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GEOTECHNICAL DESIGN REPORT

BRIDGES BRIDGE NO. 2102
MAINE DOT WIN 25105.00
WILTON, MAINE

August 2024
09.0026188.01

Prepared for:
Maine Department of Transportation
Augusta, Maine

Prepared by:
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VIA EMAIL

August 30, 2024
File No. 09.0026188.01

Ms. Laura Krusinski
Maine Department of Transportation
16 State House Station
Augusta, Maine 04333-0016

Re: Geotechnical Design Report
Bridges Bridge No. 2102
MaineDOT WIN 25105.00
Pond Road, Wilton, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the proposed Bridges Bridge No. 2102 in Wilton, Maine. Our work was completed under GZA GeoEnvironmental, Inc.'s (GZA's) June 3, 2020 General Consulting Agreement (GCA CTM202006030000000000709) with the Maine Department of Transportation (MaineDOT) Bridge Program, and incorporates GZA's Proposal No. 09.P000096.24, dated February 16, 2024, and the *Limitations* Included in **Appendix A** of this report. HNTB is serving as the bridge designer for MaineDOT.

It has been a pleasure serving MaineDOT on final design of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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Attachment: Geotechnical Design Report

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1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the replacement of bridge No. 2102 carrying Pond Road over Wilson (Coos) Stream in Wilton, Maine. Our services were completed in accordance with GZA's June 3, 2020 General Consulting Agreement (GCA CTM20200603000000000709) with the Maine Department of Transportation (MaineDOT) Bridge Program, and incorporates GZA's Proposal No. 09.P000096.24, dated February 16, 2024, and the *Limitations* Included in **Appendix A** of this report. HNTB is serving as the bridge designer for MaineDOT and our geotechnical design evaluations and recommendations have been coordinated with HNTB throughout final design.

1.1 BACKGROUND

The project includes replacement of the Bridges Bridge No. 2102 carrying Pond Road over Wilson (Coos) Stream in Wilton, Maine. The project location is shown on **Figure 1**. The existing bridge is a single span bridge with a span length of approximately 44 feet. The bridge was built in 1928 and consists of a steel beam and concrete bridge deck with bituminous overlay supported on concrete abutments and wing walls that bear on two rows of driven timber piles. The as-built plans do not indicate pile lengths.

The preferred bridge alternative includes a single-span, pile-supported, integral abutment bridge (IAB). The proposed replacement will be on the existing alignment, as shown on **Figure 2.** The new bridge will consist of a 75-foot-long, single span, 7-degree-skew, IAB with three concrete NEXT beams and approximately 12.2-foot-high backwalls and in-line wingwalls. The new abutments are anticipated to be designed with integral abutment substructures supported on driven, end-bearing H-piles. 1.75 horizontal to 1 vertical (1.75H:1V) riprapped slopes are proposed in front of the abutments. Elevations referenced herein are in feet and are referenced to the North American Vertical Datum of 1988 (NAVD88).

A full closure and temporary detour are planned for the duration of construction. Therefore, staged construction and a temporary roadway are not required.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and to provide geotechnical design recommendations for the proposed bridge. To meet these objectives, GZA completed the following Scope of Services:

- Conducted site visits to observe surficial conditions and reviewed mapped surficial and bedrock geology of the site;
- Reviewed subsurface data collected by GZA during the preliminary subsurface exploration program, which consisted of two test borings;
- Coordinated and observed a supplemental subsurface exploration program, consisting of one test boring and two seismic Cone Penetration Tests (sCPTs), to further evaluate subsurface conditions for the proposed bridge;
- Conducted a laboratory testing program to evaluate engineering and index properties of the site soils;

- Conducted geotechnical engineering analyses for soil properties; frost susceptibility; AASHTO LRFD load and resistance factors associated with geotechnical design elements; stability and settlement of approach embankments; nominal resistance of pile foundations; lateral pile design considerations; pile drivability; lateral earth pressures on abutments, and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for driven piles, lateral earth pressures, and seismic design parameters; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary design exploration program in 2023 consisting of two test borings designated as BB-WWS-101 and -102, and a final design supplemental exploration program in 2024 consisting of one boring designated as BB-WWS-201 and two sCPTs designated as sCPT-WWS-201 and -202.

Borings were drilled using 3- and 4-inch casing, and drive- or spin-and-wash drilling techniques, as noted on the boring logs. Standard penetration testing (SPT) and split spoon sampling were performed continuously or at standard 5-foot intervals using a 24-inch-long, 1-3/8-inch inside diameter sampler. The borings were generally backfilled with ¾-inch crushed stone and/or soil cuttings and topped with asphalt cold patch. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**. Additional details of each program are described below.

The as-drilled boring locations were surveyed by MaineDOT and provided to GZA and are shown on the logs and on **Figure 2.**

2.1 PRELIMINARY DESIGN BORINGS

Borings BB-WWS-101 and BB-WWS-102 were drilled between May 3 and May 4, 2023, by New England Boring Contractors of Hermon, Maine. The borings were completed using a Mobile B-53 drill carried on a CME track-mounted rig and were drilled to depths ranging from approximately 45 to 47 feet below ground surface (bgs). Approximately 10.5 to 12 feet of bedrock core was collected in these borings. SPTs were conducted using automatic hammer NEBC No. D20, which had a rated hammer energy transfer ratio of 0.742 at the time of drilling.

2.2 FINAL DESIGN PHASE BORING

Boring BB-WWS-201 was drilled on April 22, 2024, by Seaboard Drilling, LLC (Seaboard) of Bangor, Maine. The boring was drilled at the proposed abutment location ten feet behind the abutment, using a track-mounted Mobile B-53 drill rig. The boring was drilled to a depth of 47 bgs and terminated 10 feet into bedrock. SPTs were conducted using automatic hammer Seaboard No. 367, which had a rated hammer efficiency factor of 1.066 at the time of drilling.

2.3 SEISMIC CONE PENETRATION TESTING

GZA retained Seaboard to complete two sCPTs. sCPT-WWS-101 and sCPT-WWS-102 were performed on April 23, 2024.

The as-drilled sCPT locations were surveyed by MaineDOT and are shown on **Figure 2**.

The sCPTs were performed in accordance with ASTM D5778. They were advanced using an ATV-mounted Diedrich D-50 with a Vertek digital cone. The sCPTs were predrilled through the fill materials to depths of 10 to 14 feet bgs. sCPT-WWS-101 was advanced to refusal at a depth of 34 feet bgs and sCPT-WWS-102 was advanced to refusal at a depth of 24 feet bgs. Parameters obtained include cone resistance (qc), sleeve friction (fs), and piezocone pore pressure (u_2). Downhole shear wave velocity measurements were also taken at approximately 1-meter intervals using a surface impact source and a geophone near the cone to calculate shear wave velocity (V_s) at each test depth.

A data report was prepared by Seaboard on May 4, 2024 for sCPT-WWS-201 and sCPT-WWS-202, and is included in **Appendix C**. Seaboard also provided GZA with Excel files containing the raw sCPT data for use in our engineering evaluations.

GZA utilized the analytical software *CPetIT* by Geologismiki to develop reports of correlated soil types and engineering properties based on the raw data provided by Seaboard. These reports are included in **Appendix C**.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering of Cranston, Rhode Island, to assess the gradation and index properties of the soil and bedrock. The testing program included:

- Thirteen (13) gradation analyses / MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments;
- Thirteen (13) moisture content tests;
- One (1) Atterberg limits analysis; and
- Two (2) hydrometer tests.

Results of the testing are included in **Appendix D**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping[1], the surficial units in the vicinity of the site consist of artificial fill, stream alluvium, glaciofluvial, and glacial outwash areas. Artificial fill was placed over the alluvium deposits during development of the existing bridge approaches. Alluvium is described as sand, gravel, and silt. The glaciofluvial is described as sand and gravel deposits created by fluvial reworking of glacial deposits. Glacial outwash is described as sand, gravel, and silt deposited by glacial meltwater flowing down the Wilson Stream valley.

Based on available bedrock geologic mapping[2], bedrock in the vicinity of the site consists of dark grey, bedded sulfidic and graphitic quartz-rich granule metaconglomerate, metasandstone, metasiltstone, and metapelite and is mapped as the Temple Stream Member (Dst). The Fall Brook Formation (DSf) is mapped to the south of the site and consists of thickly bedded, purplish grey, calcareous metasandstone.

4.2 SUBSURFACE PROFILE

Three soil units were encountered beneath approximately 6 inches of asphalt pavement and above bedrock at the site: Fill, Alluvium, and Glacial Till. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface.

---

[1] Spigel, Lindsay J., 2018, Surficial geology of the East Dixfield quadrangle, Maine: Maine Geological Survey, Open-File Map 18-18, map, scale 1:24,000. *Maine Geological Survey Maps*. 2076. https://digitalmaine.com/mgs_maps/2076

[2] Pankiwskyj, Kost A., 1978, Reconnaissance bedrock geology of the Dixfield [15-minute] quadrangle, Maine: Maine Geological Survey, Open-File Map 78-15, map, scale 1:62,500. Maine Geological Survey Maps. 301. http://digitalmaine.com/mgs_maps/301

Soil Unit	**Approximate Encountered Thickness (ft)**	**Generalized Description**
Fill	9.6 to 13	Brown, loose to medium dense, fine to coarse SAND, trace to little silt, trace to some gravel. Contains cobbles and boulders. (USCS: SM, SP-SM, SW-SM). Typical MaineDOT Frost Classification Range= 0 to II *Encountered in all borings*
Alluvium	9 to 13.5	Stratified with layers varying <u>from</u> brown, loose to medium dense, Sandy GRAVEL, trace silt, <u>To</u> silty fine to coarse SAND, little gravel to gravelly, with layers of SILT <u>To</u> Silty CLAY with fine sand. (USCS: CL, ML, SM, GW, SP-SM). Typical MaineDOT Frost Classification = II to IV *Encountered in all borings; silty clay encountered only in BB-WWS-101 and sCPT-WWS-101.*
Glacial Till	5.5 to 13.5	Grey, medium dense to very dense, fine to coarse SAND, some silt, some gravel. Contains cobbles and boulders. (USCS: SM). Typical MaineDOT Frost Classification = II to III *Encountered in borings BB-WWS-102 and BB-WWS-201.*
Estimated Top of Bedrock	Abutment 1: Approx. El. 547.3 (34.5 feet bgs) Abutment 2: Approx. El. 547.5 to 545.1 (34.5 to 37.0 feet bgs)	

4.2.1 Bedrock

Bedrock cored in each test boring was identified as a Schist and was described as hard to very hard, fresh, aphanitic to fine grained, and grey. The primary joints are extremely close to moderately spaced, moderately dipping to high angle, planar to undulating, smooth to rough, fresh to discolored, very tight to open. The secondary joints are close to moderately spaced, horizontal to low angle, planar, rough, fresh to discolored, tight to open. The Rock Quality Designation (RQD) in the schist ranged from 0 to 86 percent, corresponding to a RQD of Very Poor to Good. Core photographs are presented in **Appendix F**.

4.2.2 Groundwater

The groundwater level was measured in the completed borings after drilling at depths of 9.2 to 10.8 feet below existing grade, corresponding approximately to El. 571.9 to 573.6. Water levels measured in the borings were likely influenced by the addition of drill water during rotary wash drilling. The groundwater level was interpreted from the sCPTs to be at a depth of 10.5 feet, corresponding approximately to El. 571.1 to El. 571.6.

Fluctuations in groundwater levels will occur due to variations in season, precipitation, stream levels and construction activity in the area. Consequently, water levels during and after construction are likely to vary from those encountered in the borings and sCPTs at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix E**.

5.2 PROPOSED CONSTRUCTION

We understand that a full bridge replacement is planned for the project. The preferred alternative includes an on-alignment bridge. The new integral abutments will be located approximately 14 feet beyond the existing abutments. A full closure and temporary detour are planned for the duration of construction.

5.3 APPROACH EMBANKMENTS

The proposed bridge includes a grade raise of 1 to 2 feet at the centerline, which include 3 to 4 feet of widening beyond the existing crest and corresponds to a maximum of 4 feet of new fill in the widened portions of the slope. The approach embankments are proposed with typical side slope angles of 2H:1V or flatter except for the ground surface in front of each abutment and side slopes within 25 feet of the abutments, which will slope down to the river level at an inclination of approximately 1.75H:1V and will be protected by riprap.

We anticipate that the embankments will be constructed over primarily very loose to medium dense Alluvium or dense Glacial Till supporting soil. Due to the typical strength and low compressibility, embankment settlement and global stability are not considered to be concerns for Abutment 2, however, due to the loose alluvium deposit underneath the Abutment 1, that approach embankment was evaluated in more detail, as discussed herein.

5.3.1 Settlement

We anticipate that settlement from the planned 2 feet of new fill will occur elastically or as recompression as the fill is placed. Therefore, post-construction settlement is anticipated to be negligible.

5.3.2 Global Stability

GZA interpreted the generalized soil profile at Abutment 1 and summarized the results in the table below. The strength parameters for each soil unit were developed using the corrected SPT N-values and soil types encountered in the borings. Supporting calculations are presented in **Appendix E**.

GENERALIZED SUBSURFACE CONDITIONS, PROPOSED APPROACH FILLS			
Soil Unit	**Total Unit Weight (pcf)**	**Effective Friction Angle (deg)**	**Estimated Thickness[1] (feet)**
Fill (Sand, Gravel)	125	32	12
Alluvium	120	30	22

Note:
1. Estimated thickness is measured from final approach embankment pavement grades.

GZA completed stability analyses to assess the factors of safety against transverse rotational instability of the proposed Abutment 1 approach (Station 3+25) slope modifications. We used the computer analytical software *Slope/W 2020*, developed by GeoSlope International, based on the Morgenstern-Price method.

The typical proposed 1.75H:1V riprap side slopes were modeled. A grid and radius search technique was used to identify the slip surface with the lowest calculated factor of safety. The search limits, slope geometry and results of analyses are included in **Appendix E**. The results indicated that the factor of safety was in excess of 1.3 for the typical 1.75H:1V or flatter Abutment 1 approach embankments, which is suitable for support of embankments without structures.

The calculated factor of safety is 1.4 in the longitudinal direction, which is less than 1.5 which is required for a slope that supports a structure. However, additional lateral resistance is available from the abutment piles, and the width of the embankment is relatively narrow and does not approach the plane-strain condition that is modeled in *Slope/W*. Based on these factors and our experience, it is our opinion that the global stability in the longitudinal direction is acceptable.

Seismic

A pseudostatic analysis was conducted at Abutment 1 including earthquake-loading based on the Newmark method, by incorporating a horizontal seismic coefficient, k_h, defined as half of the maximum peak ground acceleration at the ground surface (0.066 g). This value is justified by the likely development of some degree of deformation of the overall soil mass above the slip surface, in accordance with AASHTO LRFD Article 11.6.5.2.2. For this analysis, the stability is evaluated for the slip surface that is shown to have the lowest safety factor in the static analysis. The calculated pseudostatic factor of safety against rotational failure is greater than 1.0 for the critical slip surface beneath Abutment 1 approach, which corresponds to the lowest static factor of safety for the project, indicating that significant slope deformations are not likely.

5.4 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the approach Fill (including backfill behind abutments), Alluvium, and Glacial Till Deposits overlying bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1, considering the average SPT N-values in granular soils, and the measured shear wave velocities from the sCPTs. Given that the average SPT N-value was 28 blows per foot, and the average shear wave velocity was 990 feet per second, the bridge site should be classified as Site Class D.

Considering the loose alluvial deposit found in the borings and sCPTs, the layer could have potential to be liquefiable, therefore a liquefaction analysis was conducted for the site, as discussed in **Section 5.4.1**.

5.4.1 Liquefaction and Associated Hazards

Liquefaction evaluation is not specifically required in AASHTO for Seismic Design Category (SDC) A bridges. However, Article C6.8 of the AASHTO Seismic Guide Specification indicates that: "…a liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact bridge stability and peak ground acceleration is greater than or equal to 0.15 g."

GZA utilized the U.S. Geological Survey's online Unified Hazard Tool with PGA as the spectral period, a return period of 975 years, and Site Class B/C boundary, which uses published probabilistic seismic data, to evaluate the site-specific earthquake magnitude. The Unified Hazard Tool indicated a mean magnitude 6 earthquake and a mode of 5.1. Considering the largest contribution being close, smaller earthquakes with M=5.5 or less, GZA used a magnitude of 5.5 for the liquefaction susceptibility calculation.

GZA inputted the cone resistance (qc), sleeve friction (fs), and piezocone pore pressure (u2) collected from the sCPTs and the design earthquake and the peak ground acceleration into *CLiq* v.3.4.1.4 by Geologismiki. The *CLiq* calculation methodology used Boulanger and Idriss (2014) empirical method for the analyses. *CLiq* defaults to a factor of safety of 2 for soils that are indicated to have clay-like behavior and be non-liquefiable. There are several thin zones, 1 foot thick or less, as well as a continuous zone between about 20 feet and 27.5 feet in sCPT-WWS-201 and between 19 feet and 20.5 feet in sCPT-WWS-202 that show the clay-like behavior. These zones are consistent with the adjacent borings. The CPT results are indicative of varying fines content and density over short distances in the non-cohesive zones, typical of an alluvial deposit. Therefore, SPT results, which by default average susceptibility over a 12-inch interval every 5 feet, would be expected to be a less reliable indicator of liquefaction susceptibility. Therefore, the borings were not considered for liquefaction potential.

The results indicate the alluvial deposit has a combination of adequate density and fines content to provide a typical *FSliq* of 1.3 or greater in sCPT-WWS-201 and 1.4 or greater in sCPT-WWS-202, with zones less than 6 inches thick with lower *FSliq* values. Therefore, the alluvial layer is judged to have low liquefaction potential based on the sCPT results.

5.5 EVALUATION OF FOUNDATIONS

5.5.1 Foundation Type Assessment

Based on the data from borings, we estimate the top of rock is approximately 34 to 39 feet below finished pavement grade in the vicinity of the abutments. An HP 14x89 driven pile foundation was identified in preliminary design as the preferred alternative for support of the integral abutments based on the encountered conditions.

5.5.2 Load and Resistance Factors

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), and live load surcharge (LS) loads, using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength limit state foundation design.

The recommended LRFD resistance factors for strength limit state design of foundations were derived from LRFD Tables 10.5.5.2.2-1, 10.5.5.2.3-1 and 10.5.5.2.4-1 and are presented in the following table.

GEOTECHNICAL RESISTANCE FACTORS – STRENGTH LIMIT STATE			
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ)**	**AASHTO Reference**
DRIVEN PILES			
Nominal Geotechnical Resistance of Single Pile – Dynamic Analysis	Axial Resistance	0.65	10.5.5.2.3-1

Resistance factors for service and extreme limit state design should be taken as 1.0.

Structural resistance of the piles should be checked at the strength limit state considering a resistance factor ϕ_c=0.50, per AASHTO LRFD Article 10.7.3.2.3 for hard driving condition. Since the piles will be subject to lateral loading, the piles should also be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per LRFD Article 6.5.4.2, the axial resistance factor ϕ_{cc}=0.70 and the flexural resistance factor ϕ_f=1.0 should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).

AASHTO LRFD load factors should be applied to EH, EV and ES loads using the load factors for permanent loads (γ_p) provided in AASHTO Table 3.4.1-2 for strength and extreme limit state design. A load factor of 1.5 may be applied to the passive pressure used to design the integral backwall (end diaphragm) to account for deformation of the backwall into the soil as a result of thermal expansion or contraction of the integral bridge deck, per MaineDOT BDG Section 5.4.2.11.

5.5.3 Pile Design Considerations

Based on our experience within similar soils, we anticipate that the proposed HP14x89 piles will be driven to refusal on or near the top of rock to achieve the required axial geotechnical resistance. The soil profile will consist of medium dense Fill, loose to medium dense Alluvium and dense Glacial Till and is relatively thin. Therefore, limited geotechnical resistance will be provided by the soil profile during driving, likely on the order of 5 to 10 percent of the required nominal resistance.

Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. Axial tensile geotechnical (uplift) resistance was not evaluated because the structural loads provided by HNTB do not include uplift loading on the piles.

By utilizing steel H-piles for support of the abutments, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

5.5.4 Pile Type

The abutments are planned to be supported on ASTM A572, Grade 50 (fy=50 kips per square inch [ksi]) steel HP14x89 piles. Each abutment will include four piles. The piles should be fitted with cast steel driving shoes to limit damage during driving.

5.5.5 Pile Loads

HNTB provided a maximum factored axial load of 360 kips per pile for the strength condition; therefore, piles should be installed to a nominal axial resistance of at least 554 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.65 for piles installed under hard driving conditions. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted on one pile at each abutment during construction, in accordance with AASHTO requirements, to assess nominal geotechnical pile resistance.

5.5.6 Design-Phase Pile Drivability Analysis

GZA completed preliminary wave equation analyses to assess the drivability of an HP 14x89 pile with a nominal geotechnical resistance of 554 kips at the abutments. Analyses were completed using a Delmag D19-32 diesel hammer with a ram weight of 4,000 pounds and a maximum rated energy of 42,440 foot-pounds (ft-lbs). The 24-foot-long pile was assumed to encounter very hard driving conditions on bedrock. The results are summarized below.

SUMMARY OF WEAP ANALYSES					
Pile Analysis and Type	**Embedded Pile Length**	**Driving System**	**Required Nominal Geotechnical Resistance (kips)**	**Max Driving Stress (ksi)**	**Final Penetration Resistance (blows per inch)**
Abutment 1 HP 14x89	24 feet	Delmag D 19-32 (Fuel setting 2, 90% of maximum pressure)	554	39	8

Since the driving stresses do not exceed the limiting driving stress of 45 ksi (0.9 Fy) for ASTM A572 Grade 50 (50 ksi yield stress) steel, and the calculated penetration resistance is within the MaineDOT preferred range of 6 to 15 blows per inch, a similar hammer system is anticipated to be suitable to install the piles to the required nominal resistance noted. Results of the design phase wave equation analyses are provided in **Appendix E**.

5.5.7 Lateral Pile Analysis

GZA developed the soil profile tabulated below based on the soil conditions encountered in the test borings, sCPTs, and laboratory testing results. The ground surface was assumed to be at the ground surface in front of the abutment for thermal contraction and was assumed to be at the bottom of the approach slab for expansion. The pile was an ASTM A572 Grade 50 steel HP14x89 pile oriented for weak-axis bending. The following soil profiles were developed for lateral pile evaluations using *LPile* by Ensoft.

L-PILE® INPUT PARAMETERS ABUTMENT 1: SHORTEST PILE LENGTH APPROX. 24 FT						
Stratum	**Soil Model**	**Top of Layer Elevation (ft-NAVD 88)**	**Layer Thickness (ft)**	**k (pci) / E50**	**ϕ' (deg)/ Su (psf)**	**γ_e (pcf)**
New Fill	Reese Sand	578.2	3	83	32	125
New Fill*	Reese Sand	575.2	2.6	83	32	125
New Fill**	Reese Sand	572.6	1	55	32	62.6
Existing Fill	Reese Sand	571.6	2.8	55	32	62.6
Alluvium	Reese Sand	568.8	7.8	35	30	57.6
Clay	Stiff Clay w/o Free Water	561	7	0.008	500	47.6
Alluvium	Reese Sand	554	6.7	35	30	57.6
Top of Rock	--	547.3	--	--	--	547.3

Notes:
1. Soil strata were modelled after boring BB-WWS-101.
2. *indicates the top layer assume for the contraction evaluation, equal to the top of soil infront of the abutment.
3. ** indicates the top of layer is the approximate ground water elevation based on the boring logs.
4. pci = pounds per cubic inch, deg = degrees, psf = pounds per square foot, γ_T = total unit weight (used above anticipated groundwater level), γ_e = effective unit weight (used below anticipated groundwater), pcf = pounds per square foot.
5. The soil profiles and pile lengths for each abutment are similar. Therefore, GZA evaluated Abutment 1 considering the slightly shorter pile length.

GZA conducted lateral pile analyses based on a maximum thermal deflection of 0.366 inches for contraction and 0.243 inches for expansion, as provided by the bridge designer, HNTB. A fixed-head condition (zero rotation) was assumed and imposed the estimated thermal deflections at the pile head.

L-PILE® RESULTS							
Location	**Axial Load (kips)**	**Deflection (in.)**	**Shear Force for Lateral deflection (kips)**	**Moment at Pile Head (ft-kips)**	**Bending Stress at Pile Head (ksi)**	**Axial Stress at Pile Head (ksi)**	**Total Stress at Pile Head (ksi)**
Abutment 1 (Expansion)	360	0.243	40.4	-117.4	31.9	13.8	45.7
Abutment 1 (Contraction)	360	0.366	33.3	-123.5	33.6	13.8	47.4

The maximum combined stress for the stated load case based on our *L-PILE®* output is 47.4 ksi, which is below the yield stress of the pile.

5.5.8 Lateral Earth Pressures

Thermal expansion of the bridge will cause the backwalls and wingwalls of the integral abutment to move toward the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil.

HNTB provided a maximum expansion deflection of 0.243 inches for use in abutment design. The abutment height is approximately 11.4 feet resulting in a calculated abutment rotation of 0.0018 feet/foot. It is GZA's understanding that recent practice is to utilize The *Massachusetts Department of Transportation LRFD Bridge Design Manual* methodology, which provides the empirical equation, below, to calculate lateral earth pressure coefficient (K) based on the ratio of deflection (δ*t*) and wall height (H).

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$

Design lateral earth pressure recommendations were developed based on this equation, as presented in **Appendix E**, and are provided in **Section 6.2** of this report. AASHTO Commentary C3.10.9.1 specifies that single-span bridges are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

5.5.9 Frost Protection

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,800, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 7.5 feet. However, BDG Section 5.2.1 allows that the embedment of pile-supported integral abutments may be reduced to 4.0 feet for frost protection.

6.0 RECOMMENDATIONS

6.1 SEISMIC DESIGN

The peak ground acceleration coefficient, short- and long-period spectral acceleration coefficients were interpolated from the AASHTO design guide maps (3.10.2.1-1 through -21 as appropriate). Based on the site coordinates, the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years are summarized for the site are as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.13 g
SDs (Period = 0.2 sec)	0.28 g
SD1 (Period = 1.0 sec)	0.11 g

6.2 ABUTMENT AND WINGWALL DESIGN

- Backfill between new abutments and wingwalls and a 1.5H:1V plane extending up from the bottom of the abutment to the pavement subgrade should consist of MaineDOT 703.19 Granular Borrow for

Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:

– Internal Friction Angle of Soil = 32°;

– Soil Total Unit Weight = 125 pcf; and

– Coefficient of Passive Earth Pressure, K_p = 2.06 (use for design of backwalls and wingwalls).

- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center. Alternatively, prefabricated drainage geocomposite material can be placed against the uphill side of abutments, after holes have been created through the backing material at the weep hole locations.

6.3 PILE DESIGN

- The proposed abutments may be supported on HP14x89 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed through a combination of side friction and end-bearing on the bedrock surface.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance of 554 kips, calculated by dividing the maximum factored pile load of 360 kips by a resistance factor of 0.65.
- Preliminary wave equation analyses indicate that the piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 42,000 ft-lbs for the anticipated 24-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel), and with a final penetration resistance of 8 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings (approximately El. 547 Abutment 1, and approximately El. 548 to El. 545 at Abutment 2). A provision is recommended in the drawings for extra pile length to account for variability in the top of rock surface and the potential for piles to penetrate a short distance into the bedrock.
- We recommend that the first pile be dynamically tested at each abutment during initial driving to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require a 24-hour restrike test on each test pile, to assess potential relaxation.

- Piles shall be spliced in accordance with MaineDOT Section 501.047.
- Piles should be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per LRFD Article 6.5.4.2, the axial resistance factor ϕ_{cc}=0.7 and the flexural resistance factor ϕ_f=1.0 should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).
- Approach slabs should be constructed at each abutment to smooth the transition from the approach embankment to the bridge. The slabs should be positively connected to the backwalls.

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during pile installation, excavation, dewatering, and foundation subgrade preparation and protection. These items are discussed in the paragraphs that follow.

7.1 ABUTMENT PILE INSTALLATION CONTROL

We recommend that the H-pile installation be controlled using wave equation analysis of the contractor's proposed driving system, field logging of the pile installation, and determination of final penetration resistance based on dynamic pile testing with signal matching analysis.

AASHTO Table 10.5.5.2.3-1 requires that at least one dynamic load test with signal matching be performed per substructure to use a resistance factor of 0.65. We recommend that the first pile be dynamically tested at each abutment during initial driving to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require a 24-hour restrike test on each test pile, to assess potential relaxation.

7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavations for abutment foundations are anticipated to be on the order of 10 feet below existing pavement grades. It is our understanding that Pond Road will be out of service during construction of the new bridge. In areas where sufficient space is available and water conditions permit, the excavation adjacent to the approaches may be constructed with sloped, open cuts. If water levels are higher than the bottom of the excavations, a temporary cofferdam or sand bags and pumping water may be required. In all cases, temporary excavations should comply with Occupational Safety and Health Administration excavation safety requirements.

Considering the proximity of the required abutment excavations to the stream water level, management of water will be related to stream water levels at the time of construction. Considering Q1.1 at El. 574.7 and El. 571.3 to El. 572.3 for the proposed structure, water levels may be at or above the bottom of excavation level during construction, especially if a locally-intense precipitation event occurs. It may be desirable to over-excavate and place an 8- to 12-inch-thick crushed stone working mat to improve accessibility and allow dewatering.

We anticipate that the inflow of groundwater or surface water to excavations could be handled by open pumping from sumps installed at the bottoms of excavations if cofferdams are installed. Stacked sand bags or a porta-dam type system may be sufficient to limit inflow of surface water in normal flow conditions in lieu of a sheet pile cofferdam, given the relatively small anticipated head. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the subgrade and permit concrete placement in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, the four upper fill samples tested had approximately 10 percent passing the No. 200 sieve, indicating the fill will meet MaineDOT specifications for Granular Borrow and is suitable for embankment fill. The existing fill is not suitable for Granular Borrow for Underwater Backfill.

If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.

TABLES

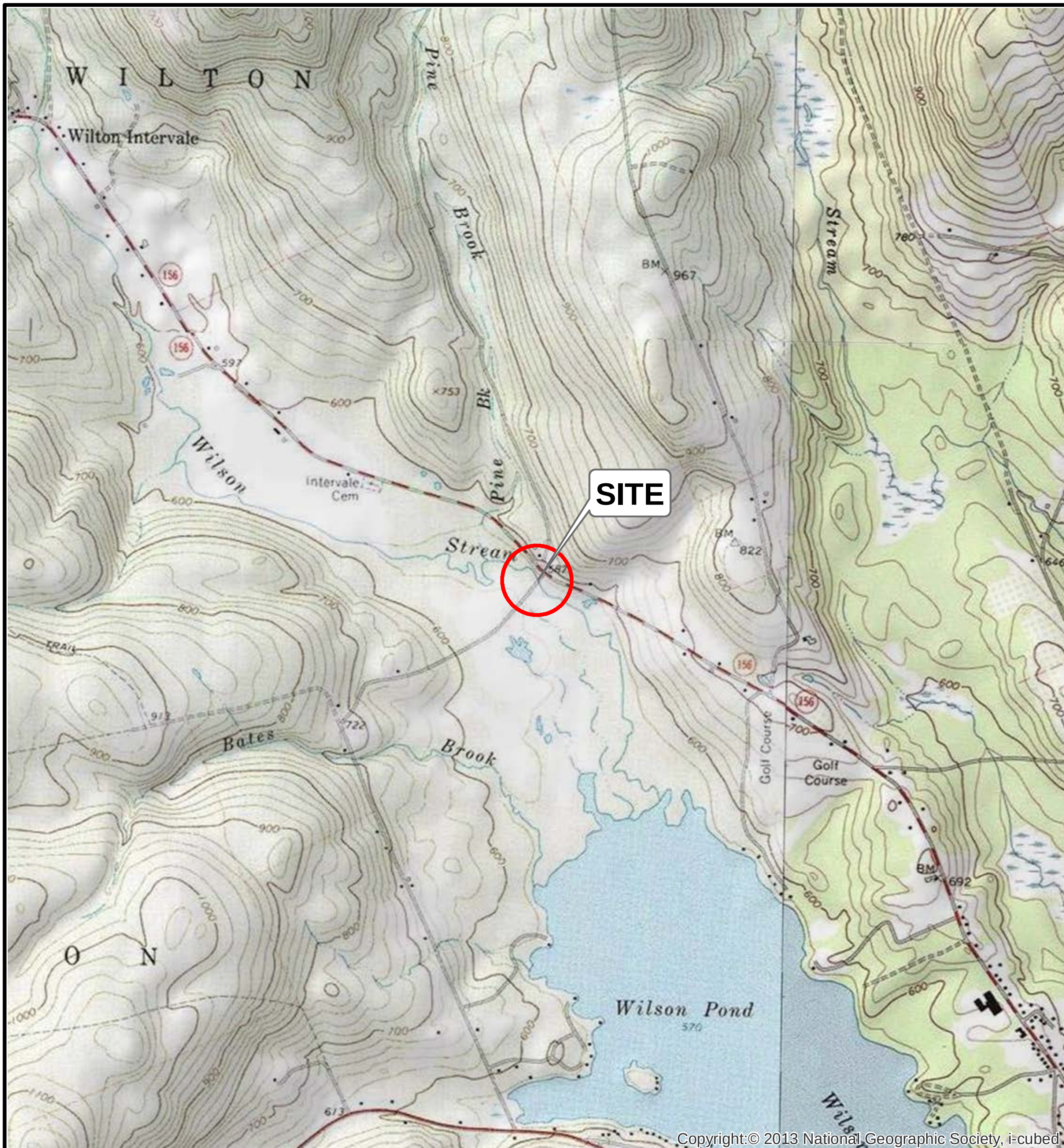
TABLE 1
Summary of Subsurface Explorations
Bridges Bridge No. 2102 Replacement
Wilton, Maine
GZA job#: 09.0026188.01

Exploration ID	Station	Offset (ft)	R/L	Ground Surface El. (ft)	Top of Stratum Elevation				Stratum Thickness (ft)			Depth Bedrock (ft)	Bottom of Exploration Depth (ft)	Bottom of Exploration El. (ft)	Groundwater	
					Fill	Alluvium	Glacial Till	Bedrock	Fill	Alluvium	Glacial Till				El. (ft)	Depth (ft)
BB-WWS-101	3+26.9	6.2	L	581.8	581.8	568.8	NE	547.3	13.0	21.5	NE	34.5	47.0	534.8	572.6	9.2
BB-WWS-102	4+03.7	4.1	R	582.0	582.0	569.0	560.0	547.5	13.0	9.0	12.5	34.5	45.0	537.0	573.6	8.4
BB-WWS-201	3+98.8	6.8	L	582.1	582.1	571.5	559.0	545.1	10.6	12.5	13.5	37.0	47.0	535.1	571.9	10.2
sCPT-WWS-201	3+30.1	5.1	R	581.6	581.6	571.1	NE	NE	10.5	23.5	NE	NE	34.0	547.6	571.1	10.5
sCPT-WWS-202	4+03.6	6.6	L	582.1	582.1	567.8	561.1	NE	14.3	6.7	>5.3	NE	26.3	555.8	571.6	10.5

El. = Elevation, NE = Not Encountered, NM = Not Measured, NP = Not Penetrated, > = Boring Terminated in Stratum

Notes:
1. Refer to the boring logs in Appendix B and the Cone Pentration Test Report in Appendix C for additional information
2. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
3. As-drilled locations and elevations were surveyed by MaineDOT and provided to GZA.
4. Stratum depths, thickness and elevations are rounded to the nearest 0.1 foot as interpreted on the boring logs, but this does not represent the precision of the data.

FIGURES



USGS QUADRANGLE LOCATION

N
W E
S

SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS MAP ALSO CONTAINS THE ESRI ARCGIS ONLINE USA COUNTIES WHICH PROVIDES DETAILED BOUNDARIES THAT ARE CONSISTENT WITH THE TRACT, BLOCK GROUP, AND STATE DATA SETS AND ARE EFFECTIVE AT REGIONAL AND STATE LEVELS.

0 1,000 2,000 4,000 6,000

SCALE IN FEET

Data Supplied by :

PROJ. MGR.: BMC
DESIGNED BY: EAF
REVIEWED BY: BMC
OPERATOR: EAF

DATE: 07-17-2024

LOCUS PLAN

BRIDGES BRIDGE - POND ROAD OVER WILSON (COOS) STREAM
WILTON, MAINE

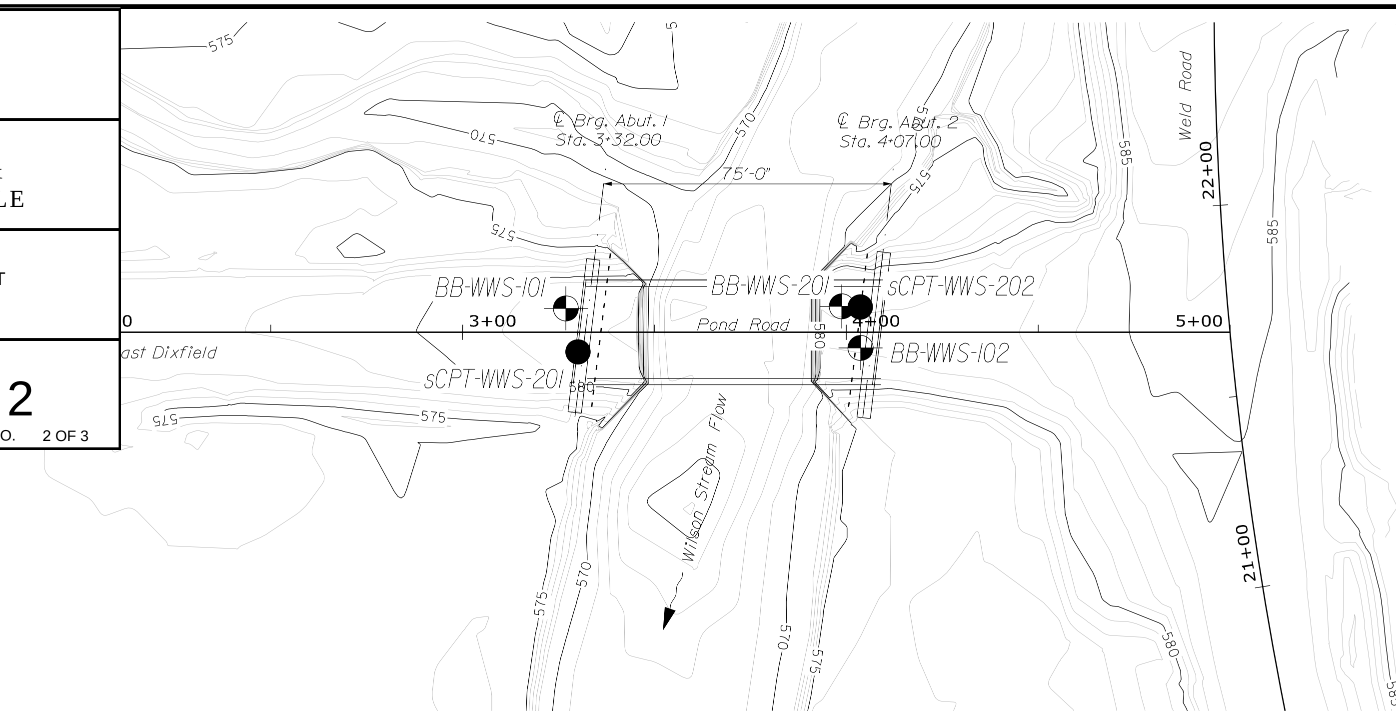
JOB NO.
09.0026188.01

FIGURE NO.
1

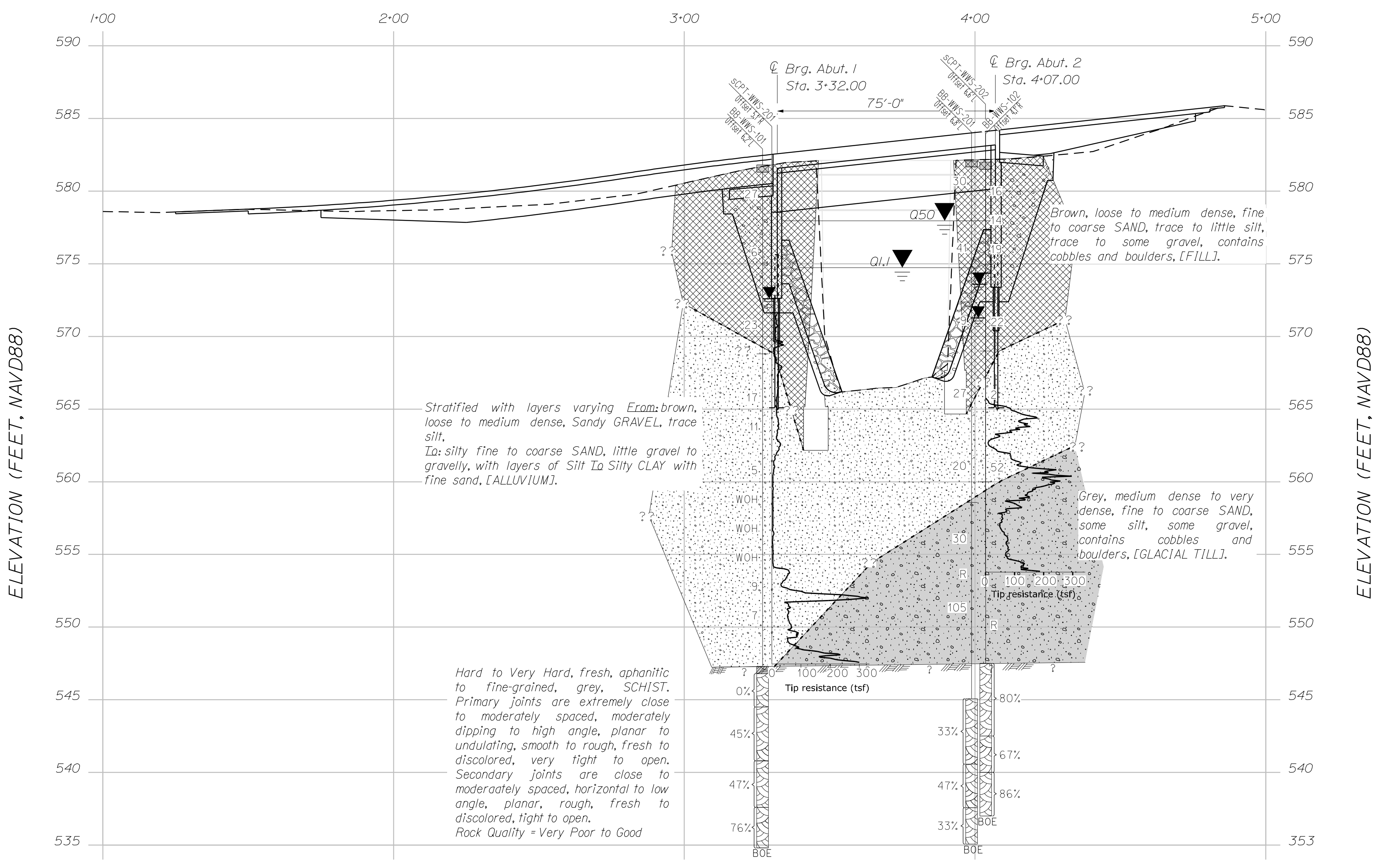
WILTON BRIDGE
MAINEDOT WIN 25105.00
WILTON, ME

BRIDGE BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINEDOT	
PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: ARB	FIG 2
DESIGNED BY: BMC	DRAWN BY: ENT	SCALE: AS SHOWN	
DATE: 8/30/24	PROJECT NO. 09.0026188.01	REVISION NO. 0	SHEET NO. 2 OF 3



BORING LOCATION PLAN



INTERPRETIVE SUBSURFACE PROFILE

NOTES

1) Base map developed from electronic files (Alignments.dgn, Bridge.dgn, BDPLAN.dgn, Countours, Profile.dgn, and Topo.dgn) provided by HNTB on July 21, 2023.

2) The as-drilled locations of the BB-WWS-101 and BB-WWS-102 test borings were surveyed and provided by MaineDOT in an electronic file (Borings.dgn) on July 17, 2023.

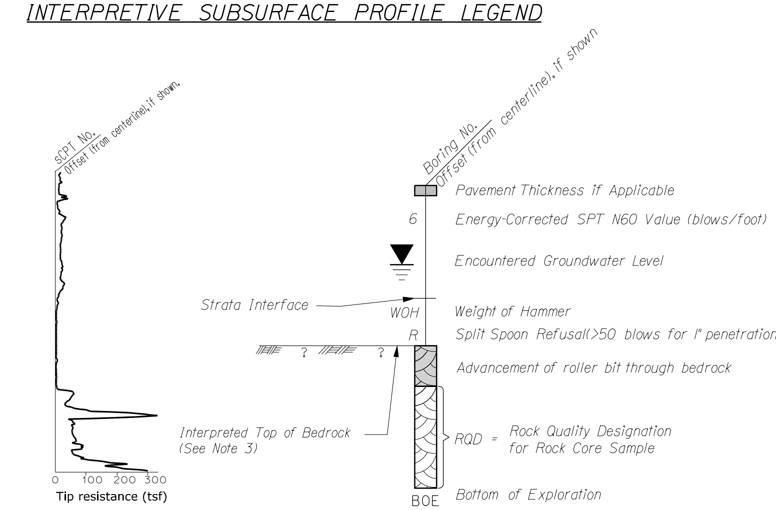
3) The as-drilled locations of the BB-WWS-201 test boring and sCPT-WWS-200 series cone penetration tests were surveyed and provided by MaineDOT in an electronic file (Borings.dgn) on May 3, 2024.

4) This generalized interpretive soil and rock profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil and rock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

BORING LOCATION PLAN LEGEND

BB-WWS-102	Location and designation of BB-WWS-100 series borings that were performed by New England Boring Contractors of Hermon, Maine between May 2 and May 4, 2023.
BB-WWS-201	Location and designation of BB-WWS-200 series borings that were performed by Seaboard Drilling of Bangor, Maine on April 22, 2024.
sCPT-WWS-202	Location and designation of sCPT-WWS-200 series seismic cone penetration tests that were performed by Seaboard Drilling of Bangor, Maine on April 23, 2024.

