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GEOTECHNICAL DESIGN REPORT

PUDDLE DOCK BRIDGE (NO. 3107)
OVER FIFTEENMILE STREAM
MAINE DOT WIN 25299.00
ALBION, MAINE

September 2024
09.0026189.01

Prepared for:
Maine Department of Transportation
Augusta, Maine

Prepared by:
GZA GeoEnvironmental, Inc.
707 Sable Oaks Drive | Suite 150 | South Portland, Maine 04106
207.879.9190

31 Offices Nationwide
www.gza.com

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Built on trust.

GEOTECHNICAL
ENVIRONMENTAL
ECOLOGICAL
WATER
CONSTRUCTION MANAGEMENT

707 Sable Oaks Drive
Suite 150
South Portland, ME 04106
T: 207.879.9190
F: 207.536.1173
www.gza.com

VIA EMAIL

September 13, 2024
File No. 09.0026189.01

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

Re: Geotechnical Design Report
Replacement of Puddle Dock Bridge (No. 3107) over Fifteenmile Stream
Maine Department of Transportation WIN 25299.00
Albion, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the replacement of Puddle Dock Bridge (No. 3107) over Fifteenmile Stream in Albion, 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.P000191.24, dated February 9, 2024, and the *Limitations* Included in **Appendix A** of this report.

It has been a pleasure serving MaineDOT on this phase 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.

Nicholas V. Williams, P.E.
Project Manager

STATE OF MAINE
Andrew
R.
Blaisdell
10957
LICENSED
PROFESSIONAL ENGINEER
09/13/2024

Christopher L. Snow, P.E.
Consultant Reviewer

Andrew R. Blaisdell, P.E.
Associate Principal

NVW/ARB/CLS:erc
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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 Puddle Dock Bridge (No. 3107) which carries South Freedom Road over Fifteenmile Stream in Albion, 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.P000191.24, dated February 9, 2024, and the *Limitations* Included in **Appendix A** of this report.

1.1 BACKGROUND

Puddle Dock Bridge (No. 3107) carries South Freedom Road over Fifteenmile Stream, as shown on the **Project Locus, Figure 1**. The existing Puddle Dock Bridge was constructed in 1933 and consists of a 60-foot-long main span and a 25-foot-long approach span. The 85-foot-long rolled steel beam superstructure originally supported a timber deck wearing surface. The existing abutment and pier substructures consist of concrete-capped granite abutments assumed to bear on bedrock. In 1993, improvements included substructure rehabilitation, timber deck replacement, and paint, followed by another timber deck replacement in 2013.

It is GZA's understanding that a full bridge replacement is planned. The replacement bridge will be constructed on the current alignment, and the road will be closed for construction. The proposed bridge is planned to consist of a 104-foot-long, single-span, integral abutment bridge. The new abutments will be constructed approximately 7 to 14 feet behind the existing abutments and will be founded on H-piles.

Grading within the limits of the existing roadway will be limited to minor cuts and fills of less than approximately 2 feet. Approach roadway grades will be raised by about 1 to 2 feet near the abutments. The embankment will be widened by approximately 5 to 7 feet laterally to the north and south. Embankment side slopes are planned to be constructed with an inclination of 2 horizontal to 1 vertical (2H:1V) or flatter, sloping downward to ditches proposed to be installed on both sides of each approach and are planned to be riprap-protected with plain riprap below El. 240. Embankment slopes in front of abutments are planned to be constructed with inclinations ranging between 1.7H:1V at Abutment 1 and 1.6H:1V at Abutment 2, where plain riprap protection is planned to be used above the Q50 stream elevation and heavy riprap protection is planned to be used below the Q50 elevation.

Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988 (NAVD 88).

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Reviewed mapped surficial and bedrock geology of the site;
- Reviewed and summarized existing subsurface data including logs from the 2022 borings;

- Reviewed results from the laboratory testing program to evaluate engineering and index properties of the site soils and bedrock;
- Conducted geotechnical engineering analyses for soil and bedrock properties; stability of raised and widened embankments; frost susceptibility; AASHTO LRFD load and resistance factors associated with geotechnical design elements; nominal resistance of pile foundations; lateral pile evaluations; abutment design parameters and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for rock-socketed piles, abutment backfill type and properties, lateral earth pressures, seismic design parameters, and recommended construction considerations; and
- Prepared this report summarizing our findings and design recommendations.

The recommendations contained in this Geotechnical Design Report were coordinated with MaineDOT, who provided the boring logs, geotechnical drawings incorporating GZA's edits, core photo logs, laboratory testing results, lateral earth pressure calculations, axial and lateral pile loading, pile displacement, and the applied plastic moment of the piles based on GZA's initial iteration of the lateral pile analyses.

2.0 SUBSURFACE EXPLORATIONS

Crews from MaineDOT and S.W. Cole drilled four test borings, BB-AFMS-101 through -104, between December 21 and 23, 2022. The as-drilled boring locations and elevations were surveyed and provided by MaineDOT (in station/offset format) and are included on the logs in **Appendix B**.

BB-AFMS-101 and -102 were located behind the existing west abutment, and BB-AFMS-103 and -104 were located behind the existing east abutment, as shown on **Figure 2**. The test borings were drilled through the overburden soil and into bedrock to depths of approximately 16.2 to 22.8 feet below ground surface (bgs). The hammer efficiency at the time of the borings was 0.92 for BB-AFMS-101 and -102 and 0.906 for BB-AFMS-103 and -104, as provided on the boring logs. Approximately 10 feet of bedrock core was obtained in each boring using NQ2 wire line coring equipment. Dry photographs of the collected rock core are presented in **Appendix C**.

MaineDOT personnel monitored the drilling work and prepared logs of each boring which are included in **Appendix B**.

3.0 LABORATORY TESTING

Laboratory testing was conducted on split-spoon soil and rock core samples retrieved during the subsurface investigation. The soil testing program consisted of four (4) gradation analysis / AASHTO Classification / Frost Classification assessments and four (4) water content tests performed on the recovered split-spoon samples. The rock core testing program consisted of unconfined compressive strength / secant modulus tests on two rock core samples. 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 bridge consists of the Presumpscot formation, wetland deposits, ice-contact deposits, and Pratt Stream delta. The units are described as follows:

- *Presumpscot:* Glaciomarine silt, clay and sand deposited on the late-glacial sea floor;
- *Wetland Deposit:* Peat, muck, silt, and clay in poorly drained areas;
- *Ice-Contact Deposit:* Sand and gravel deposited in contact with glacial ice; formed in a marine environment; and
- *Pratt Stream Delta:* Sand and gravel deposited into the sea and built up to the ocean surface. Formed near the glacier margin during recession of the most recent ice sheet.

Based on available geologic mapping[2] at the site, bedrock in the vicinity of the site is mapped as the Vassalboro Formation, described as massive bluish-gray sandstone, locally recrystallized and becomes quartzite.

4.2 SUBSURFACE PROFILE

One soil unit, Fill, was encountered along the proposed bridge alignment at the surface or beneath 6 to 9 inches of asphalt pavement and above possible weathered bedrock and competent bedrock at the site. The approximate thicknesses and generalized descriptions of the subsurface units as classified by MaineDOT are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. An interpretive subsurface profile based on the test boring results is presented on **Figure 2**. The encountered thicknesses and elevations of each stratum are summarized on the attached **Table 1**.

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[1] Weddle, Thomas K., 2015, Surficial geology of the Albion quadrangle, Maine: Maine Geological Survey, Open-File Map 15-14, map, scale 1:24,000.

[2] Osberg, Philip H., Hussey, Arthur M., II, and Boone, Gary M., (Editors), 1985, Bedrock geologic map of Maine: Maine Geological Survey, 1 plate, correlation chart, tectonic inset map, metamorphic inset map, color geologic map, cross sections, scale 1:500,000. https://digitalmaine.com/mgs_maps/23/.

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	5.5 to 11.0	Variable ranging from: Brown, damp, medium dense fine to medium SAND, with varying amounts of Gravel and Silt, to: Olive grey, moist, medium dense SILT, little Sand, with cobbles and boulders. (USCS: SM, CL) Typical MaineDOT Frost Classification = II, IV *Encountered in all borings.*
Possible Weathered Rock	0.7 to 1.5	Fragments observed when advancing roller bit or solid auger in borings BB-AFMS-101, -102, and -103.
Estimated Top of Bedrock *(See Note 1)*	Abutment 1: Approx. El. 233.5 to 239.1 Abutment 2: Approx. El. 231.9 to 234.6	

Notes:

1. Estimated Top of Bedrock corresponds to bottom of possible weathered rock. However, characteristics of possible weathered rock may be similar to competent rock from a construction standpoint.

4.2.1 Bedrock

Bedrock was cored in each test boring and was identified as GRANOFELS. Photographic logs of the recovered rock core specimens are included in **Appendix C**. Bedrock descriptions were completed by a Maine-licensed Professional Geologist from MaineDOT. The rock was described as moderately hard, moderately to severely weathered, very fine to medium-grained, grey to blue-grey. Discontinuities are indicated to consist of fairly strong foliation, which is steeply inclined, very close to close, paralleling layering, weak, fractured and stained. The Rock Quality Designation (RQD) in the Granofels ranged from 0 to 56 percent, corresponding to a Rock Quality of Very Poor to Fair.

Unconfined compressive strength testing was conducted on two core samples, the results of which are summarized in the following table.

SUMMARY OF BEDROCK STRENGTH TEST RESULTS							
Boring	**Depth below Existing Ground (ft bgs)**	**Depth below Top of Rock (ft bgs)**	**Elevation (ft NAVD 88)**	**Unconfined Compressive Strength (psi)**	**Secant Modulus @ 50% of Failure Stress (ksi)**	**Unit Weight (pcf)**	**Rock Type**
BB-AFMS-103	20.7	8.2	223.7	15,945	3,550	169	GRANOFELS
BB-AFMS-103	22.3	9.8	222.1	15,930	4,180	169	GRANOFELS

4.2.2 Groundwater

Groundwater was not encountered in BB-AFMS-101, -102, or -104. Water introduced during drilling of BB-AFSM-103 drained upon completion of drilling indicating that static water level was at or below the bottom of boring at the time of drilling.

Fluctuations in groundwater levels will occur due to variations in river level, precipitation, and construction activity in the area. Consequently, water levels during and after construction are likely to vary from those encountered in the borings 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 LRFD) 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 APPROACH AND HIGHWAY EMBANKMENTS

The proposed embankment widening will require new fills and reconstructed slopes up to about 5 feet high at the west approach and up to 9 feet high at the east approach. Approach embankment side slope inclinations will be 2H:1V or flatter. Along both sides of the west approach and the north (left) side of the east approach, the total height of embankment is about 5 feet or less, and the lower portion of the slope will be excavated to enhance drainage. The south (right) side of the east approach is an approximately 20-foot-high fill slope that roughly parallels the existing embankment slope. The MaineDOT drawings show the inclination of the slopes in front of the abutments ranges from 1.6H:1V to 1.7H:1V and will generally require excavation 3 to 4 feet below existing grades to provide the design thickness and keyway depth. Considering the likelihood of bedrock within the required keyway excavation we understand MaineDOT has developed details to secure large buttress stones at the toe of the riprap slope.

The subsurface conditions beneath the approaches include fill over bedrock, and new fill slopes will consist of engineered fill materials. Therefore, it is GZA's opinion that post-construction embankment settlement and potential for global slope instability will be negligible, in the event that oversteepened riprap slopes remain stable.

5.3 EVALUATION OF FOUNDATION TYPES

Technically feasible foundation types for this bridge include full height or semi-integral abutments with spread footings bearing on tremie seals on bedrock, and integral abutments supported on rock-socketed H-piles. Based on constructability and cost considerations and the preference to limit in-water work, MaineDOT selected an integral abutment bridge supported on steel H-piles. Based on interpolation of boring data, bedrock depths below the bottom of the abutment are anticipated to be approximately 0 to 1 foot below the bottom of abutment backwall elevations, which are considered inadequate for driven piles. Therefore, rock-socketed H-piles were selected as the preferred foundation type.

5.4 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 AASHTO 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 AASHTO LRFD Table 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 LRFD Reference**
Axial Tip Resistance	Drilled Shaft Tip Resistance in Rock (for Steel H-Pile Cast in Concrete-Filled Rock Socket)	0.50	10.5.5.2.4-1

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

The piles will be subject to lateral loading, so they 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_c=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. 6.9.2.2-2) for the upper pile segment.

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

5.5 PILE DESIGN CONSIDERATIONS

5.5.1 Pile Type and Loading

Each abutment will be supported by five, HP14x89 ASTM A572 Grade 50 steel (50 ksi yield stress) H-piles, oriented for weak-axis bending relative to the alignment of the bridge beams. MaineDOT provided a maximum factored axial compressive load of 357 kips per pile for the strength limit state.

Rock-socketed abutment piles should be installed to provide a nominal axial resistance of at least 714 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.5 for piles installed in bedrock sockets and designed for axial compressive resistance as drilled shafts end bearing in bedrock (AASHTO LRFD Table 10.5.5.2.4-1). The resistance factor is based on a static analysis method for end bearing in rock where they will be socketed.

5.5.2 Pile Design Considerations and Axial Pile Resistance

Since the piles will gain support in bedrock, there is no reduction for group interaction in axial compression. As previously noted, post-construction embankment settlement is anticipated to be minimal. Therefore, downdrag forces were not considered. Axial tensile geotechnical (uplift) resistance was not evaluated because integral abutments do not impose uplift loading. 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.

Rock sockets are planned to improve lateral resistance and limit deformation under thermal expansion and contraction of the integral abutment bridge. Based on the shallow nature of the bedrock relative to the abutment, installation of H-piles in the rock sockets should be achieved by cleaning out the rock socket, adding a plate at the base of the pile, and grouting the pile into the socket.

The grouted rock socket detail will include a steel plate welded to the tip of the pile and detailed such that grout can be reliably placed below and around the pile tip and promote full, uniform load transfer to end bearing in bedrock. For this condition, the end bearing resistance was calculated using AASHTO LRFD Equation 10.8.3.5.4C-2 and assuming a jointed rock mass, and a bearing area of 225 square inches for a 15-inch square plate, as indicated in the MaineDOT drawings. The results are representative of a lower-bound tip resistance, with calculated nominal and factored geotechnical axial compression resistances of 1,631 kips and 816 kips, respectively. Since these values are greater than the required resistance, the 15-inch by 15-inch square base plate is shown to provide adequate geotechnical resistance.

