Mark Stallcup

LOCATOR SIGNATURE: _____ DATE: 1-4-23

CUSTOMER SHOULD CALL ONE-CALL FOR PUBLICLY OWNED UTILITIES 1-800-424-5555

CC CARD INFO: VISA/MC/AMEX: _____ EXP DATE: _____

ZIP: _____ 3 DIGIT CODE ON BACK: _____ 4 DIGIT CODE ON FRONT(AMEX ONLY): _____

Washington One Call

Ticket number	23009983	**Type/header**	2 FULL BUSINESS DAYS CALL
Original call date	01/10/23 10:58 am		
Work to begin date	01/17/23 12:00 am		
Expiration date	02/24/23 11:00 am		

Excavator information

Company name	AECOM	**Type of excavator**	
Contact name	MARKUS WALBAUM	**Phone**	206-438-2197
Alt. contact	MARKUS - CALL 1ST	**Phone**	206-910-9099
Address	1111 3RD AVENUE SEATTLE, WA 98101	**Fax phone**	
Email address	markus.walbaum@aecom.com		

Excavation information

Type of work	ENVIREMENTAL DRIILING AND PAVEMENT COREING
Directional drilling	
Type(s) of excavation equipment	
Work being done for	CITY OF WOODENVILLE

Location information

State	WA	**County**	KING
Place	WOODINVILLE		
Address/Street	131ST AVE NE		
Intersecting street	NE 175TH ST		
Location of work	SITE STARTS AT THE ABV INTER MARK THE ENTIRE ST OF 131ST AVE NE INCLUDING 20FT BEHIND THE CURB LINES BOTH SIDES OF THE ST FROM THE ABV INTER UP TO NE 177TH ST AT THE ABV SITE		
Remarks	AREA MARKED IN WHITE PAINT.		

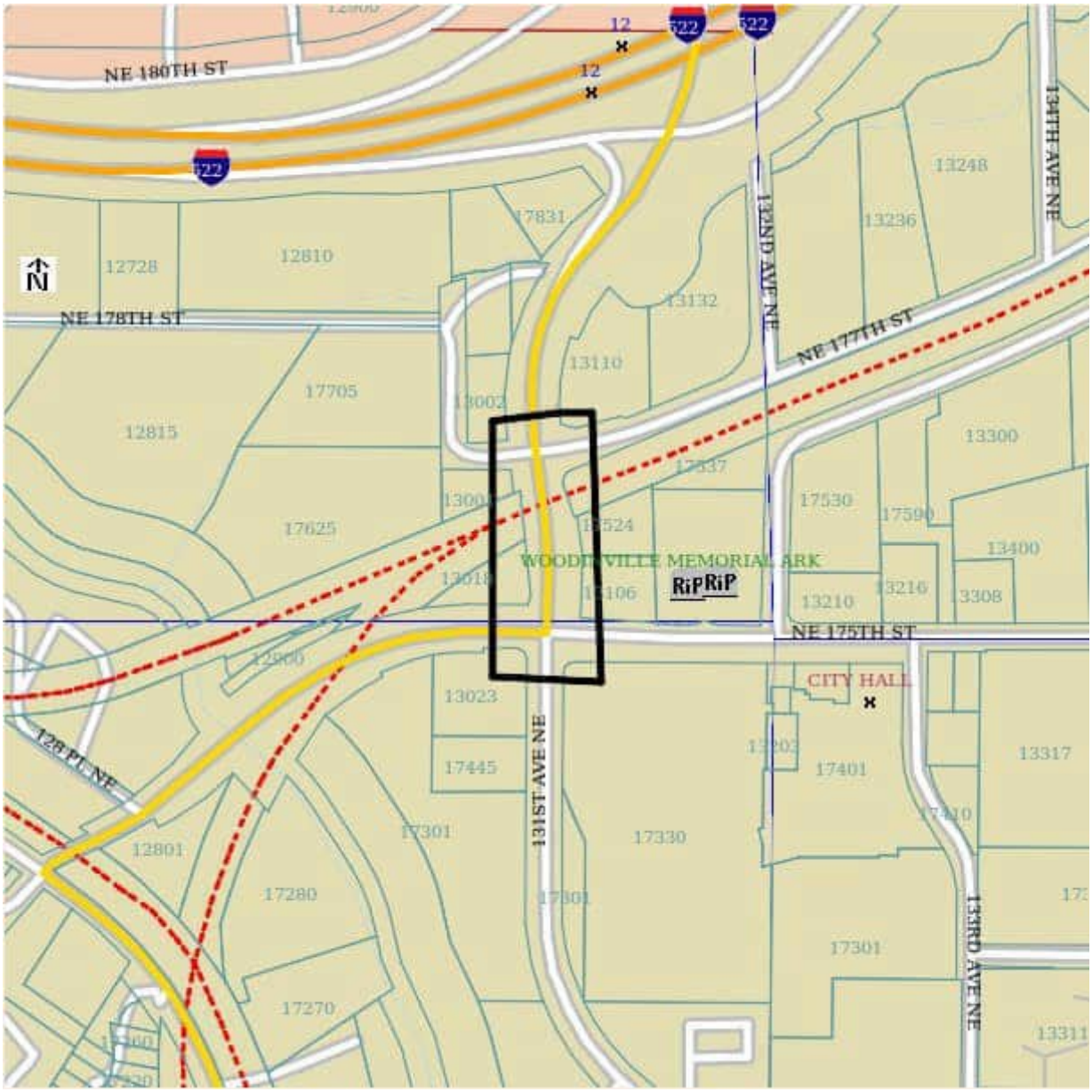
NW Lat 47.7558639 **Lon** -122.1658466 **SE Lat** 47.7542019 **Lon** -122.1648492

Members notified

District	Company name	Marking concerns	Customer service	Repair	Status
CCWAU01	COMCAST CABLE	800-778-9140	800-934-6489	855-537-6296	Not yet responded
GTE01	ZIPLY FIBER	800-778-9140	866-947-5995	866-947-5995	Clear, No Utility Owned Facility in Excavation Area
KCMTRO01	KING CNTY METRO SEWER	206-263-5722	206-263-3700	206-263-3840	Does not report
MCI01	MCI	800-289-3427	800-289-3427	800-289-3427	Does not report
NSD01	NORTHSHORE SCHOOL DISTRICT	360-829-5166	425-408-7754	425-478-6347	Duplicate Ticket (Please see contractor ticket

					WA23010599 for notes and charges.)
PUGE03	PUGET SOUND ENERGY ELECTRIC	888-728-9343	888-225-5773	888-225-5773	Marked, Complete
PUGG03	PUGET SOUND ENERGY GAS	888-728-9343	888-225-5773	888-225-5773	Marked, Complete
STARCM01	ZAYO CANADA/ALLSTREAM	800-961-6500	443-403-2023	866-236-2824	Ongoing Job
WDOTNW05	WSDOT MTCE AREA 5	425-739-3700	425-739-3700	206-440-4490	Does not report
WODH2001	WOODINVILLE WATER DISTRICT	425-487-4100	425-487-4100	425-487-4100	Does not report
WOODIN01	CITY OF WOODINVILLE	800-778-9140	425-877-2274	206-255-5088	Clear, No Utility Owned Facility in Excavation Area
WSDOT12	WSDOT - SIGNAL BRANCH OFFICE	206-442-2110	206-442-2110	206-442-2110	Does not report

Map



Appendix B – Photographic Log

AECOM

PHOTOGRAPH LOG

Project Name:	Site Location:	Project No.
Environmental Investigation SR 202 Widening and Trestle Replacement	NE 175th Street to NE 177th Street Woodinville, Washington	60683538

Photo No. 1	Date: 2/8/23

Direction Photo Taken:

North

Description:

Completion of Boring BT-1 with Hollow Stem Auger Rig. Pep Boys service center in background.

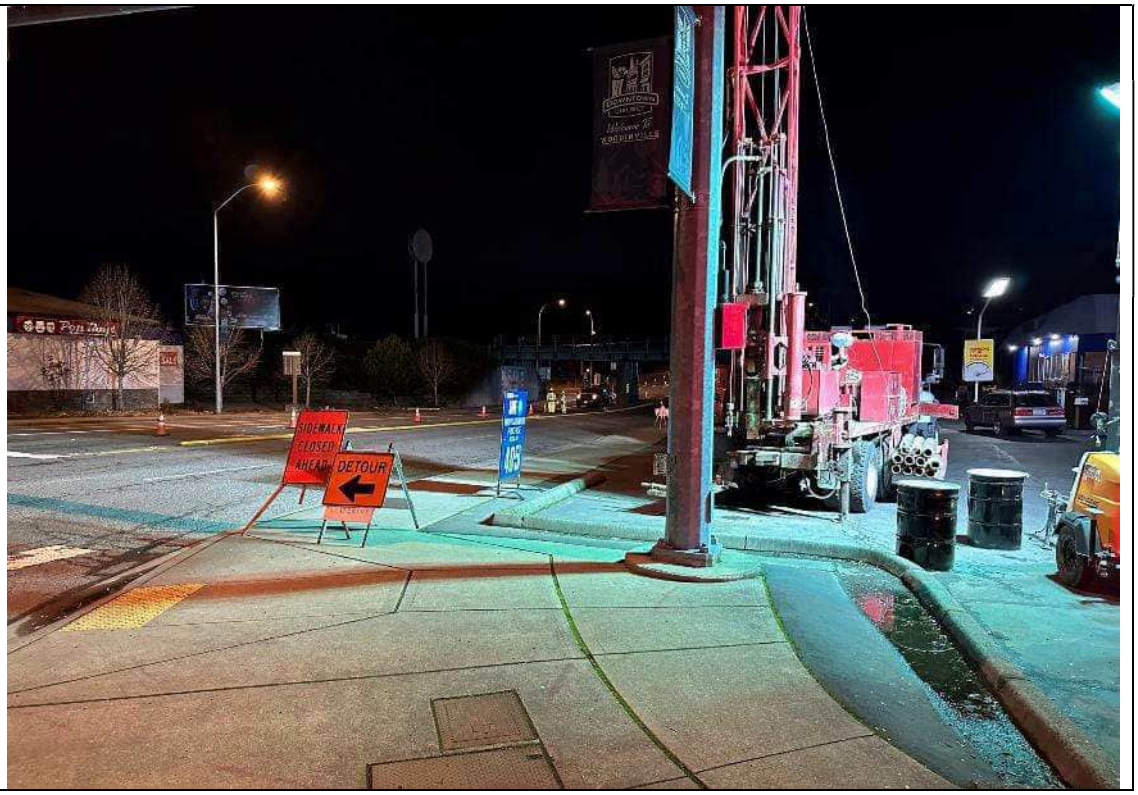


Photo No. 2	Date: 2/8/23

Direction Photo Taken:

Northwest

Description:

Completed boring BT-1, which has been patched to match surface conditions



AECOM

PHOTOGRAPH LOG

Project Name:	Site Location:	Project No.
Environmental Investigation SR 202 Widening and Trestle Replacement	NE 175th Street to NE 177th Street Woodinville, Washington	60683538

Photo No. 3	Date: 2/8/23

Direction Photo Taken:

West

Description:

Completed well with 8-inch flush mount monument at boring BT-2, Pet Boys service center in background.

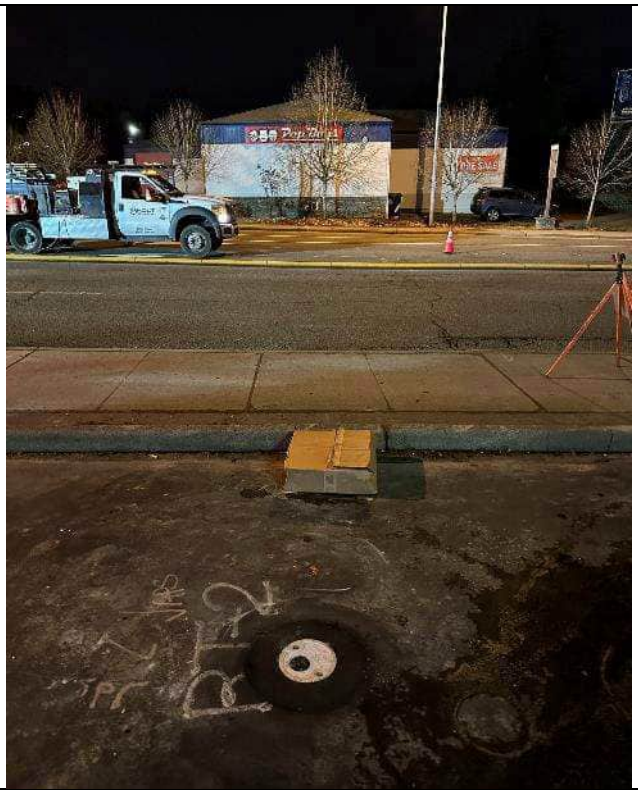


Photo No. 4	Date: 2/8/23

Direction Photo Taken:

Northwest

Description:

Completed boring BT-3, which has been patched to match surface conditions.



Appendix C – Boring Logs

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 1/4/2023 - 2/9/2023	**TEST HOLE LEGEND**
PROJECT NO: 60683538	**NUMBER OF HOLES DRILLED:** 9	PAGE 1 OF 1
LOCATION: Woodinville WA	**TOTAL LENGTH DRILLED (ft):** 248.6	

DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SOIL SAMPLES: SAMPLE TYPE AND NUMBER	SOIL SAMPLES: BLOW COUNTS (N VALUE)	SOIL SAMPLES: RECOVERY (%)	STRATIGRAPHY	MATERIAL DESCRIPTION AND REMARKS	LITHOLOGY	ROCK CORE: BOX NUMBER	ROCK CORE: RUN NUMBER	ROCK CORE: RECOVERY (%)	ROCK CORE: RQD (%)	ROCK CORE: FRACTURES PER FOOT
1	2	3	4	5	6	7	8	9	10	11	12	13

COLUMN DESCRIPTIONS

1. **Depth:** Depth in feet below the ground surface.
2. **Sample Symbol:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.
3. **Sample Type and Number:** Indicates the type of sampler used and identification number.
4. **Blow Counts:** Number of blows required to advance sampler each 6-inch interval, or length noted, using a 140-lb hammer falling 30-inches. N-value reported for SPT samples.
5. **Recovery:** Percentage of driven soil sample length actually recovered.
6. **Stratigraphy:** Abbreviation for soil depositional environment of rock formation name.
7. **Material Description and Remarks:** Description of material encountered; may include density/consistency, moisture, color, material type, grain size and other remarks.
8. **Lithology:** Graphic depiction of soil material or rock type encountered; typical symbols are explained below..
9. **Box Number:** Indicates core sample box number.
10. **Run Number:** Indicates core sample run number.
11. **Recovery:** Percentage of core run length actually recovered.
12. **RQD:** Rock Quality Designation; Percentage of intact rock core (pieces of sound core greater than 100 millimeters or about 4 inches long) in each core run.
13. **Fractures per Foot:** Approximate number of rock core fractures per foot.

TYPICAL SOIL MATERIAL GRAPHIC SYMBOLS

- Asphalt
- Concrete
- Poorly-graded Gravel
- Silt
- Poorly-graded Sand
- Low Plasticity Clay
- Fill
- Poorly-graded Gravel with silt
- Silty Sand
- Poorly-graded Sand

Relative Density or Consistency

Coarse-Grained Soils		Fine-Grained Soils	
Relative Density	N, SPT Blows / ft	Relative Consistency	N, SPT Blows / ft
Very loose sand	0 - 4	Very soft	< 2
Loose	4 - 10	Soft	2 - 4
Medium dense	10 - 30	Medium stiff	4 - 8
Dense	30 - 50	Stiff	8 - 15
Very dense	Over 50	Very stiff	15 - 30
		Hard	Over 30

TYPICAL SAMPLER GRAPHIC SYMBOLS

- GRAB: Grab Sample
- MC: Modified California Sampler
- SS: Split Spoon

WATER LEVEL LEGEND

- 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

Recovered Sample Length — Driven Sample Length

ABBREVIATIONS

ATD: At Time of Drilling
NR: No Recovery
NA: Not Applicable
bgs: Below Ground Surface

MINOR SOIL TYPE(s)

- **"trace"** When the soil type's percentage is estimated, using visual/manual procedures, to be between 1 and 15 percent of the total sample.
- **"with"** When the soil type's percentage is estimated, using visual/manual procedures, to be greater than 15 percent and less than 30 percent of the total sample.
- **"y"** When the soil type's percentage is estimated, using visual/manual procedures, to be greater than 30 percent of the total sample.

Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/9/23	**LOG NO: BT-1**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 35.5	PAGE 1 OF 1 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 16.5	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Kevin B.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-60 Truck Rig (Rig #215)	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** SW side of Shell Parking Lot
DRILL METHOD: Hollow Stem Auger / SPT	**GROUNDWATER (ft bgs):** 10.7	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE: 8", Bullet/Spade	**COMPLETION (ft):** Bentonite, Quickrete	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

ELEVATION (ft)	DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
35	0					ASPHALT, Pavement 4"									
						Moist, light tan to dark gray, fine to medium grained, silty SAND, with GRAVEL, little fine gravel, (SM), (FILL)									
						Moist, brown, medium grained, SAND, few coarse sand, (SP)									
			GRAB BT-1-HA2.5		100	BT-1-HA2.5: PID Reading 0.0ppm									
	5					Top 5.25-feet airknife/vac ex/hand-augered on 2/6/2023. Backilled with import sand and finished with cold-patch. Stop at 11:05 PM. Drilling resumed 2/8/23 at 11:20 PM.									
30			SS BT-1-5.5	3-4-4 (8)	100	Loose, moist, tan to gray, fine to medium grained, SAND, (SP) BT-1-5.5: PID Reading 0.2ppm									
			SS BT-1-8	3-6-7 (13)	83	Medium dense, moist, tan to gray, fine to medium grained, SAND, trace fine gravel, (SP) BT-1-8: PID Reading 0.4ppm									
	10														
25			SS BT-1-10	3-5-8 (13)	67	▽ Medium dense, wet, tan to gray, fine to medium grained, SAND, (SP) BT-1-10: PID Reading 0.5ppm									
			SS BT-1-12 BT-1-GW	5-6-7 (13)	78	Medium dense, wet, tan to gray, fine to medium grained, SAND, trace coarse gravel, (SP) BT-1-12: PID Reading 0.4ppm BT-1-GW: Groundwater sampled from temporary 2"-dia. monitoring well screened 5-15 ft bgs									
	15														
20			SS BT-1-15	14-24-29 (53)	89	Very dense, wet, tan to gray, fine to medium grained, SAND, trace fine gravel, (SP) BT-1-15: PID Reading 0.5ppm									
						Finish Drilling 11:51 PM on 2/8/23. Groundwater measured at 10.7 ft bgs at 12:22 AM on 2/9/23. Backfilled with bentonite chips and patched with Quikrete FastSet DOT Mix (dyed black). Bottom of hole at 16.5 feet.									

AECOM-SMART-SOIL-LAB-ELEV - URS_DENVER STD US LAB.GDT - 10/29/24 08:48 - C:\USERS\JUGGERTJ\ONEDRIVE - AECOM\DESKTOP\60683538_WOODINVILLE TRESTLE_2024.10.28.GPJ

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/8/23	**LOG NO: BT-2 (MW)**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 35.0	PAGE 1 OF 2 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 21.1	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Kevin B.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-60 Truck Rig (Rig #215)	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** West side of Shell Parking Lot
DRILL METHOD: Hollow Stem Auger / SPT	**GROUNDWATER (ft bgs):** 10.5	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE: 8", Bullet/Spade	**COMPLETION (ft):** Open Standpipe Piezometer	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

ELEVATION (ft)	DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
35	0					ASPHALT, Pavement 2"									
						Tan to gray, fine to medium grained, SAND, (SP), (FILL)									
						Light brown, fine grained, SAND, (SP)									
			GRAB BT-2-HA2.5		100	Light brown, fine grained, SAND, (SP) BT-2-HA2.5: PID Reading 0.0ppm									
30	5					Top 5.25-feet airknife/vac ex/hand-augered on 2/6/2023. Backilled with import sand and finished with cold-patch. Stop at 10:20 PM. Drilling resumed 2/8/23 at 12:22 AM.									
			SS BT-2-5.5	5-7-8 (15)	33	Medium dense, moist, tan to gray, fine to medium grained, SAND, trace fine gravel, (SP) BT-2-5.5: PID Reading 0.1ppm									
			SS BT-2-7	5-7-9 (16)	44	Medium dense, moist, tan to gray, fine to medium grained, SAND, trace fine gravel, (SP) BT-2-7: PID Reading 0.3ppm									
25	10														
			SS BT-2-10	5-7-8 (15)	44	▽ Medium dense, wet, tan to gray, fine to medium grained, SAND, trace fine gravel, (SP) BT-2-10: PID Reading 0.3ppm Groundwater 10'6" bgs at 10:35 PM on 2/8/23									
			SS BT-2-12.5	8-23-50/3"/3"	100	Very dense, wet, tan to gray, fine to medium grained, SAND, trace fine gravel, (SP) BT-2-12.5: PID Reading 0.3ppm									
20	15														
			SS BT-2-15	16-20-50/4"/4"	100	Very dense, wet, tan to gray, fine to medium grained, SAND, trace fine gravel, (SP) BT-2-15: PID Reading 0.3ppm									
			SS BT-2-17.5	18-38-50/2"/2"	100	Very dense, wet, tan to gray, fine to medium grained, SAND, few fine gravel, (SP) BT-2-17.5: PID Reading 0.4ppm									
15	20														

(Continued Next Page)

AECOM-SMART-SOIL-LAB-ELEV - URS_DENVER STD US LAB.GDT - 10/29/24 08:48 - C:\USERS\JUGGERTJ\ONEDRIVE - AECOM\DESKTOP\60683538_WOODINVILLE TRESTLE_2024.10.28.GPJ

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/8/23	**LOG NO: BT-2 (MW)**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 35.0	PAGE 2 OF 2 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 21.1	

ELEVATION (ft)	DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
15	20														
		X	SS BT-2-20	15-37-50/1"	98	Very dense, wet, tan to gray, fine to medium grained, SAND, trace fine to coarse gravel, (SP) BT-2-20: PID Reading 0.3ppm									

Finish Drilling 2/8/23 at 1:21 AM. Installed 2" PVC monitoring well to 20' bgs (see well installation log). Well developed on 2/8/2023 at 10:35 PM . Screened interval 19.83' to 9.83'. Filter pack 20' - 9'. 3/8" Bentonite chips seal/backfill 9' to 2'. 10" diameter steel flush-mount locking monument 2' - 0'. Top of PVC riser 0.32' below the monument (two 9/16" bolts for locking cover)

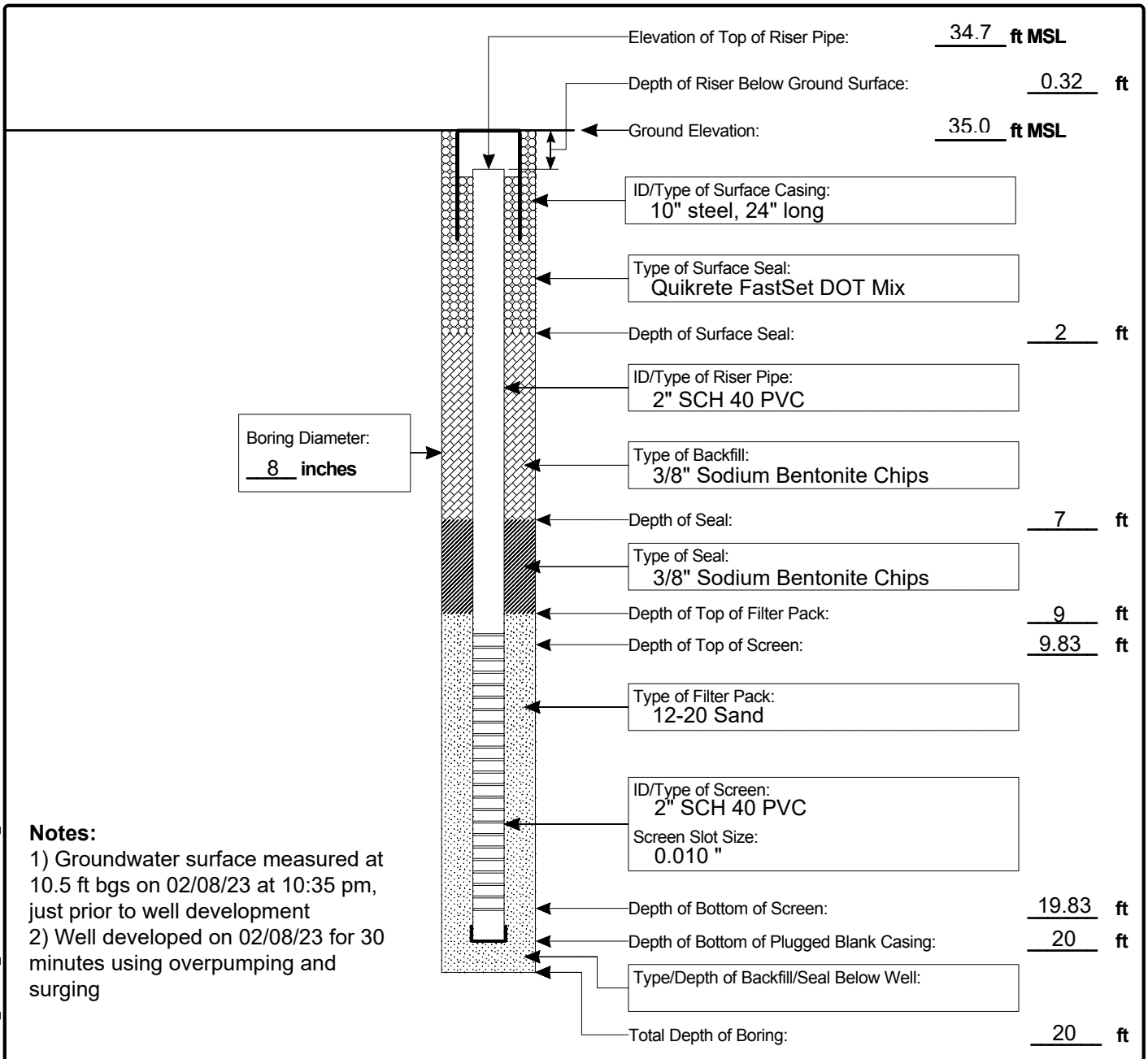
Bottom of hole at 21.1 feet.

AECOM-SMART-SOIL-LAB-ELEV - URS_DENVER STD US LAB.GDT - 10/29/24 08:48 - C:\USERS\JUGGERTJ\ONEDRIVE - AECOM\DESKTOP\60683538_WOODINVILLE TRESTLE_2024.10.28.GPJ

Project: SR 202 Widening and Trestle Replacement
Project Location: Woodinville, WA 98072
Project Number: 60683538

MONITORING WELL CONSTRUCTION LOG FOR WELL BT-2 (MW)

Well Location: In City ROW, W side of Shell Parking Lot	Date(s) Installed: 02/08/23	Time: 2:00 am
Installed By: Holt Services (Kevin B.)	Observed By: M. Black	Total Depth: 20 ft
Method of Installation: HSA		
Screened Interval: 19.83 - 9.83 ft bgs	Completion Zone: 20 - 9 ft	
Remarks: WA Dept. of Ecology Unique Well BPW 357#		



Notes:
1) Groundwater surface measured at 10.5 ft bgs on 02/08/23 at 10:35 pm, just prior to well development
2) Well developed on 02/08/23 for 30 minutes using overpumping and surging

NOTE: DIAGRAM IS NOT TO SCALE

Report: ENV_WELL_CONSTR_BELOW_GROUND; File: WC_CORP2.GPJ; 8/20/2002 -

AECOM

PROJECT NAME: SR-202 Widening and Trestle Replacement	**DATES DRILLED:** 2/6/23 - 2/7/23	**LOG NO: BT-3**
PROJECT NO: 60683538	**SURFACE ELEVATION (ft):** 32.4	PAGE 1 OF 1 AECOM
LOCATION: Woodinville WA	**TOTAL DEPTH (ft):** 16.5	

DRILLING COMPANY: Holt Services	**INCLINATION (deg):** N/A	**LOGGED BY:** Michael Black
DRILLER: Kevin B.	**AZIMUTH (deg):** N/A	**CHECKED BY:** M. Walbaum
DRILL EQUIP: Mobile B-60 Truck Rig (Rig #215)	**CASING DEPTH (ft bgs):** N/A	**HOLE LOCATION:** NE corner Pep Boys Parking Lot
DRILL METHOD: Hollow Stem Auger / SPT	**GROUNDWATER (ft bgs):** 7	**LATITUDE (deg) or NORTHING (ft):** Not Surveyed
BIT SIZE/TYPE: 8", Bullet/Spade	**COMPLETION (ft):** Bentonite, Quickrete	**LONGITUDE (deg) or EASTING (ft):** Not Surveyed

ELEVATION (ft)	DEPTH (ft)	SOIL SAMPLES: SAMPLE SYMBOL	SAMPLE TYPE AND NUMBER	BLOW COUNTS (N VALUE)	RECOVERY (%)	MATERIAL DESCRIPTION AND REMARKS	STRATIGRAPHY	LABORATORY TESTING RESULTS: Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	GRAVEL (%)	SAND (%)	FINES - Passing No. 200 Sieve (%)	CLAY - Passing 2 microns (%)
	0					ASPHALT, Pavement 2"									
						Moist, tan to gray, fine to medium grained, SAND, (SP), (FILL)									
						ASPHALT, Pavement 1"									
30						Moist, tan to gray, fine to medium grained, SAND, (SP)									
			GRAB BT-3-HA2.5		100	Fine to medium grained, SAND, (SP)									
						BT-3-HA-2.5: PID Reading 0.0ppm									
	5					Top 5.5-feet airknife/vac ex/hand-augered on 2/6/2023. Backfilled with import sand and finished with cold-patch. Stop at 9:28 PM. Drilling resumed 2/7/23 at 9:12 PM.									
			SS BT-3-5.5	5-3-5 (8)	61	Loose, moist, tan to gray, fine to medium grained, SAND, (SP) BT-3-5.5: PID Reading 0.1ppm									
25			SS BT-3-7	5-7-10 (17)	78	▽ Medium dense, wet, tan to gray, fine to medium grained, SAND, (SP) BT-3-7: PID Reading 0.1ppm Groundwater 7.0' bgs at 10:00 PM									
	10														
			SS BT-3-10 BT-3-GW	5-10-27 (37)	83	Dense, wet, tan to gray, fine to medium grained, SAND, (SP) BT-3-10: PID Reading 0.3ppm BT-3-GW: Groundwater sampled from temporary 2"-dia. monitoring well screened 5-15 ft bgs									
20			SS BT-3-12.5	16-29-50/4"/4"	100	Very dense, wet, tan to gray, fine to medium grained, SAND, (SP) BT-3-12.5: PID Reading 0.2ppm									
	15														
			SS BT-3-15	17-17-23 (40)	100	Dense, wet, tan to gray, fine to medium grained, SAND, (SP) BT-3-15: PID Reading 0.2ppm									
						Finish Drilling on 2/7/23 at 9:50 PM. Backfilled with bentonite chips and patched with Quikrete FastSet DOT Mix (dyed black). Bottom of hole at 16.5 feet.									

AECOM-SMART-SOIL-LAB-ELEV - URS_DENVER STD US LAB.GDT - 10/29/24 08:48 - C:\USERS\JUGGERTJ\ONEDRIVE - AECOM\DESKTOP\60683538_WOODINVILLE TRESTLE_2024.10.28.GPJ

Appendix D – Laboratory Analytical Reports

3600 Fremont Ave. N.
Seattle, WA 98103
T: (206) 352-3790
F: (206) 352-7178
info@fremontanalytical.com

AECOM
Markus Walbaum
1111 3rd Avenue Suite 1600
Seattle, WA 98101

RE: Woodinville SR202 Trestle Widening
Work Order Number: 2302195

February 20, 2023

Attention Markus Walbaum:

Fremont Analytical, Inc. received 11 sample(s) on 2/9/2023 for the analyses presented in the following report.

Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.
Gasoline by NWTPH-Gx
Sample Moisture (Percent Moisture)
Volatile Organic Compounds by EPA Method 8260D

This report consists of the following:

- Case Narrative
- Analytical Results
- Applicable Quality Control Summary Reports
- Chain of Custody

All analyses were performed consistent with the Quality Assurance program of Fremont Analytical, Inc. Please contact the laboratory if you should have any questions about the results.

Thank you for using Fremont Analytical.

Sincerely,

Brianna Barnes
Project Manager

Date: *03/07/2023*

CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening
Work Order: 2302195

Work Order Sample Summary

Lab Sample ID	Client Sample ID	Date/Time Collected	Date/Time Received
2302195-001	BT-1-8	02/08/2023 11:32 PM	02/09/2023 4:44 PM
2302195-002	BT-1-10	02/08/2023 11:39 PM	02/09/2023 4:44 PM
2302195-003	BT-1-12	02/08/2023 11:46 PM	02/09/2023 4:44 PM
2302195-004	BT-1-GW	02/08/2023 12:30 AM	02/09/2023 4:44 PM
2302195-005	BT-2-5.5	02/08/2023 12:32 AM	02/09/2023 4:44 PM
2302195-006	BT-2-7	02/08/2023 12:36 AM	02/09/2023 4:44 PM
2302195-007	BT-2-10	02/08/2023 12:42 AM	02/09/2023 4:44 PM
2302195-008	BT-3-5.5	02/07/2023 9:22 PM	02/09/2023 4:44 PM
2302195-009	BT-3-7	02/07/2023 9:27 PM	02/09/2023 4:44 PM
2302195-010	BT-3-10	02/07/2023 9:32 PM	02/09/2023 4:44 PM
2302195-011	BT-3-GW	02/07/2023 11:00 PM	02/09/2023 4:44 PM

Note: If no "Time Collected" is supplied, a default of 12:00AM is assigned

Case Narrative

WO#: **2302195**
Date: **2/20/2023**

CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

I. SAMPLE RECEIPT:
Samples receipt information is recorded on the attached Sample Receipt Checklist.

II. GENERAL REPORTING COMMENTS:
Results are reported on a wet weight basis unless dry-weight correction is denoted in the units field on the analytical report ("mg/kg-dry" or "ug/kg-dry").

Matrix Spike (MS) and MS Duplicate (MSD) samples are tested from an analytical batch of "like" matrix to check for possible matrix effect. The MS and MSD will provide site specific matrix data only for those samples which are spiked by the laboratory. The sample chosen for spike purposes may or may not have been a sample submitted in this sample delivery group. The validity of the analytical procedures for which data is reported in this analytical report is determined by the Laboratory Control Sample (LCS) and the Method Blank (MB). The LCS and the MB are processed with the samples and the MS/MSD to ensure method criteria are achieved throughout the entire analytical process.

III. ANALYSES AND EXCEPTIONS:
Exceptions associated with this report will be footnoted in the analytical results page(s) or the quality control summary page(s) and/or noted below.

Qualifiers:

* - Flagged value is not within established control limits
B - Analyte detected in the associated Method Blank
D - Dilution was required
E - Value above quantitation range
H - Holding times for preparation or analysis exceeded
I - Analyte with an internal standard that does not meet established acceptance criteria
J - Analyte detected below Reporting Limit
N - Tentatively Identified Compound (TIC)
Q - Analyte with an initial or continuing calibration that does not meet established acceptance criteria
S - Spike recovery outside accepted recovery limits
ND - Not detected at the Reporting Limit
R - High relative percent difference observed

Acronyms:

%Rec - Percent Recovery
CCB - Continued Calibration Blank
CCV - Continued Calibration Verification
DF - Dilution Factor
DUP - Sample Duplicate
HEM - Hexane Extractable Material
ICV - Initial Calibration Verification
LCS/LCSD - Laboratory Control Sample / Laboratory Control Sample Duplicate
MCL - Maximum Contaminant Level
MB or MBLANK - Method Blank
MDL - Method Detection Limit
MS/MSD - Matrix Spike / Matrix Spike Duplicate
PDS - Post Digestion Spike
Ref Val - Reference Value
REP - Sample Replicate
RL - Reporting Limit
RPD - Relative Percent Difference
SD - Serial Dilution
SGT - Silica Gel Treatment
SPK - Spike
Surr - Surrogate

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 11:32:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-001
Matrix: Soil
Client Sample ID: **BT-1-8**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39422		Analyst: KJ
Diesel Range Organics	ND	57.3		mg/Kg-dry	1	2/14/2023 9:01:19 AM
Heavy Oil	ND	115		mg/Kg-dry	1	2/14/2023 9:01:19 AM
Total Petroleum Hydrocarbons	ND	172		mg/Kg-dry	1	2/14/2023 9:01:19 AM
Surr: 2-Fluorobiphenyl	95.3	50 - 150		%Rec	1	2/14/2023 9:01:19 AM
Surr: o-Terphenyl	99.0	50 - 150		%Rec	1	2/14/2023 9:01:19 AM
Gasoline by NWTPH-Gx				Batch ID: 39415		Analyst: CC
Gasoline Range Organics	ND	4.64		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Surr: Toluene-d8	92.1	65 - 135		%Rec	1	2/14/2023 11:20:24 PM
Surr: 4-Bromofluorobenzene	88.2	65 - 135		%Rec	1	2/14/2023 11:20:24 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Dichlorodifluoromethane (CFC-12)	ND	0.0139	Q	mg/Kg-dry	1	2/14/2023 11:20:24 PM
Chloromethane	ND	0.0464		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Vinyl chloride	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Bromomethane	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Trichlorofluoromethane (CFC-11)	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Chloroethane	ND	0.0695		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,1-Dichloroethene	ND	0.0927		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Acetone	ND	0.232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Methylene chloride	ND	0.0324		mg/Kg-dry	1	2/14/2023 11:20:24 PM
trans-1,2-Dichloroethene	ND	0.00927		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Methyl tert-butyl ether (MTBE)	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,1-Dichloroethane	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
cis-1,2-Dichloroethene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
(MEK) 2-Butanone	ND	0.278		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Chloroform	ND	0.0162		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,1,1-Trichloroethane (TCA)	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,1-Dichloropropene	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Carbon tetrachloride	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2-Dichloroethane (EDC)	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Benzene	ND	0.0162		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Trichloroethene (TCE)	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2-Dichloropropane	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Bromodichloromethane	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Dibromomethane	ND	0.0116		mg/Kg-dry	1	2/14/2023 11:20:24 PM
cis-1,3-Dichloropropene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 11:32:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-001
Matrix: Soil
Client Sample ID: **BT-1-8**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Toluene	ND	0.0278		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Trans-1,3-Dichloropropylene	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Methyl Isobutyl Ketone (MIBK)	ND	0.0556		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,1,2-Trichloroethane	ND	0.0116		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,3-Dichloropropane	ND	0.00927		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Tetrachloroethene (PCE)	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Dibromochloromethane	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2-Dibromoethane (EDB)	ND	0.00927		mg/Kg-dry	1	2/14/2023 11:20:24 PM
2-Hexanone (MBK)	ND	0.0579		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Chlorobenzene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,1,1,2-Tetrachloroethane	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Ethylbenzene	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
m,p-Xylene	ND	0.0464		mg/Kg-dry	1	2/14/2023 11:20:24 PM
o-Xylene	ND	0.0232		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Styrene	ND	0.00927		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Isopropylbenzene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Bromoform	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,1,2,2-Tetrachloroethane	ND	0.185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
n-Propylbenzene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Bromobenzene	ND	0.0116		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,3,5-Trimethylbenzene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
2-Chlorotoluene	ND	0.0153		mg/Kg-dry	1	2/14/2023 11:20:24 PM
4-Chlorotoluene	ND	0.0153		mg/Kg-dry	1	2/14/2023 11:20:24 PM
tert-Butylbenzene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2,3-Trichloropropane	ND	0.0278		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2,4-Trichlorobenzene	ND	0.0556		mg/Kg-dry	1	2/14/2023 11:20:24 PM
sec-Butylbenzene	ND	0.139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
4-Isopropyltoluene	ND	0.185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,3-Dichlorobenzene	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,4-Dichlorobenzene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
n-Butylbenzene	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2-Dichlorobenzene	ND	0.0185		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2-Dibromo-3-chloropropane	ND	0.0278		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2,4-Trimethylbenzene	ND	0.0139		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Hexachloro-1,3-butadiene	ND	0.0371		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Naphthalene	ND	0.0927		mg/Kg-dry	1	2/14/2023 11:20:24 PM
1,2,3-Trichlorobenzene	ND	0.0556		mg/Kg-dry	1	2/14/2023 11:20:24 PM
Surr: Dibromofluoromethane	102	80 - 120		%Rec	1	2/14/2023 11:20:24 PM
Surr: Toluene-d8	110	80 - 120		%Rec	1	2/14/2023 11:20:24 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM **Collection Date:** 2/8/2023 11:32:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-001 **Matrix:** Soil
Client Sample ID: **BT-1-8**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Surr: 1-Bromo-4-fluorobenzene	97.7	80 - 120		%Rec	1	2/14/2023 11:20:24 PM

NOTES:
Q - Associated calibration verification is below acceptance criteria. Result may be low-biased

Sample Moisture (Percent Moisture)				Batch ID: R81805		Analyst: MP
Percent Moisture	18.2	0.500		wt%	1	2/13/2023 8:28:57 AM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 11:46:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-003
Matrix: Soil
Client Sample ID: **BT-1-12**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39422		Analyst: KJ
Diesel Range Organics	ND	60.8		mg/Kg-dry	1	2/14/2023 9:34:04 AM
Heavy Oil	ND	122		mg/Kg-dry	1	2/14/2023 9:34:04 AM
Total Petroleum Hydrocarbons	ND	182		mg/Kg-dry	1	2/14/2023 9:34:04 AM
Surr: 2-Fluorobiphenyl	87.7	50 - 150		%Rec	1	2/14/2023 9:34:04 AM
Surr: o-Terphenyl	90.7	50 - 150		%Rec	1	2/14/2023 9:34:04 AM
Gasoline by NWTPH-Gx				Batch ID: 39415		Analyst: CC
Gasoline Range Organics	ND	4.49		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Surr: Toluene-d8	92.2	65 - 135		%Rec	1	2/14/2023 5:38:42 PM
Surr: 4-Bromofluorobenzene	89.3	65 - 135		%Rec	1	2/14/2023 5:38:42 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Dichlorodifluoromethane (CFC-12)	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Chloromethane	ND	0.0449		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Vinyl chloride	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Bromomethane	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Trichlorofluoromethane (CFC-11)	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Chloroethane	ND	0.0673		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,1-Dichloroethene	ND	0.0897		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Acetone	ND	0.224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Methylene chloride	ND	0.0314		mg/Kg-dry	1	2/14/2023 5:38:42 PM
trans-1,2-Dichloroethene	ND	0.00897		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Methyl tert-butyl ether (MTBE)	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,1-Dichloroethane	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
cis-1,2-Dichloroethene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
(MEK) 2-Butanone	ND	0.269		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Chloroform	ND	0.0157		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,1,1-Trichloroethane (TCA)	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,1-Dichloropropene	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Carbon tetrachloride	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2-Dichloroethane (EDC)	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Benzene	ND	0.0157		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Trichloroethene (TCE)	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2-Dichloropropane	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Bromodichloromethane	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Dibromomethane	ND	0.0112		mg/Kg-dry	1	2/14/2023 5:38:42 PM
cis-1,3-Dichloropropene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM

Client: AECOM **Collection Date:** 2/8/2023 11:46:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-003 **Matrix:** Soil
Client Sample ID: **BT-1-12**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Toluene	ND	0.0269		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Trans-1,3-Dichloropropylene	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Methyl Isobutyl Ketone (MIBK)	ND	0.0538		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,1,2-Trichloroethane	ND	0.0112		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,3-Dichloropropane	ND	0.00897		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Tetrachloroethene (PCE)	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Dibromochloromethane	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2-Dibromoethane (EDB)	ND	0.00897		mg/Kg-dry	1	2/14/2023 5:38:42 PM
2-Hexanone (MBK)	ND	0.0561		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Chlorobenzene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,1,1,2-Tetrachloroethane	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Ethylbenzene	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
m,p-Xylene	ND	0.0449		mg/Kg-dry	1	2/14/2023 5:38:42 PM
o-Xylene	ND	0.0224		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Styrene	ND	0.00897		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Isopropylbenzene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Bromoform	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,1,2,2-Tetrachloroethane	ND	0.179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
n-Propylbenzene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Bromobenzene	ND	0.0112		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,3,5-Trimethylbenzene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
2-Chlorotoluene	ND	0.0148		mg/Kg-dry	1	2/14/2023 5:38:42 PM
4-Chlorotoluene	ND	0.0148		mg/Kg-dry	1	2/14/2023 5:38:42 PM
tert-Butylbenzene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2,3-Trichloropropane	ND	0.0269		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2,4-Trichlorobenzene	ND	0.0538		mg/Kg-dry	1	2/14/2023 5:38:42 PM
sec-Butylbenzene	ND	0.135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
4-Isopropyltoluene	ND	0.179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,3-Dichlorobenzene	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,4-Dichlorobenzene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
n-Butylbenzene	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2-Dichlorobenzene	ND	0.0179		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2-Dibromo-3-chloropropane	ND	0.0269		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2,4-Trimethylbenzene	ND	0.0135		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Hexachloro-1,3-butadiene	ND	0.0359		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Naphthalene	ND	0.0897		mg/Kg-dry	1	2/14/2023 5:38:42 PM
1,2,3-Trichlorobenzene	ND	0.0538		mg/Kg-dry	1	2/14/2023 5:38:42 PM
Surr: Dibromofluoromethane	98.7	80 - 120		%Rec	1	2/14/2023 5:38:42 PM
Surr: Toluene-d8	108	80 - 120		%Rec	1	2/14/2023 5:38:42 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 11:46:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-003
Matrix: Soil
Client Sample ID: **BT-1-12**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Surr: 1-Bromo-4-fluorobenzene	98.9	80 - 120		%Rec	1	2/14/2023 5:38:42 PM
Sample Moisture (Percent Moisture)				Batch ID: R81805		Analyst: MP
Percent Moisture	21.0	0.500		wt%	1	2/13/2023 8:28:57 AM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 12:30:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-004
Matrix: Groundwater
Client Sample ID: **BT-1-GW**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39419		Analyst: KJ
Diesel Range Organics	147	95.3		µg/L	1	2/14/2023 12:53:31 PM
Heavy Oil	ND	95.3		µg/L	1	2/14/2023 12:53:31 PM
Total Petroleum Hydrocarbons	ND	191		µg/L	1	2/14/2023 12:53:31 PM
Surr: 2-Fluorobiphenyl	89.6	50 - 150		%Rec	1	2/14/2023 12:53:31 PM
Surr: o-Terphenyl	96.0	50 - 150		%Rec	1	2/14/2023 12:53:31 PM
NOTES: Chromatographic pattern indicates an unresolved complex mixture, which may be weathered and/or organic material						
Gasoline by NWTPH-Gx				Batch ID: 39414		Analyst: SH
Gasoline Range Organics	ND	50.0		µg/L	1	2/13/2023 8:18:35 PM
Surr: Toluene-d8	102	65 - 135		%Rec	1	2/13/2023 8:18:35 PM
Surr: 4-Bromofluorobenzene	99.8	65 - 135		%Rec	1	2/13/2023 8:18:35 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39414		Analyst: SH
Dichlorodifluoromethane (CFC-12)	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Chloromethane	ND	0.750		µg/L	1	2/13/2023 8:18:35 PM
Vinyl chloride	ND	0.200		µg/L	1	2/13/2023 8:18:35 PM
Bromomethane	ND	3.00		µg/L	1	2/13/2023 8:18:35 PM
Trichlorofluoromethane (CFC-11)	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
Chloroethane	ND	1.00		µg/L	1	2/13/2023 8:18:35 PM
1,1-Dichloroethene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Acetone	ND	5.00		µg/L	1	2/13/2023 8:18:35 PM
Methylene chloride	ND	0.750		µg/L	1	2/13/2023 8:18:35 PM
trans-1,2-Dichloroethene	ND	0.350		µg/L	1	2/13/2023 8:18:35 PM
Methyl tert-butyl ether (MTBE)	ND	0.350		µg/L	1	2/13/2023 8:18:35 PM
1,1-Dichloroethane	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
cis-1,2-Dichloroethene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
2-Butanone (MEK)	ND	1.50		µg/L	1	2/13/2023 8:18:35 PM
Chloroform	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,1,1-Trichloroethane (TCA)	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
1,1-Dichloropropene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Carbon tetrachloride	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
1,2-Dichloroethane (EDC)	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Benzene	ND	0.440		µg/L	1	2/13/2023 8:18:35 PM
Trichloroethene (TCE)	ND	0.400		µg/L	1	2/13/2023 8:18:35 PM
1,2-Dichloropropane	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
Bromodichloromethane	ND	0.250		µg/L	1	2/13/2023 8:18:35 PM

Client: AECOM **Collection Date:** 2/8/2023 12:30:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-004 **Matrix:** Groundwater
Client Sample ID: **BT-1-GW**

Volatile Organic Compounds by EPA Method 8260D

Batch ID: 39414 Analyst: SH

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Dibromomethane	ND	0.250		µg/L	1	2/13/2023 8:18:35 PM
cis-1,3-Dichloropropene	ND	0.350		µg/L	1	2/13/2023 8:18:35 PM
Toluene	ND	1.00		µg/L	1	2/13/2023 8:18:35 PM
trans-1,3-Dichloropropylene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Methyl Isobutyl Ketone (MIBK)	ND	1.00		µg/L	1	2/13/2023 8:18:35 PM
1,1,2-Trichloroethane	ND	0.250		µg/L	1	2/13/2023 8:18:35 PM
1,3-Dichloropropane	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
Tetrachloroethene (PCE)	ND	0.350		µg/L	1	2/13/2023 8:18:35 PM
Dibromochloromethane	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
1,2-Dibromoethane (EDB)	ND	0.200		µg/L	1	2/13/2023 8:18:35 PM
2-Hexanone (MBK)	ND	1.25		µg/L	1	2/13/2023 8:18:35 PM
Chlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,1,1,2-Tetrachloroethane	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
Ethylbenzene	ND	0.400		µg/L	1	2/13/2023 8:18:35 PM
m,p-Xylene	ND	1.00		µg/L	1	2/13/2023 8:18:35 PM
o-Xylene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Styrene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Isopropylbenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Bromoform	ND	0.300		µg/L	1	2/13/2023 8:18:35 PM
1,1,2,2-Tetrachloroethane	ND	0.200		µg/L	1	2/13/2023 8:18:35 PM
n-Propylbenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Bromobenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,3,5-Trimethylbenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
2-Chlorotoluene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
4-Chlorotoluene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
tert-Butylbenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,2,3-Trichloropropane	ND	0.400		µg/L	1	2/13/2023 8:18:35 PM
1,2,4-Trichlorobenzene	ND	0.750		µg/L	1	2/13/2023 8:18:35 PM
sec-Butylbenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
4-Isopropyltoluene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,3-Dichlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,4-Dichlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
n-Butylbenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,2-Dichlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
1,2-Dibromo-3-chloropropane	ND	1.00		µg/L	1	2/13/2023 8:18:35 PM
1,2,4-Trimethylbenzene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Hexachloro-1,3-butadiene	ND	0.500		µg/L	1	2/13/2023 8:18:35 PM
Naphthalene	ND	1.25		µg/L	1	2/13/2023 8:18:35 PM
1,2,3-Trichlorobenzene	ND	0.700		µg/L	1	2/13/2023 8:18:35 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 12:30:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-004
Matrix: Groundwater
Client Sample ID: **BT-1-GW**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39414		Analyst: SH
Surr: Dibromofluoromethane	100	80 - 120		%Rec	1	2/13/2023 8:18:35 PM
Surr: Toluene-d8	102	80 - 120		%Rec	1	2/13/2023 8:18:35 PM
Surr: 1-Bromo-4-fluorobenzene	99.4	80 - 120		%Rec	1	2/13/2023 8:18:35 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 12:36:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-006
Matrix: Soil
Client Sample ID: **BT-2-7**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39422		Analyst: KJ
Diesel Range Organics	ND	51.1		mg/Kg-dry	1	2/14/2023 9:55:50 AM
Heavy Oil	ND	102		mg/Kg-dry	1	2/14/2023 9:55:50 AM
Total Petroleum Hydrocarbons	ND	153		mg/Kg-dry	1	2/14/2023 9:55:50 AM
Surr: 2-Fluorobiphenyl	92.2	50 - 150		%Rec	1	2/14/2023 9:55:50 AM
Surr: o-Terphenyl	95.0	50 - 150		%Rec	1	2/14/2023 9:55:50 AM
Gasoline by NWTPH-Gx				Batch ID: 39415		Analyst: CC
Gasoline Range Organics	ND	4.85		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Surr: Toluene-d8	93.0	65 - 135		%Rec	1	2/14/2023 6:09:40 PM
Surr: 4-Bromofluorobenzene	89.4	65 - 135		%Rec	1	2/14/2023 6:09:40 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Dichlorodifluoromethane (CFC-12)	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Chloromethane	ND	0.0485		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Vinyl chloride	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Bromomethane	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Trichlorofluoromethane (CFC-11)	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Chloroethane	ND	0.0728		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,1-Dichloroethene	ND	0.0971		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Acetone	ND	0.243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Methylene chloride	ND	0.0340		mg/Kg-dry	1	2/14/2023 6:09:40 PM
trans-1,2-Dichloroethene	ND	0.00971		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Methyl tert-butyl ether (MTBE)	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,1-Dichloroethane	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
cis-1,2-Dichloroethene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
(MEK) 2-Butanone	ND	0.291		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Chloroform	ND	0.0170		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,1,1-Trichloroethane (TCA)	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,1-Dichloropropene	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Carbon tetrachloride	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2-Dichloroethane (EDC)	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Benzene	ND	0.0170		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Trichloroethene (TCE)	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2-Dichloropropane	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Bromodichloromethane	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Dibromomethane	ND	0.0121		mg/Kg-dry	1	2/14/2023 6:09:40 PM
cis-1,3-Dichloropropene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM

Client: AECOM **Collection Date:** 2/8/2023 12:36:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-006 **Matrix:** Soil
Client Sample ID: **BT-2-7**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Toluene	ND	0.0291		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Trans-1,3-Dichloropropylene	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Methyl Isobutyl Ketone (MIBK)	ND	0.0582		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,1,2-Trichloroethane	ND	0.0121		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,3-Dichloropropane	ND	0.00971		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Tetrachloroethene (PCE)	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Dibromochloromethane	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2-Dibromoethane (EDB)	ND	0.00971		mg/Kg-dry	1	2/14/2023 6:09:40 PM
2-Hexanone (MBK)	ND	0.0607		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Chlorobenzene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,1,1,2-Tetrachloroethane	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Ethylbenzene	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
m,p-Xylene	ND	0.0485		mg/Kg-dry	1	2/14/2023 6:09:40 PM
o-Xylene	ND	0.0243		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Styrene	ND	0.00971		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Isopropylbenzene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Bromoform	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,1,2,2-Tetrachloroethane	ND	0.194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
n-Propylbenzene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Bromobenzene	ND	0.0121		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,3,5-Trimethylbenzene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
2-Chlorotoluene	ND	0.0160		mg/Kg-dry	1	2/14/2023 6:09:40 PM
4-Chlorotoluene	ND	0.0160		mg/Kg-dry	1	2/14/2023 6:09:40 PM
tert-Butylbenzene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2,3-Trichloropropane	ND	0.0291		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2,4-Trichlorobenzene	ND	0.0582		mg/Kg-dry	1	2/14/2023 6:09:40 PM
sec-Butylbenzene	ND	0.146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
4-Isopropyltoluene	ND	0.194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,3-Dichlorobenzene	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,4-Dichlorobenzene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
n-Butylbenzene	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2-Dichlorobenzene	ND	0.0194		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2-Dibromo-3-chloropropane	ND	0.0291		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2,4-Trimethylbenzene	ND	0.0146		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Hexachloro-1,3-butadiene	ND	0.0388		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Naphthalene	ND	0.0971		mg/Kg-dry	1	2/14/2023 6:09:40 PM
1,2,3-Trichlorobenzene	ND	0.0582		mg/Kg-dry	1	2/14/2023 6:09:40 PM
Surr: Dibromofluoromethane	98.7	80 - 120		%Rec	1	2/14/2023 6:09:40 PM
Surr: Toluene-d8	109	80 - 120		%Rec	1	2/14/2023 6:09:40 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 12:36:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-006
Matrix: Soil
Client Sample ID: **BT-2-7**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Surr: 1-Bromo-4-fluorobenzene	98.9	80 - 120		%Rec	1	2/14/2023 6:09:40 PM
Sample Moisture (Percent Moisture)				Batch ID: R81805		Analyst: MP
Percent Moisture	6.17	0.500		wt%	1	2/13/2023 8:28:57 AM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 12:42:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-007
Matrix: Soil
Client Sample ID: **BT-2-10**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39422		Analyst: KJ
Diesel Range Organics	ND	59.2		mg/Kg-dry	1	2/14/2023 10:06:55 AM
Heavy Oil	ND	118		mg/Kg-dry	1	2/14/2023 10:06:55 AM
Total Petroleum Hydrocarbons	ND	178		mg/Kg-dry	1	2/14/2023 10:06:55 AM
Surr: 2-Fluorobiphenyl	111	50 - 150		%Rec	1	2/14/2023 10:06:55 AM
Surr: o-Terphenyl	114	50 - 150		%Rec	1	2/14/2023 10:06:55 AM
Gasoline by NWTPH-Gx				Batch ID: 39415		Analyst: CC
Gasoline Range Organics	ND	5.00		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Surr: Toluene-d8	91.0	65 - 135		%Rec	1	2/14/2023 6:40:41 PM
Surr: 4-Bromofluorobenzene	87.7	65 - 135		%Rec	1	2/14/2023 6:40:41 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Dichlorodifluoromethane (CFC-12)	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Chloromethane	ND	0.0500		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Vinyl chloride	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Bromomethane	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Trichlorofluoromethane (CFC-11)	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Chloroethane	ND	0.0749		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,1-Dichloroethene	ND	0.0999		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Acetone	ND	0.250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Methylene chloride	ND	0.0350		mg/Kg-dry	1	2/14/2023 6:40:41 PM
trans-1,2-Dichloroethene	ND	0.00999		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Methyl tert-butyl ether (MTBE)	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,1-Dichloroethane	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
cis-1,2-Dichloroethene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
(MEK) 2-Butanone	ND	0.300		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Chloroform	ND	0.0175		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,1,1-Trichloroethane (TCA)	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,1-Dichloropropene	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Carbon tetrachloride	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2-Dichloroethane (EDC)	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Benzene	ND	0.0175		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Trichloroethene (TCE)	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2-Dichloropropane	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Bromodichloromethane	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Dibromomethane	ND	0.0125		mg/Kg-dry	1	2/14/2023 6:40:41 PM
cis-1,3-Dichloropropene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM

Client: AECOM **Collection Date:** 2/8/2023 12:42:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-007 **Matrix:** Soil
Client Sample ID: **BT-2-10**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Toluene	ND	0.0300		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Trans-1,3-Dichloropropylene	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Methyl Isobutyl Ketone (MIBK)	ND	0.0600		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,1,2-Trichloroethane	ND	0.0125		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,3-Dichloropropane	ND	0.00999		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Tetrachloroethene (PCE)	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Dibromochloromethane	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2-Dibromoethane (EDB)	ND	0.00999		mg/Kg-dry	1	2/14/2023 6:40:41 PM
2-Hexanone (MBK)	ND	0.0624		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Chlorobenzene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,1,1,2-Tetrachloroethane	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Ethylbenzene	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
m,p-Xylene	ND	0.0500		mg/Kg-dry	1	2/14/2023 6:40:41 PM
o-Xylene	ND	0.0250		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Styrene	ND	0.00999		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Isopropylbenzene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Bromoform	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,1,2,2-Tetrachloroethane	ND	0.200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
n-Propylbenzene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Bromobenzene	ND	0.0125		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,3,5-Trimethylbenzene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
2-Chlorotoluene	ND	0.0165		mg/Kg-dry	1	2/14/2023 6:40:41 PM
4-Chlorotoluene	ND	0.0165		mg/Kg-dry	1	2/14/2023 6:40:41 PM
tert-Butylbenzene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2,3-Trichloropropane	ND	0.0300		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2,4-Trichlorobenzene	ND	0.0600		mg/Kg-dry	1	2/14/2023 6:40:41 PM
sec-Butylbenzene	ND	0.150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
4-Isopropyltoluene	ND	0.200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,3-Dichlorobenzene	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,4-Dichlorobenzene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
n-Butylbenzene	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2-Dichlorobenzene	ND	0.0200		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2-Dibromo-3-chloropropane	ND	0.0300		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2,4-Trimethylbenzene	ND	0.0150		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Hexachloro-1,3-butadiene	ND	0.0400		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Naphthalene	ND	0.0999		mg/Kg-dry	1	2/14/2023 6:40:41 PM
1,2,3-Trichlorobenzene	ND	0.0600		mg/Kg-dry	1	2/14/2023 6:40:41 PM
Surr: Dibromofluoromethane	99.2	80 - 120		%Rec	1	2/14/2023 6:40:41 PM
Surr: Toluene-d8	109	80 - 120		%Rec	1	2/14/2023 6:40:41 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/8/2023 12:42:00 AM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-007
Matrix: Soil
Client Sample ID: **BT-2-10**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Surr: 1-Bromo-4-fluorobenzene	97.1	80 - 120		%Rec	1	2/14/2023 6:40:41 PM
Sample Moisture (Percent Moisture)				Batch ID: R81805		Analyst: MP
Percent Moisture	20.9	0.500		wt%	1	2/13/2023 8:28:57 AM