INTERPRETIVE SUBSURFACE PROFILE LEGEND



SCALE

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2510500
WIN 25105.00
BRIDGE PLANS
BRIDGE NO. 2102

	BY	DATE
PROJ. MANAGER	M. WIGHT	
DESIGN-DETAILED	E. TOME	B. CARDALI 07/2024
CHECKED-REVIEWED	C. SNOW	07/2024

SIGNATURE
P.E. NUMBER
DATE

WILTON
BRIDGES BRIDGE
WILSON STREAM
FRANKLIN
BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
5
OF 30

Filename: ...\BLP\BLP 5.9.24 combined.dgn Division: HIGHWAY Username: common Date:8/30/2024

APPENDIX A – LIMITATIONS

GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the contract documents, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

11. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

APPENDIX B – TEST BORINGS

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, Gravelly sands, little or no fines
			SP	Poorly-graded sands, Gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half 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 Sandy or Silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

MODIFIED BURMISTER SYSTEM

Descriptive Term	Portion of Total (%)
trace	0 - 10
little	11 - 20
some	21 - 35
adjective (e.g. Sandy, Clayey)	36 - 50

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$\text{RQD (\%)} = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Rock Quality Based on RQD

Rock Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Sample Container Labeling Requirements:

WIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bridge No. 2102 Replacement	**Boring No.:**	BB-WWS-101
Location: Wilton, Maine	**WIN:**	25105.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	581.8	**Auger ID/OD:**	4.5"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tome / L. Navarrete	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-3-23 / 5-4-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	Sta. 3+26.9, 6.2' Lt	**Casing ID/OD:**	4.25"/4.5", 3"/3.5"	**Water Level*:**	9.2'
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	581.3		0'-0.5': Asphalt. 0.5	
	1D	24/7	1.0 - 3.0	4-11-11-12	22	27				Brown, wet, medium dense, Gravelly fine to coarse SAND, little silt, (Fill).	
5	2D	24/15	5.0 - 7.0	3-3-2-2	5	6	6			Brown, moist, loose, fine to coarse SAND, little gravel, little silt, (Fill).	G#23-S-B271 A-1-b, SP-SM WC=12.1
							7				
							10				
							12				
							10				
10	3D	24/1	10.0 - 12.0	40-10-9-5	19	23	RC			Brown, wet, medium dense, GRAVEL, some silt, (Fill). hammer motion indicates probable cobble at 10.0'; cobble/likely pushed down with splitspoon. Roller cone used to push material aside.	
								568.8		13.0	
15	4D	24/5	15.0 - 17.0	7-8-6-6	14	17	37			Brown, wet, medium dense, fine to coarse SAND, little silt, little gravel, slight organic odor, (Alluvium).	
							75				
	5D	24/3	17.0 - 19.0	4-5-4-3	9	11	50			Brown, wet, medium dense, fine to coarse SAND, little silt, little gravel, (Alluvium). 5" of spoon consisted of wash return.	G#23-S-B272 A-1-a, GP-GM WC=8.8
							31				
							28				
20	6D	24/8	20.0 - 22.0	3-3-1-1	4	5	23			Top 4": Brown, wet, fine to coarse SAND, little gravel, (Alluvium). Bottom 4": Grey, wet, Silty CLAY, trace fine sand, appears water worked, (Alluvium). Pocket Penetrometer = 0 tsf	G#23-S-B273 PI=9 LL=26 PL=17 A-7-6, CL WC=27.6
							17				
	7D MV1	24/0 5/5	22.0 - 24.0 22.6 - 23.0	WOH-WOH-WOH-WOH	--		18			No recovery. Could not advance vane.	
							17				
	8D	24/3	24.0 - 26.0	WOH-WOH-WOH-1	--		PUSH			Grey, wet, Sandy SILT, trace gravel, appears water worked, (Alluvium).	G#23-S-B274 A-4(0), ML
25											

Remarks:

1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEB# D20 Energy Transfer Ratio = 0.742.
3. Water level measured immediately after removal of casing.
4. As-drilled boring locations were surveyed by MaineDOT in the field (N649630.5, E1013356.6).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bridge No. 2102 Replacement		**Boring No.:**	BB-WWS-101
Location: Wilton, Maine		**WIN:**	25105.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	581.8	**Auger ID/OD:**	4.5"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tome / L. Navarrete	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-3-23 / 5-4-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	Sta. 3+26.9, 6.2' Lt	**Casing ID/OD:**	4.25"/4.5", 3"/3.5"	**Water Level*:**	9.2'
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											WC=12.8
	9D	24/0	26.0 - 28.0	WOH-WOH-WOH-WOH	--					No recovery (3" spoon).	
	10D	24/7	28.0 - 30.0	WOR-2-5-4	7	9	30			Grey, wet, loose, Silty fine to medium SAND, trace Gravel, (Alluvium).	G#23-S-B275 A-4(0), SM WC=21.5
							40				
30	11D	24/7	30.0 - 32.0	9-3-3-6	6	7	35			Grey, wet, loose, Silty fine to coarse SAND, trace gravel, (Alluvium).	
							50				
							58				
							80				
								547.3		34.5	
35	R1	28/28	35.0 - 37.3	RQD = 0%			NQ			Increase in resistance during advancement of casing at 34.5'; probable bedrock. Advanced roller cone to 35.0' and set up to core. R1: Hard, fresh, aphanitic, grey, SCHIST. Joints are very close to close, high angle, planar to undulating, rough, fresh, tight to partially open. One vertical joint is undulating, rough, fresh, partially open. Recovery = 100% Rock Quality = Very Poor Rock Core Times (min:sec): 35.0-36.0' (2:04), 36.0-37.0' (2:04), 37.0-37.3' (0:29)	
	R2	44/42	37.3 - 41.0	RQD = 45%						R2: Hard, fresh, aphanitic, grey, SCHIST. Joints are very close to moderately spaced, high angle, planar, smooth, fresh, tight to partially open. Recovery = 95% Rock Quality = Poor Rock Core Times (min:sec): 37.3-38.0' (1:22), 38.0-39.0' (1:38), 39.0-40.0' (2:50), 40.0-41.0' (1:30)	
40											
	R3	38/38	41.0 - 44.2	RQD = 47%						R3: Hard, fresh, aphanitic, grey, SCHIST. Joints are extremely close to moderately spaced, high angle, planar, smooth, fresh, very tight to partially open. Recovery = 100% Rock Quality = Poor Rock Core Times (min:sec): 41.0-42.0' (1:28), 42.0-43.0' (1:12), 43.0-44.0' (1:13), 44.0-44.2' (0:14)	
	R4	34/34	44.2 - 47.0	RQD = 76%						R4: Hard, fresh, aphanitic, grey, SCHIST, Joints are close to moderately spaced, high angle, planar, smooth, fresh, tight to partially open. Recovery = 100% Rock Quality = Good Rock Core Times (min:sec): 44.2-45.2' (1:53), 45.2-46.2' (2:14), 46.2-47.0' (1:24)	
45											
								534.8		47.0	
										Bottom of Exploration at 47.0 feet below ground surface.	
50											

Remarks:

1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEB# D20 Energy Transfer Ratio = 0.742.
3. Water level measured immediately after removal of casing.
4. As-drilled boring locations were surveyed by MaineDOT in the field (N649630.5, E1013356.6).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bridge No. 2102 Replacement
Location: Wilton, Maine

Boring No.: BB-WWS-102
WIN: 25105.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	582.0	**Auger ID/OD:**	4.5"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tome / L. Navarrete	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-3-23 / 5-4-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	Sta. 4+03.7, 4.1' Rt	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	8.4'
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	581.5		0'-0.5': Asphalt. 0.5	
	1D	24/8	1.0 - 3.0	5-7-6-3	13	16				Brown, dry, medium dense, fine to coarse SAND, little silt, some gravel, (Fill).	G#23-S-B276 A-1-b, SM WC=5.7
	2D	24/3	3.0 - 5.0	3-6-5-7	11	14				Brown, dry, medium dense, Gravelly fine to coarse SAND, little silt, (Fill).	
5	3D	24/6	5.0 - 7.0	5-5-10-18	15	19	60			Brown, moist, medium dense, Gravelly fine to coarse SAND, little silt, (Fill). Casing advancement refusal at 7.0'. Roller bit action indicates probable cobbles and gravel from 7.0'-10.0'.	G#23-S-B277 A-1-a, SP-SM WC=6.4
							63				
							RC				
10	4D	24/9	10.0 - 12.0	10-12-6-6	18	22				Brown, wet, medium dense, Gravelly fine to coarse SAND, trace silt, (Fill).	G#23-S-B278 A-1-a, SW-SM WC=9.3
							73				
							30				
							28	569.0		13.0	
							25				
15	5D	24/10	15.0 - 17.0	5-1-1-1	2	2	20			No recovery. 3" spoon used to collect sample material. Top 4": Brown, wet, sandy GRAVEL, (Alluvium). Bottom 1": Brown, wet, Clayey SILT, some fine sand, (Alluvium). Pocket Penetrometer = 0 tsf	
							7				
							7				
							26				
							52				
20	6D	24/9	20.0 - 22.0	22-21-21-34	42	52	42			Red-brown, wet, dense, fine to coarse SAND, some gravel, little silt, (Alluvium). 4" casing advancement refusal at 22.0'. Switched to 3" casing, spin, probable cobbles from 22.0'-24.0'.	
							SPIN				
								560.0		22.0	
25											

Remarks:

1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEB# D20 Energy Transfer Ratio = 0.742.
3. Water level measured immediately after removal of casing.
4. River water level 8.25' from deck.
5. As-drilled boring locations were surveyed by MaineDOT in the field (N649682.9, E1013413.6).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Bridge No. 2102 Replacement **Location:** Wilton, Maine	**Boring No.:** BB-WWS-102 **WIN:** 25105.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	582.0	**Auger ID/OD:**	4.5"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tome / L. Navarrete	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-3-23 / 5-4-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	Sta. 4+03.7, 4.1' Rt	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	8.4'
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
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MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							RC			3" casing spin refusal at 28.0'. Advanced roller cone to 31.5'; probable cobbles from 28.0'-31.5'.	
30											
	8D	8/8	31.5 - 32.2	47-50/2"	R					Grey, wet, very dense, fine to coarse SAND, some gravel, some silt, (Glacial Till).	G#23-S-B279 A-1-b, SM WC=9.2
										34.5	
35	R1	60/60	34.5 - 39.5	RQD = 80%			NQ	547.5		Roller cone refusal at 34.5'; dark grey rock in wash return. Set up to core at 34.5'. R1: Very hard to hard, fresh, aphanitic to fine grained, grey, SCHIST. Primary joints are extremely close to moderately spaced, high angle, planar, smooth, fresh to discolored, very tight to tight. Secondary joints are close, low angle, planar, rough, fresh, tight to open. Recovery = 100% Rock Quality = Good Rock Core Times (min:sec): 34.5-35.5' (3:02), 35.5-36.5' (1:30), 36.5-37.5' (2:00), 37.5-38.5' (1:33), 38.5-39.5' (3:20)	
40	R2	30/24	39.5 - 42.0	RQD = 67%						R2: Very hard to hard, fresh, aphanitic to fine grained, grey, SCHIST. Primary joints are very close, low angle, planar, rough, fresh tight to partially open, with silt infilling. One joint is high angle, planar, smooth, fresh, tight. Recovery = 80% Rock Quality = Fair Rock Core Times (min:sec): 39.5-40.5' (2:31), 40.5-41.5' (2:04), 41.5-42.0' (3:44)	
	R3	36/36	42.0 - 45.0	RQD = 86%						R3: Very hard to hard, fresh, aphanitic to fine grained, grey, SCHIST. Primary joints are close, high angle, planar, smooth, fresh, very tight to tight. Secondary joints are close to moderately spaced, low angle to moderately dipping, planar, smooth, fresh, tight. Recovery = 100% Rock Quality = Good Rock Core Times (min:sec): 42.0-43.0' (2:00), 43.0-44.0' (2:10), 44.0-45.0' (1:15)	
45								537.0		45.0 **Bottom of Exploration at 45.0 feet below ground surface.**	
50											

Remarks:

1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEB# D20 Energy Transfer Ratio = 0.742.
3. Water level measured immediately after removal of casing.
4. River water level 8.25' from deck.
5. As-drilled boring locations were surveyed by MaineDOT in the field (N649682.9, E1013413.6).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bridge No. 2102 Replacement
Location: Wilton, Maine

Boring No.: BB-WWS-201
WIN: 25105.00

Driller:	Seaboard Drilling, Inc.	**Elevation (ft.)**	582.1	**Auger ID/OD:**	4.5" HSA
Operator:	K. Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	J. Cozens	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4-22-24 / 4-22-24	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	Sta. 3+98.8, 6.8' Lt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	10.8'
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.4 - 2.4	9-10-7-6	17	30	HSA	581.7		1D (0'-0.4'): Asphalt — 0.4	
										Brown, moist, medium dense, fine to coarse SAND & GRAVEL, trace silt, with probable cobbles and boulders, (Fill).	
										Intermittent auger resistance from 2.4'-5.0', probable cobbles and boulders.	
5	2D	24/8	5.0 - 7.0	3-2-21-13	23	41				Dark brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	24-S-1534 A-1-b, SM MC=7.0
										Intermittent auger resistance from 7.0'-10.0', probable cobbles and boulders. Less resistance starting at approximately 10.0', probable bottom of fill interface.	
10	3D	24/5	10.0 - 12.0	3-2-3-3	5	9	R/C	572.1		— 10.0 Brown, wet, loose, GRAVEL and fine to coarse Sand, trace silt, (Alluvium).	24-S-1534 A-1-4(0), SM MC=23.2
							47				
							66				
							12				
15	4D	24/5	15.0 - 17.0	7-6-9-5	15	27	17			Brown, wet, medium dense, GRAVEL and fine to coarse Sand, trace silt, (Alluvium).	24-S-1536 A-1-a, GM MC=8.7
							42				
							39				
							59				
							74				
20	5D	24/9	20.0 - 22.0	10-7-4-2	11	20	38			5D (Top 7"): Brown, wet, medium dense, fine to coarse SAND and Gravel, little silt, (Alluvium). 5D (Bottom 2"): Grey and brown, wet, fine to medium SAND, little to some silt, with gravel cuttings, (Alluvium).	24-S-1537 A-1-a, SP-GM MC=12
							56				
							35				
							36	558.6		— 23.5 Cuttings in wash return change from brown to grey, probable transition to Glacial Till.	
25							68				

Remarks:

1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer Seaboard SN 367 Energy Transfer Ratio = 1.066.
3. Water level measured immediately after removal of casing.
4. As-drilled boring locations were surveyed by MaineDOT in the field (N649686.1, E1013402.0).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bridge No. 2102 Replacement
Location: Wilton, Maine

Boring No.: BB-WWS-201
WIN: 25105.00

Driller:	Seaboard Drilling, Inc.	**Elevation (ft.)**	582.1	**Auger ID/OD:**	4.5" HSA
Operator:	K. Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	J. Cozens	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4-22-24 / 4-22-24	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	Sta. 3+98.8, 6.8' Lt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	10.8'
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/6	25.0 - 27.0	28-19-8-9	17	30	46			Grey, wet, medium dense, fine to coarse SAND and Gravel, little silt, (Glacial Till).	24-S-1538 A-1-b, SM MC=9.6
							107				
							129				
	7D	11/8	28.0 - 28.9	41-50/5"	R		224			Grey, wet, dense, Sandy GRAVEL, little silt, (Glacial Till).	
							R/C			Casing refusal at 30.0', probable cobble/boulder.	
30	8D	18/12	30.0 - 31.5	62-59-77-25/0"	59	105				Grey, wet, dense, Gravelly SAND, trace silt, (Glacial Till). Intermittent resistance at 31.5', probable boulders/cobbles, dense till.	
										Increased roller bit resistance at 37.0', probable top of rock, set up to core.	
35											
	R1	54/50	37.0 - 41.5	RQD = 33%			NQ	545.1		37.0 R1: Hard, fresh, fine grained to aphanitic, grey, SCHIST. Primary joints are very close to close, moderately high angle, undulating, smooth, fresh, tight. Secondary joints are close, horizontal, undulating, smooth to rough, discolored to fresh, tight to open. Rock Quality = Poor Recovery = 92% Rock Core Times (min:sec): 37.0-38.0' (4:15), 38.0-39.0' (3:35), 39.0-40.0' (2:58), 40.0-41.0' (3:44), 41.0-41.5' (3:12)	
40											
	R2	36/34	41.5 - 44.5	RQD = 47%						R2: Hard, fresh, fine grained to aphanitic, grey, SCHIST. Primary joints are very close to close, moderately to high angle, undulating, smooth, fresh, tight. Secondary joints are moderately close, horizontal, undulating, smooth to rough, discolored to fresh, tight to open. Rock Quality = Poor Recovery = 94% Rock Core Times (min:sec): 41.5-42.5' (1:50), 42.5-43.5' (2:33), 43.5-44.5' (2:28)	
	R3	30/28	44.5 - 47.0	RQD = 33%						R3: Hard, fresh, fine grained to aphanitic, grey, SCHIST. Joints are very close to close, moderately to high angle, undulating, smooth, fresh, tight. Rock Quality = Poor Recovery = 93% Rock Core Times (min:sec): 44.5-45.5' (2:41), 45.5-46.5' (2:57), 46.5-47.0' (1:08)	
45											
								535.1		47.0 **Bottom of Exploration at 47.0 feet below ground surface.**	
50											

Remarks:

1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer Seaboard SN 367 Energy Transfer Ratio = 1.066.
3. Water level measured immediately after removal of casing.
4. As-drilled boring locations were surveyed by MaineDOT in the field (N649686.1, E1013402.0).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

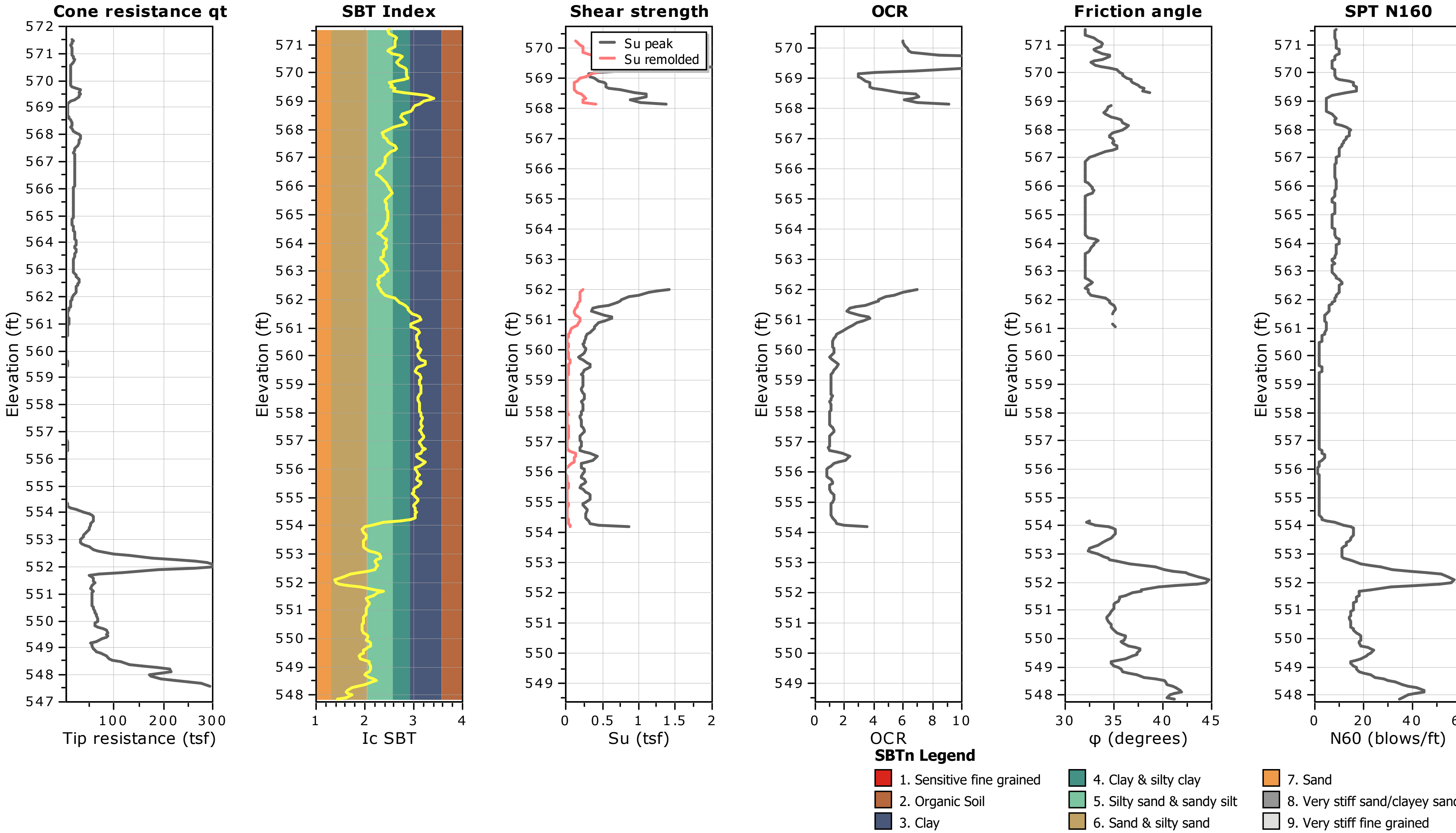
APPENDIX C – CONE PENETRATION TEST REPORTS

APPENDIX C.1 – CPT INTERPRETATION REPORT BY GZA

GZA GeoEnvironmental, Inc.
707 Sable Oaks Drive, Suite 150
South Portland, Maine

CPT: sCPT-WWS-201
Total depth: 33.99 ft, Date: 5/10/2024
Surface Elevation: 581.60 ft
Coords: X:1013367.30, Y:649625.70
Cone Type: Vertek
Cone Operator: Seaboard Drilling, LLC

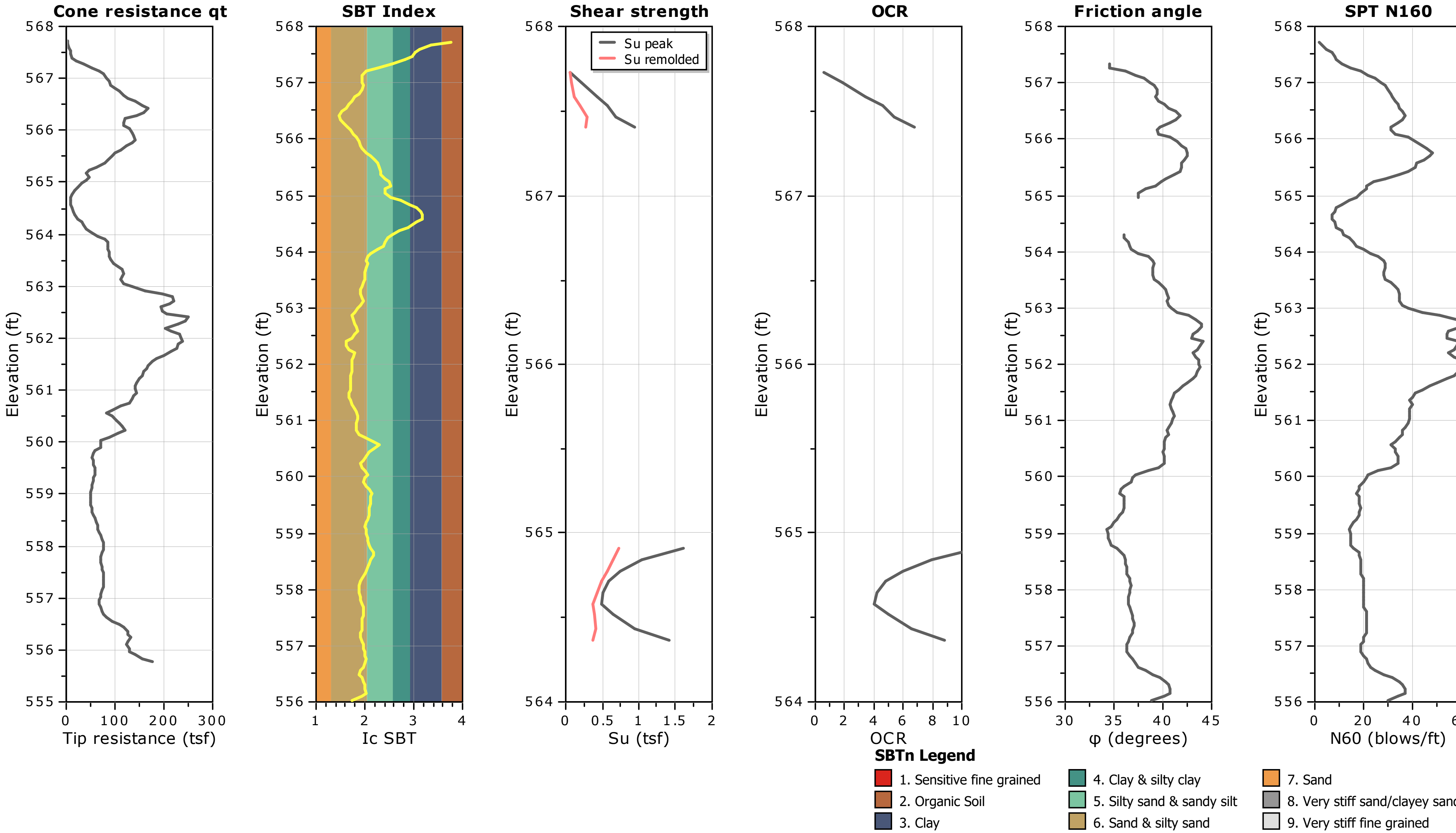
Project: Bridges Bridge No. 2102 Replacement, WIN 25105.00 (GZA File No. 09.0026188.01)
Location: Wilton, Maine



GZA GeoEnvironmental, Inc.
707 Sable Oaks Drive, Suite 150
South Portland, Maine

CPT: sCPT-WWS-202
Total depth: 26.33 ft, Date: 5/10/2024
Surface Elevation: 582.10 ft
Coords: X:1013405.40, Y:649689.70
Cone Type: Vertek
Cone Operator: Seaboard Drilling, LLC

Project: Bridges Bridge No. 2102 Replacement, WIN 25105.00 (GZA File No. 09.0026188.01)
Location: Wilton, Maine



APPENDIX C.2 – CPT DATA REPORT BY SEABOARD DRILLING LLC

23-086 S

May 4, 2024

GZA GeoEnvironmental, Inc.
Attention: Blaine Cardali, P.E.
707 Sable Oaks Drive, Suite 150
Portland, ME 04106

Subject: CPT Exploration Findings
Proposed Bridge Replacement
Bridge #2102 Over Coos Stream
Pond Road
Wilton, Maine

Dear Blaine:

In accordance with our Subcontract Agreement, dated September 11, 2023, we completed test boring and seismic piezocone penetration testing (sCPT) explorations at Bridge #2102 in Wilton. The test borings were observed and logged by GZA personnel. This report summarizes and provides data relative to the sCPT explorations.

SCPT EXPLORATION PROGRAM

Two sCPT explorations (sCPT-WWS-201 and sCPT-WWS-202) were advanced on April 23, 2024 adjacent to test boring locations. The exploration locations were selected and pre-marked at the site by GZA personnel. The sCPTs were advanced using a Diedrich D-50 track mounted drill rig utilizing Vertek piezocone equipment. The sCPT exploration program included the following:

- Two sCPT explorations were advanced at the site. The sCPT-WWS-201 location was pre-augered through granular fill materials with cobbles and boulders to a depth of 10.0 feet. sCPT-WWS-201 was subsequently advanced through sand backfill from the road surface elevation to a push refusal encountered at a depth of 34.0 feet. Hollow stem augers were advanced at sCPT-WWS-202 through granular fill materials with cobbles and boulders to a depth of 14.3 feet below the road surface. sCPT-WWS-202 was subsequently inserted into the hollow stem augers and was pushed starting at a depth of 14.3 feet extending to a depth of 26.3 feet (push refusal).

- Shear wave velocity testing was performed at rod break intervals throughout advancement of the sCPTs.

The sCPT explorations were performed in accordance with ASTM D5778. Shear wave velocity testing was performed in accordance with ASTM D7400.

SUBSURFACE CONDITIONS

The following is a summary of subsurface findings in each of the sCPT explorations.

sCPT-WWS-201		
Depth (feet)	**Predominant Soil Type**	**Soil Description**
10.0-20.8	Types 6 & 7	Sandy silt to clayey silt and silty sand to sandy silt with silt and clay layers
20.8-27.5	Type 1	Sensitive fine grained with a thin silt and clay layer at 25'
27.5-34.0*	Type 8	Sand to silty sand

*push refusal

sCPT-WWS-202		
Depth (feet)	**Predominant Soil Type**	**Soil Description**
14.3-16.3	Types 8 & 9	Sands
16.3-17.8	Types 4 & 6	Layered silts and clays
17.8-26.3*	Types 8 & 9	Sands

*push refusal

Soil behavior type profiling is based on normalized cone penetration resistance, Robertson 1986. Detailed soil type behavior is presented on the attached logs.

SEISMIC SHEAR WAVE VELOCITY TESTING

Shear wave velocity testing was performed in each of the sCPT explorations at rod break intervals. Results are presented on the logs. Waterfall plots showing shear wave velocity details are attached. A summary of the results is presented below:

SHEAR WAVE VELOCITY TEST SUMMARY			
sCPT-WSS-201		sCPT-WSS-202	
Depth (ft)	Seismic Velocity (ft/s)	Depth (ft)	Seismic Velocity (ft/s)
2.4	316	15.8	586
5.6	698	18.8	1,201
8.8	525	22.3	1,004
12.0	563	25.5	1,995
15.3	465		
18.6	454		
21.9	544		
25.1	499		
28.3	531		
31.6	2,515		
Average:	711	Average:	1,197

CLOSURE

It has been a pleasure to be of assistance to you on this project. Please let us know if you have any questions.

Sincerely,

Seaboard Drilling, LLC

Kevin J. Hanscom
Driller

SEISMIC CONE PENETRATION TEST PLOTS

sCPT-WWS-201

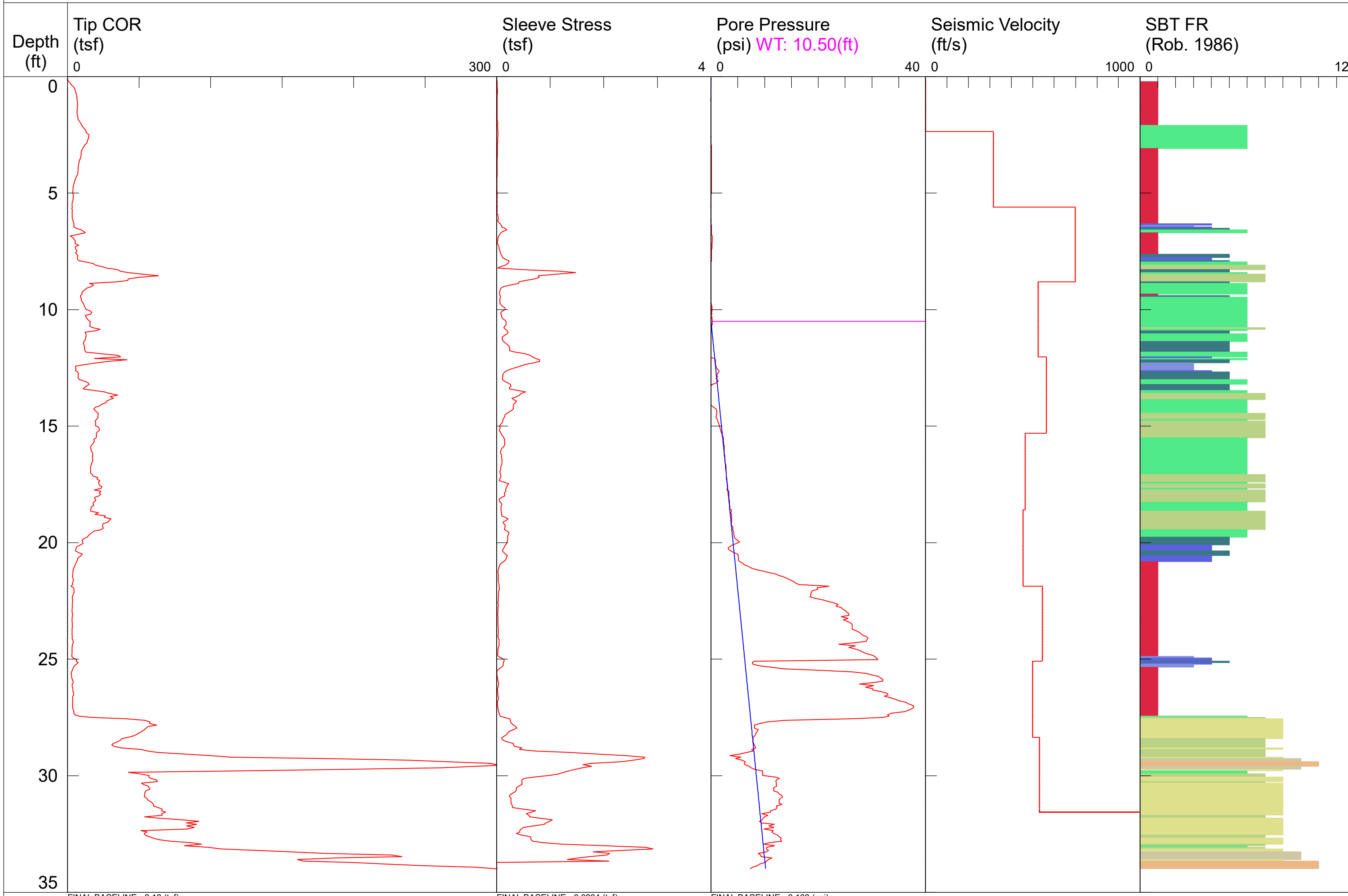
SEABOARD DRILLING

COMPANY: S.W.COLE Explorations LLC
PROJECT: MaineDOT Bridge #2102 Replacement
SITE: Pond Road
LOCATION: Wilton, ME
CLIENT: GZA GeoEnvironmental, Inc.

OPERATOR: Kevin Hanscom

FILENAME: sCPT-WWS-201.DAT

TEST ID: sCPT-WWS-201
TEST DATE: Tue 23/Apr/2024
GROUND SURFACE ELEV.: 000 +/-
TOTAL DEPTH: 33.990 ft



REMARKS

Pre-augered to 10.0', CPT pushed through sand backfill

Seismic velocity 2,515 ft/s - between 31.6'-34.0'

Push refusal at 34.0'

PROBE ID: 4644.163XX

1 Sensitive fine grained	4 Silty clay to clay	7 Silty sand to sandy silt	10 Gravelly sand to sand
2 Organic material	5 Clayey silt to silty clay	8 sand to silty sand	11 Very stiff fine grained **
3 Clays	6 Sandy silt to clayey silt	9 Sand	12 Sand to clayey sand **

*SBT: Robertson 1986; **Overconsolidated or Cemented; *SBT/SPT CORRELATION: UBC-1983

sCPT-WWS-202

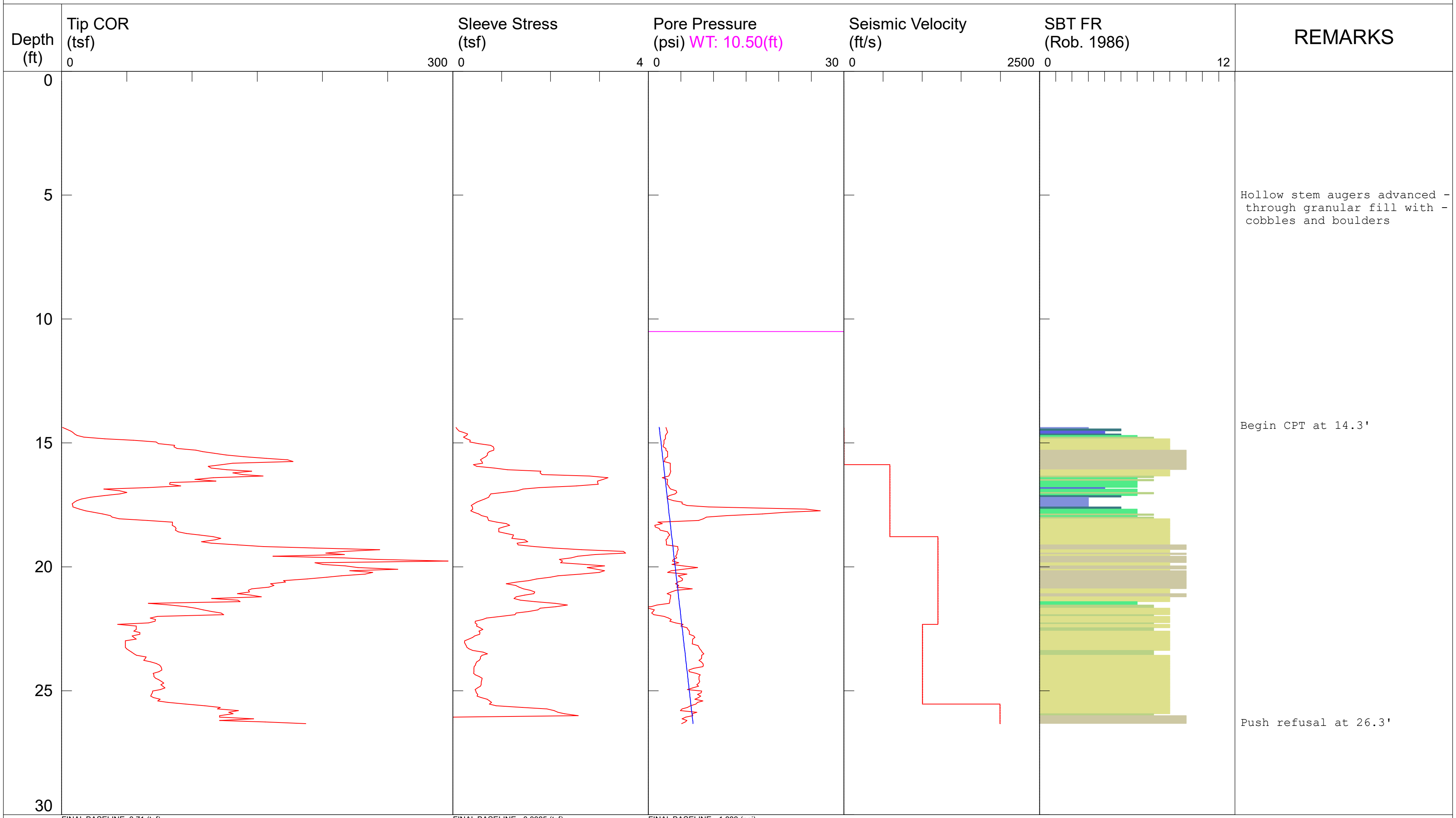
SEABOARD DRILLING

COMPANY: S.W.COLE Explorations LLC
PROJECT: MaineDOT Bridge #2102 Replacement
SITE: Pond Road
LOCATION: Wilton, ME
CLIENT: GZA GeoEnvironmental, Inc.

OPERATOR: Kevin Hanscom

FILENAME: sCPT-WWS-202.DAT

TEST ID: sCPT-WWS-202
TEST DATE: Tue 23/Apr/2024
GROUND SURFACE ELEV.: 000 +/-
TOTAL DEPTH: 26.329 ft



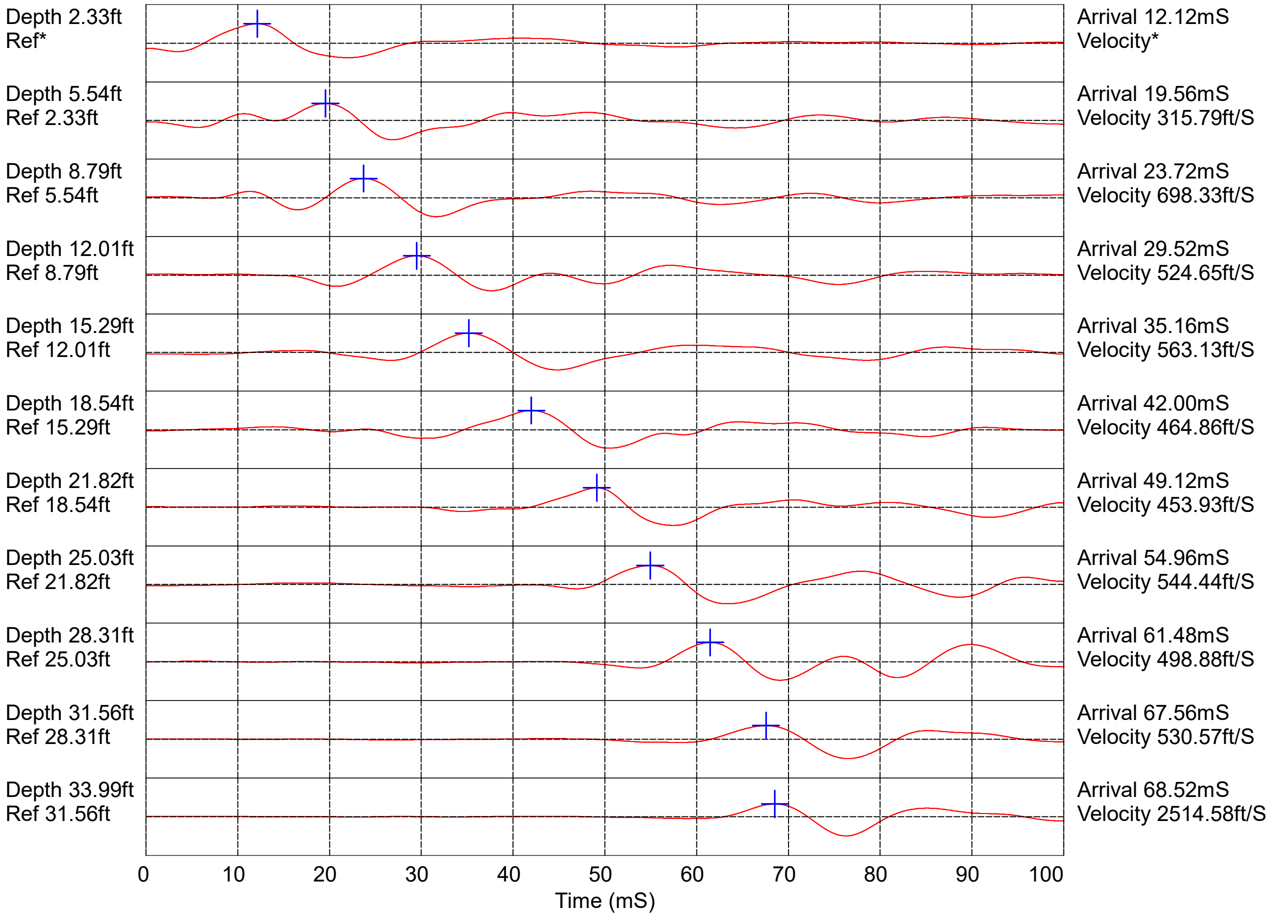
PROBE ID: 4644.163XX

1 Sensitive fine grained	4 Silty clay to clay	7 Silty sand to sandy silt	10 Gravelly sand to sand
2 Organic material	5 Clayey silt to silty clay	8 sand to silty sand	11 Very stiff fine grained **
3 Clays	6 Sandy silt to clayey silt	9 Sand	12 Sand to clayey sand **

*SBT: Robertson 1986; **Overconsolidated or Cemented; *SBT/SPT CORRELATION: UBC-1983

SEISMIC CONE PENETRATION TEST SHEAR WAVE (Vs) TRACE

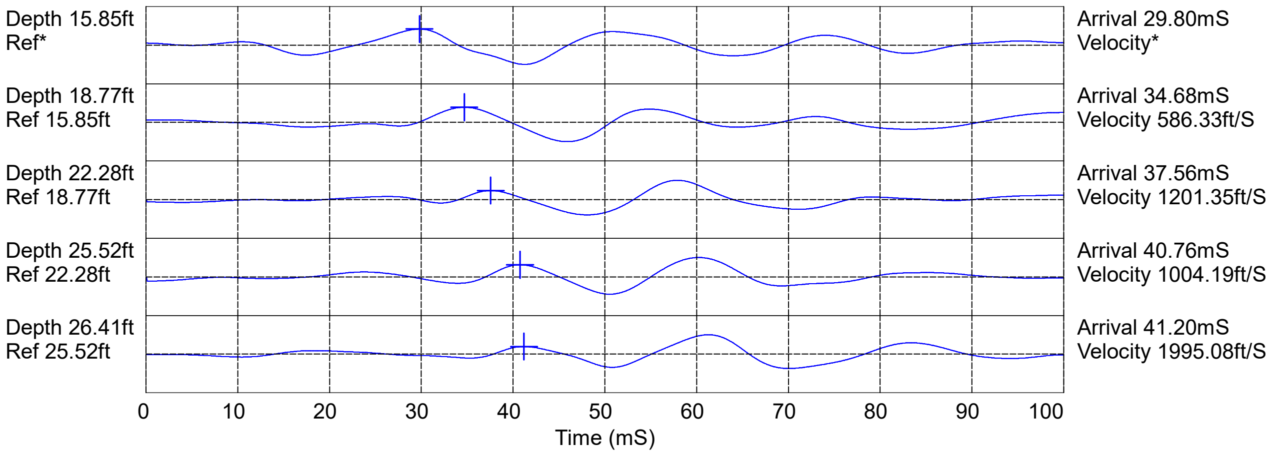
FILENAME: sCPT-WWS-201.SEI.sei



Hammer to Rod String Distance (ft): 3.51
* = Not Determined

FILENAME: sCPT-WWS-201.SEI.sei

FILENAME: sCPT-WWS-202.SEI.sei



Hammer to Rod String Distance (ft): 3.51
* = Not Determined

FILENAME: sCPT-WWS-202.SEI.sei

APPENDIX D – LABORATORY TEST RESULTS

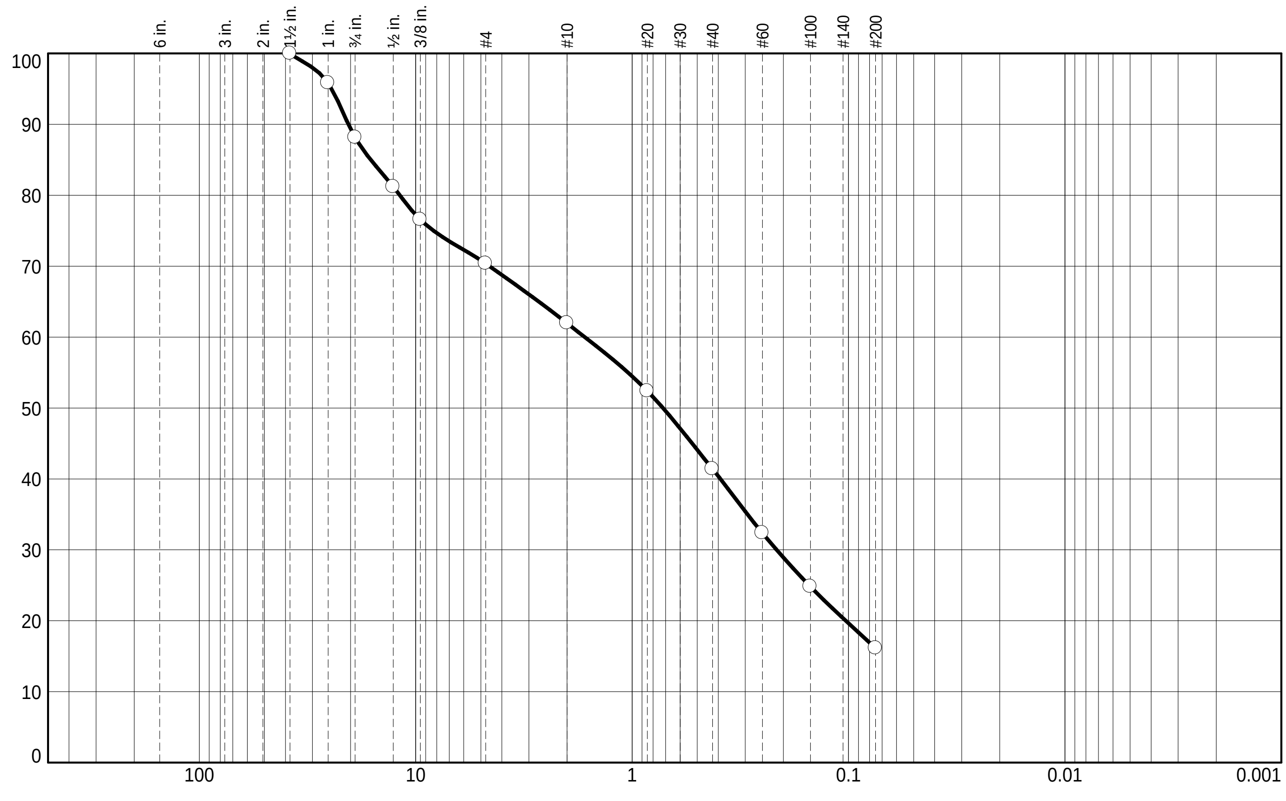
Thielsch DIVISION OF THE RISE GROUP	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 cts.thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental South Portland, ME Project Manager: John Cozens Assigned By: John Cozens Collected By: Client	Project Information: **Wilton Bridge Replacement** **Bridge #2102 over Coos Stream** Project Number: 09.0026198.01 Summary Page: 1 of 1 Report Date: 5/6/2024

LABORATORY TESTING DATA SHEET, Report No.: 7424-D-241

Boring No.	Sample ID	Depth (ft)	Laboratory No.	Identification Tests									Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Rcvd Moisture Content %	LL %	PL %	OD LL	Gravel %	Sand %	Fines %	Org. %	pH	g_d MAX (pcf) W_{opt} (%)	g_d MAX (pcf) W_{opt} (%) (Corr.)	Dry unit wt. (pcf)	Test Moisture Content %	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318			D6913			D2974	D4792	D1557								
BB-WWS-201	2D	5-7	24-S-1534	7.0				29.6	54.3	16.1											Dark Brown f-c SAND, some f-c Gravel, little Silt
BB-WWS-201	3D	10-12	24-S-1535	23.2				36.9	37.2	25.9											Brown f-c SAND and fine GRAVEL, some Silt
BB-WWS-201	4D	15-17	24-S-1536	8.7				48.2	48.2	3.6											Brown fine GRAVEL and f-c SAND, trace Silt
BB-WWS-201	5D	20-22	24-S-1537	12.0				41.1	47.3	11.6											Brown f-c SAND and f-c GRAVEL, little Silt
BB-WWS-201	6D	25-27	24-S-1538	9.6				40.3	45.9	13.8											Gray f-c SAND and f-c GRAVEL, little Silt

Date Received: **4/29/2024** Reviewed By: Date Reviewed: **5/6/2024**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.9	17.7	8.4	20.6	25.3	16.1	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	95.8		
3/4"	88.1		
1/2"	81.2		
3/8"	76.6		
#4	70.4		
#10	62.0		
#20	52.4		
#40	41.4		
#60	32.4		
#100	24.8		
#200	16.1		

* (no specification provided)

Soil Description

Dark Brown f-c SAND, some f-c Gravel, little Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 20.4842 D_{85}= 16.1361 D_{60}= 1.6544
D_{50}= 0.7202 D_{30}= 0.2148 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Remarks

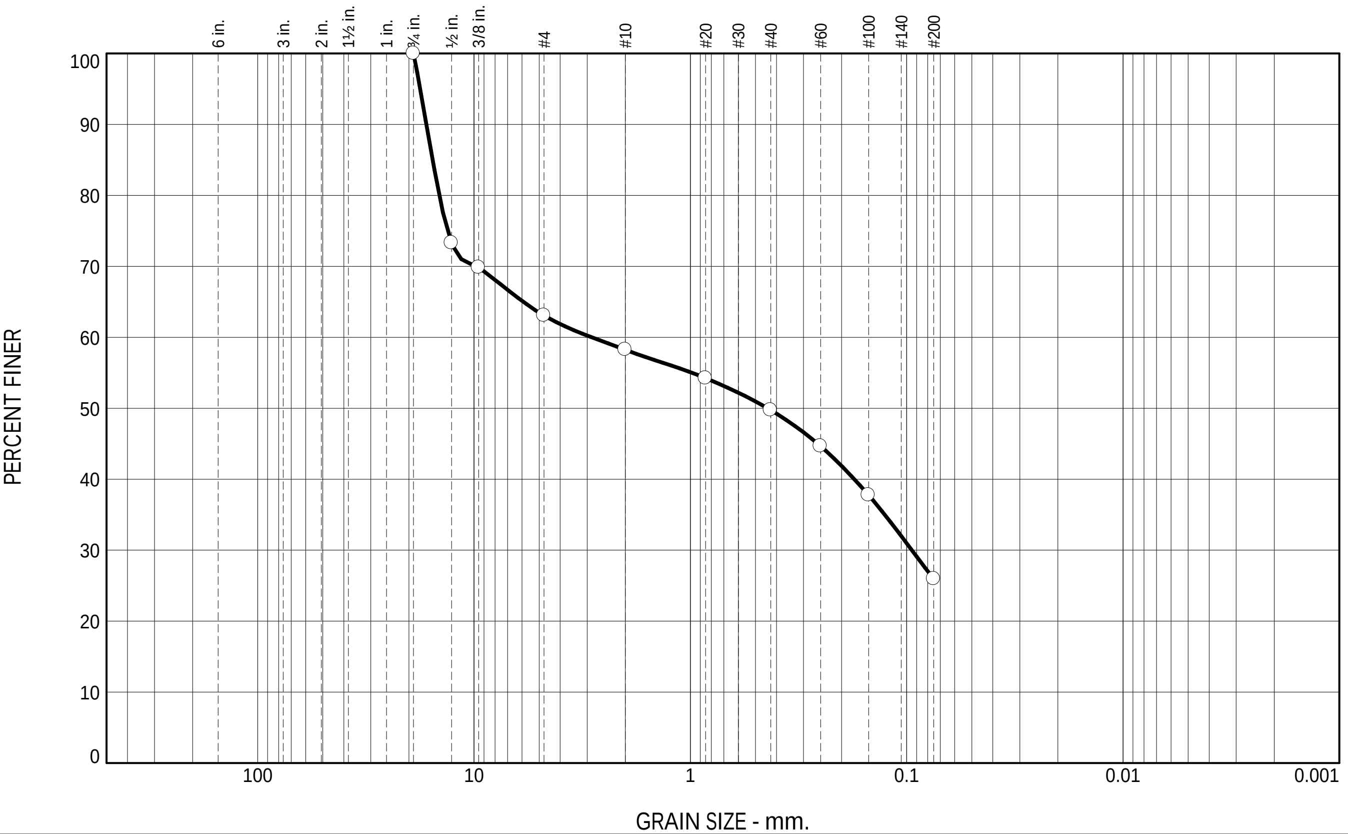
Source of Sample: BB-WWS-201 Depth: 5-7'
Sample Number: 2D

Date: 5.1.24

Thielsch Engineering Inc. Cranston, RI	Client: GZA GeoEnvironmental Project: Wilton Bridge Replacement Bridge #2102 over Coos Stream Project No: 09.0026188.01 Fig. 24-S-1534

Tested By: MCS Checked By: Kris Roland

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	36.9	4.9	8.5	23.8	25.9	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	73.3		
3/8"	69.8		
#4	63.1		
#10	58.2		
#20	54.2		
#40	49.7		
#60	44.7		
#100	37.8		
#200	25.9		

* (no specification provided)

Soil Description

Brown f-c SAND and fine GRAVEL, some Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 16.6088 D_{85}= 15.5438 D_{60}= 2.8565
D_{50}= 0.4396 D_{30}= 0.0946 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Source of Sample: BB-WWS-201 Depth: 10-12'
Sample Number: 3D

Date: 5.1.24

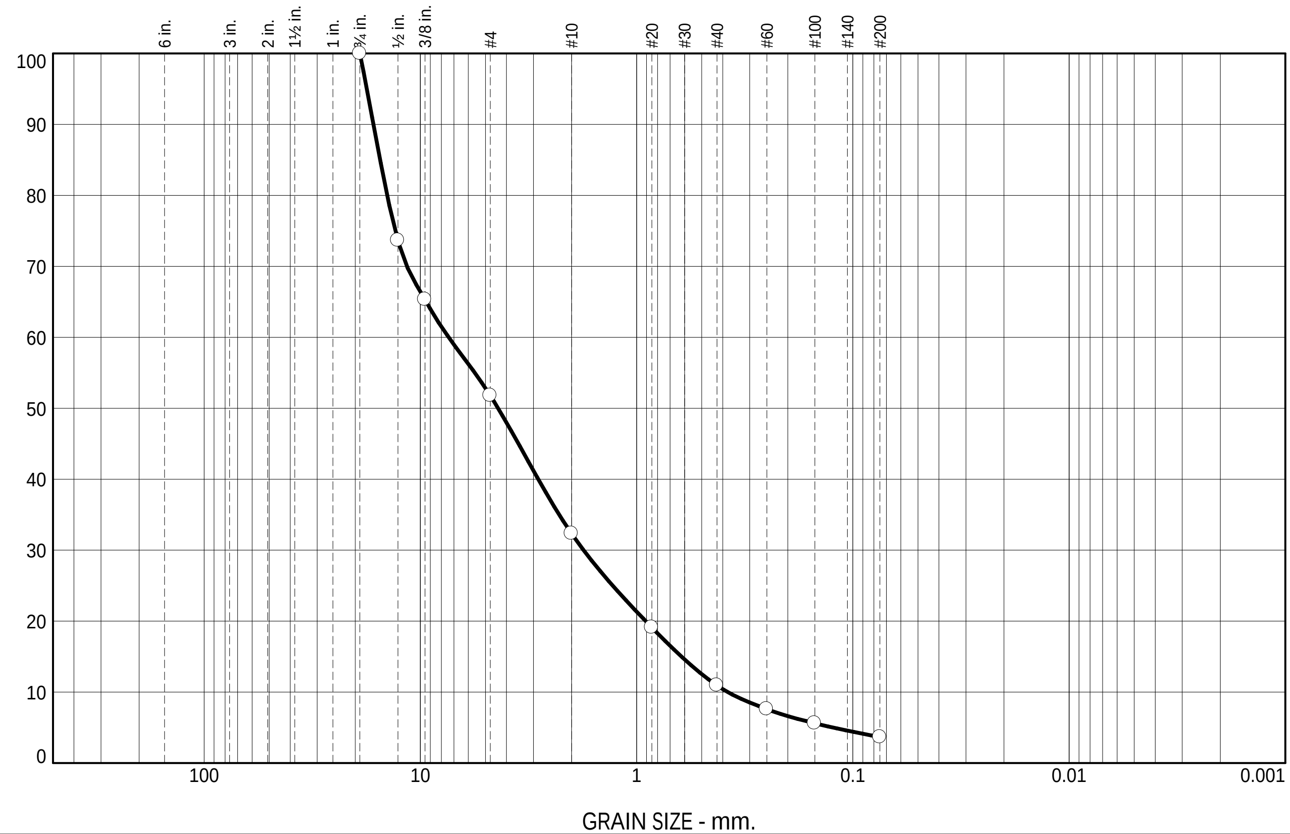
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Wilton Bridge Replacement
Bridge #2102 over Coos Stream
Project No: 09.0026188.01 Fig. 24-S-1535

Tested By: MCS Checked By: Kris Roland

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	48.2	19.5	21.4	7.3	3.6	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	73.6		
3/8"	65.3		
#4	51.8		
#10	32.3		
#20	19.1		
#40	10.9		
#60	7.6		
#100	5.6		
#200	3.6		

* (no specification provided)

Soil Description

Brown fine GRAVEL and f-c SAND, trace Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 16.4843 D_{85}= 15.3509 D_{60}= 7.3962
D_{50}= 4.3720 D_{30}= 1.7591 D_{15}= 0.6202
D_{10}= 0.3794 C_u= 19.50 C_c= 1.10

Classification

USCS= GW AASHTO= A-1-a

Remarks

Source of Sample: BB-WWS-201 Depth: 15-17'
Sample Number: 4D

Date: 5.1.24

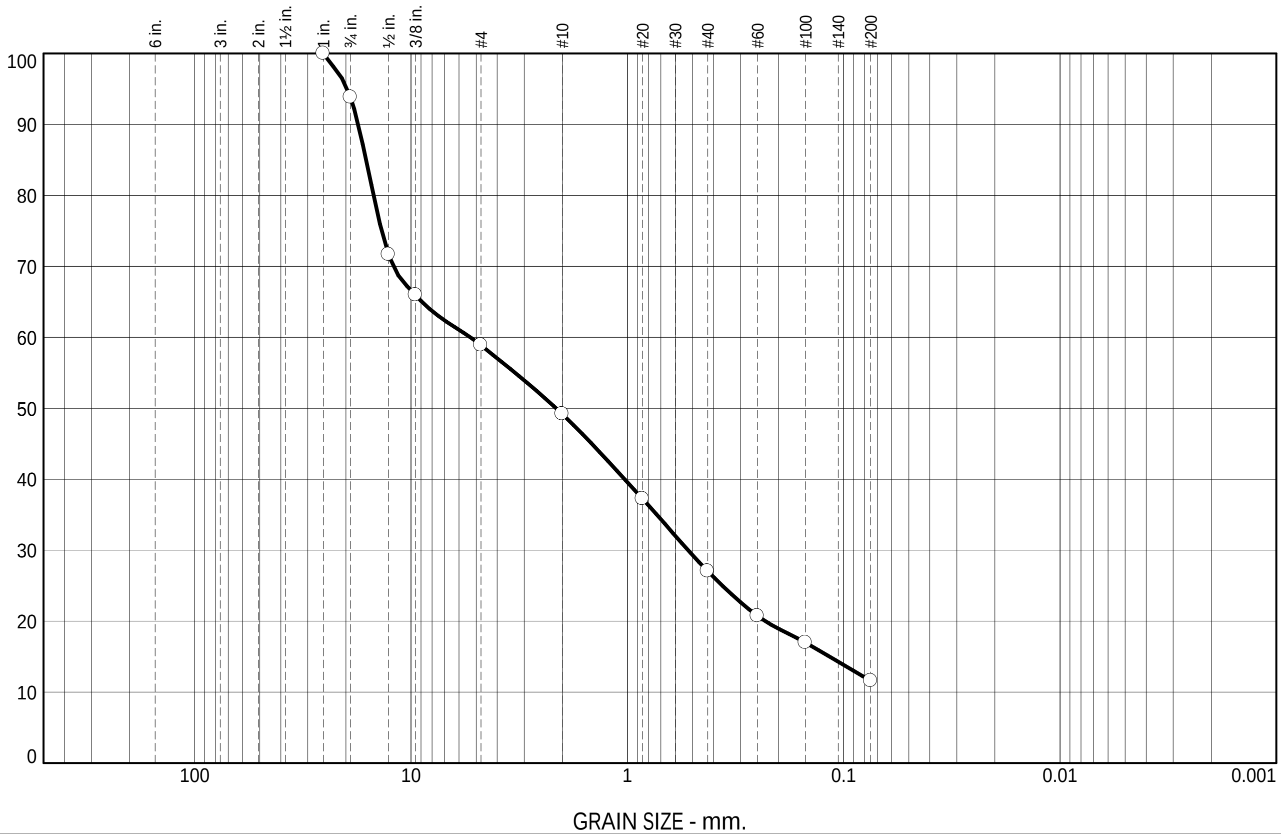
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Wilton Bridge Replacement
Bridge #2102 over Coos Stream
Project No: 09.0026188.01 Fig. 24-S-1536

Tested By: MCS Checked By: Kris Roland

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.2	34.9	9.7	22.2	15.4	11.6	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	93.8		
1/2"	71.7		
3/8"	65.9		
#4	58.9		
#10	49.2		
#20	37.2		
#40	27.0		
#60	20.7		
#100	17.0		
#200	11.6		

* (no specification provided)

Soil Description

Brown f-c SAND and f-c GRAVEL, little Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 17.5555 D_{85}= 16.1321 D_{60}= 5.3195
D_{50}= 2.1360 D_{30}= 0.5246 D_{15}= 0.1162
D_{10}= C_u= C_c=

Classification

USCS= SP-SM AASHTO= A-1-a

Remarks

Source of Sample: BB-WWS-201 Depth: 20-22'
Sample Number: 5D

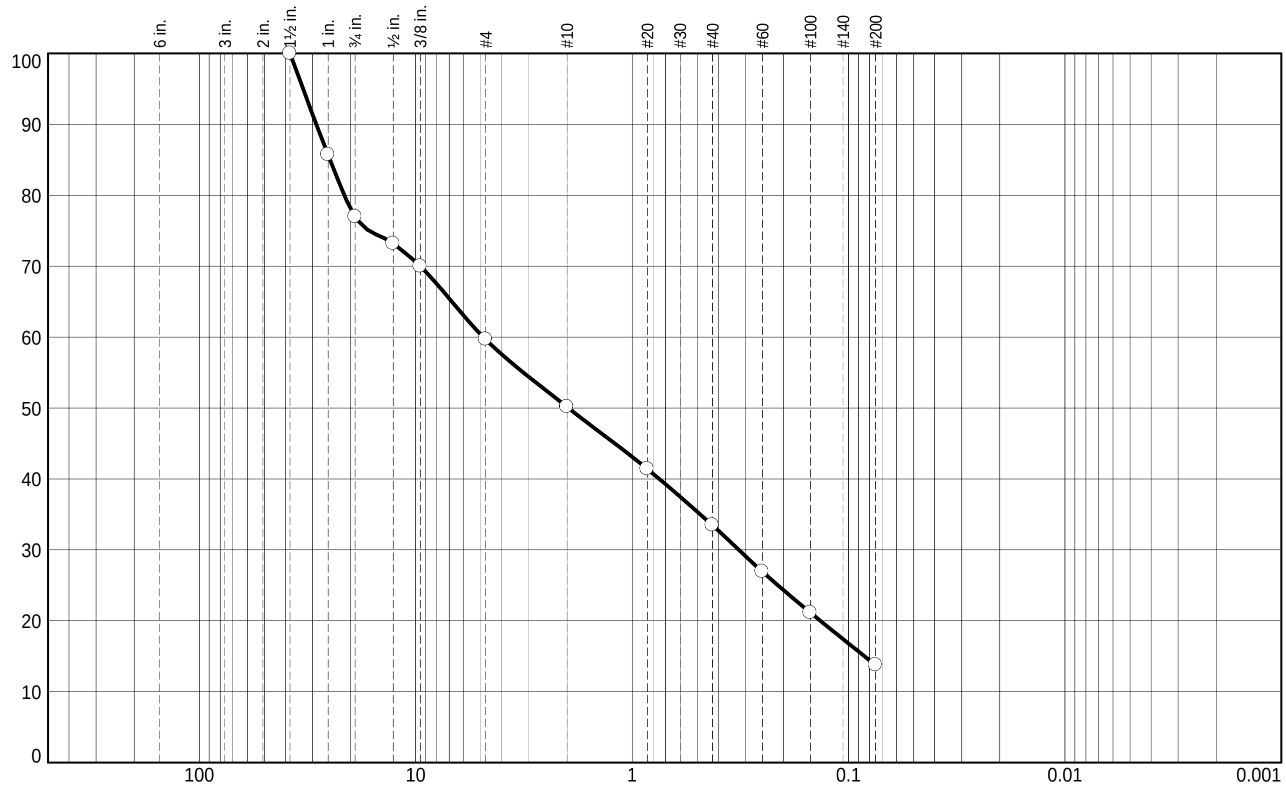
Date: 5.1.24

Thielsch Engineering Inc. Cranston, RI	Client: GZA GeoEnvironmental Project: Wilton Bridge Replacement Bridge #2102 over Coos Stream Project No: 09.0026188.01 Fig. 24-S-1537

Tested By: MCS Checked By: Kris Roland

These results are for the exclusive use of the client for whom they were obtained. This report only relates to items inspect and/or tested. No warranty, expressed or implied, is made.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	23.0	17.3	9.5	16.7	19.7	13.8	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	85.7		
3/4"	77.0		
1/2"	73.2		
3/8"	70.0		
#4	59.7		
#10	50.2		
#20	41.4		
#40	33.5		
#60	26.9		
#100	21.1		
#200	13.8		

* (no specification provided)

Soil Description

Gray f-c SAND and f-c GRAVEL, little Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 28.8011 D_{85}= 24.8555 D_{60}= 4.8732
D_{50}= 1.9694 D_{30}= 0.3214 D_{15}= 0.0844
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Remarks

Source of Sample: BB-WWS-201 Depth: 25-27'
Sample Number: 6D

Date: 5.1.24

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Wilton Bridge Replacement
Bridge #2102 over Coos Stream
Project No: 09.0026188.01 Fig. 24-S-1538

Tested By: MCS Checked By: Kris Roland

APPENDIX E – ENGINEERING CALCULATIONS

APPENDIX E.1 – GLOBAL STABILITY

Wilton Bridges Global Stability

Project:	Wilton Bridges Replacement	GZA File No.:	09.0026188.01
Location:	Wilton, Maine		
Prepared by:	J. Cozens	Date:	8/13/2023
Checked by:	C. Snow/B.Cardali	Date:	8/13/2023

Objective Evaluate global stability of the proposed approach embankment at critical transverse sections near the proposed abutments.