5.5.3 Lateral Pile Analysis

The soil/bedrock/pile geometry is similar at both abutments due to the presence of very shallow bedrock. Therefore, GZA developed soil profiles for lateral pile evaluations for the shallowest bedrock profile (Abutment 1). Because the bottoms of abutments are relatively close to the top of rock, the impact of the pile deflection on the potential densification of soil within the socket was considered. The two assumed socket fill conditions included denser soils for the expansion case, resulting in stiffer p-y springs and a greater moment generation at the top of the pile, and looser soils for the contraction case, resulting in softer p-y springs and a deeper depth to fixity. The controlling soil-pile profile appeared to be the expansion case due to the greater soil spring stiffness. Therefore, only the expansion case profile was considered for the remainder of the lateral pile analyses.

The following design profile reflects the soil conditions encountered in the test borings at both abutments. The base of the approach slab was modelled as the ground surface (El. 241) for the lateral pile evaluations. The top of pile was assumed to be located at the base of the abutment (EL. 235.5), resulting in a total pile embedment length of 13 feet. The upper foot consisted of embankment fill overlying a 12-foot rock socket. The embedded pile length was modeled considering 1 foot of granular fill over 9 feet of sand infill over 3 feet of grout. The upper 3.5 feet of socket infill sand was assumed to be densified by cyclic lateral pile movement such that the properties were the same as the embankment fill. For the purpose of modelling, the unconfined compressive strength of the grouted rock socket was limited to 2,000 psi. Lateral pile analysis used the Ensoft, Inc. LPILE® version 2022-12.011 software.

L-PILE® INPUT PARAMETERS 13' PILE: 1' NEW FILL, 9' SAND INFILL OVER 3' CONCRETE						
Stratum	**Soil Model**	**Top of Layer Elevation (ft)**	**Layer Thickness (ft)**	**k (pci) / E50/ krm**	**ϕ' (deg)/ Su (psf)/ UCS (psi)**	**γ_e (pcf)**
New Fill	Reese Sand	235.5	1.0	122	34	125
Upper Socket Infill	Reese Sand	234.5	3.5	122	34	125
Lower Socket Infill	Reese Sand	231.0	5.5	83	32	122
Top of Grout in Rock Socket	Weak Rock	225.5	3.0	0.0005	2000	169

GZA conducted initial analyses for a weak pile orientation for both a HP 14x89 and HP 14x117 with a maximum deflection at the pile head of 0.47 inches and the 307-kip maximum factored axial load (originally provided by MaineDOT). The initial results provided to MaineDOT showed high bending stress immediately below the pile cap. We understand that MaineDOT evaluated a plastic hinge of the abutment piles using the results from the initial analyses to evaluate pile section and orientation.

In response to MaineDOT's request, GZA conducted an additional lateral evaluation for the HP 14x89 pile, which included input parameters provided by MaineDOT, including a plastic moment of 1,871 in-kips, a 0.47-inch deflection, and an updated axial load of 357 kips. The summarized results of the lateral pile analyses are provided in **Appendix E**. Using these results, MaineDOT reevaluated the pile structurally and concluded that the calculated shear and bending moments in the H-pile are acceptable for weak-axis bending with a five-pile group at each abutment.

5.6 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.

Based on the estimated thermal bridge expansion of 0.47 inches provided by MaineDOT and the Abutment 1 (shorter) abutment height of approximately 11 feet, the calculated abutment rotation is 0.0036 feet/foot. The *Massachusetts Department of Transportation LRFD Bridge Design Manual* 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 calculations developed by MaineDOT based on this equation were reviewed by GZA, as presented in **Appendix E**, and are provided in **Section 6.3** 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.

5.7 SEISMIC DESIGN CONSIDERATIONS

Seismic site class was evaluated in accordance with the 2020 AASHTO LRFD, along with consideration of the 2011 AASHTO Guide Specifications for LRFD Bridge Design (Seismic Guide Specification).

The subsurface profile for seismic design includes the approach fills (including backfill behind abutments) and existing fill materials overlying bedrock. Seismic site class was determined in general accordance with AASHTO LRFD Table C3.10.3.1-1, considering the average Standard Penetration Testing (SPT) N-value of granular soils encountered in the borings. The SPT N-value used to determine the site class was evaluated by including only the soil profile, resulting in an effective profile thickness ranging from 5.5 to 12.5 feet.

The average SPT N-value for encountered granular soils is between 15 and 50 blows per foot. Therefore, the bridge is assigned to Site Class D. The available subsurface data indicates that the natural materials encountered at the site are sufficiently stiff or dense that the potential for liquefaction is low.

5.8 FROST PROTECTION

New and existing fills are anticipated to be present at the abutments and embankments. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,550, and with low moisture content (10 percent) soils, the estimated depth of frost penetration is approximately 7.0 feet.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes that are not riprap-covered should be designed with MaineDOT-typical slope angles of 2H:1V or flatter. Earth slopes flatter than 2H:1V should be provided with loam and seed for permanent erosion protection. Steeper slopes should be covered with riprap. Riprap should also be provided where the embankment side slopes will be near or below typical water levels, to protect from scour. Protective aggregate cushion and geotextile should be placed adjacent to the riprap in accordance with the MaineDOT standard details. If a keyway cannot be excavated at the toe of the riprap slopes due to bedrock, a toe buttress stone detail should be utilized to enhance the stability.

6.2 SEISMIC DESIGN

The peak ground acceleration coefficient, short- and long-period spectral acceleration coefficients were interpolated from the AASHTO LRFD design guide maps (Figures 3.10.2.1-1 through -21, as appropriate). Based on the site coordinates, the recommended AASHTO LRFD 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)	0.12 g
SDs (Period = 0.2 sec)	0.24 g
SD1 (Period = 1.0 sec)	0.10 g

6.3 ABUTMENT AND WINGWALL DESIGN

- If bedrock is encountered during abutment backwall construction, it should be excavated to a minimum depth of 1 foot below bottom of abutment and a distance of 2 feet laterally on all sides from the backwall.
- 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 = 3.21 (use for design of backwalls and wingwalls).
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent soil height (H_{eq}) values developed in accordance with AASHTO LRFD Article 3.11.6.4, based on the abutment/wingwall height and distance from the wall back-face 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.

6.4 RECOMMENDATIONS FOR FOUNDATIONS

- The proposed abutments may be supported on ASTM A572, Grade 50 steel (50 ksi yield stress) HP14x89 H-piles socketed into the bedrock. The nominal axial pile resistance of 1,631 kips will be developed through end bearing of the plate at the base of the rock socket, providing a factored axial resistance of 816 kips per pile.
- Rock socket holes should be drilled through the overburden using 3-foot minimum inside-diameter temporary casing that is seated into the top of rock prior to drilling the socket.

- Rock sockets should be at least 36 inches diameter and should be cleaned of all loose material using an airlift or vacuum truck, and the socket base should be inspected using sounding techniques prior to pile placement.
- The piles should be equipped with centralizers, 15-inch by 15-inch base plates and a 2-inch-outside-diameter tremie tube through a 3-inch diameter hole in the base plate prior to installation.
- The lower portion of the tremie tube (below the base plate) should consist of a 1-inch-minimum-diameter black steel tremie tube extending through and below the center of the base plate. The tremie tube extending above the base plate may consist of steel, PVC or plastic tubing.
- The piles should be supported above the bottom of the socket using a central shoe plate or chairs to provide a minimum clearance of 3 inches above the base, then tremie grouted up to 3 feet above the base plate using 5,000 psi minimum unconfined compressive strength grout.
- Granular infill consisting of MaineDOT 703.22 Underdrain Backfill Material, Type C should be used to backfill the remainder of the socket and the drill hole. Care should be taken to maintain at least 4 feet of granular infill above the bottom of the temporary casing as it is withdrawn.
- Rock-socketed pile construction shall be conducted in accordance with Special Provision Section 501, Foundation Piles (Rock-Socketed Pile Foundations).

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during pile installation, excavation, dewatering, and foundation subgrade preparation and protection.

7.1 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

It is anticipated that South Freedom Road will be closed during construction of the new bridge. Excavations for abutment foundations are anticipated to extend approximately 8 to 11 feet below existing pavement grades to the bottom of proposed abutments. We anticipate that it will be feasible to construct the abutments using sloped open-cut excavations. In all cases, temporary excavations should comply with Occupational Safety and Health Administration excavation safety requirements.

Excavation for abutment backwalls and riprap slope keys may encounter bedrock. If so, rock should be removed to the minimum limits recommended in **Section 6.3**. Due to the steeply inclined foliation, we anticipate that up to a few feet of rock can be removed using a hoe ram.

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. 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 should comply with all local, State, and federal regulations.

7.2 PILE INSTALLATION CONTROL

The piles will be installed in pre-drilled holes that extend from the pile cap subgrade to the bottom of the proposed rock socket. Temporary casing will be necessary to keep the holes open through the overburden soil and to provide a seal into bedrock for subsequent socket drilling. Once the casings are seated in the top of the rock, the rock sockets may be drilled. Once the socket drilling is completed, the bases should be cleaned out using an auger, cleanout bucket, air lift, or other suitable means such that no loose material remains in the base.

A temporary template or frame may be employed to establish the pile top locations. The piles may then be placed in the prepared and approved sockets and tremie grout placed through and above the base plates.

Rock-socketed pile construction shall be conducted in accordance with Special Provision Section 501, Foundation Piles (Rock-Socketed Pile Foundations).

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, none of the four fill samples tested had less than 10 percent passing the No. 200 sieve, indicating the fill generally will not meet MaineDOT specifications for Gravel Borrow and/or Granular Borrow for Underwater Backfill. The material is considered suitable for use as Common Borrow.

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
Albion Puddle Dock Bridge #3107
Albion, ME
WIN 25299.00

Exploration Designation	Ground Surface Elev. [1]	Station			Offset		Top of Stratum Elevation (ft)	Approximate Encountered Thickness of Stratum[4]	Groundwater		Top of Weathered Rock		Top of Intact Rock		Total Exploration Depth (ft)
					(feet)	Dir.	Fill	Fill	Depth (ft)	Elev. [1]	Depth (ft)	Elev. [1]	Depth (ft)	Elev. [1]	
BB-AFMS-101	245.3	103.0	+	51.1	4.4	R	245.3	5.5	NE	NE	5.5	239.8	6.2	239.1	16.2
BB-AFMS-102	244.5	103.0	+	69.7	3.6	L	244.5	9.9	NE	NE	9.9	234.6	11.0	233.5	21.0
BB-AFMS-103	244.4	104.0	+	65.6	2.4	R	244.4	11.0	NE	NE	11.0	233.4	12.5	231.9	22.8
BB-AFMS-104	244.9	104.0	+	82.8	5.1	L	244.9	10.3	NE	NE	NE	--	10.3	234.6	20.3

Notes:

1) Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

2) As-completed exploration locations and elevations for -100 series borings were surveyed by MaineDOT.

3) "NE" indicates stratum not encountered, "NM" indicates Groundwater not measured.

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.

5) Boring stations and offsets from the centerline alignment, were provided by MaineDOT.

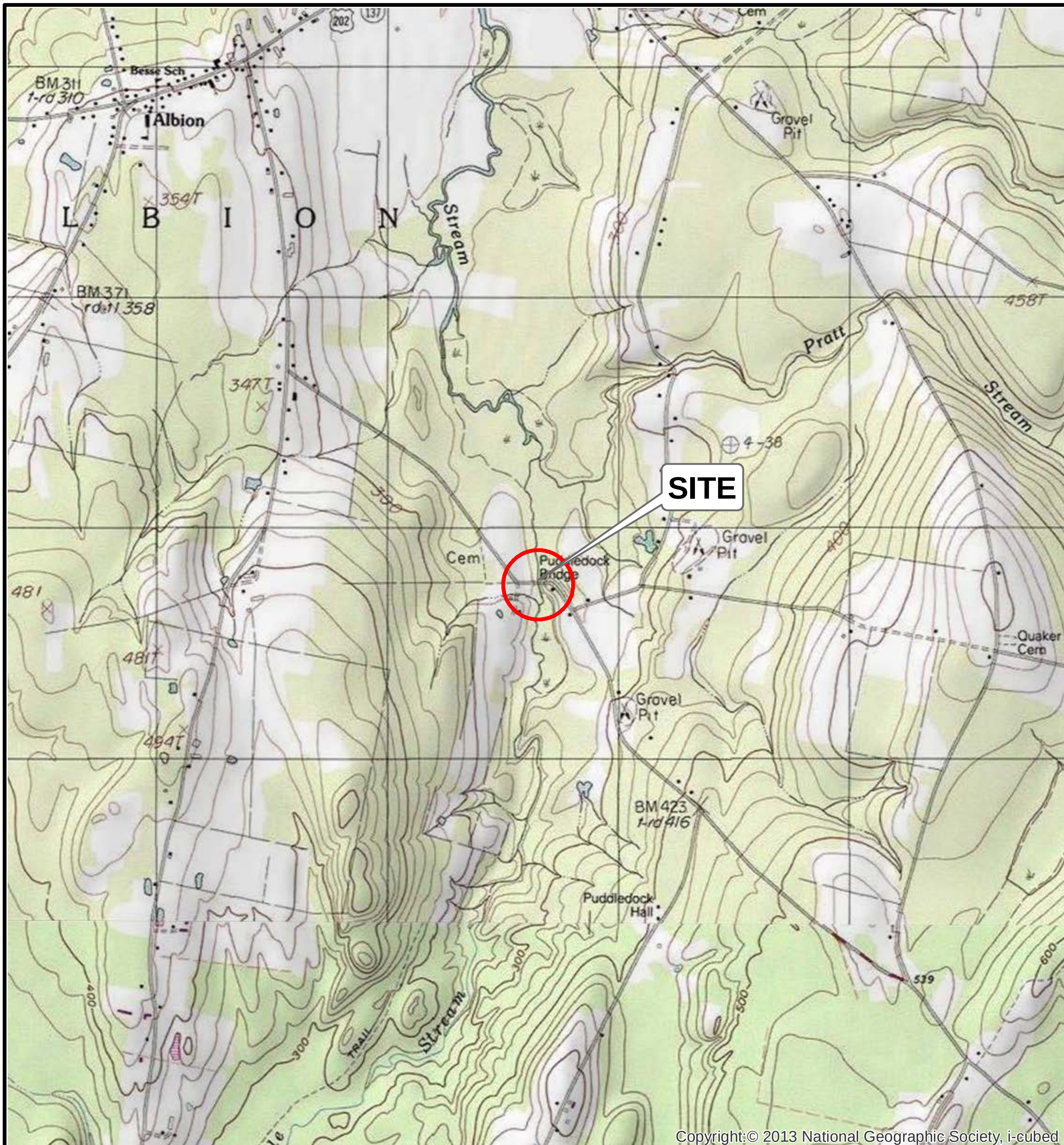
6) Drilling for borings was conducted using a truck, or atv - mounted rig provided by MaineDOT or S.W. Cole. SPT samples were conducted using calibrated automatic hammers.