Client: AECOM **Collection Date:** 2/7/2023 9:22:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-008 **Matrix:** Soil
Client Sample ID: **BT-3-5.5**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39422		Analyst: KJ
Diesel Range Organics	ND	54.0		mg/Kg-dry	1	2/14/2023 10:20:23 AM
Heavy Oil	ND	108		mg/Kg-dry	1	2/14/2023 10:20:23 AM
Total Petroleum Hydrocarbons	ND	162		mg/Kg-dry	1	2/14/2023 10:20:23 AM
Surr: 2-Fluorobiphenyl	89.1	50 - 150		%Rec	1	2/14/2023 10:20:23 AM
Surr: o-Terphenyl	92.0	50 - 150		%Rec	1	2/14/2023 10:20:23 AM
Gasoline by NWTPH-Gx				Batch ID: 39415		Analyst: CC
Gasoline Range Organics	ND	4.84		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Surr: Toluene-d8	91.6	65 - 135		%Rec	1	2/14/2023 7:11:48 PM
Surr: 4-Bromofluorobenzene	88.6	65 - 135		%Rec	1	2/14/2023 7:11:48 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Dichlorodifluoromethane (CFC-12)	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Chloromethane	ND	0.0484		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Vinyl chloride	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Bromomethane	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Trichlorofluoromethane (CFC-11)	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Chloroethane	ND	0.0727		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,1-Dichloroethene	ND	0.0969		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Acetone	ND	0.242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Methylene chloride	ND	0.0339		mg/Kg-dry	1	2/14/2023 7:11:48 PM
trans-1,2-Dichloroethene	ND	0.00969		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Methyl tert-butyl ether (MTBE)	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,1-Dichloroethane	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
cis-1,2-Dichloroethene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
(MEK) 2-Butanone	ND	0.291		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Chloroform	ND	0.0170		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,1,1-Trichloroethane (TCA)	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,1-Dichloropropene	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Carbon tetrachloride	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2-Dichloroethane (EDC)	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Benzene	ND	0.0170		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Trichloroethene (TCE)	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2-Dichloropropane	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Bromodichloromethane	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Dibromomethane	ND	0.0121		mg/Kg-dry	1	2/14/2023 7:11:48 PM
cis-1,3-Dichloropropene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM

Client: AECOM **Collection Date:** 2/7/2023 9:22:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-008 **Matrix:** Soil
Client Sample ID: **BT-3-5.5**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Toluene	ND	0.0291		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Trans-1,3-Dichloropropylene	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Methyl Isobutyl Ketone (MIBK)	ND	0.0581		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,1,2-Trichloroethane	ND	0.0121		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,3-Dichloropropane	ND	0.00969		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Tetrachloroethene (PCE)	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Dibromochloromethane	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2-Dibromoethane (EDB)	ND	0.00969		mg/Kg-dry	1	2/14/2023 7:11:48 PM
2-Hexanone (MBK)	ND	0.0606		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Chlorobenzene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,1,1,2-Tetrachloroethane	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Ethylbenzene	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
m,p-Xylene	ND	0.0484		mg/Kg-dry	1	2/14/2023 7:11:48 PM
o-Xylene	ND	0.0242		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Styrene	ND	0.00969		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Isopropylbenzene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Bromoform	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,1,2,2-Tetrachloroethane	ND	0.194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
n-Propylbenzene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Bromobenzene	ND	0.0121		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,3,5-Trimethylbenzene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
2-Chlorotoluene	ND	0.0160		mg/Kg-dry	1	2/14/2023 7:11:48 PM
4-Chlorotoluene	ND	0.0160		mg/Kg-dry	1	2/14/2023 7:11:48 PM
tert-Butylbenzene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2,3-Trichloropropane	ND	0.0291		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2,4-Trichlorobenzene	ND	0.0581		mg/Kg-dry	1	2/14/2023 7:11:48 PM
sec-Butylbenzene	ND	0.145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
4-Isopropyltoluene	ND	0.194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,3-Dichlorobenzene	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,4-Dichlorobenzene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
n-Butylbenzene	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2-Dichlorobenzene	ND	0.0194		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2-Dibromo-3-chloropropane	ND	0.0291		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2,4-Trimethylbenzene	ND	0.0145		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Hexachloro-1,3-butadiene	ND	0.0388		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Naphthalene	ND	0.0969		mg/Kg-dry	1	2/14/2023 7:11:48 PM
1,2,3-Trichlorobenzene	ND	0.0581		mg/Kg-dry	1	2/14/2023 7:11:48 PM
Surr: Dibromofluoromethane	99.0	80 - 120		%Rec	1	2/14/2023 7:11:48 PM
Surr: Toluene-d8	109	80 - 120		%Rec	1	2/14/2023 7:11:48 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/7/2023 9:22:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-008
Matrix: Soil
Client Sample ID: **BT-3-5.5**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Surr: 1-Bromo-4-fluorobenzene	98.0	80 - 120		%Rec	1	2/14/2023 7:11:48 PM
Sample Moisture (Percent Moisture)				Batch ID: R81805		Analyst: MP
Percent Moisture	11.4	0.500		wt%	1	2/13/2023 8:28:57 AM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/7/2023 9:32:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-010
Matrix: Soil
Client Sample ID: **BT-3-10**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39422		Analyst: KJ
Diesel Range Organics	ND	59.6		mg/Kg-dry	1	2/14/2023 10:31:12 AM
Heavy Oil	ND	119		mg/Kg-dry	1	2/14/2023 10:31:12 AM
Total Petroleum Hydrocarbons	ND	179		mg/Kg-dry	1	2/14/2023 10:31:12 AM
Surr: 2-Fluorobiphenyl	100	50 - 150		%Rec	1	2/14/2023 10:31:12 AM
Surr: o-Terphenyl	103	50 - 150		%Rec	1	2/14/2023 10:31:12 AM
Gasoline by NWTPH-Gx				Batch ID: 39415		Analyst: CC
Gasoline Range Organics	ND	4.43		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Surr: Toluene-d8	91.7	65 - 135		%Rec	1	2/14/2023 7:42:52 PM
Surr: 4-Bromofluorobenzene	88.7	65 - 135		%Rec	1	2/14/2023 7:42:52 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Dichlorodifluoromethane (CFC-12)	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Chloromethane	ND	0.0443		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Vinyl chloride	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Bromomethane	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Trichlorofluoromethane (CFC-11)	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Chloroethane	ND	0.0664		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,1-Dichloroethene	ND	0.0886		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Acetone	ND	0.221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Methylene chloride	ND	0.0310		mg/Kg-dry	1	2/14/2023 7:42:52 PM
trans-1,2-Dichloroethene	ND	0.00886		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Methyl tert-butyl ether (MTBE)	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,1-Dichloroethane	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
cis-1,2-Dichloroethene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
(MEK) 2-Butanone	ND	0.266		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Chloroform	ND	0.0155		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,1,1-Trichloroethane (TCA)	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,1-Dichloropropene	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Carbon tetrachloride	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2-Dichloroethane (EDC)	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Benzene	ND	0.0155		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Trichloroethene (TCE)	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2-Dichloropropane	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Bromodichloromethane	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Dibromomethane	ND	0.0111		mg/Kg-dry	1	2/14/2023 7:42:52 PM
cis-1,3-Dichloropropene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM

Client: AECOM **Collection Date:** 2/7/2023 9:32:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-010 **Matrix:** Soil
Client Sample ID: **BT-3-10**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Toluene	ND	0.0266		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Trans-1,3-Dichloropropylene	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Methyl Isobutyl Ketone (MIBK)	ND	0.0531		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,1,2-Trichloroethane	ND	0.0111		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,3-Dichloropropane	ND	0.00886		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Tetrachloroethene (PCE)	0.0326	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Dibromochloromethane	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2-Dibromoethane (EDB)	ND	0.00886		mg/Kg-dry	1	2/14/2023 7:42:52 PM
2-Hexanone (MBK)	ND	0.0554		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Chlorobenzene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,1,1,2-Tetrachloroethane	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Ethylbenzene	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
m,p-Xylene	ND	0.0443		mg/Kg-dry	1	2/14/2023 7:42:52 PM
o-Xylene	ND	0.0221		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Styrene	ND	0.00886		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Isopropylbenzene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Bromoform	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,1,2,2-Tetrachloroethane	ND	0.177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
n-Propylbenzene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Bromobenzene	ND	0.0111		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,3,5-Trimethylbenzene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
2-Chlorotoluene	ND	0.0146		mg/Kg-dry	1	2/14/2023 7:42:52 PM
4-Chlorotoluene	ND	0.0146		mg/Kg-dry	1	2/14/2023 7:42:52 PM
tert-Butylbenzene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2,3-Trichloropropane	ND	0.0266		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2,4-Trichlorobenzene	ND	0.0531		mg/Kg-dry	1	2/14/2023 7:42:52 PM
sec-Butylbenzene	ND	0.133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
4-Isopropyltoluene	ND	0.177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,3-Dichlorobenzene	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,4-Dichlorobenzene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
n-Butylbenzene	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2-Dichlorobenzene	ND	0.0177		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2-Dibromo-3-chloropropane	ND	0.0266		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2,4-Trimethylbenzene	ND	0.0133		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Hexachloro-1,3-butadiene	ND	0.0354		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Naphthalene	ND	0.0886		mg/Kg-dry	1	2/14/2023 7:42:52 PM
1,2,3-Trichlorobenzene	ND	0.0531		mg/Kg-dry	1	2/14/2023 7:42:52 PM
Surr: Dibromofluoromethane	100	80 - 120		%Rec	1	2/14/2023 7:42:52 PM
Surr: Toluene-d8	110	80 - 120		%Rec	1	2/14/2023 7:42:52 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/7/2023 9:32:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-010
Matrix: Soil
Client Sample ID: **BT-3-10**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39415		Analyst: CC
Surr: 1-Bromo-4-fluorobenzene	98.2	80 - 120		%Rec	1	2/14/2023 7:42:52 PM
Sample Moisture (Percent Moisture)				Batch ID: R81813		Analyst: MP
Percent Moisture	17.9	0.500		wt%	1	2/13/2023 11:14:33 AM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/7/2023 11:00:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-011
Matrix: Groundwater
Client Sample ID: **BT-3-GW**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.				Batch ID: 39419		Analyst: KJ
Diesel Range Organics	ND	97.7		µg/L	1	2/14/2023 1:04:28 PM
Heavy Oil	123	97.7		µg/L	1	2/14/2023 1:04:28 PM
Total Petroleum Hydrocarbons	ND	195		µg/L	1	2/14/2023 1:04:28 PM
Surr: 2-Fluorobiphenyl	108	50 - 150		%Rec	1	2/14/2023 1:04:28 PM
Surr: o-Terphenyl	81.2	50 - 150		%Rec	1	2/14/2023 1:04:28 PM
NOTES: Chromatographic pattern is not consistent with a petroleum standard						
Gasoline by NWTPH-Gx				Batch ID: 39414		Analyst: SH
Gasoline Range Organics	ND	50.0		µg/L	1	2/13/2023 8:48:42 PM
Surr: Toluene-d8	100	65 - 135		%Rec	1	2/13/2023 8:48:42 PM
Surr: 4-Bromofluorobenzene	98.6	65 - 135		%Rec	1	2/13/2023 8:48:42 PM
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39414		Analyst: SH
Dichlorodifluoromethane (CFC-12)	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Chloromethane	ND	0.750		µg/L	1	2/13/2023 8:48:42 PM
Vinyl chloride	ND	0.200		µg/L	1	2/13/2023 8:48:42 PM
Bromomethane	ND	3.00		µg/L	1	2/13/2023 8:48:42 PM
Trichlorofluoromethane (CFC-11)	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
Chloroethane	ND	1.00		µg/L	1	2/13/2023 8:48:42 PM
1,1-Dichloroethene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Acetone	ND	5.00		µg/L	1	2/13/2023 8:48:42 PM
Methylene chloride	ND	0.750		µg/L	1	2/13/2023 8:48:42 PM
trans-1,2-Dichloroethene	ND	0.350		µg/L	1	2/13/2023 8:48:42 PM
Methyl tert-butyl ether (MTBE)	ND	0.350		µg/L	1	2/13/2023 8:48:42 PM
1,1-Dichloroethane	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
cis-1,2-Dichloroethene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
2-Butanone (MEK)	ND	1.50		µg/L	1	2/13/2023 8:48:42 PM
Chloroform	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,1,1-Trichloroethane (TCA)	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
1,1-Dichloropropene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Carbon tetrachloride	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
1,2-Dichloroethane (EDC)	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Benzene	ND	0.440		µg/L	1	2/13/2023 8:48:42 PM
Trichloroethene (TCE)	ND	0.400		µg/L	1	2/13/2023 8:48:42 PM
1,2-Dichloropropane	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
Bromodichloromethane	ND	0.250		µg/L	1	2/13/2023 8:48:42 PM

Client: AECOM
Collection Date: 2/7/2023 11:00:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-011
Matrix: Groundwater
Client Sample ID: **BT-3-GW**

Volatile Organic Compounds by EPA Method 8260D

Batch ID: 39414 Analyst: SH

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Dibromomethane	ND	0.250		µg/L	1	2/13/2023 8:48:42 PM
cis-1,3-Dichloropropene	ND	0.350		µg/L	1	2/13/2023 8:48:42 PM
Toluene	4.82	1.00		µg/L	1	2/13/2023 8:48:42 PM
trans-1,3-Dichloropropylene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Methyl Isobutyl Ketone (MIBK)	ND	1.00		µg/L	1	2/13/2023 8:48:42 PM
1,1,2-Trichloroethane	ND	0.250		µg/L	1	2/13/2023 8:48:42 PM
1,3-Dichloropropane	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
Tetrachloroethene (PCE)	3.53	0.350		µg/L	1	2/13/2023 8:48:42 PM
Dibromochloromethane	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
1,2-Dibromoethane (EDB)	ND	0.200		µg/L	1	2/13/2023 8:48:42 PM
2-Hexanone (MBK)	ND	1.25		µg/L	1	2/13/2023 8:48:42 PM
Chlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,1,1,2-Tetrachloroethane	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
Ethylbenzene	ND	0.400		µg/L	1	2/13/2023 8:48:42 PM
m,p-Xylene	ND	1.00		µg/L	1	2/13/2023 8:48:42 PM
o-Xylene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Styrene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Isopropylbenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Bromoform	ND	0.300		µg/L	1	2/13/2023 8:48:42 PM
1,1,2,2-Tetrachloroethane	ND	0.200		µg/L	1	2/13/2023 8:48:42 PM
n-Propylbenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Bromobenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,3,5-Trimethylbenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
2-Chlorotoluene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
4-Chlorotoluene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
tert-Butylbenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,2,3-Trichloropropane	ND	0.400		µg/L	1	2/13/2023 8:48:42 PM
1,2,4-Trichlorobenzene	ND	0.750		µg/L	1	2/13/2023 8:48:42 PM
sec-Butylbenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
4-Isopropyltoluene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,3-Dichlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,4-Dichlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
n-Butylbenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,2-Dichlorobenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
1,2-Dibromo-3-chloropropane	ND	1.00		µg/L	1	2/13/2023 8:48:42 PM
1,2,4-Trimethylbenzene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Hexachloro-1,3-butadiene	ND	0.500		µg/L	1	2/13/2023 8:48:42 PM
Naphthalene	ND	1.25		µg/L	1	2/13/2023 8:48:42 PM
1,2,3-Trichlorobenzene	ND	0.700		µg/L	1	2/13/2023 8:48:42 PM

Analytical Report

Work Order: **2302195**
Date Reported: **2/20/2023**

Client: AECOM
Collection Date: 2/7/2023 11:00:00 PM
Project: Woodinville SR202 Trestle Widening
Lab ID: 2302195-011
Matrix: Groundwater
Client Sample ID: **BT-3-GW**

Analyses	Result	RL	Qual	Units	DF	Date Analyzed
Volatile Organic Compounds by EPA Method 8260D				Batch ID: 39414		Analyst: SH
Surr: Dibromofluoromethane	103	80 - 120		%Rec	1	2/13/2023 8:48:42 PM
Surr: Toluene-d8	103	80 - 120		%Rec	1	2/13/2023 8:48:42 PM
Surr: 1-Bromo-4-fluorobenzene	98.0	80 - 120		%Rec	1	2/13/2023 8:48:42 PM

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.

Sample ID: **MB-39422** SampType: **MBLK** Units: **mg/Kg** Prep Date: **2/13/2023** RunNo: **81847**
Client ID: **MBLKS** Batch ID: **39422** Analysis Date: **2/14/2023** SeqNo: **1697056**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Diesel Range Organics	ND	50.0									
Heavy Oil	ND	100									
Total Petroleum Hydrocarbons	ND	150									
Surr: 2-Fluorobiphenyl	11.5		10.00		115	50	150				
Surr: o-Terphenyl	12.0		10.00		120	50	150				

Sample ID: **LCS-39422** SampType: **LCS** Units: **mg/Kg** Prep Date: **2/13/2023** RunNo: **81847**
Client ID: **LCSS** Batch ID: **39422** Analysis Date: **2/14/2023** SeqNo: **1697057**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Total Petroleum Hydrocarbons	503	150	500.0	0	101	74.5	125				
Surr: 2-Fluorobiphenyl	8.83		10.00		88.3	50	150				
Surr: o-Terphenyl	11.3		10.00		113	50	150				

Sample ID: **2302195-001AMS** SampType: **MS** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81847**
Client ID: **BT-1-8** Batch ID: **39422** Analysis Date: **2/14/2023** SeqNo: **1697059**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Total Petroleum Hydrocarbons	569	172	574.2	0	99.1	50.3	140				
Surr: 2-Fluorobiphenyl	11.0		11.48		96.0	50	150				
Surr: o-Terphenyl	12.4		11.48		108	50	150				

Sample ID: **2302195-001AMSD** SampType: **MSD** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81847**
Client ID: **BT-1-8** Batch ID: **39422** Analysis Date: **2/14/2023** SeqNo: **1697060**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Total Petroleum Hydrocarbons	556	172	574.8	0	96.8	50.3	140	568.9	2.26	30	
Surr: 2-Fluorobiphenyl	10.6		11.50		91.9	50	150		0		
Surr: o-Terphenyl	12.3		11.50		107	50	150		0		

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.

Sample ID: **2302195-003ADUP**	SampType: **DUP**	Units: **mg/Kg-dry**	Prep Date: **2/13/2023**	RunNo: **81847**
Client ID: **BT-1-12**	Batch ID: **39422**		Analysis Date: **2/14/2023**	SeqNo: **1697062**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Diesel Range Organics	ND	60.4						0		30	
Heavy Oil	ND	121						0		30	
Total Petroleum Hydrocarbons	ND	181						0		30	
Surr: 2-Fluorobiphenyl	10.6		12.08		87.6	50	150		0		
Surr: o-Terphenyl	11.0		12.08		90.9	50	150		0		

Sample ID: **2302203-003ADUP**	SampType: **DUP**	Units: **mg/Kg-dry**	Prep Date: **2/13/2023**	RunNo: **81847**
Client ID: **BATCH**	Batch ID: **39422**		Analysis Date: **2/14/2023**	SeqNo: **1697369**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Diesel Range Organics	ND	52.7						0		30	
Heavy Oil	ND	105						0		30	
Total Petroleum Hydrocarbons	ND	158						0		30	
Surr: 2-Fluorobiphenyl	12.7		10.53		121	50	150		0		
Surr: o-Terphenyl	13.1		10.53		125	50	150		0		

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.

Sample ID: **MB-39419**	SampType: **MBLK**			Units: **µg/L**		Prep Date: **2/13/2023**		RunNo: **81833**		
Client ID: **MBLKW**	Batch ID: **39419**					Analysis Date: **2/13/2023**		SeqNo: **1696845**		
Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Diesel Range Organics	ND	94.1									
Heavy Oil	ND	94.1									
Total Petroleum Hydrocarbons	ND	188									
Surr: 2-Fluorobiphenyl	15.9		23.53		67.5	50	150				
Surr: o-Terphenyl	21.7		23.53		92.1	50	150				

Sample ID: **LCS-39419**	SampType: **LCS**			Units: **µg/L**		Prep Date: **2/13/2023**		RunNo: **81833**		
Client ID: **LCSW**	Batch ID: **39419**					Analysis Date: **2/13/2023**		SeqNo: **1696846**		
Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Total Petroleum Hydrocarbons	856	190	1,187	0	72.1	45.7	115				
Surr: 2-Fluorobiphenyl	22.4		23.74		94.3	50	150				
Surr: o-Terphenyl	29.5		23.74		124	50	150				

Sample ID: **LCSD-39419**	SampType: **LCSD**			Units: **µg/L**		Prep Date: **2/13/2023**		RunNo: **81833**		
Client ID: **LCSW02**	Batch ID: **39419**					Analysis Date: **2/13/2023**		SeqNo: **1696847**		
Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Total Petroleum Hydrocarbons	933	190	1,187	0	78.6	45.7	115	856.3	8.55	30	
Surr: 2-Fluorobiphenyl	21.1		23.74		88.8	50	150		0		
Surr: o-Terphenyl	29.8		23.74		125	50	150		0		

Sample ID: **2302215-010BMS**	SampType: **MS**			Units: **µg/L**		Prep Date: **2/13/2023**		RunNo: **81833**		
Client ID: **BATCH**	Batch ID: **39419**					Analysis Date: **2/16/2023**		SeqNo: **1699065**		
Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Total Petroleum Hydrocarbons	1,090	187	1,167	202.4	76.0	31.9	125				
Surr: 2-Fluorobiphenyl	20.7		23.35		88.5	50	150				
Surr: o-Terphenyl	27.1		23.35		116	50	150				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Diesel and Heavy Oil by NWTPH-Dx/Dx Ext.

Sample ID: **2302215-010BMSD** SampType: **MSD** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81833**
Client ID: **BATCH** Batch ID: **39419** Analysis Date: **2/16/2023** SeqNo: **1699066**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Total Petroleum Hydrocarbons	1,240	186	1,165	202.4	89.1	31.9	125	1,089	13.0	30	
Surr: 2-Fluorobiphenyl	21.8		23.29		93.7	50	150		0		
Surr: o-Terphenyl	28.7		23.29		123	50	150		0		

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Gasoline by NWTPH-Gx

Sample ID: **2302057-025BDUP**	SampType: **DUP**	Units: **mg/Kg-dry**	Prep Date: **2/13/2023**	RunNo: **81882**
Client ID: **BATCH**	Batch ID: **39415**		Analysis Date: **2/14/2023**	SeqNo: **1697959**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	ND	10.5						0		30	
Surr: Toluene-d8	2.44		2.629		92.8	65	135		0		
Surr: 4-Bromofluorobenzene	2.35		2.629		89.3	65	135		0		

Sample ID: **2302195-003BMS**	SampType: **MS**	Units: **mg/Kg-dry**	Prep Date: **2/13/2023**	RunNo: **81882**
Client ID: **BT-1-12**	Batch ID: **39415**		Analysis Date: **2/14/2023**	SeqNo: **1697967**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	15.5	4.49	22.43	0	69.0	65	135				
Surr: Toluene-d8	1.04		1.122		92.6	65	135				
Surr: 4-Bromofluorobenzene	1.02		1.122		90.9	65	135				

Sample ID: **2302195-001BDUP**	SampType: **DUP**	Units: **mg/Kg-dry**	Prep Date: **2/13/2023**	RunNo: **81882**
Client ID: **BT-1-8**	Batch ID: **39415**		Analysis Date: **2/14/2023**	SeqNo: **1697965**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	ND	4.64						0		30	
Surr: Toluene-d8	1.07		1.159		92.3	65	135		0		
Surr: 4-Bromofluorobenzene	1.03		1.159		88.5	65	135		0		

Sample ID: **LCS-39415**	SampType: **LCS**	Units: **mg/Kg**	Prep Date: **2/13/2023**	RunNo: **81882**
Client ID: **LCSS**	Batch ID: **39415**		Analysis Date: **2/15/2023**	SeqNo: **1698003**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	25.0	5.00	25.00	0	99.9	65	135				
Surr: Toluene-d8	1.17		1.250		93.9	65	135				
Surr: 4-Bromofluorobenzene	1.15		1.250		91.8	65	135				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Gasoline by NWTPH-Gx

Sample ID: **MB-39415** SampType: **MBLK** Units: **mg/Kg** Prep Date: **2/13/2023** RunNo: **81882**
Client ID: **MBLKS** Batch ID: **39415** Analysis Date: **2/15/2023** SeqNo: **1697998**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	ND	5.00									
Surr: Toluene-d8	1.17		1.250		93.5	65	135				
Surr: 4-Bromofluorobenzene	1.11		1.250		89.1	65	135				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Gasoline by NWTPH-Gx

Sample ID: **LCS-39414**	SampType: **LCS**	Units: **µg/L**	Prep Date: **2/13/2023**	RunNo: **81839**
Client ID: **LCSW**	Batch ID: **39414**		Analysis Date: **2/13/2023**	SeqNo: **1696962**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	553	50.0	500.0	0	111	65	135				
Surr: Toluene-d8	25.2		25.00		101	65	135				
Surr: 4-Bromofluorobenzene	25.4		25.00		102	65	135				

Sample ID: **MB-39414**	SampType: **MBLK**	Units: **µg/L**	Prep Date: **2/13/2023**	RunNo: **81839**
Client ID: **MBLKW**	Batch ID: **39414**		Analysis Date: **2/13/2023**	SeqNo: **1696955**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	ND	50.0									
Surr: Toluene-d8	25.7		25.00		103	65	135				
Surr: 4-Bromofluorobenzene	25.2		25.00		101	65	135				

Sample ID: **2302186-001ADUP**	SampType: **DUP**	Units: **µg/L**	Prep Date: **2/13/2023**	RunNo: **81839**
Client ID: **BATCH**	Batch ID: **39414**		Analysis Date: **2/13/2023**	SeqNo: **1696929**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	ND	50.0						0		30	
Surr: Toluene-d8	25.6		25.00		102	65	135		0		
Surr: 4-Bromofluorobenzene	25.2		25.00		101	65	135		0		

Sample ID: **2302215-015ADUP**	SampType: **DUP**	Units: **µg/L**	Prep Date: **2/13/2023**	RunNo: **81839**
Client ID: **BATCH**	Batch ID: **39414**		Analysis Date: **2/14/2023**	SeqNo: **1696948**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	ND	50.0						0		30	
Surr: Toluene-d8	26.1		25.00		104	65	135		0		
Surr: 4-Bromofluorobenzene	25.4		25.00		102	65	135		0		

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Gasoline by NWTPH-Gx

Sample ID: **2302195-011AMS** SampType: **MS** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81839**
Client ID: **BT-3-GW** Batch ID: **39414** Analysis Date: **2/14/2023** SeqNo: **1696932**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Gasoline Range Organics	457	50.0	500.0	47.38	82.0	65	135				
Surr: Toluene-d8	24.2		25.00		96.6	65	135				
Surr: 4-Bromofluorobenzene	24.3		25.00		97.0	65	135				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **LCS-39415** SampType: **LCS** Units: **mg/Kg** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **LCSS** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1700313**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	0.892	0.0150	1.000	0	89.2	80	120				
Chloromethane	0.964	0.0500	1.000	0	96.4	80	120				
Vinyl chloride	0.919	0.0250	1.000	0	91.9	80	120				
Bromomethane	0.881	0.0250	1.000	0	88.1	80	120				
Trichlorofluoromethane (CFC-11)	1.03	0.0200	1.000	0	103	80	120				
Chloroethane	0.990	0.0750	1.000	0	99.0	80	120				
1,1-Dichloroethene	1.04	0.100	1.000	0	104	80	120				
Acetone	2.25	0.250	2.500	0	89.9	80	120				
Methylene chloride	1.14	0.0350	1.000	0	114	80	120				
trans-1,2-Dichloroethene	1.04	0.0100	1.000	0	104	80	120				
Methyl tert-butyl ether (MTBE)	1.02	0.0200	1.000	0	102	80	120				
1,1-Dichloroethane	1.05	0.0250	1.000	0	105	80	120				
cis-1,2-Dichloroethene	1.03	0.0150	1.000	0	103	80	120				
(MEK) 2-Butanone	2.12	0.300	2.500	0	84.8	80	120				
Chloroform	0.993	0.0175	1.000	0	99.3	80	120				
1,1,1-Trichloroethane (TCA)	0.986	0.0200	1.000	0	98.6	80	120				
1,1-Dichloropropene	1.00	0.0200	1.000	0	100	80	120				
Carbon tetrachloride	1.06	0.0250	1.000	0	106	80	120				
1,2-Dichloroethane (EDC)	0.995	0.0200	1.000	0	99.5	80	120				
Benzene	0.990	0.0175	1.000	0	99.0	80	120				
Trichloroethene (TCE)	0.881	0.0150	1.000	0	88.1	80	120				
1,2-Dichloropropane	1.00	0.0250	1.000	0	100	80	120				
Bromodichloromethane	1.10	0.0250	1.000	0	110	80	120				
Dibromomethane	0.980	0.0125	1.000	0	98.0	80	120				
cis-1,3-Dichloropropene	1.00	0.0150	1.000	0	100	80	120				
Toluene	1.01	0.0300	1.000	0	101	80	120				
Trans-1,3-Dichloropropylene	1.03	0.0200	1.000	0	103	80	120				
Methyl Isobutyl Ketone (MIBK)	2.16	0.0600	2.500	0	86.4	80	120				
1,1,2-Trichloroethane	0.943	0.0125	1.000	0	94.3	80	120				
1,3-Dichloropropane	0.954	0.0100	1.000	0	95.4	80	120				
Tetrachloroethene (PCE)	1.04	0.0150	1.000	0	104	80	120				
Dibromochloromethane	1.01	0.0150	1.000	0	101	80	120				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **LCS-39415** SampType: **LCS** Units: **mg/Kg** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **LCSS** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1700313**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2-Dibromoethane (EDB)	0.929	0.0100	1.000	0	92.9	80	120				
2-Hexanone (MBK)	2.14	0.0625	2.500	0	85.6	80	120				
Chlorobenzene	1.00	0.0150	1.000	0	100	80	120				
1,1,1,2-Tetrachloroethane	1.02	0.0250	1.000	0	102	80	120				
Ethylbenzene	1.01	0.0250	1.000	0	101	80	120				
m,p-Xylene	2.26	0.0500	2.000	0	113	80	120				
o-Xylene	1.02	0.0250	1.000	0	102	80	120				
Styrene	0.995	0.0100	1.000	0	99.5	80	120				
Isopropylbenzene	1.02	0.0150	1.000	0	102	80	120				
Bromoform	1.02	0.0150	1.000	0	102	80	120				
1,1,2,2-Tetrachloroethane	0.956	0.200	1.000	0	95.6	80	120				
n-Propylbenzene	1.03	0.0150	1.000	0	103	80	120				
Bromobenzene	1.12	0.0125	1.000	0	112	80	120				
1,3,5-Trimethylbenzene	1.02	0.0150	1.000	0	102	80	120				
2-Chlorotoluene	1.02	0.0165	1.000	0	102	80	120				
4-Chlorotoluene	1.03	0.0165	1.000	0	103	80	120				
tert-Butylbenzene	1.03	0.0150	1.000	0	103	80	120				
1,2,3-Trichloropropane	0.917	0.0300	1.000	0	91.7	80	120				
1,2,4-Trichlorobenzene	1.01	0.0600	1.000	0	101	80	120				
sec-Butylbenzene	1.05	0.150	1.000	0	105	80	120				
4-Isopropyltoluene	1.05	0.200	1.000	0	105	80	120				
1,3-Dichlorobenzene	1.03	0.0200	1.000	0	103	80	120				
1,4-Dichlorobenzene	0.983	0.0150	1.000	0	98.3	80	120				
n-Butylbenzene	1.06	0.0200	1.000	0	106	80	120				
1,2-Dichlorobenzene	1.00	0.0200	1.000	0	100	80	120				
1,2-Dibromo-3-chloropropane	0.956	0.0300	1.000	0	95.6	80	120				
1,2,4-Trimethylbenzene	1.02	0.0150	1.000	0	102	80	120				
Hexachloro-1,3-butadiene	1.07	0.0400	1.000	0	107	80	120				
Naphthalene	0.924	0.100	1.000	0	92.4	80	120				
1,2,3-Trichlorobenzene	1.01	0.0600	1.000	0	101	80	120				
Surr: Dibromofluoromethane	1.25		1.250		99.9	80	120				
Surr: Toluene-d8	1.25		1.250		100	80	120				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **LCS-39415** SampType: **LCS** Units: **mg/Kg** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **LCSS** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1700313**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Surr: 1-Bromo-4-fluorobenzene	1.27		1.250		102	80	120				

Sample ID: **2302057-025BDUP** SampType: **DUP** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BATCH** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698103**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	ND	0.0316						0		30	
Chloromethane	ND	0.105						0		30	
Vinyl chloride	ND	0.0526						0		30	
Bromomethane	ND	0.0526						0		30	
Trichlorofluoromethane (CFC-11)	ND	0.0421						0		30	
Chloroethane	ND	0.158						0		30	
1,1-Dichloroethene	ND	0.210						0		30	
Acetone	ND	0.526						0		30	
Methylene chloride	ND	0.0736						0		30	
trans-1,2-Dichloroethene	ND	0.0210						0		30	
Methyl tert-butyl ether (MTBE)	ND	0.0421						0		30	
1,1-Dichloroethane	ND	0.0526						0		30	
cis-1,2-Dichloroethene	ND	0.0316						0		30	
(MEK) 2-Butanone	ND	0.631						0		30	
Chloroform	ND	0.0368						0		30	
1,1,1-Trichloroethane (TCA)	0.258	0.0421						0.2596	0.668	30	
1,1-Dichloropropene	ND	0.0421						0		30	
Carbon tetrachloride	ND	0.0526						0		30	
1,2-Dichloroethane (EDC)	ND	0.0421						0		30	
Benzene	ND	0.0368						0		30	
Trichloroethene (TCE)	0.434	0.0316						0.4302	0.974	30	
1,2-Dichloropropane	ND	0.0526						0		30	
Bromodichloromethane	ND	0.0526						0		30	
Dibromomethane	ND	0.0263						0		30	
cis-1,3-Dichloropropene	ND	0.0316						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302057-025BDUP** SampType: **DUP** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BATCH** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698103**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Toluene	ND	0.0631						0		30	
Trans-1,3-Dichloropropylene	ND	0.0421						0		30	
Methyl Isobutyl Ketone (MIBK)	ND	0.126						0		30	
1,1,2-Trichloroethane	ND	0.0263						0		30	
1,3-Dichloropropane	ND	0.0210						0		30	
Tetrachloroethene (PCE)	0.0706	0.0316						0.07117	0.731	30	
Dibromochloromethane	ND	0.0316						0		30	
1,2-Dibromoethane (EDB)	ND	0.0210						0		30	
2-Hexanone (MBK)	ND	0.131						0		30	
Chlorobenzene	ND	0.0316						0		30	
1,1,1,2-Tetrachloroethane	ND	0.0526						0		30	
Ethylbenzene	ND	0.0526						0		30	
m,p-Xylene	ND	0.105						0		30	
o-Xylene	ND	0.0526						0		30	
Styrene	ND	0.0210						0		30	
Isopropylbenzene	ND	0.0316						0		30	
Bromoform	ND	0.0316						0		30	
1,1,2,2-Tetrachloroethane	ND	0.421						0		30	
n-Propylbenzene	ND	0.0316						0		30	
Bromobenzene	ND	0.0263						0		30	
1,3,5-Trimethylbenzene	ND	0.0316						0		30	
2-Chlorotoluene	ND	0.0347						0		30	
4-Chlorotoluene	ND	0.0347						0		30	
tert-Butylbenzene	ND	0.0316						0		30	
1,2,3-Trichloropropane	ND	0.0631						0		30	
1,2,4-Trichlorobenzene	ND	0.126						0		30	
sec-Butylbenzene	ND	0.316						0		30	
4-Isopropyltoluene	ND	0.421						0		30	
1,3-Dichlorobenzene	ND	0.0421						0		30	
1,4-Dichlorobenzene	ND	0.0316						0		30	
n-Butylbenzene	ND	0.0421						0		30	
1,2-Dichlorobenzene	ND	0.0421						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302057-025BDUP** SampType: **DUP** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BATCH** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698103**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2-Dibromo-3-chloropropane	ND	0.0631						0		30	
1,2,4-Trimethylbenzene	ND	0.0316						0		30	
Hexachloro-1,3-butadiene	ND	0.0841						0		30	
Naphthalene	ND	0.210						0		30	
1,2,3-Trichlorobenzene	ND	0.126						0		30	
Surr: Dibromofluoromethane	2.68		2.629		102	80	120		0		
Surr: Toluene-d8	2.93		2.629		111	80	120		0		
Surr: 1-Bromo-4-fluorobenzene	2.60		2.629		98.9	80	120		0		

Sample ID: **2302057-034BMS** SampType: **MS** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BATCH** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698063**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	0.713	0.0145	0.9669	0	73.7	13.6	193				
Chloromethane	0.801	0.0483	0.9669	0	82.8	37.6	165				
Vinyl chloride	0.746	0.0242	0.9669	0	77.1	49.9	166				
Bromomethane	0.782	0.0242	0.9669	0	80.9	29.1	184				
Trichlorofluoromethane (CFC-11)	0.977	0.0193	0.9669	0	101	49.8	160				
Chloroethane	0.704	0.0725	0.9669	0	72.8	41.1	171				
1,1-Dichloroethene	1.05	0.0967	0.9669	0	108	64.6	149				
Acetone	2.21	0.242	2.417	0.1477	85.3	54.6	152				
Methylene chloride	1.09	0.0338	0.9669	0	113	66.1	140				
trans-1,2-Dichloroethene	0.821	0.00967	0.9669	0	84.9	73.1	137				
Methyl tert-butyl ether (MTBE)	0.920	0.0193	0.9669	0	95.1	72.4	129				
1,1-Dichloroethane	1.01	0.0242	0.9669	0	104	68.6	139				
cis-1,2-Dichloroethene	0.905	0.0145	0.9669	0	93.6	76.4	134				
(MEK) 2-Butanone	2.16	0.290	2.417	0	89.2	58.2	156				
Chloroform	0.842	0.0169	0.9669	0	87.0	77.9	132				
1,1,1-Trichloroethane (TCA)	0.816	0.0193	0.9669	0.09754	74.3	77.6	139				S
1,1-Dichloropropene	0.814	0.0193	0.9669	0	84.2	78.1	138				
Carbon tetrachloride	0.834	0.0242	0.9669	0	86.2	75.8	140				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302057-034BMS**	SampType: **MS**	Units: **mg/Kg-dry**	Prep Date: **2/13/2023**	RunNo: **81881**	
Client ID: **BATCH**	Batch ID: **39415**		Analysis Date: **2/14/2023**	SeqNo: **1698063**	

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2-Dichloroethane (EDC)	0.899	0.0193	0.9669	0	93.0	74.5	133				
Benzene	0.832	0.0169	0.9669	0.04187	81.7	76.2	134				
Trichloroethene (TCE)	0.853	0.0145	0.9669	0.1035	77.5	75.5	144				
1,2-Dichloropropane	0.975	0.0242	0.9669	0	101	70.9	135				
Bromodichloromethane	1.04	0.0242	0.9669	0	107	72.4	135				
Dibromomethane	1.04	0.0121	0.9669	0	107	75.8	134				
cis-1,3-Dichloropropene	0.936	0.0145	0.9669	0	96.8	67.1	135				
Toluene	0.950	0.0290	0.9669	0.1628	81.4	77.9	135				
Trans-1,3-Dichloropropylene	0.961	0.0193	0.9669	0	99.4	66.8	135				
Methyl Isobutyl Ketone (MIBK)	2.42	0.0580	2.417	0	99.9	55	155				
1,1,2-Trichloroethane	0.996	0.0121	0.9669	0.007287	102	71.7	137				
1,3-Dichloropropane	0.990	0.00967	0.9669	0	102	72.9	135				
Tetrachloroethene (PCE)	0.967	0.0145	0.9669	0.07066	92.7	78.3	138				
Dibromochloromethane	1.02	0.0145	0.9669	0	106	69.4	137				
1,2-Dibromoethane (EDB)	1.00	0.00967	0.9669	0	104	75.2	133				
2-Hexanone (MBK)	2.36	0.0604	2.417	0	97.8	45.8	156				
Chlorobenzene	0.859	0.0145	0.9669	0	88.9	83.2	128				
1,1,1,2-Tetrachloroethane	0.878	0.0242	0.9669	0	90.8	81	131				
Ethylbenzene	0.830	0.0242	0.9669	0.01529	84.3	81.1	138				
m,p-Xylene	1.87	0.0483	1.934	0.08680	92.3	82.2	135				
o-Xylene	0.851	0.0242	0.9669	0.04989	82.8	81.3	136				
Styrene	0.857	0.00967	0.9669	0	88.6	81.9	132				
Isopropylbenzene	0.834	0.0145	0.9669	0.008674	85.4	80.3	142				
Bromoform	0.914	0.0145	0.9669	0	94.5	63.3	143				
1,1,2,2-Tetrachloroethane	0.869	0.193	0.9669	0	89.9	61	136				
n-Propylbenzene	0.821	0.0145	0.9669	0.009091	83.9	77.4	145				
Bromobenzene	0.976	0.0121	0.9669	0	101	80.1	131				
1,3,5-Trimethylbenzene	0.850	0.0145	0.9669	0.01041	86.8	78.6	138				
2-Chlorotoluene	0.850	0.0160	0.9669	0	87.9	79.6	136				
4-Chlorotoluene	0.858	0.0160	0.9669	0	88.8	80.2	133				
tert-Butylbenzene	0.842	0.0145	0.9669	0	87.1	76.8	138				
1,2,3-Trichloropropane	0.839	0.0290	0.9669	0	86.8	68.4	134				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302057-034BMS** SampType: **MS** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BATCH** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698063**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2,4-Trichlorobenzene	0.916	0.0580	0.9669	0	94.7	71.4	134				
sec-Butylbenzene	0.826	0.145	0.9669	0	85.5	74.3	148				
4-Isopropyltoluene	0.835	0.193	0.9669	0.03073	83.2	75.4	142				
1,3-Dichlorobenzene	0.906	0.0193	0.9669	0	93.6	83.8	130				
1,4-Dichlorobenzene	0.910	0.0145	0.9669	0	94.1	83.7	130				
n-Butylbenzene	0.826	0.0193	0.9669	0	85.4	77.1	142				
1,2-Dichlorobenzene	0.916	0.0193	0.9669	0	94.8	84.8	128				
1,2-Dibromo-3-chloropropane	0.865	0.0290	0.9669	0	89.5	61.3	138				
1,2,4-Trimethylbenzene	0.850	0.0145	0.9669	0.02262	85.5	77.1	138				
Hexachloro-1,3-butadiene	0.941	0.0387	0.9669	0	97.3	70.3	148				
Naphthalene	0.915	0.0967	0.9669	0	94.6	58.7	144				
1,2,3-Trichlorobenzene	0.959	0.0580	0.9669	0	99.1	61.4	142				
Surr: Dibromofluoromethane	1.21		1.209		99.8	80	120				
Surr: Toluene-d8	1.35		1.209		112	80	120				
Surr: 1-Bromo-4-fluorobenzene	1.23		1.209		102	80	120				

NOTES:
S - Outlying spike recoveries were associated with this sample.

Sample ID: **2302195-001BDUP** SampType: **DUP** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BT-1-8** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698068**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	ND	0.0139						0		30	Q
Chloromethane	ND	0.0464						0		30	
Vinyl chloride	ND	0.0232						0		30	
Bromomethane	ND	0.0232						0		30	
Trichlorofluoromethane (CFC-11)	ND	0.0185						0		30	
Chloroethane	ND	0.0695						0		30	
1,1-Dichloroethene	ND	0.0927						0		30	
Acetone	ND	0.232						0		30	
Methylene chloride	ND	0.0324						0		30	
trans-1,2-Dichloroethene	ND	0.00927						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302195-001BDUP** SampType: **DUP** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BT-1-8** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698068**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Methyl tert-butyl ether (MTBE)	ND	0.0185						0		30	
1,1-Dichloroethane	ND	0.0232						0		30	
cis-1,2-Dichloroethene	ND	0.0139						0		30	
(MEK) 2-Butanone	ND	0.278						0		30	
Chloroform	ND	0.0162						0		30	
1,1,1-Trichloroethane (TCA)	ND	0.0185						0		30	
1,1-Dichloropropene	ND	0.0185						0		30	
Carbon tetrachloride	ND	0.0232						0		30	
1,2-Dichloroethane (EDC)	ND	0.0185						0		30	
Benzene	ND	0.0162						0		30	
Trichloroethene (TCE)	ND	0.0139						0		30	
1,2-Dichloropropane	ND	0.0232						0		30	
Bromodichloromethane	ND	0.0232						0		30	
Dibromomethane	ND	0.0116						0		30	
cis-1,3-Dichloropropene	ND	0.0139						0		30	
Toluene	ND	0.0278						0		30	
Trans-1,3-Dichloropropylene	ND	0.0185						0		30	
Methyl Isobutyl Ketone (MIBK)	ND	0.0556						0		30	
1,1,2-Trichloroethane	ND	0.0116						0		30	
1,3-Dichloropropane	ND	0.00927						0		30	
Tetrachloroethene (PCE)	ND	0.0139						0		30	
Dibromochloromethane	ND	0.0139						0		30	
1,2-Dibromoethane (EDB)	ND	0.00927						0		30	
2-Hexanone (MBK)	ND	0.0579						0		30	
Chlorobenzene	ND	0.0139						0		30	
1,1,1,2-Tetrachloroethane	ND	0.0232						0		30	
Ethylbenzene	ND	0.0232						0		30	
m,p-Xylene	ND	0.0464						0		30	
o-Xylene	ND	0.0232						0		30	
Styrene	ND	0.00927						0		30	
Isopropylbenzene	ND	0.0139						0		30	
Bromoform	ND	0.0139						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302195-001BDUP** SampType: **DUP** Units: **mg/Kg-dry** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **BT-1-8** Batch ID: **39415** Analysis Date: **2/14/2023** SeqNo: **1698068**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,1,2,2-Tetrachloroethane	ND	0.185						0		30	
n-Propylbenzene	ND	0.0139						0		30	
Bromobenzene	ND	0.0116						0		30	
1,3,5-Trimethylbenzene	ND	0.0139						0		30	
2-Chlorotoluene	ND	0.0153						0		30	
4-Chlorotoluene	ND	0.0153						0		30	
tert-Butylbenzene	ND	0.0139						0		30	
1,2,3-Trichloropropane	ND	0.0278						0		30	
1,2,4-Trichlorobenzene	ND	0.0556						0		30	
sec-Butylbenzene	ND	0.139						0		30	
4-Isopropyltoluene	ND	0.185						0		30	
1,3-Dichlorobenzene	ND	0.0185						0		30	
1,4-Dichlorobenzene	ND	0.0139						0		30	
n-Butylbenzene	ND	0.0185						0		30	
1,2-Dichlorobenzene	ND	0.0185						0		30	
1,2-Dibromo-3-chloropropane	ND	0.0278						0		30	
1,2,4-Trimethylbenzene	ND	0.0139						0		30	
Hexachloro-1,3-butadiene	ND	0.0371						0		30	
Naphthalene	ND	0.0927						0		30	
1,2,3-Trichlorobenzene	ND	0.0556						0		30	
Surr: Dibromofluoromethane	1.16		1.159		99.9	80	120		0		
Surr: Toluene-d8	1.27		1.159		110	80	120		0		
Surr: 1-Bromo-4-fluorobenzene	1.13		1.159		97.9	80	120		0		

NOTES:
Q - Associated calibration verification is below acceptance criteria. Result may be low-biased

Sample ID: **MB-39415** SampType: **MBLK** Units: **mg/Kg** Prep Date: **2/13/2023** RunNo: **81881**
Client ID: **MBLKS** Batch ID: **39415** Analysis Date: **2/15/2023** SeqNo: **1698099**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	ND	0.0150									Q
Chloromethane	ND	0.0500									

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **MB-39415**	SampType: **MBLK**	Units: **mg/Kg**	Prep Date: **2/13/2023**	RunNo: **81881**	
Client ID: **MBLKS**	Batch ID: **39415**		Analysis Date: **2/15/2023**	SeqNo: **1698099**	

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Vinyl chloride	ND	0.0250									
Bromomethane	ND	0.0250									
Trichlorofluoromethane (CFC-11)	ND	0.0200									
Chloroethane	ND	0.0750									Q
1,1-Dichloroethene	ND	0.100									
Acetone	ND	0.250									
Methylene chloride	ND	0.0350									
trans-1,2-Dichloroethene	ND	0.0100									
Methyl tert-butyl ether (MTBE)	ND	0.0200									
1,1-Dichloroethane	ND	0.0250									
cis-1,2-Dichloroethene	ND	0.0150									
(MEK) 2-Butanone	ND	0.300									
Chloroform	ND	0.0175									
1,1,1-Trichloroethane (TCA)	ND	0.0200									
1,1-Dichloropropene	ND	0.0200									
Carbon tetrachloride	ND	0.0250									
1,2-Dichloroethane (EDC)	ND	0.0200									
Benzene	ND	0.0175									
Trichloroethene (TCE)	ND	0.0150									
1,2-Dichloropropane	ND	0.0250									
Bromodichloromethane	ND	0.0250									
Dibromomethane	ND	0.0125									
cis-1,3-Dichloropropene	ND	0.0150									
Toluene	ND	0.0300									
Trans-1,3-Dichloropropylene	ND	0.0200									
Methyl Isobutyl Ketone (MIBK)	ND	0.0600									
1,1,2-Trichloroethane	ND	0.0125									
1,3-Dichloropropane	ND	0.0100									
Tetrachloroethene (PCE)	ND	0.0150									
Dibromochloromethane	ND	0.0150									
1,2-Dibromoethane (EDB)	ND	0.0100									
2-Hexanone (MBK)	ND	0.0625									

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **MB-39415**	SampType: **MBLK**	Units: **mg/Kg**	Prep Date: **2/13/2023**	RunNo: **81881**	
Client ID: **MBLKS**	Batch ID: **39415**		Analysis Date: **2/15/2023**	SeqNo: **1698099**	

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Chlorobenzene	ND	0.0150									
1,1,1,2-Tetrachloroethane	ND	0.0250									
Ethylbenzene	ND	0.0250									
m,p-Xylene	ND	0.0500									
o-Xylene	ND	0.0250									
Styrene	ND	0.0100									
Isopropylbenzene	ND	0.0150									
Bromoform	ND	0.0150									
1,1,2,2-Tetrachloroethane	ND	0.200									
n-Propylbenzene	ND	0.0150									
Bromobenzene	ND	0.0125									
1,3,5-Trimethylbenzene	ND	0.0150									
2-Chlorotoluene	ND	0.0165									
4-Chlorotoluene	ND	0.0165									
tert-Butylbenzene	ND	0.0150									
1,2,3-Trichloropropane	ND	0.0300									
1,2,4-Trichlorobenzene	ND	0.0600									
sec-Butylbenzene	ND	0.150									
4-Isopropyltoluene	ND	0.200									
1,3-Dichlorobenzene	ND	0.0200									
1,4-Dichlorobenzene	ND	0.0150									
n-Butylbenzene	ND	0.0200									
1,2-Dichlorobenzene	ND	0.0200									
1,2-Dibromo-3-chloropropane	ND	0.0300									
1,2,4-Trimethylbenzene	ND	0.0150									
Hexachloro-1,3-butadiene	ND	0.0400									
Naphthalene	ND	0.100									
1,2,3-Trichlorobenzene	ND	0.0600									
Surr: Dibromofluoromethane	1.24		1.250		99.2	80	120				
Surr: Toluene-d8	1.36		1.250		108	80	120				
Surr: 1-Bromo-4-fluorobenzene	1.23		1.250		98.6	80	120				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **MB-39415**	SampType: **MBLK**		Units: **mg/Kg**	Prep Date: **2/13/2023**		RunNo: **81881**
Client ID: **MBLKS**	Batch ID: **39415**			Analysis Date: **2/15/2023**		SeqNo: **1698099**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual

NOTES:
Q - Associated calibration verification is below acceptance criteria. Result may be low-biased

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **LCS-39414** SampType: **LCS** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **LCSW** Batch ID: **39414** Analysis Date: **2/13/2023** SeqNo: **1696900**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	19.0	0.500	20.00	0	95.2	80	120				
Chloromethane	19.6	0.750	20.00	0	97.9	80	120				
Vinyl chloride	21.0	0.200	20.00	0	105	80	120				
Bromomethane	22.7	3.00	20.00	0	113	80	120				
Trichlorofluoromethane (CFC-11)	21.4	0.300	20.00	0	107	80	120				
Chloroethane	22.1	1.00	20.00	0	110	80	120				
1,1-Dichloroethene	22.3	0.500	20.00	0	111	80	120				
Acetone	60.0	5.00	50.00	0	120	80	120				
Methylene chloride	21.0	0.750	20.00	0	105	80	120				
trans-1,2-Dichloroethene	21.5	0.350	20.00	0	107	80	120				
Methyl tert-butyl ether (MTBE)	22.5	0.350	20.00	0	112	80	120				
1,1-Dichloroethane	20.6	0.500	20.00	0	103	80	120				
cis-1,2-Dichloroethene	21.4	0.500	20.00	0	107	80	120				
2-Butanone (MEK)	52.3	1.50	50.00	0	105	80	120				
Chloroform	21.5	0.500	20.00	0	107	80	120				
1,1,1-Trichloroethane (TCA)	20.7	0.300	20.00	0	104	80	120				
1,1-Dichloropropene	22.1	0.500	20.00	0	110	80	120				
Carbon tetrachloride	21.6	0.300	20.00	0	108	80	120				
1,2-Dichloroethane (EDC)	20.5	0.500	20.00	0	103	80	120				
Benzene	21.1	0.440	20.00	0	105	80	120				
Trichloroethene (TCE)	22.0	0.400	20.00	0	110	80	120				
1,2-Dichloropropane	20.9	0.300	20.00	0	105	80	120				
Bromodichloromethane	20.9	0.250	20.00	0	105	80	120				
Dibromomethane	21.0	0.250	20.00	0	105	80	120				
cis-1,3-Dichloropropene	21.1	0.350	20.00	0	105	80	120				
Toluene	21.6	1.00	20.00	0	108	80	120				
trans-1,3-Dichloropropylene	20.7	0.500	20.00	0	104	80	120				
Methyl Isobutyl Ketone (MIBK)	47.7	1.00	50.00	0	95.4	80	120				
1,1,2-Trichloroethane	21.2	0.250	20.00	0	106	80	120				
1,3-Dichloropropane	21.1	0.300	20.00	0	106	80	120				
Tetrachloroethene (PCE)	22.6	0.350	20.00	0	113	80	120				
Dibromochloromethane	20.6	0.300	20.00	0	103	80	120				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **LCS-39414** SampType: **LCS** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **LCSW** Batch ID: **39414** Analysis Date: **2/13/2023** SeqNo: **1696900**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2-Dibromoethane (EDB)	21.0	0.200	20.00	0	105	80	120				
2-Hexanone (MBK)	48.3	1.25	50.00	0	96.7	80	120				
Chlorobenzene	20.9	0.500	20.00	0	104	80	120				
1,1,1,2-Tetrachloroethane	21.6	0.300	20.00	0	108	80	120				
Ethylbenzene	21.2	0.400	20.00	0	106	80	120				
m,p-Xylene	43.1	1.00	40.00	0	108	80	120				
o-Xylene	21.0	0.500	20.00	0	105	80	120				
Styrene	21.5	0.500	20.00	0	107	80	120				
Isopropylbenzene	21.3	0.500	20.00	0	106	80	120				
Bromoform	20.0	0.300	20.00	0	99.8	80	120				
1,1,2,2-Tetrachloroethane	22.1	0.200	20.00	0	110	80	120				
n-Propylbenzene	21.6	0.500	20.00	0	108	80	120				
Bromobenzene	20.7	0.500	20.00	0	103	80	120				
1,3,5-Trimethylbenzene	21.7	0.500	20.00	0	108	80	120				
2-Chlorotoluene	21.3	0.500	20.00	0	106	80	120				
4-Chlorotoluene	21.3	0.500	20.00	0	106	80	120				
tert-Butylbenzene	21.4	0.500	20.00	0	107	80	120				
1,2,3-Trichloropropane	20.1	0.400	20.00	0	100	80	120				
1,2,4-Trichlorobenzene	20.3	0.750	20.00	0	101	80	120				
sec-Butylbenzene	21.5	0.500	20.00	0	108	80	120				
4-Isopropyltoluene	21.5	0.500	20.00	0	107	80	120				
1,3-Dichlorobenzene	21.3	0.500	20.00	0	106	80	120				
1,4-Dichlorobenzene	21.4	0.500	20.00	0	107	80	120				
n-Butylbenzene	21.6	0.500	20.00	0	108	80	120				
1,2-Dichlorobenzene	20.6	0.500	20.00	0	103	80	120				
1,2-Dibromo-3-chloropropane	17.8	1.00	20.00	0	89.2	80	120				
1,2,4-Trimethylbenzene	21.4	0.500	20.00	0	107	80	120				
Hexachloro-1,3-butadiene	21.4	0.500	20.00	0	107	80	120				
Naphthalene	19.2	1.25	20.00	0	96.0	80	120				
1,2,3-Trichlorobenzene	19.7	0.700	20.00	0	98.7	80	120				
Surr: Dibromofluoromethane	25.2		25.00		101	80	120				
Surr: Toluene-d8	24.9		25.00		99.5	80	120				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **LCS-39414**	SampType: **LCS**			Units: **µg/L**		Prep Date: **2/13/2023**		RunNo: **81835**	
Client ID: **LCSW**	Batch ID: **39414**					Analysis Date: **2/13/2023**		SeqNo: **1696900**	
Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Surr: 1-Bromo-4-fluorobenzene	24.2		25.00		96.9	80	120				

Sample ID: **MB-39414**	SampType: **MBLK**			Units: **µg/L**		Prep Date: **2/13/2023**		RunNo: **81835**	
Client ID: **MBLKW**	Batch ID: **39414**					Analysis Date: **2/13/2023**		SeqNo: **1696899**	
Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	ND	0.500									
Chloromethane	ND	0.750									
Vinyl chloride	ND	0.200									
Bromomethane	ND	3.00									
Trichlorofluoromethane (CFC-11)	ND	0.300									
Chloroethane	ND	1.00									
1,1-Dichloroethene	ND	0.500									
Acetone	ND	5.00									
Methylene chloride	ND	0.750									
trans-1,2-Dichloroethene	ND	0.350									
Methyl tert-butyl ether (MTBE)	ND	0.350									
1,1-Dichloroethane	ND	0.500									
cis-1,2-Dichloroethene	ND	0.500									
2-Butanone (MEK)	ND	1.50									
Chloroform	ND	0.500									
1,1,1-Trichloroethane (TCA)	ND	0.300									
1,1-Dichloropropene	ND	0.500									
Carbon tetrachloride	ND	0.300									
1,2-Dichloroethane (EDC)	ND	0.500									
Benzene	ND	0.440									
Trichloroethene (TCE)	ND	0.400									
1,2-Dichloropropane	ND	0.300									
Bromodichloromethane	ND	0.250									
Dibromomethane	ND	0.250									
cis-1,3-Dichloropropene	ND	0.350									

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **MB-39414**	SampType: **MBLK**	Units: **µg/L**	Prep Date: **2/13/2023**	RunNo: **81835**	
Client ID: **MBLKW**	Batch ID: **39414**		Analysis Date: **2/13/2023**	SeqNo: **1696899**	

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Toluene	ND	1.00									
trans-1,3-Dichloropropylene	ND	0.500									
Methyl Isobutyl Ketone (MIBK)	ND	1.00									
1,1,2-Trichloroethane	ND	0.250									
1,3-Dichloropropane	ND	0.300									
Tetrachloroethene (PCE)	ND	0.350									
Dibromochloromethane	ND	0.300									
1,2-Dibromoethane (EDB)	ND	0.200									
2-Hexanone (MBK)	ND	1.25									
Chlorobenzene	ND	0.500									
1,1,1,2-Tetrachloroethane	ND	0.300									
Ethylbenzene	ND	0.400									
m,p-Xylene	ND	1.00									
o-Xylene	ND	0.500									
Styrene	ND	0.500									
Isopropylbenzene	ND	0.500									
Bromoform	ND	0.300									
1,1,2,2-Tetrachloroethane	ND	0.200									
n-Propylbenzene	ND	0.500									
Bromobenzene	ND	0.500									
1,3,5-Trimethylbenzene	ND	0.500									
2-Chlorotoluene	ND	0.500									
4-Chlorotoluene	ND	0.500									
tert-Butylbenzene	ND	0.500									
1,2,3-Trichloropropane	ND	0.400									
1,2,4-Trichlorobenzene	ND	0.750									
sec-Butylbenzene	ND	0.500									
4-Isopropyltoluene	ND	0.500									
1,3-Dichlorobenzene	ND	0.500									
1,4-Dichlorobenzene	ND	0.500									
n-Butylbenzene	ND	0.500									
1,2-Dichlorobenzene	ND	0.500									

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **MB-39414** SampType: **MBLK** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **MBLKW** Batch ID: **39414** Analysis Date: **2/13/2023** SeqNo: **1696899**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2-Dibromo-3-chloropropane	ND	1.00									
1,2,4-Trimethylbenzene	ND	0.500									
Hexachloro-1,3-butadiene	ND	0.500									
Naphthalene	ND	1.25									
1,2,3-Trichlorobenzene	ND	0.700									
Surr: Dibromofluoromethane	25.8		25.00		103	80	120				
Surr: Toluene-d8	25.5		25.00		102	80	120				
Surr: 1-Bromo-4-fluorobenzene	24.8		25.00		99.2	80	120				