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 9th edition, 2020. (AASHTO LRFD)
2. Hynes-Griffin and Franklin (1984), "Rationalizing the seismic coefficient method," Miscellaneous Paper GL-84-13, U.S. Army Corps of Engineers Waterway Experiment Station, Vicksburg, Mississippi.
3. Kramer (1996), "Geotechnical Earthquake Engineering," Prentice-Hall, Inc., Upper Saddle River, NY.

Appendices

Appendix A – Slope/W Output Files
Appendix B – Design Drawing Plan for Stability Sections

SECTION 1 –Soil Profile

Soil Profile Friction angles were developed for granular materials based on corrected N60 blow counts or seismic cone penetrations tests interpreted N60 values of existing materials or anticipated/MaineDOT BDG design properties of new fills. Design soil properties are shown below and on the Slope/W output.

Table 1: Input Soil parameters

Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Fill	125	0	32
Alluvium	120	0	30
Riprap	140	0	40

SECTION 2 – Analysis

Performance Criteria The minimum factors of safety for global stability for slopes that support structures and do not contain or support a structural component are selected as 1.5 and 1.3, respectively. These correspond to approximate resistance factors of 0.65 and 0.75, as specified in MaineBDG section 5.9.2 and AASHTO LRFD Article 11.6.2.3.

The minimum factor of safety for pseudostatic seismic stability is selected as 1.0, which is recommended by Hynes-Griffin and Franklin (1984), referenced in Geotechnical Earthquake Engineering (1996). The pseudostatic analysis includes an additional earthquake load by incorporating the horizontal seismic coefficient, kh, as defined in AASHTO 11.6.5.2.2 and recommended by Hynes-Griffin and Franklin using the Newmark method as half of the maximum peak ground acceleration at the ground surface of the embankment. This criterion is considered suit able to limit potential for large deformation of the embankments, which is considered suitable for this Extreme Event scenario. Kh is taken as .066 g for Bridges Bridge.

Transverse Section Analysis GZA evaluated the stability of the proposed approach embankments using the computer analytical software *Slope/W 2020*, developed by Geo-Slope International, based on the Morgenstern-Price method.

The highest embankment and/or steepest side slopes are at the Abutment 1 approach at station 3+25. The analyzed sections considered the typical subsurface conditions at the respective locations. A Slope-W model was developed for the proposed 22-foot-wide travel way and auxiliary lane with a traffic surcharge load of 250 psf placed over the travel way and side slopes of 1.75H:1V on both side slopes. Sections were evaluated left to right, right to left.

The attached Slope-W outputs show that the proposed embankment provides the following minimum global factors of safety in the transverse direction:

Station	Analysis	Factory of safety Against Rotation	Required Factor of Safety
3+25 Right to left	Static	1.5	1.3
3+25 left to Right	Static	1.4	1.3
3+25 left to Right	Seismic	1.2	1.0

SECTION 3 –Conclusion

Conclusion The global stability analysis meets the minimum required criteria for static and pseudostatic analysis conditions.

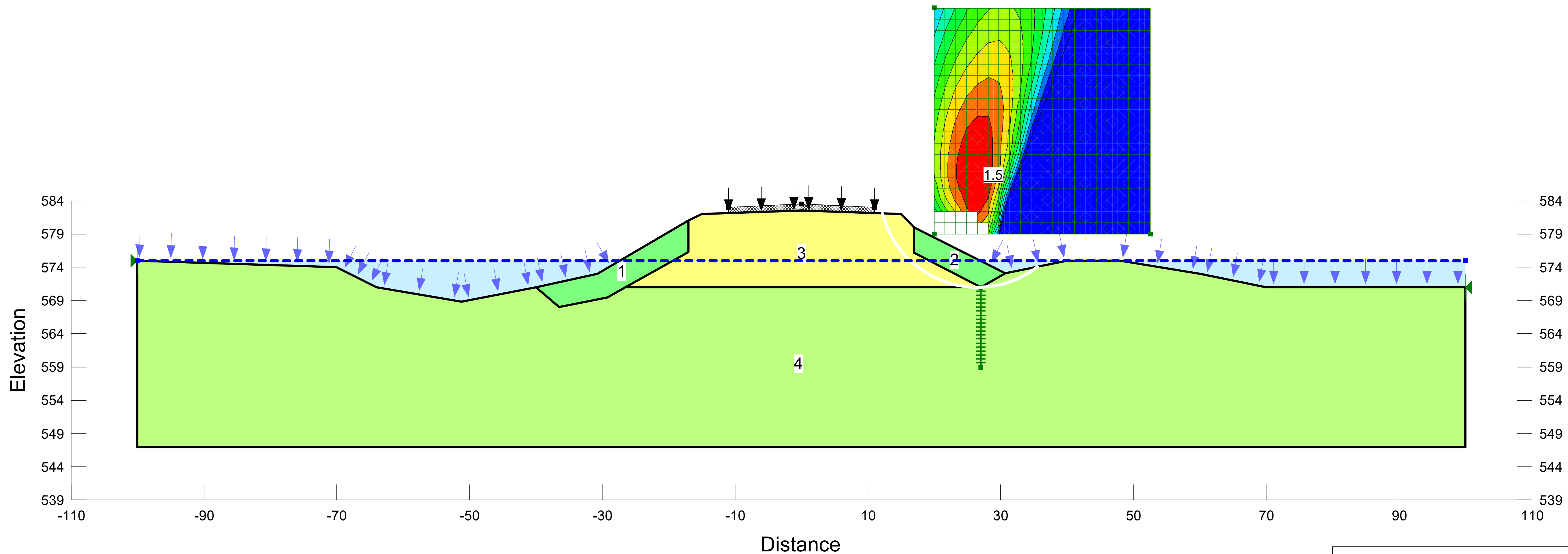
APPENDIX A

Slope/W Output Files

File Name: Wilton AB1 Stability check 8.12.24.gsz
Date: 08/30/2024

Analysis Type: Morgenstern-Price
Surcharge (Unit Weight): 250 pcf

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	1- Fill (New/Existing)	Mohr-Coulomb	125	0	32	1
	2 - Alluvium	Mohr-Coulomb	120	0	30	1
	3 - Riprap	Mohr-Coulomb	140	0	40	1



P:\09 Jobs\0026100s\09.0026188.00 - MEDOT - Wilton Bridge Replacement\09.0026188.01\Work\Calcs\Stability\

2D Geometry

Wilton-Abutment 1 L-R

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com

PREPARED FOR: MaineDOT

PROJ MGR: BMC | REVIEWED BY:CLS

CALCULATIONS: Blaine Cardali

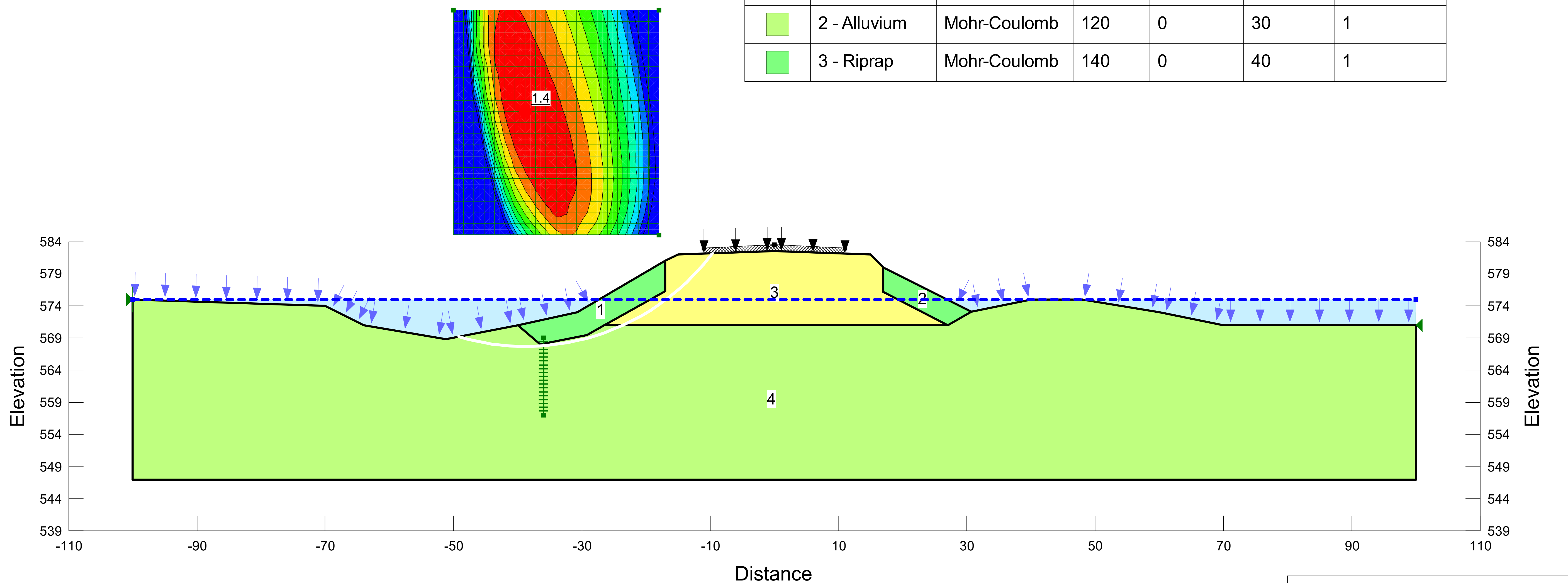
DATE: 08/30/2024 | PROJECT NO.: 09.0026188.01

FIGURE 2

File Name: Wilton AB1 Stability check 8.12.24.gsz
Date: 08/30/2024

Analysis Type: Morgenstern-Price
Surcharge (Unit Weight): 250 pcf

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	1- Fill (New/Existing)	Mohr-Coulomb	125	0	32	1
	2 - Alluvium	Mohr-Coulomb	120	0	30	1
	3 - Riprap	Mohr-Coulomb	140	0	40	1



2D Geometry

Wilton-Abutment 1 R-L

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
MaineDOT

PROJ MGR: BMC
REVIEWED BY:CLS
CALCULATIONS: Blaine Cardali
DATE:
08/30/2024
PROJECT NO.:
09.0026188.01

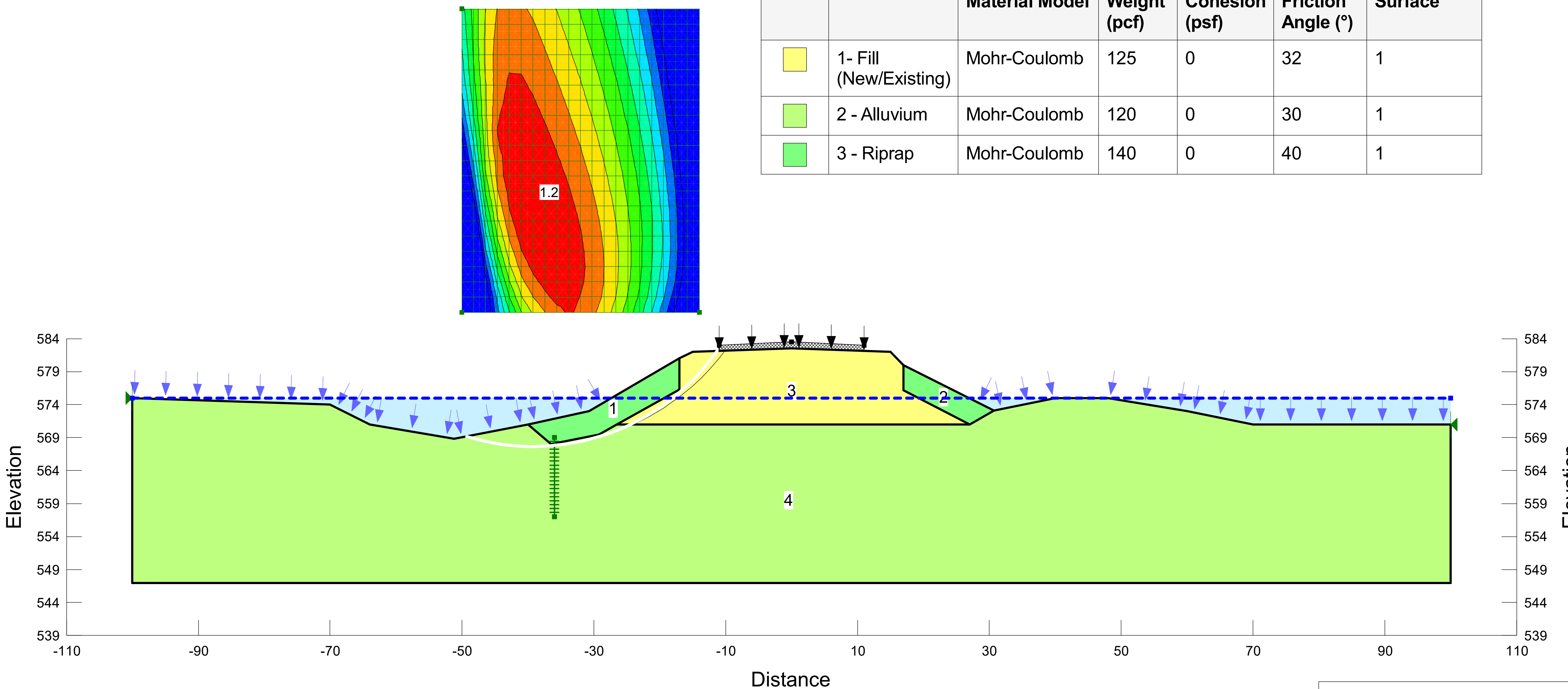
FIGURE
1

P:\09 Jobs\0026100s\09.0026188.00 - MEDOT - Wilton Bridge Replacement\09.0026188.01\Work\Calcs\Stability\

File Name: Wilton AB1 Stability check 8.12.24.gsz
Date: 08/30/2024

Analysis Type: Morgenstern-Price
Surcharge (Unit Weight): 250 pcf
Horz Seismic Coef.: 0.066

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	1- Fill (New/Existing)	Mohr-Coulomb	125	0	32	1
	2 - Alluvium	Mohr-Coulomb	120	0	30	1
	3 - Riprap	Mohr-Coulomb	140	0	40	1



2D Geometry

Wilton-Abutment 1 Psuedostatic R-L

PREPARED BY:	PREPARED FOR:	
GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com	MaineDOT	
PROJ MGR: BMC	REVIEWED BY:CLS	FIGURE 3
CALCULATIONS: Blaine Cardali		
DATE: 08/30/2024	PROJECT NO.: 09.0026188.01	

P:\09 Jobs\0026100s\09.0026188.00\09.0026188.00 - MEDOT - Wilton Bridge Replacement\09.0026188.01\Work\Calcs\Stability\

APPENDIX B

Design Drawing Plan for Stability Sections

Global Stability Section location
Wilton, Maine

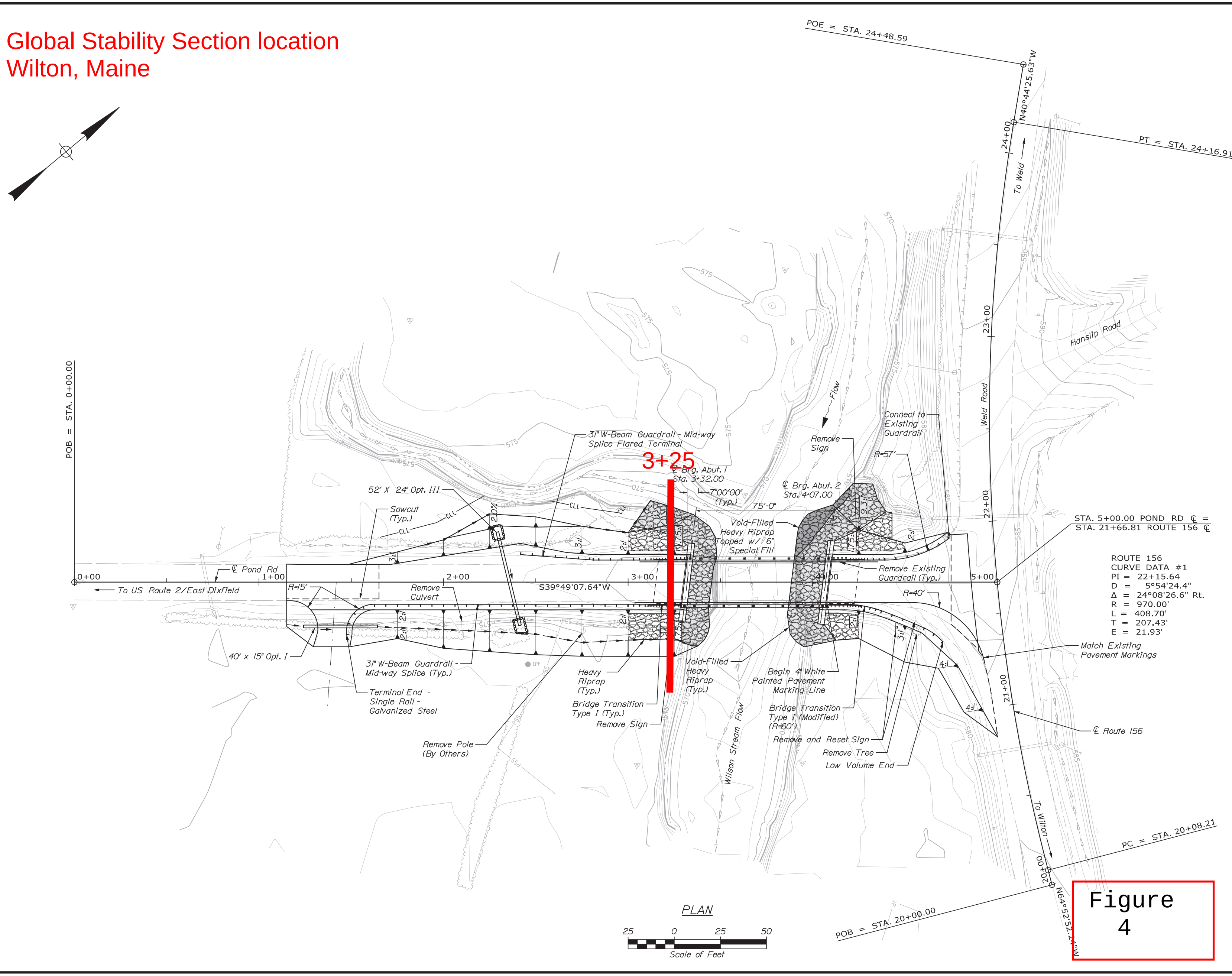


Figure 4

APPENDIX E.2 – SIESMIC DESIGN PARAMETERS

Seismic Site Class Calculation Summary

Project: Bridges Bridge Replacement **Project No.:** 09.0026188.01

Location: Wilton, ME

Evaluated By/Date: B.Cardali **Date** 7/16/2024

Checked By/Date: C.Snow **Date** 7/16/2024

Objective:

Determine seismic site class by performing calculations in accordance with the MaineDOT Bridge Manual 2003 Edition with updates in 2018, which references the AASHTO LRFD Seismic Bridge Design Specifications, 9th Edition 2020.

Subsurface Data: BB-WWS-101 and BB-WWS-102 were conducted by New England Boring Contractors between May 2 and 4, 2023 and were observed by GZA. Boring BB-WWS-201, sCPT-WWS-201 and sCPT-WWS-202 were conducted by Seaboard Drilling bewteen April 22 and 23, 2024 and were observed by GZA.

Approach:

1) Evaluate if the procedure in AASHTO LRFD Seismic Section 3.10.2.1 for classifying a site is appropriate for the site. Sites with highly variable subsurface conditions or very large sites may require multiple site class determinations or a site-specific seismic response analysis. Furthermore, classifying a site based on the 100 feet of soil and rock beneath the ground surface may be inappropriate if deep deposits of weak soils are present below 100 feet, or if foundation structures are supported on firm soil or rock below soft soils which can be justified as having little effect on the structure's seismic response.

2) Evaluate if soil properties are known in sufficient detail to determine site class. If data is not known in sufficient detail, AASHTO permits the use of Site Class D, unless conditions for Site Class E or Site Class F are likely to be present.

3) Check for the four categories of Site Class F requiring site-specific evaluation:
- Soils vulnerable to potential failure (liquefiable soils, sensitive clays, weakly cemented soils)
- Peats or highly organic clays greater than 10 feet in thickness
- Thick layers (greater than 25 feet) of highly plastic clay (PI > 75)
- Very thick soft/medium stiff clays (greater than 125 feet)

4) Check for existence of greater than 10 feet of soft clay (where s_u < 500 psf, *w* > 40%, and PI > 20). If these conditions are met, classify as Site Class E.

5) Categorize the site using one of the following three methods in AASHTO C3.10.3.1-1:
- $\overline{v}_s$ (Method A) - $\overline{N}$ (Method B) - $\overline{N}_{ch}$ *and* $\overline{s}_u$ (Method C)

If shear wave velocity data are available, they should be used to classify the site. The N and s_u methods should only be used if shear wave velocity data is not available, as the correlation between site amplification and these geotechnical parameters is more uncertain (and therefore more conservative) than the correlation with $\overline{v}_s$. If bedrock is encountered above a depth of 100 feet, 100 blow per foot should be taken as Ni.

Results: Calculations of the Seismic Site Class based on Method B as described in section 3.10.3.1 of the LRFD Seismic Bridge Design Specifications are attached. Calculations results are summarized in the table below.

Boring ID	BB-WWS-101	BB-WWS-102	BB-WWS-201	Average
N	12	25	46	28

sCPT ID	sCPT-WWS-201	sCPT-WWS-202	Average
Vs	860	1121	990

Conclusions: Based on the procedure outlined in section 3.10.3.1 and table 3.10.3.1-1 of the LRFD Seismic Bridge Design Specifications, we recommend that Site Class "D" be used for design.

Bridges Bridge Replacement
Wilton, ME

Calculated By: B.Cardali Date: 7/16/2024
Checked By: C.Snow Date: 7/16/2024

INPUT

Exploration ID: **BB-WWS-101** Ground Surface Elevation: 580.0 ft Depth of Boring: 47.0 ft

Depth to Bedrock: 34.5 ft Hammer ETR: 0.742

EQUATIONS

$$N = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals

CALCULATION

$\overline{N}$ = 12.2

Soil Strata	SPT Interval Depth Top, ft	SPT Interval Depth Bottom, ft	SPT Elevation (mid-interval)	N60	*d_i*	*d_i / Ni*	Comment
Fill	1.0	3.0	578.0	27	4.0	0.15	
	5.0	7.0	574.0	6	4.5	0.73	
	10.0	12.0	569.0	23	5.0	0.21	
Alluvium	15.0	17.0	564.0	17	3.5	0.20	
	17.0	19.0	562.0	11	2.5	0.22	
Glaciofluvial	20.0	22.0	559.0	5	2.5	0.51	
	22.0	24.0	557.0	1	2.0	1.62	
	24.0	26.0	555.0	1	2.0	1.62	
	26.0	28.0	553.0	1	2.0	1.62	
Glacial Till	28.0	30.0	551.0	9	2.0	0.23	
	30.0	32.0	549.0	7	3.3	0.44	
Bedrock	34.5	100.0	512.8	100	66.8	0.67	
Top of Bedrock	34.5						
					100.00		

Bridges Bridge Replacement
Wilton, ME

Calculated By: B.Cardali Date: 7/16/2024
Checked By: C.Snow Date: 7/16/2024

INPUT

Exploration ID: **BB-WWS-102** Ground Surface Elevation: 580.0 ft Depth of Boring: 45.0 ft

Depth to Bedrock: 34.5 ft Hammer ETR: 0.742

EQUATIONS

$$N = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals

CALCULATION

$\overline{N}$ = 25.0

Soil Strata	CPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	N60	d_i	d_i / Ni	Comment
Fill	1.0	3.0	578.0	16	3.0	0.19	
	3.0	5.0	576.0	14	2.0	0.15	
	5.0	7.0	574.0	19	3.5	0.19	
	10.0	12.0	569.0	22	5.0	0.22	
Alluvium	15.0	17.0	564.0	2	5.0	2.02	
	20.0	22.0	559.0	52	5.0	0.10	
Glacial Till	25.0	27.0	554.0	15	5.8	0.39	
	31.5	32.2	548.2	50	4.1	0.08	
Bedrock	34.5	100.0	512.8	100	66.7	0.67	
Top of Bedrock	34.5				100.0		

Bridges Bridge Replacement
Wilton, ME

Calculated By: B.Cardali Date: 7/16/2024
Checked By: C.Snow Date: 7/16/2024

INPUT

Exploration ID: **BB-WWS-201** Ground Surface Elevation: 582.1 ft Depth of Boring: 47.0 ft

Depth to Bedrock: 37.0 ft Hammer ETR: 1.066

EQUATIONS

$$N = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals

CALCULATION

$\overline{N}$ = 45.8

Soil Strata	CPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	N60	d_i	d_i / Ni	Comment
Fill	0.4	2.4	580.7	30	3.7	0.12	
	5.0	7.0	576.1	41	4.8	0.12	
Alluvium	10.0	12.0	571.1	9	5.0	0.56	
	15.0	17.0	566.1	27	5.0	0.19	
	20.0	22.0	561.1	20	5.0	0.26	
Glacial Till	25.0	27.0	556.1	30	5.0	0.17	
	30.0	31.5	551.4	50	5.8	0.12	
Bedrock	37.0	100.0	513.6	100	65.8	0.66	
Top of Bedrock	37.0				100.0		

Bridges Bridge Replacement
Wilton, ME

Calculated By: B.Cardali Date: 7/16/2024
Checked By: C.Snow Date: 7/16/2024

INPUT

Exploration ID: **sCPT-WWS-201** Ground Surface Elevation: 581.6 ft Depth of CPT: 43.8 ft

Depth to Bedrock: NE

CALCULATION

$\overline{V}$ = 859.6

Soil Strata	CPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	V_s	d_i	d_i / V_s	Comment
Fill	5.0	6.0	576.1	316	7.0	0.02	
	8.0	9.0	573.1	698	3.0	0.00	
Alluvium	11.0	13.0	569.6	525	4.0	0.01	
	15.0	16.0	566.1	563	3.0	0.01	
	18.0	19.0	563.1	465	3.0	0.01	
	21.0	22.0	560.1	454	3.5	0.01	
	25.0	26.0	556.1	544	3.5	0.01	
Glacial Till	28.0	29.0	553.1	499	3.0	0.01	
	31.0	32.0	550.1	530	3.0	0.01	
	33.0	100.0	515.1	1500	67.0	0.04	
Bottom of Boring	33.0						

Bridges Bridge Replacement
Wilton, ME

Calculated By: B.Cardali Date: 7/16/2024
Checked By: C.Snow Date: 7/16/2024

INPUT

Exploration ID: **sCPT-WWS-202** Ground Surface Elevation: 582.1 ft Depth of CPT: 26.3 ft

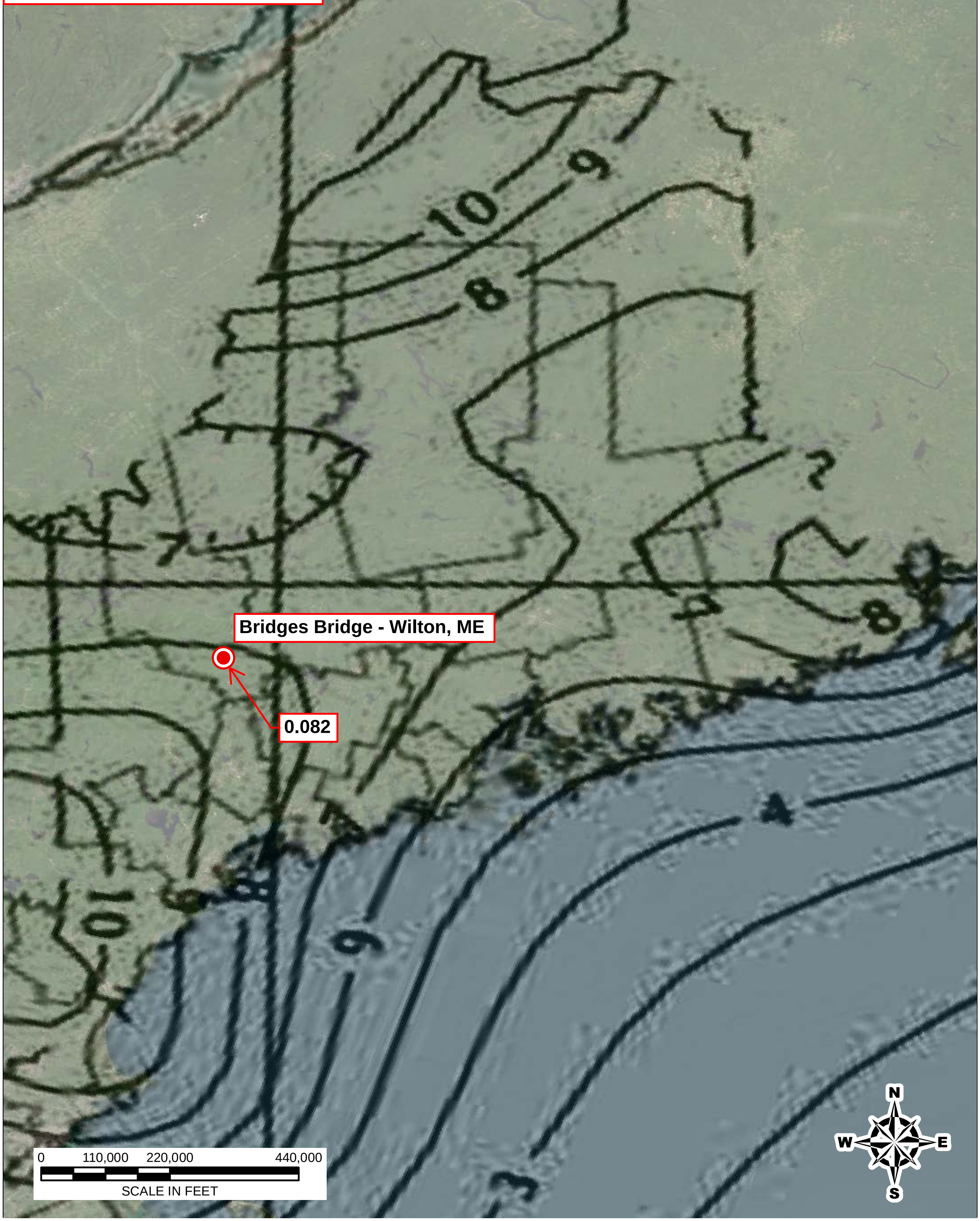
Depth to Bedrock: NE

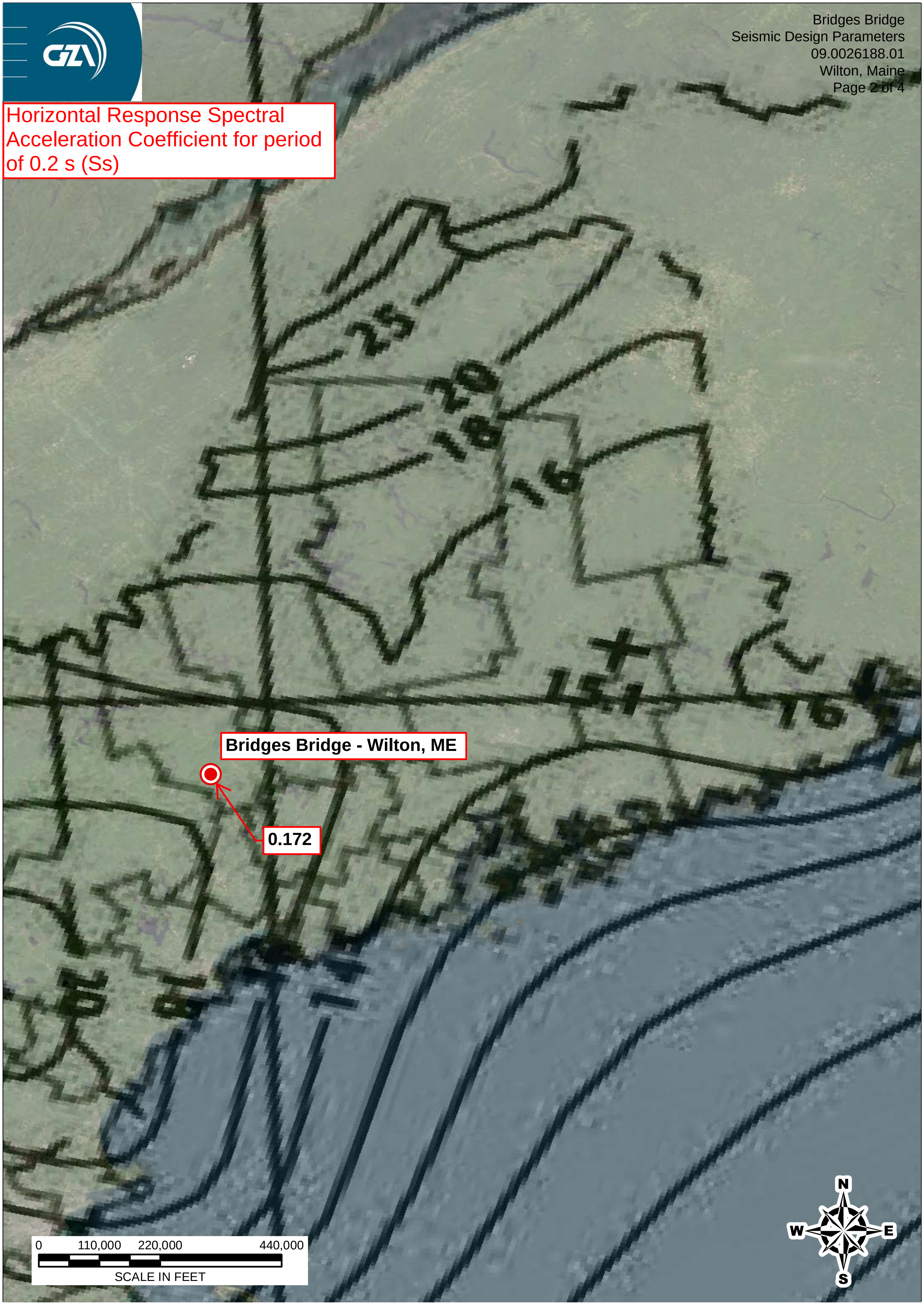
CALCULATION

$\overline{V}$ = 1120.8

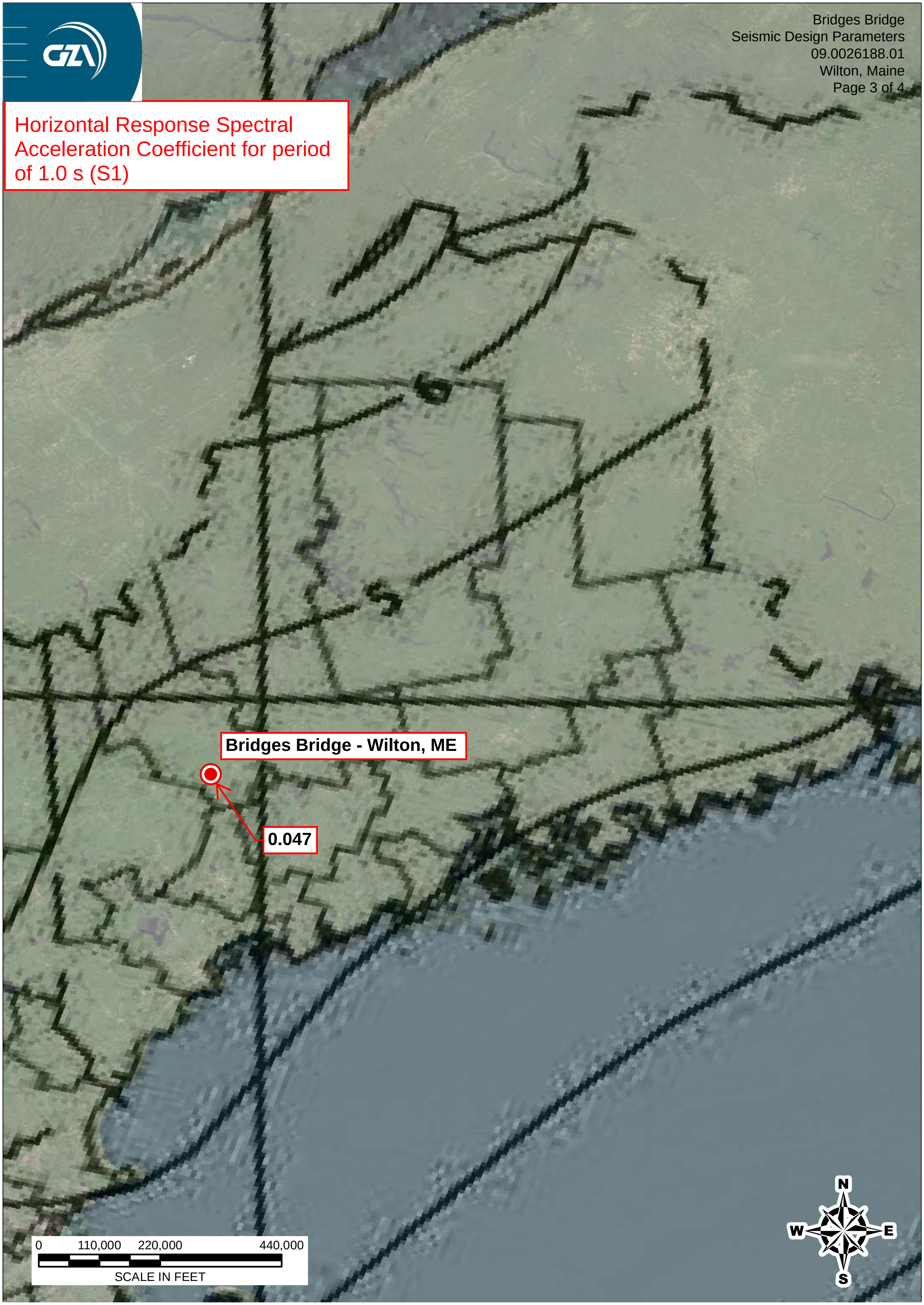
Soil Strata	CPT Interval Depth		SPT Elevation	V_s	d_i	d_i / V_s	Comment
	Top, ft	Bottom, ft	(mid-interval)				
Alluvium	18.0	19.0	563.6	586	20.5	0.03	
	22.0	23.0	559.6	1201	3.5	0.00	
	25.0	26.0	556.6	1004	2.0	0.00	
	26.0	100.0	519.1	1500	74.0	0.05	
Bottom of Boring	33.0						

Horizontal Peak Ground Acceleration Coefficient (PGA)




Bridges Bridge
Seismic Design Parameters
09.0026188.01
Wilton, Maine
Page 2 of 4
Horizontal Response Spectral Acceleration Coefficient for period of 0.2 s (Ss)
25
20
18
16
Bridges Bridge - Wilton, ME
0.172
0
110,000
220,000
440,000
SCALE IN FEET
N
W
E
S

Horizontal Response Spectral Acceleration Coefficient for period of 1.0 s (S1)



Seismic Parameter	Design Parameter[1]
Horizontal Peak ground Acceleration Coefficient	$PGA = .082$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	$S_s = 0.172$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	$S_1 = .047$

Notes: 1. AASHTO Figures 3.10.2.1-1,-2, and -3 were overlaid within GIS software. Coefficients were interpolated between lines on these figures as presented in pages 1 through 3 of this calculation.

For Class D, values of F_{PGA} and F_a = 1.6, and F_v = 2.4

Therefore:

$$A_s = F_{PGA} \times PGA = 1.6 \times 0.082 = 0.131\ g$$

$$S_{DS} = F_a \times S_s = 1.6 \times 0.172 = 0.275\ g$$

$$S_{D1} = F_v \times S_1 = 2.4 \times 0.047 = 0.113\ g$$

Summary:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.13 g
SDs (Period = 0.2 sec)	0.28 g
SD1 (Period = 1.0 sec)	0.11 g

APPENDIX E.3 –LIQUEFACTION ANALYSIS

Bridges Bridge – sCPT Based Liquefaction Susceptibility Calculation

Project:	Bridges Bridge No. 2102	GZA File No.:	09.0026188.01
Location:	Pond Road, Wilton, Maine		
Prepared by:	BMC	Date:	8/13/2024
Checked by:	CLS/ARB	Date:	8/30/2024

Objective

The purpose of this calculation package is to evaluate the liquefaction potential for the loose alluvial deposit found at the Bridges Bridge site located in Wilton Maine. The calculation is based on the raw data collected from the seismic cone penetration tests performed at the site on April 23, 2024 by Seaboard drilling LLC.

References

Documents prepared by GZA and others that were used as a basis of our evaluation include:

1. Boulanger, R.W. and Idriss, I.M. (2014). CPT and SPT Based Liquefaction Triggering Procedures. University of California Davis, Report No. UCS/CGM-14/01.
2. Zhang, G., Robertson. P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168 -1180

Appendices

Appendix A – Unified Hazard Tool Output Files
Appendix B – CLiq Output Files

Procedure

1. Determine seismic site class based on the standard Penetration Tests from the borings and the Shear Wave Velocities from the Seismic Cone Penetration Tests to determine the AASHTO Seismic Site Class.
2. Develop peak ground acceleration (PGA) for the selected Seismic Site Class and the project location for use in the Liquefaction Analysis.
3. Utilize the Unified Hazard Tool by United States Geological Survey (USGS) to evaluate the site-specific earthquake magnitude (M) for use in the Liquefaction Analysis.
4. Input raw sCPT data collected by GZA and site-specific earthquake parameters into Cliq software by Geologismiki to calculate factor of safety against liquefaction triggering and, if necessary, post-liquefaction settlement using the empirical methodology by Boulanger and Idriss (2014).

SECTION 1 –Seismic Site Class and Peak Ground Acceleration

Please refer to the seismic site classification calculation found in Appendix E.2 of the Geotechnical Design Report. The subsurface profile for seismic design includes the approach fills (including backfill behind abutments), Alluvium, and Glacial Till Deposits overlying bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1, considering the average SPT N-values in granular soils. Given an average SPT N-of 28 blows per foot from the borings, and the average shear wave velocity from the sCPTs was 990 feet per second, the bridge site should be classified as Site Class D.

Based on the site coordinates, the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years are summarized for the site are as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.13 g
SDs (Period = 0.2 sec)	0.28 g
SD1 (Period = 1.0 sec)	0.11 g

The PGA used for liquefaction evaluation is calculated as: PGA=Fpga*As. Therefore, the PGA is 0.13 g.

SECTION 2 – Site Specific Earthquake Magnitude

GZA utilized the USGS's online Unified Hazard Tool with PGA as the spectral period, a return period of 975 years, and Site Class B/C boundary, which uses published probabilistic seismic data, to evaluate the mean and mode site-specific earthquake magnitudes. Figure 2.1 below represents the degradation of various magnitude earthquakes and distances from the site. The complete deaggregation report from USGS is included in Appendix A.

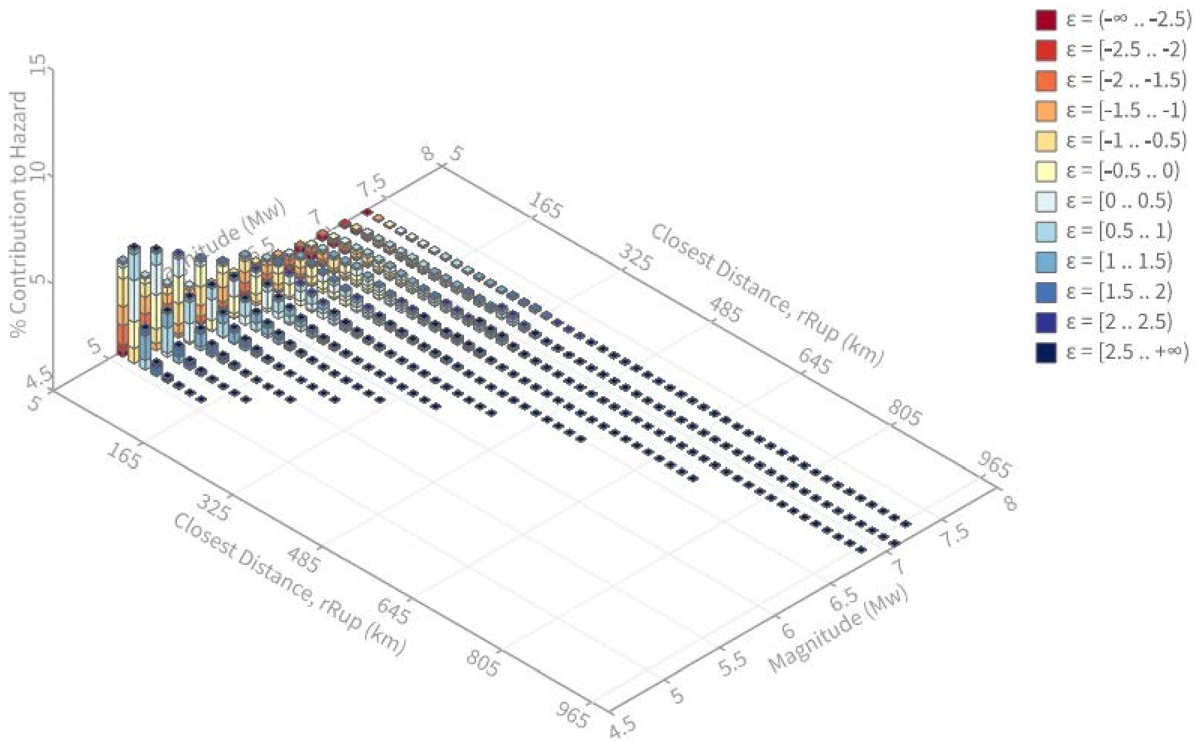


Figure 2.1 – Earthquake Deaggregation Plot, PGA, 975-year event, Site Class B

The Unified Hazard Tool report in Appendix A indicates a mean magnitude 6 earthquake and a mode of 5.1. Considering that Figure 2.1 shows the largest contribution being close, smaller earthquakes with M=5.5 or less, GZA used a magnitude of 5.5 for the liquefaction susceptibility calculation.