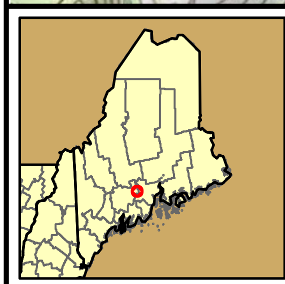
7) See boring logs in Appendix B for additional details of all borings.

TABLE 2
Summary of Rock Core
Puddle Dock Bridge over Fifteenmile Stream
Albion, ME
WIN 25299.00

Boring ID	Core Run	Ground Surface El. (ft)	Depth of Core Run below GS (ft)			Depth to Competent Rock (ft)	Depth Below Top of Rock (ft)			Length of Core Run (in)	Rec (in)	Rec (%)	RQD (in)	RQD %	Elev. (ft)		Rock Type
			Top		Bottom		Top		Bottom						Top	Bottom	
BB-AFMS-101	R1	245.3	6.2	-	11.2	6.2	0.0	-	5.0	60.0	57	95%	0	0%	239.1	234.1	Granofels
BB-AFMS-101	R2	245.3	11.2	-	16.2	6.2	5.0	-	10.0	60.0	60	100%	18	30%	234.1	229.1	Granofels
BB-AFMS-102	R1	244.5	11.0	-	14.1	11.0	0.0	-	3.1	37.2	37	100%	0	0%	233.5	230.4	Granofels
BB-AFMS-102	R2	244.5	14.1	-	16.0	11.0	3.1	-	5.0	22.8	23	100%	0	0%	230.4	228.5	Granofels
BB-AFMS-102	R3	244.5	16.0	-	21.0	11.0	5.0	-	10.0	60.0	55	92%	0	0%	228.5	223.5	Granofels
BB-AFMS-103	R1	244.4	12.8	-	17.8	12.5	0.3	-	5.3	60.0	60	100%	29	48%	231.6	226.6	Granofels
BB-AFMS-103	R2	244.4	17.8	-	22.8	12.5	5.3	-	10.3	60.0	60	100%	34	56%	226.6	221.6	Granofels
BB-AFMS-104	R1	244.9	10.3	-	15.3	10.3	0.0	-	5.0	60.0	30	50%	0	0%	234.6	229.6	Granofels
BB-AFMS-104	R2	244.9	15.3	-	20.3	10.3	5.0	-	10.0	60.0	60	100%	16	27%	229.6	224.6	Granofels

FIGURES





N
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USGS QUADRANGLE LOCATION

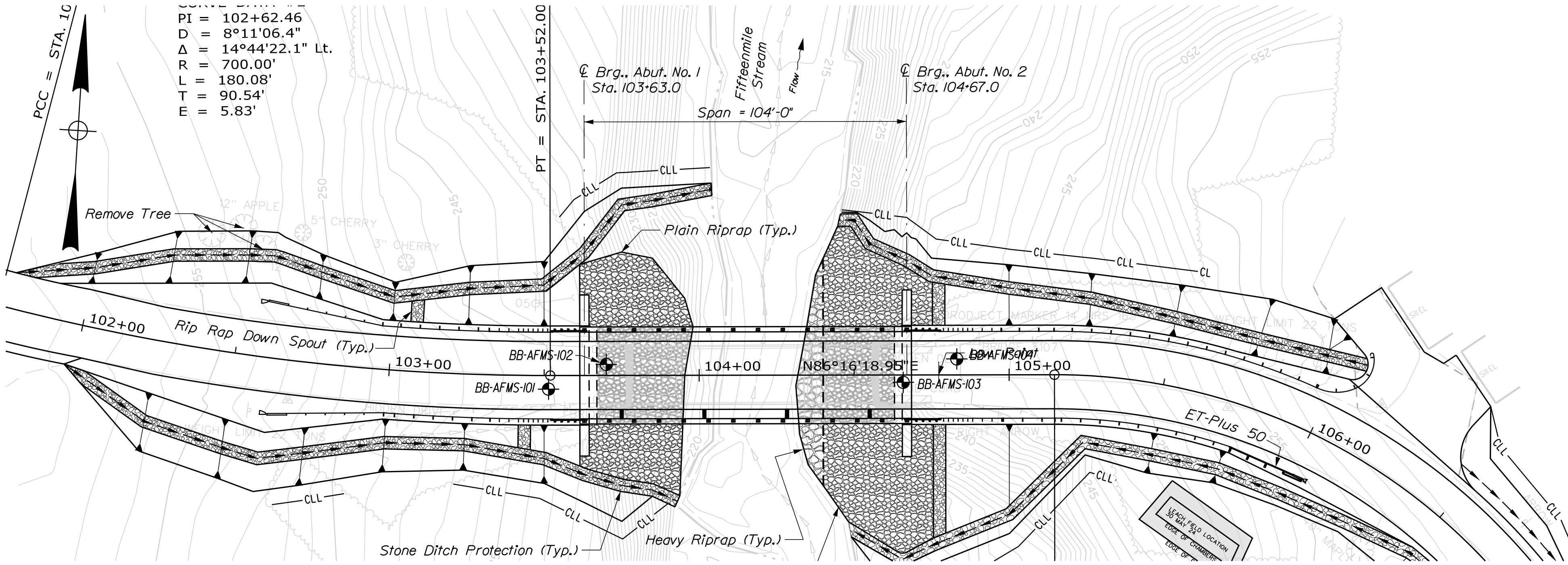
SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS 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.

Data Supplied by :

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

SCALE IN FEET

GZA	PROJ. MGR.: NVW DESIGNED BY: EAF REVIEWED BY: ARB OPERATOR: EAF DATE: 07-24-2024	**LOCUS PLAN**	JOB NO. 09.0026189.01
		PUDDLE DOCK BRIDGE ALBION, MAINE	FIGURE NO. 1



BORING LOCATION PLAN

NOTES:

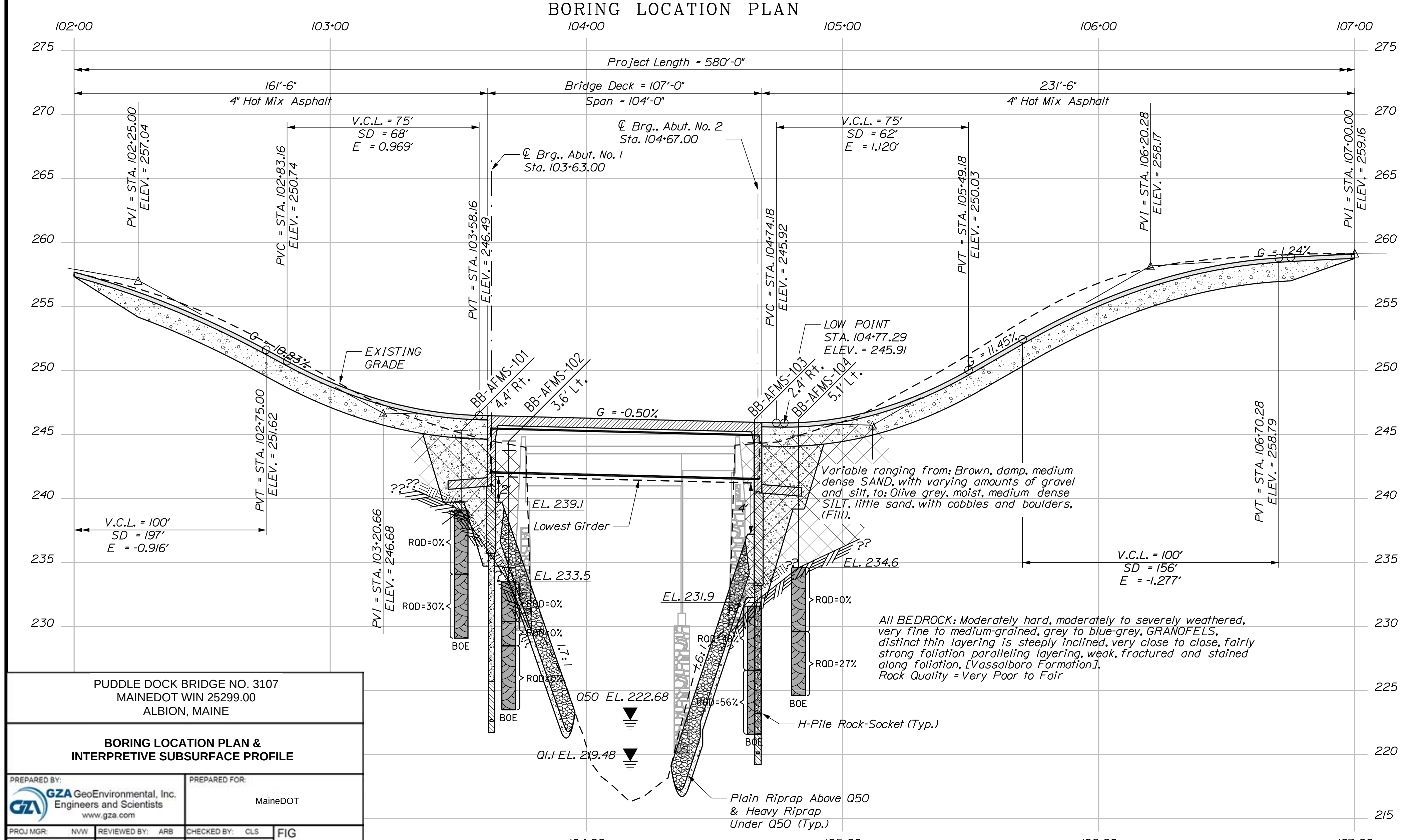
1) Base map developed from electronic files (Geoplan.dgn, Topo.dgn, Text.dgn, Contours.dgn and Bridge.dgn).

2) The as-drilled locations and elevations of the BB-AFMS-100 series test borings were surveyed by MaineDOT.

3) This generalized interpretive soil 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 bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

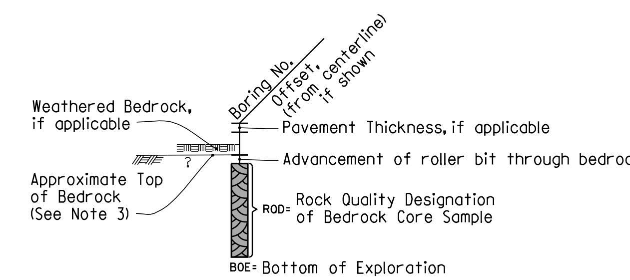
BORING LOCATION PLAN LEGEND

BB-AFMS-104 — Location and designation of BB-AFMS-100 series borings that were performed by MaineDOT between December 21 and 23, 2022.



INTERPRETIVE SUBSURFACE PROFILE

INTERPRETIVE SUBSURFACE PROFILE LEGEND



SCALE

HORIZ 25 0 25 50

VERT 5 0 5 10

PUDDLE DOCK BRIDGE NO. 3107 MAINEDOT WIN 25299.00 ALBION, MAINE	
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE	
PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com	PREPARED FOR: MaineDOT
PROJ MGR: NVW / REVIEWED BY: ARB / CHECKED BY: CLS	FIG 2
DESIGNED BY: NVW / DRAWN BY: NVW / SCALE: AS SHOWN	
DATE: 9/12/2024 / PROJECT NO. 09.0026189.01 / REVISION NO. 0	SHEET NO. 2 OF 2

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

02529900

WIN 025299.00

BRIDGE NO. 3107

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	TONY BEAULIEU	
DESIGN-DETAILED	E. BREWER	J. BRUNELLE 9/22
CHECKED-REVIEWED	-----	
DESIGN2-DETAILED2	N. WILLIAMS	T. WHITE AUG 2024
DESIGN3-DETAILED3	-----	-----
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

PUDDLE DOCK BRIDGE
FIFTEENMILE STREAM

ALBION KENNEBEC

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

5

OF 34

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:8/28/2024

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 BORING LOGS

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

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

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))

Sample Container Labeling Requirements:

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

January 2020

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #3107 carries South Freedom Road over Fifteenmile Stream
Location: Albion, Maine

Boring No.: BB-AFMS-101
WIN: 25299.00

Driller:	S.W. Cole	**Elevation (ft.)**	245.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/23/2022; 09:30-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	103+51.1, 4.4 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.92	**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	244.8		6" HMA. (0.5)	
										(Off Auger Flight) Brown, damp, SAND, little gravel, little silt, occasional small cobble, (Fill).	
5	1D	12/5	5.00 - 6.00	35/55	---		RCA	239.8		Brown, damp, SAND, some gravel, little silt, (Fill). (5.5)	G#380781 A-1-b, SM WC=4.6%
	R1	60/57	6.20 - 11.20	RQD = 0%			NQ-2	239.1		Weathered ROCK. Roller Coned ahead to 6.2 ft bgs. (6.2)	
										Top of Bedrock at Elev. 239.1 ft. R1: Bedrock:Grey to blue-grey, very fine to medium-grained, GRANOFELS, moderately hard, moderately weathered, distinct thin layering is steeply inclined, very close, fairly strong foliation paralleling layering, weak, fractured and stained along foliation. [Vassalboro Formation] Rock Quality = Very Poor. R1: Core Times (min:sec) 6.2-7.2 ft (2:34) 7.2-8.2 ft (2:38) 8.2-9.2 ft (3:01) 9.2-10.2 ft (2:48) 10.2-11.2 ft (3:36) 95% Recovery (11.2)	
10	R2	60/60	11.20 - 16.20	RQD = 30%				234.1		R2: Bedrock: Similar to R1, except frequency of foliation breaks decreases with depth. [Vassalboro Formation] Rock Quality = Poor. R2: Core Times (min:sec) 11.2-12.2 ft (3:40) 12.2-13.2 ft (4:21) 13.2-14.2 ft (3:58) 14.2-15.2 ft (3:08) 15.2-16.2 ft (3:43) 100% Recovery (16.2)	
15								229.1		**Bottom of Exploration at 16.2 feet below ground surface.**	
20											
25											

Remarks:

Auto Hammer #367

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: Puddle Dock Bridge #3107 carries South Freedom Road over Fifteenmile Stream
Location: Albion, Maine