Sample ID: **2302186-001ADUP** SampType: **DUP** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BATCH** Batch ID: **39414** Analysis Date: **2/13/2023** SeqNo: **1696875**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	ND	0.500						0		30	
Chloromethane	ND	0.750						0		30	
Vinyl chloride	ND	0.200						0		30	
Bromomethane	ND	3.00						0		30	
Trichlorofluoromethane (CFC-11)	ND	0.300						0		30	
Chloroethane	ND	1.00						0		30	
1,1-Dichloroethene	ND	0.500						0		30	
Acetone	ND	5.00						0		30	
Methylene chloride	ND	0.750						0		30	
trans-1,2-Dichloroethene	ND	0.350						0		30	
Methyl tert-butyl ether (MTBE)	ND	0.350						0		30	
1,1-Dichloroethane	ND	0.500						0		30	
cis-1,2-Dichloroethene	ND	0.500						0		30	
2-Butanone (MEK)	ND	1.50						0		30	
Chloroform	ND	0.500						0		30	
1,1,1-Trichloroethane (TCA)	ND	0.300						0		30	
1,1-Dichloropropene	ND	0.500						0		30	
Carbon tetrachloride	ND	0.300						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302186-001ADUP** SampType: **DUP** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BATCH** Batch ID: **39414** Analysis Date: **2/13/2023** SeqNo: **1696875**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2-Dichloroethane (EDC)	ND	0.500						0		30	
Benzene	ND	0.440						0		30	
Trichloroethene (TCE)	ND	0.400						0		30	
1,2-Dichloropropane	ND	0.300						0		30	
Bromodichloromethane	ND	0.250						0		30	
Dibromomethane	ND	0.250						0		30	
cis-1,3-Dichloropropene	ND	0.350						0		30	
Toluene	ND	1.00						0		30	
trans-1,3-Dichloropropylene	ND	0.500						0		30	
Methyl Isobutyl Ketone (MIBK)	ND	1.00						0		30	
1,1,2-Trichloroethane	ND	0.250						0		30	
1,3-Dichloropropane	ND	0.300						0		30	
Tetrachloroethene (PCE)	ND	0.350						0		30	
Dibromochloromethane	ND	0.300						0		30	
1,2-Dibromoethane (EDB)	ND	0.200						0		30	
2-Hexanone (MBK)	ND	1.25						0		30	
Chlorobenzene	ND	0.500						0		30	
1,1,1,2-Tetrachloroethane	ND	0.300						0		30	
Ethylbenzene	ND	0.400						0		30	
m,p-Xylene	ND	1.00						0		30	
o-Xylene	ND	0.500						0		30	
Styrene	ND	0.500						0		30	
Isopropylbenzene	ND	0.500						0		30	
Bromoform	ND	0.300						0		30	
1,1,2,2-Tetrachloroethane	ND	0.200						0		30	
n-Propylbenzene	ND	0.500						0		30	
Bromobenzene	ND	0.500						0		30	
1,3,5-Trimethylbenzene	ND	0.500						0		30	
2-Chlorotoluene	ND	0.500						0		30	
4-Chlorotoluene	ND	0.500						0		30	
tert-Butylbenzene	ND	0.500						0		30	
1,2,3-Trichloropropane	ND	0.400						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302186-001ADUP** SampType: **DUP** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BATCH** Batch ID: **39414** Analysis Date: **2/13/2023** SeqNo: **1696875**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,2,4-Trichlorobenzene	ND	0.750						0		30	
sec-Butylbenzene	ND	0.500						0		30	
4-Isopropyltoluene	ND	0.500						0		30	
1,3-Dichlorobenzene	ND	0.500						0		30	
1,4-Dichlorobenzene	ND	0.500						0		30	
n-Butylbenzene	ND	0.500						0		30	
1,2-Dichlorobenzene	ND	0.500						0		30	
1,2-Dibromo-3-chloropropane	ND	1.00						0		30	
1,2,4-Trimethylbenzene	ND	0.500						0		30	
Hexachloro-1,3-butadiene	ND	0.500						0		30	
Naphthalene	ND	1.25						0		30	
1,2,3-Trichlorobenzene	ND	0.700						0		30	
Surr: Dibromofluoromethane	25.8		25.00		103	80	120		0		
Surr: Toluene-d8	25.9		25.00		104	80	120		0		
Surr: 1-Bromo-4-fluorobenzene	24.9		25.00		99.4	80	120		0		

Sample ID: **2302215-015ADUP** SampType: **DUP** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BATCH** Batch ID: **39414** Analysis Date: **2/14/2023** SeqNo: **1696894**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	ND	0.500						0		30	
Chloromethane	ND	0.750						0		30	
Vinyl chloride	0.411	0.200						0.3328	21.0	30	
Bromomethane	ND	3.00						0		30	
Trichlorofluoromethane (CFC-11)	ND	0.300						0		30	
Chloroethane	ND	1.00						0		30	
1,1-Dichloroethene	ND	0.500						0		30	
Acetone	ND	5.00						0		30	
Methylene chloride	ND	0.750						0		30	
trans-1,2-Dichloroethene	ND	0.350						0		30	
Methyl tert-butyl ether (MTBE)	ND	0.350						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302215-015ADUP** SampType: **DUP** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BATCH** Batch ID: **39414** Analysis Date: **2/14/2023** SeqNo: **1696894**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
1,1-Dichloroethane	ND	0.500						0		30	
cis-1,2-Dichloroethene	ND	0.500						0		30	
2-Butanone (MEK)	ND	1.50						0		30	
Chloroform	ND	0.500						0		30	
1,1,1-Trichloroethane (TCA)	ND	0.300						0		30	
1,1-Dichloropropene	ND	0.500						0		30	
Carbon tetrachloride	ND	0.300						0		30	
1,2-Dichloroethane (EDC)	ND	0.500						0		30	
Benzene	ND	0.440						0		30	
Trichloroethene (TCE)	ND	0.400						0		30	
1,2-Dichloropropane	ND	0.300						0		30	
Bromodichloromethane	ND	0.250						0		30	
Dibromomethane	ND	0.250						0		30	
cis-1,3-Dichloropropene	ND	0.350						0		30	
Toluene	ND	1.00						0		30	
trans-1,3-Dichloropropylene	ND	0.500						0		30	
Methyl Isobutyl Ketone (MIBK)	ND	1.00						0		30	
1,1,2-Trichloroethane	ND	0.250						0		30	
1,3-Dichloropropane	ND	0.300						0		30	
Tetrachloroethene (PCE)	ND	0.350						0		30	
Dibromochloromethane	ND	0.300						0		30	
1,2-Dibromoethane (EDB)	ND	0.200						0		30	
2-Hexanone (MBK)	ND	1.25						0		30	
Chlorobenzene	ND	0.500						0		30	
1,1,1,2-Tetrachloroethane	ND	0.300						0		30	
Ethylbenzene	ND	0.400						0		30	
m,p-Xylene	ND	1.00						0		30	
o-Xylene	ND	0.500						0		30	
Styrene	ND	0.500						0		30	
Isopropylbenzene	ND	0.500						0		30	
Bromoform	ND	0.300						0		30	
1,1,2,2-Tetrachloroethane	ND	0.200						0		30	

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302215-015ADUP** SampType: **DUP** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BATCH** Batch ID: **39414** Analysis Date: **2/14/2023** SeqNo: **1696894**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
n-Propylbenzene	ND	0.500						0		30	
Bromobenzene	ND	0.500						0		30	
1,3,5-Trimethylbenzene	ND	0.500						0		30	
2-Chlorotoluene	ND	0.500						0		30	
4-Chlorotoluene	ND	0.500						0		30	
tert-Butylbenzene	ND	0.500						0		30	
1,2,3-Trichloropropane	ND	0.400						0		30	
1,2,4-Trichlorobenzene	ND	0.750						0		30	
sec-Butylbenzene	ND	0.500						0		30	
4-Isopropyltoluene	ND	0.500						0		30	
1,3-Dichlorobenzene	ND	0.500						0		30	
1,4-Dichlorobenzene	ND	0.500						0		30	
n-Butylbenzene	ND	0.500						0		30	
1,2-Dichlorobenzene	ND	0.500						0		30	
1,2-Dibromo-3-chloropropane	ND	1.00						0		30	
1,2,4-Trimethylbenzene	ND	0.500						0		30	
Hexachloro-1,3-butadiene	ND	0.500						0		30	
Naphthalene	ND	1.25						0		30	
1,2,3-Trichlorobenzene	ND	0.700						0		30	
Surr: Dibromofluoromethane	25.3		25.00		101	80	120		0		
Surr: Toluene-d8	25.7		25.00		103	80	120		0		
Surr: 1-Bromo-4-fluorobenzene	25.0		25.00		100	80	120		0		

Sample ID: **2302195-004AMS** SampType: **MS** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BT-1-GW** Batch ID: **39414** Analysis Date: **2/14/2023** SeqNo: **1696877**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Dichlorodifluoromethane (CFC-12)	15.2	0.500	20.00	0	76.1	1.35	172				
Chloromethane	24.0	0.750	20.00	0.4492	118	27.2	164				
Vinyl chloride	18.1	0.200	20.00	0	90.7	52.3	147				
Bromomethane	18.2	3.00	20.00	0	90.8	24.2	186				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302195-004AMS** SampType: **MS** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BT-1-GW** Batch ID: **39414** Analysis Date: **2/14/2023** SeqNo: **1696877**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Trichlorofluoromethane (CFC-11)	18.0	0.300	20.00	0	90.0	71.2	137				
Chloroethane	18.9	1.00	20.00	0	94.3	62.9	141				
1,1-Dichloroethene	18.8	0.500	20.00	0	93.9	68	152				
Acetone	59.4	5.00	50.00	0	119	56.1	148				
Methylene chloride	18.9	0.750	20.00	0	94.7	73.7	132				
trans-1,2-Dichloroethene	19.6	0.350	20.00	0	98.1	79.1	131				
Methyl tert-butyl ether (MTBE)	22.2	0.350	20.00	0	111	60.2	140				
1,1-Dichloroethane	17.7	0.500	20.00	0	88.7	64.8	148				
cis-1,2-Dichloroethene	18.8	0.500	20.00	0	94.1	78.3	131				
2-Butanone (MEK)	48.3	1.50	50.00	1.078	94.5	53.6	145				
Chloroform	18.0	0.500	20.00	0	90.1	78.9	131				
1,1,1-Trichloroethane (TCA)	18.0	0.300	20.00	0	90.1	76.6	143				
1,1-Dichloropropene	17.8	0.500	20.00	0	88.8	73.9	146				
Carbon tetrachloride	18.1	0.300	20.00	0	90.3	79.5	133				
1,2-Dichloroethane (EDC)	18.4	0.500	20.00	0	91.8	67.8	140				
Benzene	17.7	0.440	20.00	0	88.7	78.5	133				
Trichloroethene (TCE)	18.0	0.400	20.00	0	90.1	75	133				
1,2-Dichloropropane	18.3	0.300	20.00	0	91.7	71.4	139				
Bromodichloromethane	18.5	0.250	20.00	0	92.4	76.1	130				
Dibromomethane	19.0	0.250	20.00	0	95.1	75.5	130				
cis-1,3-Dichloropropene	17.9	0.350	20.00	0	89.5	68.4	128				
Toluene	23.2	1.00	20.00	0	116	77	133				
trans-1,3-Dichloropropylene	18.2	0.500	20.00	0	91.1	63.8	132				
Methyl Isobutyl Ketone (MIBK)	48.8	1.00	50.00	0	97.6	55.6	145				
1,1,2-Trichloroethane	19.6	0.250	20.00	0	98.1	70.1	138				
1,3-Dichloropropane	19.9	0.300	20.00	0	99.3	67.7	139				
Tetrachloroethene (PCE)	23.0	0.350	20.00	0	115	78	131				
Dibromochloromethane	19.3	0.300	20.00	0	96.5	72.6	129				
1,2-Dibromoethane (EDB)	20.0	0.200	20.00	0	100	67.7	137				
2-Hexanone (MBK)	49.0	1.25	50.00	0	98.0	48.5	148				
Chlorobenzene	18.6	0.500	20.00	0	92.8	80.9	124				
1,1,1,2-Tetrachloroethane	18.5	0.300	20.00	0	92.7	75	133				

Work Order: 2302195
CLIENT: AECOM
Project: Woodinville SR202 Trestle Widening

QC SUMMARY REPORT

Volatile Organic Compounds by EPA Method 8260D

Sample ID: **2302195-004AMS** SampType: **MS** Units: **µg/L** Prep Date: **2/13/2023** RunNo: **81835**
Client ID: **BT-1-GW** Batch ID: **39414** Analysis Date: **2/14/2023** SeqNo: **1696877**

Analyte	Result	RL	SPK value	SPK Ref Val	%REC	LowLimit	HighLimit	RPD Ref Val	%RPD	RPDLimit	Qual
Ethylbenzene	18.6	0.400	20.00	0	92.8	77.9	133				
m,p-Xylene	37.0	1.00	40.00	0	92.6	74.8	133				
o-Xylene	19.0	0.500	20.00	0	94.9	81.2	126				
Styrene	18.6	0.500	20.00	0	93.0	75.7	126				
Isopropylbenzene	18.9	0.500	20.00	0	94.6	79.1	132				
Bromoform	19.8	0.300	20.00	0	99.1	68.3	132				
1,1,2,2-Tetrachloroethane	23.1	0.200	20.00	0	116	62.8	148				
n-Propylbenzene	18.2	0.500	20.00	0	90.8	77.2	137				
Bromobenzene	19.0	0.500	20.00	0	95.0	79.9	124				
1,3,5-Trimethylbenzene	18.5	0.500	20.00	0	92.4	73.9	142				
2-Chlorotoluene	18.3	0.500	20.00	0	91.5	73.5	140				
4-Chlorotoluene	18.6	0.500	20.00	0	93.2	73.5	140				
tert-Butylbenzene	18.3	0.500	20.00	0	91.6	79.5	131				
1,2,3-Trichloropropane	19.0	0.400	20.00	0	94.9	63.1	139				
1,2,4-Trichlorobenzene	18.3	0.750	20.00	0	91.6	60.4	135				
sec-Butylbenzene	18.6	0.500	20.00	0	92.8	77.9	136				
4-Isopropyltoluene	18.7	0.500	20.00	0	93.7	69.9	147				
1,3-Dichlorobenzene	18.4	0.500	20.00	0	91.8	79.3	131				
1,4-Dichlorobenzene	18.8	0.500	20.00	0	93.9	79.1	131				
n-Butylbenzene	17.0	0.500	20.00	0	85.2	76	137				
1,2-Dichlorobenzene	18.6	0.500	20.00	0	92.8	79.3	131				
1,2-Dibromo-3-chloropropane	19.0	1.00	20.00	0	94.9	47.7	153				
1,2,4-Trimethylbenzene	18.6	0.500	20.00	0	93.0	74.3	142				
Hexachloro-1,3-butadiene	17.6	0.500	20.00	0	87.8	68.5	136				
Naphthalene	18.5	1.25	20.00	0	92.5	51.6	149				
1,2,3-Trichlorobenzene	18.6	0.700	20.00	0	92.8	56.6	142				
Surr: Dibromofluoromethane	25.0		25.00		100	80	120				
Surr: Toluene-d8	24.7		25.00		98.8	80	120				
Surr: 1-Bromo-4-fluorobenzene	25.4		25.00		102	80	120				

Sample Log-In Check List

Client Name:	**AECOM**	Work Order Number:	**2302195**
Logged by:	**Clare Griggs**	Date Received:	**2/9/2023 4:44:00 PM**

Chain of Custody

1.	Is Chain of Custody complete?	Yes ☑	No ☐	Not Present ☐
2.	How was the sample delivered?	Client		

Log In

3.	Coolers are present?	Yes ☑	No ☐	NA ☐
4.	Shipping container/cooler in good condition?	Yes ☑	No ☐	
5.	Custody Seals present on shipping container/cooler? (Refer to comments for Custody Seals not intact)	Yes ☑	No ☐	Not Present ☐
6.	Was an attempt made to cool the samples?	Yes ☑	No ☐	NA ☐
7.	Were all items received at a temperature of >2°C to 6°C *	Yes ☑	No ☐	NA ☐
8.	Sample(s) in proper container(s)?	Yes ☑	No ☐	
9.	Sufficient sample volume for indicated test(s)?	Yes ☑	No ☐	
10.	Are samples properly preserved?	Yes ☑	No ☐	
11.	Was preservative added to bottles?	Yes ☐	No ☑	NA ☐
12.	Is there headspace in the VOA vials?	Yes ☐	No ☑	NA ☐
13.	Did all samples containers arrive in good condition(unbroken)?	Yes ☑	No ☐	
14.	Does paperwork match bottle labels?	Yes ☑	No ☐	
15.	Are matrices correctly identified on Chain of Custody?	Yes ☑	No ☐	
16.	Is it clear what analyses were requested?	Yes ☑	No ☐	
17.	Were all holding times able to be met?	Yes ☑	No ☐	

Special Handling (if applicable)

18.	Was client notified of all discrepancies with this order?	Yes ☐	No ☐	NA ☑

Person Notified: _____ Date: _____

By Whom: _____ Via: ☐ eMail ☐ Phone ☐ Fax ☐ In Person

Regarding: _____

Client Instructions: _____

19. Additional remarks:

Item Information

Item #	Temp ºC
Sample	1.1

* Note: DoD/ELAP and TNI require items to be received at 4°C +/- 2°C

Fremont Analytical
3600 Fremont Ave N.
Seattle, WA 98103
Tel: 206-352-3790
Fax: 206-352-7178

Chain of Custody Record & Laboratory Services Agreement

Date: 2/9/2023 Page: 1 of: 2 Laboratory Project No (internal): 2302195

Project Name: Woodinville SR202 Trestle Widening

Special Remarks: Samples from HSA with SPT sampling. GW samples collected with temp. wells, peristaltic pump, disposable tubing

Client: AECOM Technical Services, Inc.

Project No: 60683438 Task 6.5.6

Address: 1111 3rd Ave. Suite 1600

Collected by: Michael Black (AECOM)

City, State, Zip: Seattle, WA, 98101

Location: Woodinville, WA

Telephone: 206-438-2197

Report To (PM): Markus Walbaum

Sample Disposal: ☐ Return to client ☒ Disposal by lab (after 30 days)

Fax:

PM Email: markus.walbaum@aecom.com

	Sample Name	Sample Date	Sample Time	Sample Type (Matrix)*	# of Cont.	VOCs (EPA 8260 / 624)	BTEX	Gasoline Range Organics (GX)	Hydrocarbon Identification (HCID)	Diesel/Heavy Oil Range Organics (DX)	SVOCs (EPA 8270 / 625)	PAHs (EPA 8270 - SIM)	PCBs (EPA 8082 / 608)	Metals** (EPA 6020 / 200.8) Total (T) | Dissolved (D)	Anions (IC)***	EDB (8011)			Comments
1	BT-1-8	2/8/23	23:32	S	3	X		X		X									2x vials, 1x 4oz jar
2	BT-1-10	2/8/23	23:39	S	3	X		X		X									" "
3	BT-1-12	2/8/23	23:46	S	3	X		X		X									" "
4	BT-1-GW	2/9/23	00:30	GW	4	X		X		X									3x VOAs, 1x 500ml amber
5	BT-2-5.5	2/8/23	00:32	S	3	X		X		X									2x vials, 1x 4oz jar
6	BT-2-7	2/8/23	00:36	S	3	X		X		X									" "
7	BT-2-10	2/8/23	00:42	S	3	X		X		X									" "
8	BT-3-5.5	2/7/23	21:22	S	3	X		X		X									" "
9	BT-3-7	2/7/23	21:27	S	3	X		X		X									" "
10	BT-3-10	2/7/23	21:32	S	3	X		X		X									" "

*Matrix: A = Air, AQ = Aqueous, B = Bulk, O = Other, P = Product, S = Soil, SD = Sediment, SL = Solid, W = Water, DW = Drinking Water, GW = Ground Water, SW = Storm Water, WW = Waste Water

**Metals (Circle): MTCA-5 RCRA-8 Priority Pollutants TAL *Individual:* Ag Al As B Ba Be Ca Cd Co Cr Cu Fe Hg K Mg Mn Mo Na Ni Pb Sb Se Sr Sn Ti Tl V Zn

***Anions (Circle): Nitrate Nitrite Chloride Sulfate Bromide O-Phosphate Fluoride Nitrate+Nitrite

Turn-around Time:
☒ Standard ☐ Next Day
☐ 3 Day ☐ Same Day
☐ 2 Day _____ (specify)

I represent that I am authorized to enter into this Agreement with Fremont Analytical on behalf of the Client named above, that I have verified Client's agreement to each of the terms on the front and backside of this Agreement.

Relinquished (Signature)	Print Name	Date/Time	Received (Signature)	Print Name	Date/Time
x M. Black	Michael Black	2/9/23 4:45pm	x	Romiro Garcia	2/9 16.44
Relinquished (Signature)	Print Name	Date/Time	Received (Signature)	Print Name	Date/Time
x			x		

Fremont Analytical
3600 Fremont Ave N.
Seattle, WA 98103
Tel: 206-352-3790
Fax: 206-352-7178

Chain of Custody Record & Laboratory Services Agreement

Date: 2/9/2023 Page: 2 of: 2

Laboratory Project No (internal): 2302195

Project Name: Woodinville SR202 Trestle Widening

Special Remarks: GW Samples collected with temp. well, peristaltic pump, disposable tubing

Project No: 60683438 Task 6.5.6

Client: AECOM Technical Services Inc.

Collected by: Michael Black (AECOM)

Address: 1111 3rd Ave. Suite 1600

Location: Woodinville, WA

City, State, Zip: Seattle, WA 98101

Report To (PM): Markus Walbaum

Telephone: 206-438-2197

Sample Disposal: ☐ Return to client ☒ Disposal by lab (after 30 days)

Fax:

PM Email: markus.walbaum@aecom.com

	Sample Name	Sample Date	Sample Time	Sample Type (Matrix)*	# of Cont.	VOCs (EPA 8260 / 624)	BTEX	Gasoline Range Organics (GX)	Hydrocarbon Identification (HCID)	Diesel/Heavy Oil Range Organics (DX)	SVOCs (EPA 8270 / 625)	PAHs (EPA 8270 - SIM)	PCBs (EPA 8082 / 608)	Metals** (EPA 6020 / 200.8)	Total (T) | Dissolved (D)	Anions (IC)***	EDB (8011)	Comments
1	BT-3-GW	2/7/23	23:00	GW	4	X		X		X								3x VOAs, 1x 500ml amber
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

*Matrix: A = Air, AQ = Aqueous, B = Bulk, O = Other, P = Product, S = Soil, SD = Sediment, SL = Solid, W = Water, DW = Drinking Water, GW = Ground Water, SW = Storm Water, WW = Waste Water

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☐ 3 Day ☐ Same Day
☐ 2 Day _____ (specify)

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Relinquished (Signature)	Print Name	Date/Time	Received (Signature)	Print Name	Date/Time
x [signature]	Michael Black	2/9/23 4:45pm	x [signature]	Ramiro Garcia	2/9 16-44
Relinquished (Signature)	Print Name	Date/Time	Received (Signature)	Print Name	Date/Time
x			x		

60683538

Chain of Custody Record & Laboratory Services Agreement

Fremont Analytical
3600 Fremont Ave N.
Seattle, WA 98103
Tel: 206-352-3790
Fax: 206-352-7178

Date: 2/9/2023 Page: 1 of: 2

Laboratory Project No (internal): 2302195

Project Name: Woodinville SR202 Trestle Widening

Special Remarks: Samples from HSA with SPT sampling. GW samples collected with temp. wells, peristaltic pump, disposable tubing

Project No: ~~60683438~~ Task 6.5.6

Client: AECOM Technical Services, Inc.

Collected by: Michael Black (AECOM)

Address: 1111 3rd Ave. Suite 1600

Location: Woodinville, WA

City, State, Zip: Seattle, WA, 98101

Telephone: 206-438-2197

Report To (PM): Markus Walbaum

Sample Disposal: ☐ Return to client ☒ Disposal by lab (after 30 days)

Fax:

PM Email: markus.walbaum@aecom.com

edits per MW 2/10/23 -cg

	Sample Name	Sample Date	Sample Time	Sample Type (Matrix)*	# of Cont.	VOCs (EPA 8260 / 624)	BTEX	Gasoline Range Organics (GX)	Hydrocarbon Identification (HCID)	Diesel/Heavy Oil Range Organics (DX)	SVOCs (EPA 8270 / 625)	PAHs (EPA 8270 - SIM)	PCBs (EPA 8082 / 608)	Metals** (EPA 6020 / 200.8) Total (T) | Dissolved (D)	Anions (IC)***	EDB (8011)	Comments
1	BT-1-8	2/8/23	23:32	S	3	X		X		X							2x vials, 1x 4oz jar
2	BT-1-10	2/8/23	23:39	S	3	X		X		X							On hold " "
3	BT-1-12	2/8/23	23:46	S	3	X		X		X							" "
4	BT-1-GW	2/9/23	00:30	GW	4	X		X		X							3x VOAs, 1x 500ml amber
5	BT-2-5.5	2/8/23	00:32	S	3	X		X		X							On hold 1x 4oz jar
6	BT-2-7	2/8/23	00:36	S	3	X		X		X							" "
7	BT-2-10	2/8/23	00:42	S	3	X		X		X							" "
8	BT-3-5.5	2/7/23	21:22	S	3	X		X		X							" "
9	BT-3-7	2/7/23	21:27	S	3	X		X		X							On hold "
10	BT-3-10	2/7/23	21:32	S	3	X		X		X							" "

*Matrix: A = Air, AQ = Aqueous, B = Bulk, O = Other, P = Product, S = Soil, SD = Sediment, SL = Solid, W = Water, DW = Drinking Water, GW = Ground Water, SW = Storm Water, WW = Waste Water

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***Anions (Circle): Nitrate Nitrite Chloride Sulfate Bromide O-Phosphate Fluoride Nitrate+Nitrite

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x M. Black	Michael Black	2/9/23 4:45pm	x	Ramiro Garcia	2/9 16.44
Relinquished (Signature)	Print Name	Date/Time	Received (Signature)	Print Name	Date/Time
x			x		

Fremont Analytical
3600 Fremont Ave N.
Seattle, WA 98103
Tel: 206-352-3790
Fax: 206-352-7178

Chain of Custody Record & Laboratory Services Agreement

Date: 2/9/2023 Page: 2 of: 2

Laboratory Project No (internal): 2302195

Project Name: Woodinville SR202 Trestle Widening

Project No: ~~60683438~~ Task 6.5.6

Collected by: Michael Black (AECOM)

Location: Woodinville, WA

Report To (PM): Markus Walbaum

PM Email: markus.walbaum@aecom.com

Special Remarks: GW Samples collected with temp. well, peristaltic pump, disposable tubing

Sample Disposal: ☐ Return to client ☒ Disposal by lab (after 30 days)

Client: AECOM Technical Services Inc.

Address: 1111 3rd Ave. Suite 1600

City, State, Zip: Seattle, WA 98101

Telephone: 206-438-2197

Fax:

	Sample Name	Sample Date	Sample Time	Sample Type (Matrix)*	# of Cont.	VOCs (EPA 8260 / 624)	BTEX	Gasoline Range Organics (GX)	Hydrocarbon Identification (HCID)	Diesel/Heavy Oil Range Organics (DX)	SVOCs (EPA 8270 / 625)	PAHs (EPA 8270 - SIM)	PCBs (EPA 8082 / 608)	Metals** (EPA 6020 / 200.8)	Total (T) / Dissolved (D)	Anions (IC)***	EDB (8011)	Comments
1	BT-3-GW	2/7/23	23:00	GW	4	X		X		X								3x VOAs, 1x 500ml amber
2																		
3																		
4																		
5																		
6																		
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*Matrix: A = Air, AQ = Aqueous, B = Bulk, O = Other, P = Product, S = Soil, SD = Sediment, SL = Solid, W = Water, DW = Drinking Water, GW = Ground Water, SW = Storm Water, WW = Waste Water

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***Anions (Circle): Nitrate Nitrite Chloride Sulfate Bromide O-Phosphate Fluoride Nitrate+Nitrite

Turn-around Time:
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x [signature]	Michael Black	2/9/23 4:45pm	x [signature]	Ramiro Garcia	2/9 16-44
Relinquished (Signature)	Print Name	Date/Time	Received (Signature)	Print Name	Date/Time
x			x		

Appendix E – Laboratory Data Quality Review

AECOM
1111 Third Ave
Suite 1600
Seattle, WA 98101
www.aecom.com

206 438 2700 tel
866 495 5288 fax

Memorandum

Subject	Laboratory Data Quality Review Summary Woodinville SR202 Trestle Widening
Date	April 10, 2023

The summary data quality review of 6 soil samples and 2 groundwater samples, collected on February 7 through February 9, 2023, has been completed. The samples were collected by AECOM personnel and were analyzed at Fremont Analytical (FA), located in Seattle, Washington, for volatile organic compounds (VOCs) by EPA Method 8260D; total petroleum hydrocarbons (TPHs) by Washington State Department of Ecology (Ecology) methods NWTPH-Gx (gasoline-range TPH) and NWTPH-Dx (diesel-range and heavy oil-range TPH) as described below. The analyses were performed in general accordance with the methods specified in EPA's *Test Methods for Evaluating Solid Waste (SW-846)* and Ecology's *Analytical Methods for Petroleum Hydrocarbons*, June 1997. The laboratory provided summary reports containing sample results and associated quality assurance (QA) and quality control (QC) data. The following samples are associated with FA laboratory group 2302195:

Sample ID	Sample Matrix	Laboratory Group	FA ID	Requested Analyses
BT-1-8	Soil	2302195	2302195-001	VOCs, TPHs
BT-1-10	Soil		2302195-002	NA
BT-1-12	Soil		2302195-003	VOCs, TPHs
BT-1-GW	Groundwater		2302195-004	VOCs, TPHs
BT-2-5.5	Soil		2302195-005	NA
BT-2-7	Soil		2302195-006	VOCs, TPHs
BT-2-10	Soil		2302195-007	VOCs, TPHs
BT-3-5.5	Soil		2302195-008	VOCs, TPHs
BT-3-7	Soil		2302195-009	NA
BT-3-10	Soil		2302195-010	VOCs, TPHs
BT-3-GW	Groundwater		2302195-011	VOCs, TPHs

NA – Not analyzed

Upon receipt by FA, the sample jar information was compared to the associated chain-of-custody (COC) and the cooler temperatures were recorded. No discrepancies related to sample ID were noted by the laboratory and the coolers were received at temperatures within the EPA-recommended limits of greater than 0°C and less than or equal to 6°C. The COCs for laboratory group 2302195 were edited by AECOM personnel to update the project number. Samples BT-1-10, BT-2-5.5 and BT-3-7 were put on hold without analysis. No results were reported for these samples.

AECOM

Laboratory Data Quality Review Summary
Woodinville SR202 Trestle Widening
Soil and Groundwater Sampling – February 2023

Holding times, field/method/trip blanks, surrogate recoveries, matrix spike/matrix spike duplicate recoveries, laboratory and/or field duplicate results, blank spike recoveries (laboratory control samples), and reporting limits were reviewed to assess compliance with applicable methods. If data qualification was required, data were qualified based on the definitions and use of qualifying flags outlined in the EPA documents *National Functional Guidelines for Organic Superfund Methods Data Review,* November 2020, and *National Functional Guidelines for Inorganic Superfund Methods Data Review*, November 2020. A summary of data qualifiers assigned to this sample set is included as Table 1.

Organic Analyses

Samples were analyzed for VOCs and TPHs by the methods described in the introduction to this report.

1. Holding Times – Acceptable

2. Blanks – Acceptable

3. Surrogates – Acceptable

4. Laboratory Control Sample/Laboratory Control Sample Duplicate (LCS/LCSD) – Acceptable

5. Matrix Spike/Matrix Spike Duplicate (MS/MSD) – Acceptable except as noted below:

   VOCs by Method 8260D – An MS was performed using BT-1-GW. Results were acceptable.

   An MS was performed using a soil sample from a project unrelated to the Woodinville Trestle Widening project. No data were qualified based on non-project QC results.

   Gasoline Range Organics by Method NWTPH-Gx – Matrix spikes were performed using soil sample BT-1-12 and groundwater sample BT-3-GW. Results were acceptable.

   Diesel and Oil Range Organics by Method NWTPH-Dx – MS/MSDs were performed using BT-1-8 and a soil sample from a project unrelated to the Woodinville Trestle Widening project. Results were acceptable.

6. Laboratory Duplicate – Acceptable

   VOCs by Method 8260D – Laboratory duplicates were performed using BT-1-8 and one soil sample and two groundwater samples from projects unrelated to the Woodinville Trestle Widening project. Results were comparable.

   Gasoline Range Organics by Method NWTPH-Gx – Laboratory duplicates were performed using BT-1-8 and one soil sample and two groundwater samples from projects unrelated to the Woodinville Trestle Widening project. Results were comparable.

   Diesel and Oil Range Organics by Method NWTPH-Dx – Laboratory duplicates for soil samples were performed using BT-1-12 and one sample from a project unrelated to the Woodinville Trestle Widening project. Results were comparable.

7. Reporting Limits – Acceptable

   General – The reporting limits for one or more analytes reported as not detected in multiple samples were elevated due to moisture content and/or limited sample volume. The elevated reporting limits do not impact the use of the data for regulatory comparison.

8. Other Items of Note:

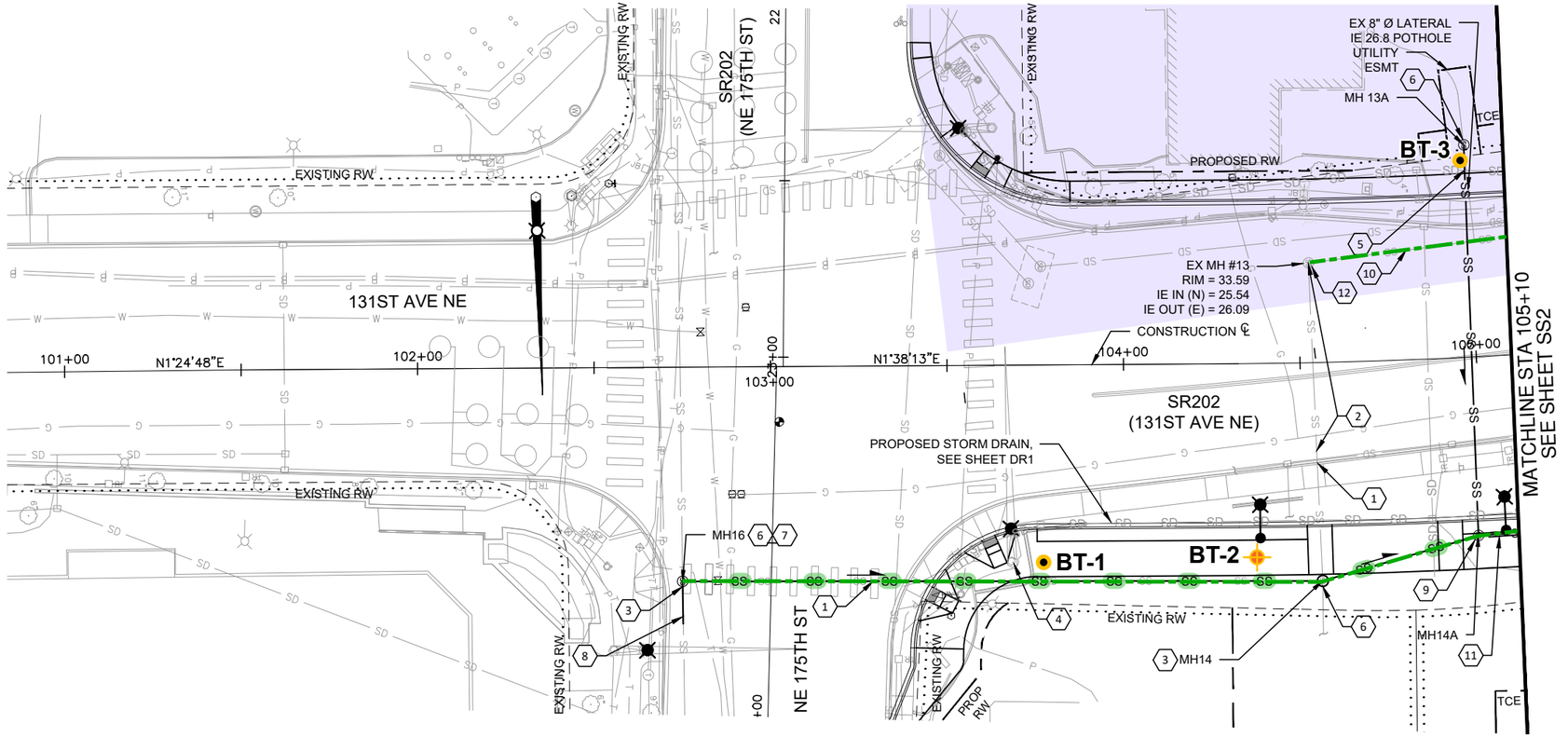
   VOCs by Method 8260D – The laboratory noted that the percent differences for dichlorodifluoromethane and chloroethane were outside acceptance criteria (low) in the continuing calibration verification (CCV) associated with batch 39415. The results for dichlorofluoromethane and chloroethane in samples BT-1-8, BT-1-12, BT-2-7, BT-2-10, BT-3-5.5, and BT-3-10 were qualified as estimated and flagged 'UJ' due this CCV result.

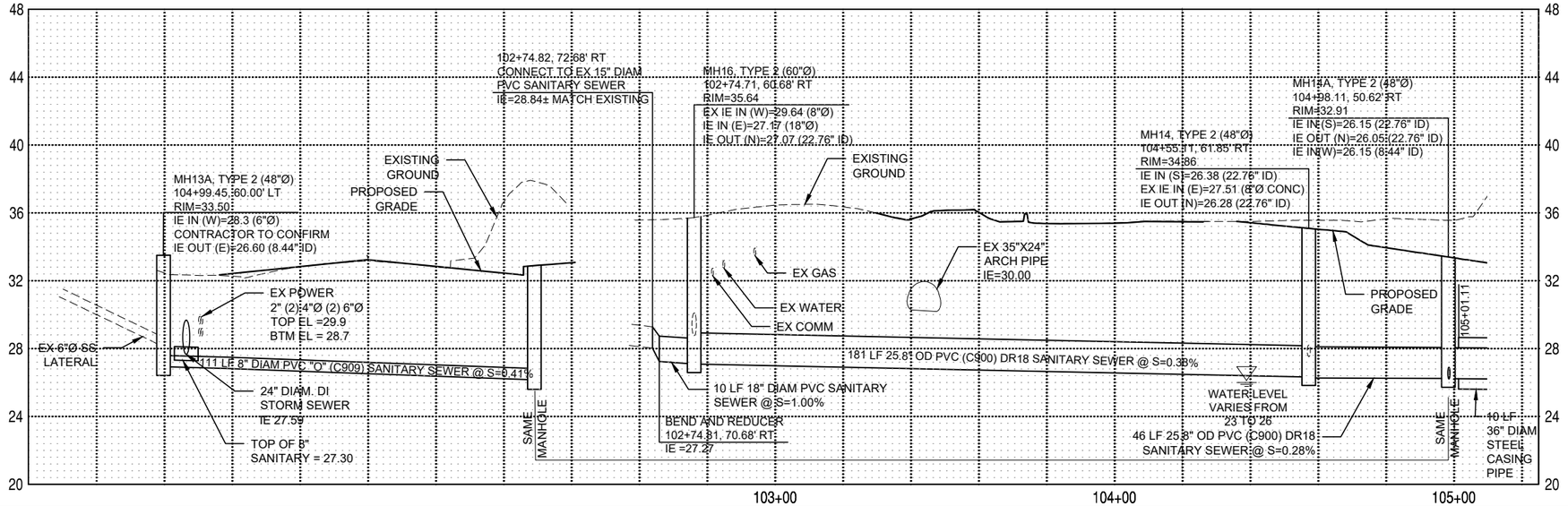
Overall Assessment of Data

The data reported in these laboratory groups, as qualified, are usable for meeting project objectives. The completeness for laboratory group 2302195 is 100%.

Appendix F – Sanitary Sewer Plan and Profile - Area of PCE Media Management

SEC. 9, T.26N., R.5E., W.M.
CITY OF WOODINVILLE





CONSTRUCTION NOTES

1. REMOVE EXISTING PIPE
2. ABANDON 36" DIA EXISTING STEEL CASING PIPE, REMOVE EXISTING 18" DIAM SANITARY SEWER PIPE, PLUG EXISTING PIPE ENDS
3. REMOVE EXISTING MANHOLE
4. PROTECT TRAFFIC SIGNAL / LIGHT POLE IN PLACE
5. PROVIDE CONCRETE SADDLE, SEE SHEET SSD1
6. CONNECTION TO EXISTING SANITARY MANHOLE OR EXISTING PIPE CONNECTION TO MANHOLE (PAID AS CONNECTION TO SANITARY MANHOLE)
7. INSTALL DROP MANHOLE CONNECTION PER WOODINVILLE WATER DISTRICT DETAIL #7
8. PROVIDE 18" DIAM PVC SANITARY SEWER PIPE, BEND, AND REDUCER FITTINGS AND CONNECTION TO EXISTING 15" DIAM PVC SANITARY SEWER PIPE
9. PROVIDE CONCRETE COLLAR PER WOODINVILLE WATER DISTRICT DETAIL #21
10. ABANDON EXISTING SANITARY SEWER, PLUG ENDS PER SPECIFICATIONS AND FILL REMAINING PIPE WITH LEAN CONCRETE (INCIDENTAL TO PAY ITEM "PLUGGING EXISTING PIPE")
11. INSTALL 36" DIAM STEEL CASING PIPE, SEE WOODINVILLE WATER DISTRICT #13 FOR INSTALLATION DETAILS
12. ABANDON EXISTING MANHOLES

GENERAL NOTES

1. ALL PROPOSED MANHOLES WILL BE FIBERGLASS LINED PER SPECIFICATIONS AND ORIENTED PER WOODINVILLE WATER DISTRICT DETAIL #18.
2. ALL FRAMES AND LIDS WILL BE WATERTIGHT PER WOODINVILLE WATER DISTRICT DETAIL #20.
3. ALL INVERTS ARE TO INSIDE DIAMETER OF PIPE.

Legend

- Boring Location
- Monitoring Well
- Planned Piping Removal or Install
- Potential Area of PCE Impacted Media (saturated zone soils & groundwater)

20 0 20 40
HORIZ SCALE IN FEET

4 0 4 8
VERT SCALE IN FEET

AECOM
1111 THIRD AVENUE, SUITE 1600
SEATTLE, WASHINGTON 98101
PHONE: (206) 438-2700
FAX: (866) 495-5288

REX W. METER STATE OF WASHINGTON 29998 REGISTERED PROFESSIONAL ENGINEER

NO.	REVISION	DATE	BY	CK

CITY OF WOODINVILLE
17301 133rd AVE NE
WOODINVILLE, WA 98072
PHONE: (425) 489-2700
FAX: (425)489-2705

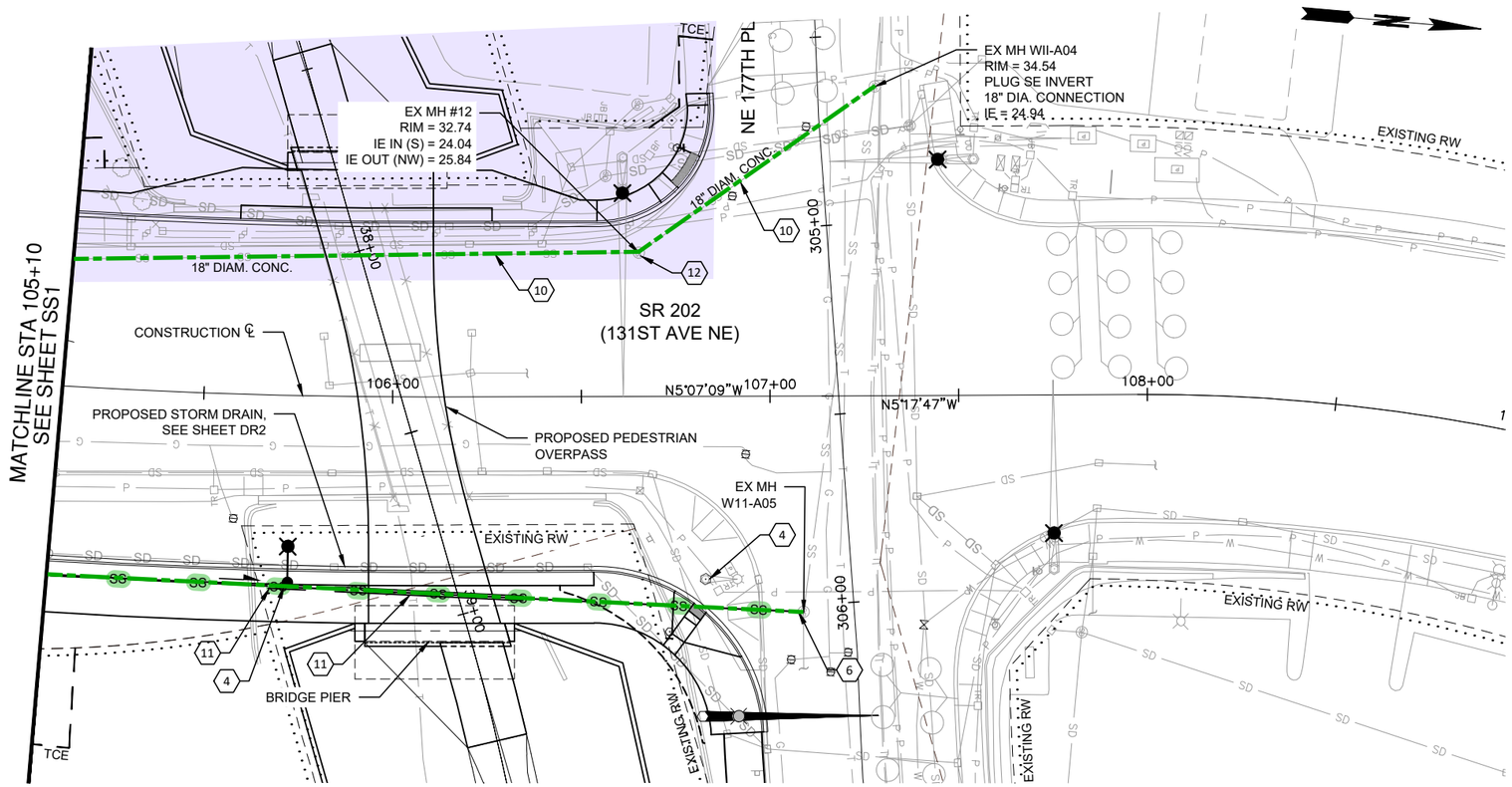
PROJECT INFORMATION
SR 202 WIDENING AND TRESTLE REPLACEMENT PROJECT

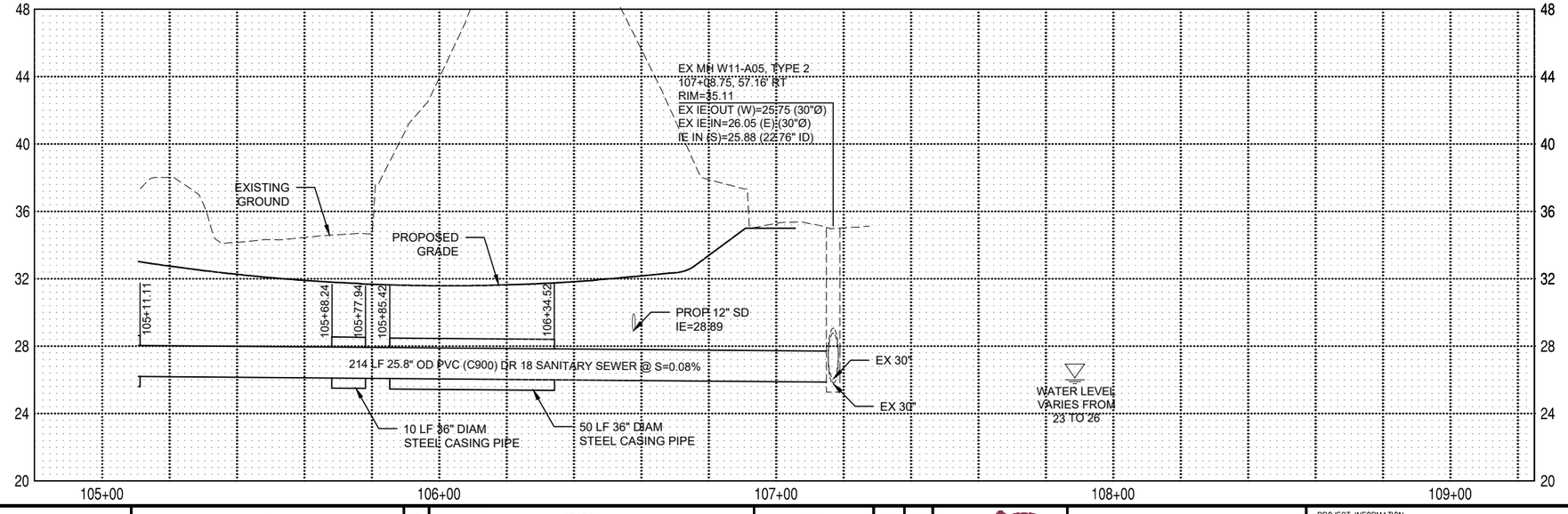
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SANITARY SEWER PLAN AND PROFILE

DATE: 4/11/25	CHECKED BY: RM
DESIGNED BY: BM	PROJECT NO.: ST-26-XXX
DRAWN BY: BM	SCALE: AS SHOWN
SHEET 18 OF XX	
DRAWING NO.: SS1	

Apr. 12, 25 - 9:42 PM brenda.mcintosh C:\Users\brenda.mcintosh\OneDrive - AECOM\Projects\Trestle\35_SS1.dwg

SEC. 9, T.26N., R.5E., W.M.
CITY OF WOODINVILLE





CONSTRUCTION NOTES

1. REMOVE EXISTING PIPE
3. REMOVE EXISTING MANHOLE
4. PROTECT TRAFFIC SIGNAL / LIGHT POLE IN PLACE
6. CONNECTION TO EXISTING SANITARY MANHOLE OR EXISTING PIPE CONNECTION TO MANHOLE (PAID AS CONNECTION TO SANITARY MANHOLE)
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11. INSTALL 36" DIAM STEEL CASING PIPE, SEE WOODINVILLE WATER DISTRICT #13 FOR INSTALLATION DETAILS
12. ABANDON EXISTING MANHOLES

Legend

- Planned Piping Removal or Install
- Potential Area of PCE Impacted Media (saturated zone soils & groundwater)

20 0 20 40
HORIZ SCALE IN FEET

4 0 4 8
VERT SCALE IN FEET

AECOM
1111 THIRD AVENUE, SUITE 1600
SEATTLE, WASHINGTON 98101
PHONE: (206) 438-2700
FAX: (866) 495-5288

REX W. MEYER
STATE OF WASHINGTON
25998
REGISTERED
PROFESSIONAL ENGINEER

NO.	REVISION	DATE	BY	CK

CITY OF WOODINVILLE
17301 133rd AVE NE
WOODINVILLE, WA 98072
PHONE: (425) 489-2700
FAX: (425)489-2705

PROJECT INFORMATION
SR 202 WIDENING AND TRESTLE REPLACEMENT PROJECT

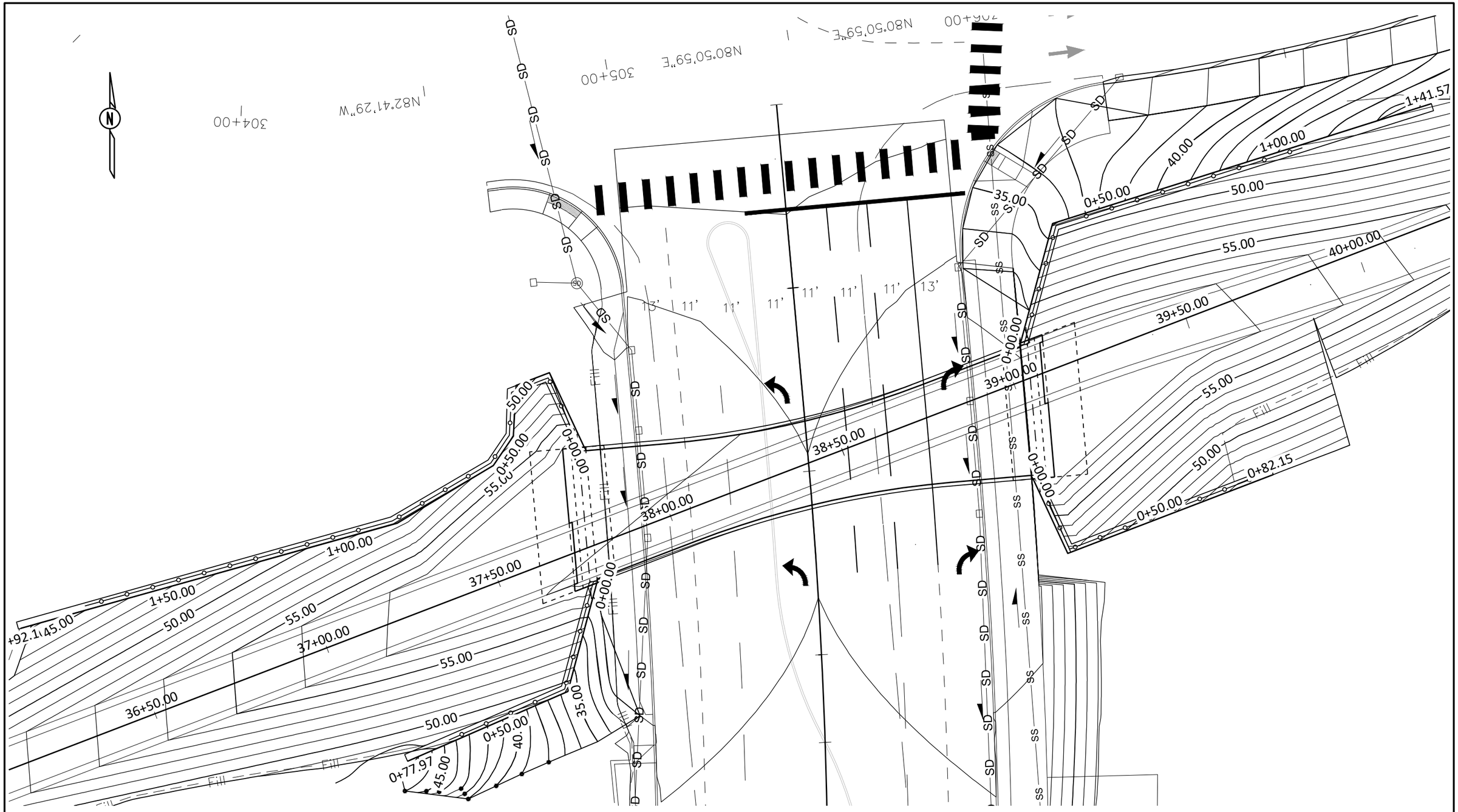
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SANITARY SEWER PLAN AND PROFILE

DATE: 4/11/25	CHECKED BY: RM
DESIGNED BY: BM	PROJECT NO.: ST-26-XXX
DRAWN BY: BM	SCALE: AS SHOWN
SHEET 19 OF XX	
DRAWING NO.: SS2	

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aecom.com

APPENDIX F RETAINING WALL GLOBAL STABILITY ANALYSIS RESULTS



AECOM

DRAWN BY	UD

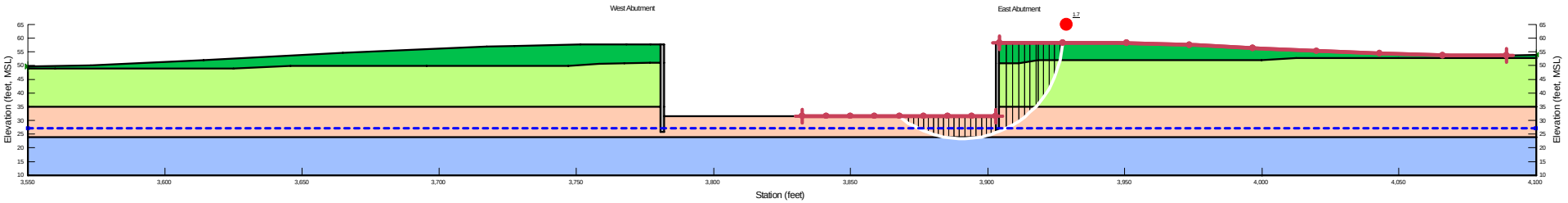
WALL PLAN EXHIBIT

F.1 Bridge Abutment Stability Analysis Results

Name: Longitudinal Section_East Abutment_Static

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 1.7

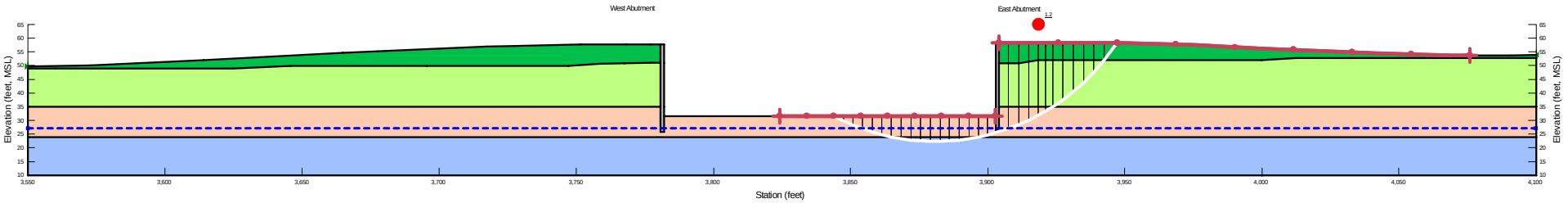
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	Dense to Very Dense Sand with Gravel and Silt	130	0	38	1
	Existing Fill	120	0	30	1
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35	1
	Retaining Wall	145			1
	WSDOT Select Borrow	130	0	36	1



Name: Longitudinal Section_East Abutment_Pseudostatic

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.25
Factor of Safety: 1.2

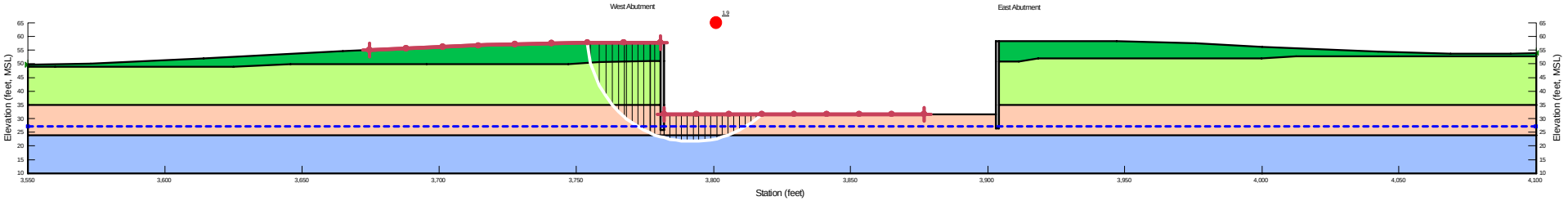
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	Dense to Very Dense Sand with Gravel and Silt	130	0	38	1
	Existing Fill	120	0	30	1
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35	1
	Retaining Wall	145			1
	WSDOT Select Borrow	130	0	36	1



Name: Longitudinal Section_West Abutment_Static

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 1.9

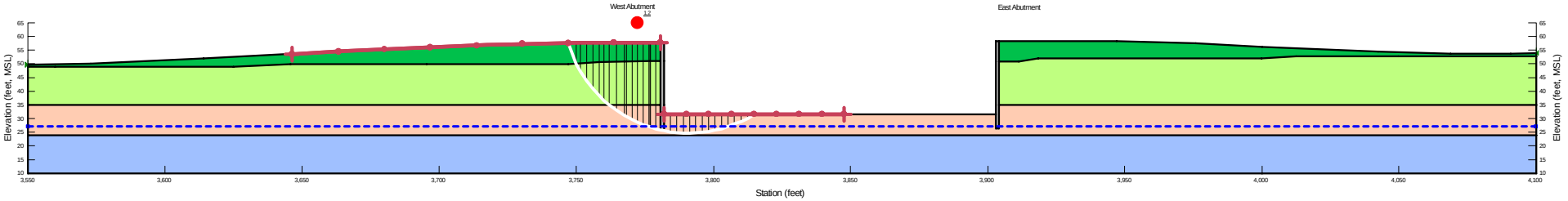
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	Dense to Very Dense Sand with Gravel and Silt	130	0	38	1
	Existing Fill	120	0	30	1
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35	1
	Retaining Wall	145			1
	WSDOT Select Borrow	130	0	36	1



Name: Longitudinal Section_West Abutment_Pseudostatic

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.25
Factor of Safety: 1.2

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	Dense to Very Dense Sand with Gravel and Silt	130	0	38	1
	Existing Fill	120	0	30	1
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35	1
	Retaining Wall	145			1
	WSDOT Select Borrow	130	0	36	1