SECTION 3 – Liquefaction Susceptibility and Conclusions

GZA input the cone resistance (q_c), sleeve friction (f_s), and piezocone pore pressure (u_2) collected from the sCPTs and the design earthquake and the peak ground acceleration into CLiq by Geologismiki. The Cliq calculation methodology used Boulanger and Idriss (2014) empirical method for the analyses. **Figure 3.1** below shows the resulting magnitude-corrected Cyclic Stress Ratio (CSR*; pink) and magnitude-corrected Cyclic Resistance Ratio ($CRR_{7.5}$; red) and the factor of safety against liquefaction for sCPT-WWS-201, which is the worst of the two sCPTs. The factor of safety (FS) is calculated as FS_{liq}=CRR*/$CSR_{7.5}$. The complete report for both sCPTs is presented in Appendix B. The equations utilized for the Boulanger and Idriss empirical method are also presented in Appendix B.

Bridges Bridge – sCPT Based Liquefaction Susceptibility Calculation

Project:	Bridges Bridge No. 2102	GZA File No.:	09.0026188.01
Location:	Pond Road, Wilton, Maine		
Prepared by:	BMC	Date:	8/13/2024
Checked by:	CLS/ARB	Date:	8/30/2024

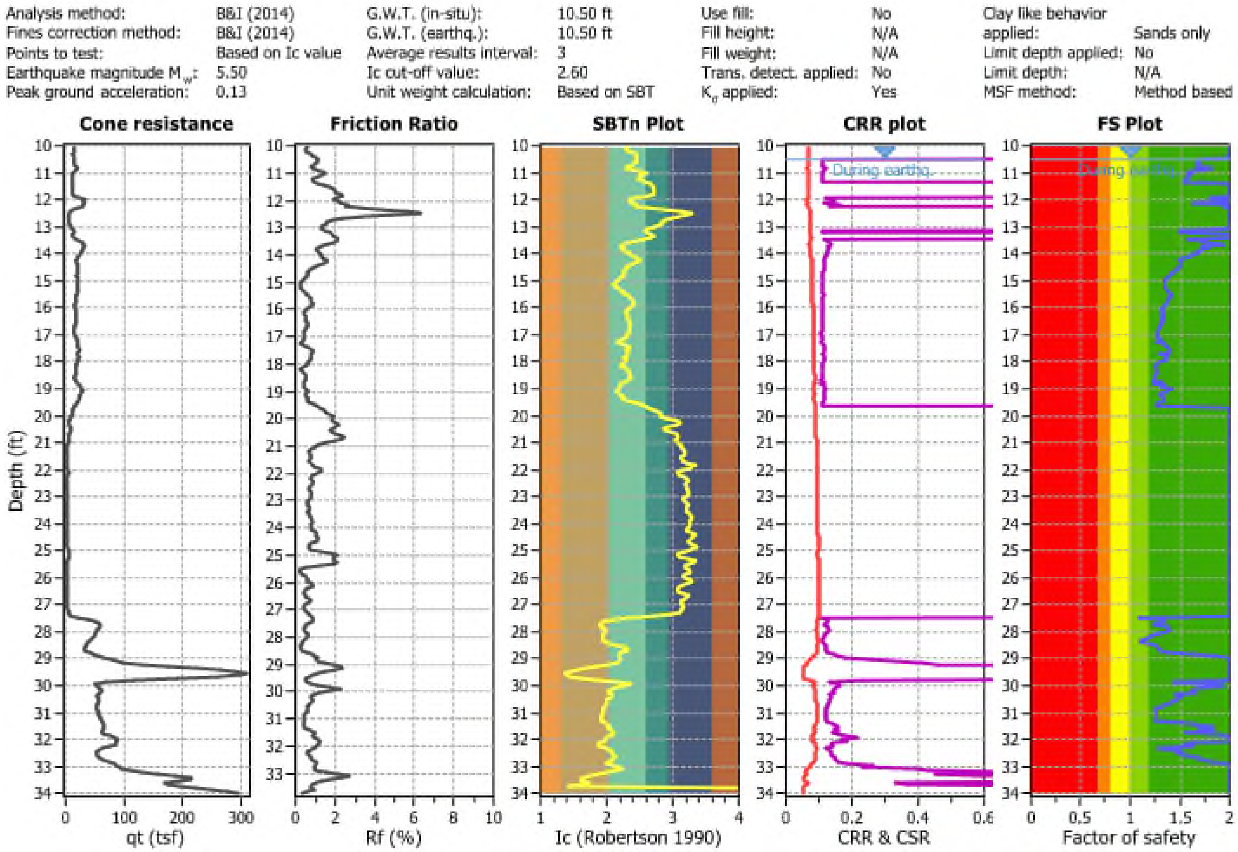


Figure 3.1 – Cliq Result Plots sCPT-WWS-201

CLiq defaults to a factor of safety of 2 for soils that are indicated to have clay-like behavior and be non-liquefiable. There are several thin zones, 1' thick or less, as well as a continuous zone between about 20' and 27.5' in sCPT-WWS-201 and between 19' and 20.5' in sCPT-WWS-202 that show the clay-like behavior. The remainder of the profile was considered to be generally non-cohesive and with fines content that would allow liquefaction so that FS_{liq} values were calculated. The CPT results are indicative of varying fines content and density over short distances, typical of an alluvial deposit. Therefore, SPT results, which by default average susceptibility over a 12" interval every 5', would be expected to be a less reliable indicator of liquefaction susceptibility. Therefore, the borings were not considered for liquefaction potential.

The results indicate the alluvial deposit has a combination of adequate density and fines content to provide a typical FS_{liq} of 1.3 or greater in sCPT-WWS-201 and 1.4 or greater in sCPT-WWS-202, with zones less than 6" thick with lower FS_{liq} values. Therefore, the alluvial layer is judged to have low liquefaction potential based on the sCPT results.

APPENDIX A

Unified Hazard Tool Output Files

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2008 (u…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

44.616

Time Horizon

Return period in years

975

Longitude

Decimal degrees, negative values for western longitudes

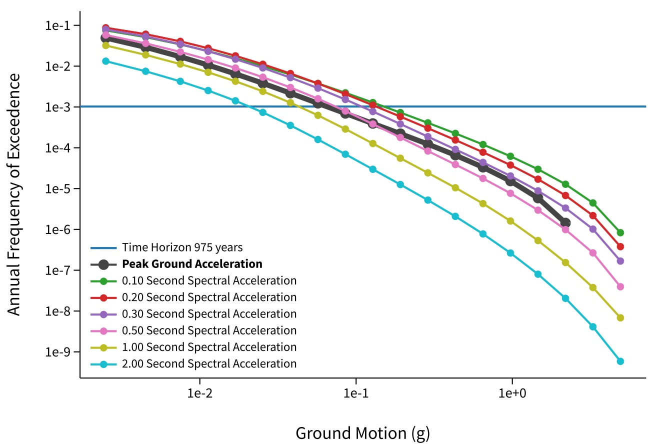
-70.263

Site Class

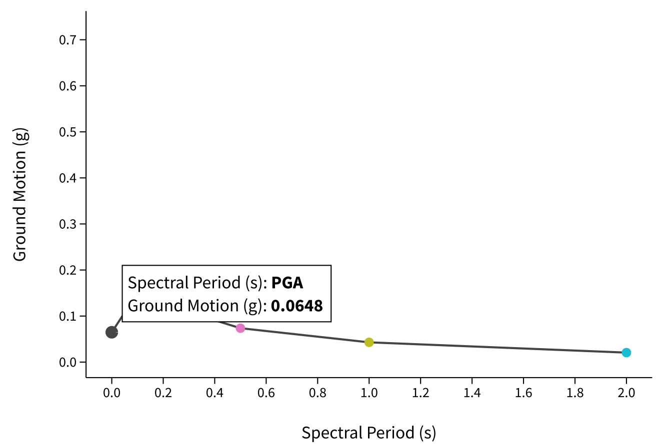
760 m/s (B/C boundary)

Hazard Curve

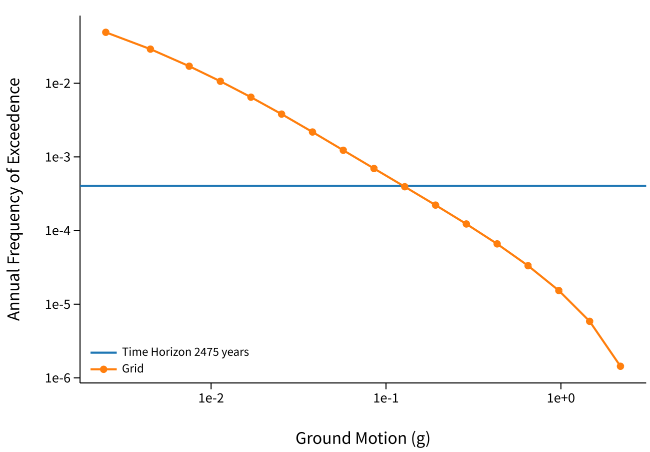
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

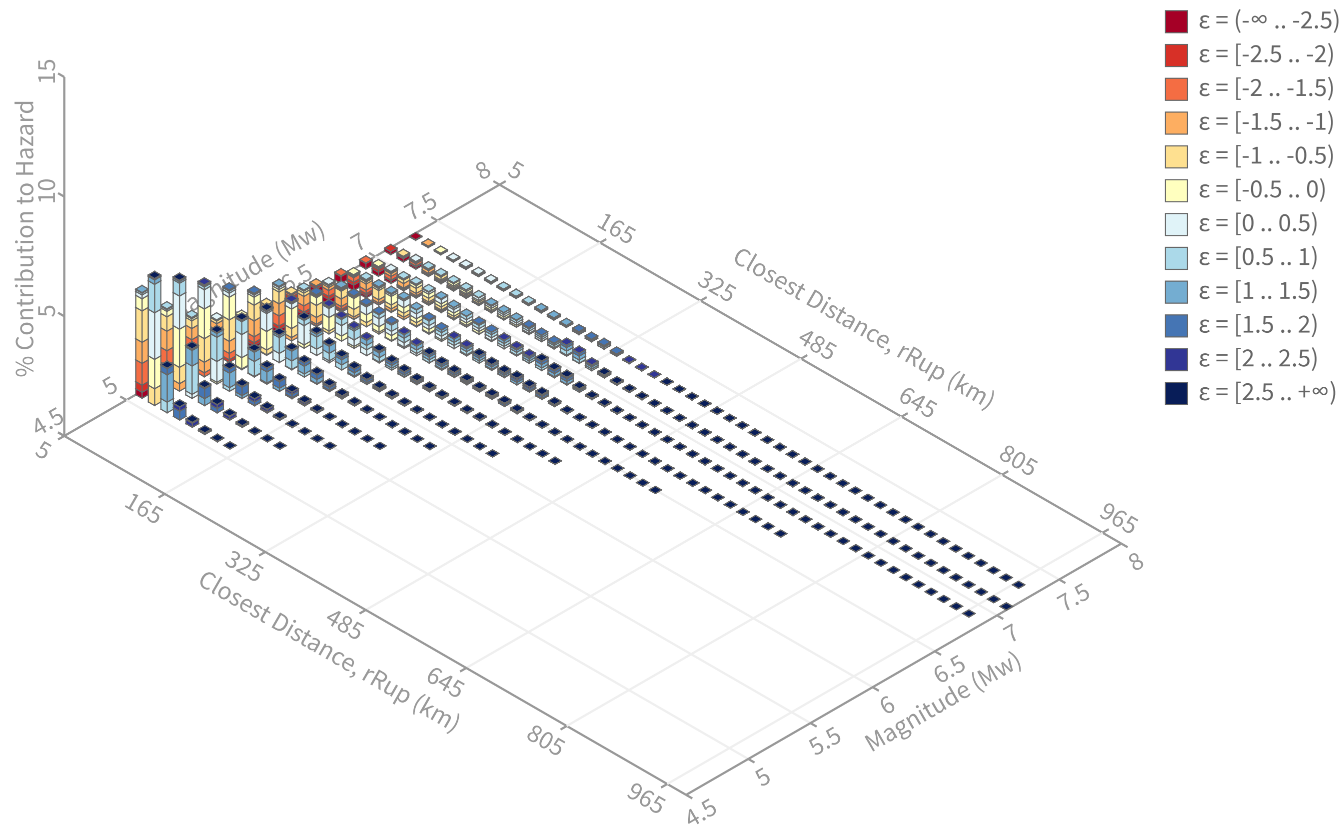


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 975 yrs
Exceedance rate: 0.001025641 yr^{-1}
PGA ground motion: 0.064796525 g

Recovered targets

Return period: 975.27256 yrs
Exceedance rate: 0.0010253544 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 1.19 %

Mean (over all sources)

m: 6
r: 83.24 km
ε_0: -0.11 σ

Mode (largest m-r bin)

m: 5.1
r: 28.92 km
ε_0: 0.17 σ
Contribution: 5.31 %

Mode (largest m-r-ε_0 bin)

m: 5.09
r: 28.78 km
ε_0: 0.23 σ
Contribution: 1.95 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
CEUS.2007all8.J.in (opt)	Grid							55.79
PointSourceFinite: -70.263, 44.818		22.37	5.53	-1.08	70.263°W	44.818°N	0.00	4.24
PointSourceFinite: -70.263, 44.953		36.62	5.67	-0.39	70.263°W	44.953°N	0.00	3.50
PointSourceFinite: -70.263, 44.728		13.06	5.47	-1.84	70.263°W	44.728°N	0.00	3.33
PointSourceFinite: -70.263, 44.863		27.11	5.57	-0.80	70.263°W	44.863°N	0.00	2.66
PointSourceFinite: -70.263, 44.998		41.36	5.72	-0.23	70.263°W	44.998°N	0.00	2.50
PointSourceFinite: -70.263, 44.908		31.87	5.62	-0.57	70.263°W	44.908°N	0.00	2.40
PointSourceFinite: -70.263, 45.043		46.07	5.77	-0.11	70.263°W	45.043°N	0.00	2.35
PointSourceFinite: -70.263, 45.088		50.77	5.83	-0.00	70.263°W	45.088°N	0.00	2.00
PointSourceFinite: -70.263, 44.773		17.66	5.50	-1.41	70.263°W	44.773°N	0.00	1.97
PointSourceFinite: -70.263, 45.133		55.45	5.88	0.09	70.263°W	45.133°N	0.00	1.78
PointSourceFinite: -70.263, 45.223		64.80	5.99	0.23	70.263°W	45.223°N	0.00	1.61
PointSourceFinite: -70.263, 45.178		60.12	5.94	0.16	70.263°W	45.178°N	0.00	1.48
PointSourceFinite: -70.263, 45.358		79.10	6.09	0.38	70.263°W	45.358°N	0.00	1.23
PointSourceFinite: -70.263, 45.448		88.67	6.15	0.41	70.263°W	45.448°N	0.00	1.18
PointSourceFinite: -70.263, 44.683		8.78	5.45	-2.39	70.263°W	44.683°N	0.00	1.15
PointSourceFinite: -70.263, 45.313		74.32	6.06	0.35	70.263°W	45.313°N	0.00	1.12
PointSourceFinite: -70.263, 45.268		69.55	6.03	0.29	70.263°W	45.268°N	0.00	1.09
CEUS.2007all8.AB.in (opt)	Grid							44.21
PointSourceFinite: -70.263, 44.818		22.24	5.56	-0.89	70.263°W	44.818°N	0.00	3.83
PointSourceFinite: -70.263, 44.728		13.02	5.49	-1.63	70.263°W	44.728°N	0.00	3.18
PointSourceFinite: -70.263, 44.953		36.29	5.72	-0.24	70.263°W	44.953°N	0.00	2.93
PointSourceFinite: -70.263, 44.863		26.93	5.61	-0.63	70.263°W	44.863°N	0.00	2.34
PointSourceFinite: -70.263, 44.908		31.62	5.67	-0.41	70.263°W	44.908°N	0.00	2.06
PointSourceFinite: -70.263, 44.998		40.94	5.79	-0.11	70.263°W	44.998°N	0.00	2.05
PointSourceFinite: -70.263, 45.043		45.56	5.85	0.00	70.263°W	45.043°N	0.00	1.90
PointSourceFinite: -70.263, 44.773		17.58	5.52	-1.22	70.263°W	44.773°N	0.00	1.84
PointSourceFinite: -70.263, 45.088		50.17	5.91	0.09	70.263°W	45.088°N	0.00	1.59
PointSourceFinite: -70.263, 45.133		54.75	5.97	0.17	70.263°W	45.133°N	0.00	1.41
PointSourceFinite: -70.263, 45.223		63.93	6.09	0.28	70.263°W	45.223°N	0.00	1.26
PointSourceFinite: -70.263, 45.178		59.33	6.03	0.23	70.263°W	45.178°N	0.00	1.16
PointSourceFinite: -70.263, 44.683		8.76	5.46	-2.16	70.263°W	44.683°N	0.00	1.14

APPENDIX B

CLiq Output Report and References

TABLE OF CONTENTS

GZA GeoEnvironmental, Inc.

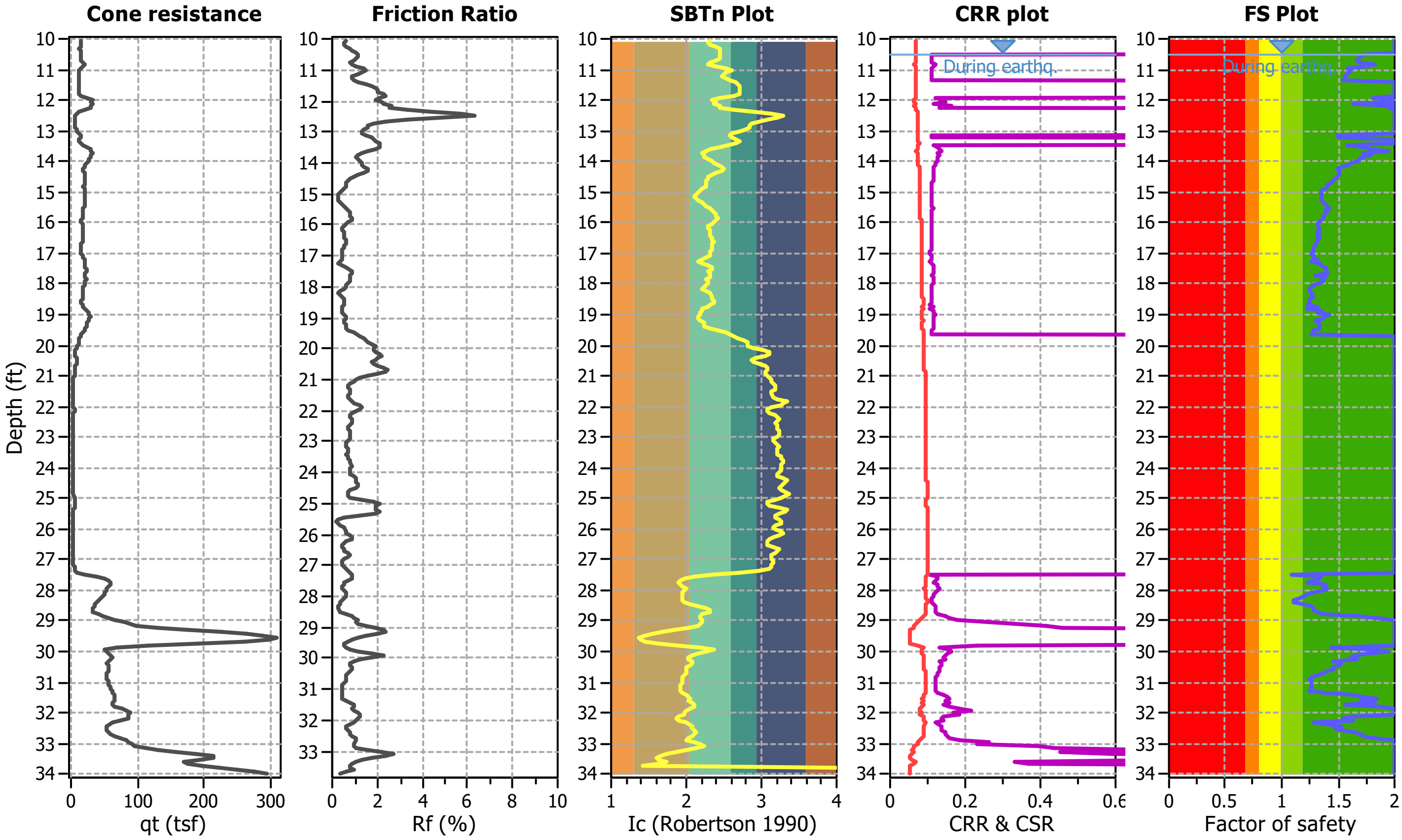
LIQUEFACTION ANALYSIS REPORT

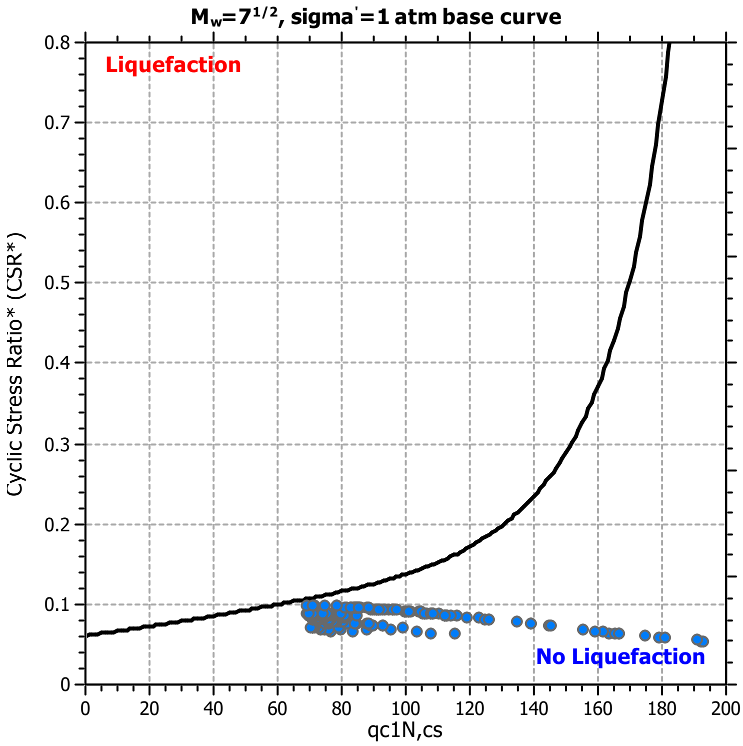
Project title : Bridges Bridge No. **Location : Wilton, Maine**

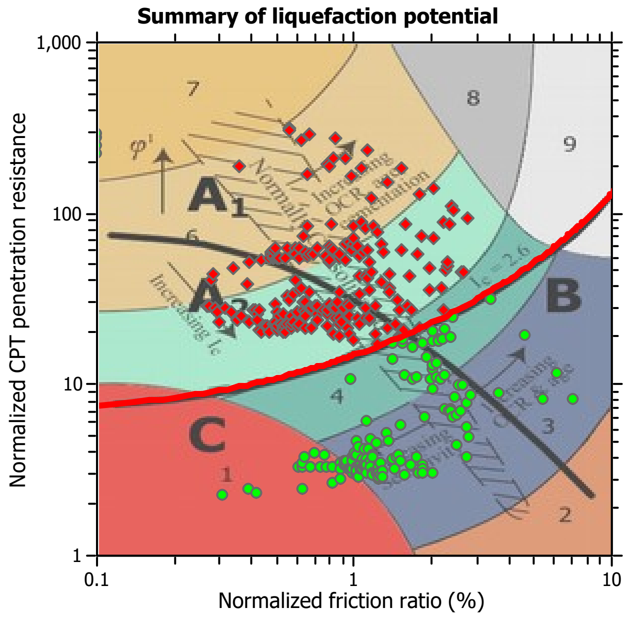
CPT file : sCPT-WWS-201

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	10.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.13	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







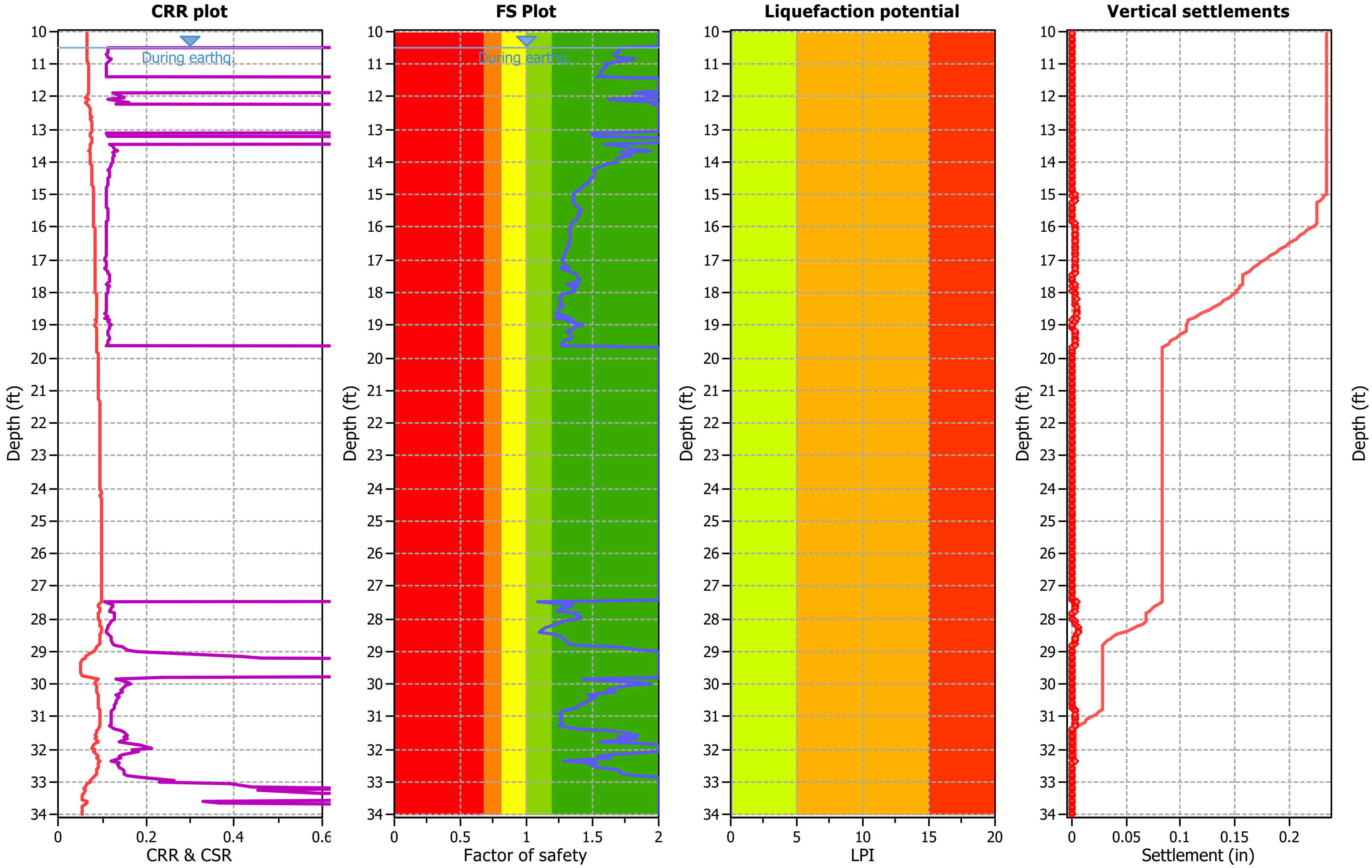
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	5.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data ::

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
1	10.06	0.50	0.00	0.50	0.95	0.080	1.69	0.048	1.06	1.00	2.000	No
2	10.12	0.51	0.00	0.51	0.95	0.080	1.69	0.048	1.06	1.00	2.000	No
3	10.19	0.51	0.00	0.51	0.95	0.080	1.69	0.048	1.06	1.00	2.000	No
4	10.25	0.51	0.00	0.51	0.95	0.080	1.69	0.048	1.06	1.00	2.000	No
5	10.32	0.52	0.00	0.52	0.95	0.080	1.69	0.048	1.06	1.00	2.000	No
6	10.38	0.52	0.00	0.52	0.95	0.080	1.69	0.048	1.06	1.00	2.000	No
7	10.45	0.52	0.00	0.52	0.95	0.080	1.69	0.047	1.06	1.00	2.000	No
8	10.52	0.53	0.00	0.53	0.95	0.080	1.69	0.048	1.06	1.00	0.066	No
9	10.58	0.53	0.00	0.53	0.95	0.080	1.69	0.048	1.06	1.00	0.066	No
10	10.65	0.53	0.00	0.53	0.95	0.081	1.69	0.048	1.06	1.00	0.067	No
11	10.71	0.54	0.01	0.53	0.95	0.081	1.69	0.048	1.06	1.00	0.067	No
12	10.78	0.54	0.01	0.53	0.95	0.081	1.69	0.048	1.06	1.00	0.067	No
13	10.84	0.54	0.01	0.53	0.95	0.081	1.69	0.048	1.06	1.00	0.066	No
14	10.91	0.55	0.01	0.53	0.94	0.082	1.69	0.048	1.06	1.00	0.067	No
15	10.97	0.55	0.01	0.54	0.94	0.082	1.69	0.049	1.06	1.00	0.069	No
16	11.04	0.55	0.02	0.54	0.94	0.082	1.69	0.049	1.06	1.00	0.068	No
17	11.11	0.56	0.02	0.54	0.94	0.083	1.69	0.049	1.06	1.00	0.069	No
18	11.17	0.56	0.02	0.54	0.94	0.083	1.69	0.049	1.06	1.00	0.069	No
19	11.24	0.56	0.02	0.54	0.94	0.083	1.69	0.049	1.06	1.00	0.069	No
20	11.30	0.57	0.03	0.54	0.94	0.083	1.69	0.049	1.06	1.00	0.070	No
21	11.37	0.57	0.03	0.54	0.94	0.084	1.69	0.050	1.06	1.00	0.070	No
22	11.43	0.57	0.03	0.55	0.94	0.084	1.69	0.050	1.06	1.00	0.070	No
23	11.50	0.58	0.03	0.55	0.94	0.084	1.69	0.050	1.06	1.00	0.070	No
24	11.56	0.58	0.03	0.55	0.94	0.084	1.69	0.050	1.06	1.00	0.070	No
25	11.63	0.58	0.04	0.55	0.94	0.085	1.69	0.050	1.06	1.00	0.071	No
26	11.70	0.59	0.04	0.55	0.94	0.085	1.69	0.050	1.06	1.00	0.071	No
27	11.76	0.59	0.04	0.55	0.94	0.085	1.69	0.050	1.06	1.00	0.071	No
28	11.83	0.60	0.04	0.55	0.94	0.085	1.69	0.051	1.06	1.00	0.071	No
29	11.89	0.60	0.04	0.56	0.94	0.085	1.69	0.051	1.06	1.00	0.068	No
30	11.96	0.60	0.05	0.56	0.94	0.086	1.69	0.051	1.07	1.00	0.064	No
31	12.02	0.61	0.05	0.56	0.94	0.086	1.69	0.051	1.07	1.00	0.063	No
32	12.09	0.61	0.05	0.56	0.94	0.086	1.69	0.051	1.06	1.00	0.071	No
33	12.16	0.61	0.05	0.56	0.94	0.086	1.69	0.051	1.08	1.00	0.061	No
34	12.22	0.62	0.05	0.56	0.94	0.087	1.69	0.051	1.06	1.00	0.067	No
35	12.29	0.62	0.06	0.57	0.94	0.087	1.69	0.051	1.06	1.00	0.070	No
36	12.35	0.63	0.06	0.57	0.93	0.087	1.69	0.052	1.05	1.00	0.072	No
37	12.42	0.63	0.06	0.57	0.93	0.087	1.69	0.052	1.05	1.00	0.074	No
38	12.48	0.63	0.06	0.57	0.93	0.087	1.69	0.052	1.05	1.00	0.075	No
39	12.55	0.64	0.06	0.57	0.93	0.088	1.69	0.052	1.05	1.00	0.075	No
40	12.62	0.64	0.07	0.57	0.93	0.088	1.69	0.052	1.05	1.00	0.075	No
41	12.68	0.64	0.07	0.57	0.93	0.088	1.69	0.052	1.05	1.00	0.075	No
42	12.75	0.65	0.07	0.58	0.93	0.088	1.69	0.052	1.05	1.00	0.075	No
43	12.81	0.65	0.07	0.58	0.93	0.089	1.69	0.052	1.05	1.00	0.075	No
44	12.88	0.65	0.07	0.58	0.93	0.089	1.69	0.053	1.05	1.00	0.076	No
45	12.94	0.66	0.08	0.58	0.93	0.089	1.69	0.053	1.05	1.00	0.076	No
46	13.01	0.66	0.08	0.58	0.93	0.089	1.69	0.053	1.05	1.00	0.076	No
47	13.07	0.66	0.08	0.58	0.93	0.089	1.69	0.053	1.05	1.00	0.075	No
48	13.14	0.67	0.08	0.58	0.93	0.090	1.69	0.053	1.05	1.00	0.075	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
49	13.21	0.67	0.08	0.59	0.93	0.090	1.69	0.053	1.05	1.00	0.075	No
50	13.27	0.67	0.09	0.59	0.93	0.090	1.69	0.053	1.05	1.00	0.075	No
51	13.34	0.68	0.09	0.59	0.93	0.090	1.69	0.053	1.05	1.00	0.076	No
52	13.40	0.68	0.09	0.59	0.93	0.090	1.69	0.054	1.05	1.00	0.076	No
53	13.47	0.68	0.09	0.59	0.93	0.091	1.69	0.054	1.05	1.00	0.074	No
54	13.53	0.69	0.09	0.59	0.93	0.091	1.69	0.054	1.06	1.00	0.072	No
55	13.60	0.69	0.10	0.59	0.93	0.091	1.69	0.054	1.06	1.00	0.072	No
56	13.66	0.69	0.10	0.60	0.93	0.091	1.69	0.054	1.06	1.00	0.070	No
57	13.73	0.70	0.10	0.60	0.92	0.091	1.69	0.054	1.06	1.00	0.073	No
58	13.80	0.70	0.10	0.60	0.92	0.091	1.69	0.054	1.06	1.00	0.072	No
59	13.86	0.71	0.10	0.60	0.92	0.092	1.69	0.054	1.06	1.00	0.073	No
60	13.93	0.71	0.11	0.60	0.92	0.092	1.69	0.054	1.05	1.00	0.074	No
61	13.99	0.71	0.11	0.60	0.92	0.092	1.69	0.055	1.05	1.00	0.074	No
62	14.06	0.72	0.11	0.61	0.92	0.092	1.69	0.055	1.05	1.00	0.075	No
63	14.12	0.72	0.11	0.61	0.92	0.092	1.69	0.055	1.05	1.00	0.075	No
64	14.19	0.72	0.12	0.61	0.92	0.093	1.69	0.055	1.05	1.00	0.076	No
65	14.26	0.73	0.12	0.61	0.92	0.093	1.69	0.055	1.05	1.00	0.077	No
66	14.32	0.73	0.12	0.61	0.92	0.093	1.69	0.055	1.05	1.00	0.077	No
67	14.39	0.73	0.12	0.61	0.92	0.093	1.69	0.055	1.05	1.00	0.077	No
68	14.45	0.74	0.12	0.61	0.92	0.093	1.69	0.055	1.05	1.00	0.077	No
69	14.52	0.74	0.13	0.62	0.92	0.093	1.69	0.055	1.05	1.00	0.077	No
70	14.58	0.74	0.13	0.62	0.92	0.094	1.69	0.056	1.05	1.00	0.078	No
71	14.65	0.75	0.13	0.62	0.92	0.094	1.69	0.056	1.05	1.00	0.078	No
72	14.71	0.75	0.13	0.62	0.92	0.094	1.69	0.056	1.05	1.00	0.078	No
73	14.78	0.75	0.13	0.62	0.92	0.094	1.69	0.056	1.05	1.00	0.079	No
74	14.85	0.76	0.14	0.62	0.92	0.094	1.69	0.056	1.05	1.00	0.079	No
75	14.91	0.76	0.14	0.62	0.92	0.094	1.69	0.056	1.05	1.00	0.080	No
76	14.98	0.76	0.14	0.62	0.92	0.095	1.69	0.056	1.04	1.00	0.080	No
77	15.04	0.77	0.14	0.63	0.91	0.095	1.69	0.056	1.04	1.00	0.080	No
78	15.11	0.77	0.14	0.63	0.91	0.095	1.69	0.056	1.04	1.00	0.080	No
79	15.17	0.77	0.15	0.63	0.91	0.095	1.69	0.056	1.04	1.00	0.081	No
80	15.24	0.78	0.15	0.63	0.91	0.095	1.69	0.057	1.04	1.00	0.081	No
81	15.30	0.78	0.15	0.63	0.91	0.095	1.69	0.057	1.04	1.00	0.081	No
82	15.37	0.78	0.15	0.63	0.91	0.096	1.69	0.057	1.04	1.00	0.080	No
83	15.44	0.79	0.15	0.63	0.91	0.096	1.69	0.057	1.05	1.00	0.080	No
84	15.50	0.79	0.16	0.63	0.91	0.096	1.69	0.057	1.05	1.00	0.080	No
85	15.57	0.79	0.16	0.64	0.91	0.096	1.69	0.057	1.04	1.00	0.081	No
86	15.63	0.80	0.16	0.64	0.91	0.096	1.69	0.057	1.04	1.00	0.081	No
87	15.70	0.80	0.16	0.64	0.91	0.096	1.69	0.057	1.04	1.00	0.081	No
88	15.76	0.80	0.16	0.64	0.91	0.097	1.69	0.057	1.04	1.00	0.081	No
89	15.83	0.81	0.17	0.64	0.91	0.097	1.69	0.057	1.04	1.00	0.081	No
90	15.90	0.81	0.17	0.64	0.91	0.097	1.69	0.057	1.04	1.00	0.082	No
91	15.96	0.81	0.17	0.64	0.91	0.097	1.69	0.058	1.04	1.00	0.082	No
92	16.03	0.82	0.17	0.64	0.91	0.097	1.69	0.058	1.04	1.00	0.082	No
93	16.09	0.82	0.17	0.65	0.91	0.097	1.69	0.058	1.04	1.00	0.083	No
94	16.16	0.82	0.18	0.65	0.91	0.097	1.69	0.058	1.04	1.00	0.083	No
95	16.22	0.83	0.18	0.65	0.91	0.098	1.69	0.058	1.04	1.00	0.083	No
96	16.29	0.83	0.18	0.65	0.91	0.098	1.69	0.058	1.04	1.00	0.083	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
97	16.36	0.83	0.18	0.65	0.90	0.098	1.69	0.058	1.04	1.00	0.083	No
98	16.42	0.84	0.18	0.65	0.90	0.098	1.69	0.058	1.04	1.00	0.083	No
99	16.49	0.84	0.19	0.65	0.90	0.098	1.69	0.058	1.04	1.00	0.083	No
100	16.55	0.84	0.19	0.65	0.90	0.098	1.69	0.058	1.04	1.00	0.083	No
101	16.62	0.85	0.19	0.66	0.90	0.099	1.69	0.058	1.04	1.00	0.083	No
102	16.68	0.85	0.19	0.66	0.90	0.099	1.69	0.058	1.04	1.00	0.084	No
103	16.75	0.85	0.19	0.66	0.90	0.099	1.69	0.059	1.04	1.00	0.084	No
104	16.81	0.86	0.20	0.66	0.90	0.099	1.69	0.059	1.04	1.00	0.084	No
105	16.88	0.86	0.20	0.66	0.90	0.099	1.69	0.059	1.04	1.00	0.084	No
106	16.95	0.86	0.20	0.66	0.90	0.099	1.69	0.059	1.04	1.00	0.085	No
107	17.01	0.87	0.20	0.66	0.90	0.099	1.69	0.059	1.04	1.00	0.085	No
108	17.08	0.87	0.21	0.66	0.90	0.099	1.69	0.059	1.04	1.00	0.085	No
109	17.14	0.87	0.21	0.67	0.90	0.100	1.69	0.059	1.04	1.00	0.085	No
110	17.21	0.88	0.21	0.67	0.90	0.100	1.69	0.059	1.04	1.00	0.084	No
111	17.27	0.88	0.21	0.67	0.90	0.100	1.69	0.059	1.04	1.00	0.085	No
112	17.34	0.88	0.21	0.67	0.90	0.100	1.69	0.059	1.04	1.00	0.084	No
113	17.41	0.89	0.22	0.67	0.90	0.100	1.69	0.059	1.04	1.00	0.084	No
114	17.47	0.89	0.22	0.67	0.90	0.100	1.69	0.059	1.04	1.00	0.084	No
115	17.54	0.89	0.22	0.67	0.90	0.100	1.69	0.060	1.04	1.00	0.084	No
116	17.60	0.90	0.22	0.67	0.90	0.101	1.69	0.060	1.04	1.00	0.083	No
117	17.67	0.90	0.22	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.084	No
118	17.73	0.90	0.23	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.085	No
119	17.80	0.91	0.23	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.084	No
120	17.86	0.91	0.23	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.085	No
121	17.93	0.91	0.23	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.085	No
122	18.00	0.92	0.23	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.085	No
123	18.06	0.92	0.24	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.086	No
124	18.13	0.92	0.24	0.68	0.89	0.101	1.69	0.060	1.04	1.00	0.087	No
125	18.19	0.93	0.24	0.69	0.89	0.102	1.69	0.060	1.04	1.00	0.087	No
126	18.26	0.93	0.24	0.69	0.89	0.102	1.69	0.060	1.04	1.00	0.087	No
127	18.32	0.93	0.24	0.69	0.89	0.102	1.69	0.060	1.04	1.00	0.087	No
128	18.39	0.94	0.25	0.69	0.89	0.102	1.69	0.060	1.04	1.00	0.087	No
129	18.45	0.94	0.25	0.69	0.89	0.102	1.69	0.061	1.04	1.00	0.087	No
130	18.52	0.94	0.25	0.69	0.89	0.102	1.69	0.061	1.04	1.00	0.087	No
131	18.59	0.95	0.25	0.69	0.89	0.102	1.69	0.061	1.04	1.00	0.087	No
132	18.65	0.95	0.25	0.69	0.89	0.102	1.69	0.061	1.04	1.00	0.088	No
133	18.72	0.95	0.26	0.70	0.89	0.103	1.69	0.061	1.04	1.00	0.086	No
134	18.78	0.96	0.26	0.70	0.89	0.103	1.69	0.061	1.04	1.00	0.088	No
135	18.85	0.96	0.26	0.70	0.89	0.103	1.69	0.061	1.04	1.00	0.086	No
136	18.91	0.96	0.26	0.70	0.88	0.103	1.69	0.061	1.04	1.00	0.086	No
137	18.98	0.97	0.26	0.70	0.88	0.103	1.69	0.061	1.04	1.00	0.085	No
138	19.05	0.97	0.27	0.70	0.88	0.103	1.69	0.061	1.04	1.00	0.086	No
139	19.11	0.97	0.27	0.70	0.88	0.103	1.69	0.061	1.04	1.00	0.086	No
140	19.18	0.98	0.27	0.71	0.88	0.103	1.69	0.061	1.04	1.00	0.087	No
141	19.24	0.98	0.27	0.71	0.88	0.103	1.69	0.061	1.04	1.00	0.087	No
142	19.31	0.98	0.27	0.71	0.88	0.103	1.69	0.061	1.04	1.00	0.087	No
143	19.37	0.99	0.28	0.71	0.88	0.104	1.69	0.061	1.04	1.00	0.087	No
144	19.44	0.99	0.28	0.71	0.88	0.104	1.69	0.061	1.04	1.00	0.087	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
145	19.50	0.99	0.28	0.71	0.88	0.104	1.69	0.062	1.03	1.00	0.088	No
146	19.57	1.00	0.28	0.71	0.88	0.104	1.69	0.062	1.03	1.00	0.088	No
147	19.64	1.00	0.29	0.71	0.88	0.104	1.69	0.062	1.03	1.00	0.088	No
148	19.70	1.00	0.29	0.72	0.88	0.104	1.69	0.062	1.03	1.00	0.089	No
149	19.77	1.01	0.29	0.72	0.88	0.104	1.69	0.062	1.03	1.00	0.089	No
150	19.83	1.01	0.29	0.72	0.88	0.104	1.69	0.062	1.03	1.00	0.090	No
151	19.90	1.01	0.29	0.72	0.88	0.104	1.69	0.062	1.03	1.00	0.090	No
152	19.96	1.02	0.30	0.72	0.88	0.104	1.69	0.062	1.03	1.00	0.090	No
153	20.03	1.02	0.30	0.72	0.88	0.104	1.69	0.062	1.03	1.00	0.090	No
154	20.09	1.02	0.30	0.72	0.88	0.105	1.69	0.062	1.03	1.00	0.090	No
155	20.16	1.03	0.30	0.73	0.88	0.105	1.69	0.062	1.03	1.00	0.091	No
156	20.23	1.03	0.30	0.73	0.87	0.105	1.69	0.062	1.03	1.00	0.091	No
157	20.29	1.03	0.31	0.73	0.87	0.105	1.69	0.062	1.03	1.00	0.091	No
158	20.36	1.04	0.31	0.73	0.87	0.105	1.69	0.062	1.03	1.00	0.092	No
159	20.42	1.04	0.31	0.73	0.87	0.105	1.69	0.062	1.03	1.00	0.091	No
160	20.49	1.04	0.31	0.73	0.87	0.105	1.69	0.062	1.03	1.00	0.091	No
161	20.55	1.05	0.31	0.73	0.87	0.105	1.69	0.062	1.03	1.00	0.091	No
162	20.62	1.05	0.32	0.73	0.87	0.105	1.69	0.062	1.03	1.00	0.091	No
163	20.69	1.05	0.32	0.74	0.87	0.105	1.69	0.062	1.03	1.00	0.092	No
164	20.75	1.06	0.32	0.74	0.87	0.105	1.69	0.063	1.03	1.00	0.092	No
165	20.82	1.06	0.32	0.74	0.87	0.106	1.69	0.063	1.03	1.00	0.092	No
166	20.88	1.06	0.32	0.74	0.87	0.106	1.69	0.063	1.03	1.00	0.092	No
167	20.95	1.07	0.33	0.74	0.87	0.106	1.69	0.063	1.03	1.00	0.092	No
168	21.01	1.07	0.33	0.74	0.87	0.106	1.69	0.063	1.03	1.00	0.093	No
169	21.08	1.07	0.33	0.74	0.87	0.106	1.69	0.063	1.03	1.00	0.093	No
170	21.14	1.08	0.33	0.74	0.87	0.106	1.69	0.063	1.03	1.00	0.093	No
171	21.21	1.08	0.33	0.74	0.87	0.106	1.69	0.063	1.03	1.00	0.093	No
172	21.28	1.08	0.34	0.75	0.87	0.106	1.69	0.063	1.03	1.00	0.093	No
173	21.34	1.08	0.34	0.75	0.87	0.106	1.69	0.063	1.03	1.00	0.093	No
174	21.41	1.09	0.34	0.75	0.86	0.106	1.69	0.063	1.03	1.00	0.094	No
175	21.47	1.09	0.34	0.75	0.86	0.106	1.69	0.063	1.03	1.00	0.094	No
176	21.54	1.09	0.34	0.75	0.86	0.107	1.69	0.063	1.03	1.00	0.094	No
177	21.60	1.10	0.35	0.75	0.86	0.107	1.69	0.063	1.03	1.00	0.094	No
178	21.67	1.10	0.35	0.75	0.86	0.107	1.69	0.063	1.03	1.00	0.094	No
179	21.74	1.10	0.35	0.75	0.86	0.107	1.69	0.063	1.03	1.00	0.094	No
180	21.80	1.10	0.35	0.75	0.86	0.107	1.69	0.063	1.03	1.00	0.094	No
181	21.87	1.11	0.35	0.75	0.86	0.107	1.69	0.063	1.03	1.00	0.094	No
182	21.93	1.11	0.36	0.75	0.86	0.107	1.69	0.064	1.03	1.00	0.094	No
183	22.00	1.11	0.36	0.75	0.86	0.107	1.69	0.064	1.03	1.00	0.094	No
184	22.06	1.12	0.36	0.76	0.86	0.107	1.69	0.064	1.03	1.00	0.094	No
185	22.13	1.12	0.36	0.76	0.86	0.107	1.69	0.064	1.03	1.00	0.094	No
186	22.20	1.12	0.36	0.76	0.86	0.107	1.69	0.064	1.03	1.00	0.094	No
187	22.26	1.13	0.37	0.76	0.86	0.108	1.69	0.064	1.03	1.00	0.095	No
188	22.33	1.13	0.37	0.76	0.86	0.108	1.69	0.064	1.03	1.00	0.095	No
189	22.39	1.13	0.37	0.76	0.86	0.108	1.69	0.064	1.03	1.00	0.095	No
190	22.46	1.13	0.37	0.76	0.86	0.108	1.69	0.064	1.03	1.00	0.095	No
191	22.52	1.14	0.38	0.76	0.86	0.108	1.69	0.064	1.03	1.00	0.095	No
192	22.59	1.14	0.38	0.76	0.86	0.108	1.69	0.064	1.03	1.00	0.095	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
193	22.65	1.14	0.38	0.76	0.85	0.108	1.69	0.064	1.03	1.00	0.095	No
194	22.72	1.15	0.38	0.76	0.85	0.108	1.69	0.064	1.02	1.00	0.095	No
195	22.79	1.15	0.38	0.76	0.85	0.108	1.69	0.064	1.02	1.00	0.095	No
196	22.85	1.15	0.39	0.77	0.85	0.108	1.69	0.064	1.02	1.00	0.096	No
197	22.92	1.15	0.39	0.77	0.85	0.108	1.69	0.064	1.02	1.00	0.096	No
198	22.98	1.16	0.39	0.77	0.85	0.109	1.69	0.064	1.02	1.00	0.096	No
199	23.05	1.16	0.39	0.77	0.85	0.109	1.69	0.064	1.02	1.00	0.096	No
200	23.11	1.16	0.39	0.77	0.85	0.109	1.69	0.064	1.02	1.00	0.096	No
201	23.18	1.17	0.40	0.77	0.85	0.109	1.69	0.064	1.02	1.00	0.096	No
202	23.25	1.17	0.40	0.77	0.85	0.109	1.69	0.065	1.02	1.00	0.096	No
203	23.31	1.17	0.40	0.77	0.85	0.109	1.69	0.065	1.02	1.00	0.096	No
204	23.38	1.17	0.40	0.77	0.85	0.109	1.69	0.065	1.02	1.00	0.096	No
205	23.44	1.18	0.40	0.77	0.85	0.109	1.69	0.065	1.02	1.00	0.096	No
206	23.51	1.18	0.41	0.77	0.85	0.109	1.69	0.065	1.02	1.00	0.096	No
207	23.57	1.18	0.41	0.77	0.85	0.109	1.69	0.065	1.02	1.00	0.096	No
208	23.64	1.19	0.41	0.78	0.85	0.109	1.69	0.065	1.02	1.00	0.096	No
209	23.70	1.19	0.41	0.78	0.85	0.109	1.69	0.065	1.02	1.00	0.097	No
210	23.77	1.19	0.41	0.78	0.85	0.109	1.69	0.065	1.02	1.00	0.097	No
211	23.84	1.19	0.42	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
212	23.90	1.20	0.42	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
213	23.97	1.20	0.42	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
214	24.03	1.20	0.42	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
215	24.10	1.21	0.42	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
216	24.16	1.21	0.43	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
217	24.23	1.21	0.43	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
218	24.30	1.21	0.43	0.78	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
219	24.36	1.22	0.43	0.79	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
220	24.43	1.22	0.43	0.79	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
221	24.49	1.22	0.44	0.79	0.84	0.110	1.69	0.065	1.02	1.00	0.097	No
222	24.56	1.23	0.44	0.79	0.84	0.110	1.69	0.065	1.02	1.00	0.098	No
223	24.62	1.23	0.44	0.79	0.84	0.110	1.69	0.065	1.02	1.00	0.098	No
224	24.69	1.23	0.44	0.79	0.84	0.111	1.69	0.066	1.02	1.00	0.098	No
225	24.75	1.23	0.44	0.79	0.84	0.111	1.69	0.066	1.02	1.00	0.098	No
226	24.82	1.24	0.45	0.79	0.84	0.111	1.69	0.066	1.02	1.00	0.098	No
227	24.89	1.24	0.45	0.79	0.84	0.111	1.69	0.066	1.02	1.00	0.098	No
228	24.95	1.24	0.45	0.79	0.84	0.111	1.69	0.066	1.02	1.00	0.098	No
229	25.02	1.25	0.45	0.79	0.84	0.111	1.69	0.066	1.02	1.00	0.097	No
230	25.08	1.25	0.45	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.097	No
231	25.15	1.25	0.46	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.097	No
232	25.21	1.26	0.46	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.097	No
233	25.28	1.26	0.46	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.098	No
234	25.34	1.26	0.46	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.098	No
235	25.41	1.27	0.47	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.098	No
236	25.48	1.27	0.47	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.098	No
237	25.54	1.27	0.47	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.098	No
238	25.61	1.27	0.47	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.099	No
239	25.67	1.28	0.47	0.80	0.83	0.111	1.69	0.066	1.02	1.00	0.099	No
240	25.74	1.28	0.48	0.81	0.83	0.111	1.69	0.066	1.02	1.00	0.099	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR*	Belongs to transition
241	25.80	1.28	0.48	0.81	0.83	0.111	1.69	0.066	1.02	1.00	0.099	No
242	25.87	1.29	0.48	0.81	0.83	0.112	1.69	0.066	1.02	1.00	0.099	No
243	25.93	1.29	0.48	0.81	0.83	0.112	1.69	0.066	1.02	1.00	0.099	No
244	26.00	1.29	0.48	0.81	0.83	0.112	1.69	0.066	1.02	1.00	0.099	No
245	26.07	1.30	0.49	0.81	0.83	0.112	1.69	0.066	1.02	1.00	0.099	No
246	26.13	1.30	0.49	0.81	0.83	0.112	1.69	0.066	1.02	1.00	0.099	No
247	26.20	1.30	0.49	0.81	0.83	0.112	1.69	0.066	1.02	1.00	0.099	No
248	26.26	1.30	0.49	0.81	0.82	0.112	1.69	0.066	1.02	1.00	0.099	No
249	26.33	1.31	0.49	0.81	0.82	0.112	1.69	0.066	1.02	1.00	0.099	No
250	26.39	1.31	0.50	0.81	0.82	0.112	1.69	0.066	1.02	1.00	0.099	No
251	26.46	1.31	0.50	0.81	0.82	0.112	1.69	0.066	1.02	1.00	0.099	No
252	26.53	1.32	0.50	0.82	0.82	0.112	1.69	0.066	1.02	1.00	0.099	No
253	26.59	1.32	0.50	0.82	0.82	0.112	1.69	0.067	1.02	1.00	0.099	No
254	26.66	1.32	0.50	0.82	0.82	0.112	1.69	0.067	1.02	1.00	0.099	No
255	26.72	1.32	0.51	0.82	0.82	0.112	1.69	0.067	1.02	1.00	0.099	No
256	26.79	1.33	0.51	0.82	0.82	0.112	1.69	0.067	1.02	1.00	0.099	No
257	26.85	1.33	0.51	0.82	0.82	0.112	1.69	0.067	1.02	1.00	0.100	No
258	26.92	1.33	0.51	0.82	0.82	0.112	1.69	0.067	1.02	1.00	0.100	No
259	26.98	1.34	0.51	0.82	0.82	0.113	1.69	0.067	1.02	1.00	0.100	No
260	27.05	1.34	0.52	0.82	0.82	0.113	1.69	0.067	1.02	1.00	0.100	No
261	27.12	1.34	0.52	0.82	0.82	0.113	1.69	0.067	1.02	1.00	0.100	No
262	27.18	1.34	0.52	0.82	0.82	0.113	1.69	0.067	1.02	1.00	0.100	No
263	27.25	1.35	0.52	0.82	0.82	0.113	1.69	0.067	1.02	1.00	0.100	No
264	27.31	1.35	0.52	0.83	0.82	0.113	1.69	0.067	1.02	1.00	0.100	No
265	27.38	1.35	0.53	0.83	0.82	0.113	1.69	0.067	1.02	1.00	0.100	No
266	27.44	1.36	0.53	0.83	0.81	0.113	1.69	0.067	1.02	1.00	0.099	No
267	27.49	1.36	0.53	0.83	0.81	0.113	1.69	0.067	1.02	1.00	0.098	No
268	27.56	1.36	0.53	0.83	0.81	0.113	1.69	0.067	1.02	1.00	0.093	No
269	27.63	1.37	0.53	0.83	0.81	0.113	1.69	0.067	1.02	1.00	0.093	No
270	27.69	1.37	0.54	0.83	0.81	0.113	1.69	0.067	1.02	1.00	0.095	No
271	27.76	1.37	0.54	0.83	0.81	0.113	1.69	0.067	1.02	1.00	0.095	No
272	27.82	1.38	0.54	0.84	0.81	0.113	1.69	0.067	1.02	1.00	0.092	No
273	27.89	1.38	0.54	0.84	0.81	0.113	1.69	0.067	1.02	1.00	0.093	No
274	27.95	1.38	0.54	0.84	0.81	0.113	1.69	0.067	1.02	1.00	0.092	No
275	28.02	1.39	0.55	0.84	0.81	0.113	1.69	0.067	1.02	1.00	0.093	No
276	28.08	1.39	0.55	0.84	0.81	0.113	1.69	0.067	1.02	1.00	0.094	No
277	28.15	1.40	0.55	0.84	0.81	0.113	1.69	0.067	1.02	1.00	0.096	No
278	28.21	1.40	0.55	0.85	0.81	0.113	1.69	0.067	1.02	1.00	0.096	No
279	28.28	1.40	0.55	0.85	0.81	0.113	1.69	0.067	1.02	1.00	0.097	No
280	28.35	1.41	0.56	0.85	0.81	0.113	1.69	0.067	1.02	1.00	0.098	No
281	28.41	1.41	0.56	0.85	0.81	0.113	1.69	0.067	1.02	1.00	0.098	No
282	28.48	1.41	0.56	0.85	0.81	0.113	1.69	0.067	1.02	1.00	0.096	No
283	28.54	1.42	0.56	0.85	0.81	0.113	1.69	0.067	1.02	1.00	0.095	No
284	28.61	1.42	0.57	0.85	0.80	0.113	1.69	0.067	1.02	1.00	0.095	No
285	28.68	1.42	0.57	0.86	0.80	0.113	1.69	0.067	1.02	1.00	0.095	No
286	28.74	1.43	0.57	0.86	0.80	0.113	1.69	0.067	1.02	1.00	0.094	No
287	28.81	1.43	0.57	0.86	0.80	0.113	1.69	0.067	1.02	1.00	0.093	No
288	28.87	1.43	0.57	0.86	0.80	0.113	1.69	0.067	1.02	1.00	0.088	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR*	Belongs to transition
289	28.94	1.44	0.58	0.86	0.80	0.113	1.69	0.067	1.02	1.00	0.085	No
290	29.00	1.44	0.58	0.86	0.80	0.113	1.69	0.067	1.03	1.00	0.081	No
291	29.07	1.45	0.58	0.87	0.80	0.113	1.69	0.067	1.03	1.00	0.072	No
292	29.13	1.45	0.58	0.87	0.80	0.113	1.69	0.067	1.04	1.00	0.063	No
293	29.20	1.45	0.58	0.87	0.80	0.113	1.69	0.067	1.04	1.00	0.062	No
294	29.27	1.46	0.59	0.87	0.80	0.113	1.69	0.067	1.05	1.00	0.053	No
295	29.33	1.46	0.59	0.88	0.80	0.113	1.69	0.067	1.06	1.00	0.053	No
296	29.40	1.47	0.59	0.88	0.80	0.113	1.69	0.067	1.06	1.00	0.053	No
297	29.46	1.47	0.59	0.88	0.80	0.113	1.69	0.067	1.06	1.00	0.053	No
298	29.53	1.48	0.59	0.88	0.80	0.113	1.69	0.067	1.05	1.00	0.053	No
299	29.59	1.48	0.60	0.88	0.80	0.113	1.69	0.067	1.05	1.00	0.053	No
300	29.66	1.48	0.60	0.89	0.80	0.113	1.69	0.067	1.05	1.00	0.053	No
301	29.72	1.49	0.60	0.89	0.80	0.113	1.69	0.067	1.04	1.00	0.053	No
302	29.79	1.49	0.60	0.89	0.79	0.113	1.69	0.067	1.03	1.00	0.074	No
303	29.86	1.50	0.60	0.89	0.79	0.113	1.69	0.067	1.02	1.00	0.092	No
304	29.92	1.50	0.61	0.89	0.79	0.113	1.69	0.067	1.02	1.00	0.086	No
305	29.99	1.50	0.61	0.90	0.79	0.112	1.69	0.067	1.02	1.00	0.084	No
306	30.05	1.51	0.61	0.90	0.79	0.112	1.69	0.067	1.02	1.00	0.086	No
307	30.12	1.51	0.61	0.90	0.79	0.112	1.69	0.067	1.02	1.00	0.088	No
308	30.18	1.52	0.61	0.90	0.79	0.112	1.69	0.067	1.02	1.00	0.089	No
309	30.25	1.52	0.62	0.90	0.79	0.112	1.69	0.067	1.02	1.00	0.088	No
310	30.32	1.52	0.62	0.91	0.79	0.112	1.69	0.067	1.02	1.00	0.091	No
311	30.38	1.53	0.62	0.91	0.79	0.112	1.69	0.067	1.02	1.00	0.090	No
312	30.45	1.53	0.62	0.91	0.79	0.112	1.69	0.067	1.02	1.00	0.090	No
313	30.51	1.53	0.62	0.91	0.79	0.112	1.69	0.067	1.02	1.00	0.091	No
314	30.58	1.54	0.63	0.91	0.79	0.112	1.69	0.067	1.02	1.00	0.092	No
315	30.64	1.54	0.63	0.91	0.79	0.112	1.69	0.067	1.01	1.00	0.092	No
316	30.71	1.55	0.63	0.92	0.79	0.112	1.69	0.067	1.01	1.00	0.093	No
317	30.77	1.55	0.63	0.92	0.79	0.112	1.69	0.067	1.01	1.00	0.094	No
318	30.84	1.55	0.63	0.92	0.79	0.112	1.69	0.067	1.01	1.00	0.095	No
319	30.91	1.56	0.64	0.92	0.79	0.112	1.69	0.067	1.01	1.00	0.095	No
320	30.97	1.56	0.64	0.92	0.78	0.112	1.69	0.067	1.01	1.00	0.095	No
321	31.04	1.56	0.64	0.92	0.78	0.112	1.69	0.067	1.01	1.00	0.095	No
322	31.10	1.57	0.64	0.92	0.78	0.112	1.69	0.067	1.01	1.00	0.095	No
323	31.17	1.57	0.64	0.93	0.78	0.112	1.69	0.067	1.01	1.00	0.095	No
324	31.23	1.57	0.65	0.93	0.78	0.112	1.69	0.067	1.01	1.00	0.095	No
325	31.30	1.58	0.65	0.93	0.78	0.112	1.69	0.067	1.01	1.00	0.095	No
326	31.36	1.58	0.65	0.93	0.78	0.112	1.69	0.067	1.01	1.00	0.093	No
327	31.43	1.59	0.65	0.93	0.78	0.112	1.69	0.067	1.01	1.00	0.089	No
328	31.50	1.59	0.66	0.93	0.78	0.112	1.69	0.067	1.01	1.00	0.087	No
329	31.56	1.59	0.66	0.94	0.78	0.112	1.69	0.066	1.01	1.00	0.086	No
330	31.63	1.60	0.66	0.94	0.78	0.112	1.69	0.066	1.01	1.00	0.088	No
331	31.69	1.60	0.66	0.94	0.78	0.112	1.69	0.066	1.01	1.00	0.086	No
332	31.76	1.60	0.66	0.94	0.78	0.112	1.69	0.066	1.01	1.00	0.090	No
333	31.82	1.61	0.67	0.94	0.78	0.112	1.69	0.066	1.01	1.00	0.085	No
334	31.89	1.61	0.67	0.95	0.78	0.112	1.69	0.066	1.01	1.00	0.080	No
335	31.95	1.62	0.67	0.95	0.78	0.112	1.69	0.066	1.02	1.00	0.077	No
336	32.02	1.62	0.67	0.95	0.78	0.112	1.69	0.066	1.01	1.00	0.083	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
337	32.09	1.62	0.67	0.95	0.78	0.112	1.69	0.066	1.01	1.00	0.081	No
338	32.15	1.63	0.68	0.95	0.77	0.112	1.69	0.066	1.01	1.00	0.089	No
339	32.22	1.63	0.68	0.95	0.77	0.112	1.69	0.066	1.01	1.00	0.090	No
340	32.28	1.64	0.68	0.96	0.77	0.112	1.69	0.066	1.01	1.00	0.089	No
341	32.35	1.64	0.68	0.96	0.77	0.112	1.69	0.066	1.01	1.00	0.095	No
342	32.41	1.64	0.68	0.96	0.77	0.112	1.69	0.066	1.01	1.00	0.090	No
343	32.48	1.65	0.69	0.96	0.77	0.112	1.69	0.066	1.01	1.00	0.091	No
344	32.55	1.65	0.69	0.96	0.77	0.112	1.69	0.066	1.01	1.00	0.090	No
345	32.61	1.65	0.69	0.96	0.77	0.112	1.69	0.066	1.01	1.00	0.088	No
346	32.68	1.66	0.69	0.97	0.77	0.112	1.69	0.066	1.01	1.00	0.088	No
347	32.74	1.66	0.69	0.97	0.77	0.112	1.69	0.066	1.01	1.00	0.087	No
348	32.81	1.67	0.70	0.97	0.77	0.112	1.69	0.066	1.01	1.00	0.086	No
349	32.87	1.67	0.70	0.97	0.77	0.112	1.69	0.066	1.01	1.00	0.080	No
350	32.94	1.67	0.70	0.97	0.77	0.112	1.69	0.066	1.01	1.00	0.072	No
351	33.01	1.68	0.70	0.98	0.77	0.112	1.69	0.066	1.01	1.00	0.075	No
352	33.07	1.68	0.70	0.98	0.77	0.111	1.69	0.066	1.01	1.00	0.065	No
353	33.14	1.69	0.71	0.98	0.77	0.111	1.69	0.066	1.01	1.00	0.063	No
354	33.20	1.69	0.71	0.98	0.77	0.111	1.69	0.066	1.02	1.00	0.057	No
355	33.27	1.70	0.71	0.99	0.77	0.111	1.69	0.066	1.01	1.00	0.062	No
356	33.33	1.70	0.71	0.99	0.76	0.111	1.69	0.066	1.01	1.00	0.059	No
357	33.40	1.70	0.71	0.99	0.76	0.111	1.69	0.066	1.02	1.00	0.054	No
358	33.47	1.71	0.72	0.99	0.76	0.111	1.69	0.066	1.02	1.00	0.054	No
359	33.53	1.71	0.72	0.99	0.76	0.111	1.69	0.066	1.01	1.00	0.056	No
360	33.60	1.72	0.72	1.00	0.76	0.111	1.69	0.066	1.01	1.00	0.067	No
361	33.66	1.72	0.72	1.00	0.76	0.111	1.69	0.066	1.01	1.00	0.066	No
362	33.73	1.72	0.72	1.00	0.76	0.111	1.69	0.066	1.01	1.00	0.054	No
363	33.79	1.73	0.73	1.00	0.76	0.111	1.69	0.066	1.02	1.00	0.054	No
364	33.86	1.73	0.73	1.00	0.76	0.111	1.69	0.066	1.02	1.00	0.054	No
365	33.92	1.73	0.73	1.00	0.76	0.111	1.69	0.066	1.02	1.00	0.054	No
366	33.99	1.74	0.73	1.00	0.76	0.111	1.69	0.066	1.02	1.00	0.054	No