Boring No.: BB-AFMS-102
WIN: 25299.00

Driller:	S.W. Cole	**Elevation (ft.)**	244.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/23/2022; 07:30-09:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	103+69.7, 3.6 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.92	**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	243.8		9" HMA. 0.8	
5	1D	24/14	5.00 - 7.00	8/9/4/2	13	20				Brown, damp, medium dense, SAND, some gravel, some silt, occasional cobble, (Fill).	G#380782 A-1-b, SM WC=12.1%
										Dense at 8.0 ft bgs. 9.9	
10	2D	6/5	10.00 - 10.50	57/6"	---		175	234.6		Weathered ROCK. Roller Coned ahead to 11.0 ft bgs. 11.0	
							~~RCA~~	233.5			
	R1	37.2/37.2	11.00 - 14.10	RQD = 0%			NQ-2			Top of Bedrock at Elev. 233.5 ft. R1: Bedrock: Grey to blue-grey, very fine to medium-grained GRANOFELS, moderately hard, moderately to severely weathered, distinct thin layering is steeply inclined, very close, fairly strong foliation paralleling layering, weak, fractured and stained along foliation. [Vassalboro Formation] Rock Quality = Very Poor. R1: Core Times (minsec) 11.0-12.0 ft (3:10) 12.0-13.0 ft (3:34) 13.0-14.0 ft (3:42) 14.0-14.1 ft (1:00) Core Blocked 100% Recovery 14.1	
	R2	22.8/22.8	14.10 - 16.00	RQD = 0%				230.4		R2: Bedrock: Similar to R1. [Vassalboro Formation] Rock Quality = Very Poor. R2: Core Times (min:sec) 14.1-15.1 ft (3:23) 15.1-16.0 ft (3:19) Core Blocked 100% Recovery 16.0	
15	R3	60/55	16.00 - 21.00	RQD = 0%				228.5		R3: Bedrock: Similar to R1. [Vassalboro Formation] Rock Quality = Very Poor. R3: Core Times (min:sec) 16.0-17.0 ft (2:43) 17.0-18.0 ft (2:45)	
20								223.5			
25											

Remarks:

Auto Hammer #367

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: Puddle Dock Bridge #3107 carries South Freedom Road over Fifteenmile Stream
Location: Albion, Maine

Boring No.: BB-AFMS-102
WIN: 25299.00

Driller:	S.W. Cole	**Elevation (ft.)**	244.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/23/2022; 07:30-09:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	103+69.7, 3.6 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.92	**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										18.0-19.0 ft (2:46) 19.0-20.0 ft (3:03) 20.0-21.0 ft (3:26) 92% Recovery	
										21.0	
										Bottom of Exploration at 21.0 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #367

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: Puddle Dock Bridge #3107 carries South Freedom Road over Fifteenmile Stream
Location: Albion, Maine

Boring No.: BB-AFMS-103
WIN: 25299.00

Driller:	MaineDOT	**Elevation (ft.)**	244.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/21/2022; 07:45-09:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+65.6, 2.4 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	Lost H2O after coring
Hammer Efficiency Factor: 0.906		**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				
5	1D	24/21	5.00 - 7.00	3/3/5/4	8	12				Brown, damp, medium dense, fine to medium SAND, some silt, trace gravel, (Fill).	G#380783 A-2-4, SM WC=13.9%
										Boulder from 8.0-9.0 ft bgs.	
10	2D	1.2/1.2	10.00 - 10.10	33(1.2")	---					Brown, damp, Silty SAND.	
								233.4		11.0	
	S1						40			Sample of weathered rock taken from Auger 11.0-12.0 ft bgs.	
										a149 blows for 0.5 ft.	
										12.5	
	R1	60/60	12.80 - 17.80	RQD = 48%			a149 NQ-2	231.9 231.6		Top of Bedrock at Elev. 231.9 ft.	
										12.8	
15										R1: Bedrock: Grey to blue-grey, very fine to medium-grained, GRANOFELS, moderately hard, distinct layering is steeply inclined, fairly strong foliation paralleling layering, weak and stained along foliation. Near vertical fractured across foliation, annealed, weathered, close. [Vassalboro Formation] Rock Quality = Poor R1: Core Times (min:sec) 12.8-13.8 ft (1:24) 13.8-14.8 ft (1:47) 14.8-15.8 ft (1:54) 15.8-16.8 ft (2:06) 16.8-17.8 ft (3:19) 100% Recovery	
	R2	60/60	17.80 - 22.80	RQD = 56%				226.6			
20										17.8	
								221.6		R2: Bedrock: Similar to R1, except core run increases in competency with depth, and two joint sets steep and moderately dipping noted, fresh and with rock flour/gouge. Rock Quality = Fair R2: Core Times (min:sec) 17.8-18.8 ft (3:18) 18.8-19.8 ft (2:52) 19.8-20.8 ft (1:49) 20.8-21.8 ft (2:17)	GTX 20.71-21.09 q_p=15,945 psi GTX 22.33-22.71 q_p=15,930 psi
25											

Remarks:

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: Puddle Dock Bridge #3107 carries South Freedom Road over Fifteenmile Stream
Location: Albion, Maine

Boring No.: BB-AFMS-103
WIN: 25299.00

Driller:	MaineDOT	**Elevation (ft.)**	244.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/21/2022; 07:45-09:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+65.6, 2.4 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	Lost H2O after coring
Hammer Efficiency Factor:	0.906	**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										21.8-22.8 ft (4:24) 100% Recovery	
										22.8	
										Bottom of Exploration at 22.8 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

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: Puddle Dock Bridge #3107 carries South Freedom Road over Fifteenmile Stream
Location: Albion, Maine

Boring No.: BB-AFMS-104
WIN: 25299.00

Driller:	MaineDOT	**Elevation (ft.)**	244.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/21/2022; 10:00-12:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+82.8, 5.1 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.906	**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				
5	1D	24/24	5.00 - 7.00	4/6/6/7	12	18				Olive grey, moist, medium dense, SILT, little sand, (Fill).	G#380784 A-4, CL WC=20.1%
10	2D R1	3.6/3 60/30	10.00 - 10.30 10.30 - 15.30	25(3.6") RQD = 0%	---		NQ-2	234.6		Brown, moist, fine to coarse SAND, some gravel, little silt, (Fill). 10.3 Top of Bedrock at Elev. 234.6 ft. R1: Bedrock: Grey to blue-grey, very fine to medium-grained, GRANOFELS, moderately hard, distinct layering is steeply inclined, weak, weathered and stained. [Vassalboro Formation] Rock Quality = Very Poor R1; Core Times (min:sec) 10.3-11.3 ft (2:05) 11.3-12.3 ft (3:15) 12.3-13.3 ft (3:50) 13.3-14.3 ft (2:17) 14.3-15.3 ft (3:43) 50% Recovery	
15	R2	60/60	15.30 - 20.30	RQD = 27%				229.6		15.3 R2: Bedrock: Similar to R1, except weakness along foliation not as prominent and competency of rock increases with depth. Near vertical joint set noted across layering, annealed with some weathering. [Vassalboro Formation] Rock Quality = Poor R2: Core Times (min:sec) 15.3-16.3 ft (1:15) 16.3-17.3 ft (1:53) 17.3-18.3 ft (4:03) 18.3-19.3 ft (1:14) 19.3-20.3 ft (2:18) 100% Recovery	
20								224.6		20.3 **Bottom of Exploration at 20.3 feet below ground surface.**	
25											

Remarks:

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 – ROCK CORE PHOTOGRAPH LOG

MaineDOT

Puddle Dock Bridge #3107 Carries South Freedom Road Over Fifteen Mile Stream
Albion, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AFMS-102	R1	11.0-14.1	37.2	37.2	0	0	GRANOFELS	1
BB-AFMS-102	R2	14.1-16.0	22.8	22.8	0	0	GRANOFELS	1
BB-AFMS-102	R3	16.0-21.0	60	55	0	0	GRANOFELS	2
BB-AFMS-101	R1	6.2-11.2	60	60	0	0	GRANOFELS	3
BB-AFMS-101	R2	11.2-16.2	60	60	18	30	GRANOFELS	4

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.

MaineDOT

Puddle Dock Bridge #3107 Carries South Freedom Road Over Fifteen Mile Stream
Albion, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AFMS-103	R1	12.8-17.8	60	60	29	48	GRANOFELS	1
BB-AFMS-103	R2	17.8-22.8	60	60	34	56	GRANOFELS	2
BB-AFMS-104	R1	10.3-15.3	60	30	0	0	GRANOFELS	3
BB-AFMS-104	R2	15.3-20.3	60	60	16	27	GRANOFELS	4

Notes: 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.

APPENDIX D – LABORATORY TEST RESULTS

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Albion   Work Number: 25299.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-AFMS-101, 1D	103+51.1	4.4 Rt.	5.0-6.0	380781	1	4.6			SM	A-1-b	II
BB-AFMS-102, 1D	103+69.7	3.6 Lt.	5.0-7.0	380782	1	12.1			SM	A-1-b	II
BB-AFMS-103, 1D	104+65.6	2.4 Rt.	5.0-7.0	380783	1	13.9			SM	A-2-4	II
BB-AFMS-104, 1D	104+82.8	5.1 Lt.	5.0-7.0	380784	1	20.1			CL	A-4	IV

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98   NP = Non Plastic

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380781	**BB-AFMS-101/1D**	**GEOTECHNICAL (DISTURBED)**	**12/23/2022**	**2/1/2023**

Sample Type: **GEOTECHNICAL** Location: Station: **103+51.1** Offset, ft: **4.4** **RT** Dbfg, ft: **5.0-6.0**

WIN/Town **025299.00 - ALBION** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**93.5**
⅜ in. [9.5 mm]	**88.6**
¼ in. [6.3 mm]	**77.3**
No. 4 [4.75 mm]	**69.9**
No. 10 [2.00 mm]	**50.5**
No. 20 [0.850 mm]	**36.9**
No. 40 [0.425 mm]	**30.3**
No. 60 [0.250 mm]	**26.9**
No. 100 [0.150 mm]	**24.1**
No. 200 [0.075 mm]	**19.3**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**4.6**

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/3/2023**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380782	BB-AFMS-102/1D	GEOTECHNICAL (DISTURBED)	12/23/2022	2/1/2023

Sample Type: **GEOTECHNICAL** Location: Station: **103+69.7** Offset, ft: **3.6 LT** Dbfg, ft: **5.0-7.0**

WIN/Town **025299.00 - ALBION** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method: Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	83.2
½ in. [12.5 mm]	76.9
⅜ in. [9.5 mm]	75.2
¼ in. [6.3 mm]	70.6
No. 4 [4.75 mm]	67.4
No. 10 [2.00 mm]	58.1
No. 20 [0.850 mm]	49.5
No. 40 [0.425 mm]	41.6
No. 60 [0.250 mm]	35.2
No. 100 [0.150 mm]	30.0
No. 200 [0.075 mm]	22.2

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	12.1

Consolidation (T 216)

Trimmings, Water Content, %:

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/3/2023**

Paper Copy: Lab File; Project File; Geotech File

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380783	**BB-AFMS-103/1D**	**GEOTECHNICAL (DISTURBED)**	**12/23/2022**	**2/1/2023**

Sample Type: **GEOTECHNICAL** Location: Station: **104+65.6** Offset, ft: **2.4** **RT** Dbfg, ft: **5.0-7.0**

WIN/Town **025299.00 - ALBION** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method: Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**98.7**
⅜ in. [9.5 mm]	**97.8**
¼ in. [6.3 mm]	**96.0**
No. 4 [4.75 mm]	**94.9**
No. 10 [2.00 mm]	**89.5**
No. 20 [0.850 mm]	**80.0**
No. 40 [0.425 mm]	**67.0**
No. 60 [0.250 mm]	**53.2**
No. 100 [0.150 mm]	**42.4**
No. 200 [0.075 mm]	**29.5**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**13.9**

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/3/2023**

Paper Copy: Lab File; Project File; Geotech File

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380784	BB-AFMS-104/1D	GEOTECHNICAL (DISTURBED)	12/23/2022	2/1/2023

Sample Type: **GEOTECHNICAL** Location: Station: **104+82.8** Offset, ft: **5.1 LT** Dbfg, ft: **5.0-7.0**

WIN/Town **025299.00 - ALBION** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method: Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	98.8
No. 40 [0.425 mm]	97.4
No. 60 [0.250 mm]	96.5
No. 100 [0.150 mm]	95.4
No. 200 [0.075 mm]	86.4

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	20.1

Consolidation (T 216)

Trimmings, Water Content, %:

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

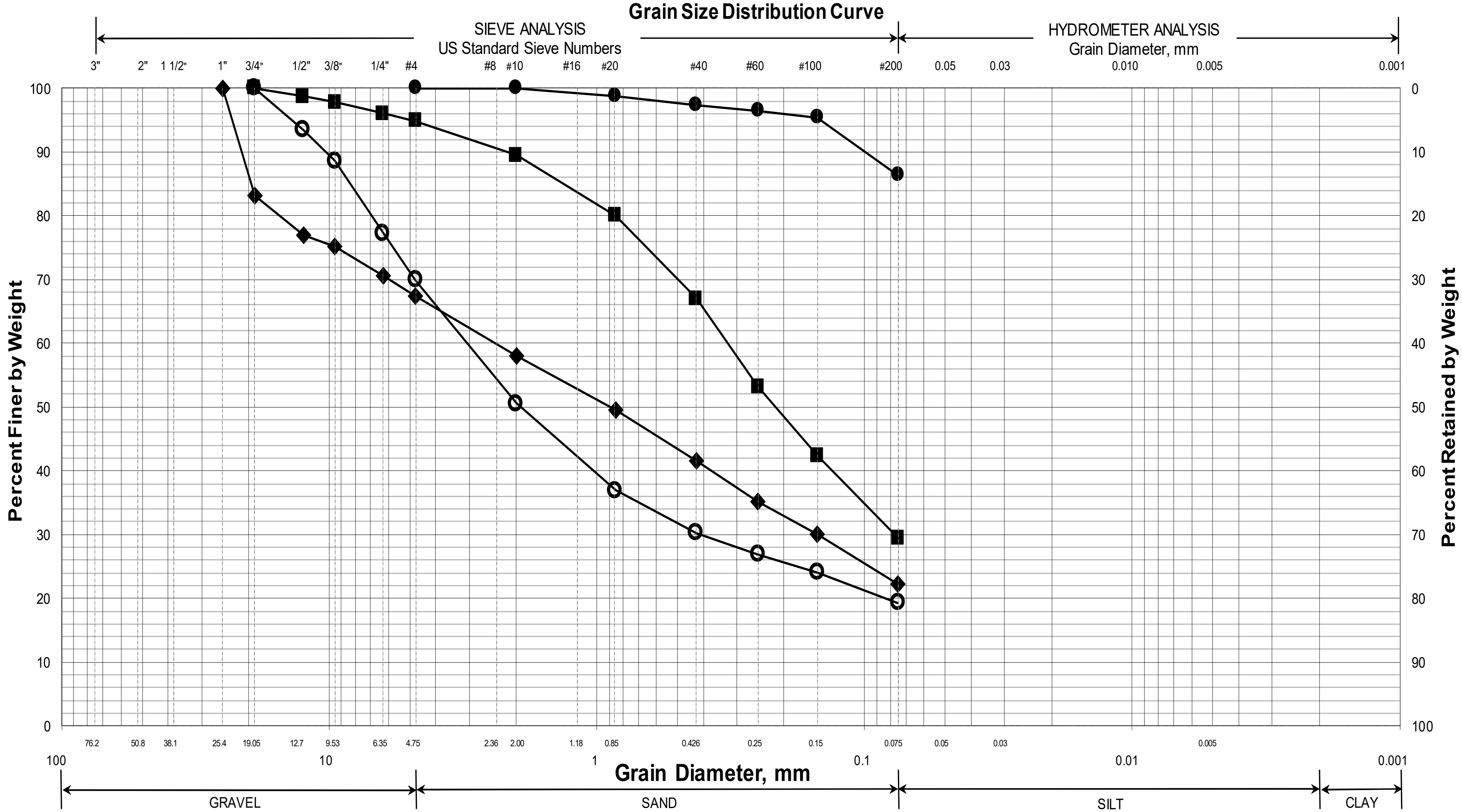
Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/3/2023**