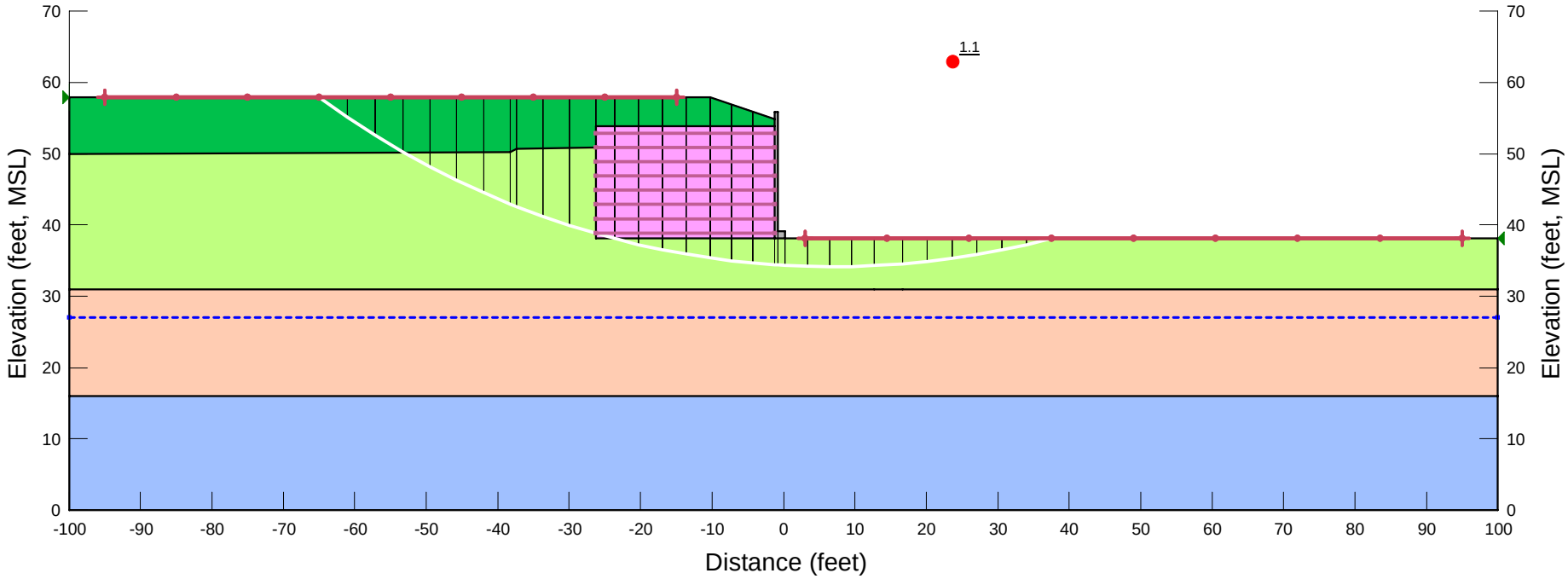


F.2 Northwest Wall Stability Analysis Results – MSE Wall

Name: NW Wall_H=17.8 ft_Pseudostatic

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

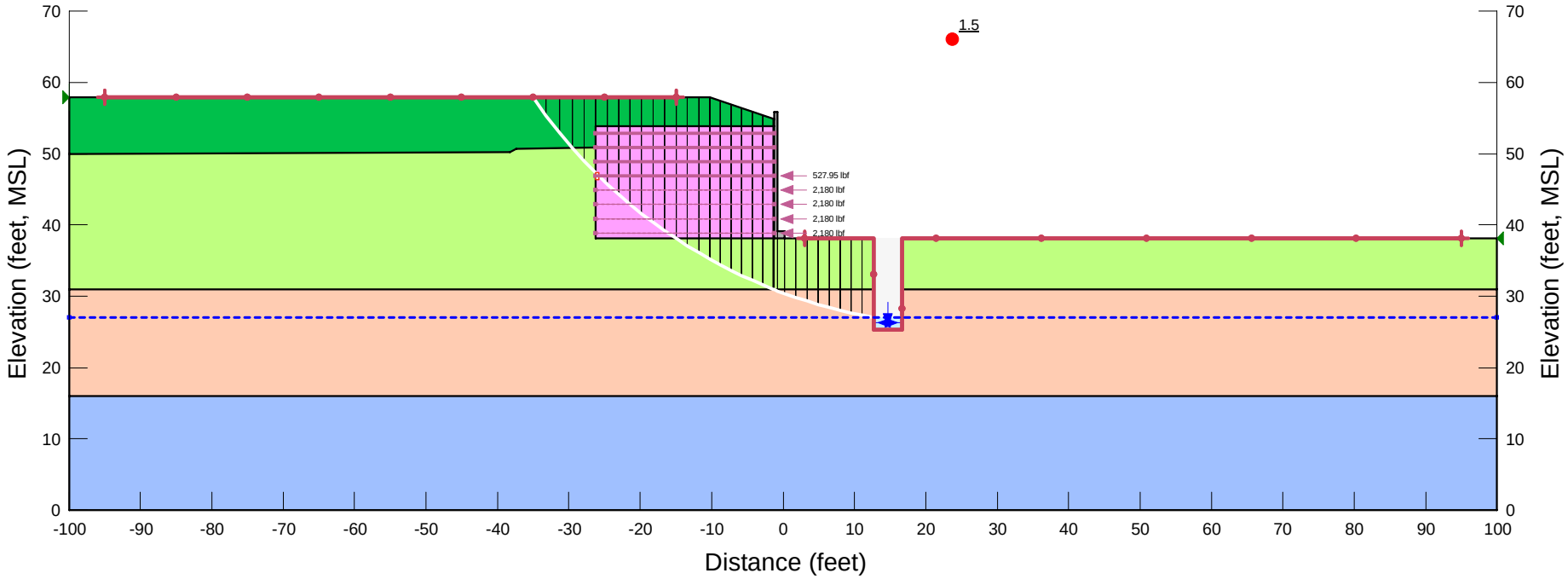


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.28
Factor of Safety: 1.1

Name: NW Wall_H=17.8 ft_SS Construction

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

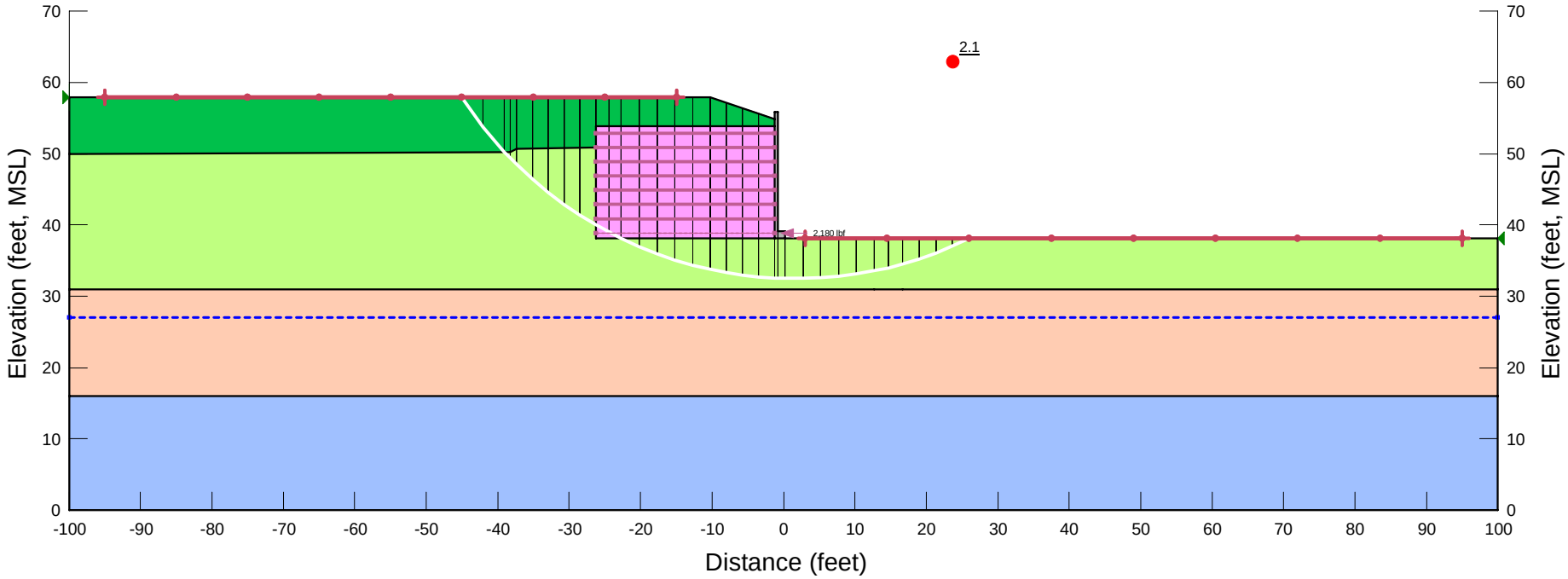


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 1.5

Name: NW Wall_H=17.8 ft_Static

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

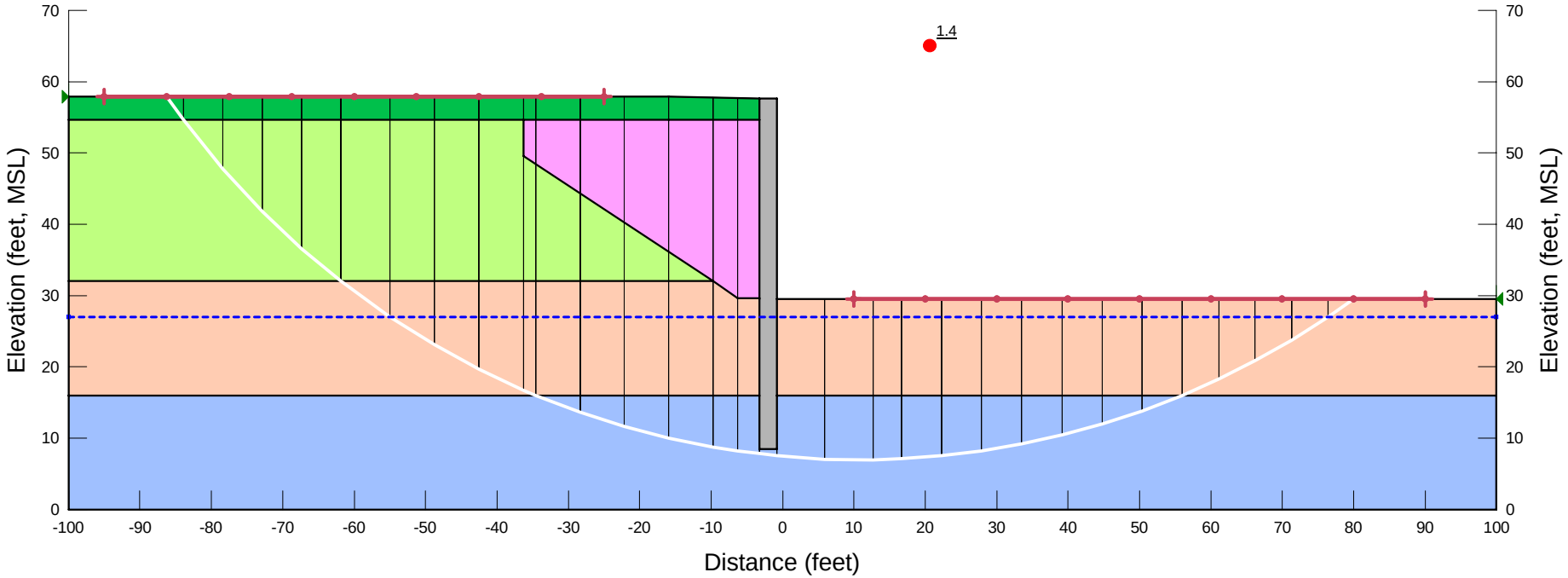


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.1

F.2.1 Northwest Wall Stability Analysis Results – Soldier Pile Wall

Name: NW Wall_H=28.2 ft_3H:1V_Pseudostatic

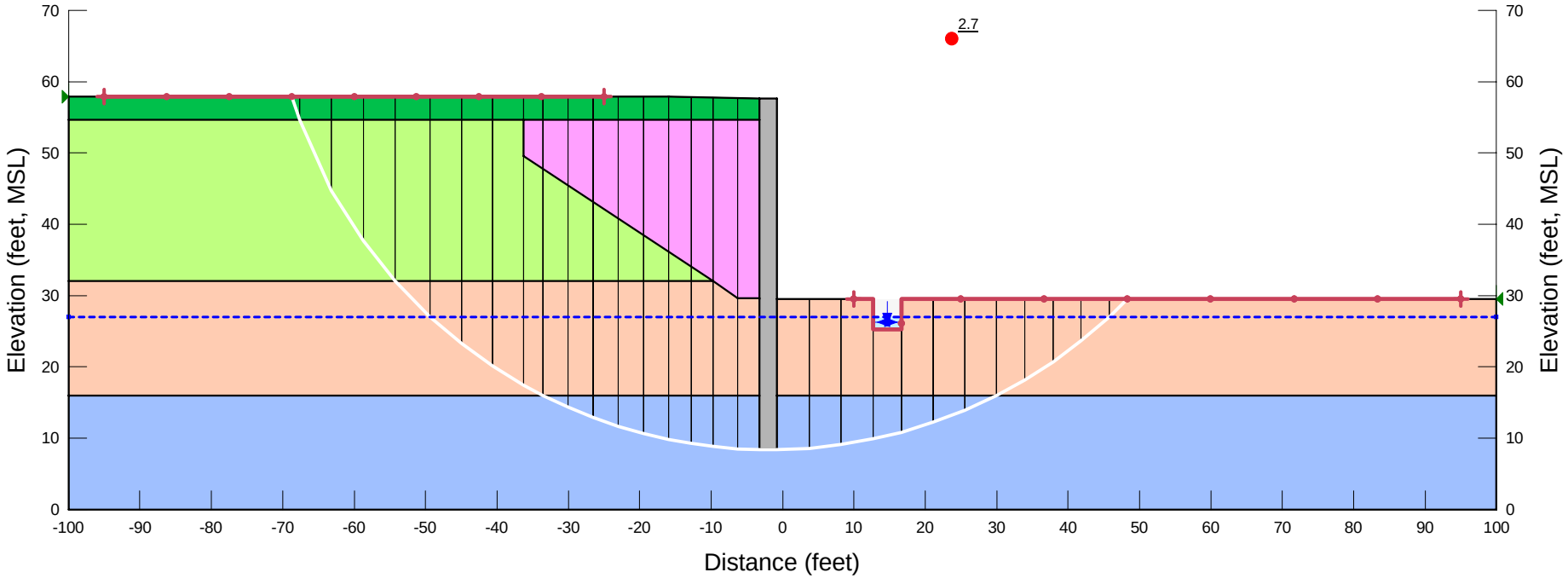
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Lightweight Fill (FGA)	25	0	41
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32



File Name: Woodinville_NW SP_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.28
Factor of Safety: 1.4

Name: NW Wall_H=28.2 ft_3H:1V_SS Construction

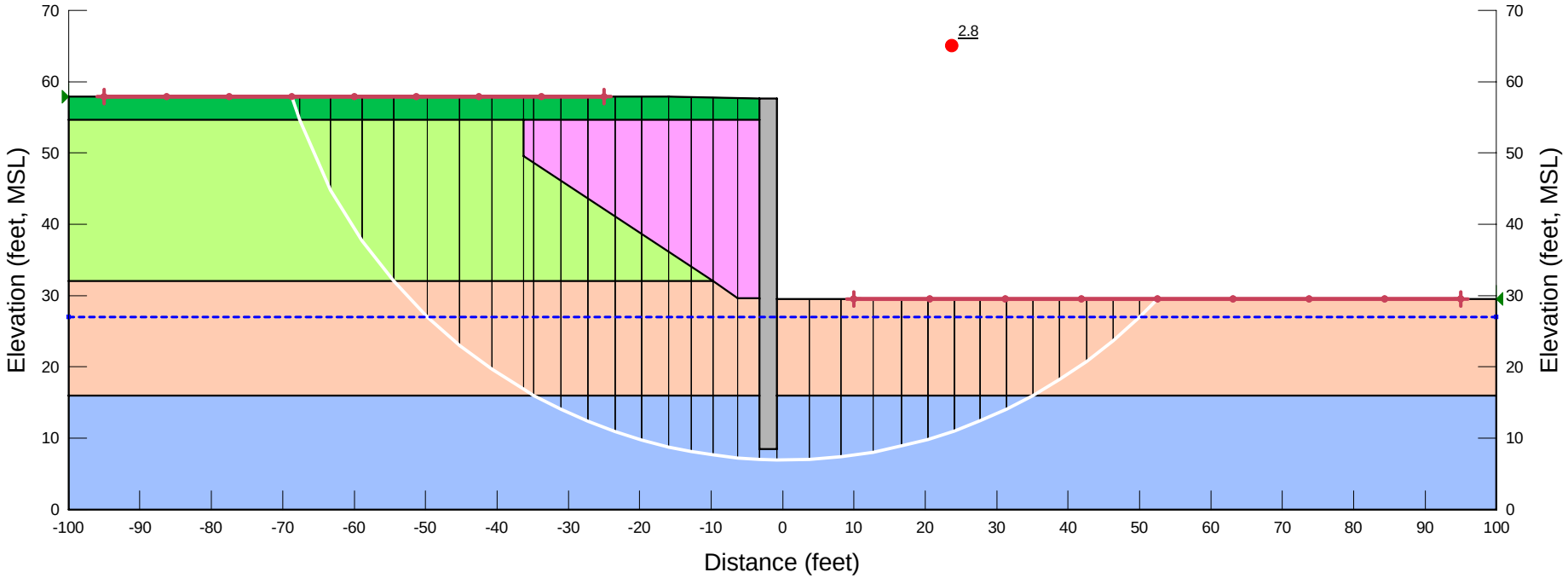
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Lightweight Fill (FGA)	25	0	41
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32



File Name: Woodinville_NW SP_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.7

Name: NW Wall_H=28.2 ft_3H:1V_Static

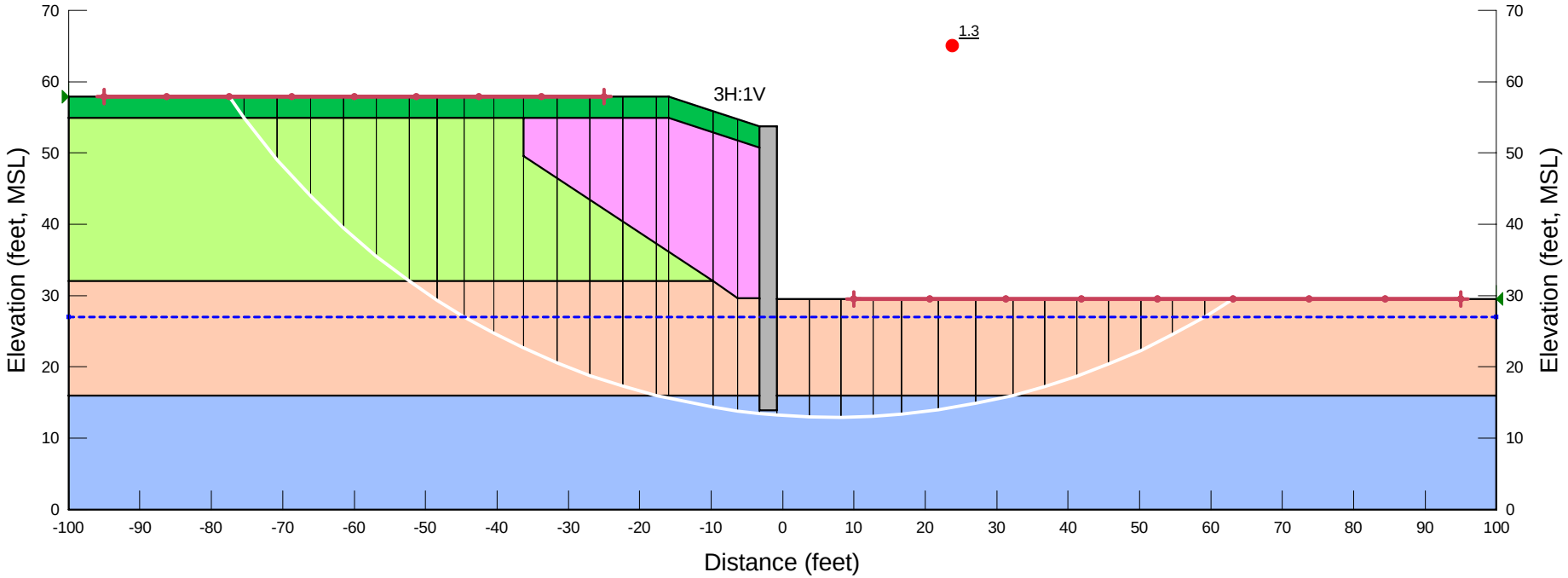
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Lightweight Fill (FGA)	25	0	41
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32



File Name: Woodinville_NW SP_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.8

Name: NW Wall_H=24.2 ft_3H:1V_Pseudostatic

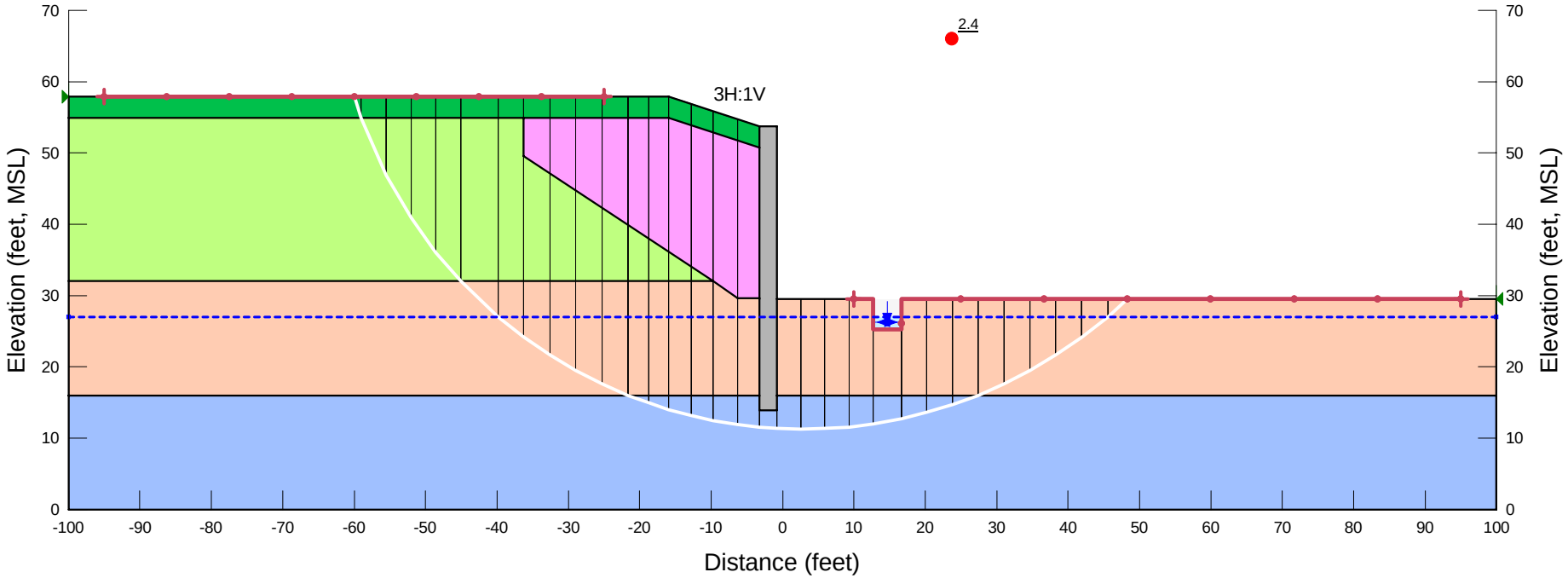
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Lightweight Fill (FGA)	25	0	41
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32



File Name: Woodinville_NW SP_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.28
Factor of Safety: 1.3

Name: NW Wall_H=24.2 ft_3H:1V_SS Construction

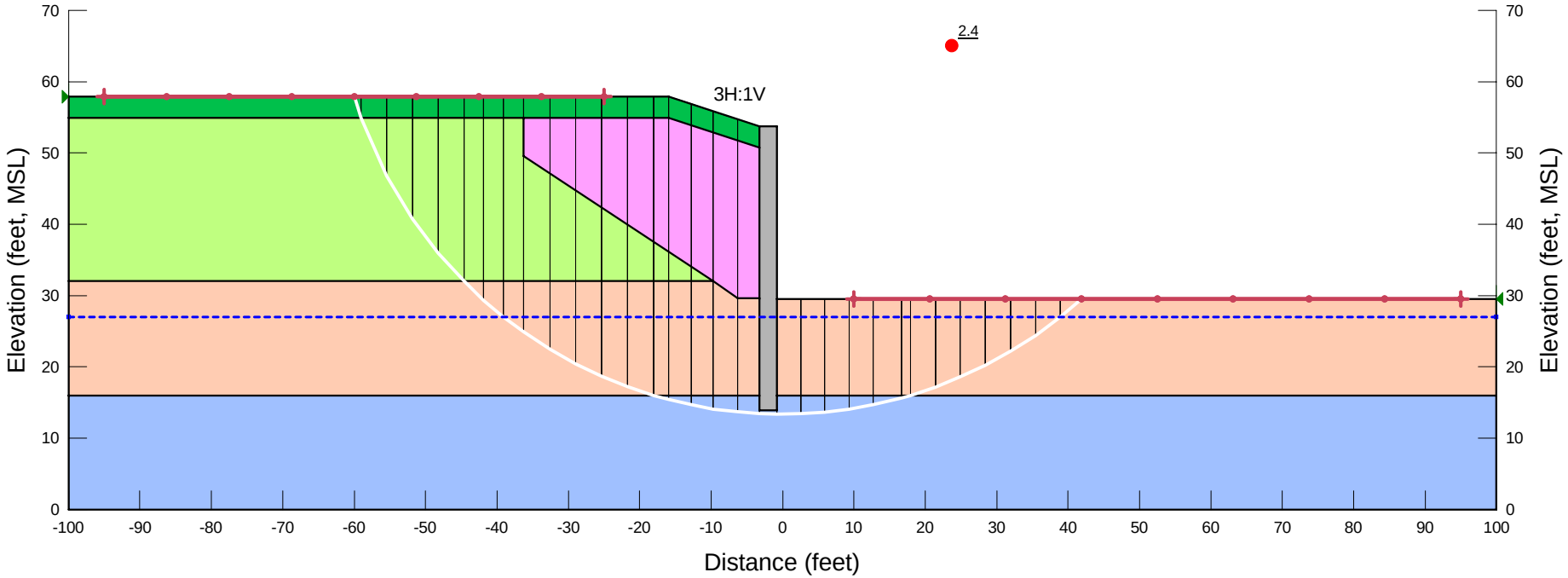
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Lightweight Fill (FGA)	25	0	41
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32



File Name: Woodinville_NW SP_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.4

Name: NW Wall_H=24.2 ft_3H:1V_Static

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Lightweight Fill (FGA)	25	0	41
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32



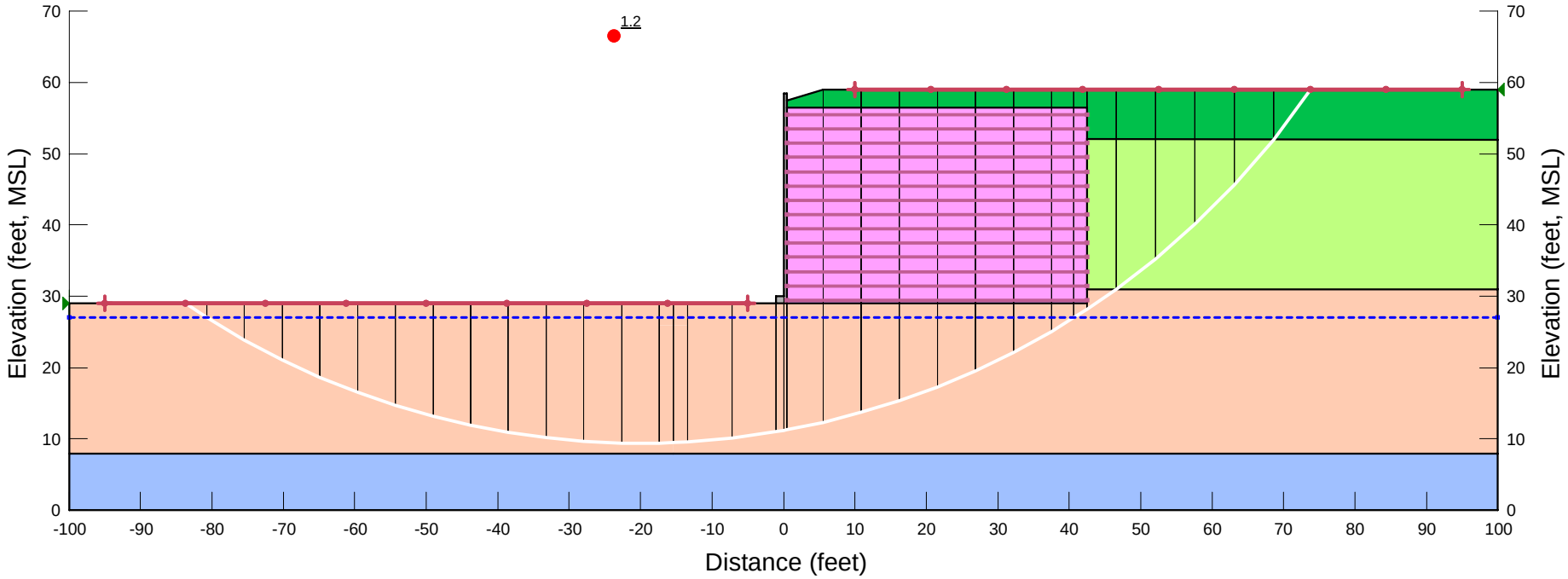
File Name: Woodinville_NW SP_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.4

F.3 Northeast Wall Stability Analysis Results – MSE Wall

Name: NE Wall_H=29.5 ft_Pseudostatic

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

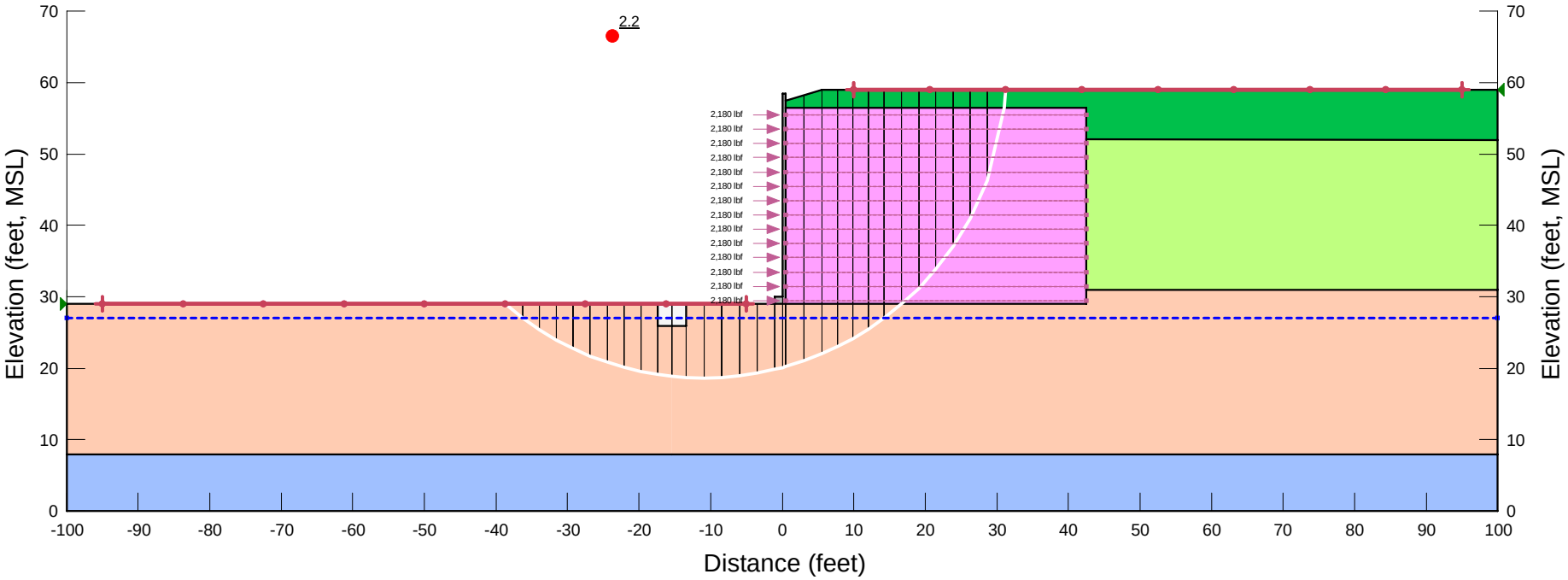


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.28
Factor of Safety: 1.2

Name: NE Wall_H=29.5 ft_SS Construction

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

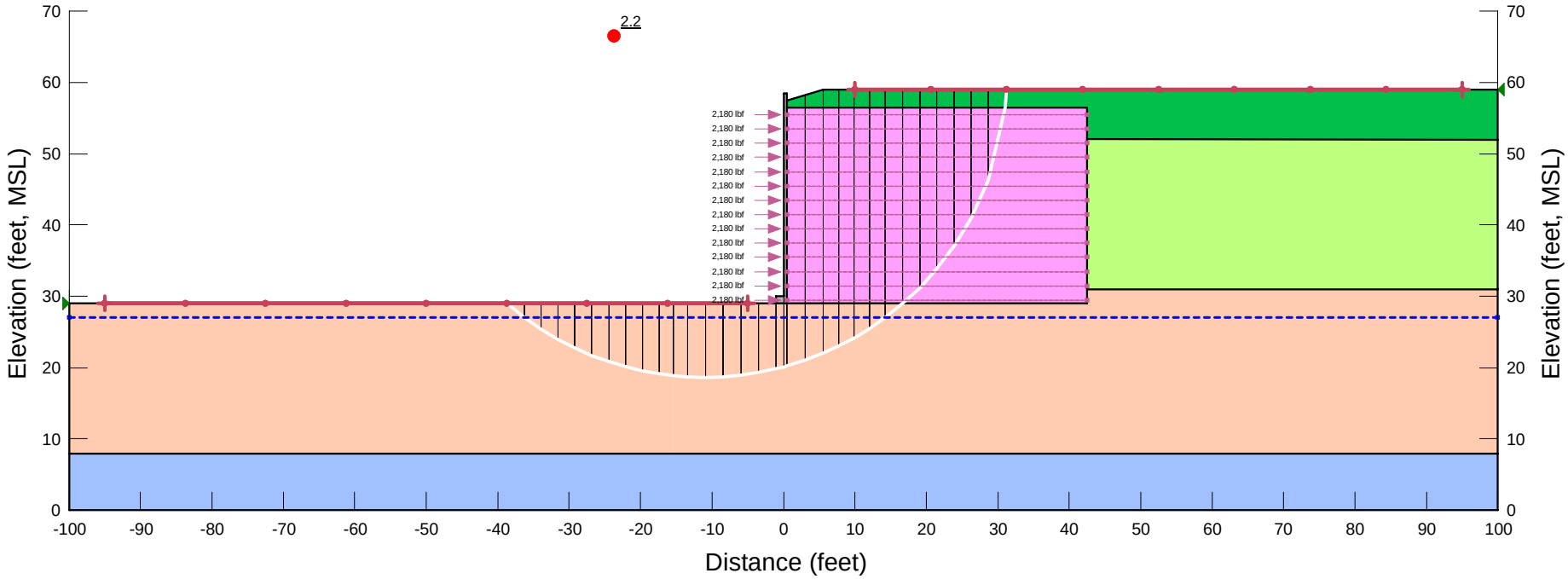


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.2

Name: NE Wall_H=29.5 ft_Static

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft



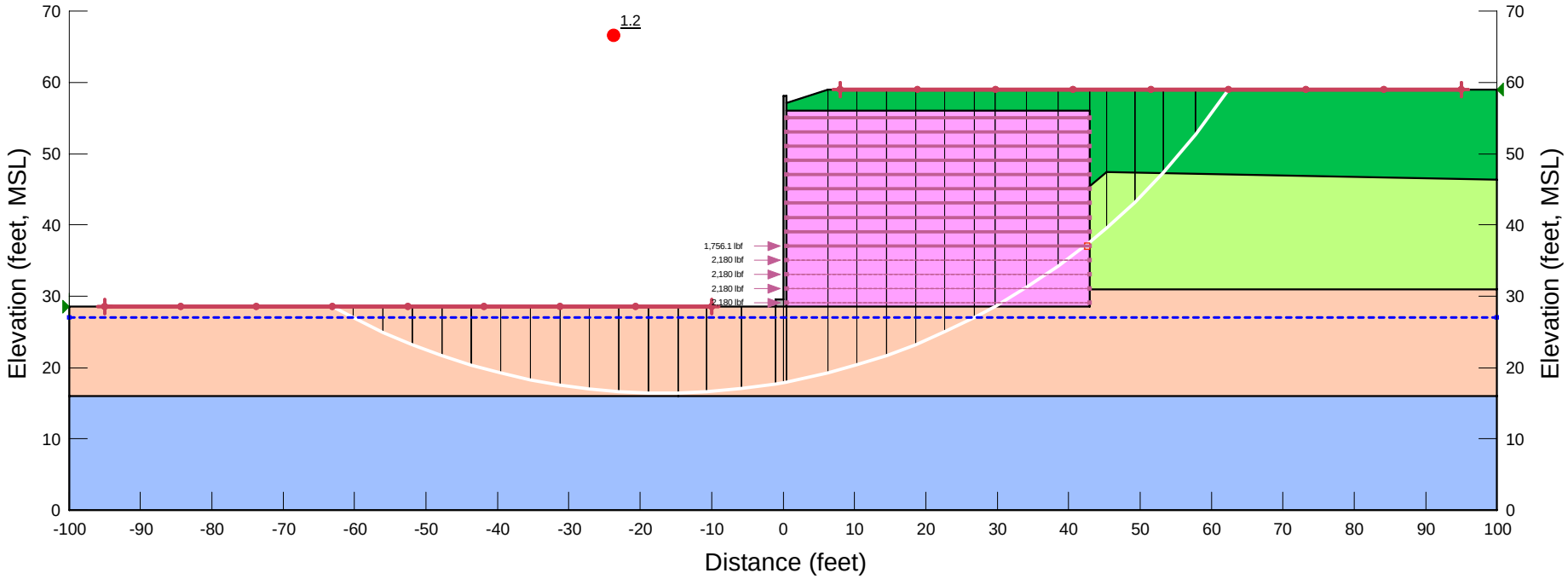
File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.2

F.4 Southwest Wall Stability Analysis Results – MSE Wall

Name: SW Wall_H=29.5 ft_Pseudostatic

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

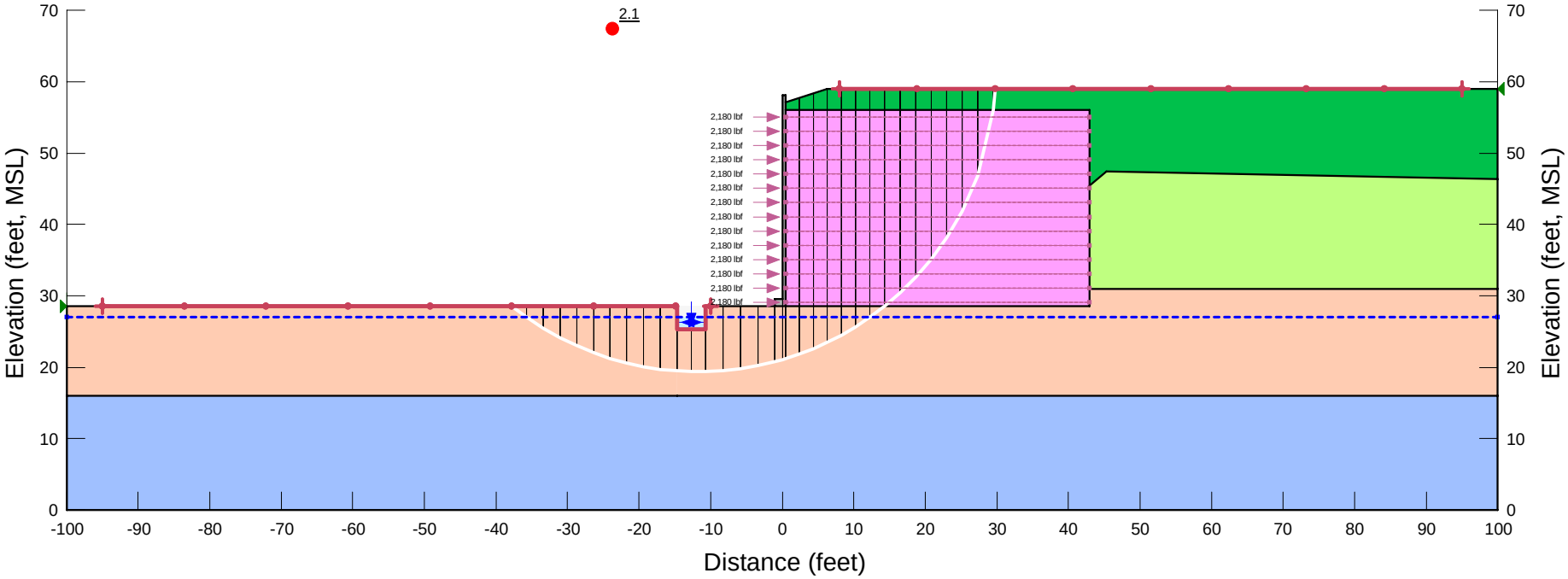


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.28
Factor of Safety: 1.2

Name: SW Wall_H=29.5 ft_SS Construction

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

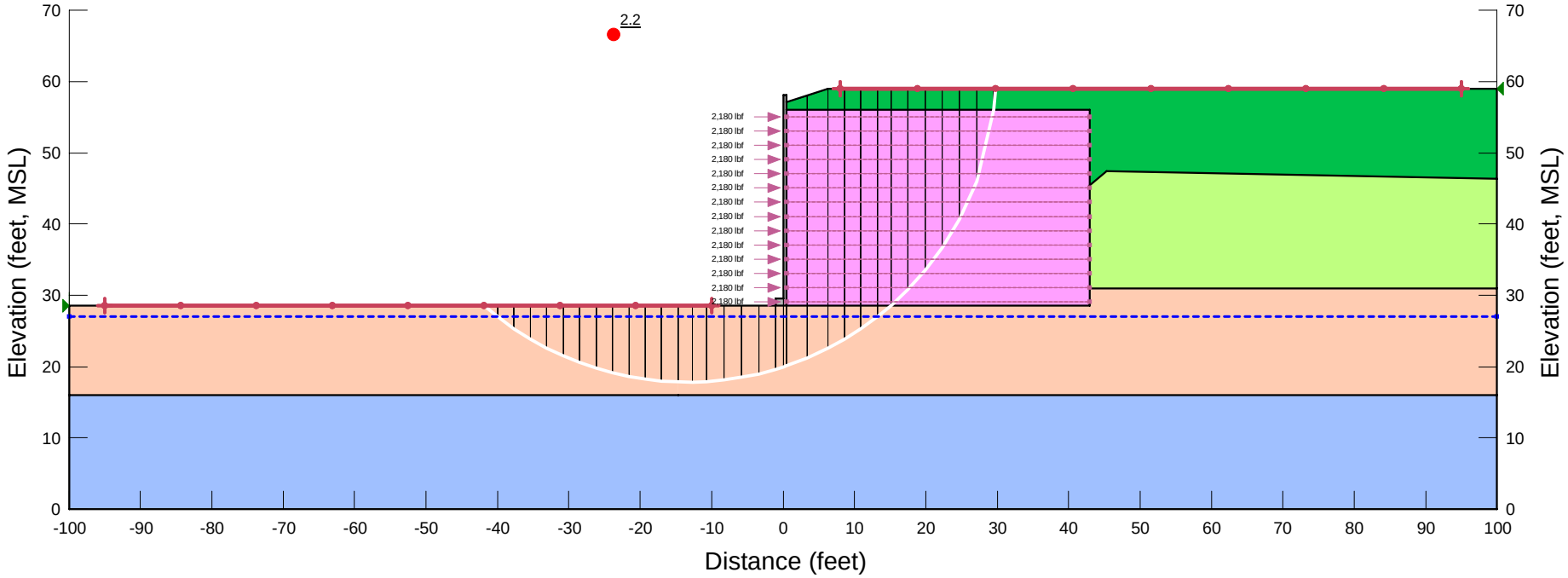


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.1

Name: SW Wall_H=29.5 ft_Static

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft



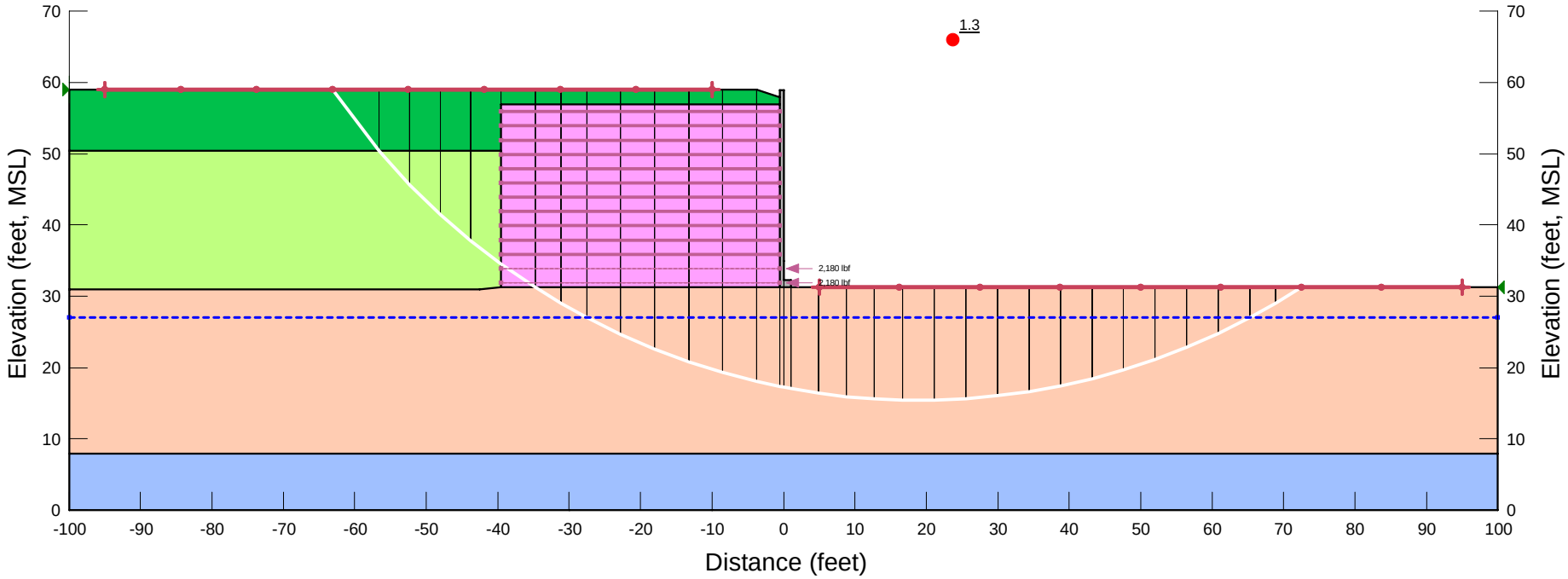
File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.2

F.5 Southeast Wall Stability Analysis Results – MSE Wall

Name: SE Wall_H=27.7 ft_Pseudostatic

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

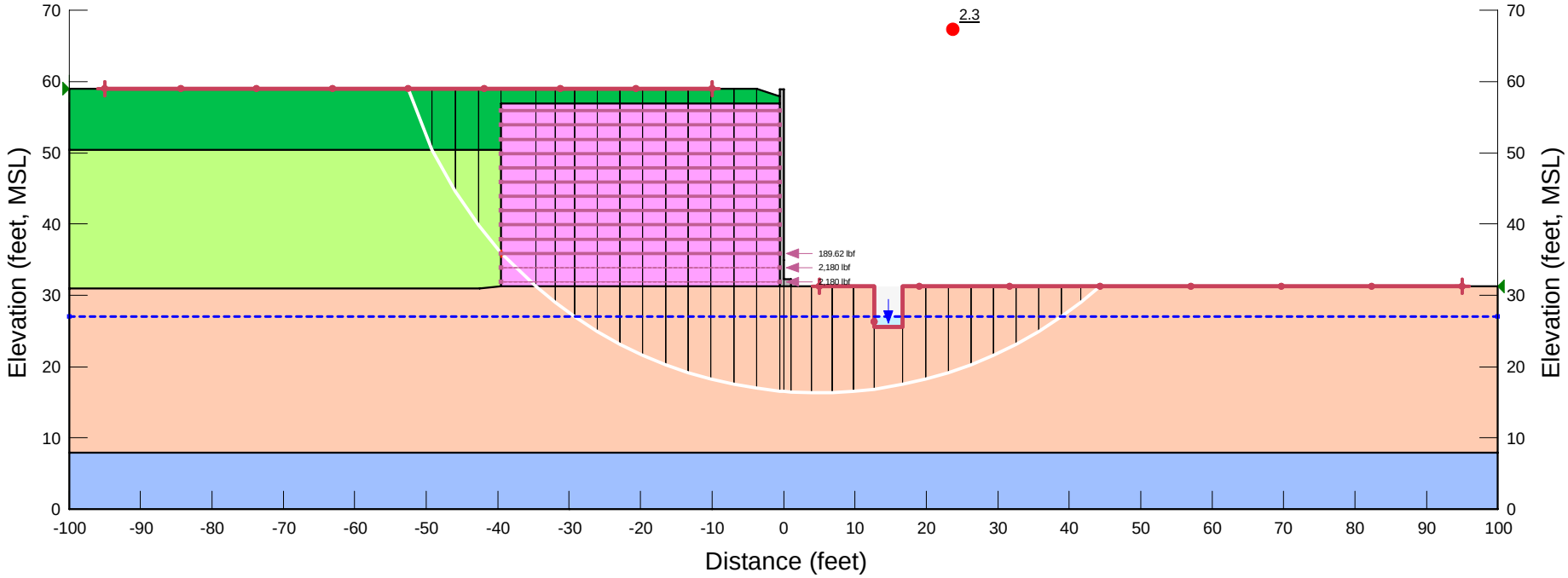


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.28
Factor of Safety: 1.3

Name: SE Wall_H=27.7 ft_SS Construction

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft

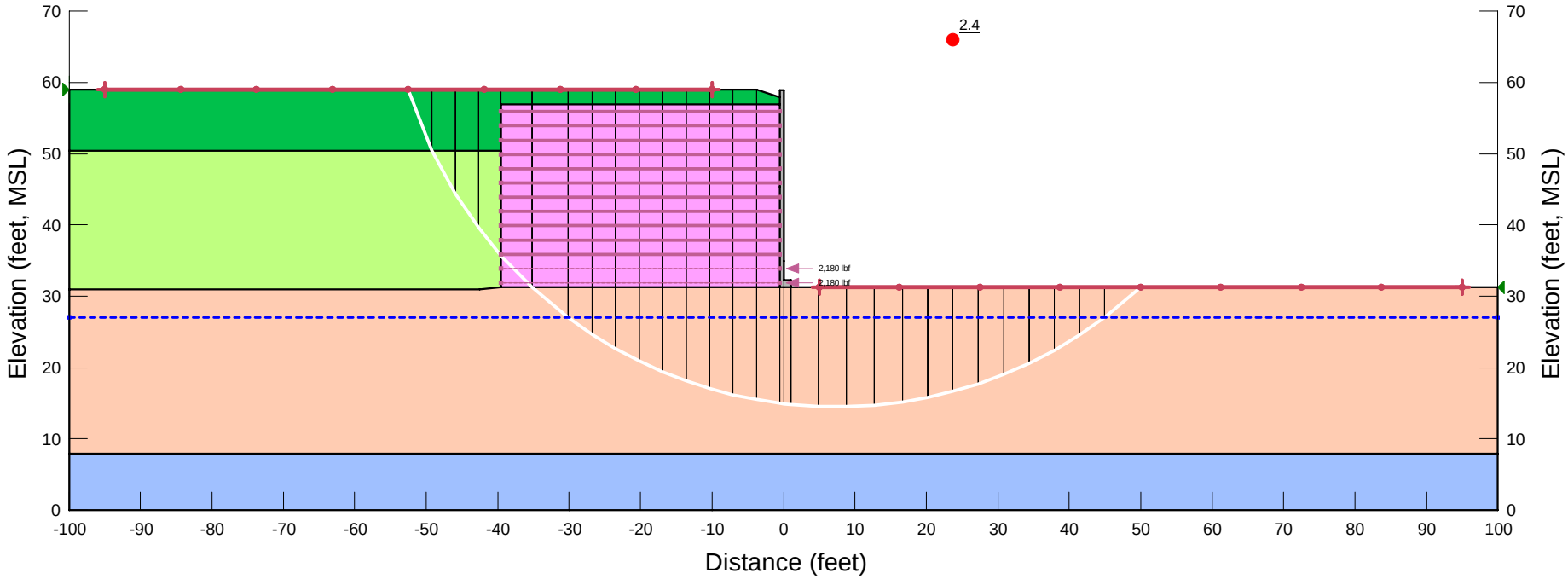


File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.3

Name: SE Wall_H=27.7 ft_Static

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	MSE Reinforced Zone	130	0	38
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	32

Color	Name	Type	Interface Adhesion (psf)	Interface Shear Angle (°)	Surface Area Factor	Tensile Capacity (lbf)	Face Anchorage	Factored Tensile Capacity
	MSE	Geosynthetic	0	23	2	2,180	Yes	2,180 lbf/ft



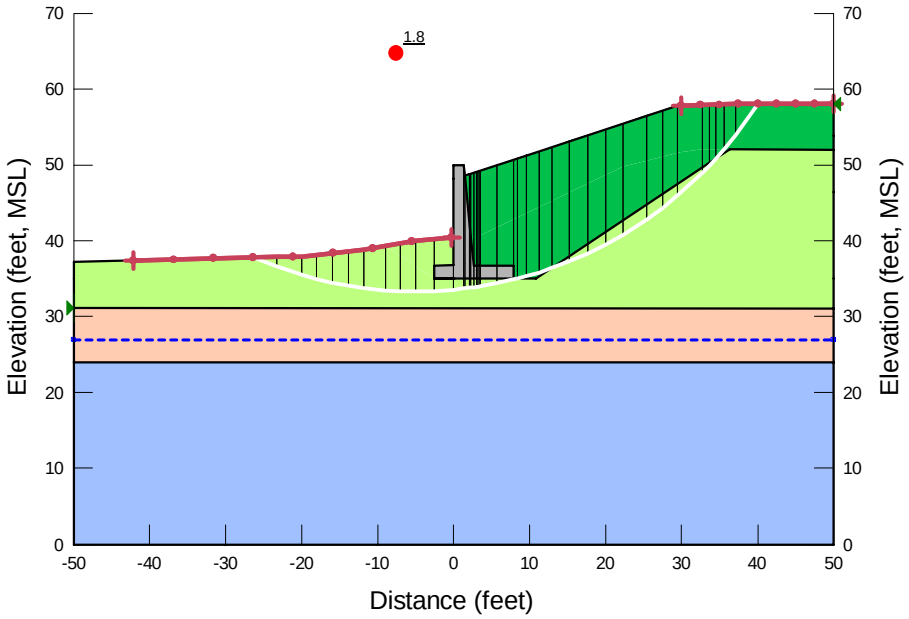
File Name: Woodinville_SSA_MSE_flexible_MIN LENGTH_2025.11.24.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.4

F.6 Northeast Wall Stability Analysis Results – Concrete Wall

Name: STA 0+70_NE Wall=15 ft_3H:1V_Static

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 1.8

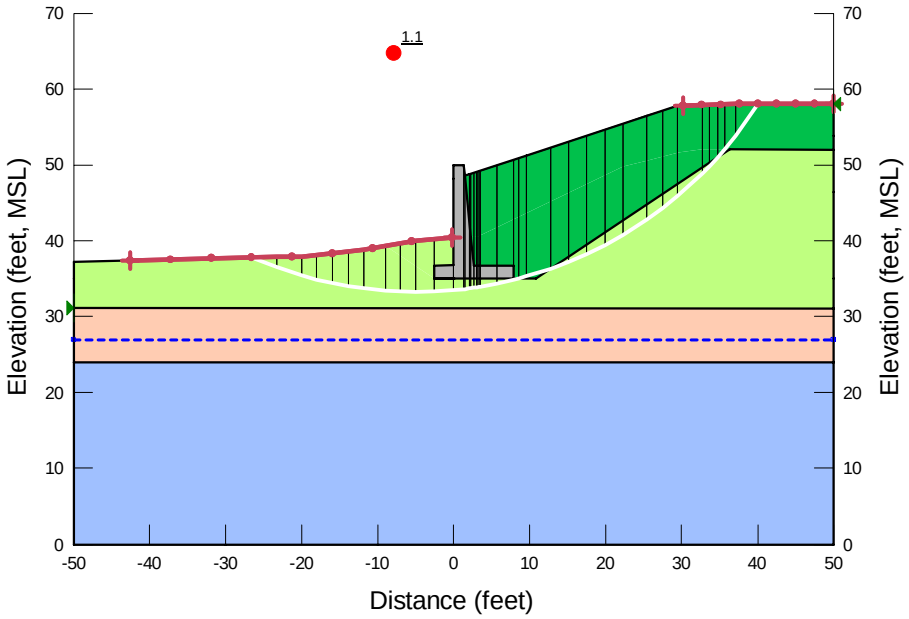
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36



Name: STA 0+70_NE Wall=15 ft_3H:1V_Pseudostatic

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.25
Factor of Safety: 1.1

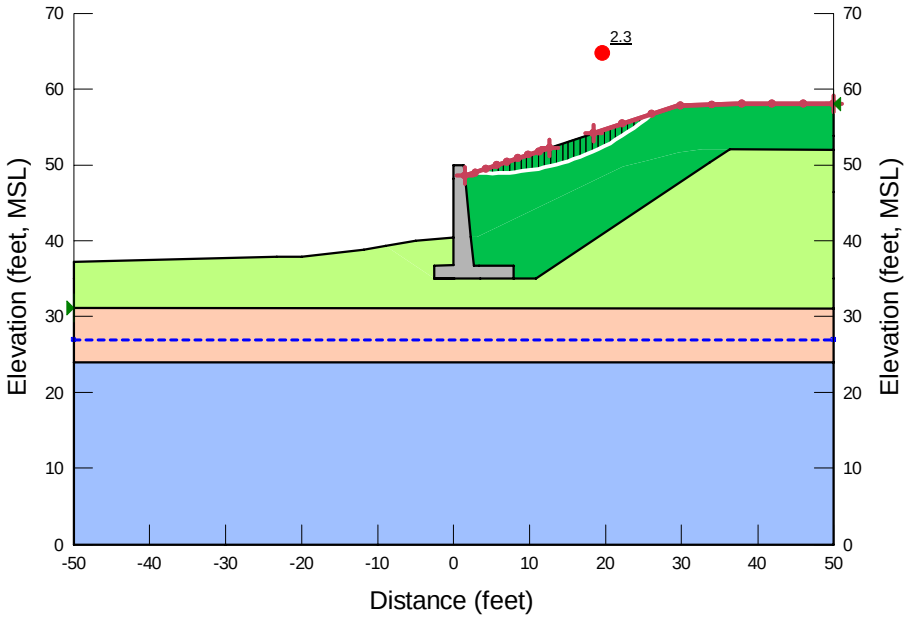
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36



Name: STA 0+70_NE Wall=15 ft_3H:1V_Static_Fill Slope

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.3

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36

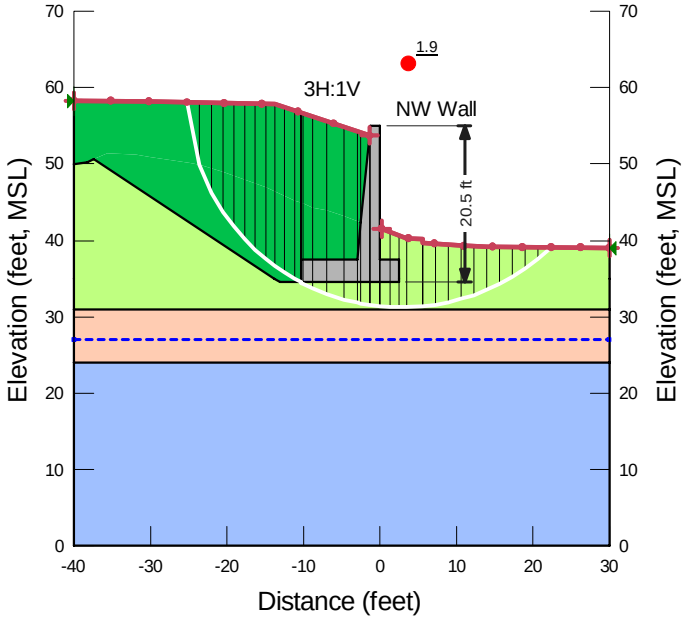


F.7 Northwest Wall Stability Analysis Results – Concrete Wall

Name: STA 0+50_NW Wall=20.5 ft_3H:1V_Static

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 1.9

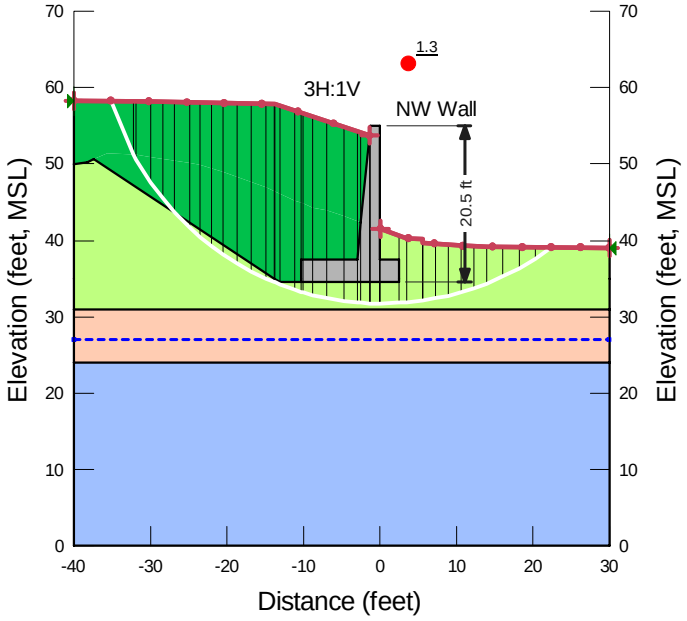
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36



Name: STA 0+50_NW Wall=20.5 ft_3H:1V_Pseudostatic

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.25
Factor of Safety: 1.3

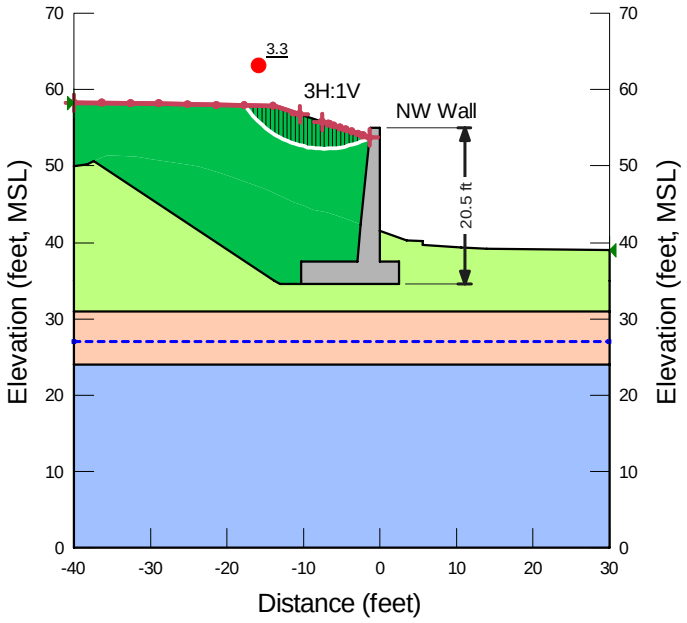
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36