Abbreviations

Depth: Depth from free surface, at which CPT was performed (ft)
σ_v: Total overburden pressure at test point (tsf)
u_0: Water pressure at test point (tsf)
σ_v': Effective overburden pressure based on GWT during earthquake (tsf)
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Magnitude Scaling Factor
CSR_{eq}: CSR adjusted for M=7.5
K_σ: Effective overburden stress factor
CSR^*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) calculation data ::

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
1	10.06	16.12	47.95	2.31	0.56	1.52	22.67	51.22	73.89	4.000	No	No	2.00
2	10.12	16.28	45.81	2.29	0.56	1.51	23.90	50.64	74.54	4.000	No	No	2.00
3	10.19	15.14	48.47	2.32	0.56	1.50	23.14	51.55	74.70	4.000	No	No	2.00
4	10.25	13.84	53.80	2.39	0.57	1.51	17.72	51.97	69.69	4.000	No	No	2.00
5	10.32	12.98	58.85	2.45	0.57	1.50	18.26	53.54	71.80	4.000	No	No	2.00
6	10.38	13.69	59.18	2.45	0.56	1.49	19.30	53.92	73.22	4.000	No	No	2.00
7	10.45	14.67	59.06	2.45	0.56	1.48	20.34	54.18	74.52	4.000	No	No	2.00
8	10.52	15.34	59.19	2.45	0.55	1.47	22.00	54.67	76.68	0.113	No	No	1.71
9	10.58	15.73	58.53	2.44	0.56	1.47	21.82	54.45	76.28	0.113	No	No	1.70
10	10.65	15.65	57.89	2.44	0.56	1.47	21.80	54.27	76.07	0.112	No	No	1.69
11	10.71	16.31	54.30	2.39	0.56	1.47	21.65	53.19	74.84	0.111	No	No	1.66
12	10.78	18.66	47.75	2.31	0.56	1.47	24.50	51.62	76.12	0.112	No	No	1.68
13	10.84	19.80	46.81	2.30	0.54	1.44	31.04	52.92	83.95	0.120	No	No	1.82
14	10.91	17.95	53.72	2.38	0.55	1.45	26.03	54.20	80.22	0.116	No	No	1.74
15	10.97	14.65	64.92	2.52	0.57	1.47	16.89	54.56	71.46	0.109	No	No	1.58
16	11.04	12.64	70.54	2.59	0.56	1.46	17.78	55.89	73.67	0.110	No	No	1.61
17	11.11	12.84	65.77	2.53	0.57	1.46	17.94	55.04	72.98	0.110	No	No	1.60
18	11.17	12.72	61.97	2.49	0.57	1.47	17.63	54.13	71.76	0.109	No	No	1.57
19	11.24	12.44	61.36	2.48	0.57	1.47	17.30	53.89	71.19	0.108	No	No	1.56
20	11.30	11.97	64.67	2.52	0.57	1.46	16.78	54.48	71.26	0.108	No	No	1.56
21	11.37	11.45	69.77	2.58	0.57	1.46	15.63	55.13	70.76	0.108	No	No	1.54
22	11.43	11.22	74.64	2.65	0.57	1.46	15.12	0.00	15.12	4.000	No	Yes	2.00
23	11.50	11.34	77.74	2.68	0.57	1.45	15.71	0.00	15.71	4.000	No	Yes	2.00
24	11.56	11.61	78.83	2.70	0.57	1.45	16.00	0.00	16.00	4.000	No	Yes	2.00
25	11.63	11.77	79.27	2.70	0.57	1.45	16.13	0.00	16.13	4.000	No	Yes	2.00
26	11.70	11.93	78.87	2.70	0.57	1.45	16.26	0.00	16.26	4.000	No	Yes	2.00
27	11.76	12.27	80.23	2.72	0.56	1.44	16.54	0.00	16.54	4.000	No	Yes	2.00
28	11.83	16.00	72.90	2.62	0.56	1.44	17.46	0.00	17.46	4.000	No	Yes	2.00
29	11.89	23.69	59.66	2.46	0.53	1.40	30.62	57.21	87.84	0.123	No	No	1.81
30	11.96	31.75	50.19	2.34	0.49	1.37	45.58	58.16	103.74	0.143	No	No	2.00
31	12.02	30.37	54.53	2.39	0.48	1.36	47.56	60.33	107.89	0.149	No	No	2.00
32	12.09	32.48	53.25	2.38	0.55	1.42	25.27	53.84	79.11	0.115	No	No	1.63
33	12.16	29.59	58.86	2.45	0.47	1.34	52.65	63.14	115.79	0.163	No	No	2.00
34	12.22	29.86	57.93	2.44	0.51	1.38	36.86	58.47	95.32	0.132	No	No	1.95
35	12.29	20.07	74.31	2.64	0.53	1.40	26.00	0.00	26.00	4.000	No	Yes	2.00
36	12.35	12.49	94.05	2.89	0.56	1.42	16.29	0.00	16.29	4.000	No	Yes	2.00
37	12.42	7.77	100.00	3.14	0.59	1.44	7.58	0.00	7.58	4.000	No	Yes	2.00
38	12.48	5.69	100.00	3.29	0.59	1.44	7.57	0.00	7.57	4.000	No	Yes	2.00
39	12.55	5.70	100.00	3.22	0.59	1.44	8.03	0.00	8.03	4.000	No	Yes	2.00
40	12.62	6.17	100.00	3.09	0.59	1.44	7.56	0.00	7.56	4.000	No	Yes	2.00
41	12.68	6.68	100.00	2.97	0.58	1.43	9.42	0.00	9.42	4.000	No	Yes	2.00
42	12.75	7.27	93.10	2.88	0.58	1.43	10.04	0.00	10.04	4.000	No	Yes	2.00
43	12.81	7.39	90.39	2.84	0.59	1.43	9.87	0.00	9.87	4.000	No	Yes	2.00
44	12.88	7.34	89.92	2.84	0.59	1.42	9.87	0.00	9.87	4.000	No	Yes	2.00
45	12.94	7.74	86.89	2.80	0.59	1.42	9.85	0.00	9.85	4.000	No	Yes	2.00
46	13.01	9.27	79.26	2.70	0.58	1.42	11.41	0.00	11.41	4.000	No	Yes	2.00
47	13.07	11.63	72.23	2.62	0.57	1.41	15.84	0.00	15.84	4.000	No	Yes	2.00
48	13.14	13.78	69.08	2.58	0.56	1.40	18.99	55.98	74.97	0.112	No	No	1.49

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
49	13.21	14.24	70.82	2.60	0.56	1.39	19.69	56.50	76.18	0.113	No	No	1.51
50	13.27	13.36	74.77	2.65	0.56	1.39	17.54	0.00	17.54	4.000	No	Yes	2.00
51	13.34	12.01	79.51	2.71	0.57	1.40	15.54	0.00	15.54	4.000	No	Yes	2.00
52	13.40	13.86	75.93	2.66	0.57	1.40	14.48	0.00	14.48	4.000	No	Yes	2.00
53	13.47	18.46	68.03	2.56	0.54	1.37	24.44	57.36	81.80	0.118	No	No	1.58
54	13.53	24.63	57.98	2.44	0.52	1.35	32.71	57.33	90.03	0.126	No	No	1.74
55	13.60	30.01	49.20	2.33	0.51	1.34	37.46	55.61	93.06	0.129	No	No	1.80
56	13.66	31.38	44.82	2.27	0.50	1.33	44.07	55.31	99.38	0.137	No	No	1.94
57	13.73	32.24	40.87	2.22	0.52	1.35	37.84	51.59	89.43	0.125	No	No	1.71
58	13.80	30.43	41.58	2.23	0.51	1.34	40.57	52.67	93.25	0.129	No	No	1.79
59	13.86	29.61	42.76	2.25	0.52	1.34	37.55	52.58	90.14	0.126	No	No	1.72
60	13.93	27.85	46.71	2.30	0.52	1.34	34.60	53.79	88.39	0.124	No	No	1.68
61	13.99	25.97	49.96	2.34	0.52	1.34	33.93	54.98	88.91	0.125	No	No	1.69
62	14.06	24.37	51.88	2.36	0.53	1.35	30.46	54.77	85.23	0.121	No	No	1.61
63	14.12	21.90	56.23	2.42	0.54	1.35	28.52	55.66	84.18	0.120	No	No	1.59
64	14.19	19.99	60.43	2.47	0.54	1.35	24.72	55.76	80.48	0.116	No	No	1.52
65	14.26	18.89	62.48	2.49	0.55	1.35	23.24	55.83	79.07	0.115	No	No	1.50
66	14.32	19.18	60.30	2.47	0.55	1.35	24.38	55.63	80.01	0.116	No	No	1.51
67	14.39	20.28	54.98	2.40	0.55	1.35	25.69	54.51	80.20	0.116	No	No	1.51
68	14.45	21.00	50.13	2.34	0.54	1.34	27.29	53.29	80.58	0.116	No	No	1.51
69	14.52	21.38	46.81	2.30	0.55	1.35	27.10	51.90	78.99	0.115	No	No	1.48
70	14.58	21.26	45.24	2.28	0.55	1.35	27.09	51.19	78.28	0.114	No	No	1.47
71	14.65	20.89	44.83	2.27	0.55	1.34	26.85	50.94	77.79	0.114	No	No	1.46
72	14.71	20.16	45.08	2.28	0.56	1.35	25.68	50.76	76.44	0.113	No	No	1.44
73	14.78	19.65	45.07	2.28	0.56	1.35	24.36	50.42	74.78	0.111	No	No	1.41
74	14.85	19.46	43.58	2.26	0.56	1.35	24.93	49.86	74.79	0.111	No	No	1.41
75	14.91	19.62	40.41	2.22	0.56	1.35	24.97	48.19	73.15	0.110	No	No	1.38
76	14.98	20.25	36.82	2.17	0.57	1.35	25.02	45.96	70.98	0.108	No	No	1.35
77	15.04	21.12	34.15	2.14	0.57	1.35	27.37	44.51	71.88	0.109	No	No	1.36
78	15.11	21.99	32.21	2.12	0.57	1.35	28.27	43.08	71.34	0.108	No	No	1.35
79	15.17	21.87	32.56	2.12	0.57	1.35	28.23	43.38	71.61	0.109	No	No	1.35
80	15.24	21.17	35.13	2.15	0.57	1.34	26.84	45.16	71.99	0.109	No	No	1.35
81	15.30	20.46	39.02	2.20	0.57	1.34	25.43	47.47	72.90	0.110	No	No	1.36
82	15.37	20.11	42.40	2.24	0.56	1.34	25.34	49.36	74.70	0.111	No	No	1.38
83	15.44	20.07	44.94	2.27	0.56	1.33	25.27	50.59	75.86	0.112	No	No	1.40
84	15.50	19.49	48.34	2.32	0.55	1.33	25.05	52.00	77.05	0.113	No	No	1.41
85	15.57	19.02	50.64	2.35	0.56	1.33	23.01	52.33	75.34	0.112	No	No	1.39
86	15.63	18.43	52.71	2.37	0.56	1.33	23.39	53.15	76.54	0.113	No	No	1.40
87	15.70	18.07	53.78	2.38	0.56	1.32	22.78	53.34	76.12	0.112	No	No	1.39
88	15.76	17.33	55.57	2.41	0.56	1.32	21.60	53.56	75.17	0.112	No	No	1.38
89	15.83	16.62	56.60	2.42	0.56	1.33	20.58	53.58	74.16	0.111	No	No	1.36
90	15.90	16.31	55.38	2.40	0.56	1.33	20.14	53.10	73.24	0.110	No	No	1.34
91	15.96	16.35	52.47	2.37	0.57	1.32	20.42	52.28	72.70	0.110	No	No	1.34
92	16.03	16.58	49.37	2.33	0.57	1.32	20.72	51.26	71.98	0.109	No	No	1.32
93	16.09	16.98	47.07	2.30	0.57	1.32	21.01	50.43	71.44	0.109	No	No	1.31
94	16.16	17.29	47.02	2.30	0.57	1.32	21.84	50.63	72.47	0.109	No	No	1.33
95	16.22	17.45	47.78	2.31	0.57	1.32	21.81	50.93	72.74	0.110	No	No	1.33
96	16.29	17.49	48.56	2.32	0.57	1.32	21.50	51.16	72.66	0.110	No	No	1.32

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
97	16.36	17.41	49.20	2.33	0.56	1.32	21.89	51.51	73.40	0.110	No	No	1.33
98	16.42	17.57	49.11	2.33	0.57	1.31	21.45	51.36	72.80	0.110	No	No	1.32
99	16.49	17.37	50.20	2.34	0.56	1.31	21.98	51.90	73.88	0.111	No	No	1.33
100	16.55	17.22	50.58	2.34	0.57	1.31	21.11	51.81	72.91	0.110	No	No	1.32
101	16.62	16.71	51.48	2.36	0.57	1.31	20.79	52.04	72.84	0.110	No	No	1.32
102	16.68	16.39	51.18	2.35	0.57	1.31	20.07	51.74	71.82	0.109	No	No	1.30
103	16.75	16.12	50.87	2.35	0.57	1.31	19.91	51.59	71.51	0.109	No	No	1.29
104	16.81	16.04	50.35	2.34	0.57	1.31	19.76	51.37	71.12	0.108	No	No	1.29
105	16.88	16.00	49.54	2.33	0.57	1.31	19.75	51.07	70.81	0.108	No	No	1.28
106	16.95	16.00	48.81	2.32	0.57	1.31	19.73	50.79	70.53	0.108	No	No	1.28
107	17.01	16.40	47.82	2.31	0.57	1.31	19.73	50.40	70.13	0.107	No	No	1.27
108	17.08	17.07	47.22	2.30	0.57	1.30	21.12	50.52	71.64	0.109	No	No	1.29
109	17.14	18.80	43.51	2.26	0.57	1.30	22.13	49.12	71.25	0.108	No	No	1.28
110	17.21	19.94	39.82	2.21	0.56	1.30	25.93	48.08	74.00	0.111	No	No	1.31
111	17.27	21.47	35.17	2.15	0.57	1.30	25.31	44.84	70.15	0.108	No	No	1.26
112	17.34	21.59	38.97	2.20	0.56	1.29	27.56	47.95	75.51	0.112	No	No	1.33
113	17.41	21.98	45.22	2.28	0.55	1.29	26.18	50.95	77.13	0.113	No	No	1.35
114	17.47	21.91	49.98	2.34	0.55	1.28	26.37	52.99	79.35	0.115	No	No	1.38
115	17.54	22.73	49.55	2.33	0.55	1.28	27.03	53.00	80.02	0.116	No	No	1.39
116	17.60	23.24	47.08	2.30	0.54	1.28	28.95	52.49	81.44	0.117	No	No	1.41
117	17.67	22.02	48.90	2.32	0.54	1.28	28.07	53.01	81.09	0.117	No	No	1.40
118	17.73	21.77	48.89	2.32	0.56	1.28	22.68	51.60	74.28	0.111	No	No	1.30
119	17.80	21.42	48.70	2.32	0.54	1.27	27.95	52.91	80.86	0.117	No	No	1.39
120	17.86	22.92	44.90	2.27	0.55	1.28	26.81	50.97	77.78	0.114	No	No	1.34
121	17.93	22.39	45.15	2.28	0.55	1.27	27.87	51.35	79.22	0.115	No	No	1.36
122	18.00	21.29	45.89	2.29	0.55	1.27	26.06	51.23	77.29	0.114	No	No	1.33
123	18.06	19.95	44.84	2.27	0.57	1.28	22.89	49.94	72.83	0.110	No	No	1.27
124	18.13	19.24	41.39	2.23	0.57	1.28	23.19	48.30	71.48	0.109	No	No	1.26
125	18.19	19.05	39.93	2.21	0.57	1.28	23.61	47.58	71.18	0.108	No	No	1.25
126	18.26	18.58	43.81	2.26	0.57	1.28	22.15	49.27	71.42	0.109	No	No	1.25
127	18.32	18.14	47.31	2.30	0.57	1.28	21.40	50.63	72.04	0.109	No	No	1.26
128	18.39	17.56	49.70	2.33	0.56	1.27	21.91	51.70	73.61	0.110	No	No	1.28
129	18.45	17.08	50.30	2.34	0.57	1.27	19.96	51.40	71.36	0.108	No	No	1.25
130	18.52	16.40	51.56	2.36	0.57	1.27	19.65	51.76	71.41	0.109	No	No	1.24
131	18.59	16.20	52.43	2.37	0.57	1.27	19.42	51.99	71.42	0.109	No	No	1.24
132	18.65	17.94	48.24	2.32	0.57	1.27	19.23	50.44	69.67	0.107	No	No	1.22
133	18.72	18.94	46.38	2.29	0.55	1.26	25.66	51.34	77.00	0.113	No	No	1.31
134	18.78	22.44	39.25	2.20	0.57	1.27	23.01	47.03	70.04	0.107	No	No	1.22
135	18.85	23.86	39.22	2.20	0.55	1.25	31.38	49.02	80.41	0.116	No	No	1.36
136	18.91	27.59	37.04	2.18	0.55	1.26	30.59	47.39	77.99	0.114	No	No	1.32
137	18.98	28.42	38.28	2.19	0.53	1.25	35.74	49.45	85.19	0.121	No	No	1.42
138	19.05	29.28	36.08	2.16	0.54	1.25	34.16	47.53	81.69	0.117	No	No	1.37
139	19.11	27.39	36.68	2.17	0.54	1.25	33.45	47.80	81.25	0.117	No	No	1.36
140	19.18	25.86	38.57	2.19	0.55	1.25	29.22	48.10	77.32	0.114	No	No	1.31
141	19.24	24.56	42.12	2.24	0.55	1.25	28.73	50.05	78.78	0.115	No	No	1.32
142	19.31	24.65	42.49	2.24	0.55	1.25	28.83	50.28	79.11	0.115	No	No	1.33
143	19.37	24.29	42.91	2.25	0.55	1.24	29.33	50.62	79.95	0.116	No	No	1.34
144	19.44	22.29	48.09	2.31	0.55	1.24	27.36	52.51	79.87	0.116	No	No	1.33

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
145	19.50	19.62	56.69	2.42	0.56	1.25	21.76	53.93	75.69	0.112	No	No	1.28
146	19.57	17.03	65.67	2.53	0.56	1.25	19.93	55.59	75.52	0.112	No	No	1.27
147	19.64	15.65	70.19	2.59	0.56	1.25	18.27	55.97	74.24	0.111	No	No	1.26
148	19.70	14.55	73.03	2.63	0.57	1.25	16.89	0.00	16.89	4.000	No	Yes	2.00
149	19.77	13.06	77.86	2.69	0.57	1.25	16.04	0.00	16.04	4.000	No	Yes	2.00
150	19.83	11.69	83.37	2.75	0.58	1.25	13.02	0.00	13.02	4.000	No	Yes	2.00
151	19.90	10.68	87.79	2.81	0.58	1.25	12.05	0.00	12.05	4.000	No	Yes	2.00
152	19.96	10.56	87.72	2.81	0.58	1.25	12.46	0.00	12.46	4.000	No	Yes	2.00
153	20.03	10.04	89.13	2.83	0.58	1.24	12.57	0.00	12.57	4.000	No	Yes	2.00
154	20.09	8.69	94.96	2.90	0.58	1.25	10.21	0.00	10.21	4.000	No	Yes	2.00
155	20.16	7.10	100.00	3.01	0.59	1.25	7.72	0.00	7.72	4.000	No	Yes	2.00
156	20.23	6.11	100.00	3.09	0.59	1.25	7.02	0.00	7.02	4.000	No	Yes	2.00
157	20.29	5.76	100.00	3.11	0.60	1.25	6.74	0.00	6.74	4.000	No	Yes	2.00
158	20.36	6.28	100.00	3.05	0.60	1.25	6.46	0.00	6.46	4.000	No	Yes	2.00
159	20.42	7.83	97.59	2.93	0.59	1.24	8.80	0.00	8.80	4.000	No	Yes	2.00
160	20.49	9.10	93.21	2.88	0.58	1.24	12.07	0.00	12.07	4.000	No	Yes	2.00
161	20.55	9.31	93.98	2.89	0.58	1.24	10.84	0.00	10.84	4.000	No	Yes	2.00
162	20.62	8.21	100.00	2.98	0.58	1.24	9.47	0.00	9.47	4.000	No	Yes	2.00
163	20.69	7.34	100.00	3.04	0.59	1.24	8.23	0.00	8.23	4.000	No	Yes	2.00
164	20.75	6.72	100.00	3.07	0.59	1.24	7.81	0.00	7.81	4.000	No	Yes	2.00
165	20.82	6.29	100.00	3.07	0.59	1.24	7.27	0.00	7.27	4.000	No	Yes	2.00
166	20.88	5.83	100.00	3.05	0.60	1.24	6.72	0.00	6.72	4.000	No	Yes	2.00
167	20.95	5.33	100.00	3.05	0.60	1.24	6.17	0.00	6.17	4.000	No	Yes	2.00
168	21.01	4.88	100.00	3.07	0.60	1.24	5.50	0.00	5.50	4.000	No	Yes	2.00
169	21.08	4.42	100.00	3.11	0.60	1.24	5.09	0.00	5.09	4.000	No	Yes	2.00
170	21.14	4.13	100.00	3.12	0.60	1.24	4.56	0.00	4.56	4.000	No	Yes	2.00
171	21.21	3.93	100.00	3.13	0.60	1.24	4.43	0.00	4.43	4.000	No	Yes	2.00
172	21.28	3.85	100.00	3.15	0.61	1.24	4.32	0.00	4.32	4.000	No	Yes	2.00
173	21.34	3.76	100.00	3.17	0.61	1.24	4.20	0.00	4.20	4.000	No	Yes	2.00
174	21.41	3.71	100.00	3.19	0.61	1.24	4.07	0.00	4.07	4.000	No	Yes	2.00
175	21.47	3.81	100.00	3.17	0.61	1.23	4.08	0.00	4.08	4.000	No	Yes	2.00
176	21.54	3.99	100.00	3.13	0.60	1.23	4.50	0.00	4.50	4.000	No	Yes	2.00
177	21.60	4.04	100.00	3.12	0.60	1.23	4.64	0.00	4.64	4.000	No	Yes	2.00
178	21.67	3.94	100.00	3.17	0.61	1.23	4.24	0.00	4.24	4.000	No	Yes	2.00
179	21.74	3.80	100.00	3.22	0.61	1.23	4.11	0.00	4.11	4.000	No	Yes	2.00
180	21.80	3.39	100.00	3.34	0.61	1.23	4.11	0.00	4.11	4.000	No	Yes	2.00
181	21.87	3.62	100.00	3.30	0.61	1.23	2.70	0.00	2.70	4.000	No	Yes	2.00
182	21.93	3.89	100.00	3.25	0.60	1.23	4.84	0.00	4.84	4.000	No	Yes	2.00
183	22.00	4.57	100.00	3.12	0.60	1.23	4.98	0.00	4.98	4.000	No	Yes	2.00
184	22.06	4.68	100.00	3.08	0.60	1.22	5.09	0.00	5.09	4.000	No	Yes	2.00
185	22.13	4.59	100.00	3.07	0.60	1.22	5.22	0.00	5.22	4.000	No	Yes	2.00
186	22.20	4.35	100.00	3.10	0.60	1.22	4.67	0.00	4.67	4.000	No	Yes	2.00
187	22.26	3.95	100.00	3.18	0.61	1.22	4.26	0.00	4.26	4.000	No	Yes	2.00
188	22.33	3.73	100.00	3.22	0.61	1.22	3.85	0.00	3.85	4.000	No	Yes	2.00
189	22.39	3.63	100.00	3.23	0.61	1.22	3.87	0.00	3.87	4.000	No	Yes	2.00
190	22.46	3.70	100.00	3.20	0.61	1.22	3.89	0.00	3.89	4.000	No	Yes	2.00
191	22.52	3.70	100.00	3.20	0.61	1.22	4.04	0.00	4.04	4.000	No	Yes	2.00
192	22.59	3.69	100.00	3.20	0.61	1.22	3.78	0.00	3.78	4.000	No	Yes	2.00

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
193	22.65	3.62	100.00	3.22	0.61	1.22	3.79	0.00	3.79	4.000	No	Yes	2.00
194	22.72	3.63	100.00	3.22	0.61	1.22	3.78	0.00	3.78	4.000	No	Yes	2.00
195	22.79	3.60	100.00	3.22	0.61	1.22	3.80	0.00	3.80	4.000	No	Yes	2.00
196	22.85	3.58	100.00	3.23	0.61	1.22	3.66	0.00	3.66	4.000	No	Yes	2.00
197	22.92	3.55	100.00	3.24	0.61	1.22	3.67	0.00	3.67	4.000	No	Yes	2.00
198	22.98	3.64	100.00	3.21	0.61	1.22	3.67	0.00	3.67	4.000	No	Yes	2.00
199	23.05	3.76	100.00	3.17	0.61	1.21	3.95	0.00	3.95	4.000	No	Yes	2.00
200	23.11	3.79	100.00	3.16	0.61	1.21	4.08	0.00	4.08	4.000	No	Yes	2.00
201	23.18	3.71	100.00	3.18	0.61	1.21	3.78	0.00	3.78	4.000	No	Yes	2.00
202	23.25	3.59	100.00	3.22	0.61	1.21	3.67	0.00	3.67	4.000	No	Yes	2.00
203	23.31	3.56	100.00	3.23	0.61	1.21	3.65	0.00	3.65	4.000	No	Yes	2.00
204	23.38	3.60	100.00	3.21	0.61	1.21	3.66	0.00	3.66	4.000	No	Yes	2.00
205	23.44	3.66	100.00	3.20	0.61	1.21	3.80	0.00	3.80	4.000	No	Yes	2.00
206	23.51	3.66	100.00	3.20	0.61	1.21	3.80	0.00	3.80	4.000	No	Yes	2.00
207	23.57	3.63	100.00	3.22	0.61	1.21	3.67	0.00	3.67	4.000	No	Yes	2.00
208	23.64	3.55	100.00	3.25	0.61	1.21	3.66	0.00	3.66	4.000	No	Yes	2.00
209	23.70	3.52	100.00	3.27	0.61	1.21	3.53	0.00	3.53	4.000	No	Yes	2.00
210	23.77	3.49	100.00	3.28	0.61	1.21	3.54	0.00	3.54	4.000	No	Yes	2.00
211	23.84	3.47	100.00	3.28	0.61	1.21	3.54	0.00	3.54	4.000	No	Yes	2.00
212	23.90	3.48	100.00	3.27	0.61	1.21	3.41	0.00	3.41	4.000	No	Yes	2.00
213	23.97	3.53	100.00	3.25	0.61	1.20	3.55	0.00	3.55	4.000	No	Yes	2.00
214	24.03	3.58	100.00	3.26	0.61	1.20	3.69	0.00	3.69	4.000	No	Yes	2.00
215	24.10	3.71	100.00	3.27	0.61	1.20	3.56	0.00	3.56	4.000	No	Yes	2.00
216	24.16	3.98	100.00	3.24	0.61	1.20	3.96	0.00	3.96	4.000	No	Yes	2.00
217	24.23	4.20	100.00	3.21	0.60	1.20	4.62	0.00	4.62	4.000	No	Yes	2.00
218	24.30	4.15	100.00	3.21	0.61	1.20	4.31	0.00	4.31	4.000	No	Yes	2.00
219	24.36	3.85	100.00	3.28	0.61	1.20	3.86	0.00	3.86	4.000	No	Yes	2.00
220	24.43	3.60	100.00	3.33	0.61	1.20	3.64	0.00	3.64	4.000	No	Yes	2.00
221	24.49	3.52	100.00	3.32	0.61	1.20	3.49	0.00	3.49	4.000	No	Yes	2.00
222	24.56	3.48	100.00	3.29	0.61	1.20	3.51	0.00	3.51	4.000	No	Yes	2.00
223	24.62	3.51	100.00	3.25	0.61	1.20	3.52	0.00	3.52	4.000	No	Yes	2.00
224	24.69	3.53	100.00	3.25	0.61	1.20	3.52	0.00	3.52	4.000	No	Yes	2.00
225	24.75	3.47	100.00	3.26	0.61	1.19	3.53	0.00	3.53	4.000	No	Yes	2.00
226	24.82	3.41	100.00	3.31	0.61	1.19	3.28	0.00	3.28	4.000	No	Yes	2.00
227	24.89	3.63	100.00	3.36	0.61	1.19	3.29	0.00	3.29	4.000	No	Yes	2.00
228	24.95	4.74	100.00	3.26	0.61	1.19	4.22	0.00	4.22	4.000	No	Yes	2.00
229	25.02	5.71	100.00	3.19	0.59	1.19	6.98	0.00	6.98	4.000	No	Yes	2.00
230	25.08	6.82	100.00	3.07	0.59	1.18	6.87	0.00	6.87	4.000	No	Yes	2.00
231	25.15	6.75	100.00	3.07	0.59	1.18	8.29	0.00	8.29	4.000	No	Yes	2.00
232	25.21	6.29	100.00	3.11	0.59	1.18	7.11	0.00	7.11	4.000	No	Yes	2.00
233	25.28	5.14	100.00	3.25	0.60	1.18	5.29	0.00	5.29	4.000	No	Yes	2.00
234	25.34	4.26	100.00	3.34	0.61	1.18	4.41	0.00	4.41	4.000	No	Yes	2.00
235	25.41	3.81	100.00	3.32	0.61	1.18	4.05	0.00	4.05	4.000	No	Yes	2.00
236	25.48	3.49	100.00	3.25	0.61	1.18	3.62	0.00	3.62	4.000	No	Yes	2.00
237	25.54	3.24	100.00	3.19	0.61	1.18	3.06	0.00	3.06	4.000	No	Yes	2.00
238	25.61	3.10	100.00	3.19	0.61	1.18	2.97	0.00	2.97	4.000	No	Yes	2.00
239	25.67	3.11	100.00	3.22	0.61	1.18	2.99	0.00	2.99	4.000	No	Yes	2.00
240	25.74	3.21	100.00	3.25	0.61	1.18	3.00	0.00	3.00	4.000	No	Yes	2.00

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
241	25.80	3.43	100.00	3.25	0.61	1.18	3.27	0.00	3.27	4.000	No	Yes	2.00
242	25.87	3.83	100.00	3.19	0.61	1.18	3.67	0.00	3.67	4.000	No	Yes	2.00
243	25.93	4.13	100.00	3.16	0.61	1.18	4.32	0.00	4.32	4.000	No	Yes	2.00
244	26.00	4.09	100.00	3.19	0.61	1.18	4.29	0.00	4.29	4.000	No	Yes	2.00
245	26.07	3.80	100.00	3.26	0.61	1.18	3.59	0.00	3.59	4.000	No	Yes	2.00
246	26.13	3.63	100.00	3.29	0.61	1.18	3.37	0.00	3.37	4.000	No	Yes	2.00
247	26.20	3.77	100.00	3.23	0.61	1.18	3.74	0.00	3.74	4.000	No	Yes	2.00
248	26.26	3.96	100.00	3.15	0.61	1.17	4.02	0.00	4.02	4.000	No	Yes	2.00
249	26.33	4.11	100.00	3.11	0.61	1.17	4.01	0.00	4.01	4.000	No	Yes	2.00
250	26.39	4.33	100.00	3.08	0.61	1.17	4.17	0.00	4.17	4.000	No	Yes	2.00
251	26.46	4.52	100.00	3.07	0.60	1.17	4.70	0.00	4.70	4.000	No	Yes	2.00
252	26.53	4.48	100.00	3.09	0.60	1.17	4.57	0.00	4.57	4.000	No	Yes	2.00
253	26.59	4.21	100.00	3.16	0.61	1.17	4.04	0.00	4.04	4.000	No	Yes	2.00
254	26.66	3.99	100.00	3.22	0.61	1.17	3.80	0.00	3.80	4.000	No	Yes	2.00
255	26.72	3.94	100.00	3.22	0.61	1.17	3.81	0.00	3.81	4.000	No	Yes	2.00
256	26.79	3.97	100.00	3.17	0.61	1.17	3.82	0.00	3.82	4.000	No	Yes	2.00
257	26.85	3.99	100.00	3.13	0.61	1.17	3.83	0.00	3.83	4.000	No	Yes	2.00
258	26.92	4.01	100.00	3.12	0.61	1.17	3.84	0.00	3.84	4.000	No	Yes	2.00
259	26.98	3.99	100.00	3.14	0.61	1.17	3.85	0.00	3.85	4.000	No	Yes	2.00
260	27.05	4.03	100.00	3.16	0.61	1.17	3.72	0.00	3.72	4.000	No	Yes	2.00
261	27.12	4.22	100.00	3.16	0.61	1.17	3.97	0.00	3.97	4.000	No	Yes	2.00
262	27.18	4.52	100.00	3.14	0.60	1.16	4.48	0.00	4.48	4.000	No	Yes	2.00
263	27.25	4.80	100.00	3.12	0.60	1.16	4.72	0.00	4.72	4.000	No	Yes	2.00
264	27.31	5.11	100.00	3.10	0.60	1.16	4.93	0.00	4.93	4.000	No	Yes	2.00
265	27.38	6.30	100.00	2.98	0.60	1.16	5.56	0.00	5.56	4.000	No	Yes	2.00
266	27.44	10.38	80.77	2.72	0.59	1.16	8.65	0.00	8.65	4.000	No	Yes	2.00
267	27.49	22.23	51.16	2.35	0.58	1.15	18.21	51.24	69.45	0.107	No	No	1.09
268	27.56	37.49	31.85	2.11	0.52	1.14	43.68	46.02	89.69	0.125	No	No	1.35
269	27.63	50.96	20.13	1.96	0.52	1.13	57.86	31.58	89.44	0.125	No	No	1.34
270	27.69	56.42	16.27	1.92	0.53	1.14	61.71	23.12	84.82	0.120	No	No	1.27
271	27.76	59.12	15.22	1.90	0.54	1.14	61.60	20.40	81.99	0.118	No	No	1.24
272	27.82	58.78	17.32	1.93	0.52	1.13	66.30	26.24	92.55	0.128	No	No	1.39
273	27.89	57.99	19.59	1.96	0.52	1.13	60.26	30.77	91.03	0.127	No	No	1.37
274	27.95	55.16	21.98	1.99	0.51	1.13	58.47	35.29	93.76	0.130	No	No	1.41
275	28.02	53.71	21.26	1.98	0.52	1.13	57.14	33.72	90.85	0.127	No	No	1.36
276	28.08	52.18	19.49	1.96	0.53	1.13	55.61	29.93	85.54	0.121	No	No	1.28
277	28.15	50.49	18.41	1.94	0.54	1.13	53.80	27.37	81.18	0.117	No	No	1.22
278	28.21	48.60	18.12	1.94	0.55	1.13	51.95	26.50	78.45	0.115	No	No	1.19
279	28.28	45.96	17.78	1.93	0.56	1.13	49.72	25.49	75.21	0.112	No	No	1.15
280	28.35	42.41	18.39	1.94	0.57	1.13	45.48	26.33	71.81	0.109	No	No	1.11
281	28.41	39.23	20.70	1.97	0.57	1.13	40.67	30.29	70.96	0.108	No	No	1.10
282	28.48	36.67	26.93	2.05	0.55	1.13	39.31	39.67	78.98	0.115	No	No	1.19
283	28.54	34.67	35.83	2.16	0.53	1.12	36.89	47.96	84.85	0.120	No	No	1.27
284	28.61	32.66	43.21	2.25	0.53	1.12	33.84	51.90	85.74	0.121	No	No	1.28
285	28.68	31.84	47.33	2.30	0.53	1.12	32.68	53.57	86.25	0.122	No	No	1.29
286	28.74	33.77	47.34	2.30	0.53	1.12	34.11	53.94	88.06	0.124	No	No	1.32
287	28.81	40.69	40.10	2.21	0.52	1.11	39.73	51.58	91.31	0.127	No	No	1.37
288	28.87	48.66	36.89	2.17	0.48	1.10	53.94	52.69	106.63	0.147	No	No	1.67

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
289	28.94	57.06	38.24	2.19	0.47	1.10	58.52	54.79	113.31	0.158	No	No	1.85
290	29.00	66.87	40.63	2.22	0.45	1.10	65.16	58.08	123.25	0.179	No	No	2.00
291	29.07	81.30	39.52	2.21	0.41	1.09	83.21	61.64	144.85	0.258	No	No	2.00
292	29.13	98.51	35.72	2.16	0.38	1.08	101.46	62.56	164.02	0.417	No	No	2.00
293	29.20	130.66	25.05	2.03	0.38	1.08	116.53	50.62	167.15	0.459	No	No	2.00
294	29.27	176.40	11.02	1.85	0.34	1.07	179.12	13.67	192.79	1.290	No	No	2.00
295	29.33	226.38	0.00	1.68	0.28	1.06	236.16	0.00	236.16	4.000	No	No	2.00
296	29.40	265.36	0.00	1.54	0.26	1.05	262.63	0.00	262.63	4.000	No	No	2.00
297	29.46	287.77	0.00	1.45	0.26	1.05	292.18	0.00	292.18	4.000	No	No	2.00
298	29.53	310.11	0.00	1.37	0.26	1.05	301.47	0.00	301.47	4.000	No	No	2.00
299	29.59	299.11	0.00	1.38	0.26	1.05	328.52	0.00	328.52	4.000	No	No	2.00
300	29.66	261.98	0.00	1.45	0.26	1.05	258.94	0.00	258.94	4.000	No	No	2.00
301	29.72	189.90	0.00	1.62	0.34	1.06	193.14	0.00	193.14	1.312	No	No	2.00
302	29.79	116.95	14.42	1.89	0.42	1.08	117.12	22.56	139.68	0.233	No	No	2.00
303	29.86	69.71	38.00	2.19	0.51	1.09	43.91	51.18	95.08	0.131	No	No	1.43
304	29.92	50.29	51.77	2.36	0.47	1.08	52.03	60.51	112.54	0.157	No	No	1.83
305	29.99	55.07	43.28	2.25	0.46	1.08	58.17	58.03	116.20	0.163	No	No	1.94
306	30.05	57.57	35.49	2.16	0.48	1.08	58.14	52.51	110.65	0.153	No	No	1.77
307	30.12	59.37	28.69	2.07	0.49	1.08	59.65	45.78	105.43	0.145	No	No	1.64
308	30.18	61.29	24.41	2.02	0.49	1.08	63.71	40.38	104.08	0.143	No	No	1.61
309	30.25	59.05	25.77	2.03	0.48	1.08	63.96	42.56	106.52	0.147	No	No	1.67
310	30.32	55.98	28.38	2.07	0.51	1.08	52.77	44.05	96.82	0.133	No	No	1.46
311	30.38	53.82	29.54	2.08	0.50	1.08	54.23	45.74	99.97	0.137	No	No	1.52
312	30.45	55.62	26.50	2.04	0.50	1.08	57.31	42.43	99.75	0.137	No	No	1.52
313	30.51	57.11	23.69	2.01	0.51	1.08	58.16	38.25	96.41	0.133	No	No	1.45
314	30.58	56.64	23.30	2.00	0.51	1.08	58.71	37.68	96.38	0.133	No	No	1.45
315	30.64	55.62	23.53	2.01	0.51	1.08	55.83	37.60	93.43	0.129	No	No	1.40
316	30.71	54.45	23.14	2.00	0.52	1.08	54.98	36.81	91.78	0.128	No	No	1.37
317	30.77	54.11	21.24	1.98	0.52	1.08	55.11	33.39	88.51	0.124	No	No	1.32
318	30.84	54.08	19.53	1.96	0.53	1.08	54.78	29.91	84.69	0.120	No	No	1.27
319	30.91	54.52	18.87	1.95	0.54	1.08	54.88	28.50	83.39	0.119	No	No	1.25
320	30.97	55.30	18.80	1.95	0.53	1.08	56.36	28.54	84.90	0.121	No	No	1.27
321	31.04	56.55	18.32	1.94	0.53	1.08	57.03	27.56	84.58	0.120	No	No	1.27
322	31.10	57.84	17.38	1.93	0.54	1.08	58.54	25.54	84.08	0.120	No	No	1.26
323	31.17	59.19	16.56	1.92	0.54	1.07	60.17	23.70	83.87	0.120	No	No	1.26
324	31.23	59.97	16.48	1.92	0.53	1.07	60.95	23.58	84.53	0.120	No	No	1.26
325	31.30	61.30	15.98	1.91	0.54	1.07	60.78	22.31	83.09	0.119	No	No	1.24
326	31.36	63.30	18.28	1.94	0.52	1.07	63.89	28.28	92.17	0.128	No	No	1.38
327	31.43	65.21	22.98	2.00	0.49	1.06	66.44	38.36	104.81	0.144	No	No	1.62
328	31.50	66.97	25.85	2.04	0.48	1.06	65.94	43.04	108.98	0.150	No	No	1.73
329	31.56	66.86	26.51	2.04	0.47	1.06	68.64	44.52	113.17	0.158	No	No	1.84
330	31.63	66.94	25.18	2.03	0.48	1.06	65.93	42.00	107.93	0.149	No	No	1.70
331	31.69	62.01	28.56	2.07	0.47	1.06	65.92	46.85	112.77	0.157	No	No	1.83
332	31.76	61.31	31.77	2.11	0.49	1.06	53.87	48.09	101.96	0.140	No	No	1.56
333	31.82	65.90	32.34	2.12	0.47	1.06	63.53	50.73	114.27	0.160	No	No	1.87
334	31.89	78.51	26.78	2.05	0.45	1.05	79.25	46.89	126.14	0.186	No	No	2.00
335	31.95	84.99	23.88	2.01	0.43	1.05	90.83	43.99	134.83	0.214	No	No	2.00
336	32.02	88.45	20.80	1.97	0.46	1.05	82.62	36.46	119.08	0.169	No	No	2.00

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
337	32.09	85.66	20.06	1.96	0.45	1.05	89.31	35.70	125.01	0.183	No	No	2.00
338	32.15	87.45	15.27	1.90	0.49	1.05	82.79	22.39	105.18	0.145	No	No	1.63
339	32.22	85.17	12.04	1.86	0.50	1.05	88.02	13.13	101.15	0.139	No	No	1.54
340	32.28	74.55	15.25	1.90	0.49	1.05	82.69	22.32	105.01	0.144	No	No	1.63
341	32.35	63.53	22.11	1.99	0.53	1.05	51.17	34.40	85.57	0.121	No	No	1.28
342	32.41	53.60	29.18	2.08	0.50	1.05	55.04	45.47	100.50	0.138	No	No	1.53
343	32.48	54.56	28.86	2.07	0.50	1.05	53.04	44.69	97.73	0.135	No	No	1.48
344	32.55	55.16	31.12	2.10	0.50	1.05	53.66	47.37	101.04	0.139	No	No	1.54
345	32.61	57.18	32.62	2.12	0.48	1.05	56.60	49.54	106.13	0.146	No	No	1.66
346	32.68	60.33	31.91	2.11	0.48	1.04	58.74	49.28	108.02	0.149	No	No	1.70
347	32.74	65.25	28.54	2.07	0.48	1.04	62.75	46.20	108.95	0.150	No	No	1.73
348	32.81	73.91	24.57	2.02	0.47	1.04	70.90	41.88	112.77	0.157	No	No	1.83
349	32.87	83.83	23.45	2.01	0.45	1.04	83.95	42.06	126.01	0.186	No	No	2.00
350	32.94	87.12	30.56	2.09	0.41	1.03	91.34	54.52	145.86	0.264	No	No	2.00
351	33.01	91.40	37.56	2.18	0.42	1.03	80.07	59.32	139.39	0.232	No	No	2.00
352	33.07	95.93	41.47	2.23	0.38	1.03	95.83	66.17	162.00	0.393	No	No	2.00
353	33.14	113.33	34.21	2.14	0.38	1.03	104.08	61.56	165.64	0.438	No	No	2.00
354	33.20	132.44	23.21	2.00	0.36	1.03	130.17	49.07	179.23	0.705	No	No	2.00
355	33.27	156.88	11.81	1.86	0.38	1.03	151.19	15.45	166.64	0.452	No	No	2.00
356	33.33	187.77	2.02	1.74	0.36	1.03	174.81	0.00	174.81	0.596	No	No	2.00
357	33.40	213.78	0.00	1.65	0.31	1.02	218.81	0.00	218.81	4.000	No	No	2.00
358	33.47	216.13	0.00	1.62	0.30	1.02	225.26	0.00	225.26	4.000	No	No	2.00
359	33.53	194.12	0.00	1.64	0.36	1.02	181.06	0.00	181.06	0.759	No	No	2.00
360	33.60	171.17	2.05	1.74	0.39	1.02	155.65	0.00	155.65	0.331	No	No	2.00
361	33.66	174.98	0.00	1.62	0.39	1.02	159.46	0.00	159.46	0.366	No	No	2.00
362	33.73	195.40	0.00	1.43	0.34	1.02	191.53	0.00	191.53	1.212	No	No	2.00
363	33.79	222.87	100.00	4.06	0.26	1.01	213.03	0.00	213.03	4.000	No	Yes	2.00
364	33.86	251.55	100.00	4.06	0.26	1.01	237.19	0.00	237.19	4.000	No	Yes	2.00
365	33.92	277.60	100.00	4.06	0.26	1.01	273.08	0.00	273.08	4.000	No	Yes	2.00
366	33.99	295.24	100.00	4.06	0.26	1.01	287.83	0.00	287.83	4.000	No	Yes	2.00

Abbreviations

Depth: Depth from free surface, at which CPT was performed (ft)
q_t: Total cone resistance
FC: Fines content (%)
I_c: Soil behavior type index
m: Stress exponent
C_N: Overburden correction factor
q_{c1N}: Normalized and adjusted cone resistance
Δq_{c1N}: Cone resistance correction factor due to fines
$q_{c1N,cs}$: Normalized and adjusted cone resistance
$CRR_{7.5}$: Cyclic resistance ratio for M_w=7.5
FS: Factor of safety against soil liquefaction

GZA GeoEnvironmental, Inc.