Paper Copy: Lab File; Project File; Geotech File

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

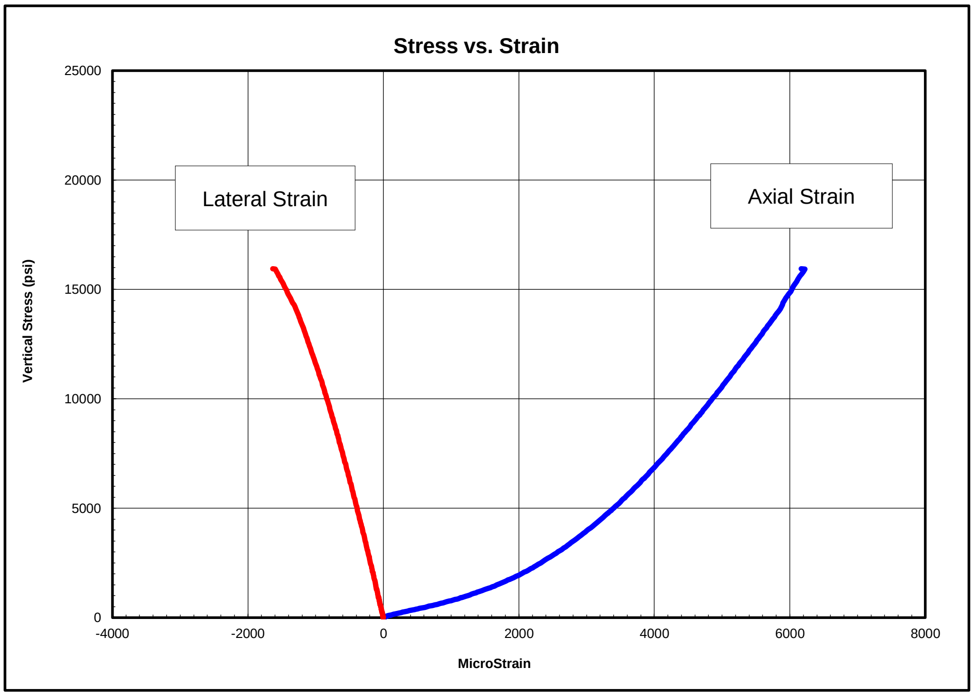
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-AFMS-101/1D	103+51.1	4.4 RT	5.0-6.0	SAND, some gravel, little silt.	4.6			
◆	BB-AFMS-102/1D	103+69.7	3.6 LT	5.0-7.0	SAND, some gravel, some silt.	12.1			
■	BB-AFMS-103/1D	104+65.6	2.4 RT	5.0-7.0	SAND, some silt, trace gravel.	13.9			
●	BB-AFMS-104/1D	104+82.8	5.1 LT	5.0-7.0	SILT, little sand.	20.1			
▲									
×									

WIN
025299.00
Town
Albion
Reported by/Date
WHITE, TERRY A 2/6/2023

SHEET 1

Client:	Maine Department of Transportation
Project Name:	Puddle Dock Bridge
Project Location:	Albion, ME
GTX #:	317079
Test Date:	5/2/2023
Tested By:	jab
Checked By:	jsc
Boring ID:	R2
Sample ID:	BB-AFMS-103, R2a
Depth, ft:	20.71-21.09
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 15,945 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1600-5800	2,220,000	0.18
5800-10100	3,550,000	0.32
10100-14400	4,120,000	0.46

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Maine Department of Transportation	Test Date:	5/1/2023
Project Name:	Puddle Dock Bridge	Tested By:	jab
Project Location:	Albion, ME	Checked By:	smd
GTX #:	317079		
Boring ID:	R2		
Sample ID:	BB-AFMS-103, R2a		
Depth:	20.71-21.09 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.47	4.47	4.47	
Specimen Diameter, in:	1.98	1.98	1.98	
Specimen Mass, g:	612.05			
Bulk Density, lb/ft^3	169	**Minimum Diameter Tolerence Met?**		**YES**
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**		**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

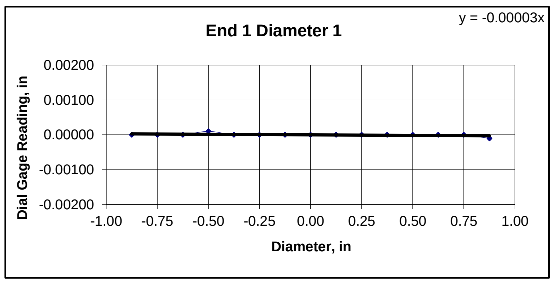
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

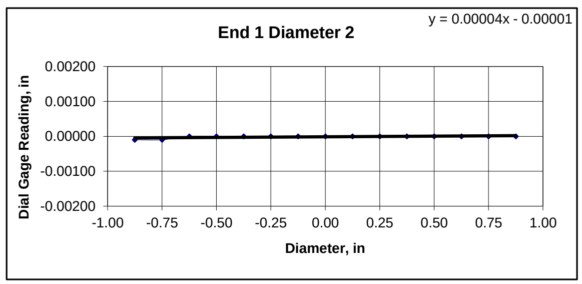
Difference between max and min readings, in:
0° = 0.00020 90° = 0.00010

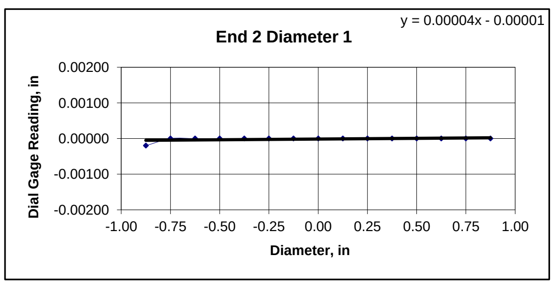
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

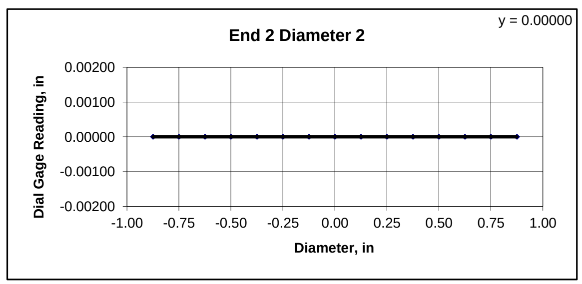
Difference between max and min readings, in:
0° = 0.0002 90° = 0

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00010
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00003
Angle of Best Fit Line: 0.00180

End 2:
Slope of Best Fit Line 0.00004
Angle of Best Fit Line: 0.00229

Maximum Angular Difference: 0.00049

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00004
Angle of Best Fit Line: 0.00213

End 2:
Slope of Best Fit Line 0.00000
Angle of Best Fit Line: 0.00000

Maximum Angular Difference: 0.00213

Parallelism Tolerance Met? **YES**
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00000	1.980	0.00000	0.000	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

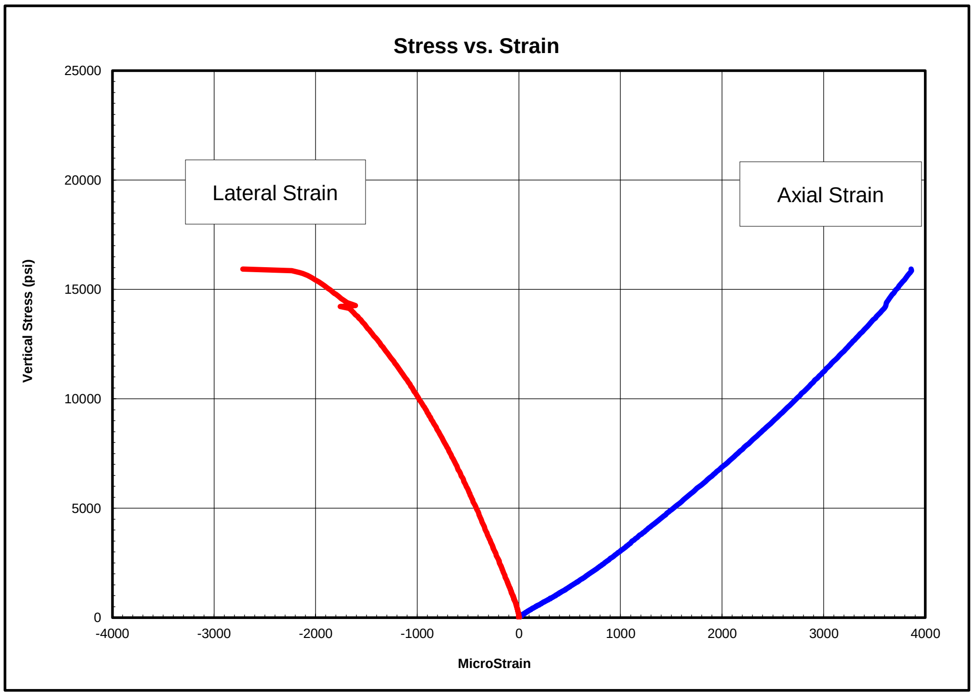
Client:	Maine Department of Transportation
Project Name:	Puddle Dock Bridge
Project Location:	Albion, ME
GTX #:	317079
Test Date:	5/2/2023
Tested By:	jab
Checked By:	smd
Boring ID:	R2
Sample ID:	BB-AFMS-103, R2a
Depth, ft:	20.71-21.09

After cutting and grinding

After break

Client:	Maine Department of Transportation
Project Name:	Puddle Dock Bridge
Project Location:	Albion, ME
GTX #:	317079
Test Date:	5/2/2023
Tested By:	jab
Checked By:	jsc
Boring ID:	R2
Sample ID:	BB-AFMS-103, R2b
Depth, ft:	22.33-22.71
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 15,930 psi

The strain values recorded within the third stress range for this test produce values of Poisson's Ratio that exceed maximum values found in rocks.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1600-5800	3,630,000	0.33
5800-10100	4,180,000	0.48
10100-14300	4,810,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Maine Department of Transportation	Test Date:	5/1/2023
Project Name:	Puddle Dock Bridge	Tested By:	jab
Project Location:	Albion, ME	Checked By:	smd
GTX #:	317079		
Boring ID:	R2		
Sample ID:	BB-AFMS-103, R2b		
Depth:	22.33-22.71 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.36	4.35	4.36		
Specimen Diameter, in:	1.99	1.98	1.99		
Specimen Mass, g:	600.17				
Bulk Density, lb/ft^3	169			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010

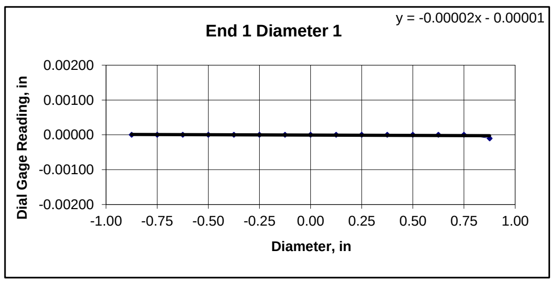
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00010

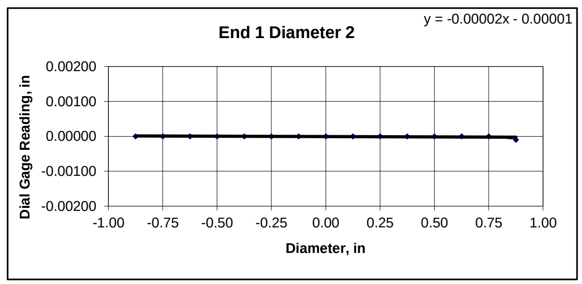
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

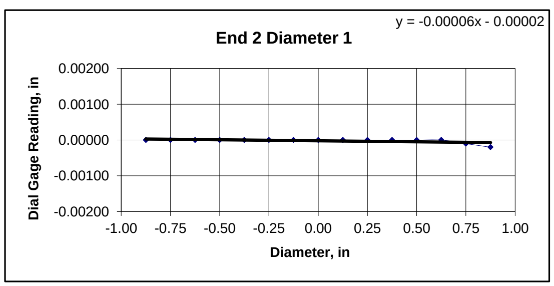
Difference between max and min readings, in:
0° = 0.0002 90° = 0

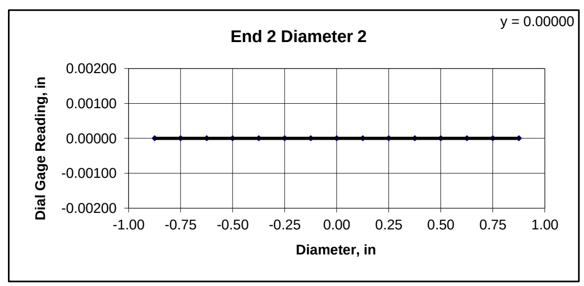
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00010

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00002
	Angle of Best Fit Line:	0.00115
End 2:		
	Slope of Best Fit Line	0.00006
	Angle of Best Fit Line:	0.00327
Maximum Angular Difference:		0.00213

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00002
	Angle of Best Fit Line:	0.00115
End 2:		
	Slope of Best Fit Line	0.00000
	Angle of Best Fit Line:	0.00000
Maximum Angular Difference:		0.00115

Parallelism Tolerance Met? **YES**
Spherically Seated

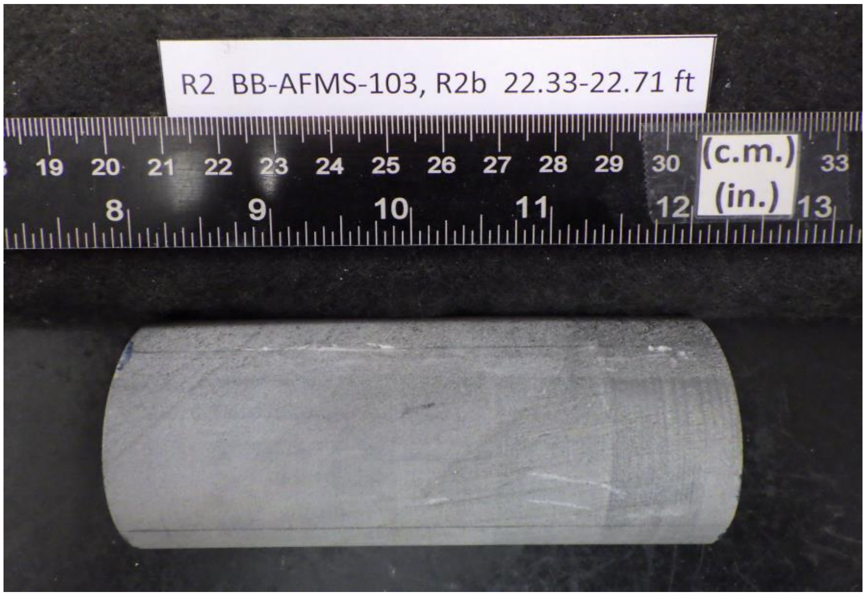
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.985	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00010	1.985	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00020	1.985	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00000	1.985	0.00000	0.000	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Maine Department of Transportation
Project Name:	Puddle Dock Bridge
Project Location:	Albion, ME
GTX #:	317079
Test Date:	5/2/2023
Tested By:	jab
Checked By:	smd
Boring ID:	R2
Sample ID:	BB-AFMS-103, R2b
Depth, ft:	22.33-22.71



After cutting and grinding

After break

APPENDIX E – CALCULATIONS

Seismic Calculation

Seismic Site Class Calculation Summary

Project: Puddle Dock Bridge over Fifteenmile Stream **Project No.:** 09.0026189.01
Location: Albion, Maine
Evaluated By/Date: NVW **Date** 8/7/2024
Checked By/Date: ARB **Date**

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

Subsurface Data: Borings BB-AFMS-102 through -104 were drilled by MaineDOT between December 21 and 23, 2022.