Name: STA 0+50_NW Wall=20.5 ft_3H:1V_Static_Fill Slope

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 3.3

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36

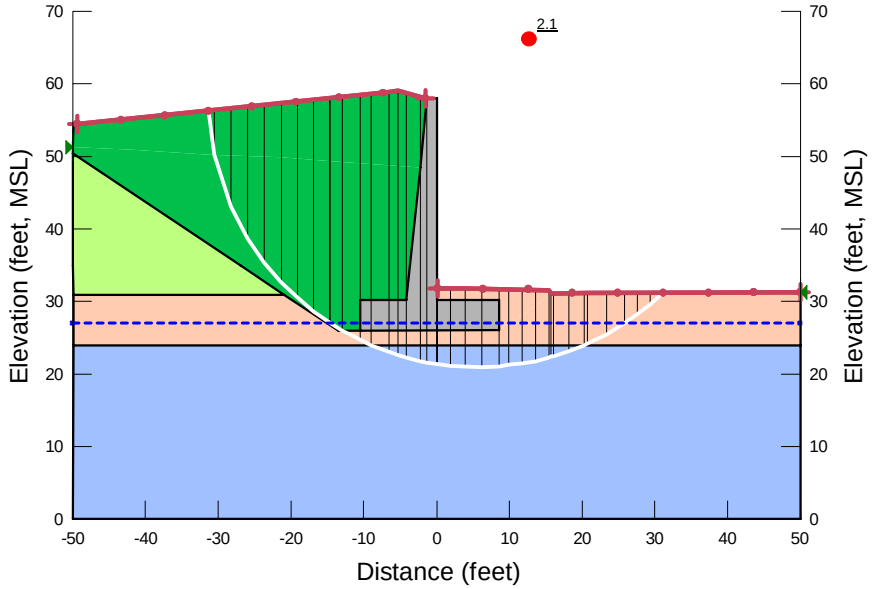


F.8 Southeast Wall Stability Analysis Results – Concrete Wall

Name: STA 0+01_SE Wall=32 ft_3H:1V_Static

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.1

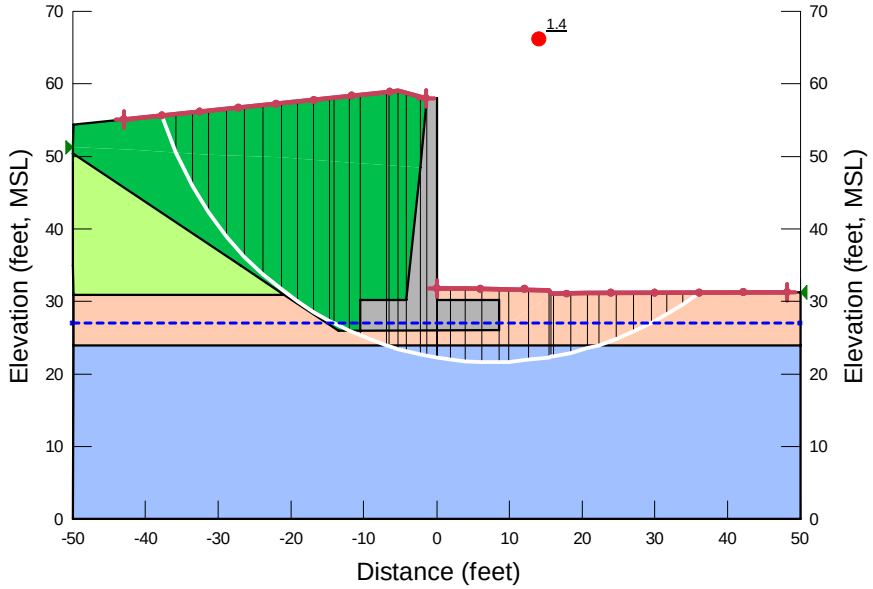
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36



Name: STA 0+01_SE Wall=32 ft_3H:1V_Pseudostatic

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.25
Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36

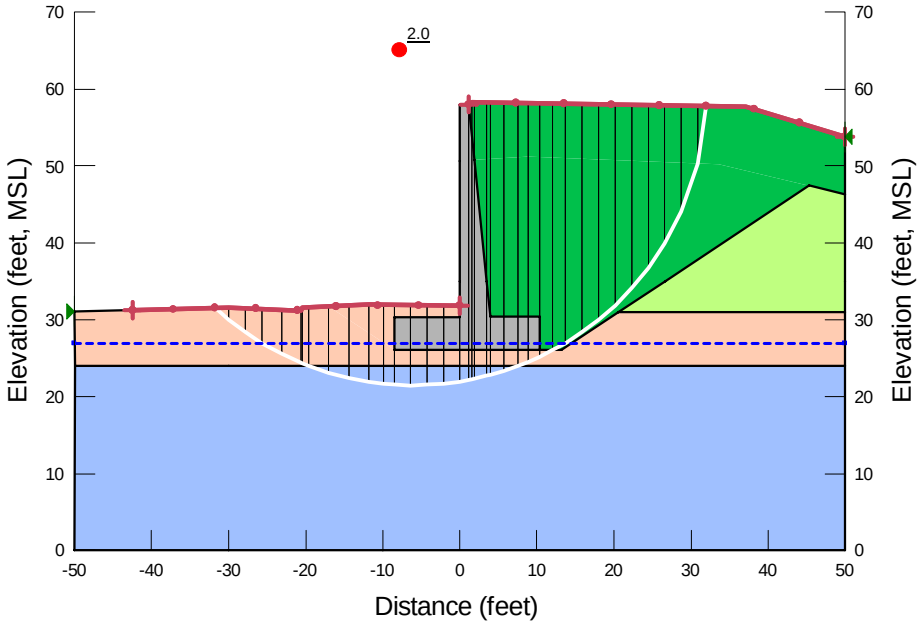


F.9 Southwest Wall Stability Analysis Results – Concrete Wall

Name: STA 0+00_SW Wall=32 ft_3H:1V_Static

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0
Factor of Safety: 2.0

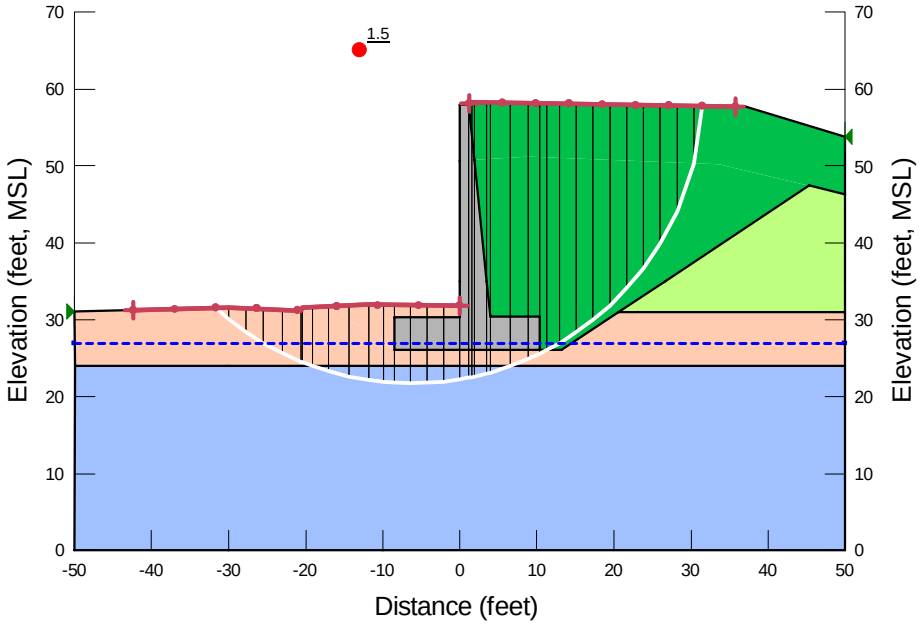
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36



Name: STA 0+00_SW Wall=32 ft_3H:1V_Pseudostatic

File Name: Woodinville_Trestle_Retaining Wall Stability.gsz
Analysis Type: Spencer
Horz Seismic Coef.: 0.25
Factor of Safety: 1.5

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense to Very Dense Sand with Gravel and Silt	130	0	38
	Existing Fill	120	0	30
	Medium Dense to Dense Sand with Gravel and Silt	125	0	35
	Retaining Wall	145		
	WSDOT Select Borrow	130	0	36



APPENDIX G DRILLED SHAFT AXIAL CAPACITY FOR EAST AND WEST BRIDGE ABUTMENTS

G.1 West Abutment Axial Capacity Curves (No Downdrag)

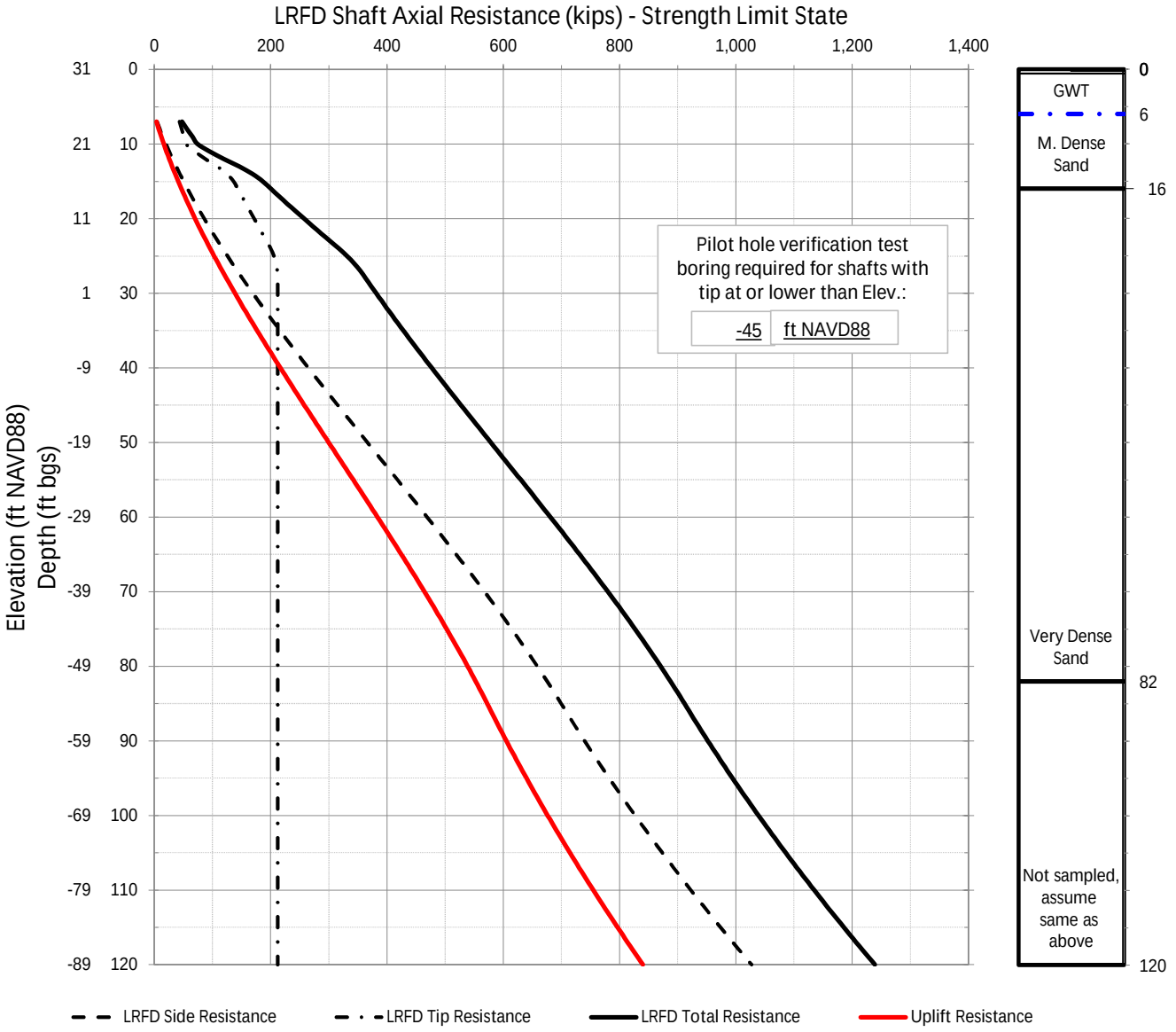
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

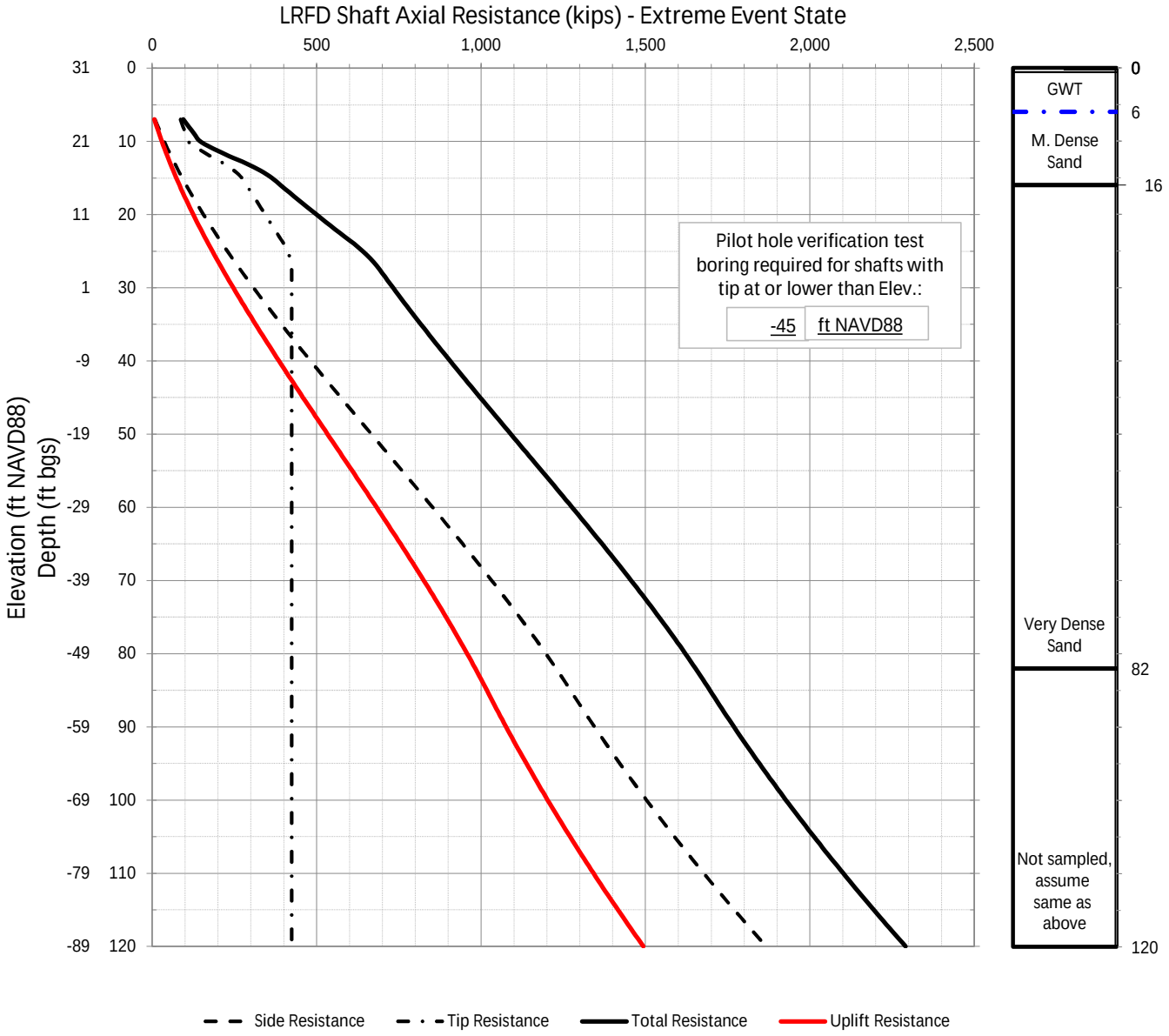
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

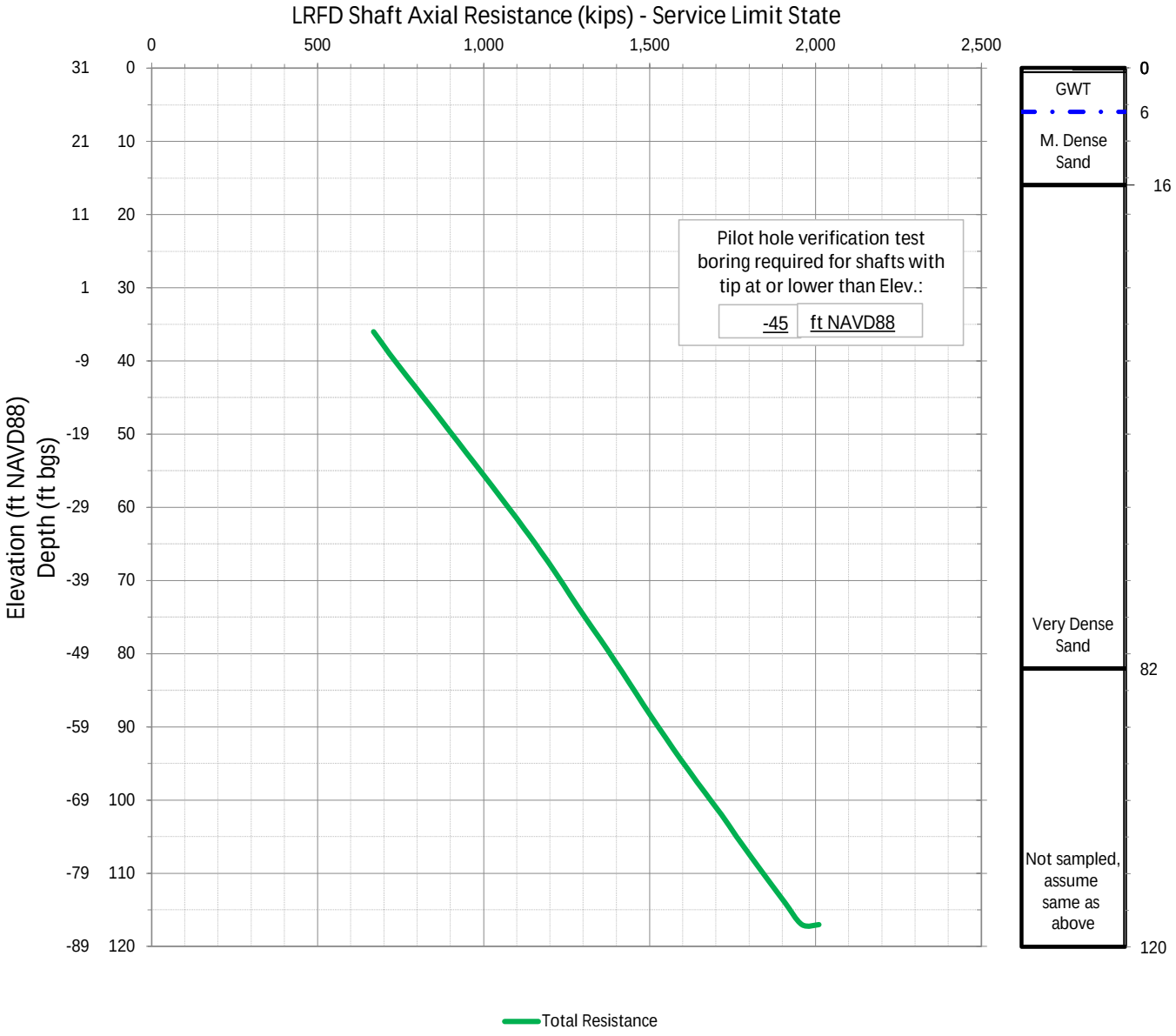
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

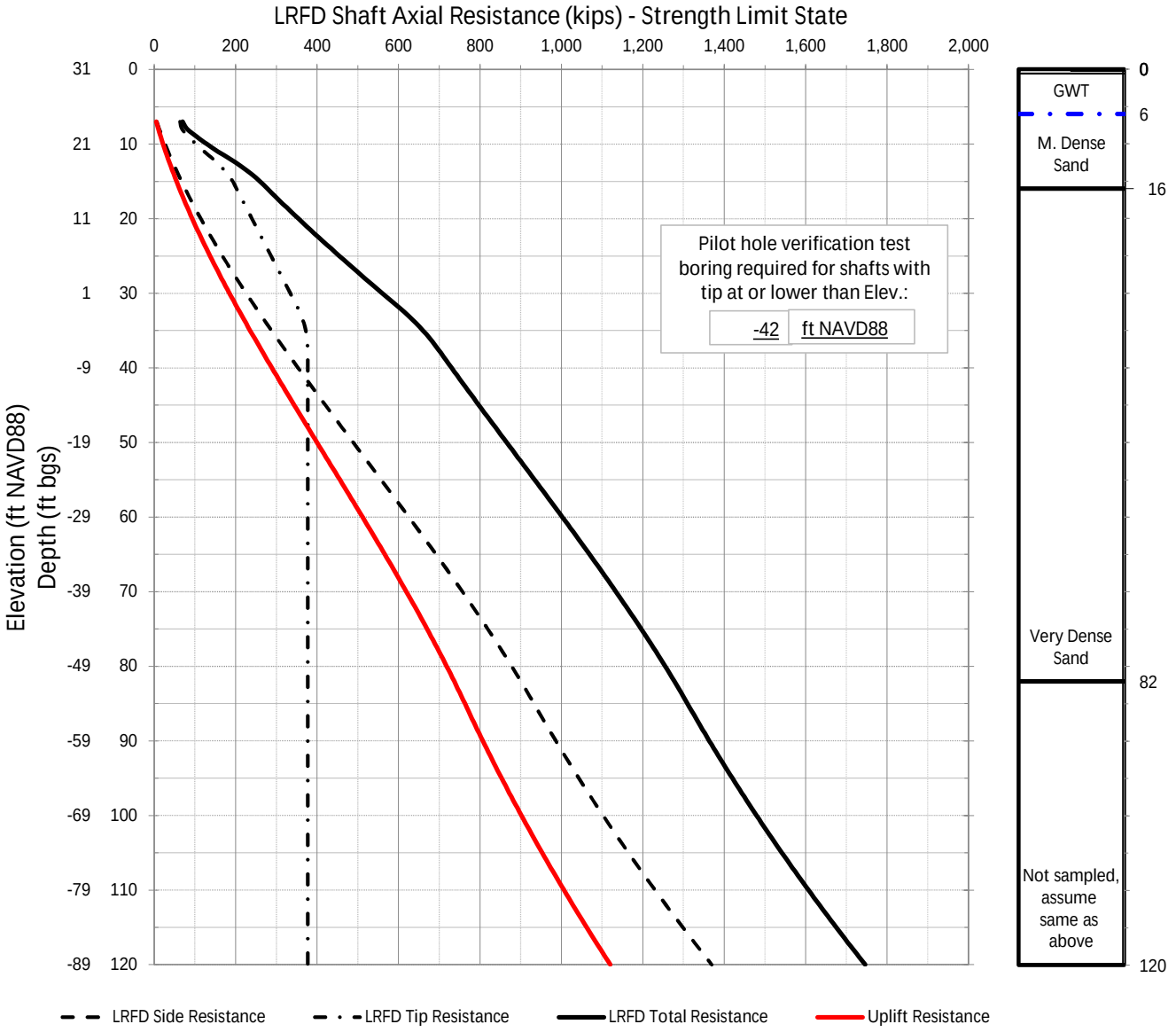
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

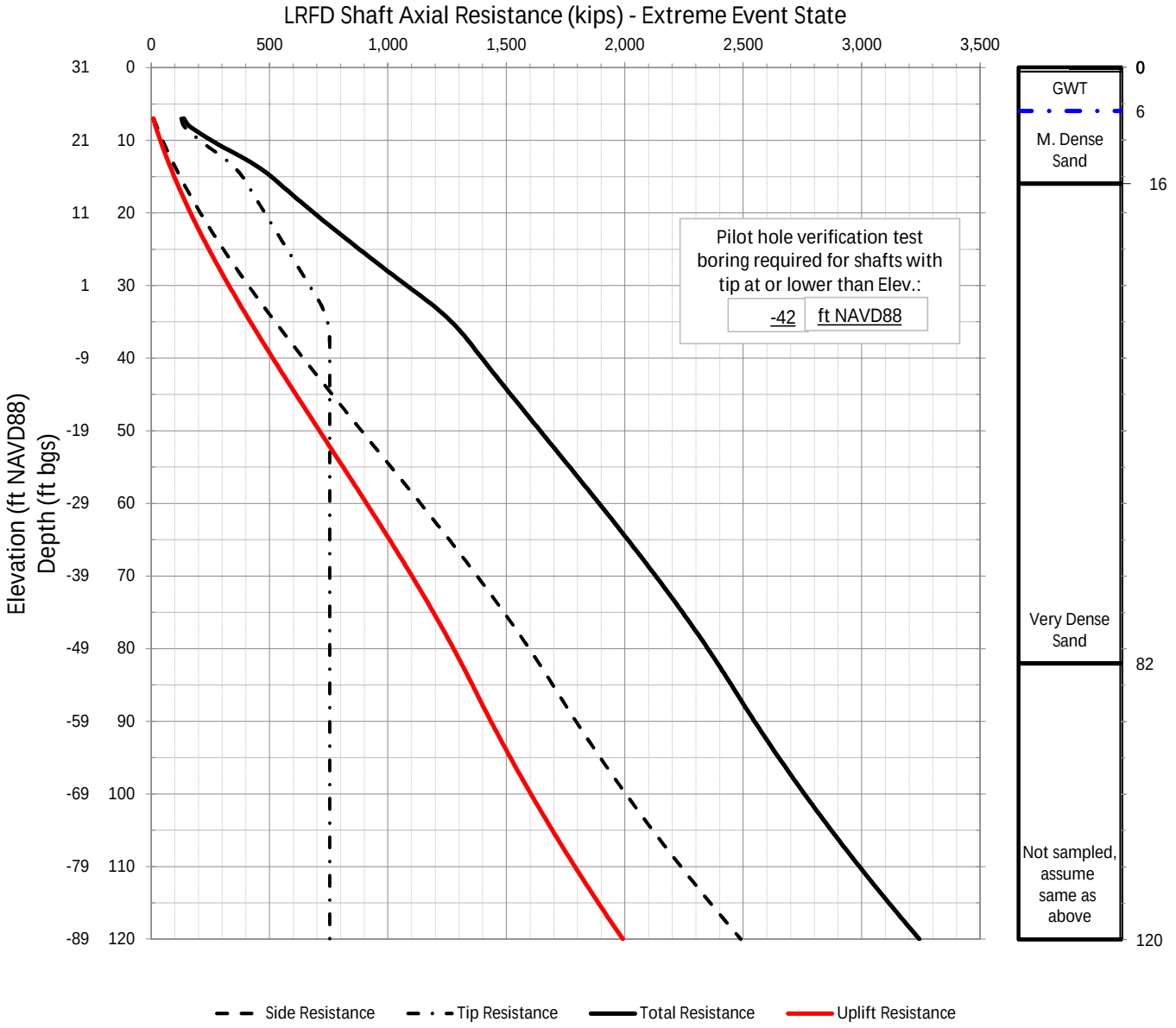
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

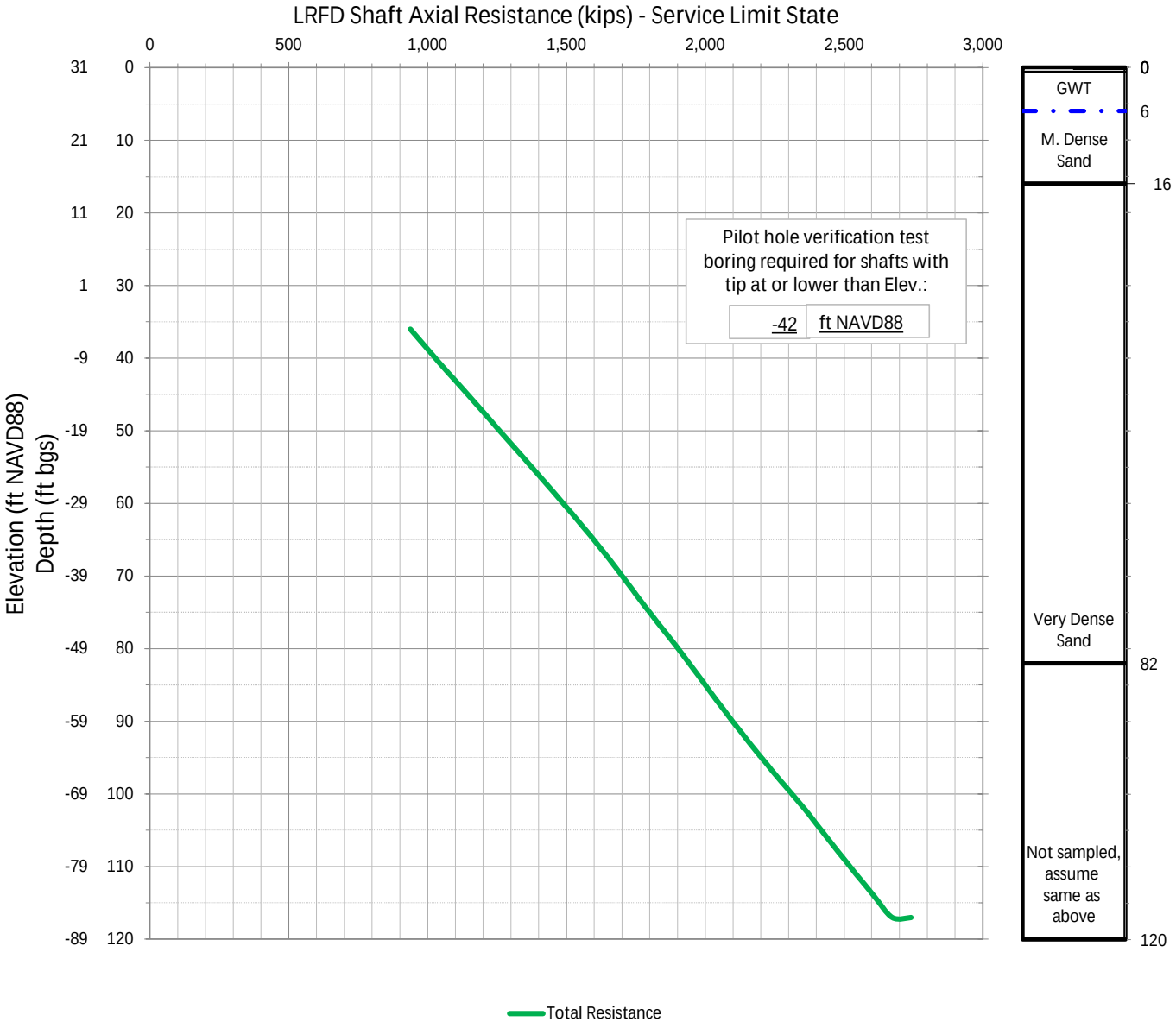
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

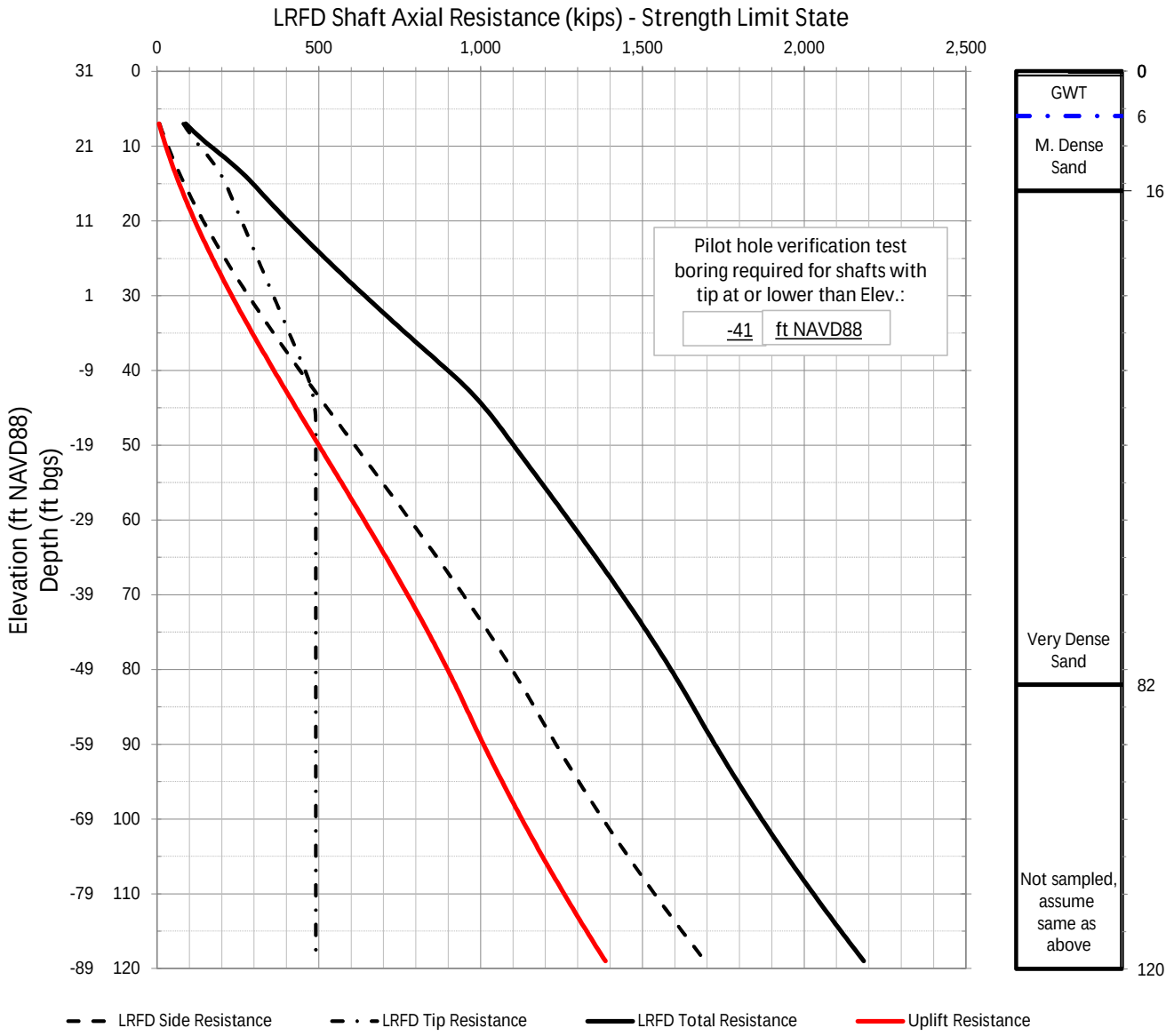
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

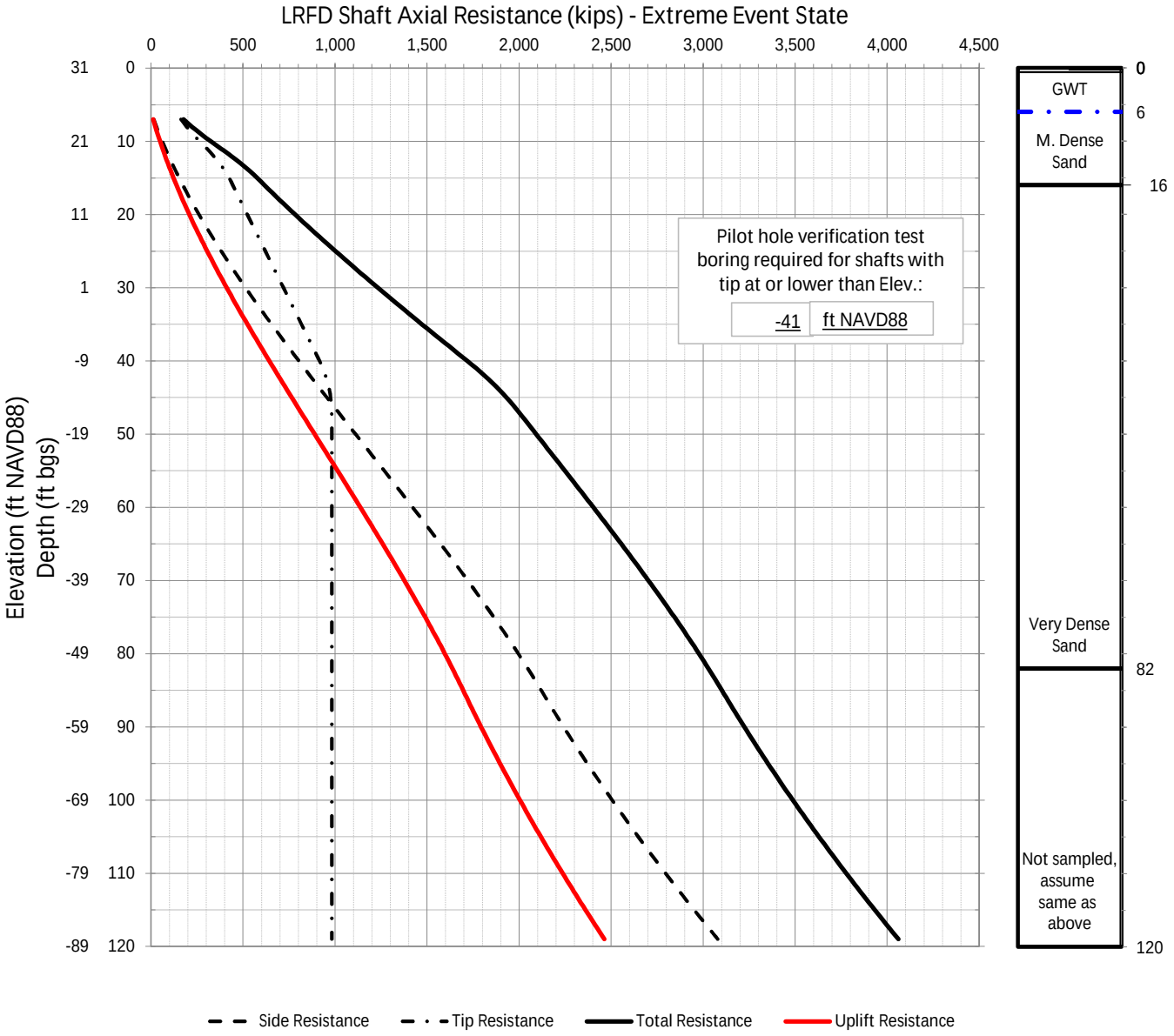
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

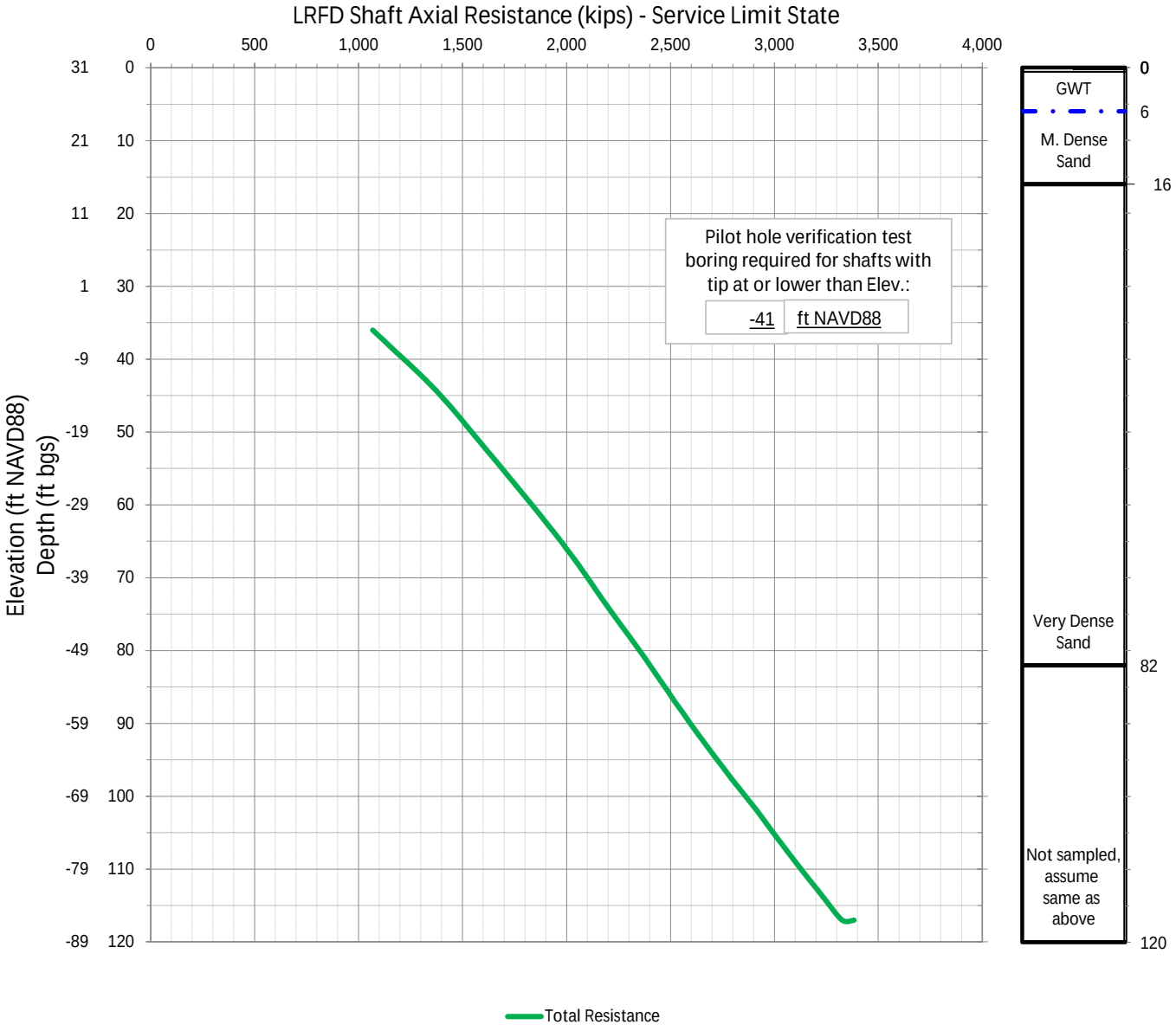
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

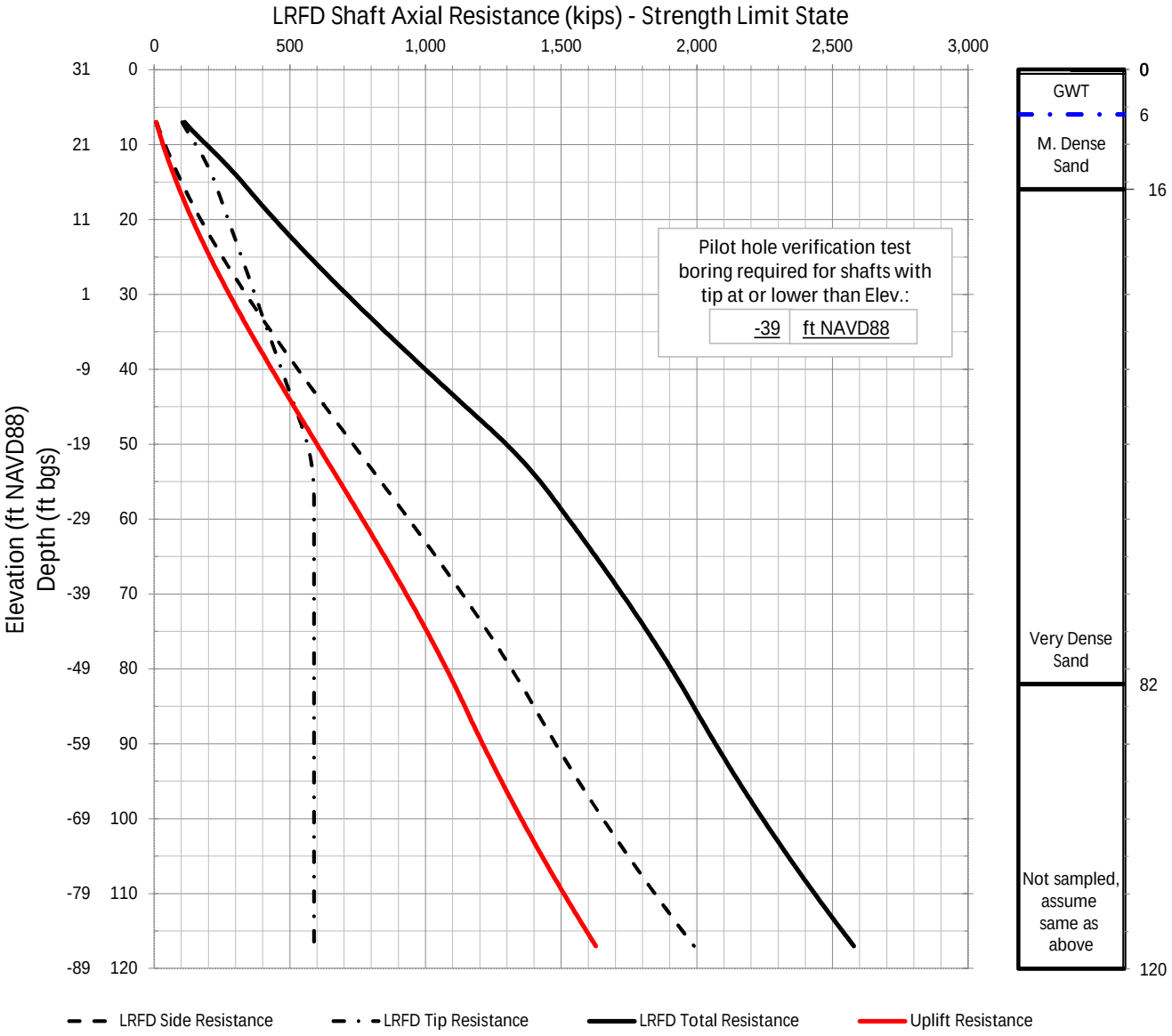
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

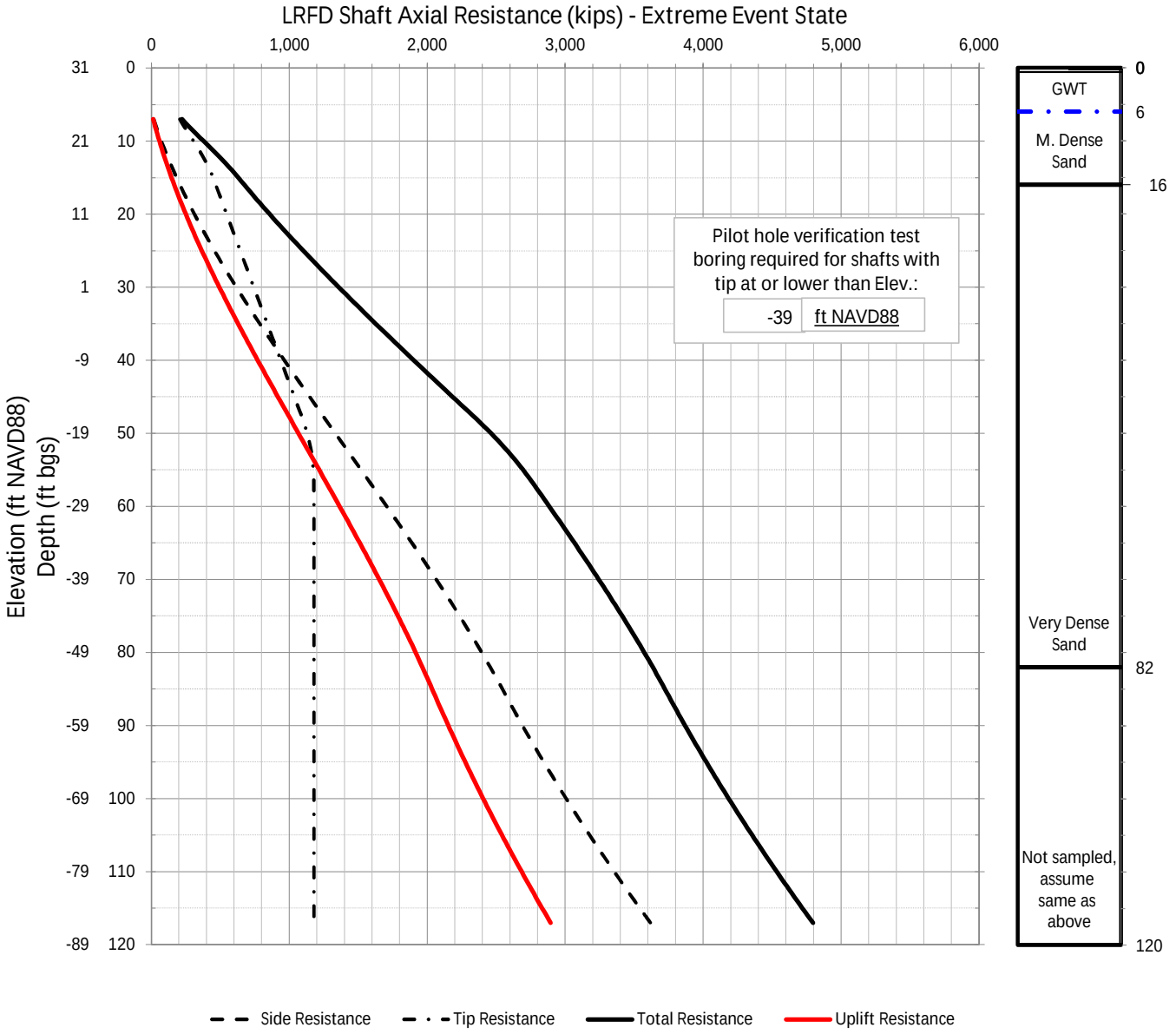
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

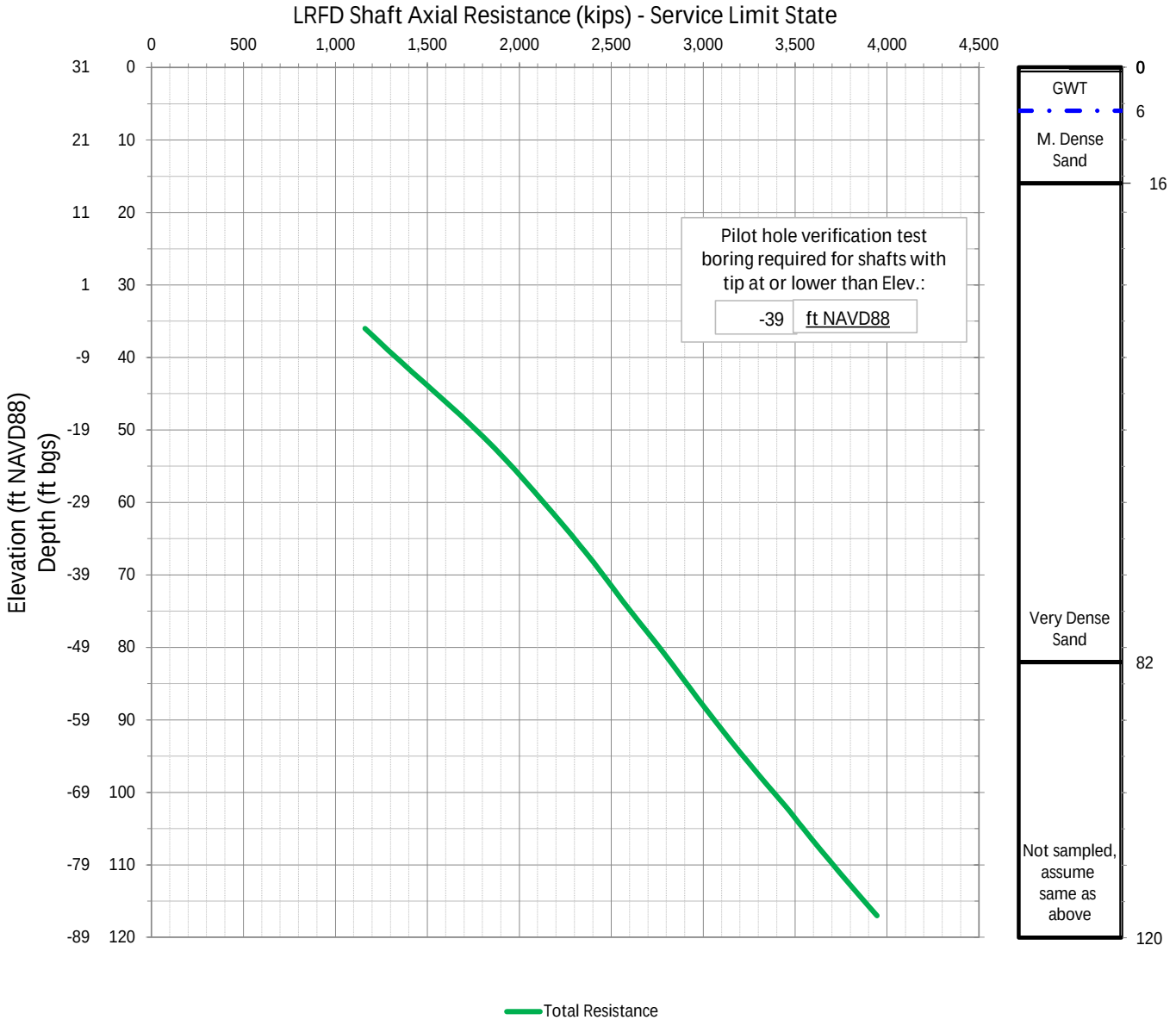
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
6.0 ft Diameter Drilled Shaft

Performed By: L. Finnefrock
Date: 6/10/2025
Checked By: Y. Soltanpour
Date: 6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

G.2 East Abutment Axial Capacity Curves (No Downdrag)

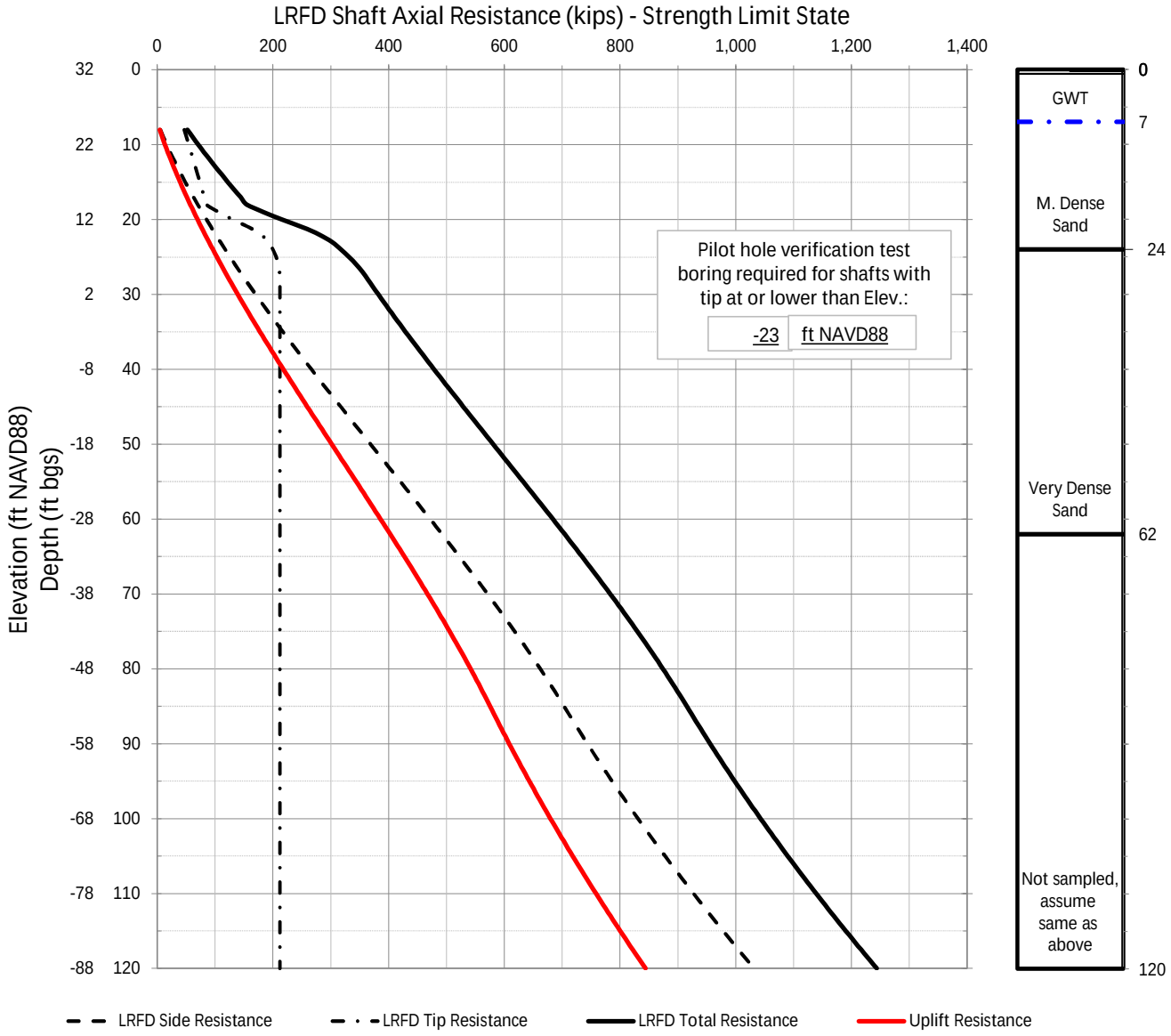
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

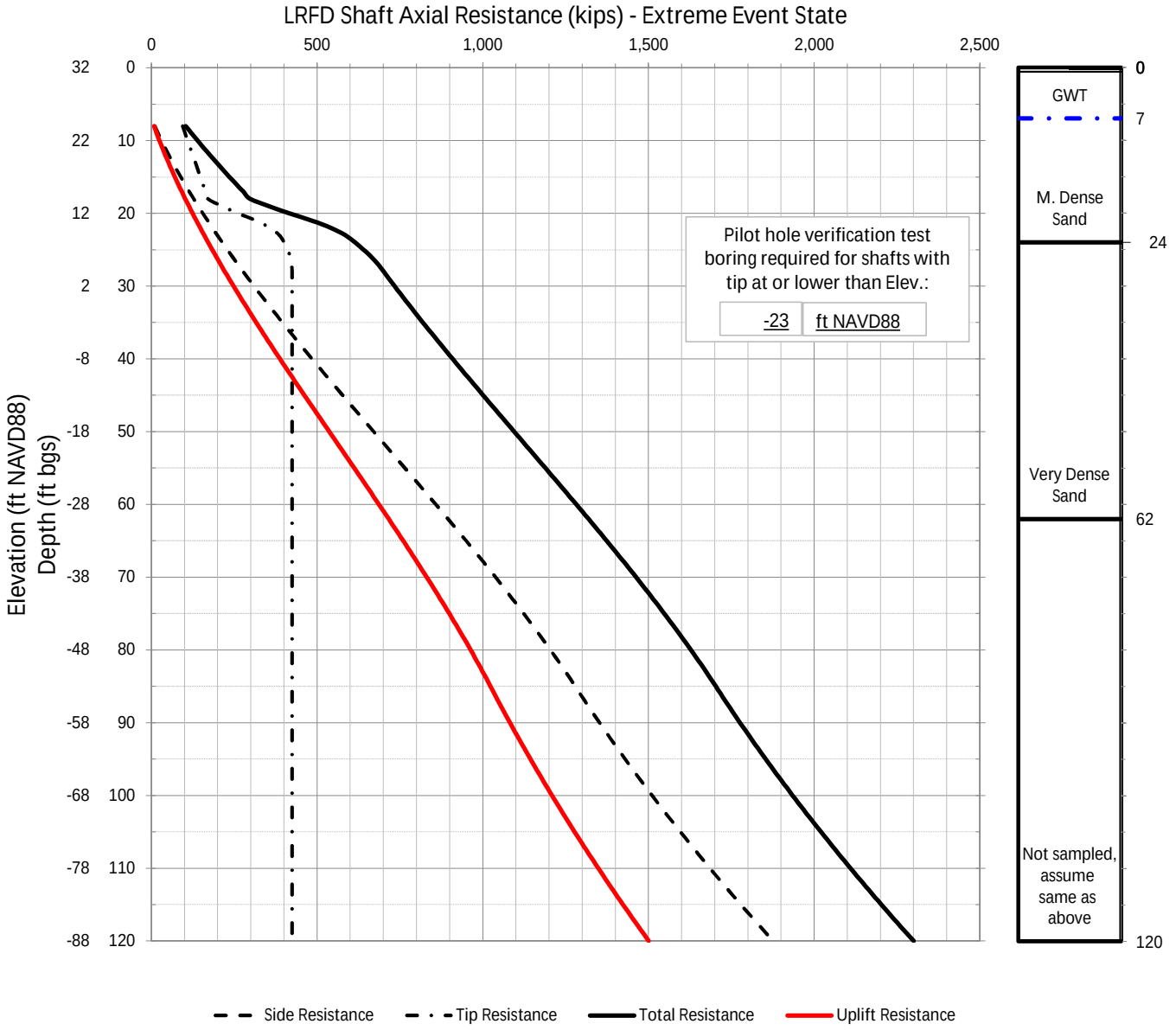
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
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5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