LIQUEFACTION ANALYSIS REPORT

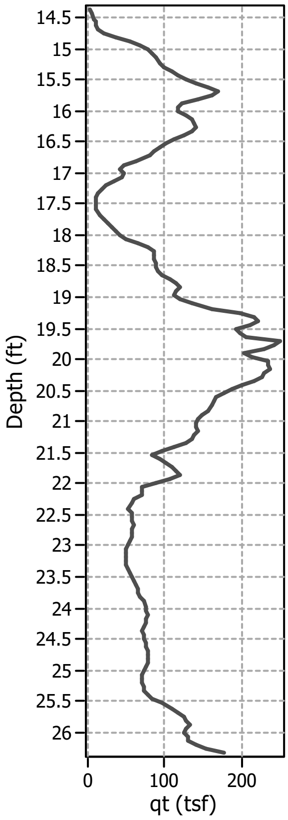
Project title : Bridges Bridge No. **Location : Wilton, Maine**

CPT file : sCPT-WWS-202

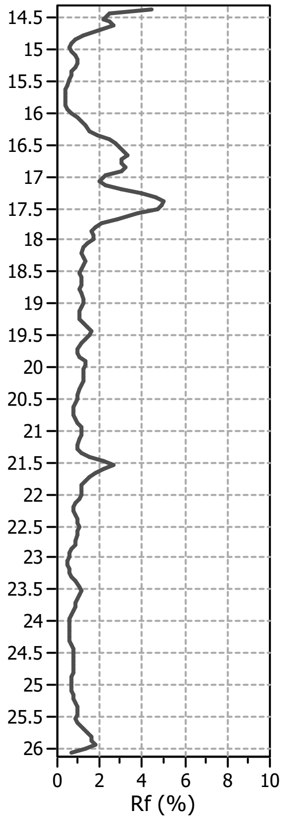
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	10.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.13	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

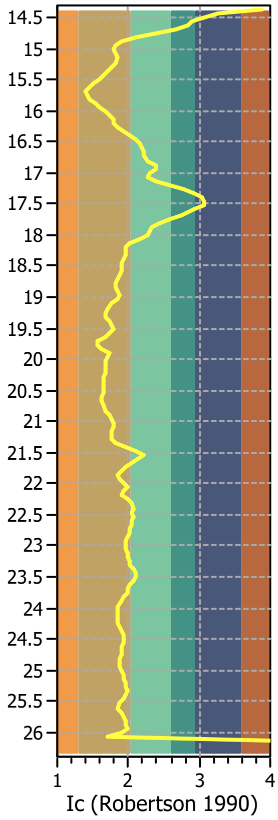
Cone resistance



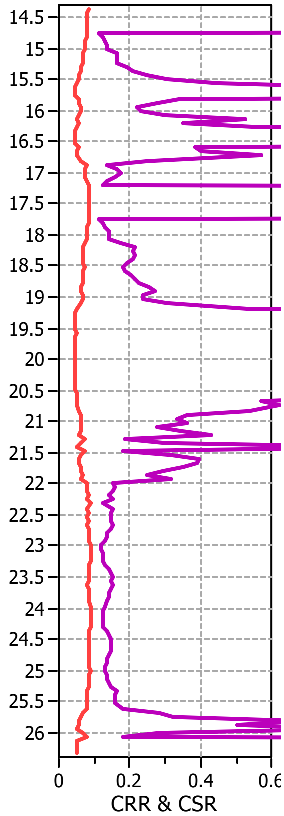
Friction Ratio



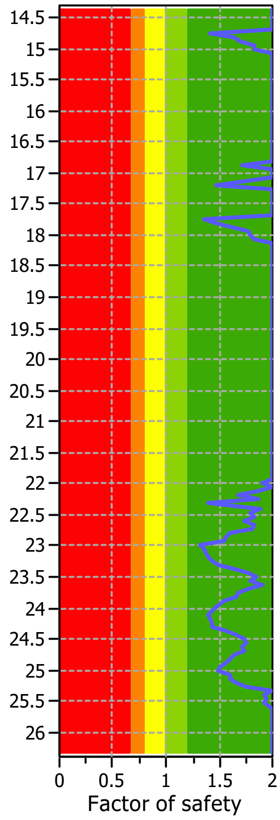
SBTn Plot



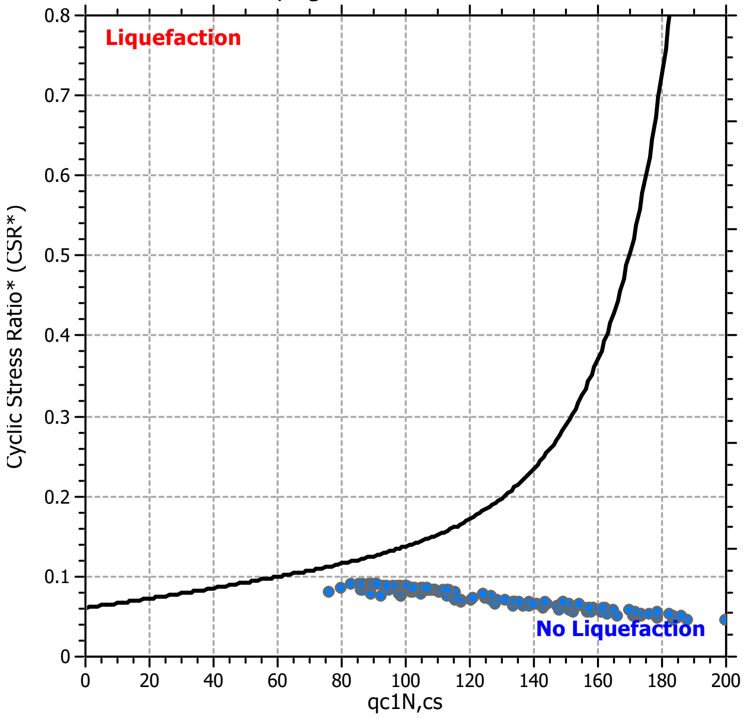
CRR plot



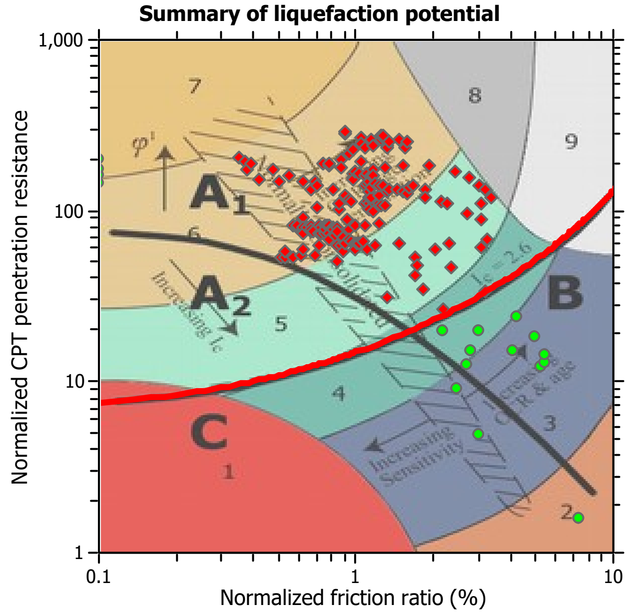
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



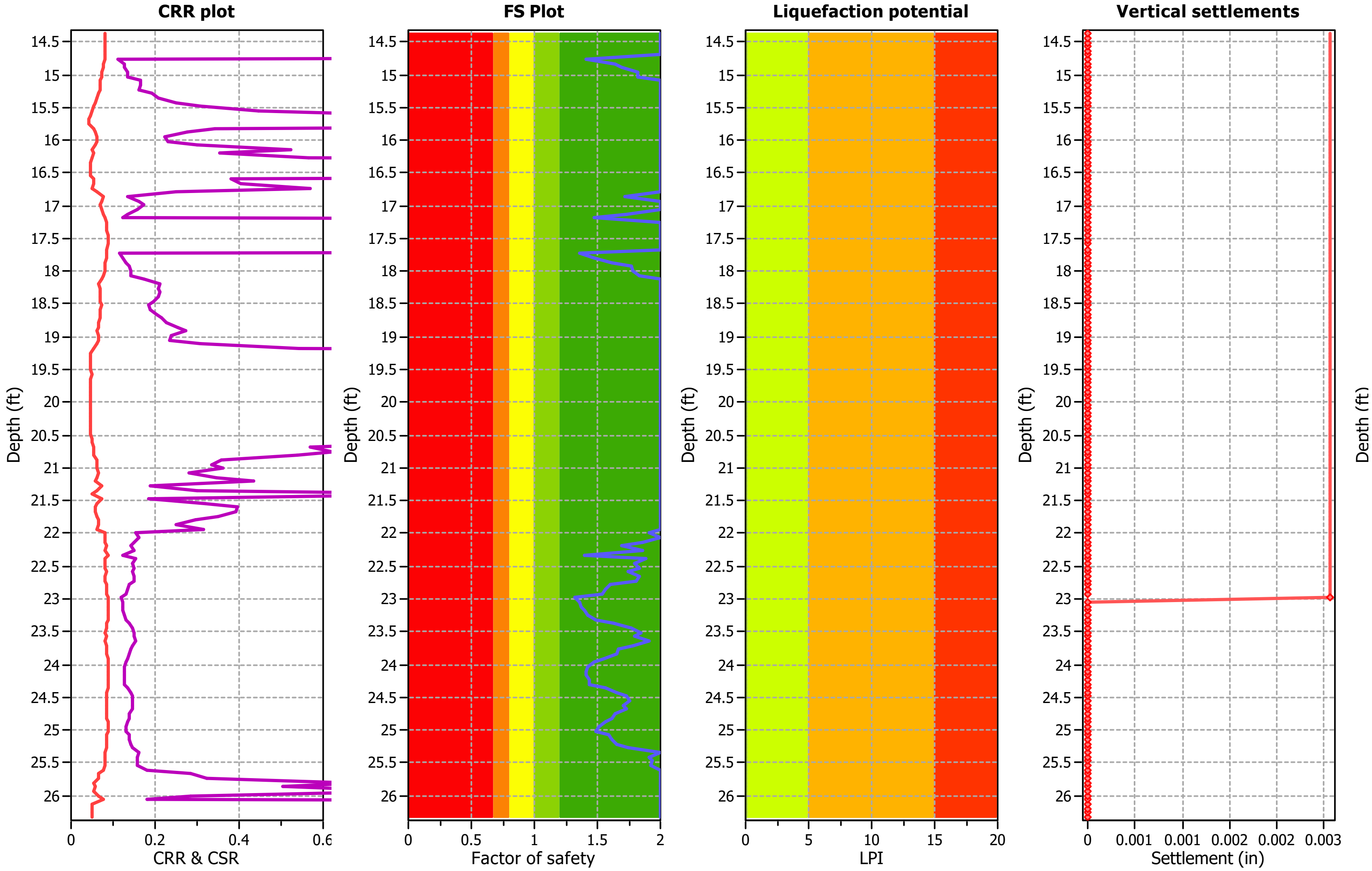
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data ::

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
1	14.37	0.66	0.12	0.54	0.92	0.095	1.69	0.056	1.05	1.00	0.082	No
2	14.44	0.67	0.12	0.54	0.92	0.095	1.69	0.056	1.05	1.00	0.082	No
3	14.52	0.67	0.13	0.55	0.92	0.095	1.69	0.057	1.05	1.00	0.081	No
4	14.57	0.67	0.13	0.55	0.92	0.096	1.69	0.057	1.05	1.00	0.081	No
5	14.63	0.68	0.13	0.55	0.92	0.096	1.69	0.057	1.06	1.00	0.080	No
6	14.70	0.68	0.13	0.55	0.92	0.096	1.69	0.057	1.06	1.00	0.080	No
7	14.76	0.68	0.13	0.55	0.92	0.096	1.69	0.057	1.06	1.00	0.080	No
8	14.83	0.69	0.14	0.55	0.92	0.096	1.69	0.057	1.06	1.00	0.077	No
9	14.89	0.69	0.14	0.55	0.92	0.097	1.69	0.057	1.06	1.00	0.076	No
10	14.96	0.69	0.14	0.56	0.92	0.097	1.69	0.057	1.07	1.00	0.074	No
11	15.03	0.70	0.14	0.56	0.92	0.097	1.69	0.057	1.07	1.00	0.074	No
12	15.09	0.70	0.14	0.56	0.91	0.097	1.69	0.058	1.08	1.00	0.069	No
13	15.16	0.71	0.15	0.56	0.91	0.097	1.69	0.058	1.08	1.00	0.069	No
14	15.22	0.71	0.15	0.56	0.91	0.097	1.69	0.058	1.08	1.00	0.069	No
15	15.29	0.71	0.15	0.57	0.91	0.098	1.69	0.058	1.08	1.00	0.065	No
16	15.35	0.72	0.15	0.57	0.91	0.098	1.69	0.058	1.09	1.00	0.063	No
17	15.42	0.72	0.15	0.57	0.91	0.098	1.69	0.058	1.09	1.00	0.059	No
18	15.49	0.73	0.16	0.57	0.91	0.098	1.69	0.058	1.10	1.00	0.056	No
19	15.55	0.73	0.16	0.57	0.91	0.098	1.69	0.058	1.10	1.00	0.051	No
20	15.62	0.73	0.16	0.57	0.91	0.098	1.69	0.058	1.10	1.00	0.045	No
21	15.68	0.74	0.16	0.58	0.91	0.098	1.69	0.058	1.10	1.00	0.044	No
22	15.75	0.74	0.16	0.58	0.91	0.099	1.69	0.058	1.10	1.00	0.044	No
23	15.81	0.75	0.17	0.58	0.91	0.099	1.69	0.059	1.10	1.00	0.055	No
24	15.88	0.75	0.17	0.58	0.91	0.099	1.69	0.059	1.09	1.00	0.058	No
25	15.95	0.75	0.17	0.58	0.91	0.099	1.69	0.059	1.09	1.00	0.063	No
26	16.01	0.76	0.17	0.59	0.91	0.099	1.69	0.059	1.09	1.00	0.062	No
27	16.08	0.76	0.17	0.59	0.91	0.099	1.69	0.059	1.10	1.00	0.057	No
28	16.14	0.77	0.18	0.59	0.91	0.099	1.69	0.059	1.10	1.00	0.050	No
29	16.21	0.77	0.18	0.59	0.91	0.100	1.69	0.059	1.10	1.00	0.055	No
30	16.27	0.77	0.18	0.59	0.91	0.100	1.69	0.059	1.10	1.00	0.049	No
31	16.34	0.78	0.18	0.60	0.91	0.100	1.69	0.059	1.10	1.00	0.045	No
32	16.40	0.78	0.18	0.60	0.90	0.100	1.69	0.059	1.10	1.00	0.045	No
33	16.47	0.79	0.19	0.60	0.90	0.100	1.69	0.059	1.10	1.00	0.047	No
34	16.54	0.79	0.19	0.60	0.90	0.100	1.69	0.059	1.10	1.00	0.045	No
35	16.60	0.80	0.19	0.60	0.90	0.100	1.69	0.059	1.10	1.00	0.054	No
36	16.67	0.80	0.19	0.61	0.90	0.100	1.69	0.060	1.10	1.00	0.053	No
37	16.73	0.80	0.19	0.61	0.90	0.101	1.69	0.060	1.10	1.00	0.049	No
38	16.80	0.81	0.20	0.61	0.90	0.101	1.69	0.060	1.08	1.00	0.062	No
39	16.86	0.81	0.20	0.61	0.90	0.101	1.69	0.060	1.06	1.00	0.078	No
40	16.93	0.82	0.20	0.62	0.90	0.101	1.69	0.060	1.07	1.00	0.072	No
41	17.00	0.82	0.20	0.62	0.90	0.101	1.69	0.060	1.07	1.00	0.071	No
42	17.06	0.82	0.20	0.62	0.90	0.101	1.69	0.060	1.06	1.00	0.074	No
43	17.13	0.83	0.21	0.62	0.90	0.101	1.69	0.060	1.06	1.00	0.079	No
44	17.19	0.83	0.21	0.62	0.90	0.101	1.69	0.060	1.05	1.00	0.082	No
45	17.26	0.83	0.21	0.62	0.90	0.102	1.69	0.060	1.05	1.00	0.085	No
46	17.32	0.84	0.21	0.63	0.90	0.102	1.69	0.060	1.05	1.00	0.086	No
47	17.39	0.84	0.21	0.63	0.90	0.102	1.69	0.060	1.04	1.00	0.086	No
48	17.45	0.85	0.22	0.63	0.90	0.102	1.69	0.060	1.04	1.00	0.087	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
49	17.52	0.85	0.22	0.63	0.90	0.102	1.69	0.060	1.04	1.00	0.087	No
50	17.59	0.85	0.22	0.63	0.90	0.102	1.69	0.061	1.04	1.00	0.087	No
51	17.67	0.86	0.22	0.63	0.89	0.102	1.69	0.061	1.04	1.00	0.086	No
52	17.73	0.86	0.23	0.64	0.89	0.102	1.69	0.061	1.05	1.00	0.085	No
53	17.80	0.86	0.23	0.64	0.89	0.103	1.69	0.061	1.05	1.00	0.083	No
54	17.86	0.87	0.23	0.64	0.89	0.103	1.69	0.061	1.05	1.00	0.081	No
55	17.93	0.87	0.23	0.64	0.89	0.103	1.69	0.061	1.05	1.00	0.079	No
56	18.00	0.88	0.23	0.64	0.89	0.103	1.69	0.061	1.05	1.00	0.079	No
57	18.06	0.88	0.24	0.64	0.89	0.103	1.69	0.061	1.05	1.00	0.078	No
58	18.13	0.88	0.24	0.65	0.89	0.103	1.69	0.061	1.06	1.00	0.072	No
59	18.19	0.89	0.24	0.65	0.89	0.103	1.69	0.061	1.07	1.00	0.067	No
60	18.26	0.89	0.24	0.65	0.89	0.103	1.69	0.061	1.07	1.00	0.068	No
61	18.32	0.90	0.24	0.65	0.89	0.103	1.69	0.061	1.07	1.00	0.067	No
62	18.39	0.90	0.25	0.65	0.89	0.103	1.69	0.061	1.07	1.00	0.068	No
63	18.45	0.90	0.25	0.66	0.89	0.104	1.69	0.061	1.06	1.00	0.070	No
64	18.52	0.91	0.25	0.66	0.89	0.104	1.69	0.061	1.06	1.00	0.071	No
65	18.59	0.91	0.25	0.66	0.89	0.104	1.69	0.061	1.06	1.00	0.071	No
66	18.65	0.92	0.25	0.66	0.89	0.104	1.69	0.062	1.06	1.00	0.069	No
67	18.72	0.92	0.26	0.66	0.89	0.104	1.69	0.062	1.07	1.00	0.067	No
68	18.78	0.92	0.26	0.67	0.89	0.104	1.69	0.062	1.07	1.00	0.066	No
69	18.85	0.93	0.26	0.67	0.89	0.104	1.69	0.062	1.07	1.00	0.064	No
70	18.91	0.93	0.26	0.67	0.88	0.104	1.69	0.062	1.07	1.00	0.063	No
71	18.98	0.94	0.26	0.67	0.88	0.104	1.69	0.062	1.07	1.00	0.065	No
72	19.05	0.94	0.27	0.67	0.88	0.104	1.69	0.062	1.07	1.00	0.066	No
73	19.11	0.94	0.27	0.68	0.88	0.104	1.69	0.062	1.07	1.00	0.061	No
74	19.18	0.95	0.27	0.68	0.88	0.104	1.69	0.062	1.09	1.00	0.052	No
75	19.24	0.95	0.27	0.68	0.88	0.104	1.69	0.062	1.10	1.00	0.047	No
76	19.31	0.96	0.27	0.68	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
77	19.37	0.96	0.28	0.68	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
78	19.44	0.97	0.28	0.69	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
79	19.50	0.97	0.28	0.69	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
80	19.57	0.97	0.28	0.69	0.88	0.105	1.69	0.062	1.09	1.00	0.050	No
81	19.64	0.98	0.29	0.69	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
82	19.70	0.98	0.29	0.70	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
83	19.77	0.99	0.29	0.70	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
84	19.83	0.99	0.29	0.70	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
85	19.90	1.00	0.29	0.70	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
86	19.96	1.00	0.30	0.70	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
87	20.03	1.00	0.30	0.71	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
88	20.09	1.01	0.30	0.71	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
89	20.16	1.01	0.30	0.71	0.88	0.105	1.69	0.062	1.10	1.00	0.047	No
90	20.23	1.02	0.30	0.71	0.87	0.105	1.69	0.062	1.10	1.00	0.047	No
91	20.29	1.02	0.31	0.72	0.87	0.105	1.69	0.062	1.10	1.00	0.047	No
92	20.36	1.03	0.31	0.72	0.87	0.105	1.69	0.063	1.10	1.00	0.047	No
93	20.42	1.03	0.31	0.72	0.87	0.105	1.69	0.063	1.10	1.00	0.047	No
94	20.49	1.03	0.31	0.72	0.87	0.106	1.69	0.063	1.10	1.00	0.047	No
95	20.55	1.04	0.31	0.72	0.87	0.106	1.69	0.063	1.08	1.00	0.049	No
96	20.62	1.04	0.32	0.73	0.87	0.106	1.69	0.063	1.08	1.00	0.048	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
97	20.69	1.05	0.32	0.73	0.87	0.106	1.69	0.063	1.07	1.00	0.053	No
98	20.75	1.05	0.32	0.73	0.87	0.106	1.69	0.063	1.08	1.00	0.052	No
99	20.82	1.05	0.32	0.73	0.87	0.106	1.69	0.063	1.07	1.00	0.054	No
100	20.88	1.06	0.32	0.73	0.87	0.106	1.69	0.063	1.06	1.00	0.060	No
101	20.95	1.06	0.33	0.74	0.87	0.106	1.69	0.063	1.06	1.00	0.061	No
102	21.01	1.07	0.33	0.74	0.87	0.106	1.69	0.063	1.06	1.00	0.060	No
103	21.08	1.07	0.33	0.74	0.87	0.106	1.69	0.063	1.06	1.00	0.064	No
104	21.14	1.07	0.33	0.74	0.87	0.106	1.69	0.063	1.06	1.00	0.060	No
105	21.21	1.08	0.33	0.74	0.87	0.106	1.69	0.063	1.06	1.00	0.057	No
106	21.28	1.08	0.34	0.75	0.87	0.106	1.69	0.063	1.05	1.00	0.074	No
107	21.34	1.09	0.34	0.75	0.87	0.106	1.69	0.063	1.06	1.00	0.063	No
108	21.41	1.09	0.34	0.75	0.86	0.106	1.69	0.063	1.08	1.00	0.049	No
109	21.47	1.10	0.34	0.75	0.86	0.106	1.69	0.063	1.04	1.00	0.075	No
110	21.54	1.10	0.34	0.76	0.86	0.106	1.69	0.063	1.06	1.00	0.062	No
111	21.60	1.10	0.35	0.76	0.86	0.106	1.69	0.063	1.06	1.00	0.059	No
112	21.67	1.11	0.35	0.76	0.86	0.106	1.69	0.063	1.06	1.00	0.059	No
113	21.74	1.11	0.35	0.76	0.86	0.106	1.69	0.063	1.06	1.00	0.061	No
114	21.80	1.12	0.35	0.76	0.86	0.106	1.69	0.063	1.05	1.00	0.064	No
115	21.87	1.12	0.35	0.77	0.86	0.106	1.69	0.063	1.05	1.00	0.067	No
116	21.93	1.12	0.36	0.77	0.86	0.107	1.69	0.063	1.05	1.00	0.063	No
117	22.00	1.13	0.36	0.77	0.86	0.107	1.69	0.063	1.04	1.00	0.080	No
118	22.06	1.13	0.36	0.77	0.86	0.107	1.69	0.063	1.04	1.00	0.079	No
119	22.13	1.14	0.36	0.77	0.86	0.107	1.69	0.063	1.04	1.00	0.081	No
120	22.20	1.14	0.36	0.77	0.86	0.107	1.69	0.063	1.03	1.00	0.083	No
121	22.26	1.14	0.37	0.78	0.86	0.107	1.69	0.063	1.04	1.00	0.081	No
122	22.33	1.15	0.37	0.78	0.86	0.107	1.69	0.063	1.03	1.00	0.088	No
123	22.39	1.15	0.37	0.78	0.86	0.107	1.69	0.063	1.04	1.00	0.081	No
124	22.46	1.15	0.37	0.78	0.86	0.107	1.69	0.063	1.03	1.00	0.082	No
125	22.52	1.16	0.38	0.78	0.86	0.107	1.69	0.063	1.03	1.00	0.082	No
126	22.59	1.16	0.38	0.79	0.86	0.107	1.69	0.063	1.03	1.00	0.083	No
127	22.65	1.17	0.38	0.79	0.85	0.107	1.69	0.063	1.03	1.00	0.082	No
128	22.72	1.17	0.38	0.79	0.85	0.107	1.69	0.063	1.03	1.00	0.082	No
129	22.79	1.17	0.38	0.79	0.85	0.107	1.69	0.064	1.03	1.00	0.085	No
130	22.85	1.18	0.39	0.79	0.85	0.107	1.69	0.064	1.03	1.00	0.086	No
131	22.92	1.18	0.39	0.79	0.85	0.107	1.69	0.064	1.03	1.00	0.086	No
132	22.98	1.18	0.39	0.80	0.85	0.107	1.69	0.064	1.03	1.00	0.090	No
133	23.05	1.19	0.39	0.80	0.85	0.107	1.69	0.064	1.03	1.00	0.089	No
134	23.11	1.19	0.39	0.80	0.85	0.107	1.69	0.064	1.03	1.00	0.089	No
135	23.18	1.20	0.40	0.80	0.85	0.107	1.69	0.064	1.03	1.00	0.089	No
136	23.25	1.20	0.40	0.80	0.85	0.107	1.69	0.064	1.03	1.00	0.088	No
137	23.31	1.20	0.40	0.80	0.85	0.107	1.69	0.064	1.03	1.00	0.087	No
138	23.38	1.21	0.40	0.80	0.85	0.108	1.69	0.064	1.03	1.00	0.085	No
139	23.44	1.21	0.40	0.81	0.85	0.108	1.69	0.064	1.03	1.00	0.083	No
140	23.51	1.21	0.41	0.81	0.85	0.108	1.69	0.064	1.03	1.00	0.082	No
141	23.57	1.22	0.41	0.81	0.85	0.108	1.69	0.064	1.03	1.00	0.083	No
142	23.64	1.22	0.41	0.81	0.85	0.108	1.69	0.064	1.03	1.00	0.081	No
143	23.70	1.23	0.41	0.81	0.85	0.108	1.69	0.064	1.03	1.00	0.083	No
144	23.77	1.23	0.41	0.82	0.85	0.108	1.69	0.064	1.03	1.00	0.085	No

:: Cyclic Stress Ratio fully adjusted (CSR*) calculation data :: (continued)

Point ID	Depth (ft)	σ_v (tsf)	u_0 (tsf)	σ_v' (tsf)	r_d	CSR	MSF	CSR_{eq}	K_σ	User FS	CSR^*	Belongs to transition
145	23.84	1.23	0.42	0.82	0.84	0.108	1.69	0.064	1.03	1.00	0.085	No
146	23.90	1.24	0.42	0.82	0.84	0.108	1.69	0.064	1.03	1.00	0.087	No
147	23.97	1.24	0.42	0.82	0.84	0.108	1.69	0.064	1.03	1.00	0.088	No
148	24.03	1.24	0.42	0.82	0.84	0.108	1.69	0.064	1.02	1.00	0.089	No
149	24.10	1.25	0.42	0.82	0.84	0.108	1.69	0.064	1.02	1.00	0.089	No
150	24.16	1.25	0.43	0.83	0.84	0.108	1.69	0.064	1.02	1.00	0.089	No
151	24.23	1.26	0.43	0.83	0.84	0.108	1.69	0.064	1.02	1.00	0.089	No
152	24.30	1.26	0.43	0.83	0.84	0.108	1.69	0.064	1.02	1.00	0.089	No
153	24.36	1.26	0.43	0.83	0.84	0.108	1.69	0.064	1.03	1.00	0.087	No
154	24.43	1.27	0.43	0.83	0.84	0.108	1.69	0.064	1.03	1.00	0.085	No
155	24.49	1.27	0.44	0.83	0.84	0.108	1.69	0.064	1.03	1.00	0.084	No
156	24.56	1.27	0.44	0.84	0.84	0.108	1.69	0.064	1.03	1.00	0.084	No
157	24.62	1.28	0.44	0.84	0.84	0.108	1.69	0.064	1.03	1.00	0.084	No
158	24.69	1.28	0.44	0.84	0.84	0.108	1.69	0.064	1.03	1.00	0.084	No
159	24.75	1.29	0.44	0.84	0.84	0.108	1.69	0.064	1.02	1.00	0.085	No
160	24.82	1.29	0.45	0.84	0.84	0.108	1.69	0.064	1.02	1.00	0.086	No
161	24.89	1.29	0.45	0.85	0.84	0.108	1.69	0.064	1.02	1.00	0.087	No
162	24.95	1.30	0.45	0.85	0.84	0.108	1.69	0.064	1.02	1.00	0.088	No
163	25.02	1.30	0.45	0.85	0.84	0.108	1.69	0.064	1.02	1.00	0.088	No
164	25.08	1.31	0.45	0.85	0.83	0.108	1.69	0.064	1.02	1.00	0.086	No
165	25.15	1.31	0.46	0.85	0.83	0.108	1.69	0.064	1.02	1.00	0.086	No
166	25.21	1.31	0.46	0.85	0.83	0.108	1.69	0.064	1.02	1.00	0.086	No
167	25.28	1.32	0.46	0.86	0.83	0.108	1.69	0.064	1.02	1.00	0.084	No
168	25.34	1.32	0.46	0.86	0.83	0.108	1.69	0.064	1.03	1.00	0.081	No
169	25.41	1.32	0.47	0.86	0.83	0.108	1.69	0.064	1.02	1.00	0.082	No
170	25.48	1.33	0.47	0.86	0.83	0.108	1.69	0.064	1.02	1.00	0.082	No
171	25.54	1.33	0.47	0.86	0.83	0.108	1.69	0.064	1.02	1.00	0.082	No
172	25.61	1.34	0.47	0.87	0.83	0.108	1.69	0.064	1.03	1.00	0.078	No
173	25.67	1.34	0.47	0.87	0.83	0.108	1.69	0.064	1.03	1.00	0.067	No
174	25.74	1.34	0.48	0.87	0.83	0.108	1.69	0.064	1.03	1.00	0.065	No
175	25.80	1.35	0.48	0.87	0.83	0.108	1.69	0.064	1.04	1.00	0.054	No
176	25.87	1.35	0.48	0.87	0.83	0.108	1.69	0.064	1.04	1.00	0.058	No
177	25.93	1.36	0.48	0.88	0.83	0.108	1.69	0.064	1.04	1.00	0.053	No
178	26.00	1.36	0.48	0.88	0.83	0.108	1.69	0.064	1.03	1.00	0.067	No
179	26.07	1.37	0.49	0.88	0.83	0.108	1.69	0.064	1.02	1.00	0.078	No
180	26.13	1.37	0.49	0.88	0.83	0.108	1.69	0.064	1.06	1.00	0.051	No
181	26.20	1.37	0.49	0.88	0.83	0.108	1.69	0.064	1.05	1.00	0.051	No
182	26.26	1.37	0.49	0.88	0.82	0.109	1.69	0.064	1.05	1.00	0.051	No
183	26.33	1.38	0.49	0.88	0.82	0.109	1.69	0.064	1.05	1.00	0.051	No

Abbreviations

Depth: Depth from free surface, at which CPT was performed (ft)
σ_v: Total overburden pressure at test point (tsf)
u_0: Water pressure at test point (tsf)
σ_v': Effective overburden pressure based on GWT during earthquake (tsf)
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Magnitude Scaling Factor
CSR_{eq}: CSR adjusted for M=7.5
K_σ: Effective overburden stress factor
CSR^*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) calculation data ::

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
1	14.37	1.65	100.00	3.88	0.62	1.51	1.07	0.00	1.07	4.000	No	Yes	2.00
2	14.44	3.74	100.00	3.26	0.60	1.49	4.73	0.00	4.73	4.000	No	Yes	2.00
3	14.52	6.30	100.00	2.99	0.58	1.47	9.75	0.00	9.75	4.000	No	Yes	2.00
4	14.57	8.47	94.23	2.89	0.58	1.46	11.67	0.00	11.67	4.000	No	Yes	2.00
5	14.63	10.04	89.75	2.83	0.57	1.46	13.55	0.00	13.55	4.000	No	Yes	2.00
6	14.70	12.88	77.34	2.68	0.57	1.45	16.07	0.00	16.07	4.000	No	Yes	2.00
7	14.76	20.63	53.64	2.38	0.56	1.44	23.00	53.35	76.35	0.113	No	No	1.42
8	14.83	35.01	31.39	2.10	0.52	1.40	43.94	45.63	89.57	0.125	No	No	1.64
9	14.89	53.47	14.88	1.90	0.51	1.39	72.32	20.39	92.71	0.129	No	No	1.70
10	14.96	67.18	8.56	1.82	0.50	1.38	94.43	4.10	98.53	0.136	No	No	1.82
11	15.03	77.77	7.30	1.80	0.50	1.38	96.73	1.93	98.66	0.136	No	No	1.82
12	15.09	82.38	9.72	1.83	0.46	1.34	109.99	7.21	117.20	0.165	No	No	2.00
13	15.16	87.14	10.06	1.84	0.46	1.34	109.20	8.14	117.34	0.166	No	No	2.00
14	15.22	92.58	7.92	1.81	0.47	1.34	112.30	3.07	115.37	0.162	No	No	2.00
15	15.29	99.88	3.85	1.76	0.44	1.32	128.28	0.01	128.30	0.193	No	No	2.00
16	15.35	109.45	0.00	1.69	0.43	1.31	133.61	0.00	133.61	0.209	No	No	2.00
17	15.42	117.17	0.00	1.64	0.41	1.29	143.29	0.00	143.29	0.250	No	No	2.00
18	15.49	127.68	0.00	1.58	0.40	1.28	152.45	0.00	152.45	0.306	No	No	2.00
19	15.55	140.95	0.00	1.52	0.38	1.26	166.27	0.00	166.27	0.447	No	No	2.00
20	15.62	156.70	0.00	1.44	0.35	1.24	183.91	0.00	183.91	0.855	No	No	2.00
21	15.68	169.30	0.00	1.39	0.33	1.22	200.05	0.00	200.05	1.897	No	No	2.00
22	15.75	160.68	0.00	1.42	0.32	1.22	203.84	0.00	203.84	2.366	No	No	2.00
23	15.81	143.87	0.00	1.46	0.39	1.27	156.91	0.00	156.91	0.342	No	No	2.00
24	15.88	122.22	0.00	1.54	0.41	1.27	148.11	0.00	148.11	0.277	No	No	2.00
25	15.95	116.71	0.00	1.60	0.43	1.29	136.84	0.00	136.84	0.221	No	No	2.00
26	16.01	117.97	0.00	1.67	0.42	1.28	139.05	0.00	139.05	0.230	No	No	2.00
27	16.08	129.04	2.10	1.74	0.40	1.27	151.55	0.00	151.55	0.299	No	No	2.00
28	16.14	134.59	5.33	1.78	0.37	1.24	170.82	0.32	171.15	0.524	No	No	2.00
29	16.21	138.61	7.35	1.80	0.39	1.25	155.66	2.44	158.10	0.353	No	No	2.00
30	16.27	141.52	10.57	1.84	0.37	1.24	162.04	11.38	173.42	0.567	No	No	2.00
31	16.34	136.50	17.24	1.93	0.31	1.20	174.56	37.77	212.34	4.000	No	No	2.00
32	16.40	124.29	25.47	2.03	0.35	1.22	133.81	54.48	188.29	1.039	No	No	2.00
33	16.47	112.28	30.72	2.10	0.36	1.23	118.28	60.28	178.57	0.687	No	No	2.00
34	16.54	101.22	34.98	2.15	0.32	1.20	134.23	69.16	203.40	2.303	No	No	2.00
35	16.60	94.76	37.81	2.19	0.39	1.24	97.43	63.60	161.03	0.383	No	No	2.00
36	16.67	85.74	41.25	2.23	0.38	1.24	96.72	66.23	162.95	0.404	No	No	2.00
37	16.73	80.03	40.67	2.22	0.37	1.22	105.58	67.94	173.52	0.569	No	No	2.00
38	16.80	63.22	45.84	2.29	0.42	1.26	78.34	64.60	142.94	0.248	No	No	2.00
39	16.86	47.60	53.52	2.38	0.50	1.32	40.16	57.96	98.12	0.135	No	No	1.72
40	16.93	42.29	54.97	2.40	0.46	1.29	53.95	62.22	116.17	0.163	No	No	2.00
41	17.00	46.48	46.25	2.29	0.46	1.28	60.47	60.24	120.70	0.173	No	No	2.00
42	17.06	42.57	45.98	2.29	0.47	1.29	54.54	58.57	113.12	0.158	No	No	2.00
43	17.13	33.18	55.03	2.40	0.50	1.31	40.31	58.52	98.83	0.136	No	No	1.73
44	17.19	23.32	70.58	2.59	0.53	1.33	27.43	58.71	86.14	0.122	No	No	1.48
45	17.26	16.32	85.85	2.79	0.55	1.34	19.33	0.00	19.33	4.000	No	Yes	2.00
46	17.32	12.28	97.02	2.93	0.56	1.35	14.79	0.00	14.79	4.000	No	Yes	2.00
47	17.39	9.90	100.00	3.03	0.57	1.35	12.45	0.00	12.45	4.000	No	Yes	2.00
48	17.45	8.86	100.00	3.07	0.58	1.35	10.37	0.00	10.37	4.000	No	Yes	2.00

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
49	17.52	8.54	100.00	3.07	0.58	1.35	10.81	0.00	10.81	4.000	No	Yes	2.00
50	17.59	10.50	97.45	2.93	0.58	1.35	11.15	0.00	11.15	4.000	No	Yes	2.00
51	17.67	13.85	83.72	2.76	0.56	1.33	17.25	0.00	17.25	4.000	No	Yes	2.00
52	17.73	19.12	68.72	2.57	0.55	1.32	22.76	57.01	79.76	0.116	No	No	1.36
53	17.80	25.00	57.94	2.44	0.53	1.31	30.17	56.61	86.78	0.122	No	No	1.47
54	17.86	31.28	50.51	2.34	0.51	1.29	38.44	56.39	94.83	0.131	No	No	1.62
55	17.93	36.01	47.31	2.30	0.49	1.28	45.13	56.79	101.92	0.140	No	No	1.77
56	18.00	40.18	44.23	2.27	0.49	1.28	46.84	55.71	102.55	0.141	No	No	1.78
57	18.06	49.63	35.81	2.16	0.49	1.27	53.07	51.63	104.70	0.144	No	No	1.84
58	18.13	64.99	25.03	2.03	0.45	1.25	77.68	43.82	121.50	0.175	No	No	2.00
59	18.19	78.56	19.40	1.95	0.43	1.24	99.19	35.40	134.58	0.213	No	No	2.00
60	18.26	84.87	19.06	1.95	0.43	1.23	99.02	34.50	133.53	0.209	No	No	2.00
61	18.32	85.58	19.62	1.96	0.43	1.23	98.65	35.89	134.54	0.213	No	No	2.00
62	18.39	86.56	18.07	1.94	0.43	1.23	101.36	32.16	133.53	0.209	No	No	2.00
63	18.45	87.35	16.05	1.91	0.44	1.23	102.47	26.48	128.95	0.194	No	No	2.00
64	18.52	88.16	14.97	1.90	0.45	1.24	101.87	23.11	124.98	0.183	No	No	2.00
65	18.59	90.77	14.82	1.90	0.44	1.23	104.31	22.86	127.17	0.189	No	No	2.00
66	18.65	96.93	13.93	1.89	0.44	1.23	110.93	20.48	131.41	0.202	No	No	2.00
67	18.72	105.79	11.74	1.86	0.43	1.22	121.86	13.81	135.67	0.217	No	No	2.00
68	18.78	114.61	8.73	1.82	0.42	1.22	133.51	5.12	138.63	0.229	No	No	2.00
69	18.85	118.66	8.50	1.82	0.41	1.21	139.57	4.65	144.22	0.255	No	No	2.00
70	18.91	115.71	11.48	1.86	0.41	1.20	134.05	13.50	147.55	0.273	No	No	2.00
71	18.98	113.26	13.10	1.88	0.42	1.21	122.60	18.49	141.09	0.240	No	No	2.00
72	19.05	118.94	10.12	1.84	0.42	1.21	131.18	8.93	140.11	0.235	No	No	2.00
73	19.11	134.75	5.02	1.78	0.40	1.20	152.30	0.19	152.50	0.306	No	No	2.00
74	19.18	161.07	0.34	1.72	0.37	1.18	172.18	0.00	172.18	0.543	No	No	2.00
75	19.24	197.44	0.00	1.67	0.32	1.15	210.50	0.00	210.50	3.605	No	No	2.00
76	19.31	218.15	0.00	1.69	0.26	1.12	258.74	0.00	258.74	4.000	No	No	2.00
77	19.37	221.14	1.98	1.74	0.29	1.13	232.29	0.00	232.29	4.000	No	No	2.00
78	19.44	212.14	4.71	1.77	0.31	1.14	218.57	0.14	218.71	4.000	No	No	2.00
79	19.50	193.91	6.37	1.79	0.29	1.13	231.81	1.31	233.12	4.000	No	No	2.00
80	19.57	198.63	1.84	1.74	0.36	1.17	178.47	0.00	178.47	0.684	No	No	2.00
81	19.64	206.40	0.00	1.68	0.29	1.13	231.62	0.00	231.62	4.000	No	No	2.00
82	19.70	251.18	0.00	1.55	0.26	1.12	253.57	0.00	253.57	4.000	No	No	2.00
83	19.77	243.62	0.00	1.56	0.26	1.12	312.65	0.00	312.65	4.000	No	No	2.00
84	19.83	230.34	0.00	1.62	0.32	1.14	209.22	0.00	209.22	3.314	No	No	2.00
85	19.90	204.09	1.14	1.73	0.31	1.14	214.96	0.00	214.96	4.000	No	No	2.00
86	19.96	215.11	0.41	1.72	0.29	1.13	231.36	0.00	231.36	4.000	No	No	2.00
87	20.03	234.26	0.00	1.68	0.28	1.12	240.33	0.00	240.33	4.000	No	No	2.00
88	20.09	235.39	0.00	1.67	0.26	1.11	270.77	0.00	270.77	4.000	No	No	2.00
89	20.16	239.17	0.00	1.67	0.29	1.12	234.03	0.00	234.03	4.000	No	No	2.00
90	20.23	230.86	0.00	1.68	0.27	1.11	250.61	0.00	250.61	4.000	No	No	2.00
91	20.29	228.67	0.00	1.65	0.27	1.11	244.93	0.00	244.93	4.000	No	No	2.00
92	20.36	216.87	0.00	1.64	0.29	1.12	227.16	0.00	227.16	4.000	No	No	2.00
93	20.42	202.02	0.00	1.64	0.31	1.13	216.21	0.00	216.21	4.000	No	No	2.00
94	20.49	187.25	0.00	1.65	0.33	1.13	201.63	0.00	201.63	2.077	No	No	2.00
95	20.55	176.73	0.00	1.64	0.35	1.14	183.68	0.00	183.68	0.846	No	No	2.00
96	20.62	167.35	0.00	1.63	0.35	1.14	185.02	0.00	185.02	0.897	No	No	2.00

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
97	20.69	164.90	0.00	1.62	0.37	1.15	173.56	0.00	173.56	0.570	No	No	2.00
98	20.75	160.65	0.00	1.64	0.36	1.14	175.97	0.00	175.97	0.622	No	No	2.00
99	20.82	155.87	0.00	1.69	0.37	1.15	171.95	0.00	171.95	0.539	No	No	2.00
100	20.88	149.33	1.64	1.73	0.39	1.15	158.79	0.00	158.79	0.360	No	No	2.00
101	20.95	144.42	4.94	1.77	0.39	1.15	155.99	0.17	156.16	0.335	No	No	2.00
102	21.01	140.76	7.24	1.80	0.39	1.15	156.65	2.26	158.91	0.361	No	No	2.00
103	21.08	141.42	6.48	1.79	0.41	1.16	147.33	1.16	148.49	0.279	No	No	2.00
104	21.14	144.39	3.55	1.76	0.39	1.15	157.52	0.01	157.53	0.348	No	No	2.00
105	21.21	137.74	3.59	1.76	0.38	1.14	165.23	0.01	165.24	0.433	No	No	2.00
106	21.28	134.43	4.31	1.77	0.44	1.17	126.76	0.05	126.80	0.188	No	No	2.00
107	21.34	128.99	8.46	1.82	0.40	1.15	146.82	4.65	151.47	0.299	No	No	2.00
108	21.41	112.85	19.09	1.95	0.35	1.13	145.64	40.50	186.13	0.942	No	No	2.00
109	21.47	95.54	31.25	2.10	0.45	1.16	73.13	51.59	124.72	0.183	No	No	2.00
110	21.54	81.81	40.42	2.22	0.40	1.14	90.03	63.97	154.00	0.318	No	No	2.00
111	21.60	93.87	32.72	2.12	0.38	1.14	102.73	59.58	162.30	0.397	No	No	2.00
112	21.67	102.25	26.18	2.04	0.38	1.14	110.25	51.59	161.84	0.391	No	No	2.00
113	21.74	108.79	20.72	1.97	0.39	1.14	116.61	41.09	157.70	0.349	No	No	2.00
114	21.80	115.29	15.55	1.91	0.40	1.14	124.15	26.91	151.05	0.296	No	No	2.00
115	21.87	120.53	10.92	1.85	0.41	1.14	132.05	11.51	143.56	0.251	No	No	2.00
116	21.93	106.64	13.45	1.88	0.40	1.14	133.35	20.42	153.77	0.316	No	No	2.00
117	22.00	88.55	18.18	1.94	0.48	1.16	80.84	30.05	110.89	0.154	No	No	1.91
118	22.06	71.21	23.17	2.00	0.47	1.16	74.36	39.99	114.35	0.160	No	No	2.00
119	22.13	70.59	18.29	1.94	0.48	1.16	79.20	30.13	109.33	0.151	No	No	1.86
120	22.20	70.17	15.83	1.91	0.49	1.17	78.93	23.62	102.55	0.141	No	No	1.69
121	22.26	60.46	21.48	1.98	0.48	1.16	72.95	36.49	109.44	0.151	No	No	1.86
122	22.33	55.67	25.79	2.03	0.53	1.18	47.75	39.69	87.43	0.123	No	No	1.39
123	22.39	52.55	29.91	2.09	0.48	1.16	62.57	47.85	110.41	0.153	No	No	1.89
124	22.46	57.29	27.29	2.05	0.48	1.16	62.58	44.51	107.09	0.148	No	No	1.80
125	22.52	56.67	28.44	2.07	0.48	1.16	62.40	46.00	108.40	0.150	No	No	1.83
126	22.59	57.60	27.69	2.06	0.49	1.16	60.52	44.66	105.18	0.145	No	No	1.74
127	22.65	58.55	26.18	2.04	0.48	1.15	65.34	43.43	108.78	0.150	No	No	1.84
128	22.72	58.08	25.53	2.03	0.48	1.15	65.31	42.44	107.75	0.149	No	No	1.81
129	22.79	56.51	24.94	2.02	0.50	1.16	59.01	40.42	99.43	0.137	No	No	1.61
130	22.85	55.61	22.53	1.99	0.50	1.16	60.47	36.61	97.07	0.134	No	No	1.56
131	22.92	53.88	20.73	1.97	0.51	1.16	62.64	33.47	96.10	0.133	No	No	1.54
132	22.98	51.75	19.49	1.96	0.54	1.17	53.74	29.69	83.43	0.119	No	No	1.33
133	23.05	48.91	20.58	1.97	0.53	1.16	53.73	31.91	85.64	0.121	No	No	1.36
134	23.11	48.94	21.22	1.98	0.53	1.16	53.51	33.12	86.62	0.122	No	No	1.37
135	23.18	48.91	22.32	1.99	0.52	1.16	53.46	35.13	88.59	0.124	No	No	1.40
136	23.25	49.27	23.44	2.01	0.52	1.16	53.28	37.03	90.31	0.126	No	No	1.43
137	23.31	50.20	25.03	2.03	0.51	1.15	54.26	39.73	93.98	0.130	No	No	1.49
138	23.38	51.82	28.72	2.07	0.50	1.15	55.88	45.08	100.96	0.139	No	No	1.63
139	23.44	53.72	31.90	2.11	0.48	1.14	57.80	49.06	106.86	0.147	No	No	1.77
140	23.51	55.65	32.34	2.12	0.48	1.14	59.69	49.92	109.60	0.152	No	No	1.85
141	23.57	59.39	28.69	2.07	0.48	1.14	61.71	46.18	107.89	0.149	No	No	1.80
142	23.64	62.22	24.91	2.02	0.47	1.13	69.49	42.20	111.68	0.155	No	No	1.90
143	23.70	64.07	22.91	2.00	0.48	1.14	68.69	38.60	107.29	0.148	No	No	1.78
144	23.77	65.17	21.27	1.98	0.49	1.14	67.64	35.27	102.91	0.141	No	No	1.67