Assumptions: Soil borings extended to depths between 16 and 23 feet below the roadway level and bedrock was encountered in the soil borings.

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 $\overline{N}$ and $\overline{s}_u$ 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 $\overline{v}_s$.

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-ASFM-102	BB-ASFM-103	BB-ASFM-104	Average
N-Value	20	12	18	17

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.

Puddle Dock Bridge over Fifteenmile Stream
Albion, Maine

Calculated By: NVW   Date: 8/7/2024
Checked By: ARB   Date: 1/0/1900

INPUT

Exploration ID: **All Borings**   Depth of Boring: Varies

Depth to Bedrock: Varies

EQUATIONS

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

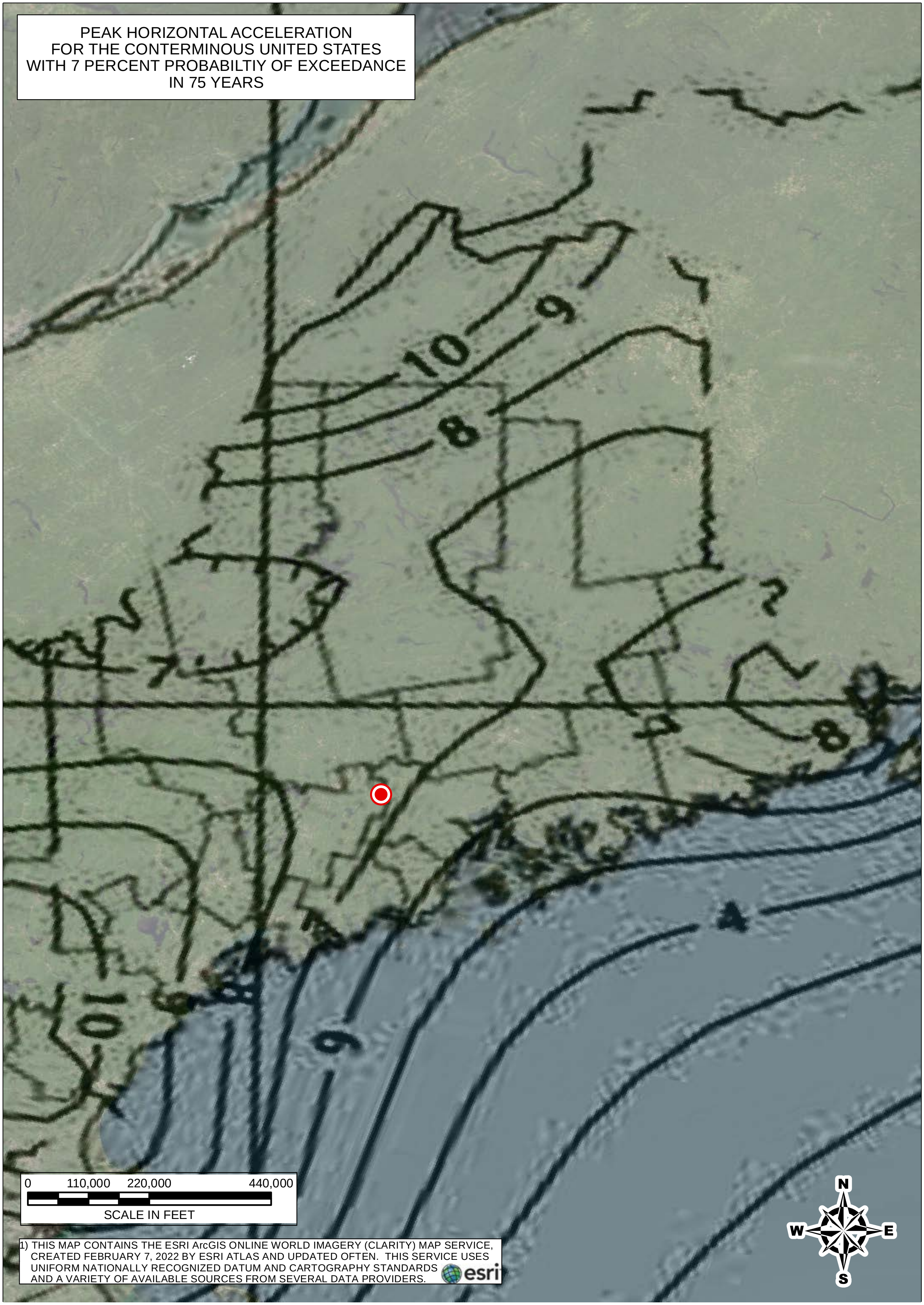
where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated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

Note: Because each boring had only one blowcount recorded through the overburden, d_i was assumed to be the full thickness of the overburden. (unless noted otherwise)

CALCULATION

Boring ID	Soil Strata	SPT Interval Depth Top, ft	SPT Interval Depth Bottom, ft	SPT N-value	d_i	d_i / N_i	$\overline{N}$	Comment
BB-ASFM-102	Fill	5.0	7.0	20	9.9	0.50	20	One Spoon was sampled through the overburden in each boring.
BB-ASFM-103		5.0	7.0	12	11.0	0.92	12	
BB-ASFM-104		5.0	7.0	18	10.3	0.57	18	
						Average	16.7	

Seismic Design Parameters



PEAK HORIZONTAL ACCELERATION
FOR THE CONTERMINOUS UNITED STATES
WITH 7 PERCENT PROBABILTIY OF EXCEEDANCE
IN 75 YEARS
10
9
8
7
7
2
7
8
10
4
6
0 110,000 220,000 440,000
SCALE IN FEET
1) THIS MAP CONTAINS THE ESRI ArcGIS ONLINE WORLD IMAGERY (CLARITY) MAP SERVICE, CREATED FEBRUARY 7, 2022 BY ESRI ATLAS AND UPDATED OFTEN. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS.
esri
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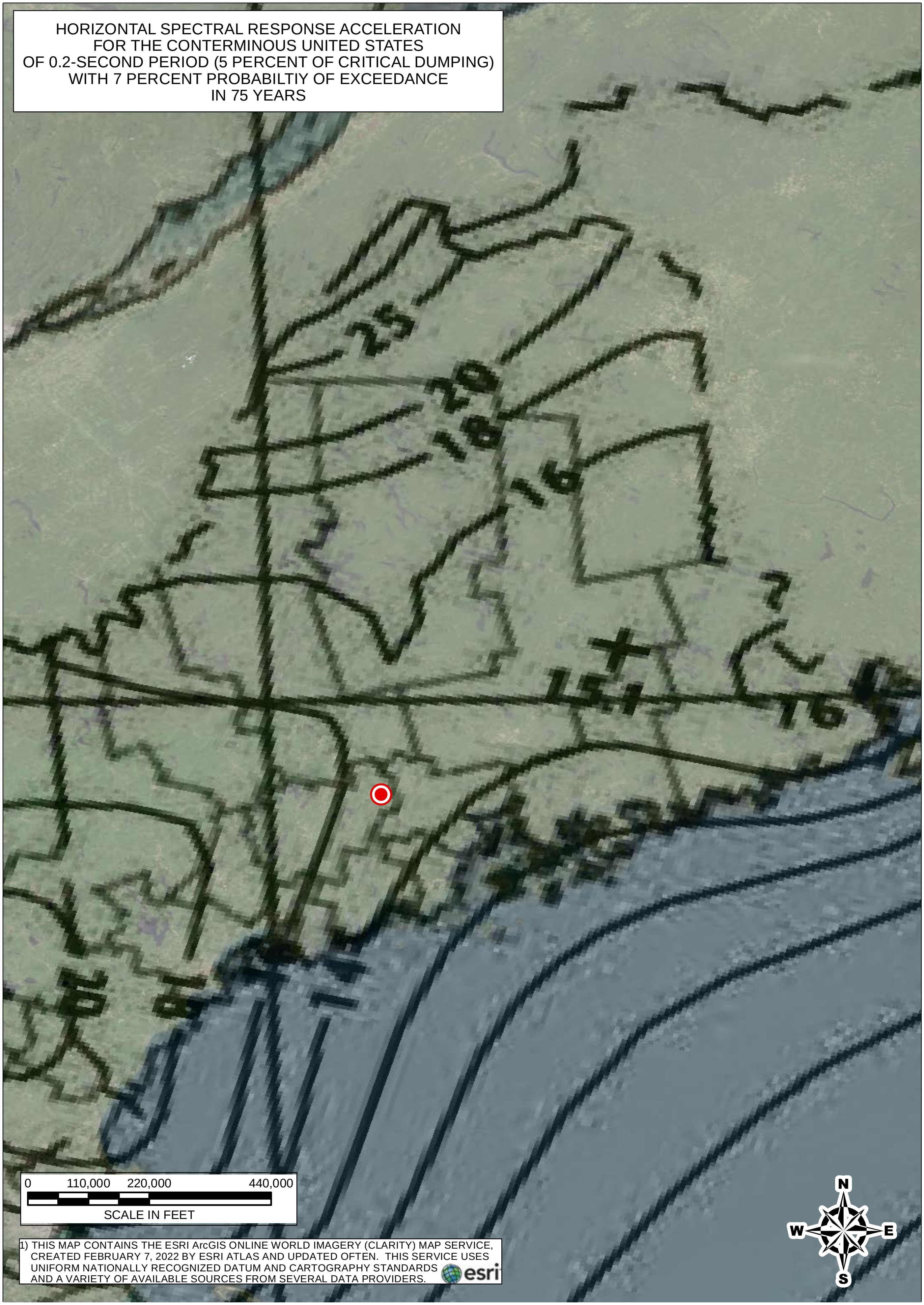
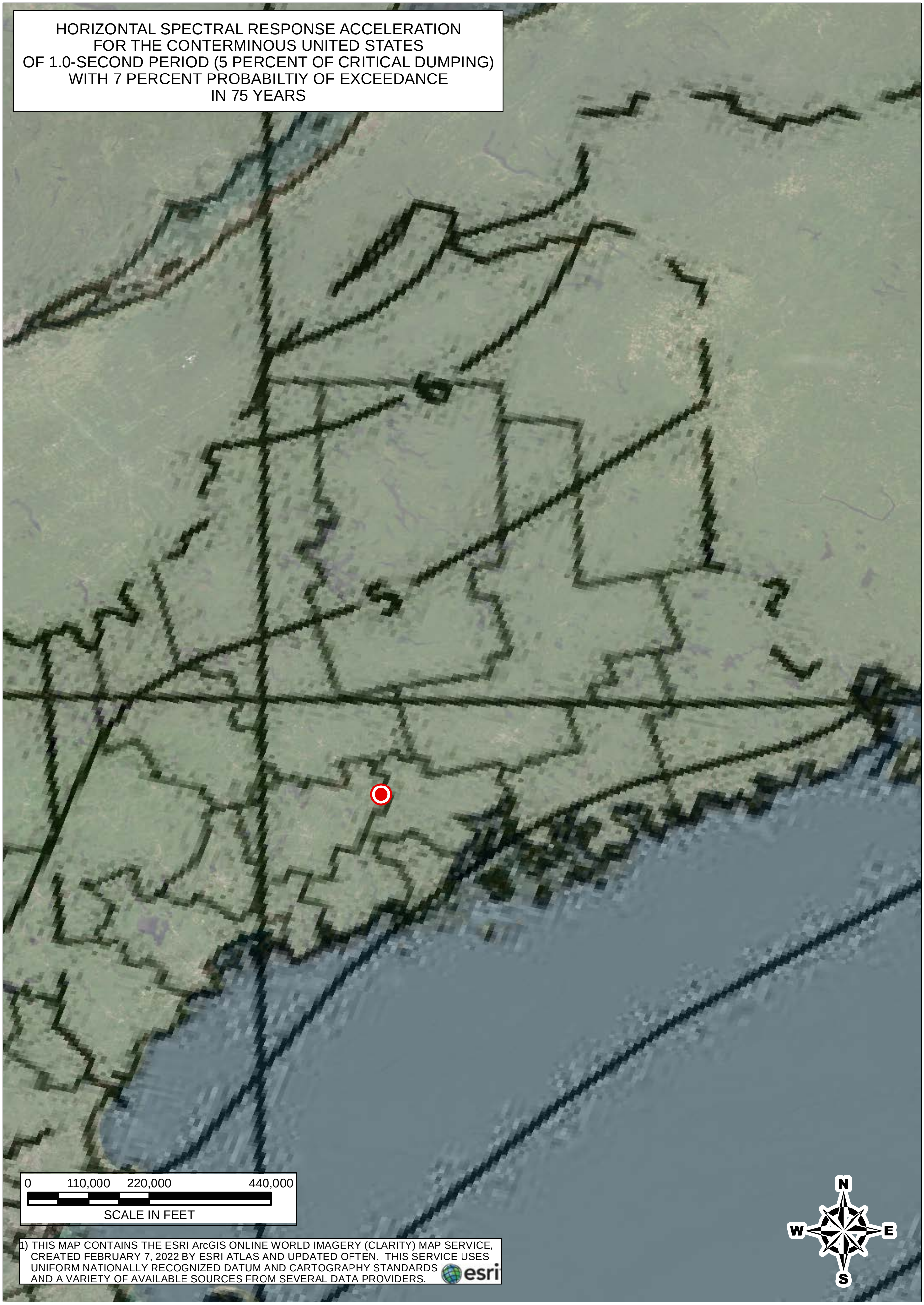
HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND PERIOD (5 PERCENT OF CRITICAL DUMPING) WITH 7 PERCENT PROBABILTIY OF EXCEEDANCE IN 75 YEARS



1) THIS MAP CONTAINS THE ESRI ArcGIS ONLINE WORLD IMAGERY (CLARITY) MAP SERVICE, CREATED FEBRUARY 7, 2022 BY ESRI ATLAS AND UPDATED OFTEN. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS.