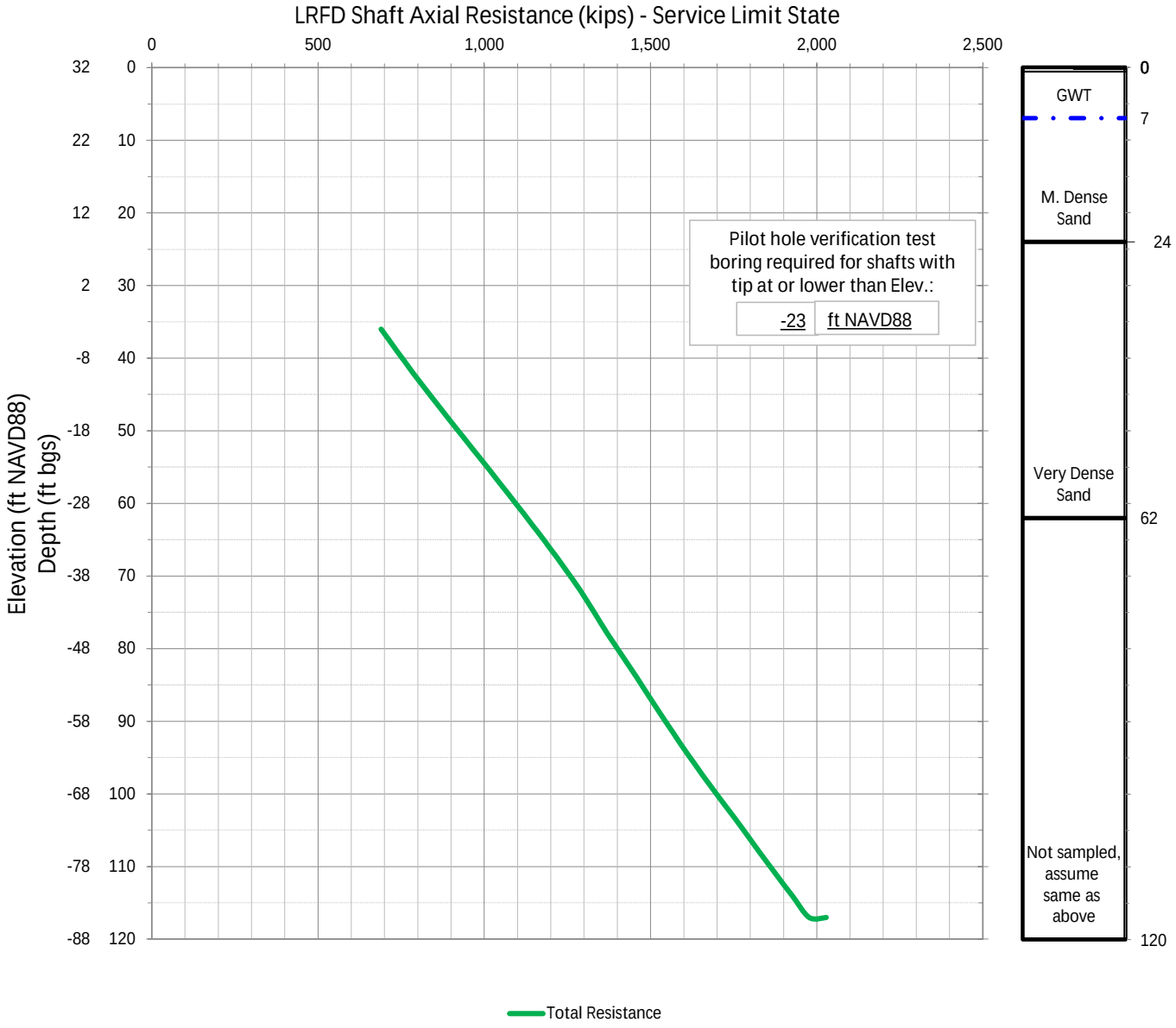
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **East Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

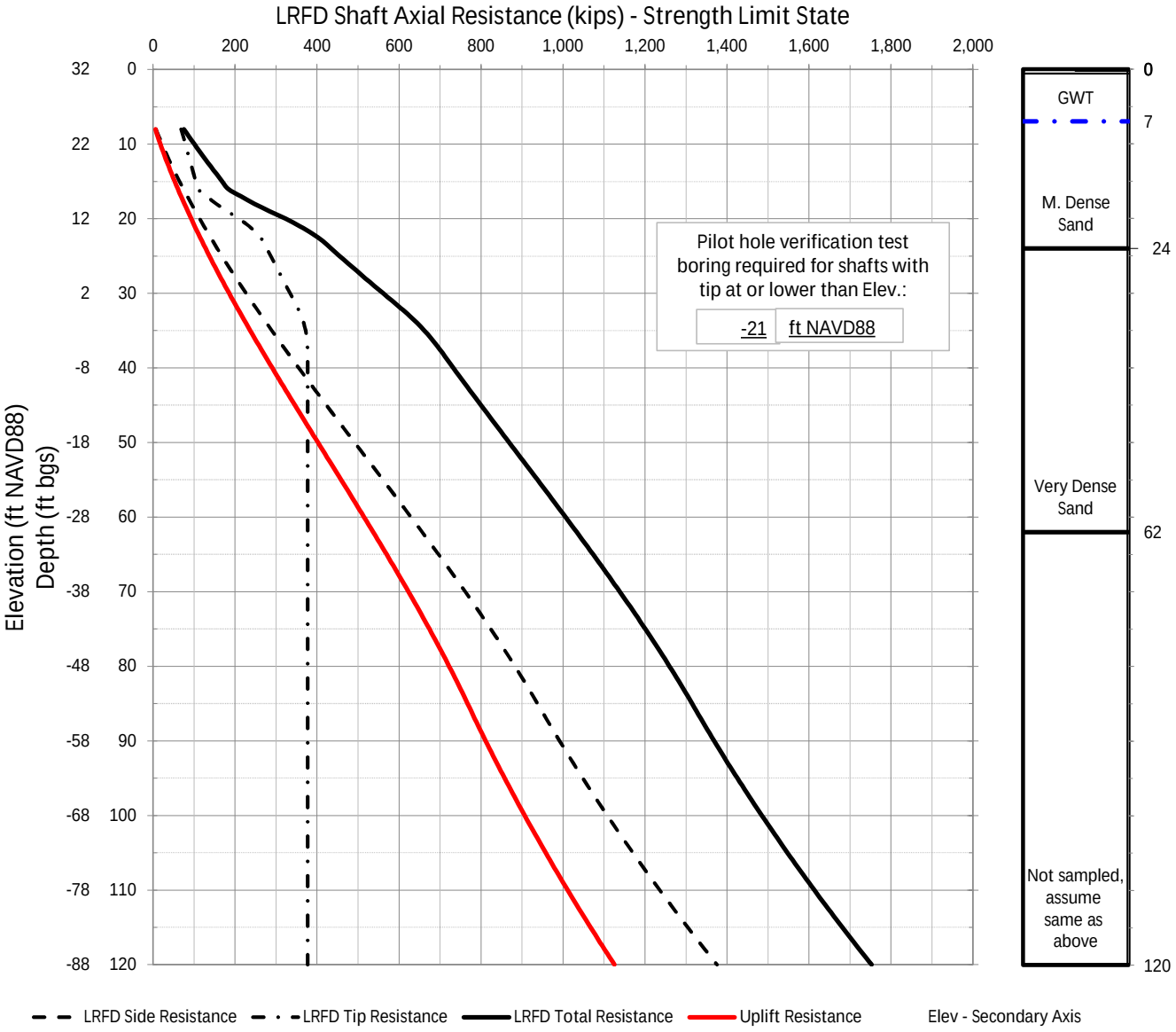
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

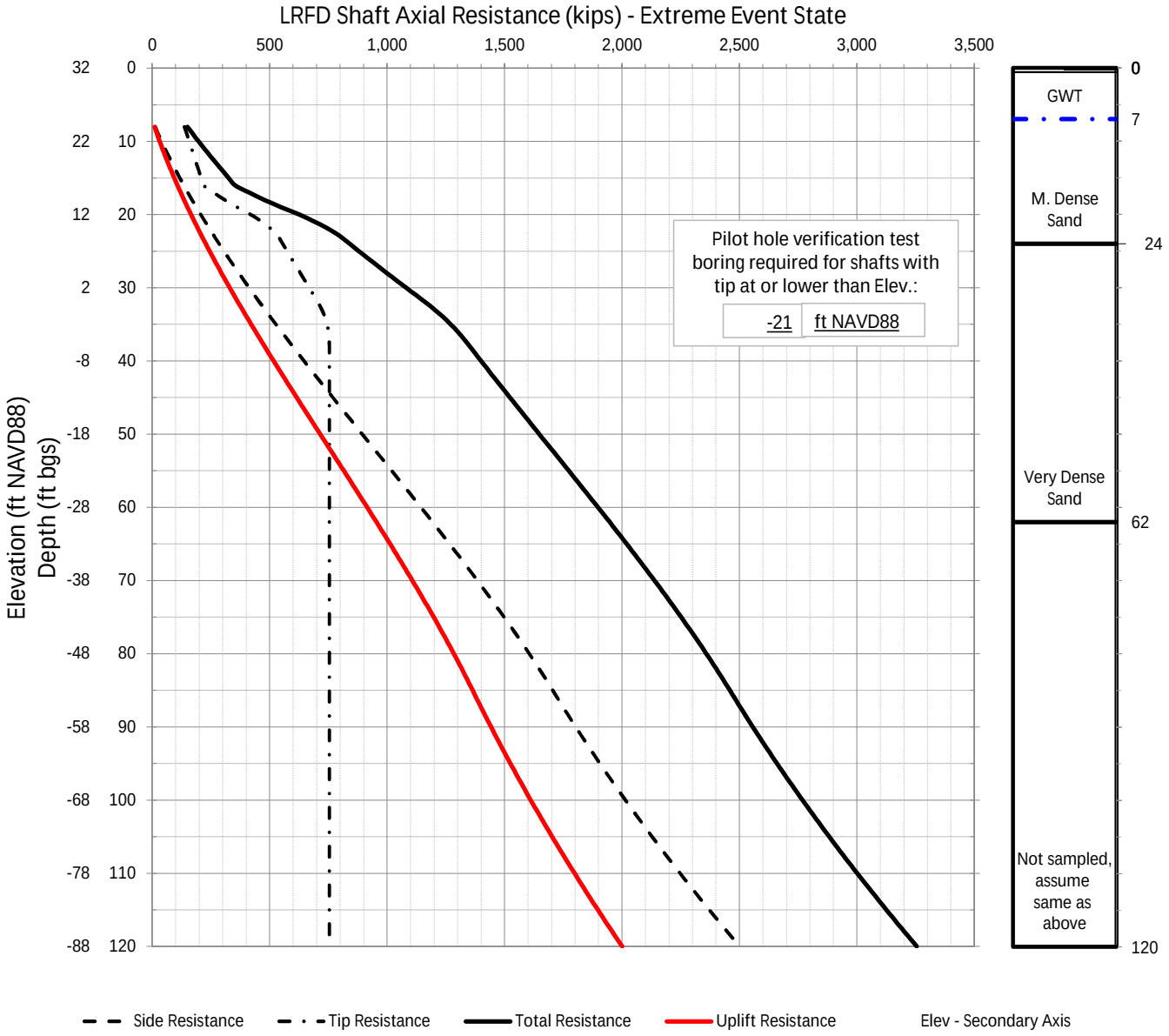
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

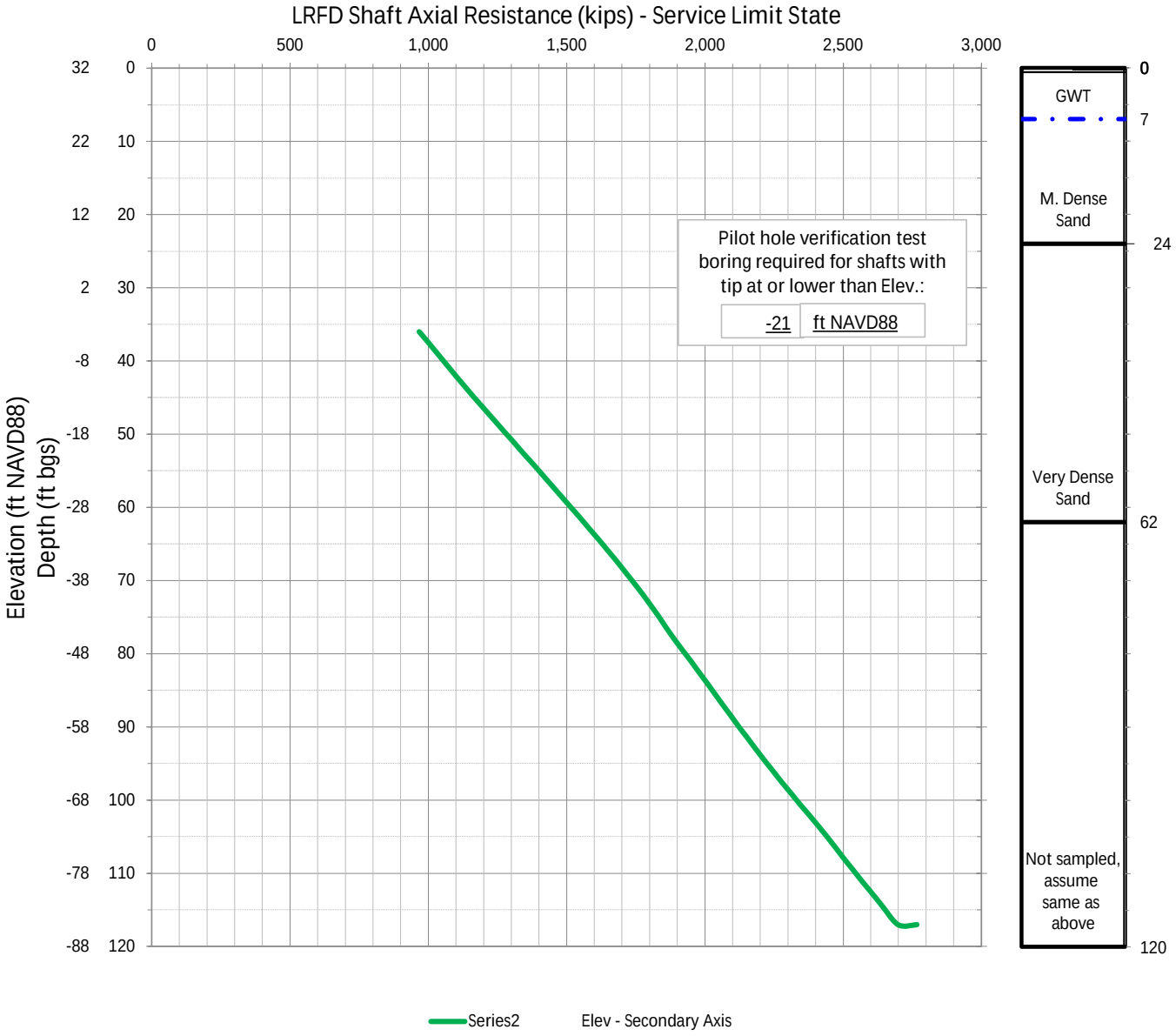
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **East Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

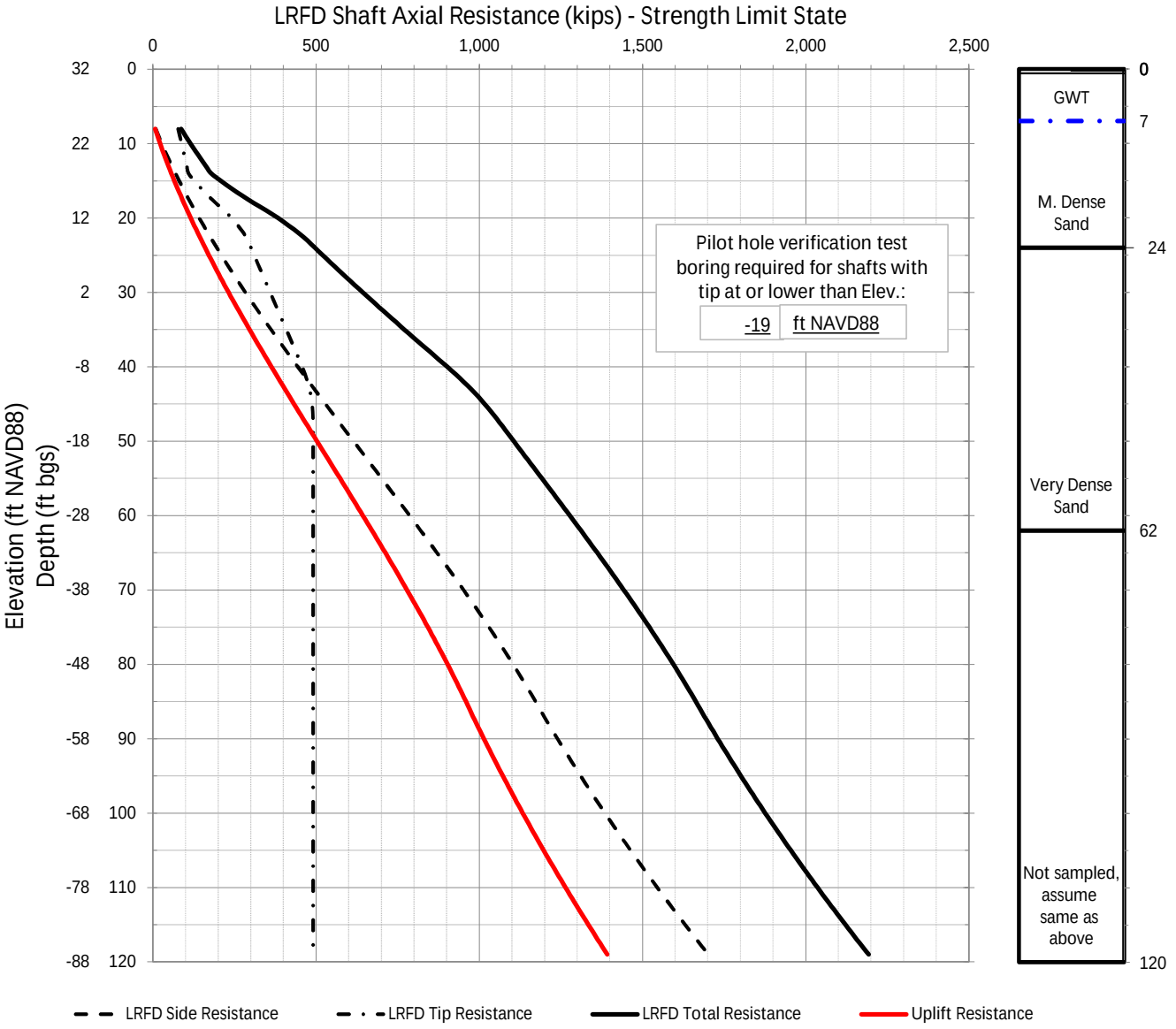
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

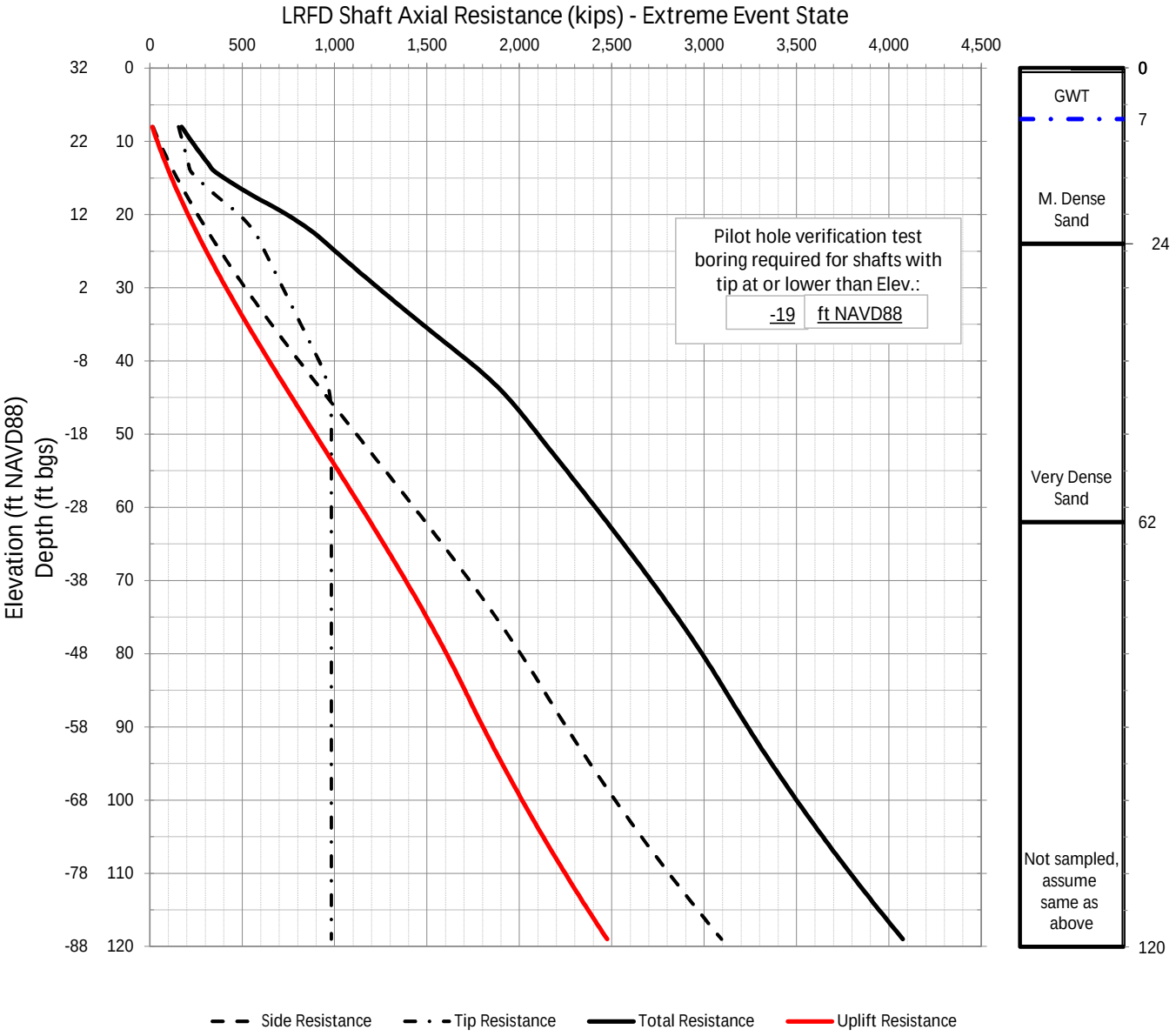
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

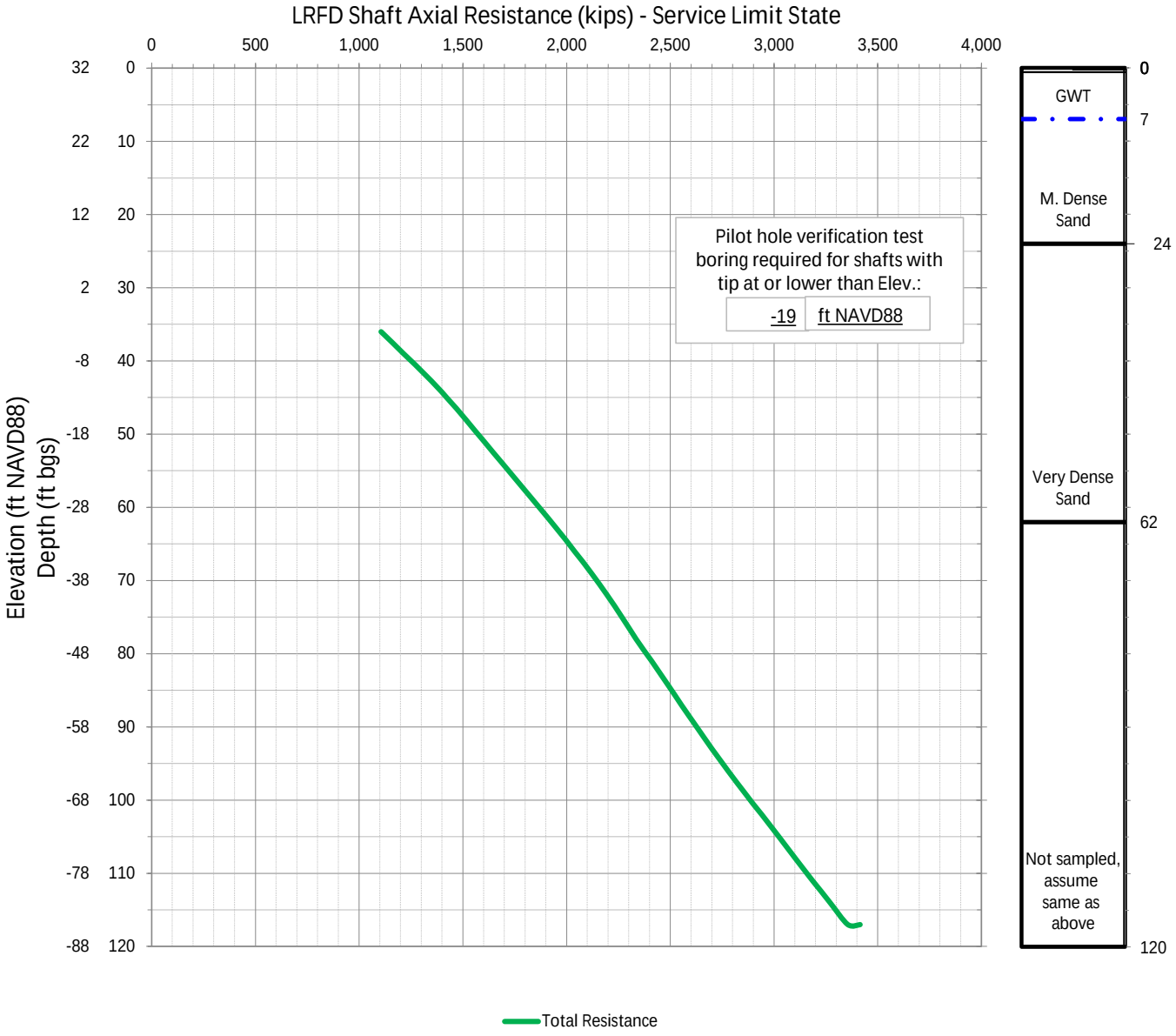
Subject: Axial Capacity - Service Limit State: **East Abutment**

5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88

Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

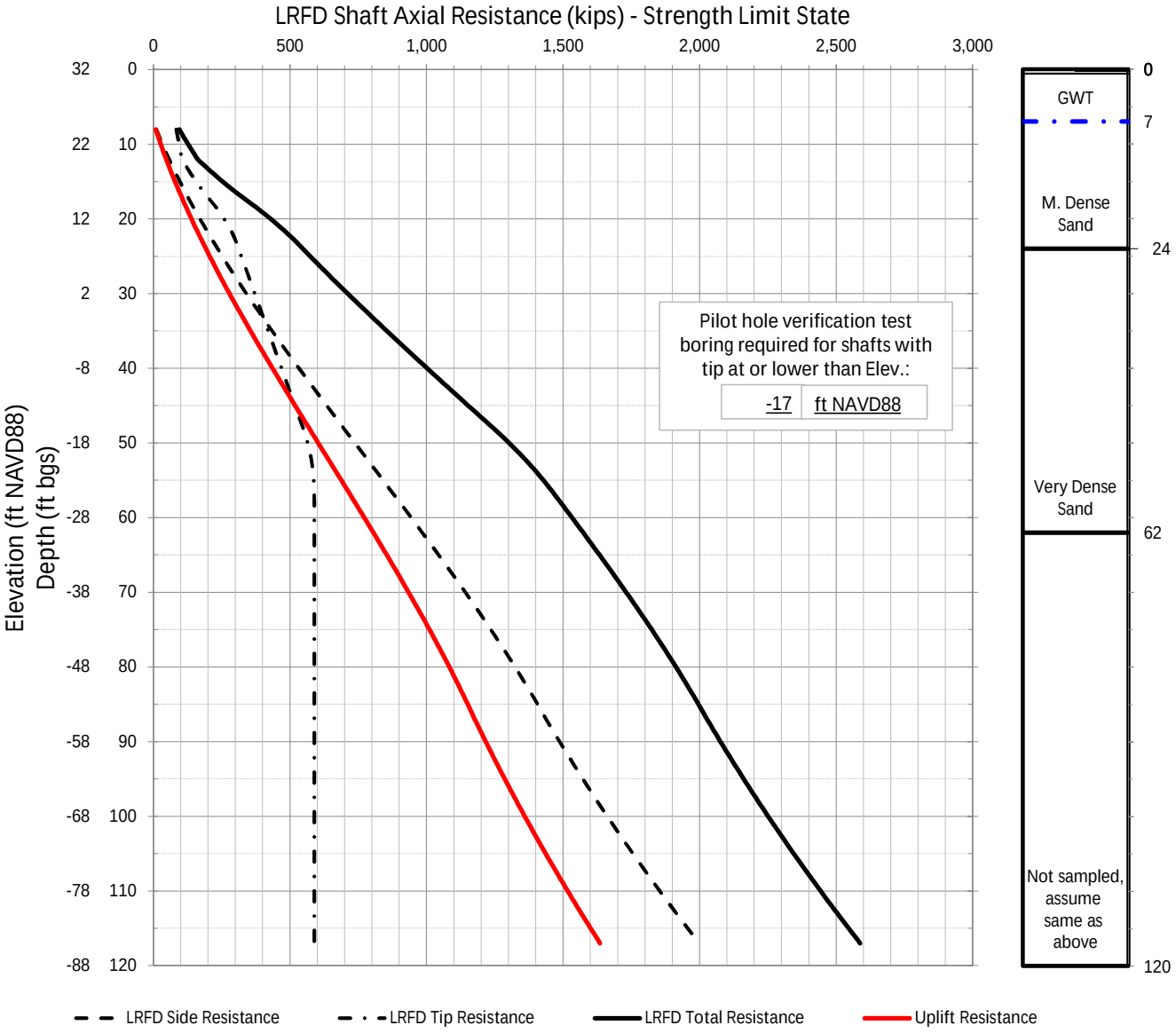
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

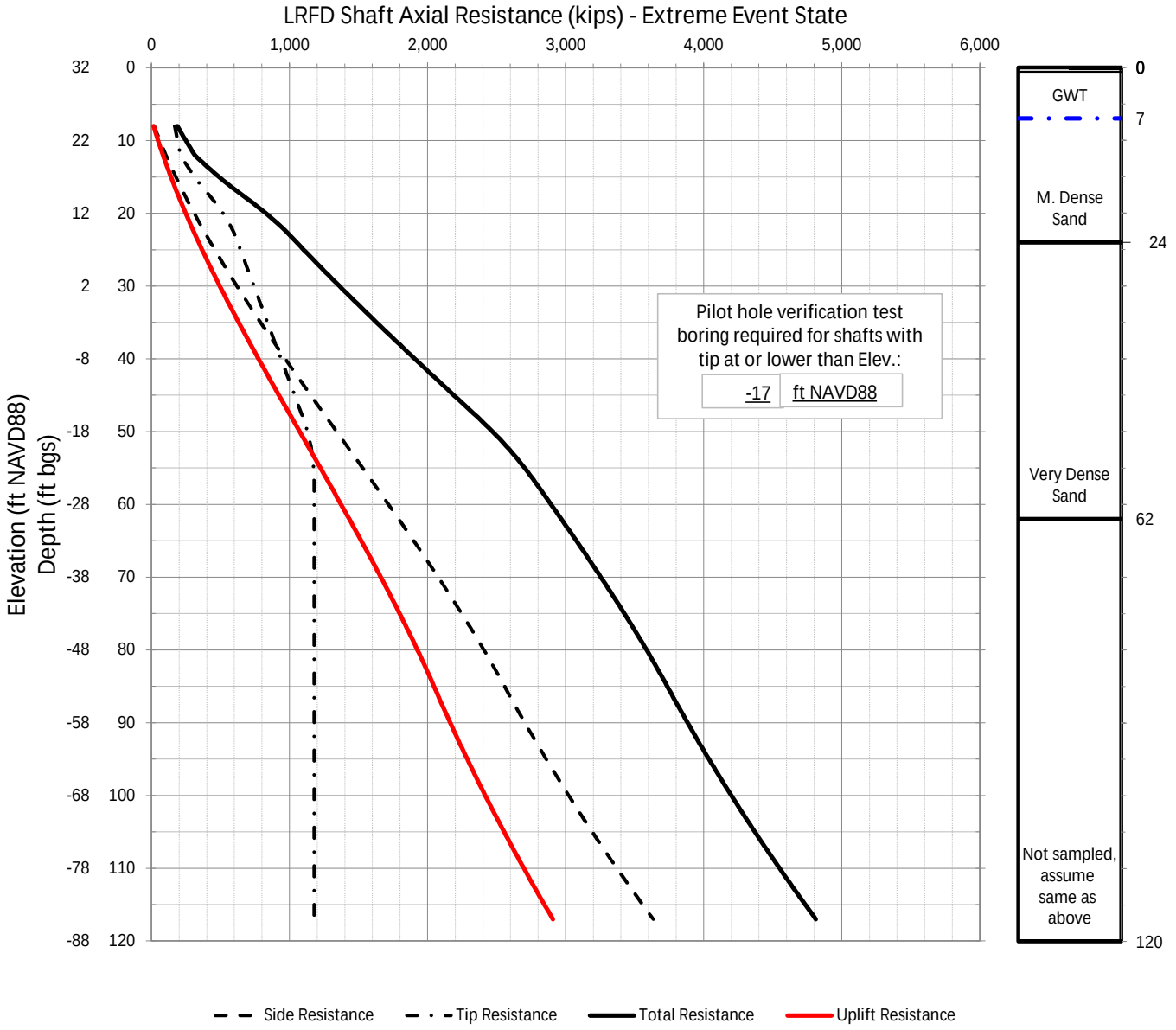
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
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AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

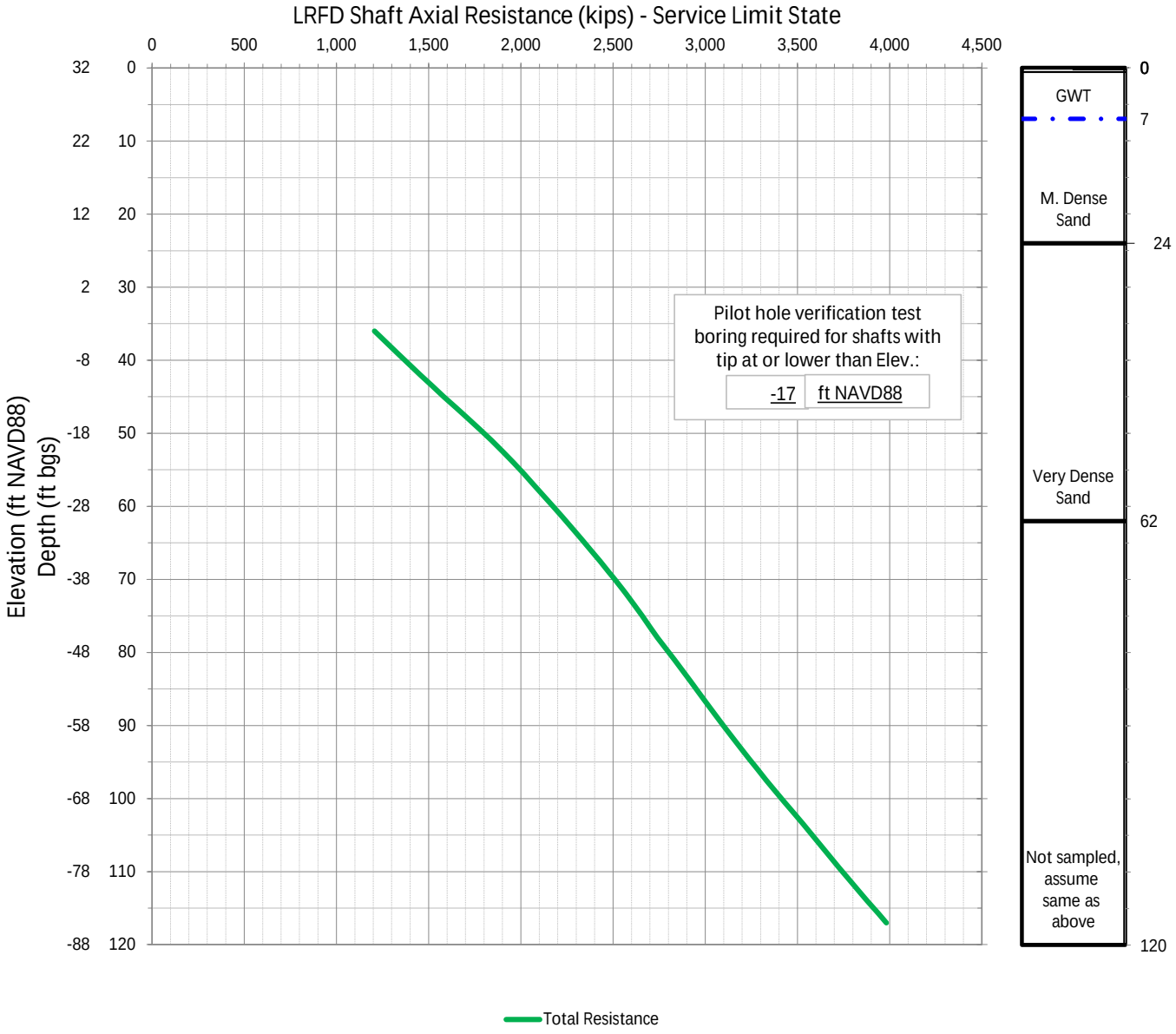
Subject: Axial Capacity - Service Limit State: **East Abutment**

6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88

Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

G.3 West Abutment Axial Capacity Curves (With Downdrag)

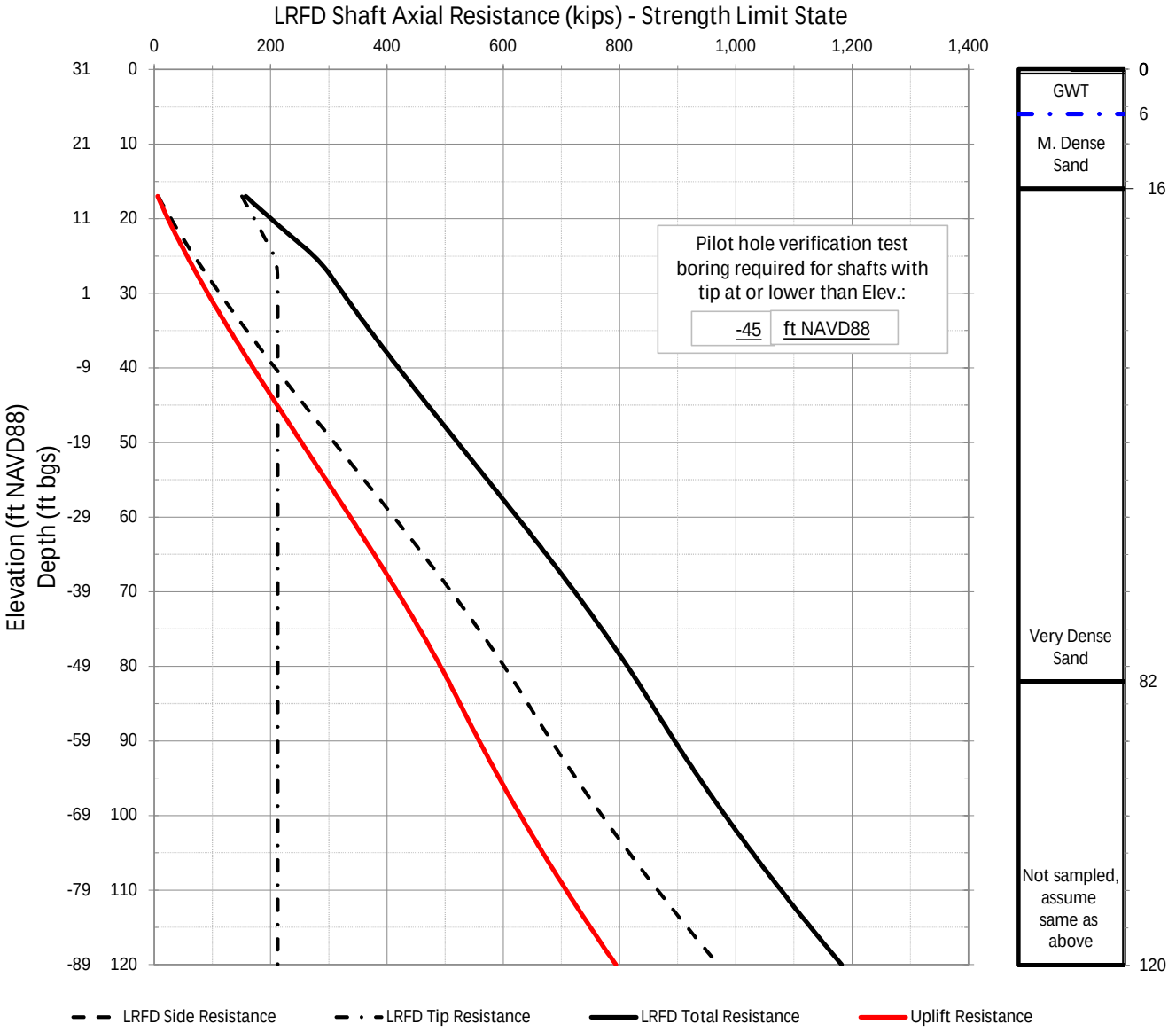
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
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6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

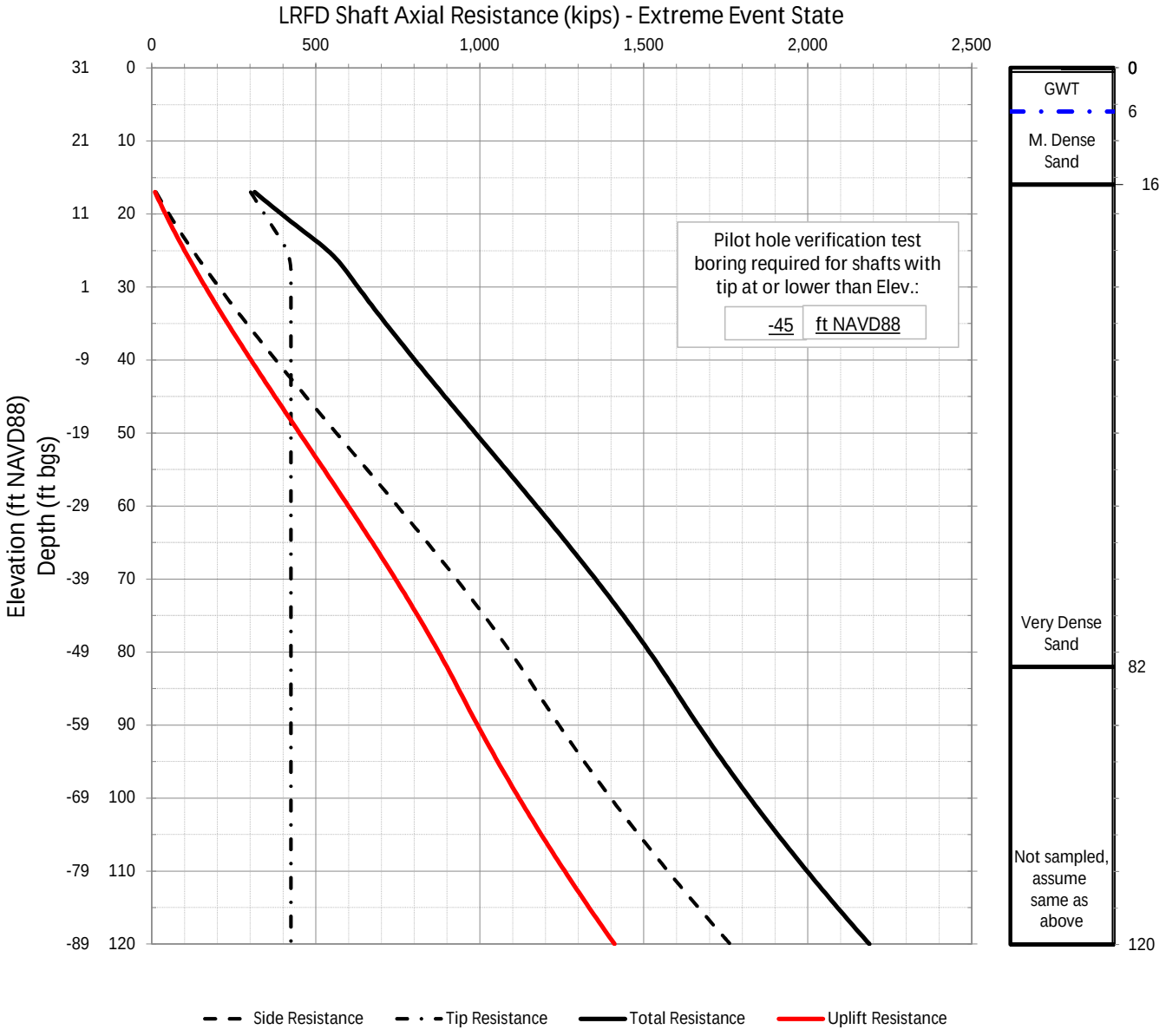
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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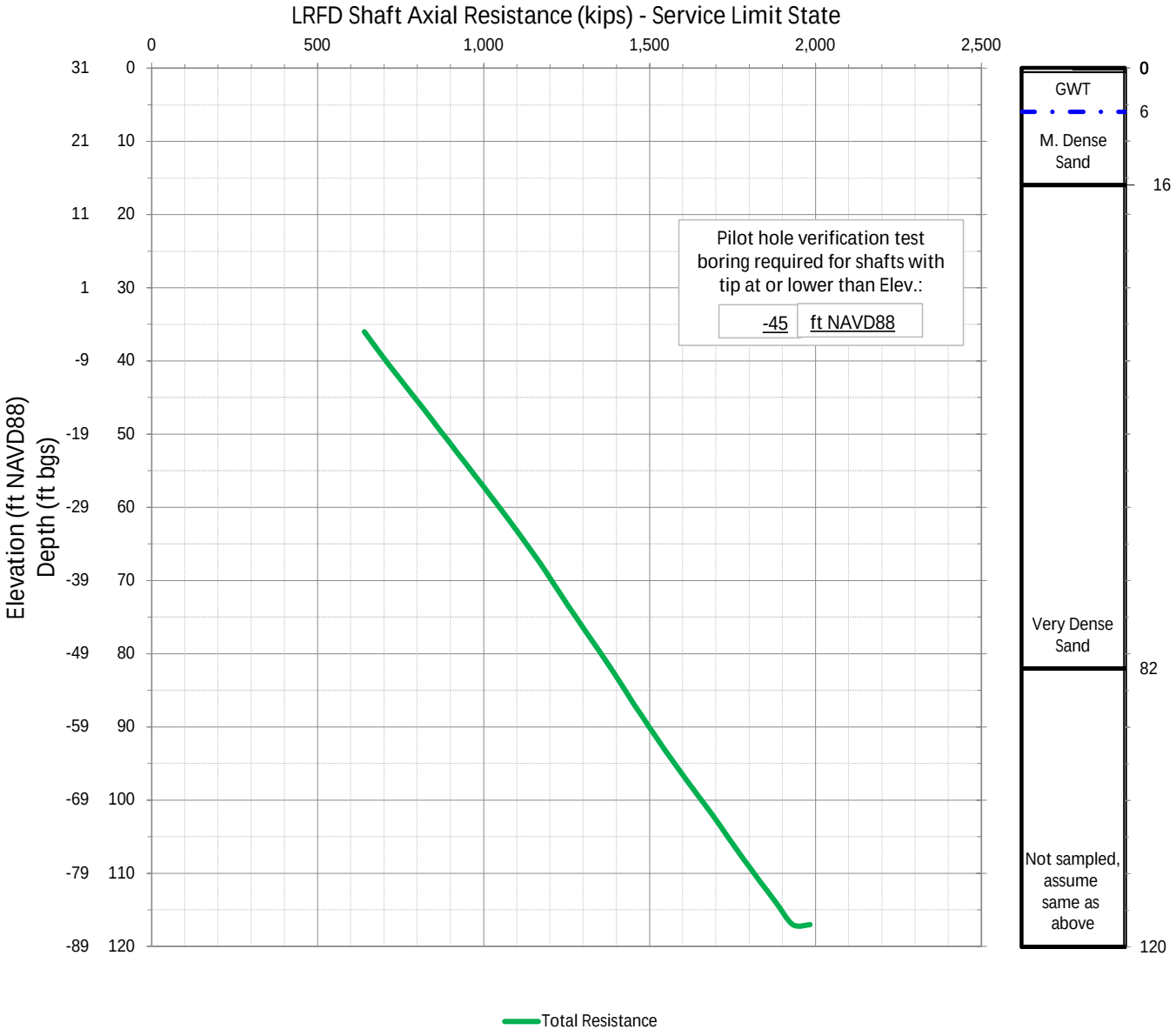
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

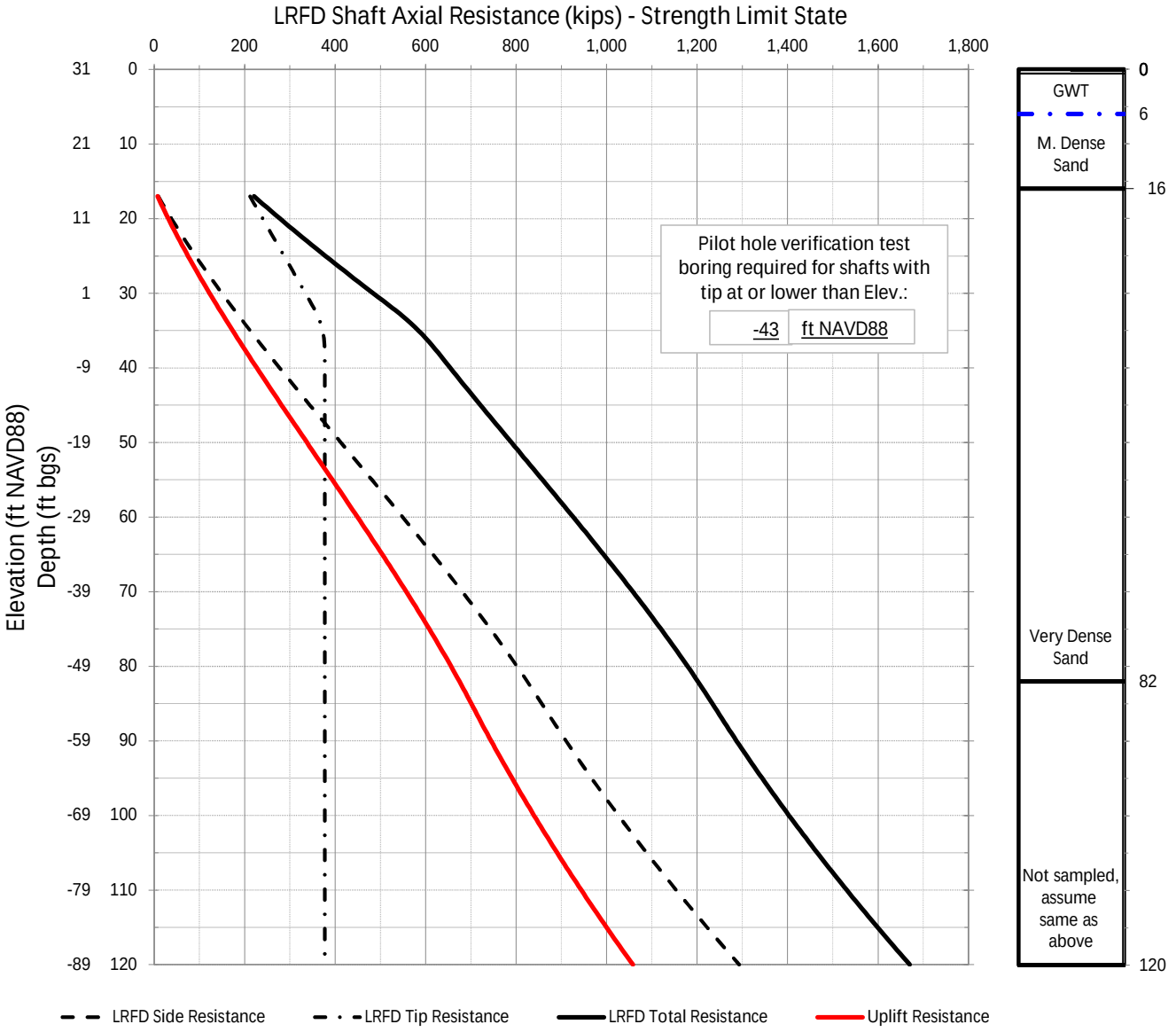
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
4.0 ft Diameter Drilled Shaft

Performed By: L. Finnefrock
Date: 6/10/2025
Checked By: Y. Soltanpour
Date: 6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

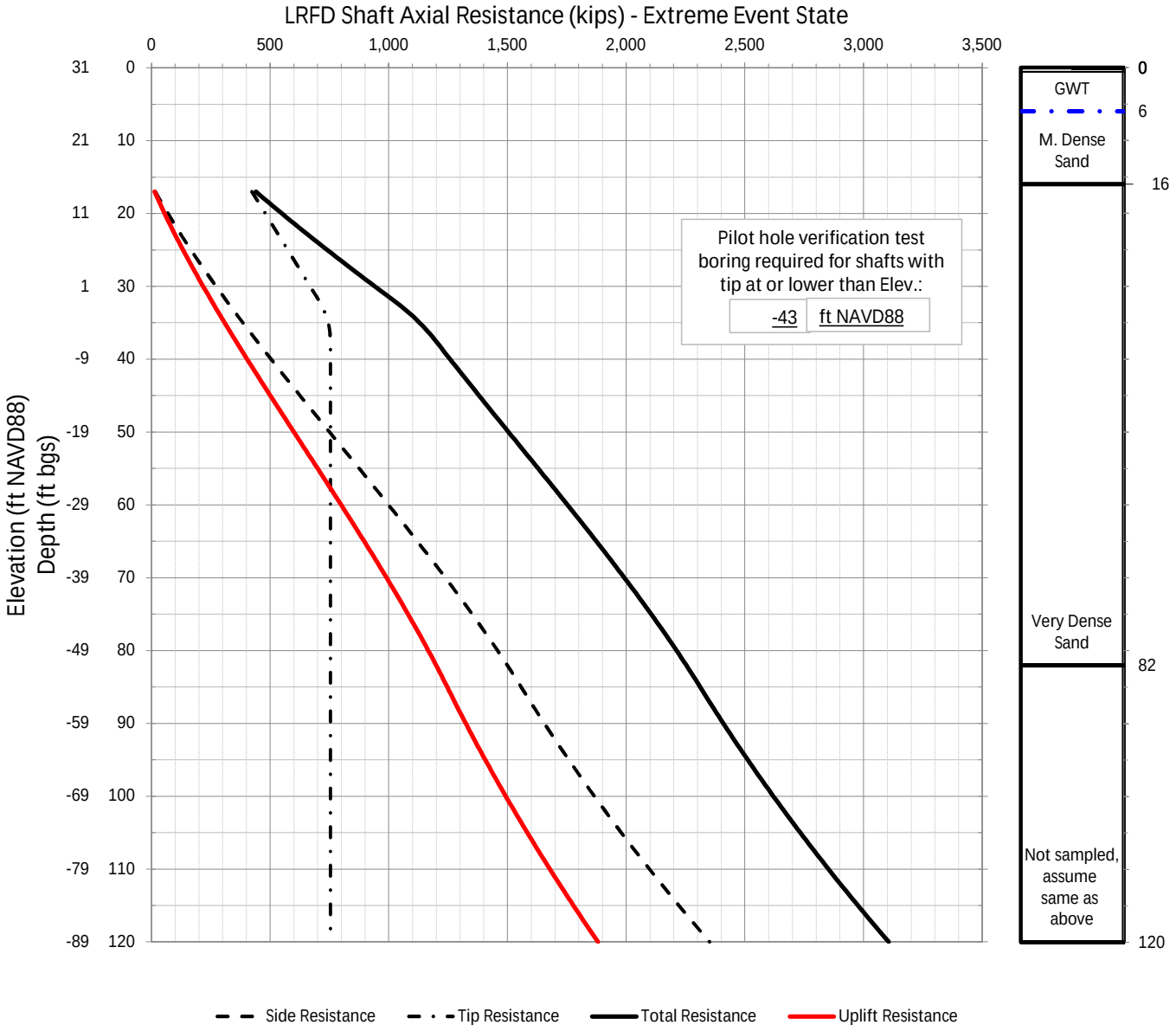
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
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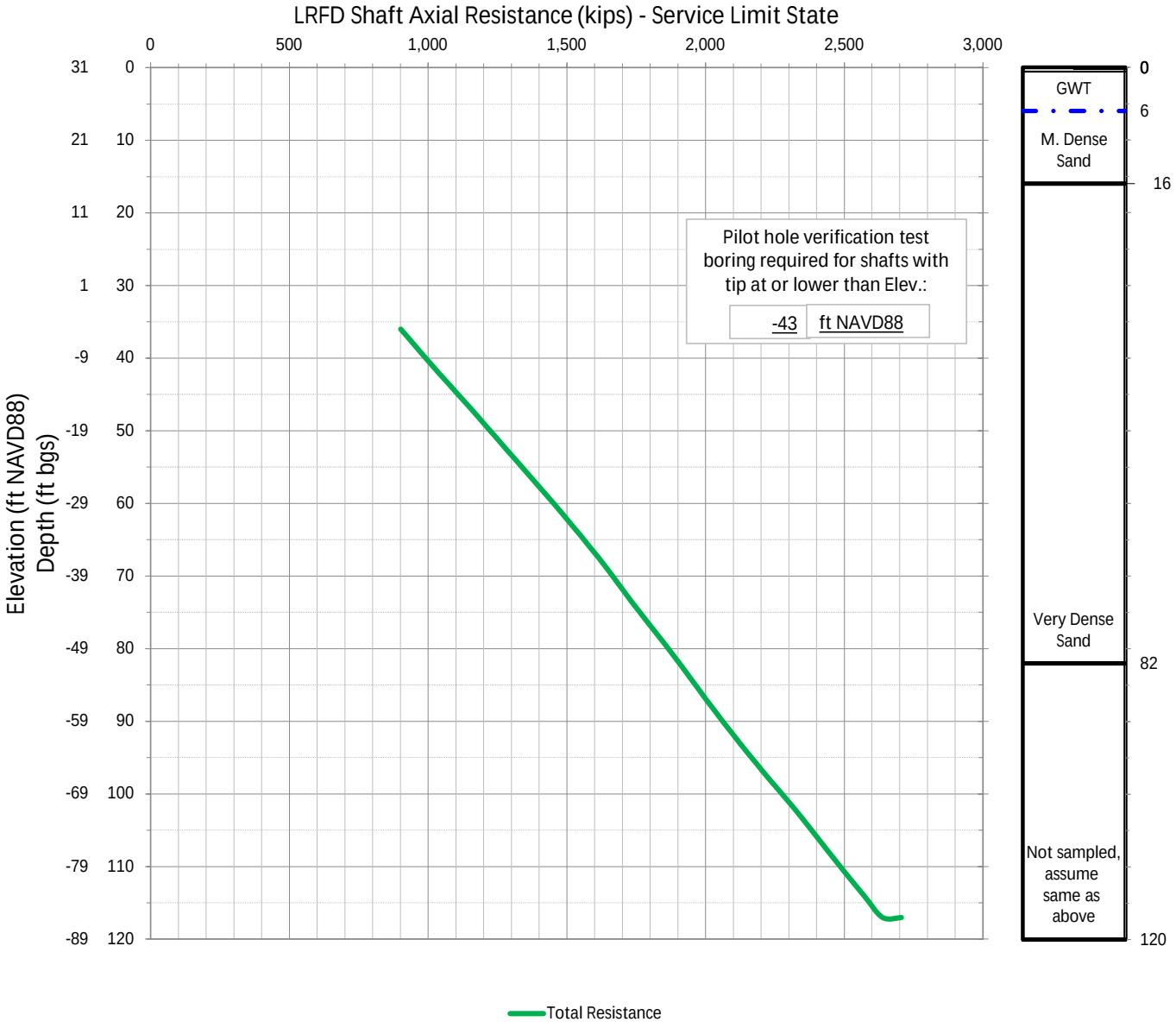
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

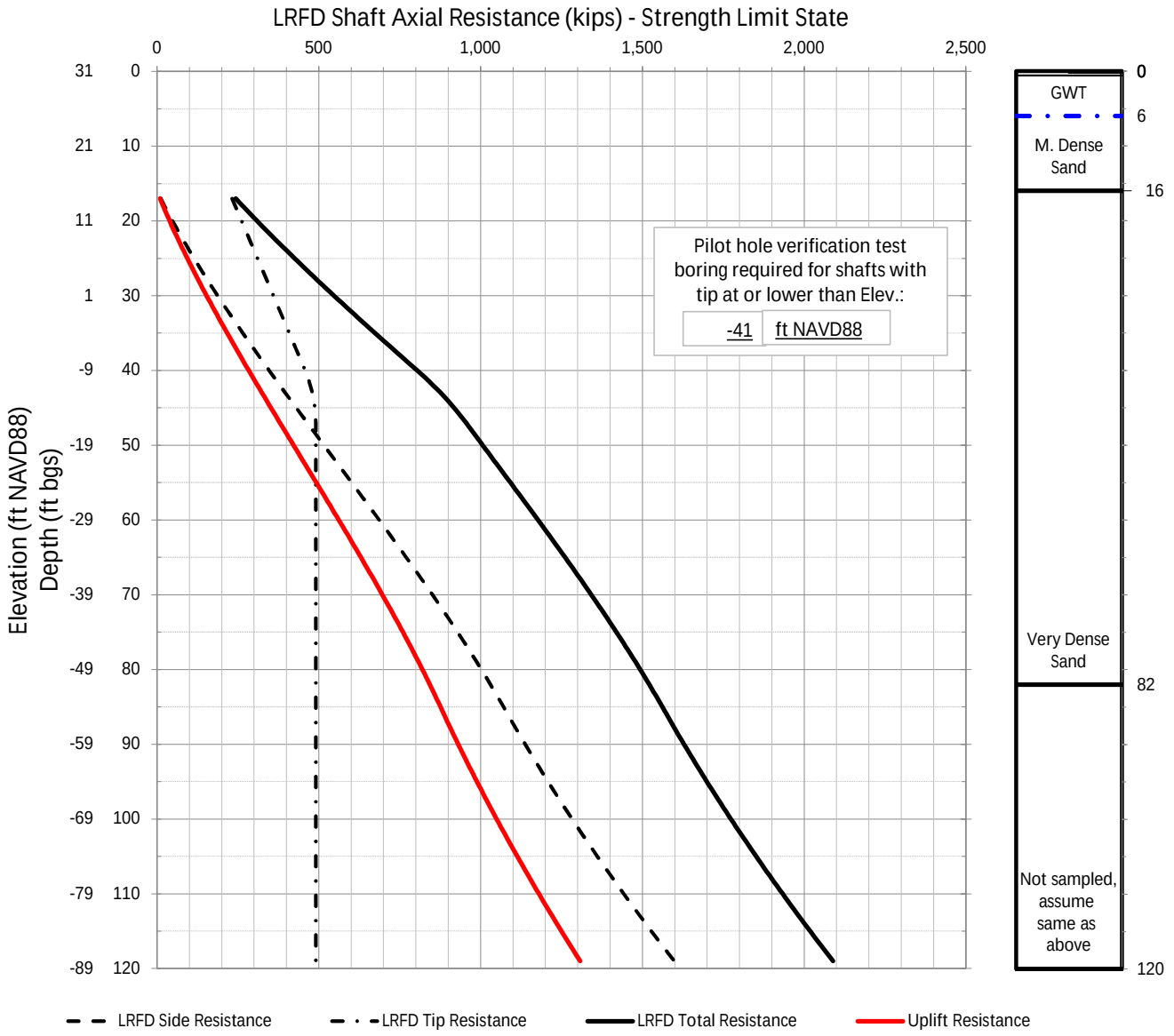
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