:: Cyclic Resistance Ratio (CRR) calculation data :: (continued)

Point ID	Depth (ft)	q_t (tsf)	FC (%)	Ic	m	C_N	q_{c1N}	Δq_{c1N}	$q_{c1N,cs}$	$CRR_{7.5}$	Belongs to trans. layer	Clay-like behaviour	FS
145	23.84	68.01	18.10	1.94	0.49	1.14	73.18	28.95	102.13	0.140	No	No	1.65
146	23.90	72.07	14.33	1.89	0.50	1.14	78.02	19.27	97.29	0.134	No	No	1.55
147	23.97	74.71	11.93	1.86	0.51	1.14	80.91	12.45	93.36	0.129	No	No	1.47
148	24.03	76.08	10.56	1.84	0.52	1.14	81.97	8.67	90.64	0.126	No	No	1.43
149	24.10	76.65	9.83	1.84	0.52	1.14	82.56	6.78	89.34	0.125	No	No	1.40
150	24.16	76.16	10.04	1.84	0.52	1.14	82.68	7.33	90.01	0.126	No	No	1.41
151	24.23	73.99	11.40	1.86	0.52	1.14	80.13	10.91	91.05	0.127	No	No	1.43
152	24.30	71.92	13.19	1.88	0.52	1.13	75.25	15.79	91.04	0.127	No	No	1.43
153	24.36	70.64	15.59	1.91	0.50	1.13	75.39	22.63	98.03	0.135	No	No	1.56
154	24.43	71.27	17.36	1.93	0.49	1.13	75.25	27.32	102.56	0.141	No	No	1.65
155	24.49	72.53	18.16	1.94	0.49	1.12	76.43	29.46	105.89	0.146	No	No	1.73
156	24.56	74.42	17.44	1.93	0.48	1.12	78.82	27.93	106.74	0.147	No	No	1.75
157	24.62	76.51	15.94	1.91	0.49	1.12	80.93	24.12	105.04	0.144	No	No	1.71
158	24.69	77.06	15.45	1.91	0.49	1.12	82.81	22.90	105.72	0.145	No	No	1.73
159	24.75	77.41	15.09	1.90	0.49	1.12	80.49	21.65	102.14	0.140	No	No	1.64
160	24.82	77.72	14.08	1.89	0.50	1.12	81.96	18.86	100.82	0.139	No	No	1.61
161	24.89	77.58	12.76	1.87	0.50	1.12	83.82	15.06	98.88	0.136	No	No	1.57
162	24.95	74.99	13.18	1.88	0.51	1.12	80.09	16.08	96.17	0.133	No	No	1.52
163	25.02	71.77	14.99	1.90	0.51	1.12	73.73	20.81	94.54	0.131	No	No	1.48
164	25.08	69.46	17.28	1.93	0.50	1.11	73.32	26.90	100.22	0.138	No	No	1.60
165	25.15	68.99	17.98	1.94	0.50	1.11	72.34	28.56	100.90	0.139	No	No	1.61
166	25.21	69.26	19.08	1.95	0.49	1.11	71.71	31.10	102.81	0.141	No	No	1.65
167	25.28	71.50	19.80	1.96	0.48	1.11	73.74	33.02	106.75	0.147	No	No	1.75
168	25.34	73.28	21.36	1.98	0.47	1.10	78.66	37.09	115.75	0.163	No	No	2.00
169	25.41	76.94	21.05	1.98	0.47	1.10	76.73	36.15	112.88	0.157	No	No	1.92
170	25.48	83.48	17.55	1.93	0.47	1.10	84.73	28.86	113.59	0.158	No	No	1.94
171	25.54	95.51	12.28	1.87	0.47	1.10	98.90	14.44	113.34	0.158	No	No	1.93
172	25.61	108.89	11.02	1.85	0.45	1.09	113.50	11.15	124.65	0.183	No	No	2.00
173	25.67	116.98	14.96	1.90	0.40	1.08	124.50	24.96	149.46	0.285	No	No	2.00
174	25.74	125.55	17.44	1.93	0.40	1.08	121.98	32.69	154.68	0.323	No	No	2.00
175	25.80	127.73	19.71	1.96	0.36	1.07	137.29	41.23	178.52	0.686	No	No	2.00
176	25.87	131.76	19.65	1.96	0.37	1.07	129.98	40.10	170.08	0.505	No	No	2.00
177	25.93	127.01	23.30	2.00	0.35	1.07	132.86	49.71	182.58	0.808	No	No	2.00
178	26.00	124.71	15.18	1.90	0.40	1.08	123.55	25.65	149.19	0.284	No	No	2.00
179	26.07	129.94	0.00	1.70	0.45	1.09	124.42	0.00	124.42	0.182	No	No	2.00
180	26.13	129.93	100.00	4.06	0.27	1.05	146.26	0.00	146.26	4.000	No	Yes	2.00
181	26.20	141.58	100.00	4.06	0.31	1.06	121.28	0.00	121.28	4.000	No	Yes	2.00
182	26.26	154.93	100.00	4.06	0.26	1.05	154.84	0.00	154.84	4.000	No	Yes	2.00
183	26.33	176.94	100.00	4.06	0.26	1.05	185.60	0.00	185.60	4.000	No	Yes	2.00

Abbreviations

Depth: Depth from free surface, at which CPT was performed (ft)
q_t: Total cone resistance
FC: Fines content (%)
I_c: Soil behavior type index
m: Stress exponent
C_N: Overburden correction factor
q_{c1N}: Normalized and adjusted cone resistance
Δq_{c1N}: Cone resistance correction factor due to fines
$q_{c1N,cs}$: Normalized and adjusted cone resistance
$CRR_{7.5}$: Cyclic resistance ratio for M_w=7.5
FS: Factor of safety against soil liquefaction

Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{\max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012-1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106+0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1-C_\sigma \ln\left(\frac{\sigma'_v}{P_a}\right) \le 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}} \le 0.3$$

$$MSF = 1+\left(MSF_{max}-1\right)\left(8.64\exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{c1Ncs}}{180}\right)^3 \le 2.2$$

σ'_v *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N}+\Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{N}\frac{q_c}{P_a}$$

$$C_{N} = \left(\frac{P_a}{\sigma'_v}\right)^m \le 1.7$$

$$m = 1.338-0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264 \le m \le 0.782$$

$$FC = 80\left(I_C+C_{FC}\right)-137 \quad with \quad 0\% \le FC \le 100\%$$

$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vc}}\right)^n \; with\; 0.5 \le n \le 1.0 \; per\; Robertson\,\&\,Wride\,(1998)$$

$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

σ'_v *at time of CPT sounding*

APPENDIX E.4 – AXIAL PILE ANALYSIS

Objective

Evaluate the axial geotechnical resistance of the abutment piles for the Bridges Bridge Replacement in Wilton, ME. Evaluations were conducted to assess a suitable driving system to install HP14x89 piles to the required geotechnical nominal resistance of 554 kips to support the abutments.

Methodology

Evaluate proposed pile section for governing factored axial compression resistance as follows.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 9th edition. (AASHTO LRFD)

Soil Properties

Consider Bridges Bridge Interpretive Subsurface Profile (see Figure 2), subsurface layering and properties relative to pile design are presented in the Apile outputs on pages 7-12.

Structural Properties

HP14x89, ASTM A572, Gr. 50

Yield Strength of Steel $F_y := 50\text{ksi}$

Area of section $A_s := 26.1\text{in}^2$

Young's Modulus of Steel $E_s := 30000 \cdot \text{ksi}$

Radius of gyration (weak axis) $r_x := 5.88\text{in}$

1. Nominal Structural Compressive Resistance P_n

Nominal Compressive Resistance: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ AASHTO Eq. 6.9.5.1-1

Determine normalized column slenderness factor λ

$$\lambda := \left(\frac{K \cdot l}{r_s \cdot \pi}\right)^2 \cdot \frac{F_y}{E}$$ AASHTO Eq. 6.9.4.1-3 pg. 6-74

$\lambda := 0$ Where the pile is fully embedded, AASHTO 10.7.3.13.1.

Giving: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = 1305 \cdot kip$

2. Factored Structural Compressive Resistance - Strength Limit State:

Factor for piles in compression under hard driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance for Strength Limit State:

$P_r := \phi_c \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

$P_r = 653 \cdot kip$

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State:

$P_r := \phi \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

$P_r = 1305 \cdot kip$

4. Geotechnical Axial Resistance - Static Analysis

AASHTO Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state or by the drivability of the pile.

Required nominal resistance of 554 kips based on hard driving conditions, a maximum factored pile load of 360 kips, and a 0.65 resistance factor.

The estimated % skin friction is 10% for the piles at the required nominal resistance, based on the estimated side resistance from Apile.

5. Geotechnical Axial Resistance - Drivability Analysis

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ AASHTO Eq. 10.7.8.1

$f_y := 50ksi$ yield Strength of steel

$\phi_{da} := 1.0$ AASHTO Table 10.5.5.2.3-1 Refers to Article 6.5.4.2, Pg. 6-28

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ Driving Stress in pile cannot exceed 45 ksi

Abutment piles - Drive plumb through 24 feet of soil to rock with a toe quake representative of tip on rock (0.04 in) and no plug. Model pile length 29 feet (5 feet of stick-up).

Drive piles with a Delmag D19-32 open-ended diesel hammer with a rated energy of 42,440 ft-lb (fuel setting 2, 1 below maximum).

GRLWEAP Output is attached on Sheets 5 through 6.

$R_{ndr1} := 554kip$ Required nominal geotechnical resistance, pile driving stress=38 ksi, final penetration resistance=8 bpi.

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

PDA, WEAP and CAPWAP used to establishing driving criteria $\phi_{dyn} := 0.65$ AASHTO Table 10.5.5.2.3-1

$$R_{ndr1\_factored} := R_{ndr1} \cdot \phi_{dyn}$$

$$R_{ndr1\_factored} = 360 \cdot kip$$

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States: $\phi_{serv\_ext} := 1$

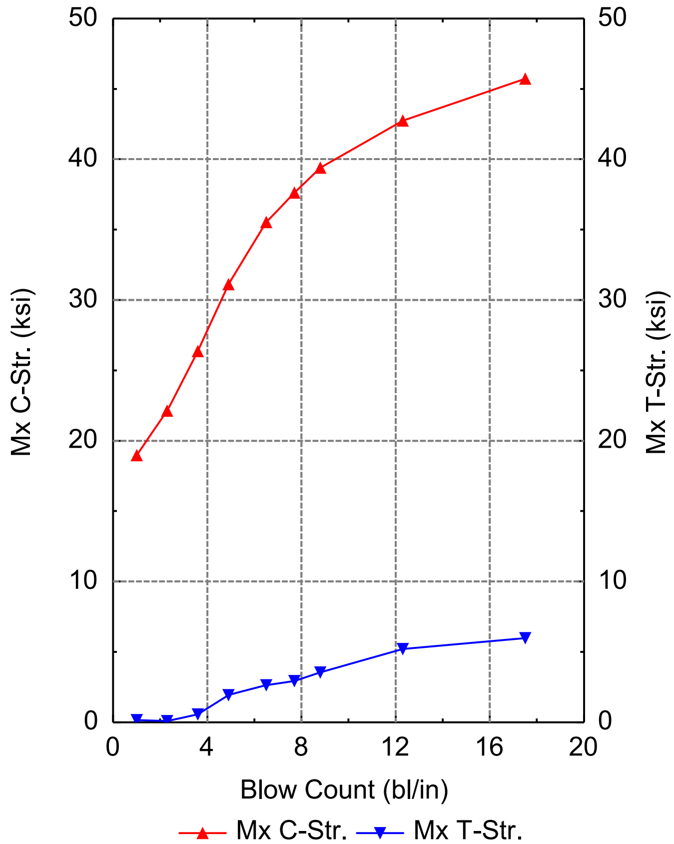
From Article 10.5.5.1 and 10.5.5.3

$$R_{ndr1\_serv\_ext} := R_{ndr1} \cdot \phi_{serv\_ext}$$

$$R_{ndr1\_serv\_ext} = 554 \cdot kip$$

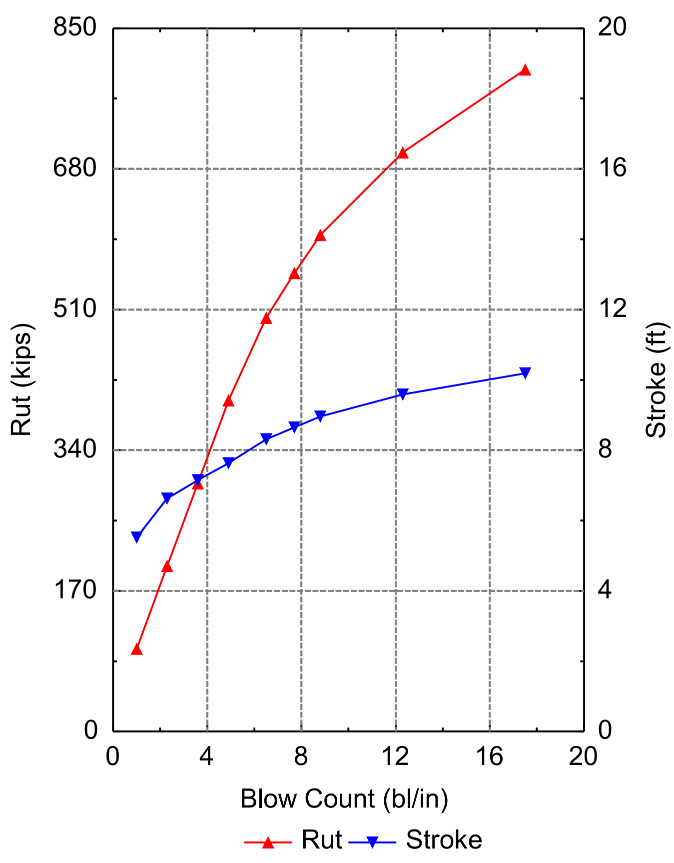
Summary of Results - Axial Loading:

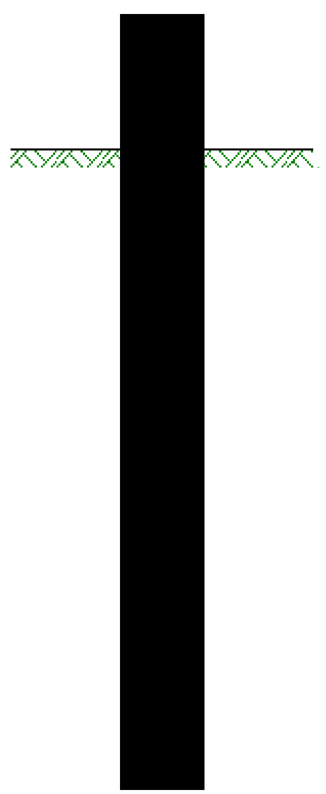
Results indicate that the piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 42,440 ft-lbs for the anticipated 24-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel), and with a final penetration resistance of 8 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch. Therefore, the analyzed hammer system is judged to be acceptable to install the piles to the required nominal resistance.



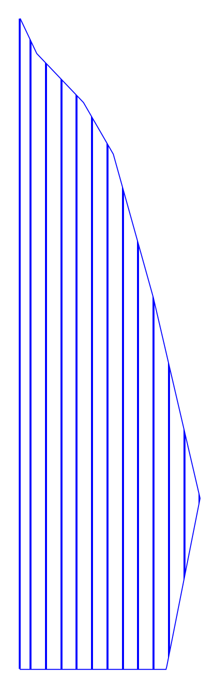
DELMAG D 19-32

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1420.0 (90%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	29.00	ft
Pile Penetration	24.00	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model



Shaft=10%
(Prop.)

Bearing Graph Summary — DELMAG D 19-32

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/in	ft	kip-ft	DELMAG	%
100.0	18.98	18.89	0.16	1.0	5.52	16.13	D 19-32	38.0
200.0	22.13	22.02	0.08	2.3	6.63	15.03	D 19-32	35.4
300.0	26.37	23.37	0.55	3.6	7.15	14.86	D 19-32	35.0
400.0	31.11	24.55	1.94	4.9	7.63	15.48	D 19-32	36.5
500.0	35.52	28.71	2.64	6.6	8.31	16.42	D 19-32	38.7
554.0	37.61	31.58	2.92	7.7	8.65	16.92	D 19-32	39.9
600.0	39.38	33.82	3.53	8.8	8.96	17.40	D 19-32	41.0
700.0	42.73	38.12	5.19	12.3	9.59	18.56	D 19-32	43.7
800.0	45.71	41.81	5.97	17.5	10.19	19.95	D 19-32	47.0

```
=========================================================================

                 APILE for Windows, Version 2023.10.5

                  Serial Number : 653550831

                  A Program for Analyzing the Axial Capacity
                  and Short-term Settlement of Driven Piles
                            under Axial Loading.
                (c) Copyright ENSOFT, Inc., 1987-2023
                      All Rights Reserved

=========================================================================


This program is licensed to :

GZA GeoEnvironmental, Inc.
Portland, OR

     Path to file locations      : P:\09 Jobs\0026100s\09.0026188.00 - MEDOT - Wilton Bridge
Replacement\09.0026188.01\Work\Calcs\Apile\
     Name of input data file     : Apile 14x89.ap10d
     Name of output file         : Apile 14x89.ap10o
     Name of plot output file    : Apile 14x89.ap10p

-------------------------------------------------------------------------
                     Time and Date of Analysis
-------------------------------------------------------------------------

          Date:  July 03, 2024     Time:  09:46:00


1
     *********************
     * INPUT INFORMATION *
     *********************

     PROJECT DESCRIPTION   :
     14x89
     DESIGNER   : E. Tome
     JOB NUMBER : 09.0026188.00
```

```
     METHOD FOR UNIT LOAD TRANSFERS :
     - USACE (U.S. Army Corps of Engineers)
       Unfactored Unit Side Friction and Unit Side Resistance are used.

     COMPUTATION METHOD(S) FOR PILE CAPACITY :
     - USACE (U.S. Army Corps of Engineers)
       # Critical Depth Method for Sand:
         10 to 20 Pile Diameter based on the Density

     TYPE OF LOADING :
     - COMPRESSION

     PILE TYPE :
     H-Pile/Steel Pile

      AVERAGE DEPTH TO ESTIMATE TIP RESISTANCE:
     - USE 1.5 DIAMETERS ABOVE AND BELOW TIP

     DATA FOR AXIAL STIFFNESS :
     - MODULUS OF ELASTICITY     = 0.290E+08 PSI
     - CROSS SECTION AREA        =    202.86 IN2

     NONCIRCULAR PILE PROPERTIES :
     - TOTAL PILE LENGTH, TL     =     23.90 FT.
     - BATTER ANGLE              =      0.00 DEG
     - PILE STICKUP LENGTH, PSL  =      0.00 FT.
     - ZERO FRICTION LENGTH, ZFL =      0.00 FT.
     - PERIMETER OF PILE         =     57.00 IN.
     - TIP AREA OF PILE          =    202.86 IN2
     - INCREMENT OF PILE LENGTH
       USED IN COMPUTATION       =      0.50 FT.
     - PRINTING INCREMENT        =         1


     SOIL INFORMATIONS :

                     LATERAL    EFFECTIVE    FRICTION      Nq
              SOIL   EARTH      UNIT         ANGLE        FACTOR
      DEPTH   TYPE   PRESSURE   WEIGHT       DEGREES      ARMY C.
       FT.                      LB/FT^3
        0.00  SAND      1.00      125.00        32.00     30.00**
        2.60  SAND      1.00      125.00        32.00     30.00**
```

2.60	SAND	1.00	62.60	32.00	30.00**
3.60	SAND	1.00	62.60	32.00	30.00**
3.60	SAND	1.00	62.60	32.00	30.00**
6.40	SAND	1.00	62.60	32.00	30.00**
6.40	SAND	1.00	57.60	30.00	23.00**
14.20	SAND	1.00	57.60	30.00	23.00**
14.20	CLAY	1.00*	47.60	0.00	1.00**
21.20	CLAY	1.00*	47.60	0.00	1.00**
21.20	SAND	1.00	57.60	30.00	23.00**
27.90	SAND	1.00	57.60	30.00	23.00**

* VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.50	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.50	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
WERE SET TO LARGE VALUES INDICATING THAT APILE
USES THE LIMITS SPECIFIED BY EACH SELECTED
CRITERIA (IF ANY).

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
2.60	1.000	1.000
2.60	1.000	1.000
3.60	1.000	1.000
3.60	1.000	1.000
6.40	1.000	1.000
6.40	1.000	1.000
14.20	1.000	1.000
14.20	1.000	1.000
21.20	1.000	1.000
21.20	1.000	1.000
27.90	1.000	1.000

1

****
* COMPUTATION RESULT *
****

****
* ARMY CORPS METHOD *
****

PILE LENGTH BELOW GND. FT.	SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	3.0	3.0
0.50	0.0	4.5	4.5
1.00	0.2	6.2	6.4
1.50	0.4	8.1	8.5
2.00	0.6	10.1	10.8
2.50	1.0	12.3	13.3
3.00	1.4	14.4	15.8
3.50	1.9	16.3	18.2
4.00	2.4	18.0	20.5
4.50	3.0	19.3	22.3
5.00	3.6	20.1	23.7
5.50	4.3	20.7	25.0
6.00	5.0	21.2	26.2

6.50	5.7	21.7	27.4
7.00	6.4	22.2	28.6
7.50	7.2	22.6	29.8
8.00	8.0	23.0	31.0
8.50	8.8	23.3	32.1
9.00	9.6	23.9	33.5
9.50	10.5	24.8	35.4
10.00	11.4	25.8	37.2
10.50	12.4	26.7	39.1
11.00	13.4	27.6	41.0
11.50	14.4	28.6	43.0
12.00	15.4	29.5	44.9
12.50	16.5	28.8	45.3
13.00	17.6	26.5	44.2
13.50	18.8	24.2	43.0
14.00	20.0	21.8	41.7
14.50	21.2	19.2	40.4
15.00	22.4	16.5	38.9
15.50	23.6	13.8	37.4
16.00	24.8	10.9	35.7
16.50	26.0	8.0	33.9
17.00	27.1	6.3	33.5
17.50	28.3	6.3	34.7
18.00	29.5	6.3	35.9
18.50	30.7	6.3	37.0
19.00	31.9	6.3	38.2
19.50	33.1	8.0	41.1
20.00	34.3	11.2	45.5
20.50	35.5	14.4	49.8
21.00	36.6	17.6	54.2
21.50	37.8	20.7	58.6
22.00	39.1	23.9	63.0
22.50	40.3	27.1	67.4
23.00	41.6	30.3	71.9
23.50	42.9	33.4	76.3

NOTES:
- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
  IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
  OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

```
*************************************************
* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING                      *
*************************************************
```

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.4167E-01		
			0.0000E+00	0.0000E+00
			0.1871E-01	0.1000E-01
			0.1897E-01	0.2000E-01
			0.1910E-01	0.4000E-01
			0.1914E-01	0.6000E-01
			0.1917E-01	0.8000E-01
			0.1919E-01	0.1200E+00
			0.1920E-01	0.1600E+00
			0.1922E-01	0.5000E+00
			0.1923E-01	0.1000E+02
2	10	0.1300E+01		
			0.0000E+00	0.0000E+00
			0.3219E+00	0.1000E-01
			0.4190E+00	0.2000E-01
			0.4934E+00	0.4000E-01
			0.5245E+00	0.6000E-01
			0.5415E+00	0.8000E-01
			0.5597E+00	0.1200E+00
			0.5693E+00	0.1600E+00
			0.5898E+00	0.5000E+00
			0.5995E+00	0.1000E+02
3	10	0.2558E+01		
			0.0000E+00	0.0000E+00
			0.4373E+00	0.1000E-01
			0.6382E+00	0.2000E-01
			0.8286E+00	0.4000E-01
			0.9201E+00	0.6000E-01
			0.9738E+00	0.8000E-01
			0.1034E+01	0.1200E+00
			0.1067E+01	0.1600E+00
			0.1142E+01	0.5000E+00
			0.1179E+01	0.1000E+02

4	10	0.2642E+01		
			0.0000E+00	0.0000E+00
			0.4424E+00	0.1000E-01
			0.6493E+00	0.2000E-01
			0.8473E+00	0.4000E-01
			0.9433E+00	0.6000E-01
			0.9998E+00	0.8000E-01
			0.1064E+01	0.1200E+00
			0.1099E+01	0.1600E+00
			0.1178E+01	0.5000E+00
			0.1217E+01	0.1000E+02
5	10	0.3100E+01		
			0.0000E+00	0.0000E+00
			0.4650E+00	0.1000E-01
			0.6991E+00	0.2000E-01
			0.9343E+00	0.4000E-01
			0.1052E+01	0.6000E-01
			0.1123E+01	0.8000E-01
			0.1204E+01	0.1200E+00
			0.1249E+01	0.1600E+00
			0.1353E+01	0.5000E+00
			0.1405E+01	0.1000E+02
6	10	0.3558E+01		
			0.0000E+00	0.0000E+00
			0.4760E+00	0.1000E-01
			0.7243E+00	0.2000E-01
			0.9798E+00	0.4000E-01
			0.1110E+01	0.6000E-01
			0.1190E+01	0.8000E-01
			0.1281E+01	0.1200E+00
			0.1332E+01	0.1600E+00
			0.1450E+01	0.5000E+00
			0.1510E+01	0.1000E+02
7	10	0.3642E+01		
			0.0000E+00	0.0000E+00
			0.4779E+00	0.1000E-01
			0.7287E+00	0.2000E-01
			0.9878E+00	0.4000E-01
			0.1121E+01	0.6000E-01
			0.1201E+01	0.8000E-01
			0.1295E+01	0.1200E+00
			0.1347E+01	0.1600E+00
			0.1468E+01	0.5000E+00
			0.1530E+01	0.1000E+02
8	10	0.5000E+01		
			0.0000E+00	0.0000E+00
			0.5047E+00	0.1000E-01
			0.7927E+00	0.2000E-01
			0.1109E+01	0.4000E-01
			0.1280E+01	0.6000E-01
			0.1386E+01	0.8000E-01
			0.1512E+01	0.1200E+00
			0.1584E+01	0.1600E+00
			0.1754E+01	0.5000E+00
			0.1842E+01	0.1000E+02
9	10	0.6358E+01		
			0.0000E+00	0.0000E+00
			0.5256E+00	0.1000E-01
			0.8455E+00	0.2000E-01
			0.1215E+01	0.4000E-01
			0.1423E+01	0.6000E-01
			0.1556E+01	0.8000E-01
			0.1716E+01	0.1200E+00
			0.1809E+01	0.1600E+00
			0.2034E+01	0.5000E+00
			0.2154E+01	0.1000E+02
10	10	0.6442E+01		
			0.0000E+00	0.0000E+00
			0.4716E+00	0.1000E-01
			0.7755E+00	0.2000E-01
			0.1144E+01	0.4000E-01
			0.1359E+01	0.6000E-01
			0.1501E+01	0.8000E-01
			0.1675E+01	0.1200E+00
			0.1778E+01	0.1600E+00
			0.2033E+01	0.5000E+00
			0.2172E+01	0.1000E+02
11	10	0.1030E+02		
			0.0000E+00	0.0000E+00
			0.4949E+00	0.1000E-01
			0.8404E+00	0.2000E-01
			0.1291E+01	0.4000E-01
			0.1572E+01	0.6000E-01
			0.1764E+01	0.8000E-01
			0.2009E+01	0.1200E+00
			0.2160E+01	0.1600E+00

			0.2548E+01	0.5000E+00
			0.2771E+01	0.1000E+02
12	10	0.1416E+02		
			0.0000E+00	0.0000E+00
			0.5145E+00	0.1000E-01
			0.8986E+00	0.2000E-01
			0.1434E+01	0.4000E-01
			0.1789E+01	0.6000E-01
			0.2042E+01	0.8000E-01
			0.2378E+01	0.1200E+00
			0.2591E+01	0.1600E+00
			0.3172E+01	0.5000E+00
			0.3524E+01	0.1000E+02
13	10	0.1424E+02		
			0.0000E+00	0.0000E+00
			0.6411E+00	0.1000E-01
			0.1353E+01	0.2000E-01
			0.2814E+01	0.4000E-01
			0.3455E+01	0.6000E-01
			0.3562E+01	0.8000E-01
			0.3455E+01	0.1200E+00
			0.3312E+01	0.1600E+00
			0.3312E+01	0.5000E+00
			0.3312E+01	0.1000E+02
14	10	0.1770E+02		
			0.0000E+00	0.0000E+00
			0.6250E+00	0.1000E-01
			0.1319E+01	0.2000E-01
			0.2743E+01	0.4000E-01
			0.3368E+01	0.6000E-01
			0.3472E+01	0.8000E-01
			0.3368E+01	0.1200E+00
			0.3229E+01	0.1600E+00
			0.3229E+01	0.5000E+00
			0.3229E+01	0.1000E+02
15	10	0.2116E+02		
			0.0000E+00	0.0000E+00
			0.6250E+00	0.1000E-01
			0.1319E+01	0.2000E-01
			0.2743E+01	0.4000E-01
			0.3368E+01	0.6000E-01
			0.3472E+01	0.8000E-01
			0.3368E+01	0.1200E+00

			0.3229E+01	0.1600E+00
			0.3229E+01	0.5000E+00
			0.3229E+01	0.1000E+02
16	10	0.2124E+02		
			0.0000E+00	0.0000E+00
			0.5129E+00	0.1000E-01
			0.8938E+00	0.2000E-01
			0.1422E+01	0.4000E-01
			0.1770E+01	0.6000E-01
			0.2017E+01	0.8000E-01
			0.2345E+01	0.1200E+00
			0.2552E+01	0.1600E+00
			0.3113E+01	0.5000E+00
			0.3452E+01	0.1000E+02
17	10	0.2455E+02		
			0.0000E+00	0.0000E+00
			0.5179E+00	0.1000E-01
			0.9090E+00	0.2000E-01
			0.1461E+01	0.4000E-01
			0.1831E+01	0.6000E-01
			0.2097E+01	0.8000E-01
			0.2453E+01	0.1200E+00
			0.2680E+01	0.1600E+00
			0.3306E+01	0.5000E+00
			0.3691E+01	0.1000E+02
18	10	0.2786E+02		
			0.0000E+00	0.0000E+00
			0.5179E+00	0.1000E-01
			0.9090E+00	0.2000E-01
			0.1461E+01	0.4000E-01
			0.1831E+01	0.6000E-01
			0.2097E+01	0.8000E-01
			0.2453E+01	0.1200E+00
			0.2680E+01	0.1600E+00
			0.3306E+01	0.5000E+00
			0.3691E+01	0.1000E+02

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00

0.3510E+00	0.1000E-03
0.2482E+01	0.5000E-02
0.3510E+01	0.1000E-01
0.7849E+01	0.5000E-01
0.1110E+02	0.1000E+00
0.1570E+02	0.2000E+00
0.2482E+02	0.5000E+00
0.3510E+02	0.1000E+01
0.4964E+02	0.2000E+01

LOAD VERSUS SETTLEMENT CURVE
****

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.4317E+00	0.1192E-03	0.3510E+00	0.1000E-03
0.1503E+01	0.1056E-02	0.7425E+00	0.1000E-02
0.6267E+01	0.5220E-02	0.2482E+01	0.5000E-02
0.1102E+02	0.1037E-01	0.3510E+01	0.1000E-01
0.1843E+02	0.2061E-01	0.4595E+01	0.2000E-01
0.3532E+02	0.5119E-01	0.7849E+01	0.5000E-01
0.4233E+02	0.8144E-01	0.9800E+01	0.8000E-01
0.4460E+02	0.1015E+00	0.1110E+02	0.1000E+00
0.5154E+02	0.2018E+00	0.1570E+02	0.2000E+00
0.6454E+02	0.5024E+00	0.2482E+02	0.5000E+00
0.7080E+02	0.8027E+00	0.3099E+02	0.8000E+00
0.7497E+02	0.1003E+01	0.3510E+02	0.1000E+01
0.8981E+02	0.2004E+01	0.4964E+02	0.2000E+01

APPENDIX E.5 – LATERAL PILE EVALUATIONS

LPile Input Parameters
MaineDOT - Bridges Bridge No. 2102
Wilton, Maine

GZA FILE NO. 09.0026188.01
CALCULATED BY E. Tome 4/5/2024
CHECKED BY B. Cardali 4/5/2024

Objective: To estimate the horizontal modulus of subgrade reaction (k) or E50 of subsurface strata for use in lateral analyses. K values are estimated using strata internal friction angles (ϕ') or shear strength.

Methods Correlations between the horizontal modulus of subgrade reaction and the soil internal friction angle of a given stratum are based on Figure 3-34 presented in the 2022 LPile Technical Manual.

Given Information: SPT measurements, In-situ vanes, and subsurface conditions in borings BB-WWS-101 and -102 were performed by New England Borings Contractors between May 3 and May 4, 2023. Boring BB-WWS-201 and Cone Penetration Tests sCPT-WWS-201 and -202 were performed by Seaboard Drilling, Inc. between April 22 and 23, 2024.

Abutment 1: Expansion, Pile length = 23.9'					
Stratum	**Soil Model**	**Top of Layer Elevation (NAVD88 ft)**	**k (pci) / E50**	**ϕ' (deg)/ Su (psf)**	**γ_e (pcf)**
New Fill	Reese Sand	578.2	83	32	125
New Fill**	Reese Sand	572.6	55	32	62.6
Existing Fill	Reese Sand	571.6	55	32	62.6
Alluvium	Reese Sand	568.8	35	30	57.6
Clay	Stiff Clay w/o Free Water	561	0.008	500	47.6
Alluvium	Reese Sand	554	35	30	57.6
Top of Rock	--	547.3	--	--	--

Abutment 1: Contraction, Pile length = 23.9'					
Stratum	**Soil Model**	**Top of Layer Elevation (NAVD88 ft)**	**k (pci) / E50**	**ϕ' (deg)/ Su (psf)**	**γ_e (pcf)**
New Fill	Reese Sand	575.2	83	32	125
New Fill**	Reese Sand	572.6	55	32	62.6
Existing Fill	Reese Sand	571.6	55	32	62.6
Alluvium	Reese Sand	568.8	35	30	57.6
Clay	Stiff Clay w/o Free Water	561	0.008	500	47.6
Alluvium	Reese Sand	554	35	30	57.6
Top of Rock	--	547.3	--	--	--

Notes: 1. Pile tip elevation should be assumed to be top of Rock.

2. ** indicates the top of layer is the approximate ground water elevation based on the boring logs.
3. pci = pounds per cubic inch, deg = degrees, psi =pounds per square inch, γe = effective unit weight, pcf = pounds per square foot.
4. The soil profiles and pile lengths for each abutment are similar. Therefore, GZA evaluated Abutment 1 considering the slightly shorter pile length.
5. These parameters do not include reductions for group interaction. Reduction factors should be applied in accordance with AASHTO 10.7.2.4 for spacing of 3 to 5 pile diameters or less.

Table 2 - LPile Output Summary
MaineDOT - Bridges Bridge No. 2102
GZA GeoEnvironmental, Inc.

GZA FILE NO.	09.0026188.01
CALCULATED BY	E. Tome 4/5/2024
CHECKED BY	B. Cardali 4/5/2024

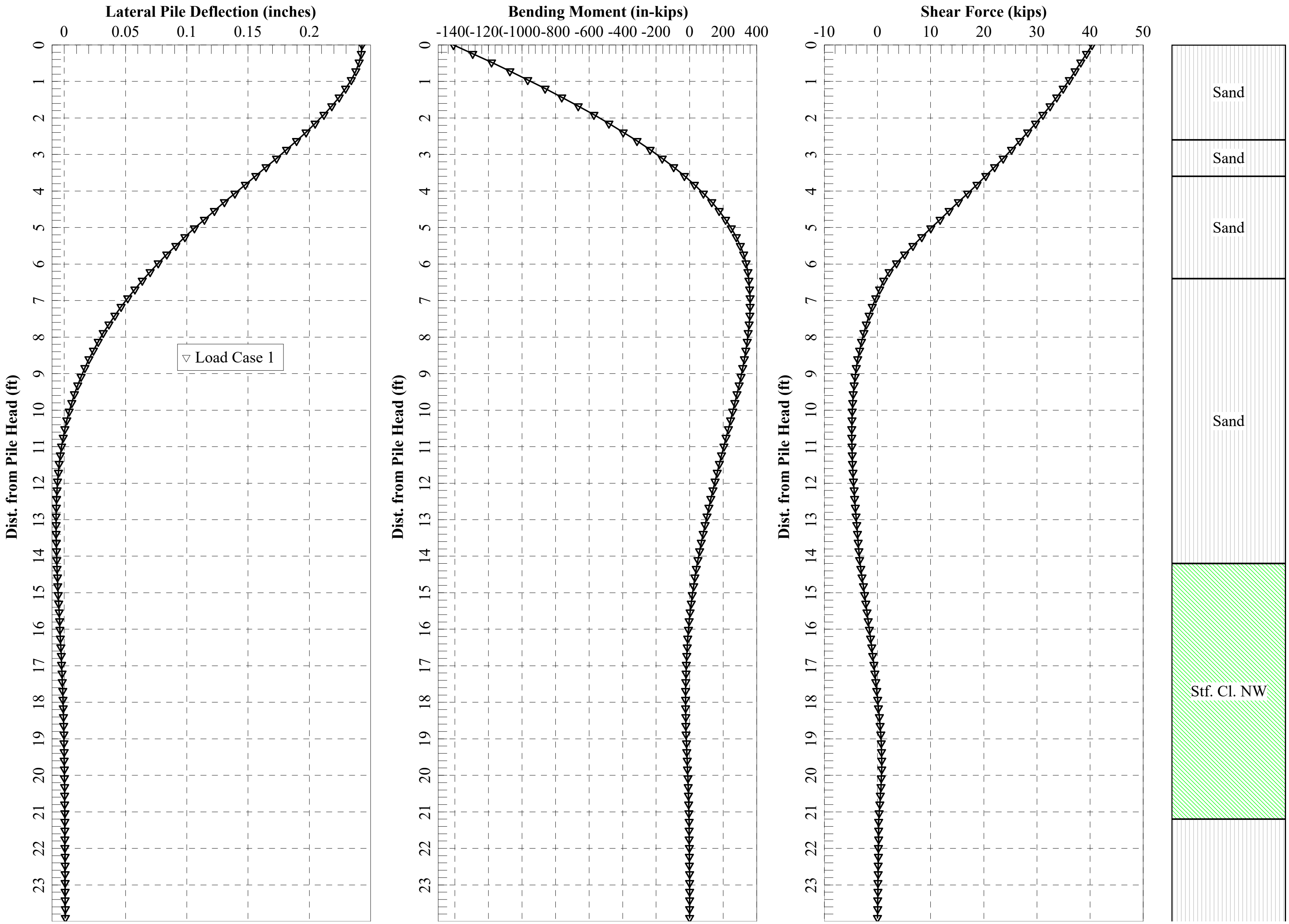
Abutment 1: Expansion, Pile Length = 23.9 ft								
Pile Section	**Axial Load (kips)**	**Shear Force for Lateral deflection of 0.243 in. (kips)**	**Moment at Pile Head (in-lbs)**	**Moment at Pile Head (kip-ft)**	**Depth below Pile Head to Fixity (ft)**	**Total Stress at Pile Head (ksi)**	**Bending Stress at Pile Head (ksi)**	**Axial Stress at Pile Head (ksi)**
HP 14x89 (Weak axis)	360	40.4	-1408675	-117.4	17.9	45.7	31.9	13.8

Abutment 1: Contraction, Pile Length = 23.9 ft								
Pile Section	**Axial Load (kips)**	**Shear Force for Lateral deflection of 0.366 in. (kips)**	**Moment at Pile Head (in-lbs)**	**Moment at Pile Head (kip-ft)**	**Depth below Pile Head to Fixity (ft)**	**Total Stress at Pile Head (ksi)**	**Bending Stress at Pile Head (ksi)**	**Axial Stress at Pile Head (ksi)**
HP 14x89 (Weak axis)	360	33.3	-1482408	-123.5	19.5	47.4	33.6	13.8

Notes:

1. Soil layering and properties are presented in Table 1.
2. The axial load is the maximum Factored Axial Load provided by HNTB.
3. For the expansion evaluations the Lpile model included imposed lateral deflection of 0.243 inches (provided by HNTB) and zero rotation applied at the pile head.
4. For the contraction evaluations the Lpile model included imposed lateral deflection of 0.366 inches (provided by HNTB) and zero rotation applied at the pile head.
5. The axial stress at pile head is calculated as the axial load divided by the pile cross-sectional area.

Abutment 1: HP14x89
Weak Axis - Expansion



```
===============================================================================
                     LPile for Windows, Version 2022-12.011

                 Analysis of Individual Piles and Drilled Shafts
                Subjected to Lateral Loading Using the p-y Method
                           © 1985-2022 by Ensoft, Inc.
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===============================================================================

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-------------------------------------------------------------------------------
                             Files Used for Analysis
-------------------------------------------------------------------------------

Path to file locations:
\09 Jobs\0026100s\09.0026188.00 - MEDOT - Wilton Bridge Replacement\09.0026188.01\Work\Calcs\Lpile\Lpile Files\

Name of input data file:
AB1_14x89_Expansion.lp12d

Name of output report file:
AB1_14x89_Expansion.lp12o

Name of plot output file:
AB1_14x89_Expansion.lp12p

Name of runtime message file:
AB1_14x89_Expansion.lp12r


-------------------------------------------------------------------------------
                            Date and Time of Analysis
-------------------------------------------------------------------------------

               Date:  June 26, 2024              Time:   9:49:16


-------------------------------------------------------------------------------
                                  Problem Title
-------------------------------------------------------------------------------

Project Name: Wilton Bridges Bridge


Job Number: 09.0026188.01


Client: MEDOT


Engineer: E. Tome


Description:



-------------------------------------------------------------------------------
                           Program Options and Settings
-------------------------------------------------------------------------------

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)
```

```
Analysis Control Options:
 - Maximum number of iterations allowed                =          500
 - Deflection tolerance for convergence                =   1.0000E-05 in
 - Maximum allowable deflection                        =     100.0000 in
 - Number of pile increments                           =          100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats



--------------------------------------------------------------------------------
                     Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Number of pile sections defined                        =            1
Total length of pile                                   =       23.900 ft
Depth of ground surface below top of pile              =      -3.9000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile. A summary of values of pile diameter vs. depth follows.
```

```
           Depth Below          Pile
Point       Pile Head         Diameter
 No.          feet             inches
-----     -------------     -------------
  1           0.000           14.7000
  2          23.900           14.7000


Input Structural Properties for Pile Sections:
----------------------------------------------

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                   =    23.900000 ft
   Pile width                                          =    13.800000 in


--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 6 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer           =     -3.90000 ft
   Distance from top of pile to bottom of layer        =     2.600000 ft
   Effective unit weight at top of layer               =   125.000000 pcf
   Effective unit weight at bottom of layer            =   125.000000 pcf
   Friction angle at top of layer                      =    32.000000 deg.
   Friction angle at bottom of layer                   =    32.000000 deg.
   Subgrade k at top of layer                          =    83.000000 pci
   Subgrade k at bottom of layer                       =    83.000000 pci


Layer 2 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer           =     2.600000 ft
   Distance from top of pile to bottom of layer        =     3.600000 ft
   Effective unit weight at top of layer               =    62.600000 pcf
```

```
   Effective unit weight at bottom of layer          =    62.600000 pcf
   Friction angle at top of layer                    =    32.000000 deg.
   Friction angle at bottom of layer                 =    32.000000 deg.
   Subgrade k at top of layer                        =    55.000000 pci
   Subgrade k at bottom of layer                     =    55.000000 pci


Layer 3 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =     3.600000 ft
   Distance from top of pile to bottom of layer      =     6.400000 ft
   Effective unit weight at top of layer             =    62.600000 pcf
   Effective unit weight at bottom of layer          =    62.600000 pcf
   Friction angle at top of layer                    =    32.000000 deg.
   Friction angle at bottom of layer                 =    32.000000 deg.
   Subgrade k at top of layer                        =    55.000000 pci
   Subgrade k at bottom of layer                     =    55.000000 pci


Layer 4 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =     6.400000 ft
   Distance from top of pile to bottom of layer      =    14.200000 ft
   Effective unit weight at top of layer             =    57.600000 pcf
   Effective unit weight at bottom of layer          =    57.600000 pcf
   Friction angle at top of layer                    =    30.000000 deg.
   Friction angle at bottom of layer                 =    30.000000 deg.
   Subgrade k at top of layer                        =    35.000000 pci
   Subgrade k at bottom of layer                     =    35.000000 pci


Layer 5 is stiff clay without free water

   Distance from top of pile to top of layer         =    14.200000 ft
   Distance from top of pile to bottom of layer      =    21.200000 ft
   Effective unit weight at top of layer             =    47.600000 pcf
   Effective unit weight at bottom of layer          =    47.600000 pcf
   Undrained cohesion at top of layer                =   500.000000 psf
   Undrained cohesion at bottom of layer             =   500.000000 psf
   Epsilon-50 at top of layer                        =     0.008000
   Epsilon-50 at bottom of layer                     =     0.008000


Layer 6 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =    21.200000 ft
   Distance from top of pile to bottom of layer      =    27.900000 ft
   Effective unit weight at top of layer             =    57.600000 pcf
   Effective unit weight at bottom of layer          =    57.600000 pcf
   Friction angle at top of layer                    =    30.000000 deg.
   Friction angle at bottom of layer                 =    30.000000 deg.
   Subgrade k at top of layer                        =    35.000000 pci
   Subgrade k at bottom of layer                     =    35.000000 pci

 (Depth of the lowest soil layer extends 4.000 ft below the pile tip)
```

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	-3.900	125.0000	--	32.0000	--	83.0000
	(Reese, et al.)	2.6000	125.0000	--	32.0000	--	83.0000
2	Sand	2.6000	62.6000	--	32.0000	--	55.0000
	(Reese, et al.)	3.6000	62.6000	--	32.0000	--	55.0000
3	Sand	3.6000	62.6000	--	32.0000	--	55.0000
	(Reese, et al.)	6.4000	62.6000	--	32.0000	--	55.0000
4	Sand	6.4000	57.6000	--	30.0000	--	35.0000
	(Reese, et al.)	14.2000	57.6000	--	30.0000	--	35.0000
5	Stiff Clay	14.2000	47.6000	500.0000	--	0.00800	--
	w/o Free Water	21.2000	47.6000	500.0000	--	0.00800	--
6	Sand	21.2000	57.6000	--	30.0000	--	35.0000
	(Reese, et al.)	27.9000	57.6000	--	30.0000	--	35.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.243000 in	S = 0.0000 in/in	360000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

```
Length of Section                                   =    23.900000 ft
Flange Width                                        =    14.700000 in
Section Depth                                       =    13.800000 in
Flange Thickness                                    =     0.615000 in
Web Thickness                                       =     0.615000 in
Yield Stress of Pipe                                =    50.000000 ksi
Elastic Modulus                                     =       29000. ksi
Cross-sectional Area                                =    25.811550 sq. in.
Moment of Inertia                                   =   325.837265 in^4
Elastic Bending Stiffness                           =     9449281. kip-in^2
Plastic Modulus, Z                                  =    67.636247in^3
Plastic Moment Capacity = Fy Z                      =        3382.in-kip
```

Axial Structural Capacities:

```
Nom. Axial Structural Capacity = Fy As              =     1290.578 kips
Nominal Axial Tensile Capacity                      =    -1290.578 kips
```

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	360.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 360.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000444	41.9374124	9448389.	115.7044043	14.8838666	
0.00000888	83.8748248	9448389.	61.5272029	15.8204887	
0.00001332	125.8122373	9448389.	43.4681348	16.7571105	
0.00001775	167.7496497	9448389.	34.4386011	17.6937326	
0.00002219	209.6870621	9448389.	29.0208809	18.6303549	
0.00002663	251.6244745	9448389.	25.4090674	19.5669769	
0.00003107	293.5618869	9448389.	22.8292006	20.5035990	