HORIZONTAL SPECTRAL RESPONSE ACCELERATION
FOR THE CONTERMINOUS UNITED STATES
OF 1.0-SECOND PERIOD (5 PERCENT OF CRITICAL DUMPING)
WITH 7 PERCENT PROBABILTIY OF EXCEEDANCE
IN 75 YEARS



1) THIS MAP CONTAINS THE ESRI ArcGIS ONLINE WORLD IMAGERY (CLARITY) MAP SERVICE, CREATED FEBRUARY 7, 2022 BY ESRI ATLAS AND UPDATED OFTEN. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS.

Site Class Seismic Design Parameters

Puddle Dock Bridge #3107 over Fifteenmile Stream
Albion, Maine
GZA Job#: 09.0026189.01

Seismic Interpolation Coefficients		
Seismic Parameter	**Interpolated Value from Maps[1]**	**Design Parameter**
Horizontal Peak Ground Acceleration Coefficient (PGA)	7.2	0.07
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s, Ss	15.0	0.15
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s, S1	4.3	0.04

Notes:

1. AASHTO Figures 3.10.2.1-1,-2, and -3 were overlaid within the ESRI ArcGIS Online World Imagery (Clarity) Map Service. Coefficients were interpolated between lines on these figures as presented in pages 1 through 3 of this calculation.

	For Site Class D
F_{PGA} =	1.6
F_a =	1.6
F_v =	2.4

Therefore:

$A_s = F_{PGA} \times PGA =$	0.12
$S_{DS} = F_a \times S_s =$	0.24
$S_{D1} = F_v \times S_1 =$	0.10

Summary:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.12
SDs (Period = 0.2 sec)	0.24
SD1 (Period = 1.0 sec)	0.10

Frost Calculation

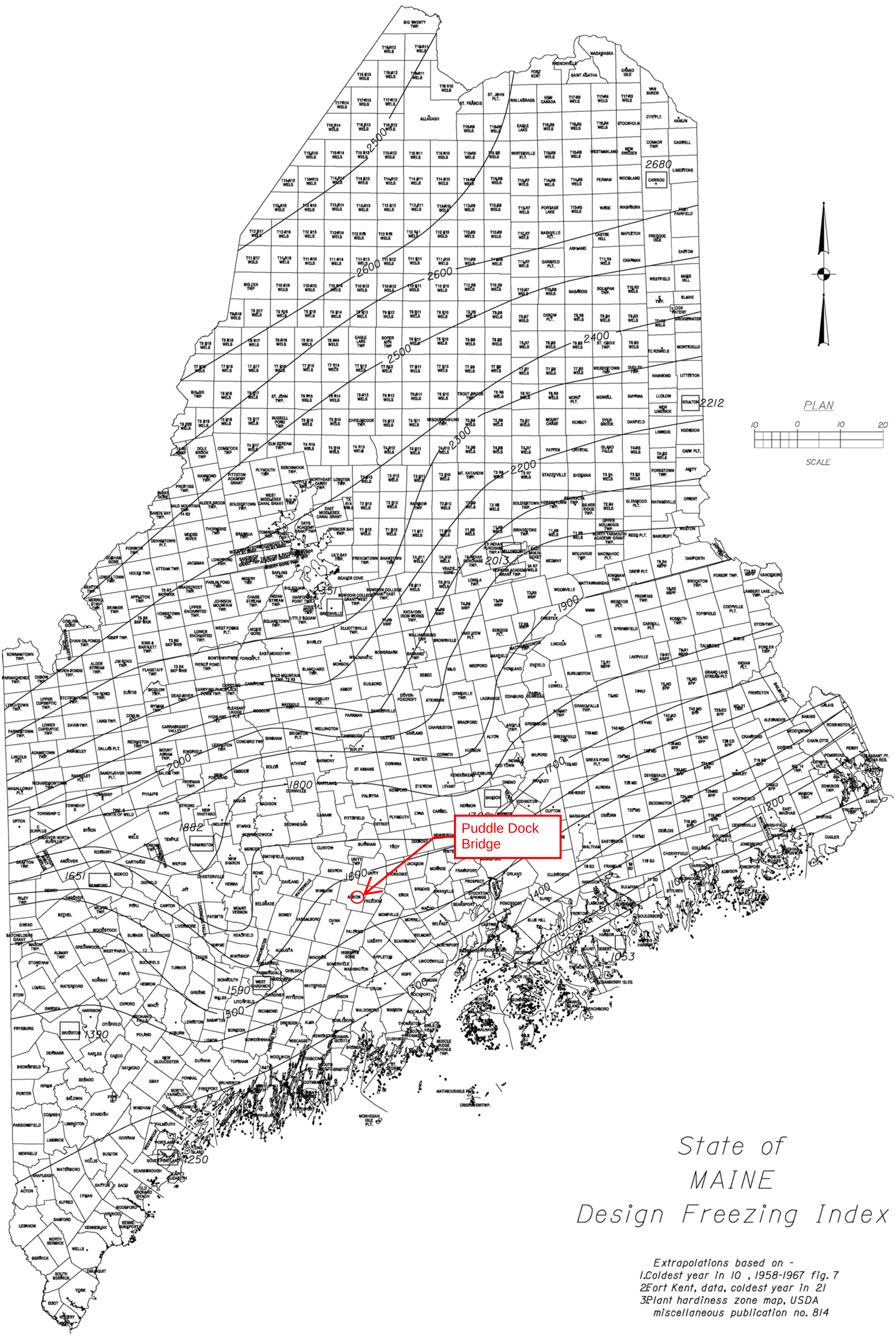


Figure 5-1 Maine Design Freezing Index Map

Table 5-1 Depth of Frost Penetration

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	[illegible] 83.7" = 7.0'	[illegible]	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 1,560 the estimated depth of frost penetration is 84 inches.

Lateral Earth Pressures

Include << R:\Region0\Bridge\Public_Teams\South\Pre Award Projects\Albion, Puddle Dock Bridge #3107 WIN 25299.00\Design\Substructure Design\Loads on Piles - Albion.mcdx

Load Factors

$\gamma_{EH} := 1.5$ for Strength Table 3.4.1-2

$\gamma_{EH.Serv} := 1$ for Service I Table 3.4.1-1

$\gamma_{LS} := 1.75$ live load surcharge factor Table 3.4.1-1

Properties

$\gamma_{soil} = 0.125 \dfrac{kip}{ft^3}$ unit weight of granular borrow BDG Table 3-3

$\gamma_w := 0.0625 \dfrac{kip}{ft^3}$ unit weight of water

Horizontal Earth Pressure Loads - Abutment Bottom

This is the horizontal loads on the abutment below the construction joint and approach slab.

Abutment Elevations

Abutment 1

$Elev_{bot1} := 235.73\ ft$ abutment bottom elevation from Microstation

$Elev_{joint1} := 241.47\ ft$ max elevation of the construction joint

$Elev_{joint.min1} := 240.50\ ft$ min elevation of the construction joint

$Elev_{top1} := 246.79\ ft$ abut top elevation from Microstation, 6" below top of parapet

$H_1 := Elev_{top1} - Elev_{bot1} = 11.06\ ft$ total abutment height

Abutment 2

$Elev_{bot2} := 233.21\ ft$ abutment bottom elevation from Microstation

$Elev_{joint2} := 240.95\ ft$ max elevation of the construction joint

$Elev_{joint.min2} := 240.10\ ft$ min elevation of the construction joint

$Elev_{top2} := 246.26\ ft$ abut top elevation from Microstation, 6" below top of parapet

$H_2 := Elev_{top2} - Elev_{bot2} = 13.05\ ft$ total abutment height

Water

$Elev_{Q50} := 222.68\ ft$

Earth Pressure Coefficient

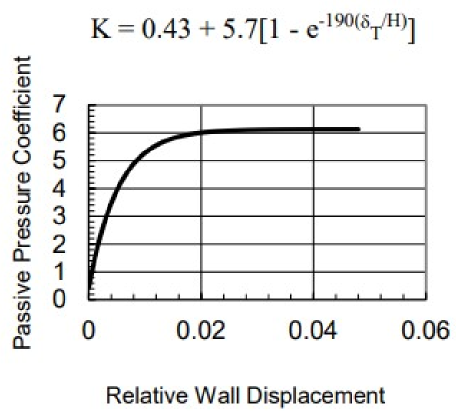
$\delta_T := \Delta_{pile} = 0.47\ in$ thermal movement at one abutment

$H_{abut} := min(H_1, H_2)$ choose minimum of the 2 abuts, since that controls

$H_{abut} = 11.06\ ft$

passive earth pressure coefficient

$$K_p := 0.43 + 5.7 \cdot \left(1 - e^{-190 \cdot \left(\frac{\delta_T}{H_{abut}}\right)}\right) = 3.21$$



MassDOT LRFD Figure 3.10.8-1

$\gamma_{soil} = 0.125\ \frac{kip}{ft^3}$ granular borrow unit weight

$\gamma_w := 0.0625\ \frac{kip}{ft^3}$ unit weight of water

Abutment 1

$H_{water} := 3\ ft$ BDG 3.6.2

$$z_{w1} := \begin{array}{l} \text{if } Elev_{Q50} - Elev_{bot1} \le 0 \\ \quad \| \ z_{w1} \leftarrow H_{water} \\ \text{else} \\ \quad \| \ z_{w1} \leftarrow Elev_{Q50} - Elev_{bot1} + H_{water} \end{array} = 3\ ft$$

even if the Q50 elevation is below the bottom of the abutment, assume 3ft worth of pore water pressure to be conservative

$p_{w1} := K_p \cdot \gamma_w \cdot z_{w1} = 0.6\ ksf$ acts on bottom 3' of abutment (3.11.5.1-1)

$p_{soil1} := K_p \cdot \gamma_{soil} \cdot H_1 = 4.43\ ksf$ acts along full abutment (3.11.5.1-1)

Abutment 2

$H_{water} := 3\ ft$ BDG 3.6.2

$$z_{w2} := \begin{array}{l} \text{if } Elev_{Q50} - Elev_{bot1} \le 0 \\ \quad \| \; z_{w2} \leftarrow H_{water} \\ \text{else} \\ \quad \| \; z_{w2} \leftarrow Elev_{Q50} - Elev_{bot1} + H_{water} \end{array} = 3\ ft$$

even if the Q50 elevation is below the bottom of the abutment, assume 3ft worth of pore water pressure to be conservative

$p_{w2} := K_p \cdot \gamma_w \cdot z_{w2} = 0.6\ ksf$ (3.11.5.1-1)

$p_{soil2} := K_p \cdot \gamma_{soil} \cdot H_2 = 5.23\ ksf$ (3.11.5.1-1)

Live Load Surcharge 3.11.6.4

This is only applied to the abutments and not the wingwall since traffic load will not be applied behind the wingwalls, only at the abutment sections.

$H_{abut} := \begin{bmatrix} H_1 \\ H_2 \end{bmatrix}$ max height of abutment 1 and abutment 2 bottom

$$h_{eq} := \begin{array}{l} \text{for } i \in 1..2 \\ \quad \| \; h_{eq_i} \leftarrow \dfrac{-1.5}{5} \cdot \left(H_{abut_i} - 5\ ft\right) + 5\ ft \\ \quad \| \; h_{eq} \end{array} = \begin{bmatrix} 3.18 \\ 2.59 \end{bmatrix} ft$$

Table 3.11.6.4-2

$\Delta_p := K_p \cdot \gamma_{soil} \cdot h_{eq} = \begin{bmatrix} 1.28 \\ 1.04 \end{bmatrix} ksf$ constant horizontal earth pressure due to live load surcharge (3.11.6.4-1)

The top spot in the vector applies to abutment 1 and the bottom spot in the vector applies to abutment 2

Axial Pile Analyses

Socketed Pile Design Parameters in Bedrock

Project:	Puddle Dock Bridge over Fifteenmile Stream	**Project No.:**	09.0026189.01
Location:	Albion, ME		
Calculated By:	N. Williams	**Date:**	2/22/2024
Checked By:	A. Blaisdell	**Date:**	8/16/2024

Objective: Develop foundation design parameters for rock socketed H-piles in bedrock using AASHTO methodology for support of the Puddle Dock bridge. The assumed foundation type is 14x89 H-piles, and a 36-inch diameter rock socket will be created using a cluster drill bit.

Inputs: Rock core data from BB-AFMS-100 series test borings, including rock type and RQD, and laboratory test data including unit weight, unconfined compressive strength, Youngs Modulus, and Poisson's ratio for specimens, were assessed as follows:

1. Rock was primarily classified as Granofels across the site. However, foliation was noted in the core runs, which suggests a potentially schistose rock type.
2. The design RQD was conservatively selected as 0%.
3. Design Unconfined Compressive Strength and Youngs Modulus were averaged from laboratory test results.

Approach:

1. Evaluate Geologic Strength Index (GSI) in accordance with AASHTO methodology. See page 5.
2. Develop LPILE/FBPier inputs for Weak Rock model based on bedrock structure characterization and laboratory test results.
3. Calculate unit tip resistance in rock for socketed piles using AASHTO Eq. 10.8.3.5.4c-1 (intact or tightly jointed rock) and Eq. 10.8.3.5.4c-2 (random joint orientation). Considering low RQD at Abutment 1, use the resistance calculated using 10.8.3.5.4c-2 for the design unit tip resistance (jointed rock mass). See page 3.
4. Calculate tip resistance using end area of a 14" square plate that will be welded to the pile. See page 3.