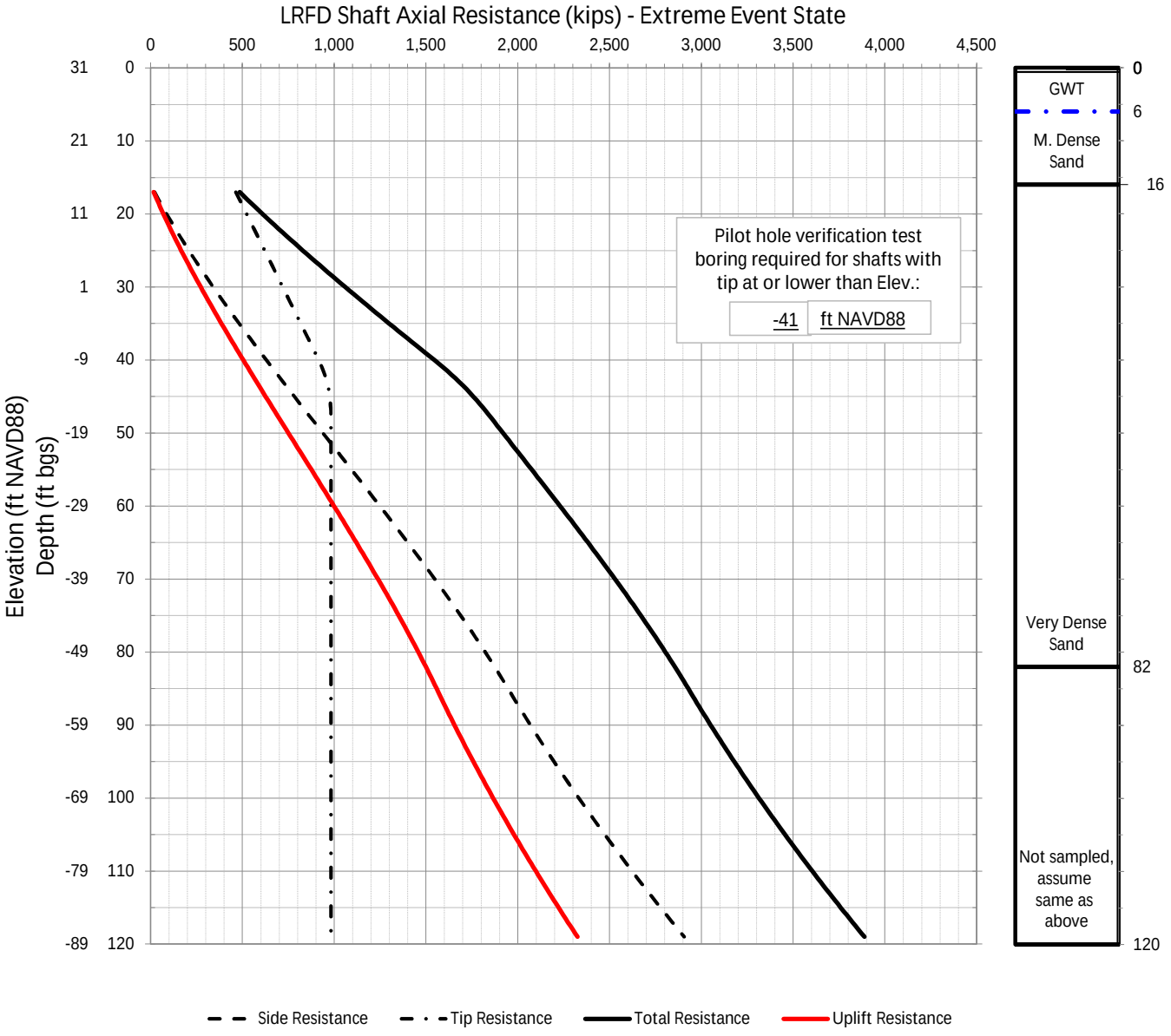
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

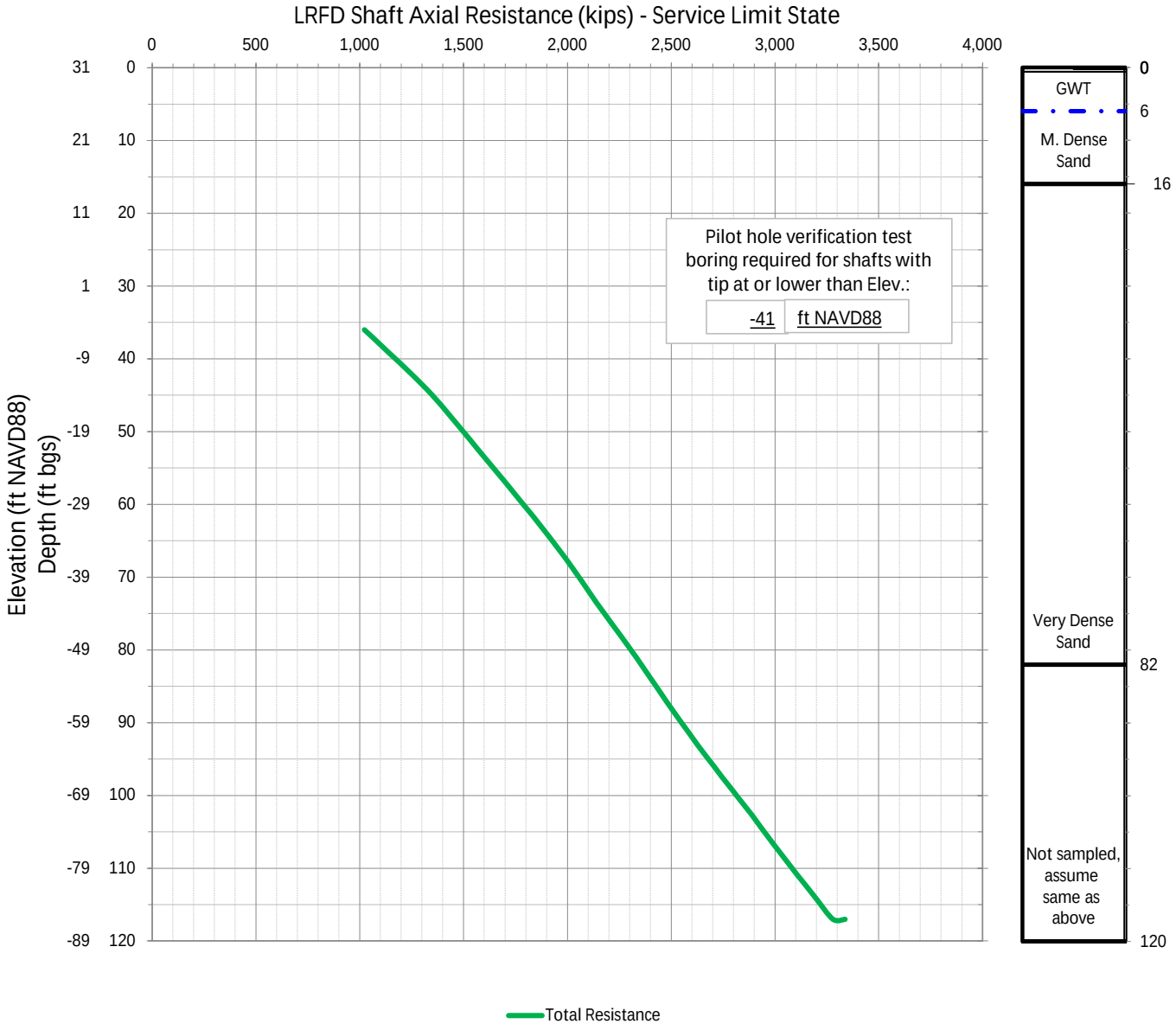
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
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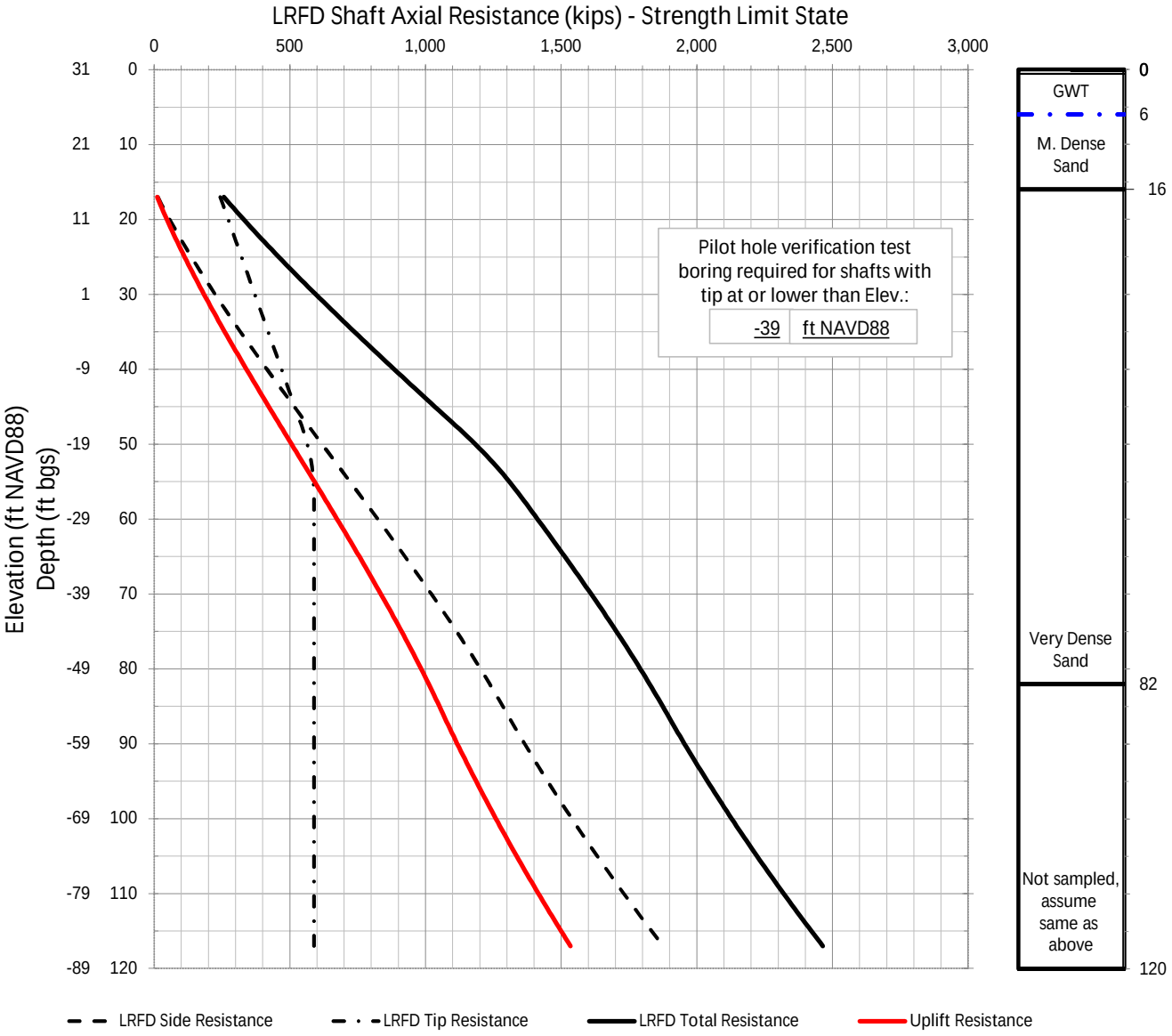
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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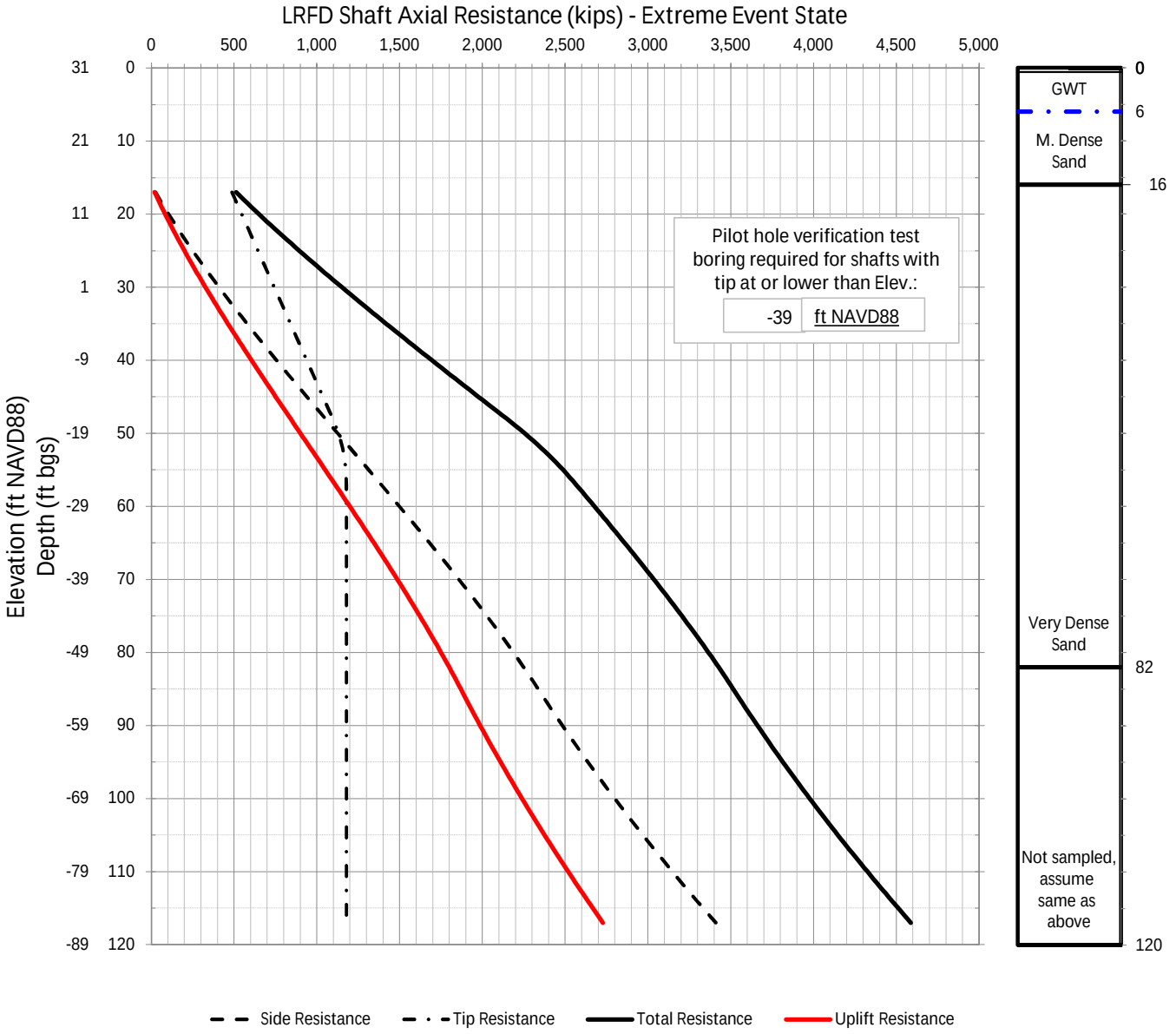
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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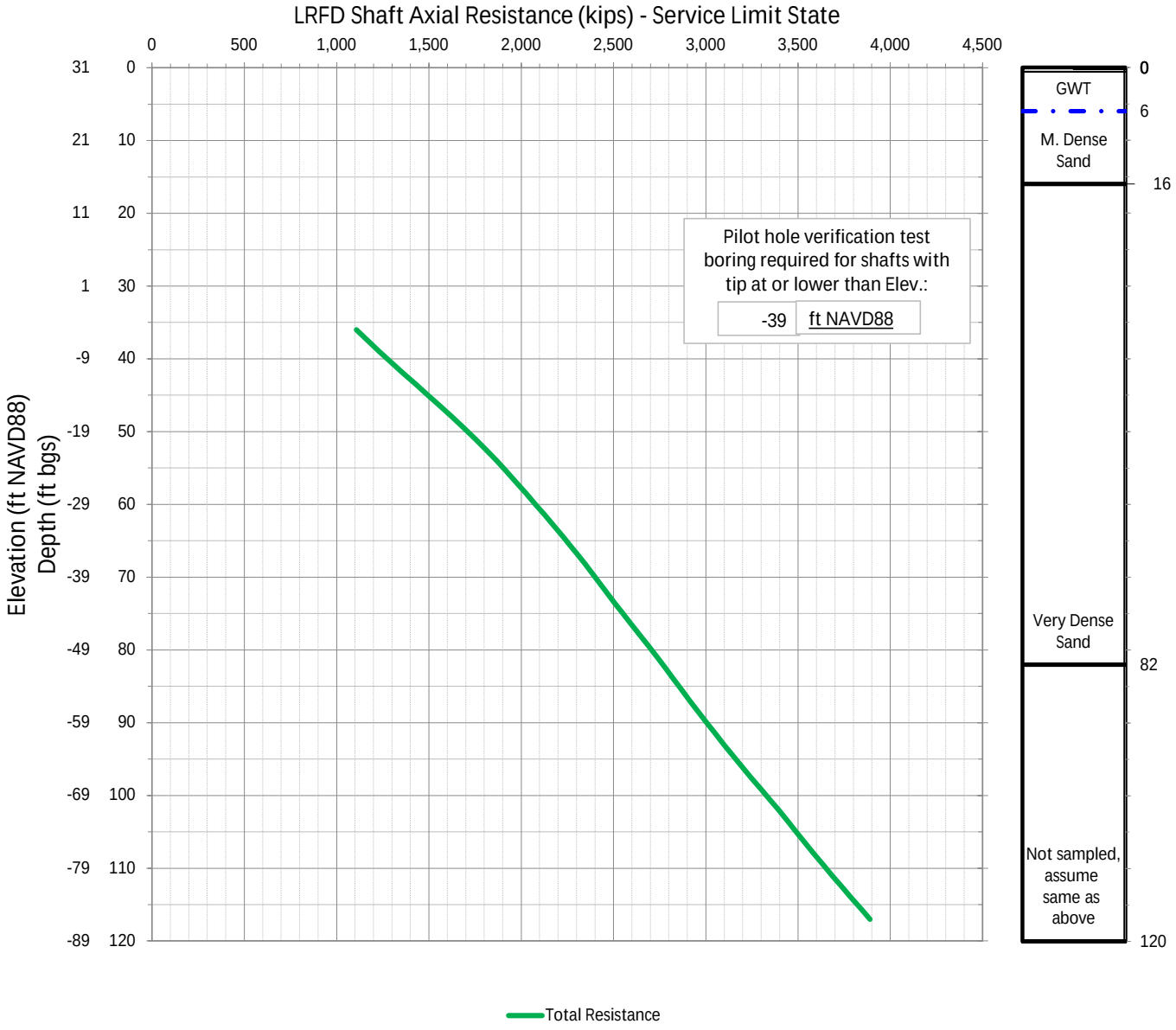
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

G.4 East Abutment Axial Capacity Curves (With Downdrag)

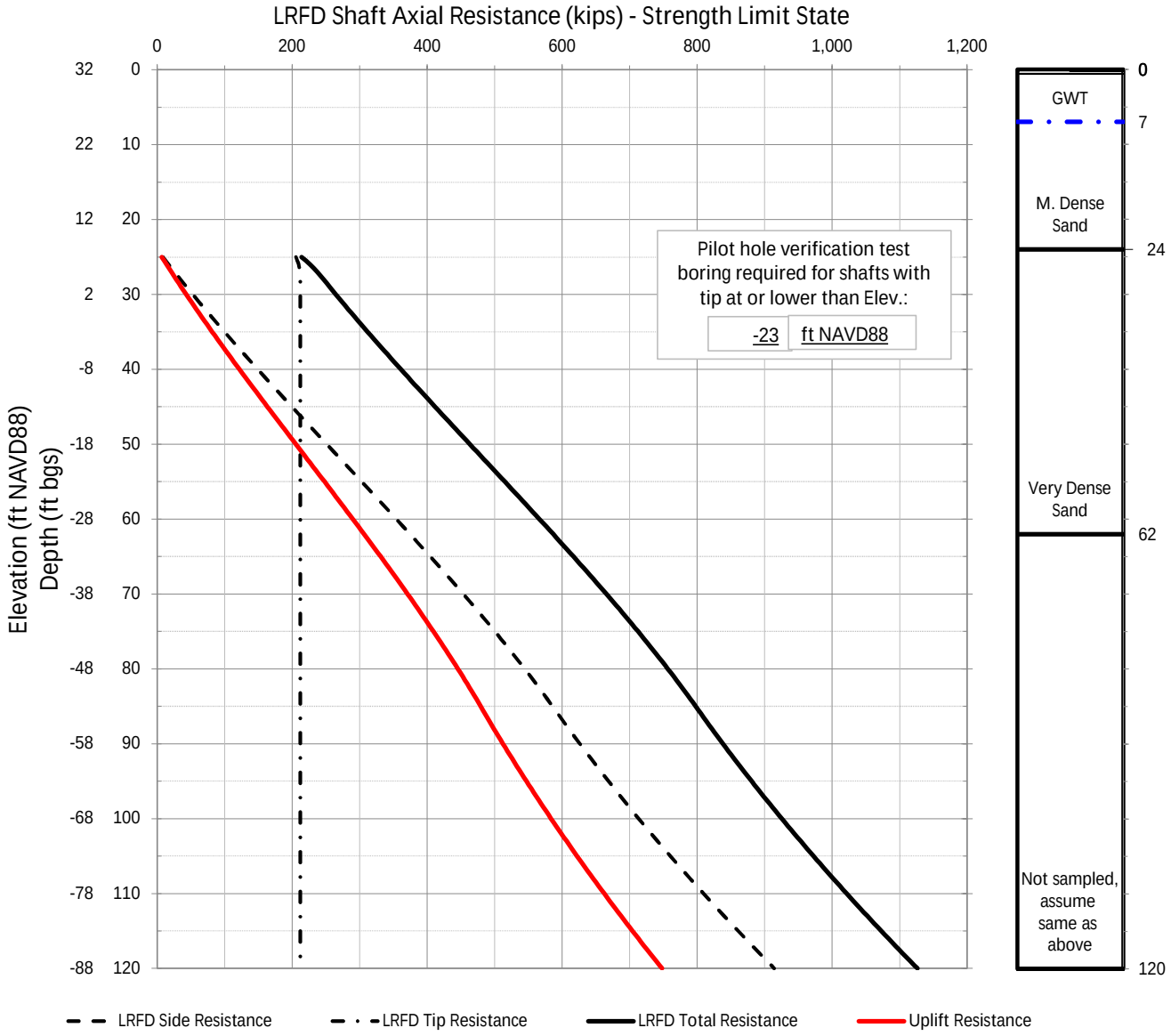
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
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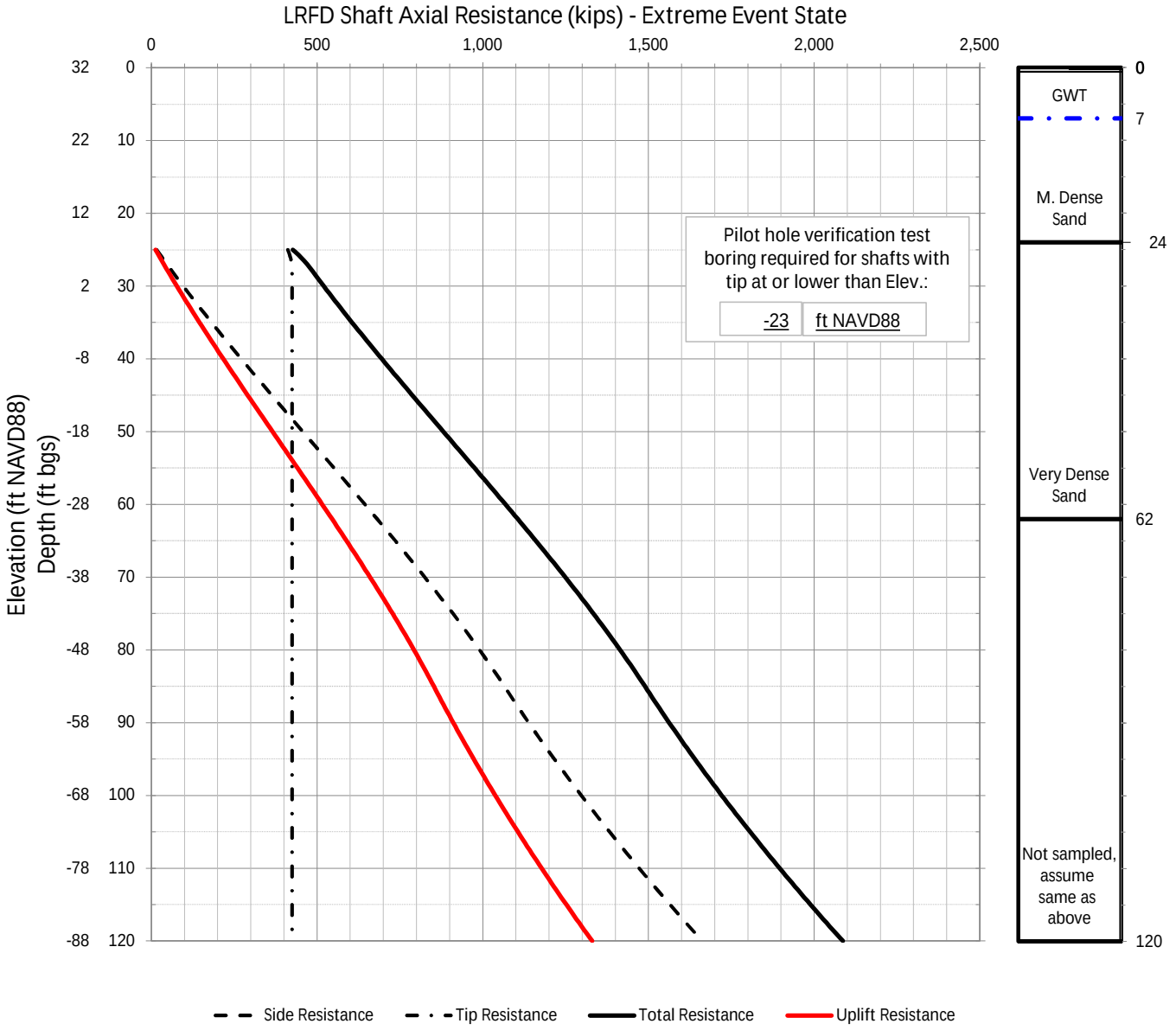
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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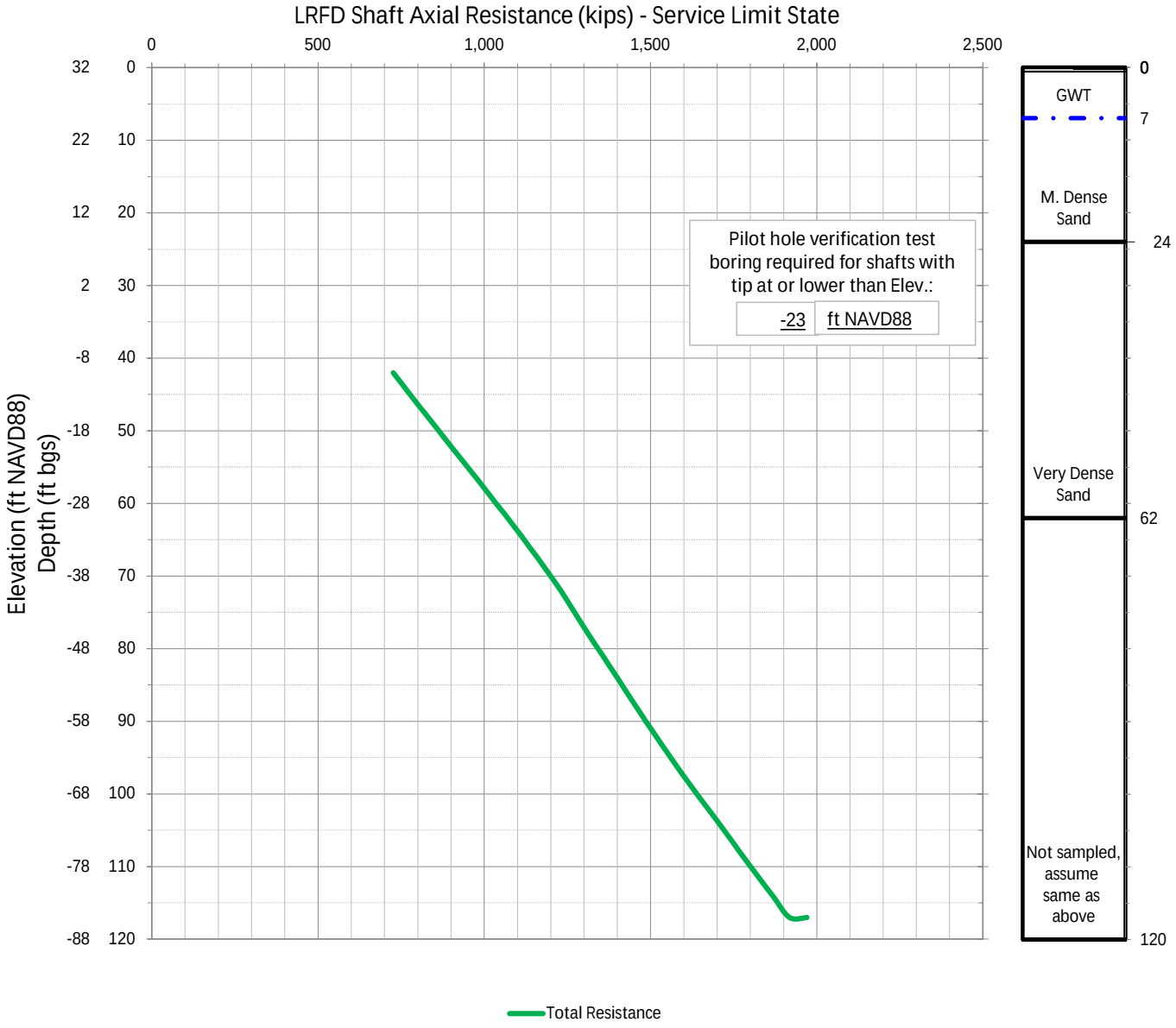
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **East Abutment**
3.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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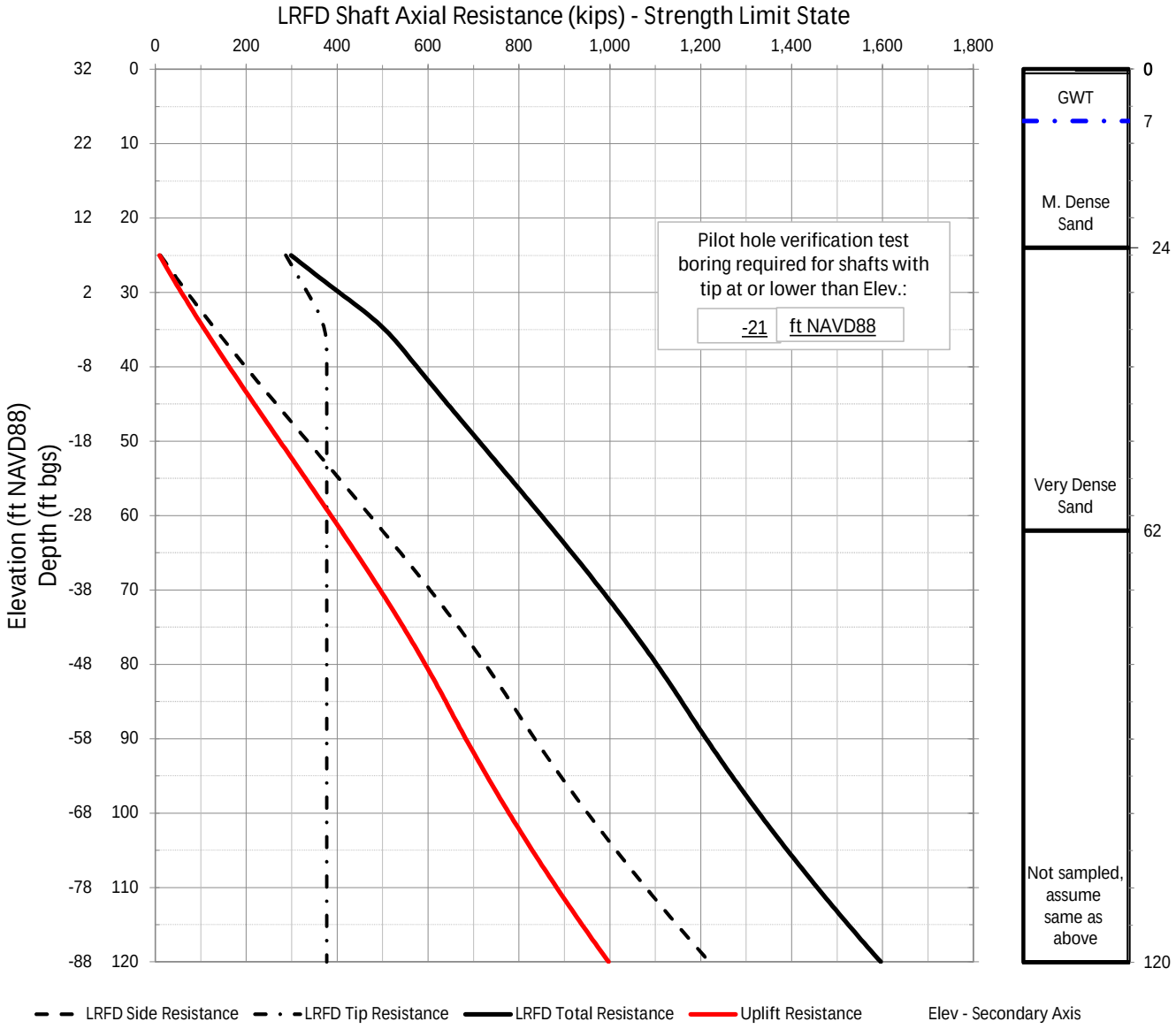
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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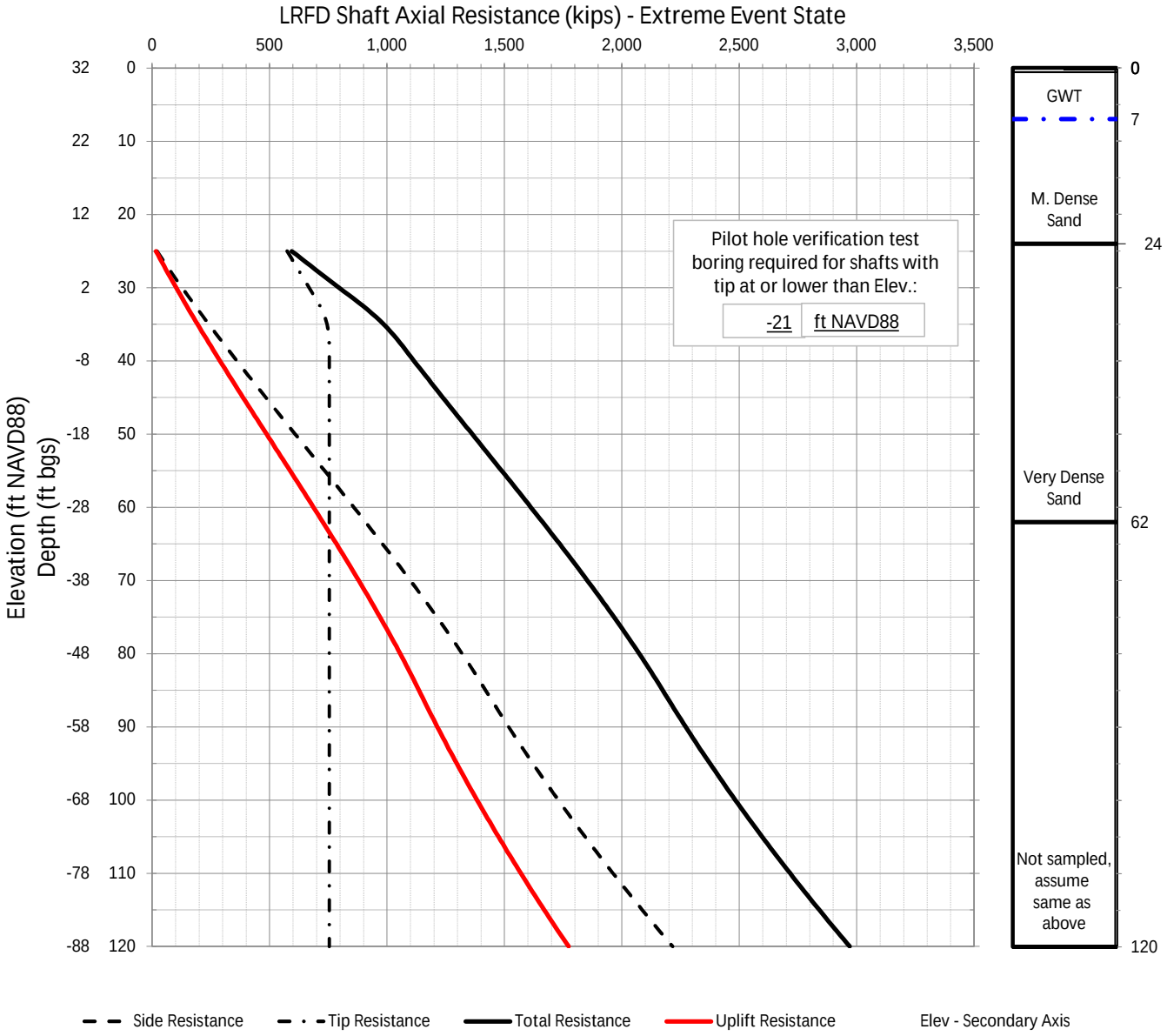
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

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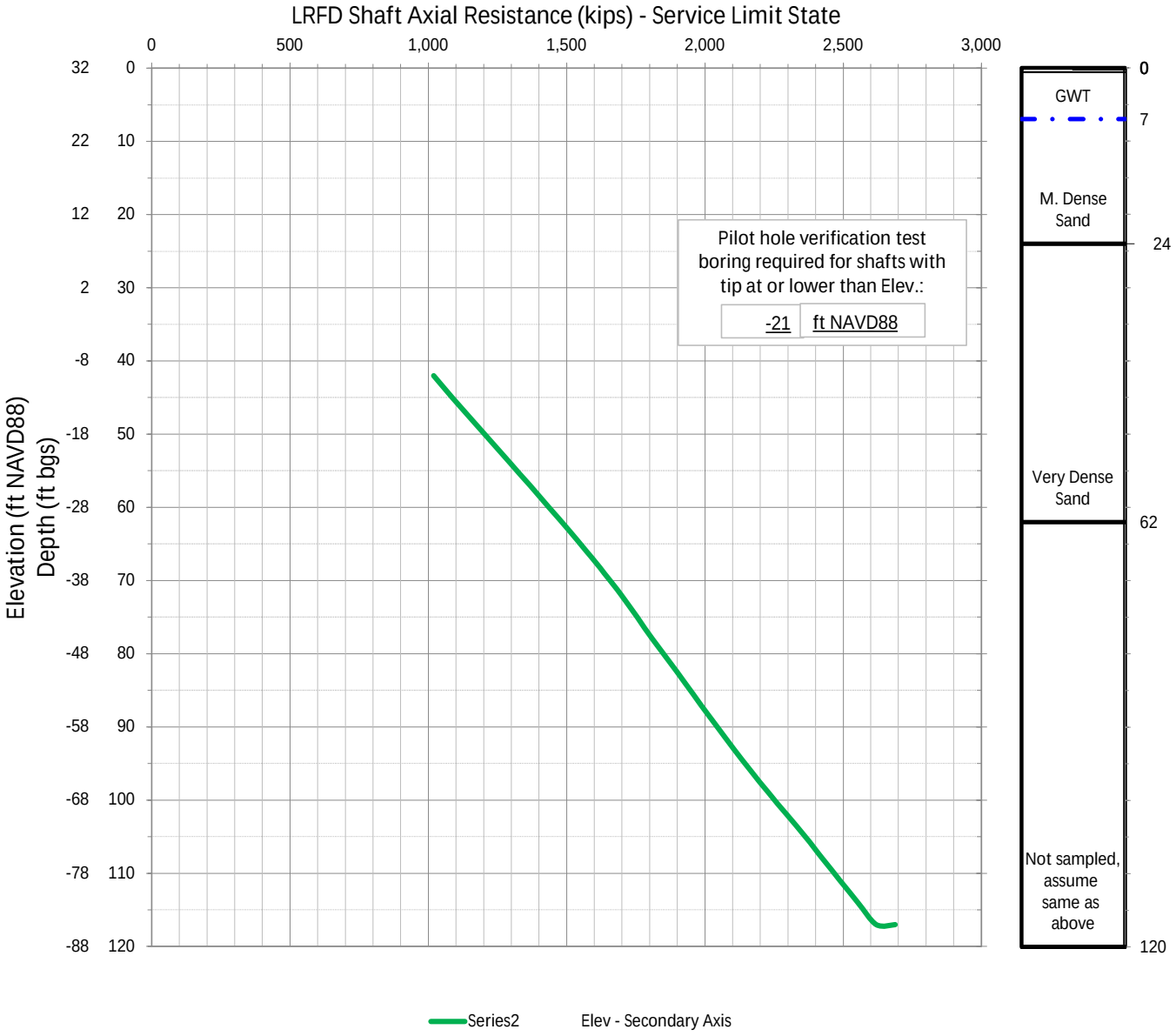
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **East Abutment**
4.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



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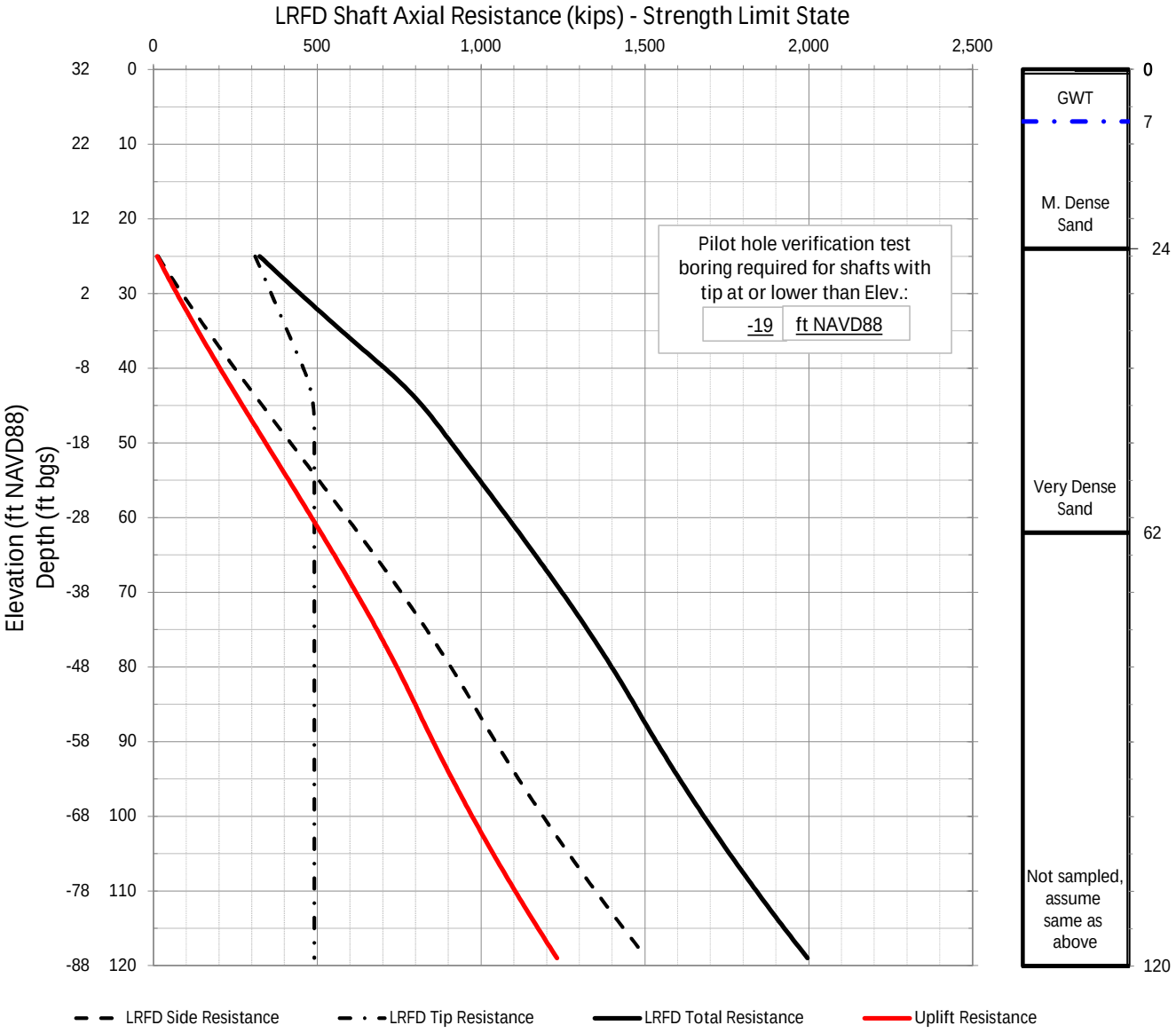
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

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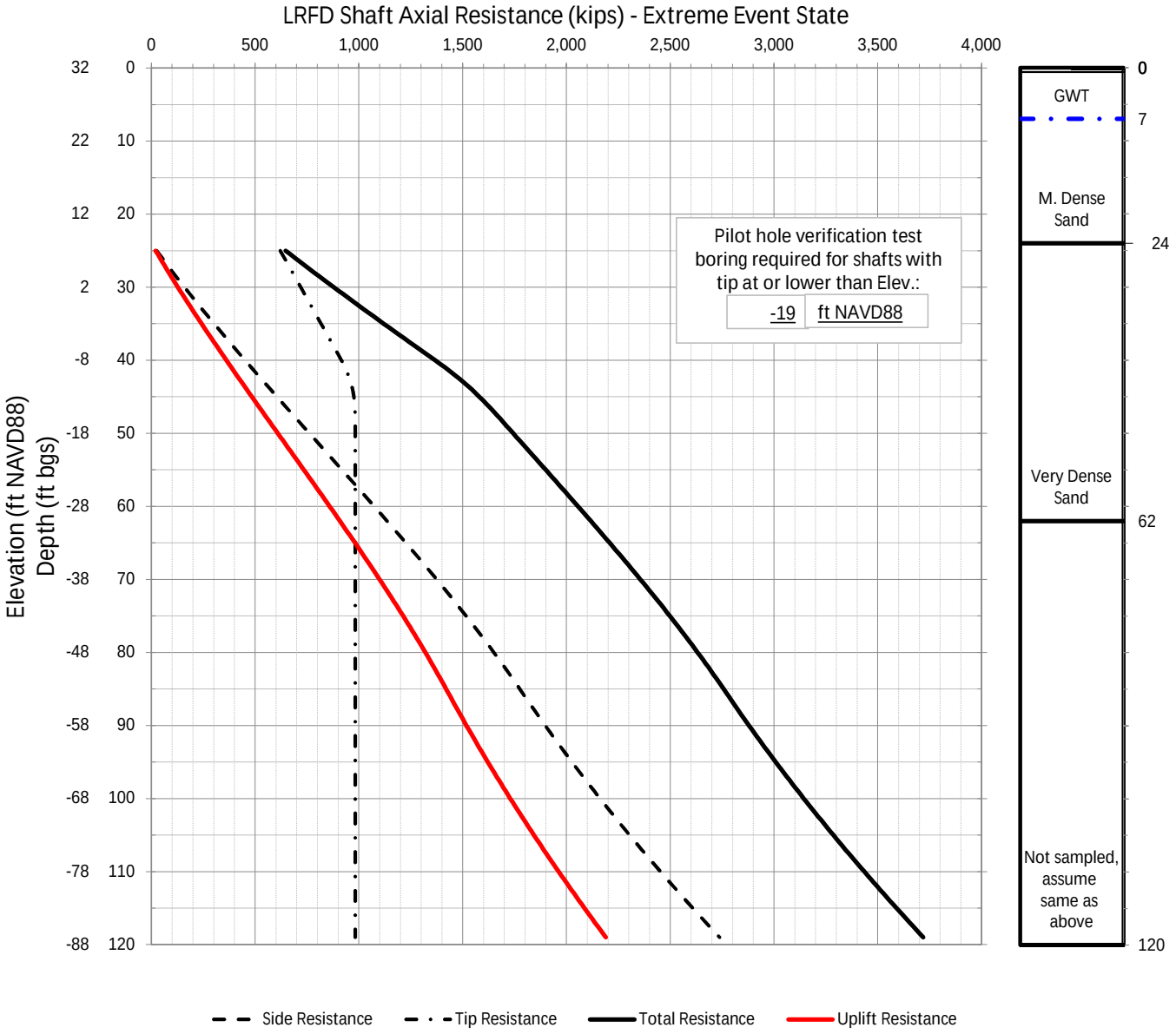
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

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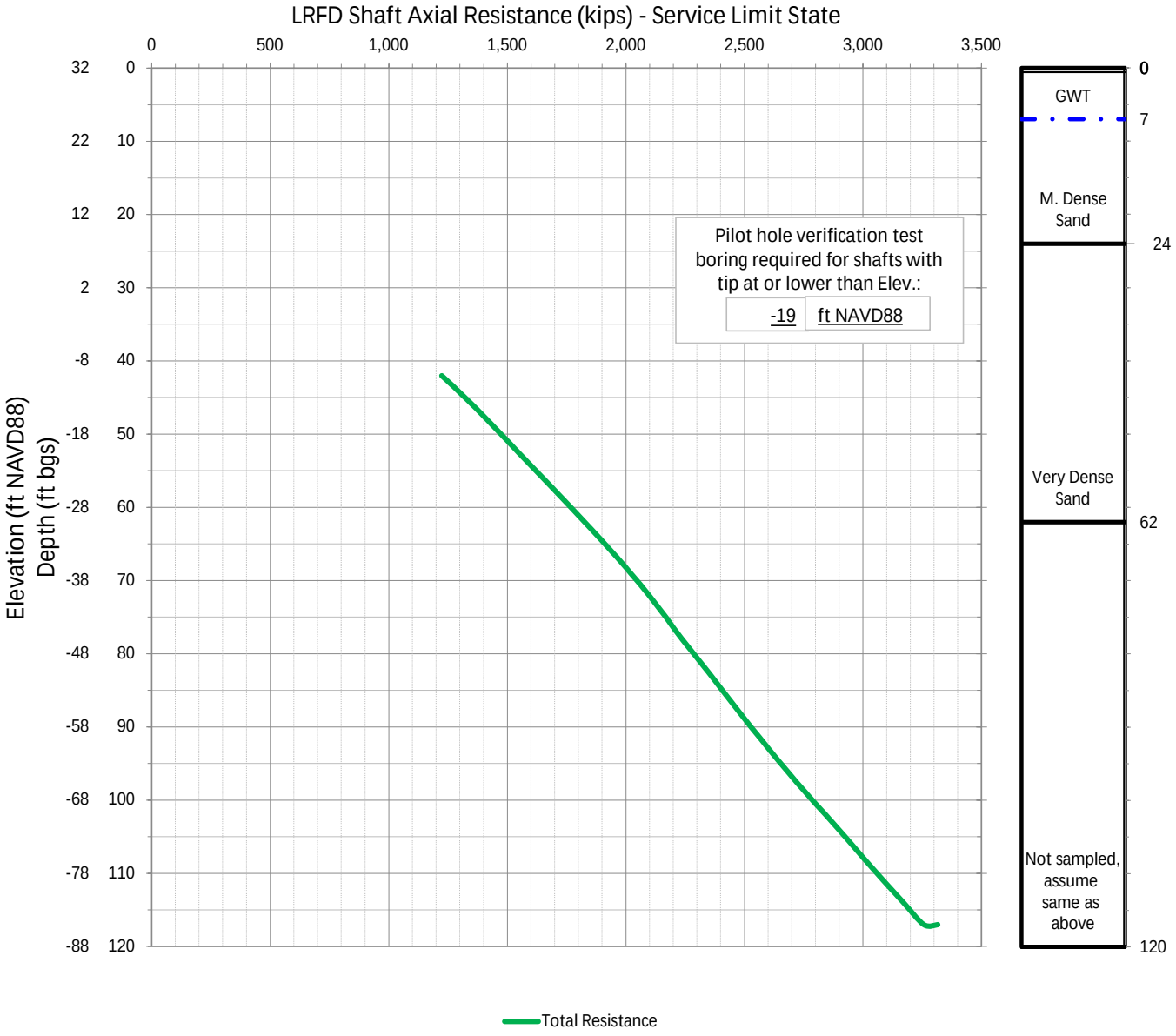
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **East Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

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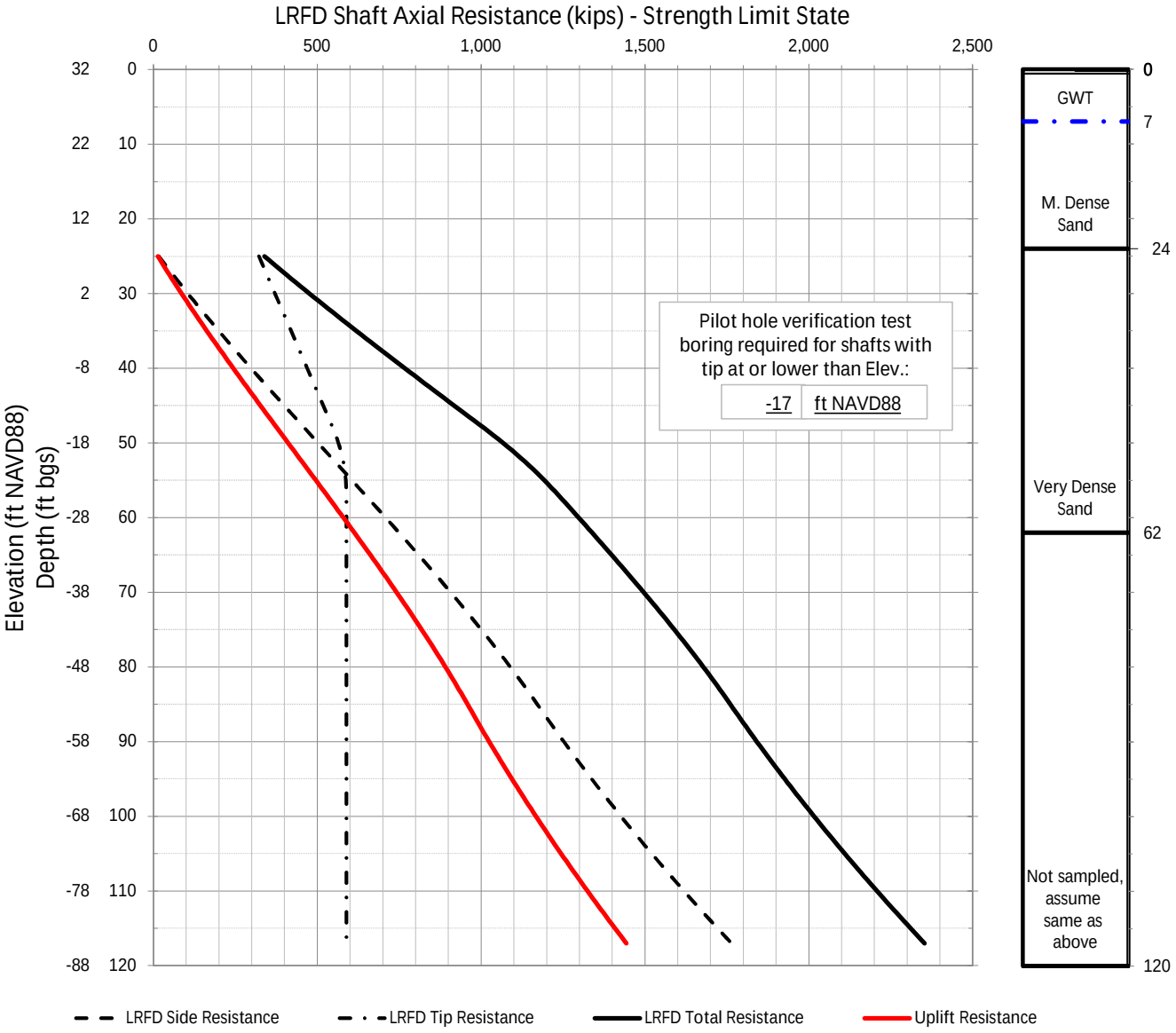
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

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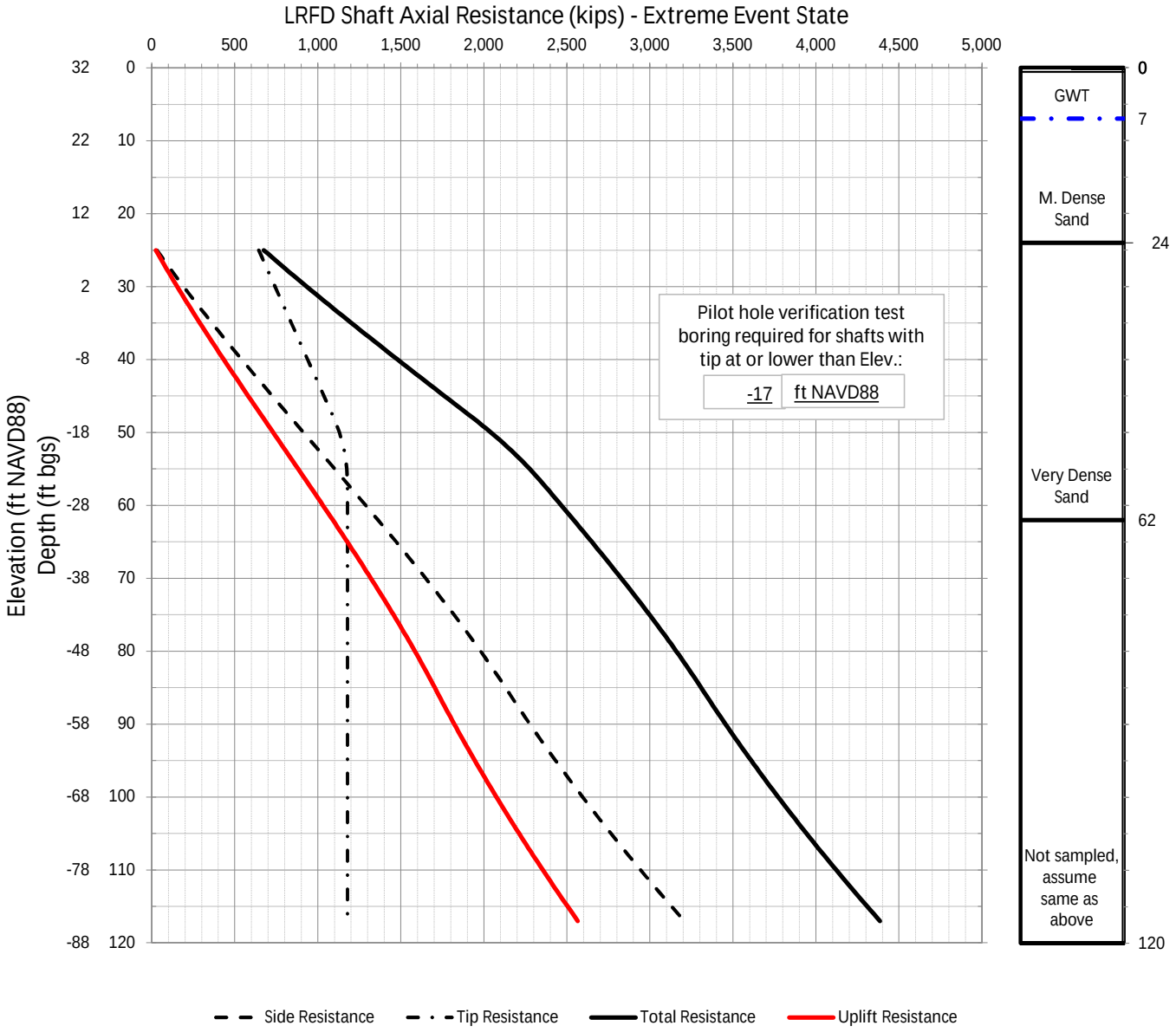
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
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Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



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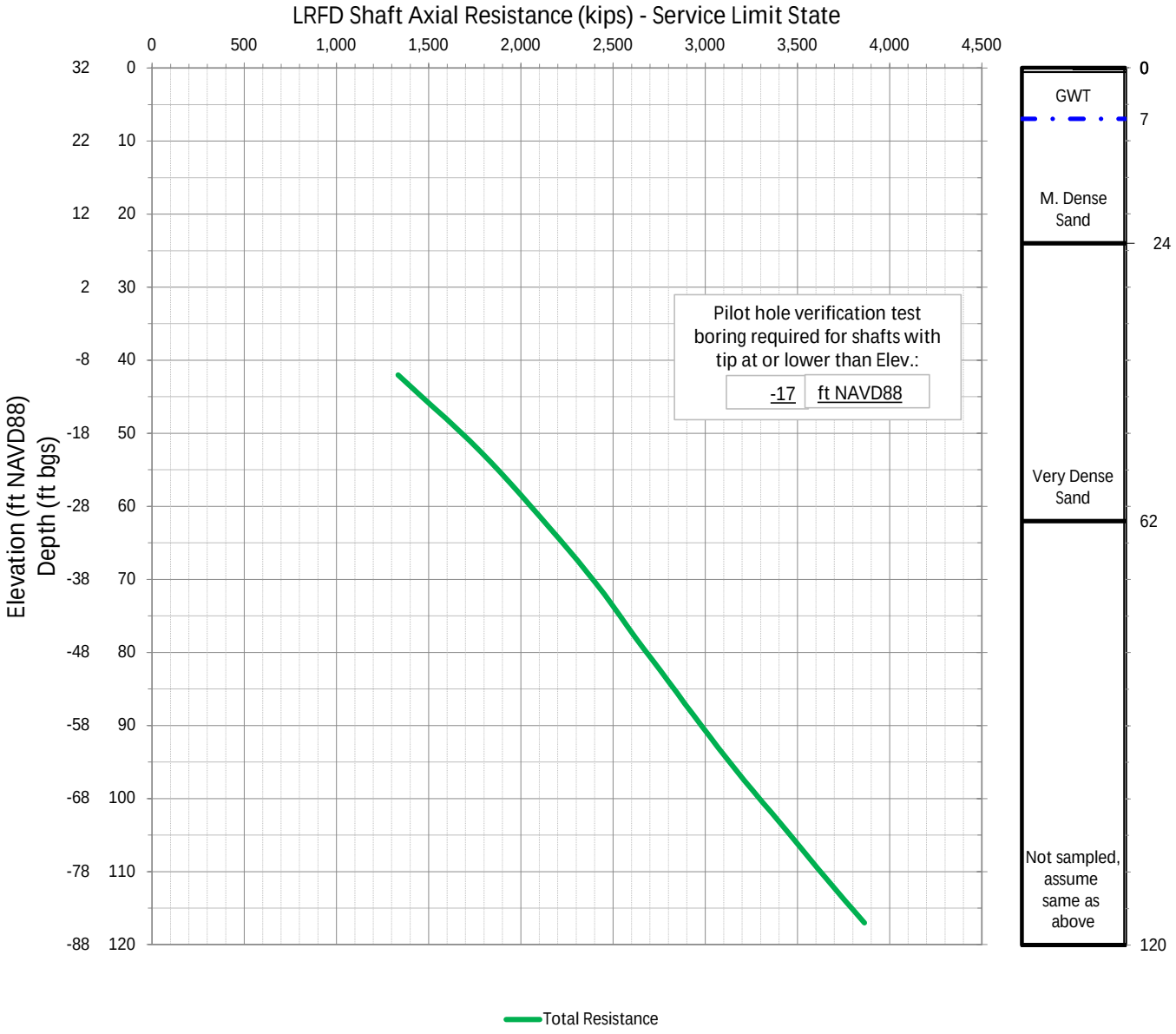
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **East Abutment**
6.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
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Design Ground Elevation = 32.0 ft NAVD88
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aecom.com

AECOM
1111 Third Ave., Suite 1600
Seattle, WA 98101
aecom.com

March 5, 2026

Our Reference
60683538

Ms. Asha D'Souza
City of Woodinville
17301 133rd Ave NE
Woodinville, Washington 98072

Addendum #1 to Geotechnical Report - Neutral Plane Method for Bridge Foundations

SR 202 Widening and Trestle Replacement, Woodinville, WA

Dear Ms. D'Souza:

AECOM Technical Services, Inc. (AECOM) is pleased to submit herewith this Addendum #1 to the Geotechnical Report (Addendum) for the "Phase B" elements of the SR 202 Widening and Trestle Replacement Project in Woodinville, Washington.