0.00003551	335.4992993	9448389.	20.8943005	21.4402210	
0.00003995	377.4367118	9448389.	19.3893783	22.3768431	
0.00004439	419.3741242	9448389.	18.1854404	23.3134651	
0.00004882	461.3115366	9448389.	17.2004004	24.2500873	
0.00005326	503.2489490	9448389.	16.3795337	25.1867094	
0.00005770	545.1863614	9448389.	15.6849542	26.1233314	
0.00006214	587.1237739	9448389.	15.0896003	27.0599535	
0.00006658	629.0611863	9448389.	14.5736270	27.9965755	
0.00007102	670.9985987	9448389.	14.1221503	28.9331977	
0.00007546	712.9360111	9448389.	13.7237885	29.8698197	
0.00007989	754.8734235	9448389.	13.3696891	30.8064417	
0.00008433	796.8108359	9448389.	13.0528634	31.7430638	
0.00008877	838.7482484	9448389.	12.7677202	32.6796859	
0.00009321	880.6856608	9448389.	12.5097335	33.6163080	
0.00009765	922.6230732	9448389.	12.2752002	34.5529301	
0.0001021	964.5604856	9448389.	12.0610611	35.4895521	
0.0001065	1006.	9448389.	11.8647668	36.4261741	
0.0001110	1048.	9448389.	11.6841762	37.3627963	
0.0001154	1090.	9448389.	11.5174771	38.2994183	
0.0001198	1132.	9448389.	11.3631261	39.2360404	
0.0001243	1174.	9448389.	11.2198002	40.1726624	
0.0001287	1216.	9448389.	11.0863588	41.1092845	
0.0001332	1258.	9448389.	10.9618135	42.0459066	
0.0001376	1300.	9448389.	10.8453034	42.9825287	
0.0001420	1342.	9448389.	10.7360751	43.9191507	
0.0001465	1384.	9448389.	10.6334668	44.8557728	
0.0001509	1426.	9448389.	10.5368942	45.7923949	
0.0001554	1468.	9448389.	10.4458401	46.7290169	
0.0001598	1510.	9448389.	10.3598446	47.6656390	
0.0001642	1552.	9448389.	10.2784974	48.6022611	
0.0001687	1594.	9448389.	10.2014317	49.5388831	
0.0001731	1635.	9444750.	10.1289863	50.0000000	Y
0.0001820	1713.	9413633.	9.9992637	50.0000000	Y
0.0001909	1786.	9356714.	9.8872130	50.0000000	Y
0.0001997	1854.	9282029.	9.7898027	50.0000000	Y
0.0002086	1918.	9194494.	9.7048009	50.0000000	Y
0.0002175	1979.	9098100.	9.6303188	50.0000000	Y
0.0002264	2036.	8994945.	9.5650217	50.0000000	Y
0.0002352	2091.	8888017.	9.5074665	50.0000000	Y
0.0002441	2143.	8778986.	9.4565910	50.0000000	Y
0.0002530	2193.	8668920.	9.4115629	50.0000000	Y
0.0002619	2241.	8558522.	9.3717055	50.0000000	Y
0.0002708	2288.	8449149.	9.3362109	50.0000000	Y
0.0002796	2332.	8341141.	9.3045903	50.0000000	Y
0.0002885	2376.	8234786.	9.2764128	50.0000000	Y
0.0002974	2418.	8130322.	9.2512965	50.0000000	Y
0.0003063	2459.	8027951.	9.2289014	50.0000000	Y
0.0003151	2498.	7927838.	9.2089234	50.0000000	Y
0.0003240	2536.	7826452.	9.1901020	50.0000000	Y
0.0003329	2571.	7722229.	9.1716604	50.0000000	Y
0.0003418	2603.	7616213.	9.1535363	50.0000000	Y
0.0003506	2633.	7509294.	9.1356714	50.0000000	Y
0.0003595	2661.	7401927.	9.1181404	50.0000000	Y
0.0003684	2687.	7294361.	9.1010928	50.0000000	Y
0.0003773	2712.	7187800.	9.0842155	50.0000000	Y
0.0003862	2735.	7082167.	9.0677298	50.0000000	Y
0.0003950	2756.	6977769.	9.0516791	50.0000000	Y
0.0004039	2777.	6875341.	9.0358222	50.0000000	Y
0.0004128	2796.	6773314.	9.0202493	50.0000000	Y
0.0004217	2814.	6673782.	9.0047922	50.0000000	Y
0.0004305	2831.	6575919.	8.9900453	50.0000000	Y
0.0004394	2847.	6479851.	8.9751447	50.0000000	Y
0.0004483	2863.	6385518.	8.9607041	50.0000000	Y
0.0004572	2877.	6293682.	8.9466590	50.0000000	Y
0.0004661	2891.	6203152.	8.9324190	50.0000000	Y
0.0004749	2904.	6114858.	8.9189260	50.0000000	Y
0.0004838	2917.	6028398.	8.9052627	50.0000000	Y
0.0004927	2928.	5943987.	8.8920604	50.0000000	Y
0.0005016	2940.	5861203.	8.8790726	50.0000000	Y
0.0005104	2951.	5780639.	8.8661964	50.0000000	Y
0.0005193	2961.	5701655.	8.8535386	50.0000000	Y
0.0005282	2971.	5624547.	8.8413214	50.0000000	Y
0.0005637	3006.	5333233.	8.7937380	50.0000000	Y
0.0005992	3036.	5067276.	8.7489644	50.0000000	Y
0.0006347	3062.	4824626.	8.7070032	50.0000000	Y
0.0006702	3085.	4602649.	8.6760298	50.0000000	Y
0.0007057	3104.	4398462.	8.6302531	50.0000000	Y
0.0007412	3121.	4210968.	8.5948757	50.0000000	Y
0.0007768	3137.	4038007.	8.5616612	50.0000000	Y
0.0008123	3150.	3878345.	8.5300009	50.0000000	Y
0.0008478	3162.	3730010.	8.5000691	50.0000000	Y
0.0008833	3173.	3592368.	8.4713113	50.0000000	Y
0.0009188	3183.	3464307.	8.4442310	50.0000000	Y
0.0009543	3192.	3344890.	8.4186520	50.0000000	Y
0.0009898	3200.	3232997.	8.3937145	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	360.0000000000	3200.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-3.900	0.00	N.A.	No	0.00	36064.
2	2.6000	6.4998	Yes	No	36064.	14549.
3	3.6000	7.6752	Yes	No	50613.	59024.
4	6.4000	11.4981	Yes	No	109637.	295118.
5	14.2000	76.9101	No	No	404755.	38587.
6	21.2000	21.6444	No	No	443343.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 0.243000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.2430	-1408675.	40365.	0.00	45723.	9.45E+09	-342.794	2023.	0.00
0.2390	0.2424	-1294183.	39328.	-4.10E-04	43140.	9.45E+09	-359.477	4253.	0.00
0.4780	0.2406	-1182242.	38278.	-7.86E-04	40615.	9.45E+09	-373.036	4446.	0.00
0.7170	0.2379	-1073000.	37193.	-0.00113	38151.	9.45E+09	-382.976	4617.	0.00
0.9560	0.2342	-966571.	36087.	-0.00144	35750.	9.45E+09	-388.798	4762.	0.00
1.1950	0.2296	-863037.	34942.	-0.00172	33415.	9.45E+09	-409.359	5113.	0.00
1.4340	0.2243	-762599.	33735.	-0.00196	31149.	9.45E+09	-432.836	5534.	0.00
1.6730	0.2184	-665483.	32461.	-0.00218	28959.	9.45E+09	-454.970	5975.	0.00
1.9120	0.2118	-571901.	31127.	-0.00237	26848.	9.45E+09	-475.517	6438.	0.00
2.1510	0.2048	-482051.	29736.	-0.00253	24821.	9.45E+09	-494.239	6921.	0.00
2.3900	0.1973	-396115.	28288.	-0.00266	22883.	9.45E+09	-515.868	7497.	0.00
2.6290	0.1895	-314298.	26778.	-0.00277	21037.	9.45E+09	-537.142	8128.	0.00
2.8680	0.1815	-236801.	25221.	-0.00285	19289.	9.45E+09	-548.418	8668.	0.00
3.1070	0.1732	-163741.	23634.	-0.00291	17641.	9.45E+09	-558.212	9244.	0.00
3.3460	0.1648	-95221.	22022.	-0.00295	16095.	9.45E+09	-566.475	9861.	0.00
3.5850	0.1563	-31331.	20387.	-0.00297	14654.	9.45E+09	-573.167	10520.	0.00
3.8240	0.1477	27855.	18701.	-0.00297	14576.	9.45E+09	-602.520	11698.	0.00
4.0630	0.1392	82076.	16969.	-0.00295	15799.	9.45E+09	-605.443	12473.	0.00
4.3020	0.1308	131291.	15231.	-0.00292	16909.	9.45E+09	-606.689	13306.	0.00
4.5410	0.1224	175475.	13492.	-0.00288	17905.	9.45E+09	-606.253	14200.	0.00

4.7800	0.1143	214617.	11756.	-0.00282	18788.	9.45E+09	-604.137	15162.	0.00
5.0190	0.1063	248723.	10029.	-0.00275	19558.	9.45E+09	-600.352	16199.	0.00
5.2580	0.09852	277813.	8315.	-0.00267	20214.	9.45E+09	-594.911	17318.	0.00
5.4970	0.09100	301922.	6652.	-0.00258	20758.	9.45E+09	-564.362	17787.	0.00
5.7360	0.08373	321294.	5079.	-0.00248	21195.	9.45E+09	-532.523	18240.	0.00
5.9750	0.07675	336185.	3598.	-0.00238	21531.	9.45E+09	-500.216	18692.	0.00
6.2140	0.07006	346857.	2210.	-0.00228	21771.	9.45E+09	-467.660	19145.	0.00
6.4530	0.06367	353573.	1143.	-0.00217	21923.	9.45E+09	-276.854	12471.	0.00
6.6920	0.05759	357901.	378.3954	-0.00207	22021.	9.45E+09	-256.192	12759.	0.00
6.9310	0.05182	360010.	-327.018	-0.00196	22068.	9.45E+09	-235.727	13047.	0.00
7.1700	0.04636	360067.	-974.166	-0.00185	22069.	9.45E+09	-215.561	13334.	0.00
7.4090	0.04122	358237.	-1564.	-0.00174	22028.	9.45E+09	-195.789	13622.	0.00
7.6480	0.03639	354686.	-2098.	-0.00163	21948.	9.45E+09	-176.497	13910.	0.00
7.8870	0.03187	349571.	-2577.	-0.00152	21833.	9.45E+09	-157.763	14198.	0.00
8.1260	0.02765	343049.	-3004.	-0.00142	21685.	9.45E+09	-139.659	14486.	0.00
8.3650	0.02373	335271.	-3379.	-0.00132	21510.	9.45E+09	-122.246	14774.	0.00
8.6040	0.02010	326382.	-3706.	-0.00122	21310.	9.45E+09	-105.580	15062.	0.00
8.8430	0.01676	316522.	-3986.	-0.00112	21087.	9.45E+09	-89.706	15350.	0.00
9.0820	0.01369	305826.	-4222.	-0.00102	20846.	9.45E+09	-74.663	15638.	0.00
9.3210	0.01089	294419.	-4416.	-9.32E-04	20589.	9.45E+09	-60.483	15925.	0.00
9.5600	0.00835	282423.	-4570.	-8.44E-04	20318.	9.45E+09	-47.190	16213.	0.00
9.7990	0.00605	269950.	-4688.	-7.61E-04	20037.	9.45E+09	-34.800	16501.	0.00
10.0380	0.00398	257106.	-4771.	-6.81E-04	19747.	9.45E+09	-23.324	16789.	0.00
10.2770	0.00214	243989.	-4823.	-6.05E-04	19451.	9.45E+09	-12.767	17077.	0.00
10.5160	5.16E-04	230691.	-4845.	-5.33E-04	19151.	9.45E+09	-3.126	17365.	0.00
10.7550	-9.11E-04	217296.	-4842.	-4.65E-04	18849.	9.45E+09	5.6049	17653.	0.00
10.9940	-0.00215	203878.	-4815.	-4.01E-04	18546.	9.45E+09	13.4394	17941.	0.00
11.2330	-0.00321	190507.	-4766.	-3.41E-04	18245.	9.45E+09	20.3942	18229.	0.00
11.4720	-0.00410	177244.	-4699.	-2.85E-04	17945.	9.45E+09	26.4912	18516.	0.00
11.7110	-0.00484	164143.	-4615.	-2.33E-04	17650.	9.45E+09	31.7560	18804.	0.00
11.9500	-0.00544	151252.	-4518.	-1.85E-04	17359.	9.45E+09	36.2181	19092.	0.00
12.1890	-0.00591	138611.	-4409.	-1.41E-04	17074.	9.45E+09	39.9104	19380.	0.00
12.4280	-0.00625	126256.	-4290.	-1.01E-04	16795.	9.45E+09	42.8687	19668.	0.00
12.6670	-0.00649	114213.	-4164.	-6.46E-05	16524.	9.45E+09	45.1314	19956.	0.00
12.9060	-0.00662	102506.	-4032.	-3.17E-05	16259.	9.45E+09	46.7394	20244.	0.00
13.1450	-0.00667	91151.	-3897.	-2.32E-06	16003.	9.45E+09	47.7358	20532.	0.00
13.3840	-0.00664	80160.	-3759.	2.37E-05	15755.	9.45E+09	48.1655	20820.	0.00
13.6230	-0.00653	69540.	-3621.	4.64E-05	15516.	9.45E+09	48.0750	21108.	0.00
13.8620	-0.00637	59294.	-3484.	6.60E-05	15285.	9.45E+09	47.5122	21395.	0.00
14.1010	-0.00615	49420.	-3349.	8.25E-05	15062.	9.45E+09	46.5263	21683.	0.00
14.3400	-0.00590	39914.	-3158.	9.60E-05	14848.	9.45E+09	86.4890	42071.	0.00
14.5790	-0.00560	31106.	-2912.	1.07E-04	14649.	9.45E+09	85.3991	43711.	0.00
14.8180	-0.00528	22991.	-2669.	1.15E-04	14466.	9.45E+09	84.1583	45684.	0.00
15.0570	-0.00494	15561.	-2429.	1.21E-04	14298.	9.45E+09	82.7763	48022.	0.00
15.2960	-0.00459	8807.	-2194.	1.25E-04	14146.	9.45E+09	81.2614	50773.	0.00
15.5350	-0.00423	2718.	-1963.	1.26E-04	14009.	9.45E+09	79.6203	53994.	0.00
15.7740	-0.00387	-2716.	-1738.	1.26E-04	14009.	9.45E+09	77.8587	57763.	0.00
16.0130	-0.00350	-7509.	-1517.	1.25E-04	14117.	9.45E+09	75.9810	62177.	0.00
16.2520	-0.00315	-11675.	-1302.	1.22E-04	14211.	9.45E+09	73.9904	67361.	0.00
16.4910	-0.00281	-15228.	-1093.	1.18E-04	14291.	9.45E+09	71.8890	73480.	0.00
16.7300	-0.00247	-18186.	-889.687	1.13E-04	14357.	9.45E+09	69.6776	80747.	0.00
16.9690	-0.00216	-20564.	-693.182	1.07E-04	14411.	9.45E+09	67.3551	89450.	0.00
17.2080	-0.00186	-22382.	-503.502	1.00E-04	14452.	9.45E+09	64.9186	99980.	0.00
17.4470	-0.00158	-23659.	-320.981	9.33E-05	14481.	9.45E+09	62.3622	112887.	0.00
17.6860	-0.00133	-24416.	-145.978	8.60E-05	14498.	9.45E+09	59.6758	128965.	0.00
17.9250	-0.00109	-24674.	21.1095	7.85E-05	14504.	9.45E+09	56.8430	149414.	0.00
18.1640	-8.77E-04	-24457.	179.8246	7.11E-05	14499.	9.45E+09	53.8370	176146.	0.00
18.4030	-6.83E-04	-23790.	329.6071	6.38E-05	14484.	9.45E+09	50.6138	212430.	0.00
18.6420	-5.11E-04	-22698.	469.7241	5.67E-05	14459.	9.45E+09	47.0968	264432.	0.00
18.8810	-3.58E-04	-21212.	599.1268	5.00E-05	14426.	9.45E+09	43.1422	345581.	0.00
19.1200	-2.24E-04	-19365.	705.5670	4.39E-05	14384.	9.45E+09	31.0838	398446.	0.00
19.3590	-1.06E-04	-17256.	771.3185	3.83E-05	14336.	9.45E+09	14.7680	398446.	0.00
19.5980	-3.88E-06	-15020.	793.2688	3.34E-05	14286.	9.45E+09	0.5391	398446.	0.00
19.8370	8.55E-05	-12775.	777.0156	2.92E-05	14235.	9.45E+09	-11.873	398446.	0.00
20.0760	1.64E-04	-10623.	727.3796	2.57E-05	14187.	9.45E+09	-22.740	398446.	0.00
20.3150	2.33E-04	-8655.	648.4187	2.27E-05	14142.	9.45E+09	-32.323	398446.	0.00
20.5540	2.94E-04	-6951.	543.5520	2.04E-05	14104.	9.45E+09	-40.806	397932.	0.00
20.7930	3.49E-04	-5580.	423.8971	1.85E-05	14073.	9.45E+09	-42.635	349881.	0.00
21.0320	4.00E-04	-4557.	299.4871	1.69E-05	14050.	9.45E+09	-44.122	316340.	0.00
21.2710	4.47E-04	-3897.	229.4462	1.56E-05	14035.	9.45E+09	-4.721	30320.	0.00
21.5100	4.90E-04	-3274.	215.1809	1.46E-05	14021.	9.45E+09	-5.227	30608.	0.00
21.7490	5.30E-04	-2692.	199.4973	1.37E-05	14008.	9.45E+09	-5.710	30896.	0.00
21.9880	5.68E-04	-2157.	182.4520	1.29E-05	13996.	9.45E+09	-6.176	31184.	0.00
22.2270	6.04E-04	-1673.	164.0883	1.23E-05	13985.	9.45E+09	-6.630	31472.	0.00
22.4660	6.39E-04	-1242.	144.4378	1.19E-05	13975.	9.45E+09	-7.074	31759.	0.00
22.7050	6.72E-04	-868.667	123.5205	1.16E-05	13967.	9.45E+09	-7.513	32047.	0.00
22.9440	7.05E-04	-557.118	101.3463	1.14E-05	13960.	9.45E+09	-7.950	32335.	0.00
23.1830	7.37E-04	-310.790	77.9162	1.12E-05	13954.	9.45E+09	-8.389	32623.	0.00
23.4220	7.70E-04	-133.364	53.2239	1.12E-05	13950.	9.45E+09	-8.830	32911.	0.00
23.6610	8.01E-04	-28.531	27.2572	1.11E-05	13948.	9.45E+09	-9.277	33199.	0.00
23.9000	8.33E-04	0.00	0.00	1.11E-05	13947.	9.45E+09	-9.730	16743.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be inter-

```
  polated from the output for nonlinear bending properties relative to the
  magnitude of bending moment developed in the pile.


Output Summary for Load Case No. 1:

Pile-head deflection             =     0.24300000 inches
Computed slope at pile head      =       0.000000 radians
Maximum bending moment           =      -1408675. inch-lbs
Maximum shear force              =         40365. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =       0.000000 feet below pile head
Number of iterations             =            11
Number of zero deflection points =             2


--------------------------------------------------------------------------------
            Summary of Pile-head Responses for Conventional Analyses
--------------------------------------------------------------------------------

Definitions of Pile-head Loading Conditions:

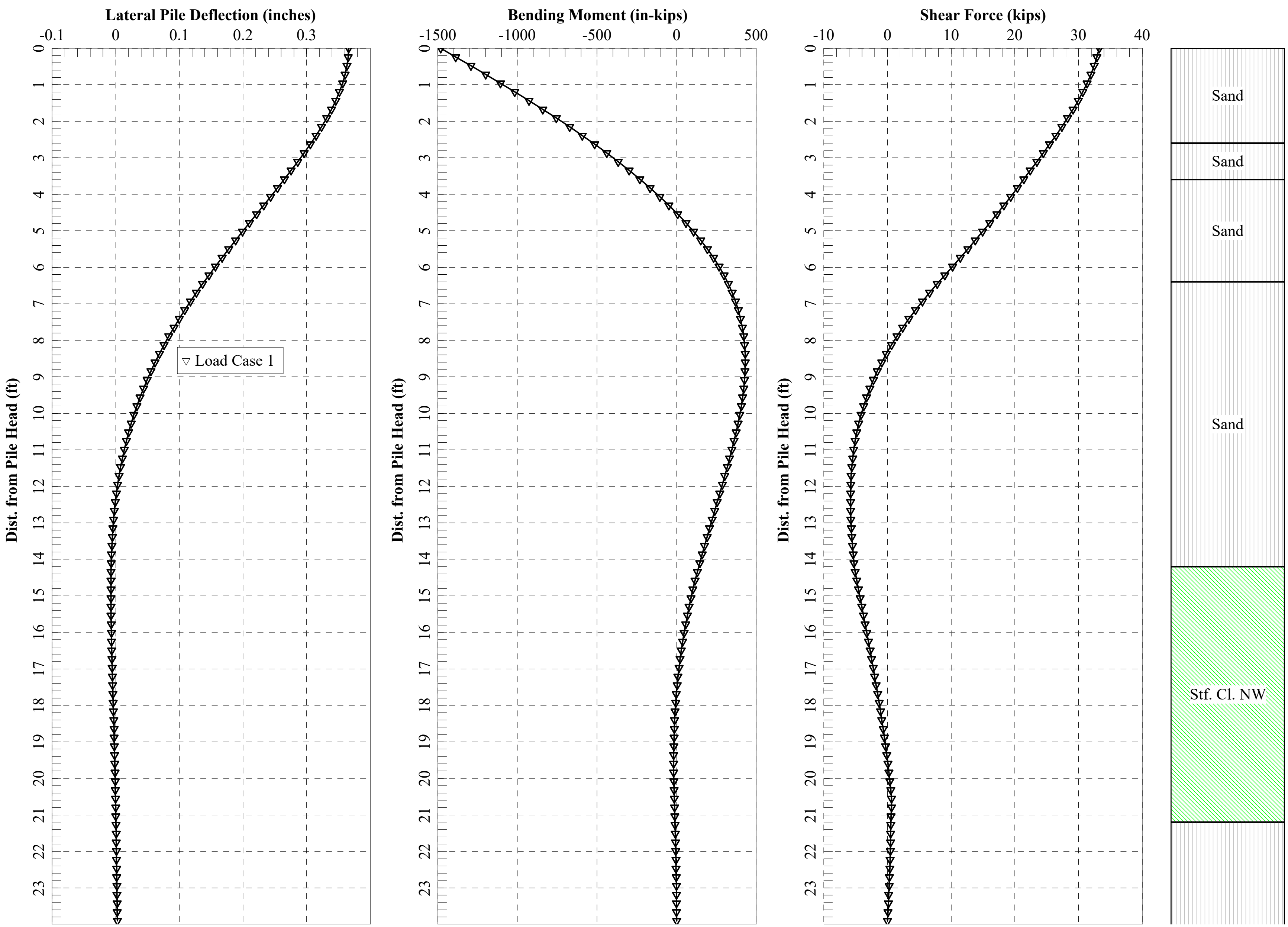
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians


Load Load               Load                 Axial    Pile-head  Pile-head  Max Shear Max Moment
Case Type   Pile-head    Type     Pile-head   Loading  Deflection  Rotation    in Pile    in Pile
 No.  1      Load 1       2        Load 2       lbs      inches     radians      lbs      in-lbs
---- ----- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
  1  y, in     0.2430   S, rad         0.00    360000.     0.2430       0.00     40365.  -1408675.


Maximum pile-head deflection = 0.2430000000 inches
Maximum pile-head rotation   = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.
```

Abutment 1: HP14x89
Weak Axis - Contraction



```
===============================================================================
                    LPile for Windows, Version 2022-12.011

                Analysis of Individual Piles and Drilled Shafts
               Subjected to Lateral Loading Using the p-y Method
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-------------------------------------------------------------------------------
                             Files Used for Analysis
-------------------------------------------------------------------------------

Path to file locations:
\09 Jobs\0026100s\09.0026188.00 - MEDOT - Wilton Bridge Replacement\09.0026188.01\Work\Calcs\Lpile\Lpile Files\

Name of input data file:
AB1_14x89_Contraction.lp12d

Name of output report file:
AB1_14x89_Contraction.lp12o

Name of plot output file:
AB1_14x89_Contraction.lp12p

Name of runtime message file:
AB1_14x89_Contraction.lp12r


-------------------------------------------------------------------------------
                            Date and Time of Analysis
-------------------------------------------------------------------------------

               Date:  June 26, 2024              Time:   9:55:26


-------------------------------------------------------------------------------
                                  Problem Title
-------------------------------------------------------------------------------

Project Name: Wilton Bridges Bridge


Job Number: 09.0026188.01


Client: MEDOT


Engineer: E. Tome


Description:



-------------------------------------------------------------------------------
                          Program Options and Settings
-------------------------------------------------------------------------------

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)
```

```
Analysis Control Options:
 - Maximum number of iterations allowed               =          500
 - Deflection tolerance for convergence               =   1.0000E-05 in
 - Maximum allowable deflection                       =     100.0000 in
 - Number of pile increments                          =          100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats



--------------------------------------------------------------------------------
                     Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Number of pile sections defined                       =            1
Total length of pile                                  =       23.900 ft
Depth of ground surface below top of pile             =      -0.9000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile. A summary of values of pile diameter vs. depth follows.


            Depth Below          Pile
Point        Pile Head         Diameter
 No.           feet             inches
-----      -------------     -------------
  1            0.000            14.7000
  2           23.900            14.7000


Input Structural Properties for Pile Sections:
----------------------------------------------

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                  =    23.900000 ft
   Pile width                                         =    13.800000 in



--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 6 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =     -0.90000 ft
   Distance from top of pile to bottom of layer       =     2.600000 ft
   Effective unit weight at top of layer              =   125.000000 pcf
   Effective unit weight at bottom of layer           =   125.000000 pcf
   Friction angle at top of layer                     =    32.000000 deg.
   Friction angle at bottom of layer                  =    32.000000 deg.
   Subgrade k at top of layer                         =    83.000000 pci
   Subgrade k at bottom of layer                      =    83.000000 pci


Layer 2 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =     2.600000 ft
   Distance from top of pile to bottom of layer       =     3.600000 ft
   Effective unit weight at top of layer              =    62.600000 pcf
```

```
   Effective unit weight at bottom of layer           =    62.600000 pcf
   Friction angle at top of layer                     =    32.000000 deg.
   Friction angle at bottom of layer                  =    32.000000 deg.
   Subgrade k at top of layer                         =    55.000000 pci
   Subgrade k at bottom of layer                      =    55.000000 pci


Layer 3 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =     3.600000 ft
   Distance from top of pile to bottom of layer       =     6.400000 ft
   Effective unit weight at top of layer              =    62.600000 pcf
   Effective unit weight at bottom of layer           =    62.600000 pcf
   Friction angle at top of layer                     =    32.000000 deg.
   Friction angle at bottom of layer                  =    32.000000 deg.
   Subgrade k at top of layer                         =    55.000000 pci
   Subgrade k at bottom of layer                      =    55.000000 pci


Layer 4 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =     6.400000 ft
   Distance from top of pile to bottom of layer       =    14.200000 ft
   Effective unit weight at top of layer              =    57.600000 pcf
   Effective unit weight at bottom of layer           =    57.600000 pcf
   Friction angle at top of layer                     =    30.000000 deg.
   Friction angle at bottom of layer                  =    30.000000 deg.
   Subgrade k at top of layer                         =    35.000000 pci
   Subgrade k at bottom of layer                      =    35.000000 pci


Layer 5 is stiff clay without free water

   Distance from top of pile to top of layer          =    14.200000 ft
   Distance from top of pile to bottom of layer       =    21.200000 ft
   Effective unit weight at top of layer              =    47.600000 pcf
   Effective unit weight at bottom of layer           =    47.600000 pcf
   Undrained cohesion at top of layer                 =   500.000000 psf
   Undrained cohesion at bottom of layer              =   500.000000 psf
   Epsilon-50 at top of layer                         =     0.008000
   Epsilon-50 at bottom of layer                      =     0.008000


Layer 6 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =    21.200000 ft
   Distance from top of pile to bottom of layer       =    27.900000 ft
   Effective unit weight at top of layer              =    57.600000 pcf
   Effective unit weight at bottom of layer           =    57.600000 pcf
   Friction angle at top of layer                     =    30.000000 deg.
   Friction angle at bottom of layer                  =    30.000000 deg.
   Subgrade k at top of layer                         =    35.000000 pci
   Subgrade k at bottom of layer                      =    35.000000 pci

 (Depth of the lowest soil layer extends 4.000 ft below the pile tip)


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                        Summary of Input Soil Properties
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Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	-0.900	125.0000	--	32.0000	--	83.0000
	(Reese, et al.)	2.6000	125.0000	--	32.0000	--	83.0000
2	Sand	2.6000	62.6000	--	32.0000	--	55.0000
	(Reese, et al.)	3.6000	62.6000	--	32.0000	--	55.0000
3	Sand	3.6000	62.6000	--	32.0000	--	55.0000
	(Reese, et al.)	6.4000	62.6000	--	32.0000	--	55.0000
4	Sand	6.4000	57.6000	--	30.0000	--	35.0000
	(Reese, et al.)	14.2000	57.6000	--	30.0000	--	35.0000
5	Stiff Clay	14.2000	47.6000	500.0000	--	0.00800	--
	w/o Free Water	21.2000	47.6000	500.0000	--	0.00800	--
6	Sand	21.2000	57.6000	--	30.0000	--	35.0000
	(Reese, et al.)	27.9000	57.6000	--	30.0000	--	35.0000

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--------------------------------------------------------------------------------
                              Static Loading Type
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Static loading criteria were used when computing p-y curves for all analyses.
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Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.366000 in	S = 0.0000 in/in	360000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	23.900000 ft
Flange Width	=	14.700000 in
Section Depth	=	13.800000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.811550 sq. in.
Moment of Inertia	=	325.837265 in^4
Elastic Bending Stiffness	=	9449281. kip-in^2
Plastic Modulus, Z	=	67.636247in^3
Plastic Moment Capacity = Fy Z	=	3382.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1290.578 kips
Nominal Axial Tensile Capacity	=	-1290.578 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	360.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 360.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000444	41.9374124	9448389.	115.7044043	14.8838666	
0.00000888	83.8748248	9448389.	61.5272029	15.8204887	
0.00001332	125.8122373	9448389.	43.4681348	16.7571105	
0.00001775	167.7496497	9448389.	34.4386011	17.6937326	
0.00002219	209.6870621	9448389.	29.0208809	18.6303549	
0.00002663	251.6244745	9448389.	25.4090674	19.5669769	
0.00003107	293.5618869	9448389.	22.8292006	20.5035990	

0.00003551	335.4992993	9448389.	20.8943005	21.4402210	
0.00003995	377.4367118	9448389.	19.3893783	22.3768431	
0.00004439	419.3741242	9448389.	18.1854404	23.3134651	
0.00004882	461.3115366	9448389.	17.2004004	24.2500873	
0.00005326	503.2489490	9448389.	16.3795337	25.1867094	
0.00005770	545.1863614	9448389.	15.6849542	26.1233314	
0.00006214	587.1237739	9448389.	15.0896003	27.0599535	
0.00006658	629.0611863	9448389.	14.5736270	27.9965755	
0.00007102	670.9985987	9448389.	14.1221503	28.9331977	
0.00007546	712.9360111	9448389.	13.7237885	29.8698197	
0.00007989	754.8734235	9448389.	13.3696891	30.8064417	
0.00008433	796.8108359	9448389.	13.0528634	31.7430638	
0.00008877	838.7482484	9448389.	12.7677202	32.6796859	
0.00009321	880.6856608	9448389.	12.5097335	33.6163080	
0.00009765	922.6230732	9448389.	12.2752002	34.5529301	
0.0001021	964.5604856	9448389.	12.0610611	35.4895521	
0.0001065	1006.	9448389.	11.8647668	36.4261741	
0.0001110	1048.	9448389.	11.6841762	37.3627963	
0.0001154	1090.	9448389.	11.5174771	38.2994183	
0.0001198	1132.	9448389.	11.3631261	39.2360404	
0.0001243	1174.	9448389.	11.2198002	40.1726624	
0.0001287	1216.	9448389.	11.0863588	41.1092845	
0.0001332	1258.	9448389.	10.9618135	42.0459066	
0.0001376	1300.	9448389.	10.8453034	42.9825287	
0.0001420	1342.	9448389.	10.7360751	43.9191507	
0.0001465	1384.	9448389.	10.6334668	44.8557728	
0.0001509	1426.	9448389.	10.5368942	45.7923949	
0.0001554	1468.	9448389.	10.4458401	46.7290169	
0.0001598	1510.	9448389.	10.3598446	47.6656390	
0.0001642	1552.	9448389.	10.2784974	48.6022611	
0.0001687	1594.	9448389.	10.2014317	49.5388831	
0.0001731	1635.	9444750.	10.1289863	50.0000000	Y
0.0001820	1713.	9413633.	9.9992637	50.0000000	Y
0.0001909	1786.	9356714.	9.8872130	50.0000000	Y
0.0001997	1854.	9282029.	9.7898027	50.0000000	Y
0.0002086	1918.	9194494.	9.7048009	50.0000000	Y
0.0002175	1979.	9098100.	9.6303188	50.0000000	Y
0.0002264	2036.	8994945.	9.5650217	50.0000000	Y
0.0002352	2091.	8888017.	9.5074665	50.0000000	Y
0.0002441	2143.	8778986.	9.4565910	50.0000000	Y
0.0002530	2193.	8668920.	9.4115629	50.0000000	Y
0.0002619	2241.	8558522.	9.3717055	50.0000000	Y
0.0002708	2288.	8449149.	9.3362109	50.0000000	Y
0.0002796	2332.	8341141.	9.3045903	50.0000000	Y
0.0002885	2376.	8234786.	9.2764128	50.0000000	Y
0.0002974	2418.	8130322.	9.2512965	50.0000000	Y
0.0003063	2459.	8027951.	9.2289014	50.0000000	Y
0.0003151	2498.	7927838.	9.2089234	50.0000000	Y
0.0003240	2536.	7826452.	9.1901020	50.0000000	Y
0.0003329	2571.	7722229.	9.1716604	50.0000000	Y
0.0003418	2603.	7616213.	9.1535363	50.0000000	Y
0.0003506	2633.	7509294.	9.1356714	50.0000000	Y
0.0003595	2661.	7401927.	9.1181404	50.0000000	Y
0.0003684	2687.	7294361.	9.1010928	50.0000000	Y
0.0003773	2712.	7187800.	9.0842155	50.0000000	Y
0.0003862	2735.	7082167.	9.0677298	50.0000000	Y
0.0003950	2756.	6977769.	9.0516791	50.0000000	Y
0.0004039	2777.	6875341.	9.0358222	50.0000000	Y
0.0004128	2796.	6773314.	9.0202493	50.0000000	Y
0.0004217	2814.	6673782.	9.0047922	50.0000000	Y
0.0004305	2831.	6575919.	8.9900453	50.0000000	Y
0.0004394	2847.	6479851.	8.9751447	50.0000000	Y
0.0004483	2863.	6385518.	8.9607041	50.0000000	Y
0.0004572	2877.	6293682.	8.9466590	50.0000000	Y
0.0004661	2891.	6203152.	8.9324190	50.0000000	Y
0.0004749	2904.	6114858.	8.9189260	50.0000000	Y
0.0004838	2917.	6028398.	8.9052627	50.0000000	Y
0.0004927	2928.	5943987.	8.8920604	50.0000000	Y
0.0005016	2940.	5861203.	8.8790726	50.0000000	Y
0.0005104	2951.	5780639.	8.8661964	50.0000000	Y
0.0005193	2961.	5701655.	8.8535386	50.0000000	Y
0.0005282	2971.	5624547.	8.8413214	50.0000000	Y
0.0005637	3006.	5333233.	8.7937380	50.0000000	Y
0.0005992	3036.	5067276.	8.7489644	50.0000000	Y
0.0006347	3062.	4824626.	8.7070032	50.0000000	Y
0.0006702	3085.	4602649.	8.6760298	50.0000000	Y
0.0007057	3104.	4398462.	8.6302531	50.0000000	Y
0.0007412	3121.	4210968.	8.5948757	50.0000000	Y
0.0007768	3137.	4038007.	8.5616612	50.0000000	Y
0.0008123	3150.	3878345.	8.5300009	50.0000000	Y
0.0008478	3162.	3730010.	8.5000691	50.0000000	Y
0.0008833	3173.	3592368.	8.4713113	50.0000000	Y
0.0009188	3183.	3464307.	8.4442310	50.0000000	Y
0.0009543	3192.	3344890.	8.4186520	50.0000000	Y
0.0009898	3200.	3232997.	8.3937145	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	360.0000000000	3200.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-0.900	0.00	N.A.	No	0.00	10086.
2	2.6000	3.5000	Yes	No	10086.	5880.
3	3.6000	4.7197	Yes	No	15966.	26586.
4	6.4000	8.3543	Yes	No	42552.	183877.
5	14.2000	44.6653	No	No	226428.	38587.
6	21.2000	18.4078	No	No	265016.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 0.366000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3660	-1482408.	33259.	0.00	47386.	9.45E+09	-102.788	402.7281	0.00
0.2390	0.3654	-1387353.	32871.	-4.36E-04	45242.	9.45E+09	-133.273	1046.	0.00
0.4780	0.3635	-1292960.	32446.	-8.42E-04	43113.	9.45E+09	-163.398	1289.	0.00
0.7170	0.3605	-1199506.	31936.	-0.00122	41005.	9.45E+09	-192.158	1529.	0.00
0.9560	0.3565	-1107256.	31347.	-0.00157	38924.	9.45E+09	-218.561	1758.	0.00
1.1950	0.3515	-1016457.	30682.	-0.00189	36876.	9.45E+09	-245.189	2000.	0.00
1.4340	0.3456	-927356.	29945.	-0.00219	34866.	9.45E+09	-268.657	2229.	0.00
1.6730	0.3390	-840174.	29144.	-0.00246	32899.	9.45E+09	-289.687	2451.	0.00
1.9120	0.3316	-755112.	28287.	-0.00270	30981.	9.45E+09	-308.118	2665.	0.00
2.1510	0.3235	-672348.	27383.	-0.00292	29114.	9.45E+09	-322.073	2855.	0.00
2.3900	0.3148	-592022.	26437.	-0.00311	27302.	9.45E+09	-337.894	3078.	0.00
2.6290	0.3057	-514290.	25453.	-0.00327	25548.	9.45E+09	-348.375	3269.	0.00
2.8680	0.2960	-439262.	24453.	-0.00342	23856.	9.45E+09	-348.847	3380.	0.00
3.1070	0.2860	-366966.	23446.	-0.00354	22225.	9.45E+09	-353.459	3544.	0.00
3.3460	0.2757	-297462.	22430.	-0.00364	20657.	9.45E+09	-355.267	3695.	0.00
3.5850	0.2652	-230787.	21412.	-0.00372	19153.	9.45E+09	-354.044	3829.	0.00
3.8240	0.2544	-166952.	20388.	-0.00378	17713.	9.45E+09	-360.661	4066.	0.00
4.0630	0.2435	-106032.	19339.	-0.00382	16339.	9.45E+09	-370.370	4363.	0.00
4.3020	0.2324	-48124.	18265.	-0.00385	15033.	9.45E+09	-378.812	4674.	0.00
4.5410	0.2214	6683.	17168.	-0.00385	14098.	9.45E+09	-385.816	4998.	0.00

4.7800	0.2103	58314.	16054.	-0.00384	15263.	9.45E+09	-391.286	5336.	0.00
5.0190	0.1993	106708.	14926.	-0.00382	16354.	9.45E+09	-395.447	5690.	0.00
5.2580	0.1884	151816.	13780.	-0.00378	17372.	9.45E+09	-403.420	6141.	0.00
5.4970	0.1776	193558.	12613.	-0.00373	18313.	9.45E+09	-410.320	6625.	0.00
5.7360	0.1670	231865.	11428.	-0.00366	19177.	9.45E+09	-416.101	7145.	0.00
5.9750	0.1566	266676.	10228.	-0.00359	19963.	9.45E+09	-420.719	7704.	0.00
6.2140	0.1465	297943.	9017.	-0.00350	20668.	9.45E+09	-424.139	8306.	0.00
6.4530	0.1365	325628.	7804.	-0.00341	21293.	9.45E+09	-421.673	8857.	0.00
6.6920	0.1269	349742.	6619.	-0.00330	21836.	9.45E+09	-404.670	9145.	0.00
6.9310	0.1176	370419.	5484.	-0.00320	22303.	9.45E+09	-386.736	9433.	0.00
7.1700	0.1086	387798.	4402.	-0.00308	22695.	9.45E+09	-368.022	9721.	0.00
7.4090	0.09991	402028.	3374.	-0.00296	23016.	9.45E+09	-348.679	10009.	0.00
7.6480	0.09160	413265.	2402.	-0.00284	23269.	9.45E+09	-328.852	10297.	0.00
7.8870	0.08364	421667.	1488.	-0.00271	23459.	9.45E+09	-308.684	10584.	0.00
8.1260	0.07605	427397.	632.1200	-0.00258	23588.	9.45E+09	-288.310	10872.	0.00
8.3650	0.06884	430623.	-165.429	-0.00245	23661.	9.45E+09	-267.861	11160.	0.00
8.6040	0.06199	431510.	-904.397	-0.00232	23681.	9.45E+09	-247.459	11448.	0.00
8.8430	0.05553	430226.	-1585.	-0.00219	23652.	9.45E+09	-227.220	11736.	0.00
9.0820	0.04944	426939.	-2208.	-0.00206	23578.	9.45E+09	-207.254	12024.	0.00
9.3210	0.04371	421813.	-2774.	-0.00193	23462.	9.45E+09	-187.660	12312.	0.00
9.5600	0.03836	415011.	-3285.	-0.00180	23309.	9.45E+09	-168.530	12600.	0.00
9.7990	0.03337	406693.	-3742.	-0.00168	23121.	9.45E+09	-149.950	12888.	0.00
10.0380	0.02873	397014.	-4146.	-0.00156	22903.	9.45E+09	-131.994	13175.	0.00
10.2770	0.02444	386125.	-4500.	-0.00144	22657.	9.45E+09	-114.730	13463.	0.00
10.5160	0.02048	374171.	-4805.	-0.00132	22388.	9.45E+09	-98.215	13751.	0.00
10.7550	0.01685	361293.	-5065.	-0.00121	22097.	9.45E+09	-82.501	14039.	0.00
10.9940	0.01354	347622.	-5280.	-0.00110	21789.	9.45E+09	-67.629	14327.	0.00
11.2330	0.01052	333286.	-5454.	-0.00100	21465.	9.45E+09	-53.634	14615.	0.00
11.4720	0.00780	318404.	-5589.	-9.01E-04	21130.	9.45E+09	-40.541	14903.	0.00
11.7110	0.00536	303090.	-5688.	-8.07E-04	20784.	9.45E+09	-28.370	15191.	0.00
11.9500	0.00317	287447.	-5753.	-7.17E-04	20431.	9.45E+09	-17.132	15479.	0.00
12.1890	0.00124	271572.	-5787.	-6.32E-04	20073.	9.45E+09	-6.832	15766.	0.00
12.4280	-4.52E-04	255557.	-5793.	-5.52E-04	19712.	9.45E+09	2.5329	16054.	0.00
12.6670	-0.00193	239482.	-5774.	-4.77E-04	19349.	9.45E+09	10.9703	16342.	0.00
12.9060	-0.00319	223423.	-5732.	-4.07E-04	18987.	9.45E+09	18.4944	16630.	0.00
13.1450	-0.00426	207445.	-5669.	-3.42E-04	18627.	9.45E+09	25.1250	16918.	0.00
13.3840	-0.00515	191609.	-5589.	-2.81E-04	18269.	9.45E+09	30.8869	17206.	0.00
13.6230	-0.00587	175968.	-5493.	-2.25E-04	17917.	9.45E+09	35.8098	17494.	0.00
13.8620	-0.00644	160565.	-5385.	-1.74E-04	17569.	9.45E+09	39.9280	17782.	0.00
14.1010	-0.00687	145441.	-5265.	-1.28E-04	17228.	9.45E+09	43.2797	18070.	0.00
14.3400	-0.00717	130627.	-5073.	-8.57E-05	16894.	9.45E+09	90.8473	36328.	0.00
14.5790	-0.00736	116520.	-4812.	-4.82E-05	16576.	9.45E+09	91.4407	35626.	0.00
14.8180	-0.00745	103128.	-4549.	-1.49E-05	16274.	9.45E+09	91.7131	35312.	0.00
15.0570	-0.00745	90458.	-4286.	1.45E-05	15988.	9.45E+09	91.7085	35321.	0.00
15.2960	-0.00737	78514.	-4023.	4.01E-05	15718.	9.45E+09	91.4611	35613.	0.00
15.5350	-0.00722	67298.	-3762.	6.23E-05	15465.	9.45E+09	90.9975	36165.	0.00
15.7740	-0.00701	56809.	-3502.	8.11E-05	15229.	9.45E+09	90.3389	36968.	0.00
16.0130	-0.00675	47046.	-3244.	9.69E-05	15008.	9.45E+09	89.5024	38022.	0.00
16.2520	-0.00645	38004.	-2988.	1.10E-04	14805.	9.45E+09	88.5020	39334.	0.00
16.4910	-0.00612	29678.	-2736.	1.20E-04	14617.	9.45E+09	87.3488	40923.	0.00
16.7300	-0.00576	22061.	-2488.	1.28E-04	14445.	9.45E+09	86.0517	42814.	0.00
16.9690	-0.00539	15145.	-2243.	1.34E-04	14289.	9.45E+09	84.6179	45041.	0.00
17.2080	-0.00500	8920.	-2002.	1.37E-04	14148.	9.45E+09	83.0523	47653.	0.00
17.4470	-0.00460	3376.	-1767.	1.39E-04	14023.	9.45E+09	81.3582	50713.	0.00
17.6860	-0.00420	-1500.	-1536.	1.39E-04	13981.	9.45E+09	79.5366	54301.	0.00
17.9250	-0.00380	-5722.	-1311.	1.38E-04	14076.	9.45E+09	77.5866	58529.	0.00
18.1640	-0.00341	-9303.	-1091.	1.36E-04	14157.	9.45E+09	75.5041	63543.	0.00
18.4030	-0.00302	-12261.	-877.683	1.33E-04	14224.	9.45E+09	73.2815	69548.	0.00
18.6420	-0.00265	-14611.	-670.918	1.29E-04	14277.	9.45E+09	70.9063	76834.	0.00
18.8810	-0.00228	-16375.	-471.212	1.24E-04	14317.	9.45E+09	68.3587	85828.	0.00
19.1200	-0.00194	-17570.	-279.103	1.19E-04	14344.	9.45E+09	65.6084	97193.	0.00
19.3590	-0.00160	-18221.	-95.241	1.13E-04	14358.	9.45E+09	62.6081	112012.	0.00
19.5980	-0.00129	-18351.	79.5492	1.08E-04	14361.	9.45E+09	59.2816	132213.	0.00
19.8370	-9.85E-04	-17987.	244.1436	1.02E-04	14353.	9.45E+09	55.4983	161620.	0.00
20.0760	-6.99E-04	-17161.	396.8777	9.69E-05	14334.	9.45E+09	51.0109	209184.	0.00
20.3150	-4.29E-04	-15911.	534.9317	9.19E-05	14306.	9.45E+09	45.2611	302681.	0.00
20.5540	-1.72E-04	-14283.	634.1421	8.73E-05	14269.	9.45E+09	23.9232	398446.	0.00
20.7930	7.20E-05	-12454.	654.0976	8.33E-05	14228.	9.45E+09	-10.007	398446.	0.00
21.0320	3.05E-04	-10703.	581.0741	7.98E-05	14189.	9.45E+09	-40.916	384210.	0.00
21.2710	5.29E-04	-9285.	515.3307	7.67E-05	14157.	9.45E+09	-4.931	26706.	0.00
21.5100	7.45E-04	-7905.	498.1985	7.41E-05	14126.	9.45E+09	-7.017	26994.	0.00
21.7490	9.55E-04	-6581.	475.1151	7.19E-05	14096.	9.45E+09	-9.081	27282.	0.00
21.9880	0.00116	-5328.	446.1309	7.01E-05	14067.	9.45E+09	-11.131	27570.	0.00
22.2270	0.00136	-4166.	411.2709	6.87E-05	14041.	9.45E+09	-13.178	27858.	0.00
22.4660	0.00155	-3111.	370.5350	6.76E-05	14017.	9.45E+09	-15.229	28146.	0.00
22.7050	0.00174	-2180.	323.8992	6.68E-05	13996.	9.45E+09	-17.292	28434.	0.00
22.9440	0.00193	-1391.	271.3178	6.62E-05	13979.	9.45E+09	-19.375	28722.	0.00
23.1830	0.00212	-760.847	212.7253	6.59E-05	13964.	9.45E+09	-21.484	29009.	0.00
23.4220	0.00231	-307.016	148.0398	6.57E-05	13954.	9.45E+09	-23.624	29297.	0.00
23.6610	0.00250	-47.410	77.1660	6.57E-05	13948.	9.45E+09	-25.800	29585.	0.00
23.9000	0.00269	0.00	0.00	6.57E-05	13947.	9.45E+09	-28.012	14937.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be inter-

polated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     0.36600000 inches
Computed slope at pile head      =       0.000000 radians
Maximum bending moment           =      -1482408. inch-lbs
Maximum shear force              =         33259. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =       0.000000 feet below pile head
Number of iterations             =             13
Number of zero deflection points =              2
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	0.3660	S, rad	0.00	360000.	0.3660	0.00	33259.	-1482408.

Maximum pile-head deflection = 0.3660000000 inches
Maximum pile-head rotation   = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.

APPENDIX E.6 – LATERAL EARTH PRESSURES

GZA
GeoEnvironmental, Inc
707 Sable Oaks Drive
Suite 150
South Portland, Maine 04106
207-879-9190
Fax 207-879-0099

Engineers and Scientists

Subject:

Evaluate lateral earth pressure coefficients of backfill behind abutments and wingwalls for the Bridges Bridge No. 2102 in Wilton, Maine

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 9th Edition (2023)

Input Parameters:

$\beta := 0\text{deg}$ Angle of backfill to the horizontal

$\theta := 90\text{deg}$ Angle of backface of wall to the horizontal

$\phi := 32\text{deg}$ Effective angle of internal friction *(Granular borrow, Soil Type 4, BDG Table 3-3)*

$\delta_f := 19.5\text{deg}$ Average value of friction angle between, precast concrete and clean sand/siltys and-gravel mixture *(AASHTO LRFD Table 3.11.5.3-1)*

Passive Earth Pressure on Integral Backwall:

Per BDG Section 5.4.2.11, developing full passive pressure requires that ratio of lateral abutment movement (y) to abutment height (Hb) exceeds 0.005. If the calculated rotation is significantly less, Rankine earth pressure may be considered. However, we understand that recent practice by MaineDOT is to utilize methodology consistent with MassDOT Section 3.10.8.

$y := 0.243\text{in}$ Maximum deflection from thermal expansion/contraction provided by structural engineer.

$H_b := 11.4\text{ft}$ Abutment Height use abutment 1, which is slightly shorter

$\frac{y}{H_b} = 0.0018$ Ratio of lateral movement to abutment height

Earth Pressure Coefficients:

MassDOT Section 3.10.8 presents the plot and calculation shown below for a gravel borrow material.

$$\omega := \frac{y}{H_b} = 0.0018$$

$$K_{p.mass} := 0.43 + 5.7\left(1 - \exp\left(-190 \cdot \frac{y}{H_b}\right)\right)$$

$$\boxed{K_{p.mass} = 2.06}$$

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$

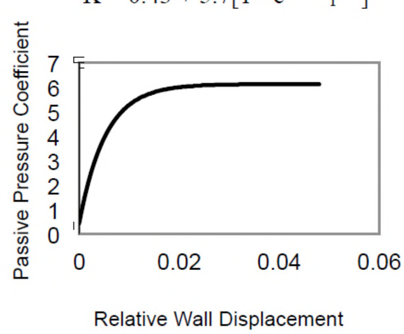

Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

GZA recommends that a passive earth pressure coefficient of 2.06, as calculated by the MassDOT methodology, and is recommended for the abutment design for the Bridges bridge project.

APPENDIX E.7 – FROST PENETRATION

March 2014

Figure 5-1 Maine Design Freezing Index Map

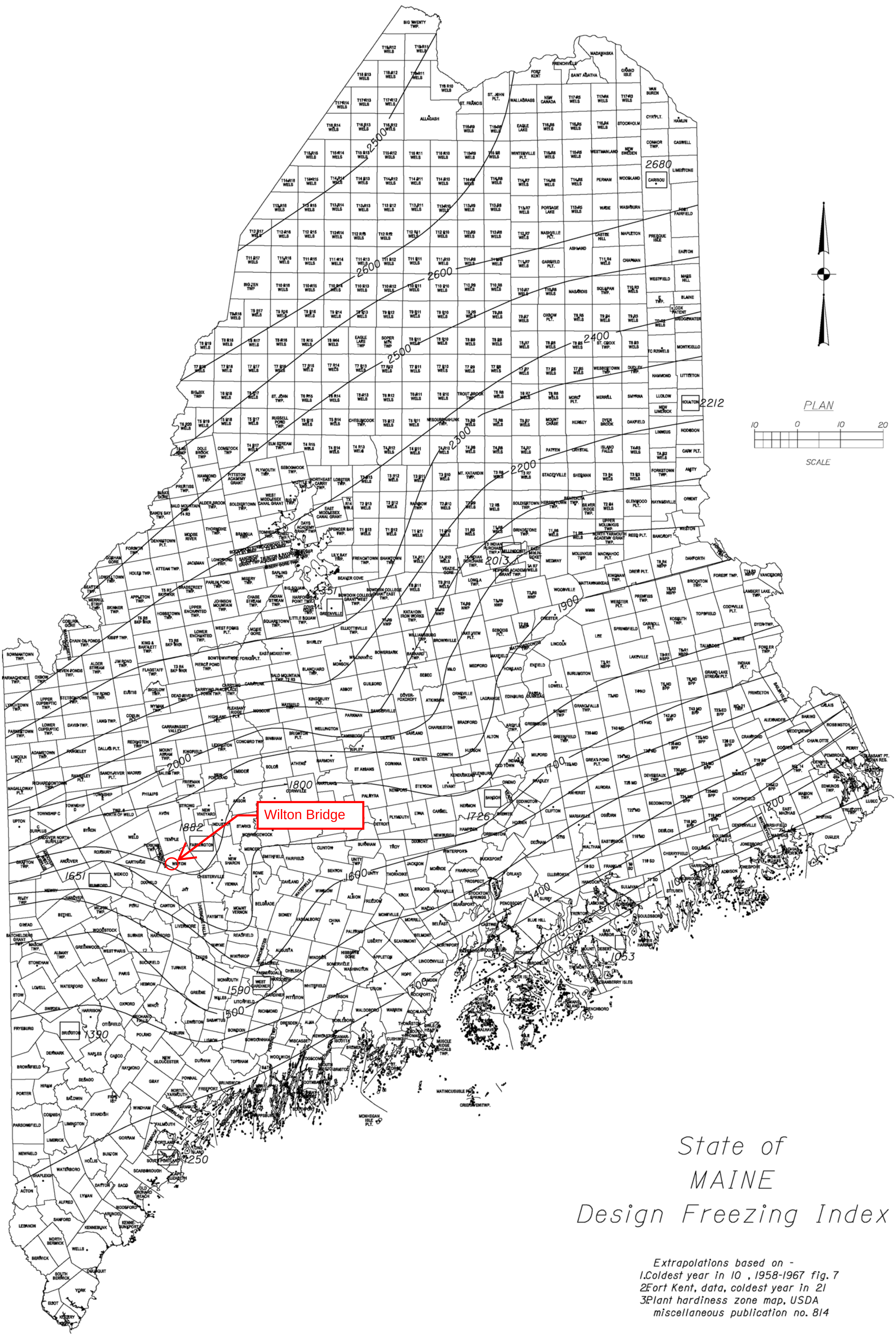

Table 5-1 Depth of Frost Penetration

90.1" = 7.5'

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

Granular materials anticipated near the abutment bearing elevations have an average water content of 10 percent. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing index of 1,800 the estimated depth of frost penetration is 90 inches.

APPENDIX F – ROCK CORE PHOTOGRAPHS

MaineDOT Wilton Bridge No. 2102
Carries Pond Road over Wilson Stream
Wilton, ME
WIN 25105.00

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WWS-102	R1	34.5 - 39.5	60	100	48	80	SCHIST	1
BB-WWS-102	R2	39.5 - 42	24	80	20	67	SCHIST	2
BB-WWS-102	R3	42 - 45	36	100	31	86	SCHIST	2
BB-WWS-101	R1	35 - 37.3	28	100	0	0	SCHIST	3
BB-WWS-101	R2	37.3 - 41	42	95	20	45	SCHIST	3,4
BB-WWS-101	R3	41 - 44.2	38	100	18	47	SCHIST	4
BB-WWS-101	R4	44.2 - 47	34	100	26	76	SCHIST	5

Notes:
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 5=Bottom.
2. Top photo is dry, bottom photo is wet.

MaineDOT Wilton Bridge No. 2102
Carries Pond Road over Wilson Stream
Wilton, ME
WIN 25105.00
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WWS-201	R1	37	-	41.5	50	92	18	33	SCHIST	1
BB-WWS-201	R2	41.5	-	44.5	34	94	17	47	SCHIST	1&2
BB-WWS-201	R3	44.5	-	47	28	95	10	33	SCHIST	2

Notes:
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 5=Bottom.
2. Top photo is dry, bottom photo is wet.