Services provided for this project were performed in general accordance with the Professional Services Agreement No. 22-025 between AECOM and the City of Woodinville effective as of April 20, 2022.

1. Introduction

This section provides pertinent project background information and states the purpose and scope of this Addendum.

1.1 Background

Phase B of the subject project includes widening and/or partial reconstruction of 131st Avenue NE between NE 175th Street and NE 177th Street, replacement of the existing railroad trestle over 131st Avenue NE with a new pedestrian bridge and retaining walls, and construction of a multi-user trail (Eastrail Trail) extending 1,256 feet east from the Sammamish River Trail to approximately 380 feet east of the bridge.

Geotechnical recommendations for project design and construction were provided in AECOM's *Report of Geotechnical Investigation* (Report) dated November 25, 2025 (AECOM, 2025). Recommendations in that report were implemented into the 95 percent design package. The Report provided geotechnical recommendations for drilled shaft foundations for the bridge abutments. The Report estimated that the sandy soils of Stratum 2 will experience maximum immediate elastic settlements of about 2.9 to 3.7 inches at the west and east abutments, respectively, due to the increased surface loading imparted by new retaining walls and approximate 10-foot raise of the approach embankments near the bridge abutment. Based on the maximum allowable service settlement of 1.0 inches for the drilled shafts, the Stratum 2 soils will experience relative settlement of at least 0.4 inches more than the drilled shaft foundations and thereby impart "negative skin friction" (i.e., drag load) to this portion of the shafts. Accordingly, the Report concluded that drilled shafts should be designed to account for downdrag and associated drag loading imparted by relative settlement within Stratum 2. Downdrag loads were not considered in either Stratum 1 because this unit comprises the existing embankment above the proposed drilled shaft elevations, or Stratum 3 because relative settlement between the drilled shafts is less than 0.4 inches.

The Report provided downdrag recommendations according to the “top load method” or “explicit method” recommended by the AASHTO Bridge Design Specifications 9th Edition (BDS) (AASHTO, 2020) and referenced in the WSDOT Geotechnical Design Manual (GDM) (WSDOT, 2022a) and WSDOT Bridge Design Manual (BDM) (WSDOT, 2022b). For the geotechnical limit state, the AASHTO explicit method neglects all side resistance of settling soils within the downdrag zone and considers the accumulated drag load imparted by the settling zone as an external load applied to the top of the drilled shaft in combination with all relevant loads applicable to the service, strength, and extreme limit states. The Report provided estimated drag loads for each shaft and modified axial resistance curves which neglect side resistance in the downdrag zone.

The AASHTO explicit method is recognized as a simple procedure that produces conservative results. The resulting drilled shaft lengths shown in the 95 percent design drawings are 115 and 130 feet for the west and east abutments, respectively. The Report suggested that the alternative “neutral plane method” (after Fellenius et al., 2004), which provides a more rigorous treatment of downdrag, could be evaluated to reduce conservatism, refine drilled shaft lengths, and potentially reduce estimated constructed costs. Accordingly, the AECOM geotechnical team performed calculations using the neutral plane method of analysis to compare those results with results of the AASHTO explicit method analysis.

1.2 Purpose and Scope

The purpose of this Addendum is to provide geotechnical analysis and recommendations for bridge drilled shaft foundations according to the neutral plane method to evaluate the effects of downdrag. The analysis is limited to 5-foot-diameter drilled shafts, which have been selected for final design, and incorporates the updated final individual shaft loads provided by the AECOM structural engineers. Downdrag evaluation for this project is limited to settlement under static loading; seismic downdrag associated with liquefaction-induced settlement is not considered applicable to this project based on the low potential for liquefaction triggering discussed in the Report.

The neutral plane method is specified as an alternative analysis method per AASHTO BDS Article 3.11.8. The WDOT GDM states that the neutral plane method may be used to assess liquefaction-induced downdrag with special approval by the State Engineer but does not specifically address usage of the neutral plane method for static loading. The WSDOT BDM does not address the neutral plane method. A WSDOT-sponsored research report (Muhunthan et al., 2017) provides detailed guidelines for use of the neutral plane method when analyzing deep foundations which may be subject to liquefaction-induced soil settlement, drag forces, and downdrag.

We understand compliance with WSDOT design guidance is preferred for this project, but that the City of Woodinville has discretion to allow variances provided there is an engineering justification. Accordingly, we recommend the neutral plane method be accepted by the City of Woodinville for final design of bridge foundations on this project, noting that the method is allowed by the AASHTO BDS.

2. Neutral Plane Method

A discussion of the neutral plane method and differences from the AASHTO explicit method are presented in the following paragraphs.

2.1 Summary of Neutral Plane Method

The premise of the neutral plane method is that there is a location (i.e., the neutral plane) within a deep foundation where the downward-acting forces are equal to the upward-acting forces and there is no relative movement between the deep foundation and the surrounding soil. The phenomenon of the neutral plane has been demonstrated by full-scale instrumented load tests in published case histories (Fellenius, 2004). Research has also demonstrated the neutral plane method is valid for conditions involving soil settlement caused by both static loads and liquefaction-induced settlement (Muhunthan et al., 2017).

The location of the neutral plane is estimated by locating the intersection of two curves on a plot of depth (or elevation) along the deep foundation profile: 1) the sustained permanent external axial load on the

foundation plus the negative skin friction accumulated from the top of the foundation downward (i.e., the downward-acting forces); and 2) the foundation tip resistance plus the foundation side resistance summed from the bottom of the foundation upward (i.e., the upward-acting forces). The loads and resistances are unfactored for the neutral plane plots. Once the neutral plane has been located, the drag force on the deep foundation can be calculated by subtracting the sustained permanent axial load from the load at the intersection of the two curves. This is illustrated in **Figure 1**.

Additionally, since relative movement between the deep foundation and the surrounding soil is zero at the location of the neutral plane, the settlement of the deep foundation is equal to the settlement of the surrounding soil at the neutral plane. For final evaluation of structure settlement at the top of the foundation element, elastic compression of the foundation also needs to be considered. This concept is illustrated in **Figure 2**.

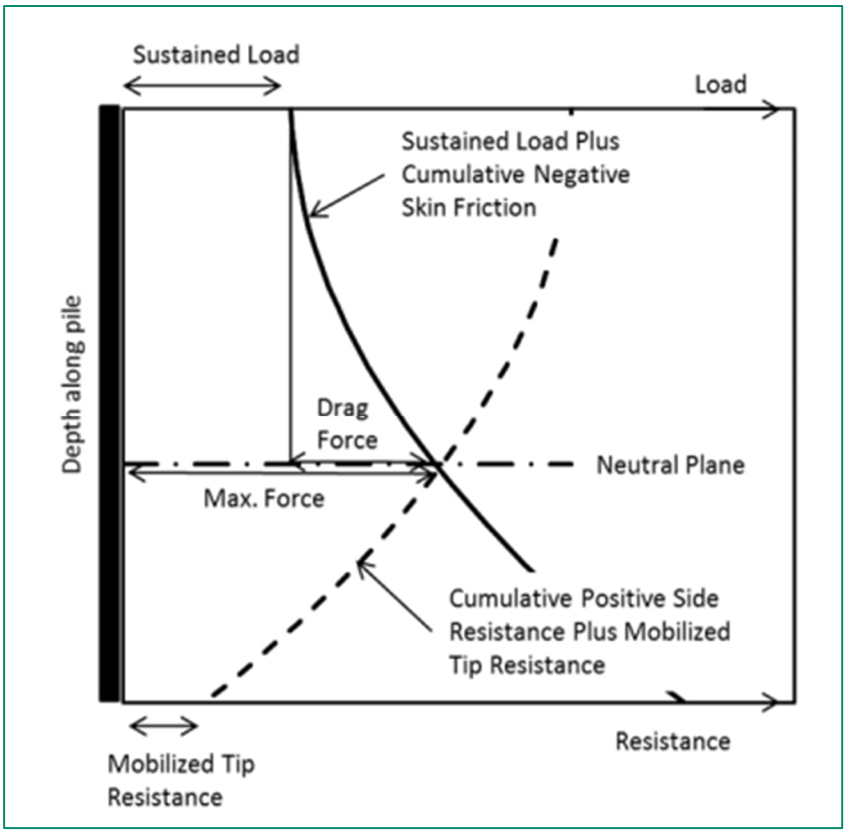


Figure 1. Plot of Neutral Plane Method (from Siegel et al., 2014)

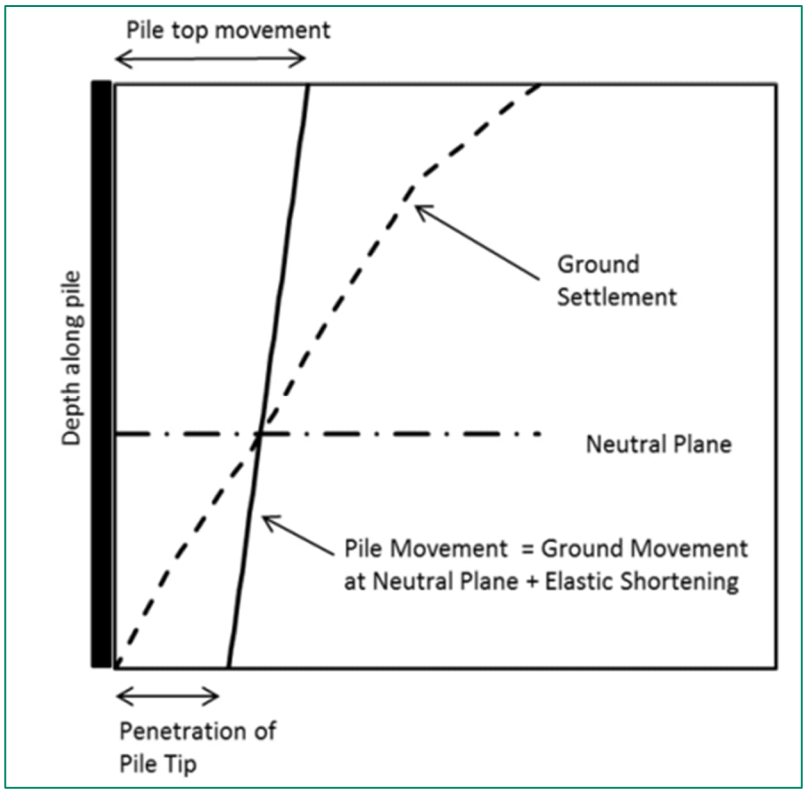


Figure 2. Estimation of Pile Settlement from Neutral Plane Method (from Siegel et al., 2014)

2.2 Comparison of Neutral Plane Method and AASHTO Explicit Method

According to Fellenius (2004), the drag load is only a concern for the structural strength of the deep foundation and has no consequence for the bearing capacity or settlement of the deep foundation or deep foundation group. Effectively, the drag load is treated as an internal structure force, much like a prestressing load in a prestressed concrete pile. This is the primary difference between the neutral plane method and the AASHTO explicit method: the neutral plane treats the drag load as an additional internal structure force that must be accounted for in the structural limit state only, whereas the AASHTO explicit method treats the drag load as an external load that must be accounted for in both the geotechnical and structural limit states. This interpretation is consistent with the guidance presented in the Federal Highway Administration's (FHWA) *Drilled Shaft Manual* (Brown et al., 2010) and Siegel et al. (2014).

Additionally, the cumulative side resistance along the entire deep foundation is considered in the neutral plane method, unlike the AAHSTO explicit method which ignores side resistance in the downdrag zone. This is because at the geotechnical strength limit state, the entire deep foundation is moving downward relative to the soil and therefore negative skin friction is not present (Siegel et al., 2014). Further, application of external transient loads to the deep foundation tends to reduce or eliminate the drag load (AASHTO, 2020; Siegel et al., 2014; Fellenius, 2004).

A summary of the differences between the neutral plane method and the AASHTO explicit method is shown below in **Table 1**.

Table 1. Neutral Plane Method versus AASHTO Explicit Method (adapted from Siegel et al., 2014)

Consideration	Neutral Plane Method	AASHTO Explicit Method
Applicability	All piles and drilled shafts	Only certain pile/drilled shaft conditions
Drag load	Treated as an internal structural force	Treated as external top load on foundation
Structural limit state	Includes drag force as internal load	Includes drag force as an external top load
Geotechnical limit state	Excludes drag force	Includes drag force as an external top load
Foundation Settlement	Excludes drag load (settlement is function of neutral plane location)	Includes drag load as external load causing settlement

3. Drilled Shaft Analysis

A discussion of the analysis procedures, input parameters, and results are provided in the following paragraphs.

3.1 Drilled Shaft External Loads

Final structural loads for individual 5-foot-diameter drilled shafts provided by the AECOM structural design team on January 27, 2026 are summarized in **Table 2**. The loads correspond to foundations for the east abutment, which are similar to the loads for the west abutment according to correspondence with the structural team; drag forces are not included in these loads.

Table 2. Summary of Factored Permanent and Transient Loads for 5-foot-diameter Drilled Shafts

Load Combination	Type	Minimum Axial Load (kips)	Maximum Axial Load (kips)
1	Service I	-67	1,526
2	Strength Ia	-305	1,591
3	Strength Ib	-125	1,909
7	Strength IVa	-305	1,591
8	Strength IVb	-80	2,030
10	Extreme Ib	-1,434	2,757

Additionally, the neutral plane method requires the unfactored permanent axial load as an input to estimate the location of the neutral plane. The permanent factored loads were developed by the structural design team according to permanent and transient load definitions in AASHTO BDS Section 3.3.2 and provided on January 27, 2026 via email. The permanent factored load combinations are summarized in **Table 3**; drag forces are not included in these permanent loads per guidance of Siegel et al. (2013). The Service I permanent load combination presented in **Table 3** was adopted as the unfactored permanent axial load for design analyses, since it incorporates load factors equal to unity for permanent loads and excludes transient loads. Additionally, the Service I permanent load combination produces similar results to the Strength IV load combination when changing all load factors equal to unity which is recommended by Siegel et al. (2013) as most closely approximating the unfactored permanent load. Accordingly, based on values in **Table 3** and understanding that a smaller top load tends to produce a larger drag force, the permanent unfactored load used in our neutral plane analysis is 1,224 kips for both the east and west abutments.

Table 3. Summary of Factored Permanent Loads for 5-foot-diameter Drilled Shafts

Load Combination	Type	Minimum Axial Load (kips)	Maximum Axial Load (kips)
1	Service I	162	1,224
2	Strength Ia	-75	1,385
3	Strength Ib	133	1,576
7	Strength IVa	-75	1,385
8	Strength IVb	220	1,730
10	Extreme Ib	165	1,210

3.2 Geotechnical Limit State

As stated previously, the drag load is not considered at the geotechnical limit state evaluation in the neutral plane method, and the cumulative skin friction along the deep foundation can be considered in design. Accordingly, the preliminary length of the drilled shaft to satisfy strength, service, and extreme geotechnical limit states was obtained from the respective axial capacity curves presented in the Report which have not been adjusted for negative skin friction (drag force).

Annotated copies of the curves illustrating the applicable factored load combination and minimum shaft length for each limit state are presented as Attachments A1 (east abutment) and A2 (west abutment). The results indicate the strength limit state controls, so the recommended preliminary, minimum, drilled shaft length is 102 feet for the east abutment and 102 feet for the west abutment.

3.3 Neutral Plane Analysis

Based on the preliminary drilled shaft length for the geotechnical limit state (102 feet for both the east and west abutments), the neutral plane analysis was conducted to estimate the internal drag force and the settlement of the drilled shaft. The neutral plane analysis requires the following additional input parameters:

1. Surrounding soil settlement profile
2. Deep foundation stiffness properties

The settlement of the surrounding soil was obtained from the Settle3 analysis results in the Report. The drilled shaft stiffness properties were then used to calculate the elastic compression of the shaft under the sustained permanent unfactored load, and were estimated as follows:

- Gross cross-section area, A = 2,827.4 square-inches
- Steel reinforcement area, As = 1 percent of gross cross-sectional area
- Concrete compressive strength, f'c = 3.6 ksi
- Elastic modulus of steel, Es = 29,000 ksi
- Elastic modulus of concrete, Ec = 3,420 ksi
- Composite shaft stiffness, A*E = 1,039,380 kips

Elastic compression is then calculated according to the formula $s_c = P*L / A*E$ where P is the unfactored permanent load and L is shaft length.

The neutral plane analysis plots were prepared according to the method described previously. The plotted curves of upward-acting and downward-acting forces assume 100 percent mobilization of both shaft skin friction and tip resistance, which produces a conservative estimate of drag force. At tip resistance mobilization of less than 100 percent, the neutral plane method estimates lower drag forces and slightly higher shaft settlement; however, downdrag settlement is expected to be less than 1 inch regardless of tip mobilization based on examination of the plotted curves.

The location of the neutral plane and the load at the neutral plane (Qn) were graphically estimated. The drag force was obtained by subtracting the unfactored permanent dead load from the Qn value. Settlement at the neutral plane was estimated graphically, and the settlement at the top of the shaft is the sum of the neutral plane settlement plus elastic compression of the shaft. The neutral plane plot for the east abutment is presented in Attachment A3 and the plot for the west abutment is presented in Attachment A4. Results of the analyses are summarized in **Table 4**.

Table 4. Summary of Neutral Plane Analysis Results

Parameter	Unit	East Abutment	West Abutment
Shaft Length	(feet)	102	102
Tip Elevation	(feet NAVD88)	-77	-77
Elevation of Neutral Plane	(feet NAVD88)	-27.5	-27.5
Unfactored Drag Force, DD	(kips)	1,361	1,346
Settlement at Neutral Plane	(inch)	0.40	0.26
Settlement at Top of Shaft	(inch)	0.48	0.34

4. Conclusions and Recommendations

The neutral plane method of analysis results in a more economical minimum required drilled shaft design length as compared to the AASHTO explicit method, e.g., required minimum shaft lengths of about 102 feet compared to the 115 to 130 feet lengths estimated from the AASHTO explicit method. Estimated top of shaft settlement, including the downdrag (downward foundation movement) attributed to the estimated drag force, is about 0.5 inches or less, and is less than the maximum allowable serviceability settlement of 1.0 inches.

The unfactored drag force (DD) should be considered in structural design of the drilled shafts as an additional internal force. The structural strength limit state should incorporate load factors for DD according to Table 3.4.1-2 of the AASHTO BDS, which are 1.25 maximum (for shafts in axial compression loading) and 0.35 minimum (for shafts in tension loading). A load factor of 1.0 is applicable to DD at the structural service and extreme limit states.

5. References

American Association of State Highway and Transportation Officials (AASHTO). (2020). *LRFD Bridge Design Specifications, 9th Edition*

AECOM. (2025). *Report of Geotechnical Investigation SR 202 Widening and Trestle Replacement, NE 175th Street to NE 177th Street, Woodinville, Washington*. Prepared for the City of Woodinville. Project number: 60683538. November 25.

Brown et al. (2010). *Drilled Shafts: Construction Procedures and LRFD Design Methods.* NHI Course No. 132014, Publication No. FHWA-NHI-10-016, FHWA GEC 010.

Fellenius, B. H. (2004). *Unified Design of Piled Foundations with Emphasis on Settlement Analysis.* Geo-Institute Geo-TRANS Conference, Los Angeles, July 27-30, ASCE Special Publication (GSP 125): "Current Practice and Future Trends in Deep Foundations", pp. 253-275.

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Muhunthan, B., M.; Vijayathasan, N. V.; and Abbasia, B. (2017). *Liquefaction-Induced Downdrag on Drilled Shafts*. Washington State Department of Transportation (WSDOT) Research Report No. WA-RD 865.1. April.

Siegel, T. C.; Lamb, R.; Dasenbrock, D.; and Axtell, P. J. (2013). *Alternative Design Approach for Drag Load and Downdrag of Deep Foundations within the LRFD Framework.*

Siegel, T. C.; Lamb, R.; Dasenbrock, D.; and Axtell, P. J. (2014). *Neutral Plane Method for Drag Force of Deep Foundations and the AASHTO LRFD Bridge Design Specifications.*

Washington State Department of Transportation (WSDOT). (2022a). *Geotechnical Design Manual M 46-03.16*. February.

Washington State Department of Transportation (WSDOT). (2022b). *Bridge Design Manual (LRFD) M 23-50.21*. June.

Washington State Department of Transportation (WSDOT). (2022c). *Design Manual M 22-01.21*. September.

6. Limitations

This Addendum has been prepared by AECOM for the exclusive use of the City of Woodinville and the SR 202 Widening and Trestle Replacement Project design team members. The report was prepared using the degree of care and skill ordinarily exercised under similar circumstances by responsible engineers and geologists practicing in the same general location. No other warranty or representation, either expressed or implied, is made as to the findings and professional advice in this report.

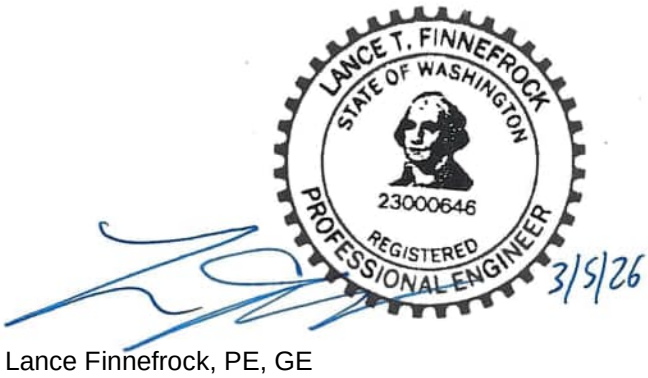
The analyses and recommendations contained in this report are based on the data obtained from previous and current investigations of field exploration, laboratory testing, soil design parameters, and provided plans. The boring logs indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. They do not necessarily reflect variations that may exist between locations or possible changes that may take place with time and depth. If variations in subsurface conditions from those described in this report are noted during construction, the recommendations presented in this report should be reviewed and updated, as needed.

If a change in the nature, design, or location of the proposed structures occurs, the conclusions and recommendations of this report should not be considered valid unless such changes are reviewed and the AECOM engineer of record modifies or verifies the conclusions of this report in writing. AECOM is not responsible for any claims, damages, or liability associated with the reinterpretation or reuse of the subsurface data or engineering analyses in this report by others.

7. Closure

AECOM thanks you for the opportunity to be of service on this project. Please contact AECOM Project Manager Bryan Huschka (bryan.huschka@aecom.com) or Markus Walbaum should questions arise or additional information is required.

Yours sincerely,



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Stoney Mather (AECOM)
Ruifen Liu, PE (AECOM)
Audra Rodgers, PE (AECOM)

Attachments

A1 – Axial Resistance Curves (East Abutment)

A2 – Axial Resistance Curves (West Abutment)

A3 – Neutral Plane Output Plots (East Abutment)

A4 – Neutral Plane Output Plots (West Abutment)

Attachment A1 – Axial Resistance Curves (East Abutment)

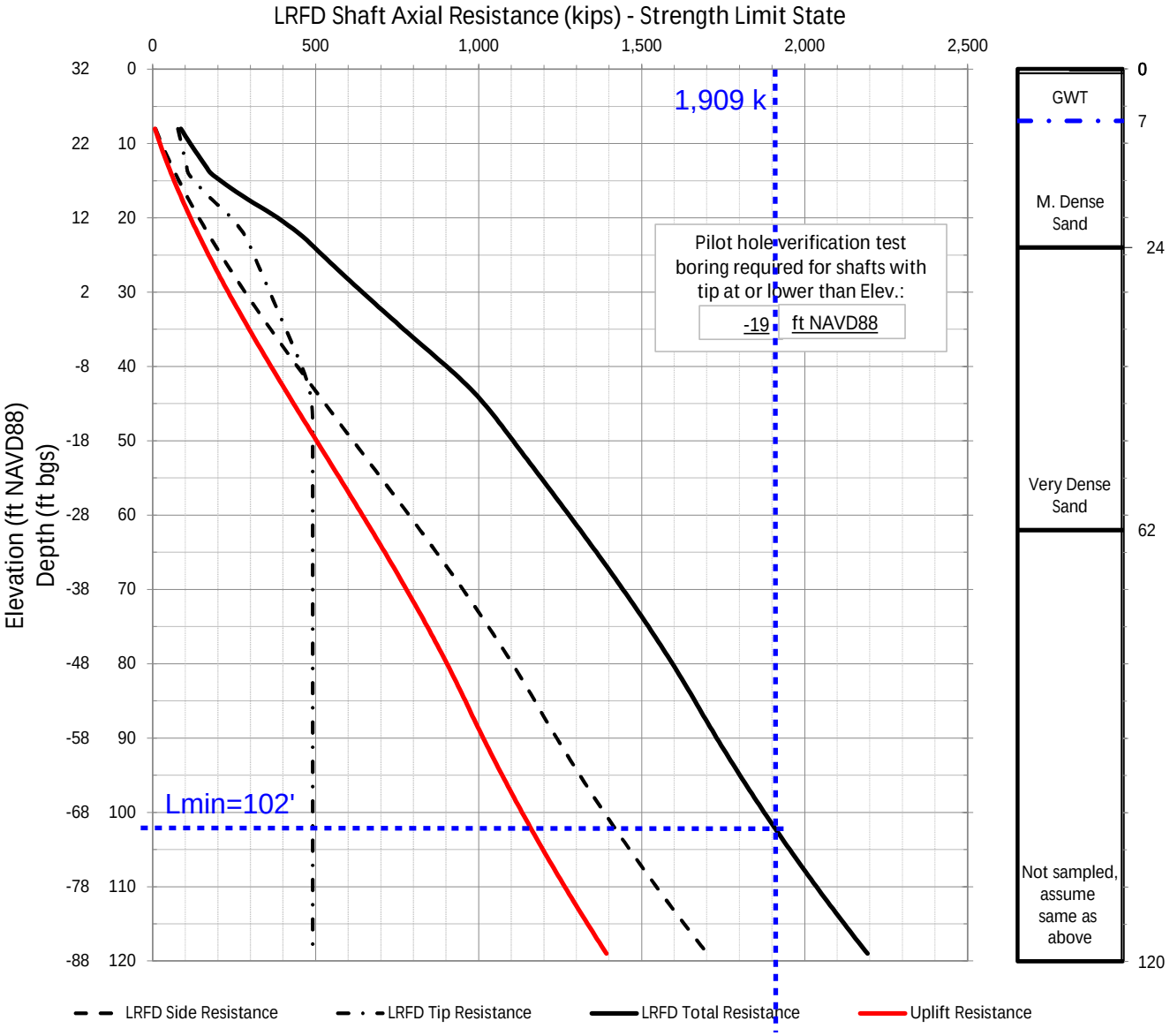
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **East Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

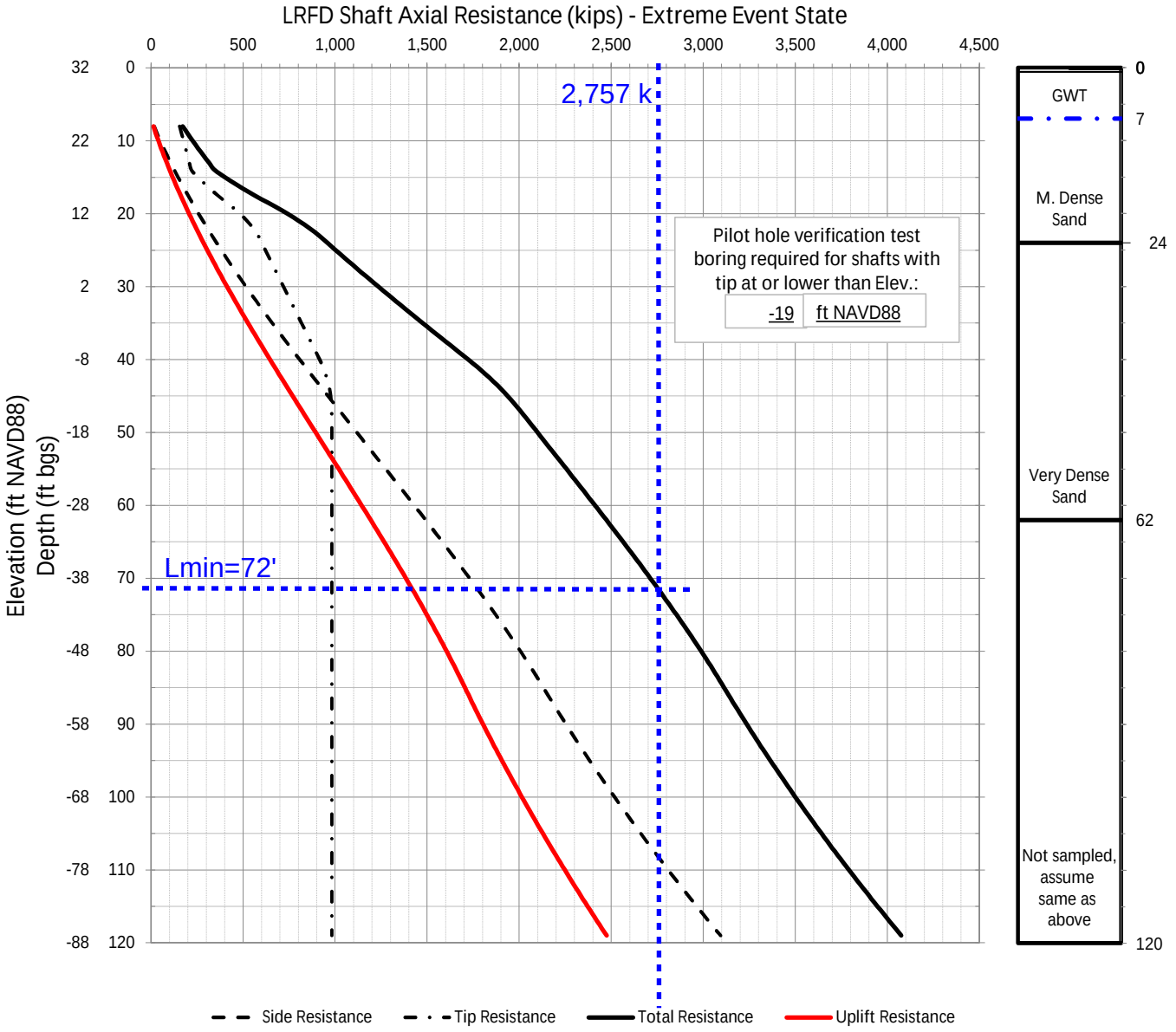
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **East Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88
Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (φ) of 1.0 for compression resistance and 0.80 for uplift resistance.

AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

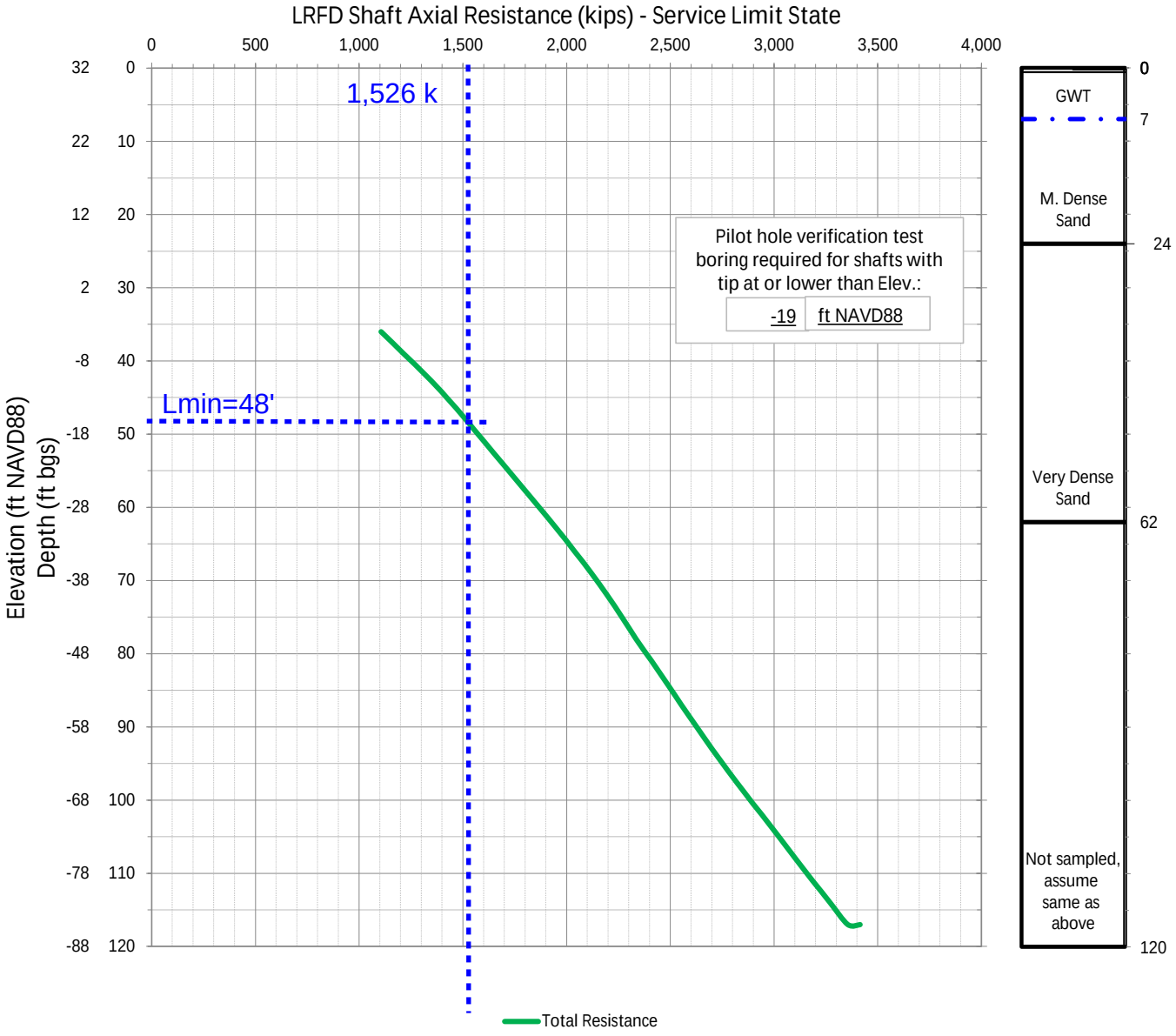
Subject: Axial Capacity - Service Limit State: **East Abutment**

5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 32.0 ft NAVD88

Depth to Groundwater = 7.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

Attachment A2 – Axial Resistance Curves (West Abutment)

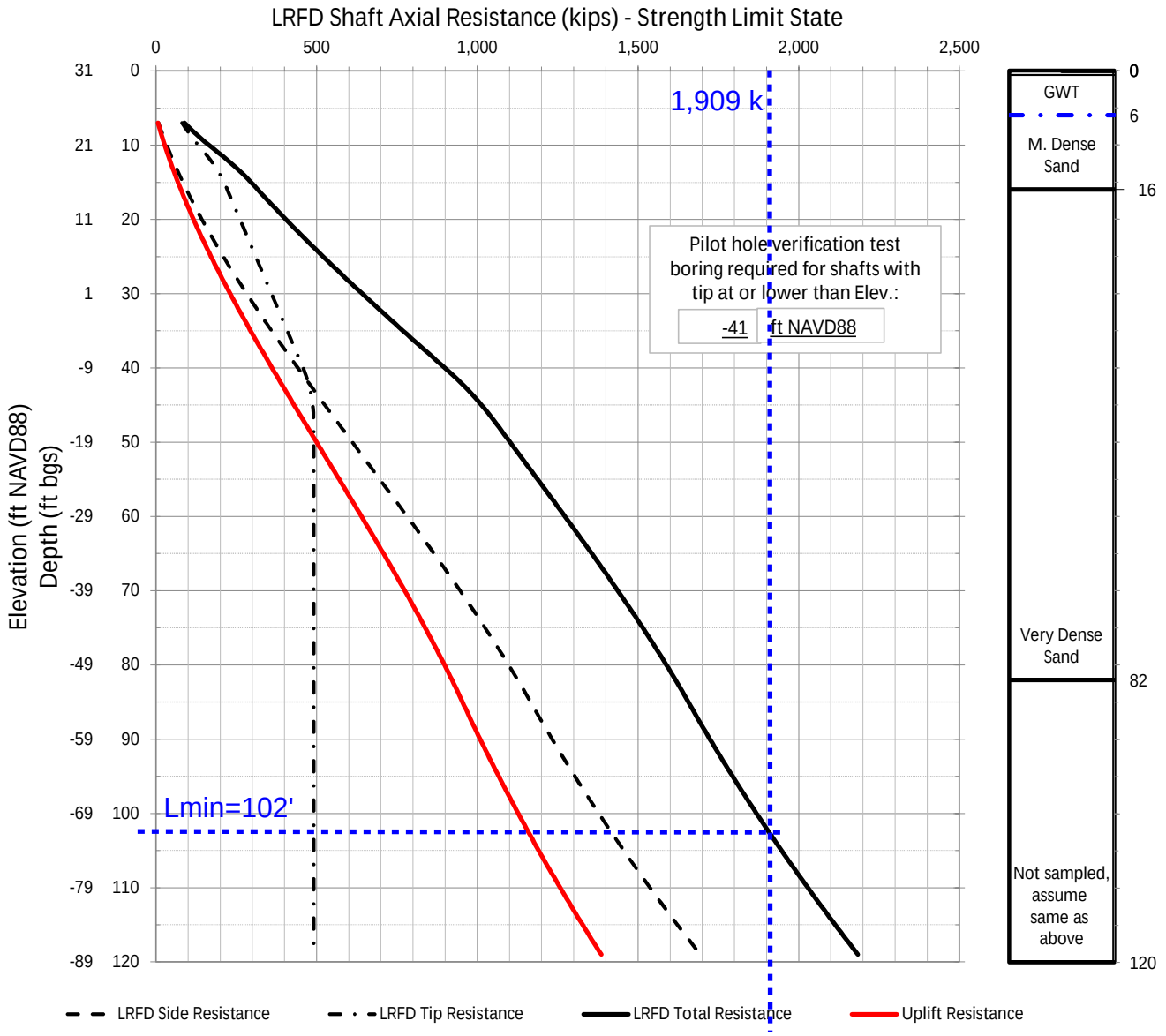
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Strength Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. The axial capacities are applied with resistance factors per 2020 AASHTO LRFD BDS.
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.
4. End bearing is considered assuming that no weak layer is present within two shaft diameters below the design tip and assuming proper method is utilized to achieve and inspect bottom cleanliness of the drilled shaft.
5. Uplift resistance is calculated from side resistance only. Weight of the pile is not included in uplift resistance.
6. Capacity curves consider resistance factors (ϕ) of 0.55, 0.50, and 0.45 for compression side resistance, tip resistance, and uplift resistance, respectively

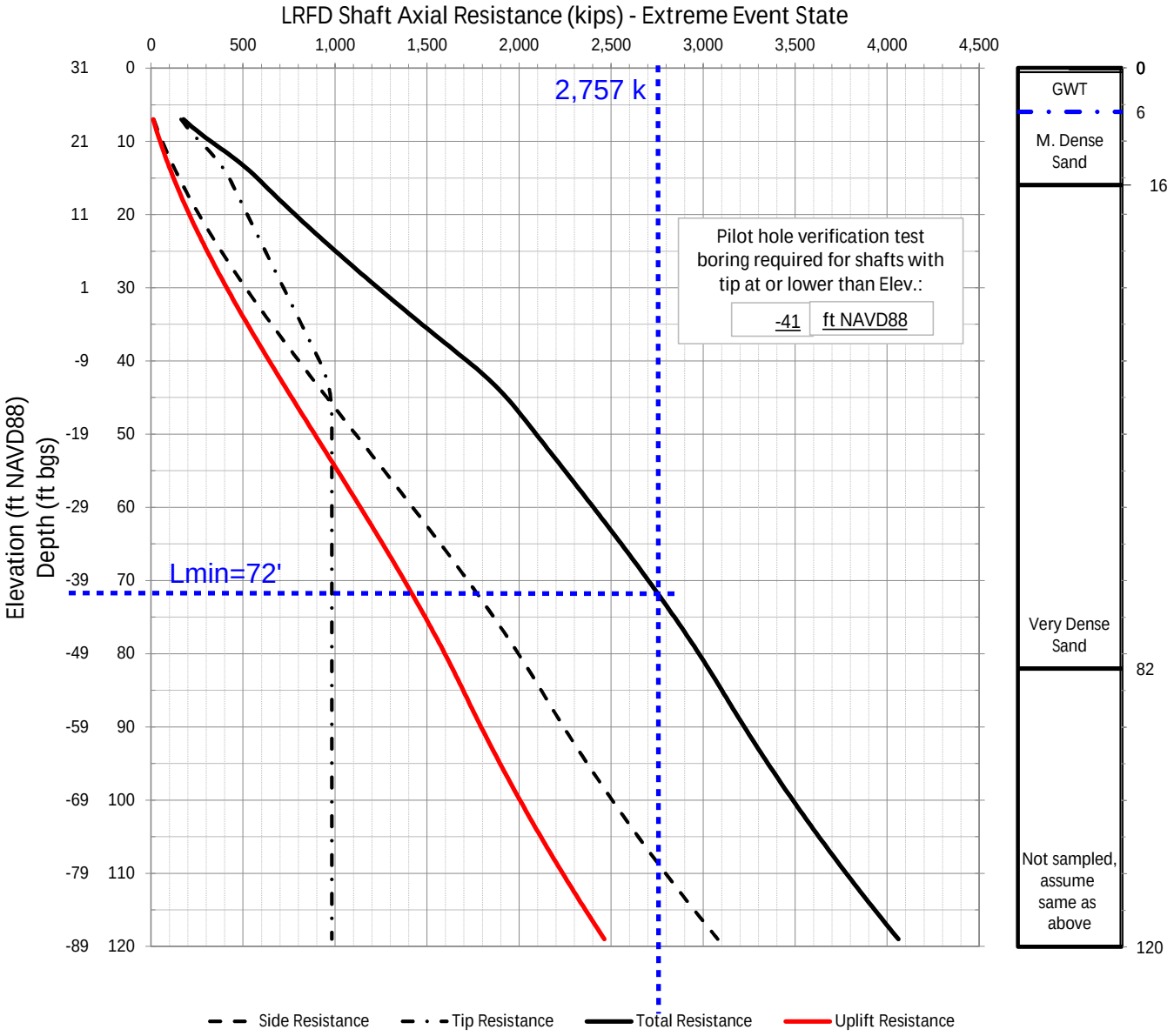
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Extreme Event Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft



Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
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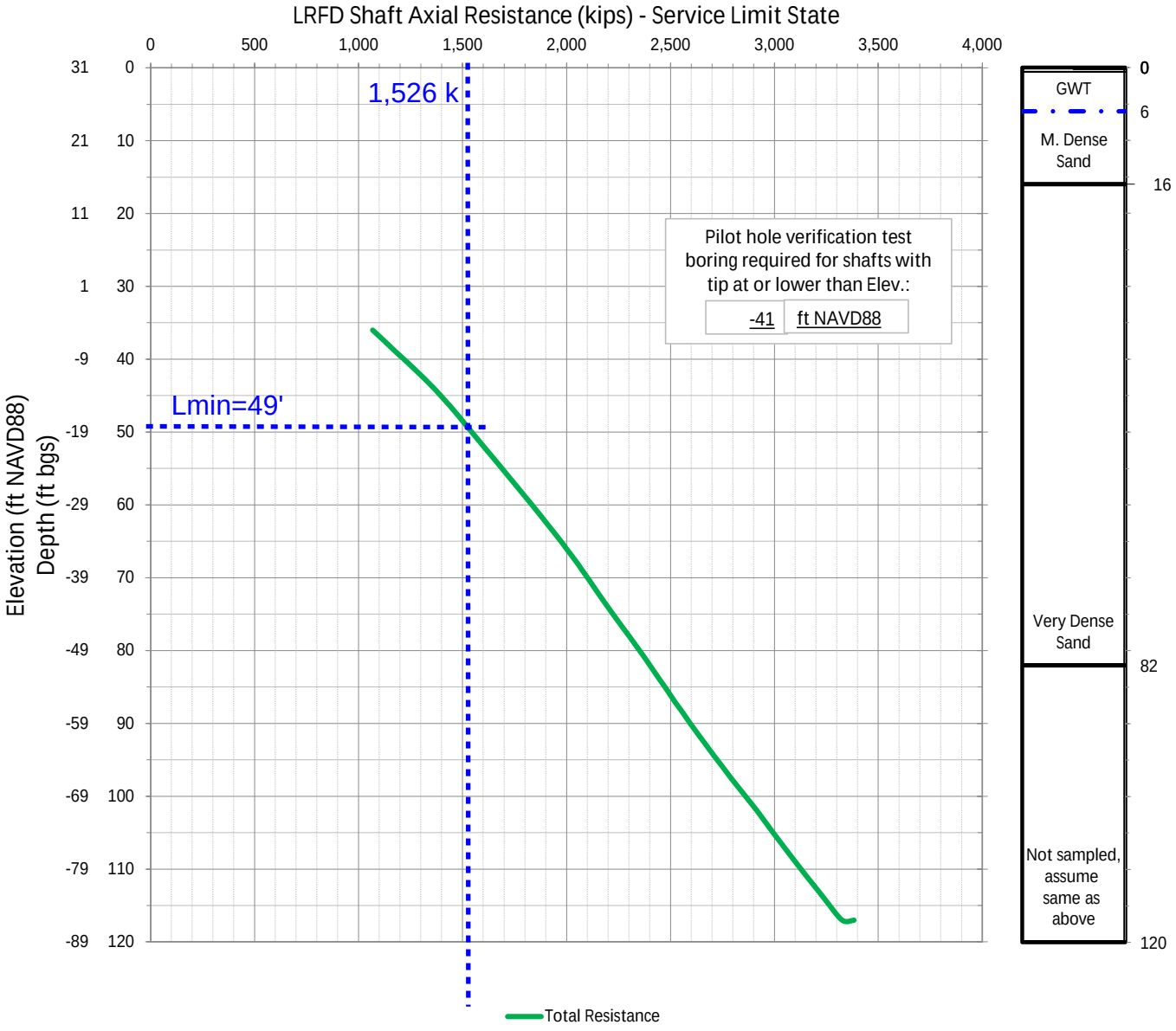
AECOM

Project: Woodinville-SR202 Widenning and Trestle Replacement

Subject: Axial Capacity - Service Limit State: **West Abutment**
5.0 ft Diameter Drilled Shaft

Performed By:	L. Finnefrock
Date:	6/10/2025
Checked By:	Y. Soltanpour
Date:	6/12/2025

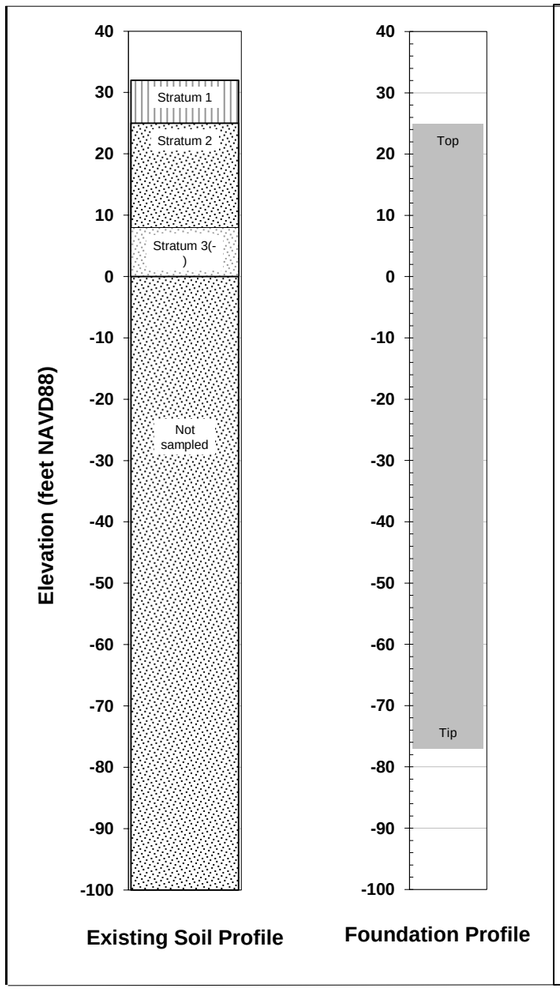
Design Ground Elevation = 31.0 ft NAVD88
Depth to Groundwater = 6.0 ft

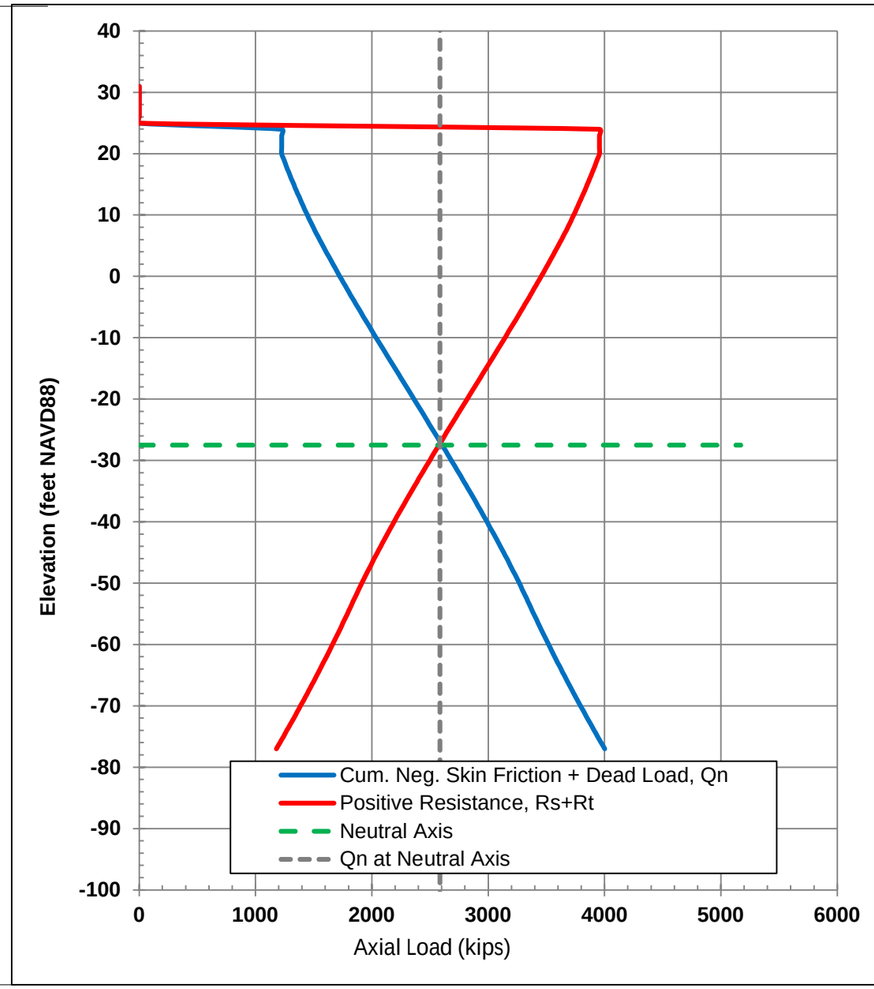


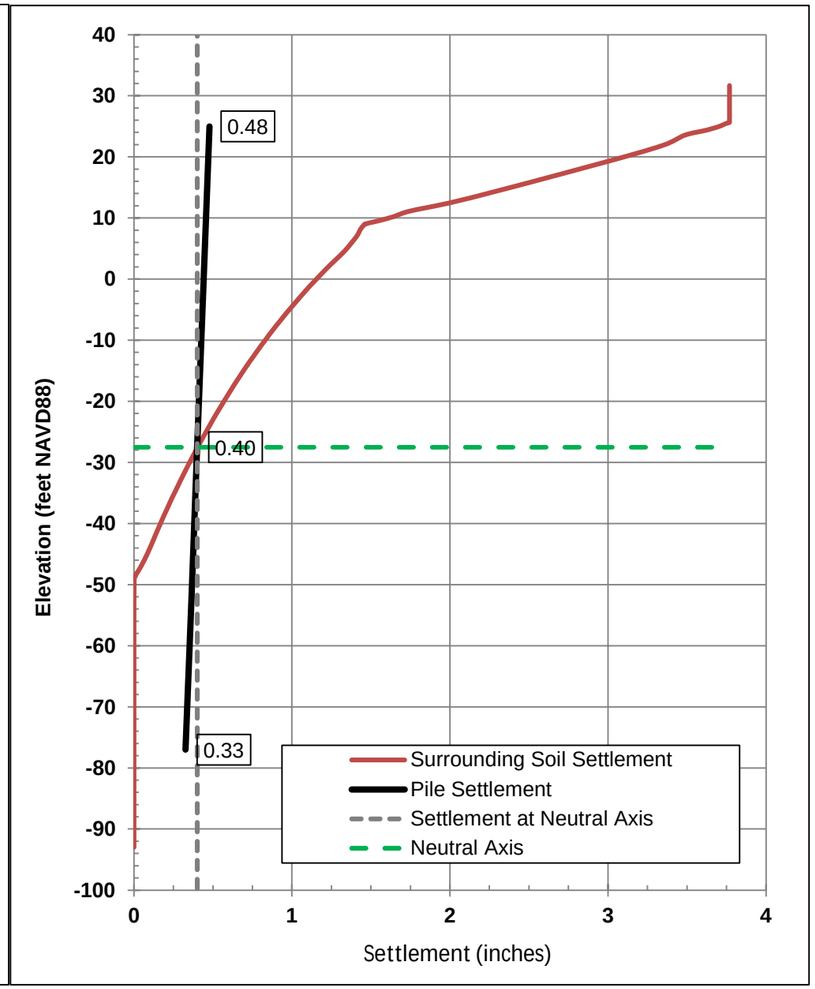
Notes:

1. Assumes bottom of shaft cap at El. +25 ft NAVD88. No exclusion zone for axial capacity in upper part of shaft is necessary for project per AASHTO.
2. Service limit state axial capacity is calculated based on 1 inch movement of drilled shaft tip
3. Capacity curves shown are for single isolated drilled shaft. Group reduction shall be applied per AASHTO BDS or Project Criteria.

Attachment A3 – Neutral Plane Output Plots (East Abutment)







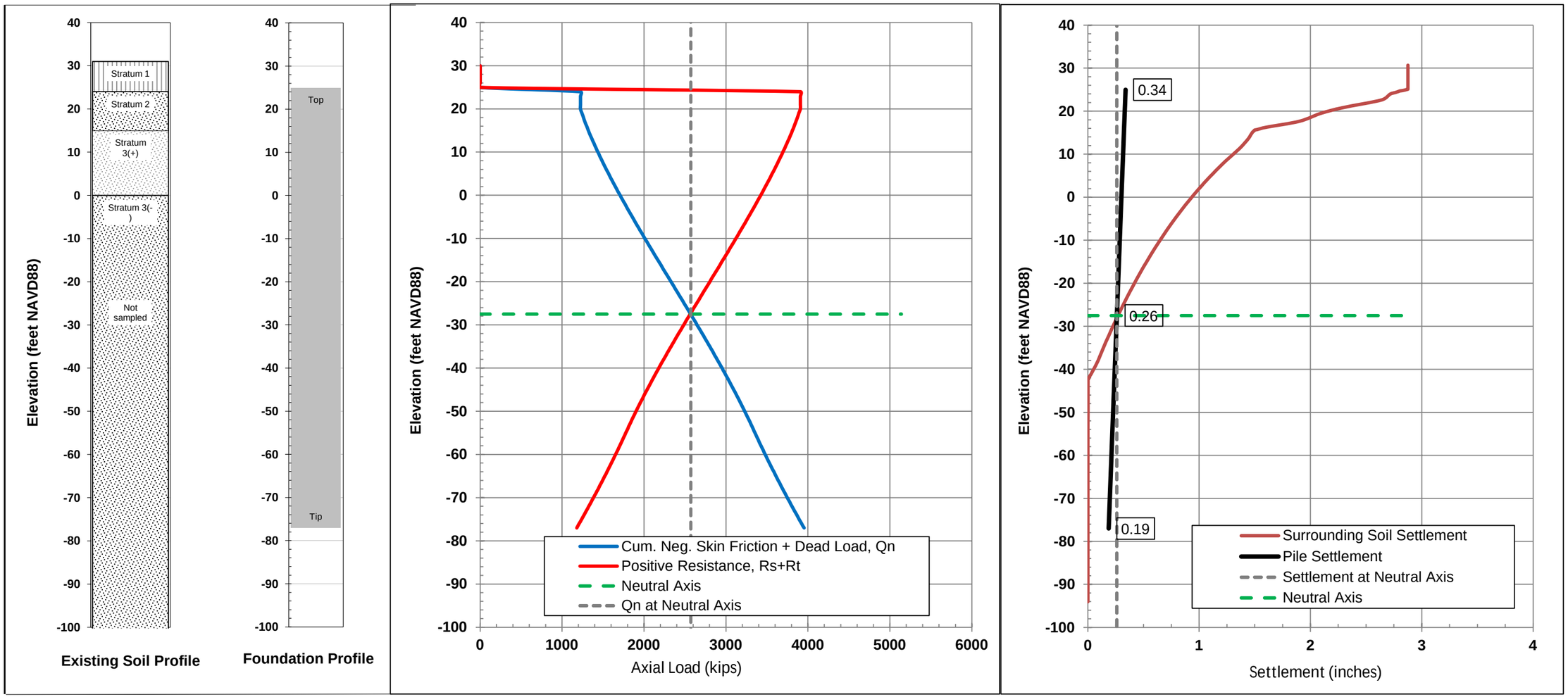
General Notes:

1. Neutral Axis Elev: -27.5 ft NAVD88
2. Qn at Neutral Axis: 2,585 kips
3. Applied Dead load: 1,224 kips
4. Consol. Drag load: 1,361 kips
5. All loads and resistances are nominal (unfactored). Appropriate load and resistance factors should be applied to these values.
6. Rs = Cumulative Skin Friction Resistance Summed from Pile Bottom
7. Rt = Tip Resistance at Pile Bottom
8. Qn = Downdrag Skin Friction + Dead Load

FIGURE NO. A3 (East)

Neutral Axis Method for Estimating Consolidation Downdrag on Deep Foundations

Attachment A4 – Neutral Plane Output Plots (West Abutment)



General Notes:

1. Neutral Axis Elev: -27.5 ft NAVD88
2. Qn at Neutral Axis: 2,570 kips
3. Applied Dead load: 1,224 kips
4. Consol. Drag load: 1,346 kips
5. All loads and resistances are nominal (unfactored). Appropriate load and resistance factors should be applied to these values.
6. Rs = Cumulative Skin Friction Resistance Summed from Pile Bottom
7. Rt = Tip Resistance at Pile Bottom
8. Qn = Downdrag Skin Friction + Dead Load

FIGURE NO. A4 (West)

Neutral Axis Method for Estimating Consolidation Downdrag on Deep Foundations