Conclusion: The nominal tip resistance for the proposed 14x89 pile is approximately 1631 kips, resulting in a factored tip resistance of 816 kips, for the assumed bedrock

Calculated by N. Williams Date: 2/22/2024
Checked by ______________ Date: ______________

Calculation: Rock Socket Lateral Parameters - Typical Roc

References: 1) AASHTO LRFD Bridge Design Specifications, 9th Ed. (2020)

Parameter Description	Parameter Symbol	Design Values	Reference
Rock Type		Granofels	Rock ID per boring logs/geologic mapping
Comparable Rock Type/Texture		Schist	AASHTO Table 10.4.6.4-1 *(Noted some foliation in photographs, Granofels typically not foliated)*
Unconfined compressive strength of intact rock	q_u (psi)	15,938	Average of samples
Elastic Secant Modulus of Intact Rock	E_i (ksi)	3,865	Average of Lab Samples (mid range)
Poisson's Ratio of Intact Rock	ν	0.25	Typical for bedrock (Lab sample average value too high)
LPILE Input Unconfined compressive strength	**$q_{u\text{-}LPILE}$ (psi)**	**2,000**	Limit UCS to 2,000 psi for LPILE
Design Strain Factor	**krm**	**0.0005**	Use krm value from Lpile manual
Rock Total Unit Weight	**γ' (psf)**	**169**	Average value from lab UCS tests
Rock Effective Unit Weight	γ' (psf)	106.6	
Geological Strength Index	GSI	50	AASHTO FIG 10.4.6.4-1
Hoek - Brown 2002	D	0	Hoek - Brown 2002
Rock Quality Designation	**RQD (%)**	**0**	Borings 101 and 102 all RQD=0
Empirically determined rock mass parameter	s	0.003868	AASHTO Eq. 10.4.6.4-2
Empirically determined rock mass parameter	a	0.506	AASHTO Eq. 10.4.6.4-3
Rock group constant	m_i	13	AASHTO Table 10.4.6.4-1 *(Schist, Upper Bound)*
Empirically determined rock mass parameter	m_b	2.18	AASHTO Eq. 10.4.6.4-4
Equivalent Rock Mass Modulus, Hoek & Brown	E_m1 (ksi)	1,450	AASHTO Table 10.4.6.5-1 - eq. from table
Equivalent Rock Mass Modulus, Yang	E_m2 (ksi)	387	AASHTO Table 10.4.6.5-1 - eq. from table
Design Basis Equivalent Rock Modulus	E_m (ksi)	919	
Initial Modulus of Rock Mass, Lpile K_{ir} Value	**K_{ir} (psi)**	**9,187**	Calculated as 1/100th of the calculated Design basis equivalent Rock Modulus, based on the KSDOT research.
Shear Modulus	**G (ksi)**	**367.5**	Calculated as G=Em/(2*(1+v))

Notes:

1. Yellow cell is user input, white is calculated.
2. Red text are input values for LPILE/FBPier Weak Rock model.
3. A reduced modulus for use in LPILE was based on research by Kansas Department of Transportation (KSDOT) presented in a paper entitled " Lateral Capacity of Rock Sockets in Limestone Under Cyclic and Repeated Loading" dated August 2010.

Puddle Dock Bridge over Fifteenmile Stream
Bridge Pile Evaluations
Albion, ME
09.0026189.01

Calculated by N. Williams Date: 2/22/2024
Checked by A. Blaisdell Date: 8/16/2024

Calculation: Socketed Pipe Pile Tip Resistance in Rock
References: 1) AASHTO LRFD Bridge Design Specifications, 9th Ed. (2020)

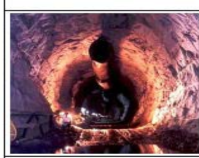
Parameter Description	**Parameter Symbol**	**Bedrock Values**	**Reference**
Pile Tip Size	*B (in)*	*15*	
Pile Tip Area	*At (in^2)*	*225.00*	
Unconfined compressive strength of intact rock	q_u (psi)	15,938	Lab data for rock
Nominal Unit Tip Resistance, Intact Rock	$q_{p\ intact}$ (ksf)	5,738	AASHTO Eq. 10.8.3.5.4C-1
Geological Strength Index	GSI	50	AASHTO Fig. 10.4.6.4-1
Hoek - Brown 2002	D	0	Hoek - Brown 2002
Empirically determined rock mass parameter	s	0.00387	AASHTO Eq. 10.4.6.4-2
Empirically determined rock mass parameter	a	0.506	AASHTO Eq. 10.4.6.4-3
Rock group constant	m_i	13	AASHTO Table 10.4.6.4-1
Empirically determined rock mass parameter	m_b	2.2	AASHTO Eq. 10.4.6.4-4
Vertical effective stress at the socket bearing elevation	$\sigma'_{v,b}$ (psf)	1066	Vertical effective stress at bottom of socket
	$\sigma'_{v,b}$ (psi)	7.4	
Fracturing coefficient	A	1087	AASHTO Eq. 10.8.3.5.4C-3 (Turner and Ramey, 2010)
Nominal Unit Tip Resistance, Jointed Rock Mass	$q_{p\ jointed}$ (ksf)	1044	AASHTO Eq. 10.8.3.5.4C-2
Design Nominal Unit Tip Resistance	*$q_{p\ design}$ (ksf)*	*1,044*	*Use Jointed value for rock based on all RQD=0 material at Abutment 1*
Nominal Tip Resistance	***$R_{p,i}$ (kips)***	***1,631***	
Resistance Factor	ϕ_{qp}	0.5	AASHTO TABLE 10.5.5.2.5-1, no load testing
Factored Tip Resistance	***$R_{R,i}$ (kips)***	***816***	AASHTO Eq. 10.8.3.5-1

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand	β-method (Brown et al., 2010)	0.55
	Tip resistance in sand	Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010)	0.60
	Tip resistance in cohesive IGMs	Brown et al. (2010)	0.55
	Side resistance in rock	Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	0.50
Block Failure, φ_{b1}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (Brown et al., 2010)	0.35
	Sand	β-method (Brown et al., 2010)	0.45
	Rock	Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

Table 1: Guidelines for estimating disturbance factor D

From Hoek et al., 2002

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel. No Blasting, No Disturbance	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$\boxed{D = 0}$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

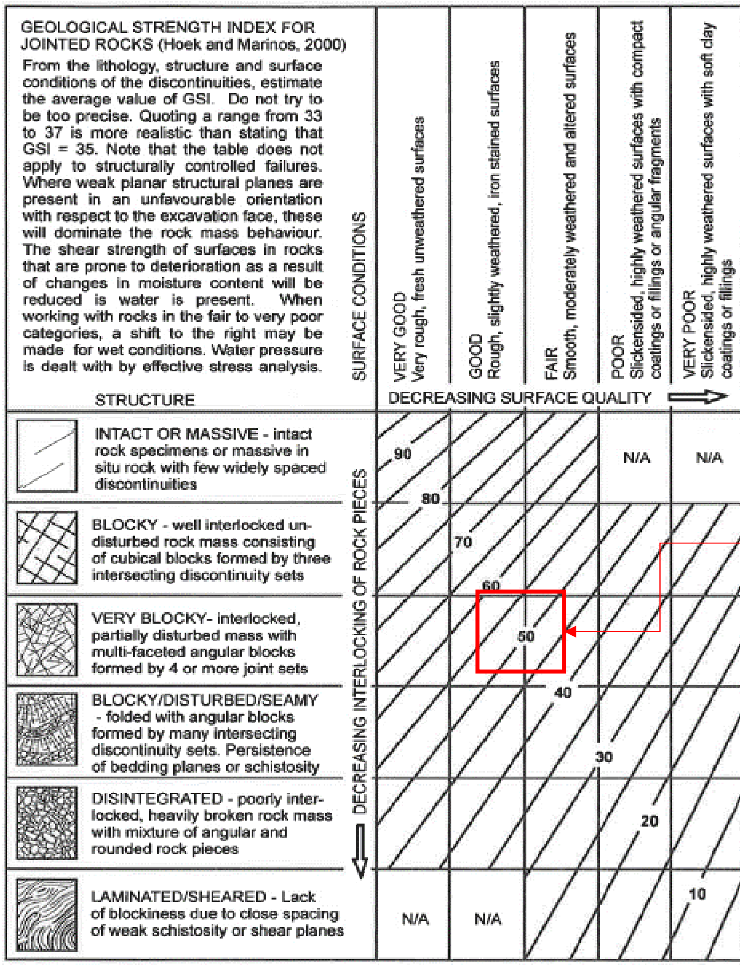


Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

Best Estimate
Structure: Very Blocky
Surface Conditions: Good/Fair
Select GSI=50

Table 10.4.6.4-1—Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000; with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Rock classified as Granofels by MaineDOT but shows foliation, assume Schist upper bound (13)

Lateral Pile Analyses
(Preliminary Run)

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 and subsurface conditions in borings BB-AFMS-101 through -104 performed by S. W. Cole between December 21 and December 23, 2022.

Expansion, Dense Fill, Pile length = 13'					
Stratum	**Soil Model**	**Top of Layer Elevation (NAVD88 ft)**	**k (pci) / krm**	**ϕ' (deg)/ UCS (psi)**	**γ_e (pcf)**
New Fill, Upper Socket (Dense)	Reese Sand	241	122	34	125
New Fill, Lower Socket (Dense)	Reese Sand	231	83	32	122
Top of Grout in Rock Socket	Weak Rock	225.5	krm=0.0005	2000	169

Notes: 1. Pile tip elevation should be assumed to be embedded into rock socket and 3 feet of grout.

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. Since the pile geometry is the same at both abutments, only Abutment 1 was evaluated.

Table 2 - LPile Output Summary
MaineDOT - Puddle Dock Bridge
GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0026189.01
CALCULATED BY E. Tome 4/1/2024
CHECKED BY A. Blaisdell 4/3/2024

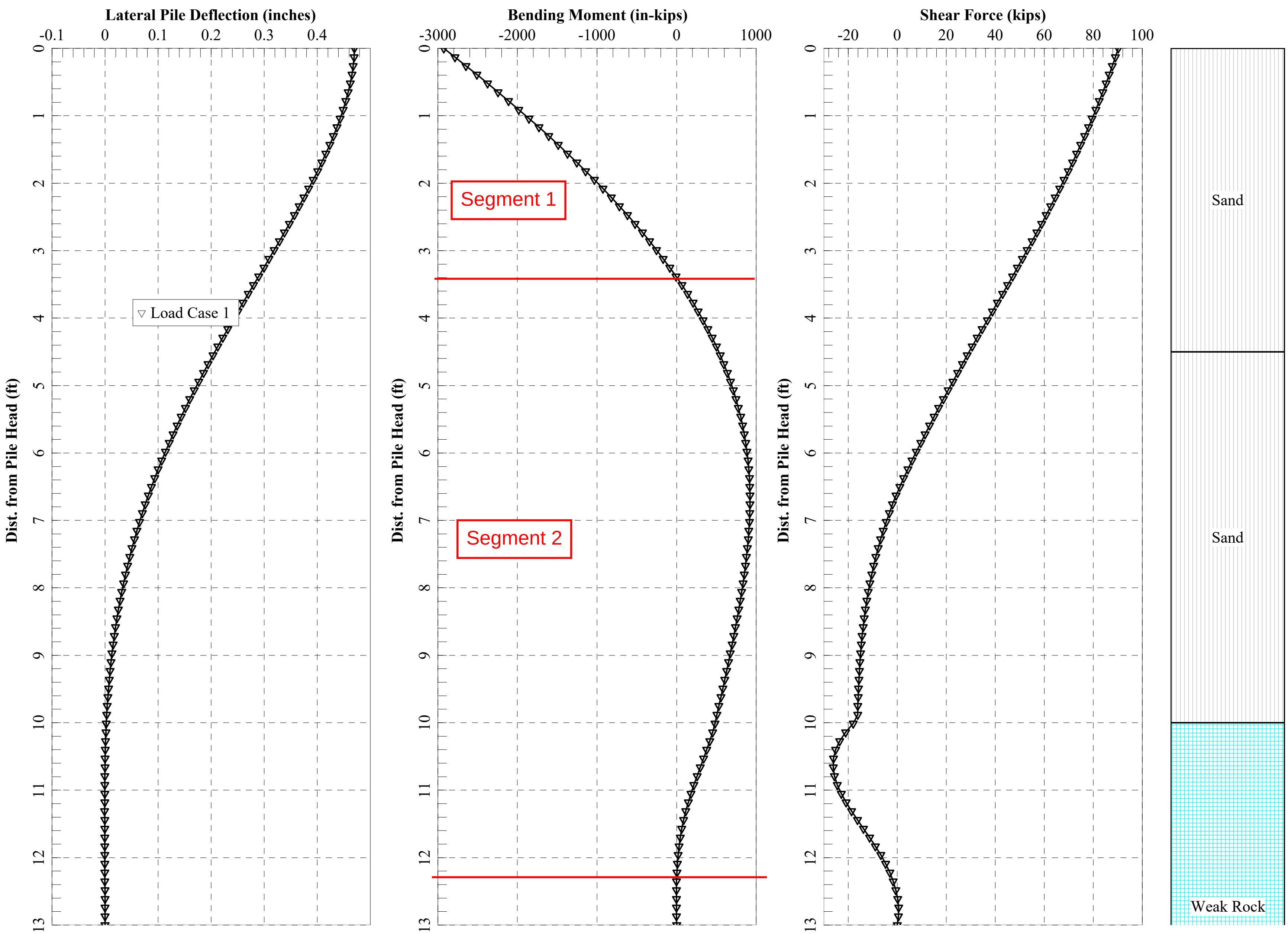
Abutment 1									
Pile Section	**Pile Length (ft)**	**Axial Load[2] (kips)**	**Deflection at Pile Head (in)**	**Shear Force at Pile Head (kips)**	**Total Stress at Pile Head (ksi)**	**Max Moment in Upper Section (in-kip)**	**Upper Section Length[3] (ft)**	**Max Moment in Lower Section (in-kip)**	**Lower Section Length[3] (ft)**
HP 14x89 (Weak axis)	13	307.3	0.47	90.2	77.9	-2926.0	3.4	922.1	8.9
HP 14x117 (Weak axis)	13	307.3	0.47	103.1	70.5	-3671.0	3.7	1136.3	9.3

Notes:

1. Soil layering and properties are presented in Table 1.
2. The axial load is the maximum Factored Axial Load.
3. The upper section length is measured from the top of pile to first moment inflection point. The lower section length is measured between first and second moment inflection points.
4. Standard weak axis analyses included applied deflection at pile head and no pile head rotation.

HP14x89 (Weak Axis)

Applied at Pile Head: Defl. = 0.47 in, Axial Load = 307.3 kips



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                    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\09.0026189.00 - MEDOT - Puddle Dock Bridge\09.0026189.01\Work\Calcs\LPile\

Name of input data file:
AB1_Exp_dense fill.lp12d

Name of output report file:
AB1_Exp_dense fill.lp12o

Name of plot output file:
AB1_Exp_dense fill.lp12p
```

```
Name of runtime message file:
AB1_Exp_dense fill.lp12r


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

               Date:  March 27, 2024             Time:  15:10:30


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

Project Name: Albion Puddle Dock Bridge


Job Number: 09.0026189.00


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)
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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                                  =       13.000 ft
Depth of ground surface below top of pile             =      -5.0000 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           13.000           14.7000


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

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                  =    13.000000 ft
   Pile width                                         =    13.800000 in


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

The soil profile is modelled using 3 layers

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

   Distance from top of pile to top of layer          =     -5.00000 ft
   Distance from top of pile to bottom of layer       =     4.500000 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                     =    34.000000 deg.
   Friction angle at bottom of layer                  =    34.000000 deg.
   Subgrade k at top of layer                         =   122.000000 pci
   Subgrade k at bottom of layer                      =   122.000000 pci


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

   Distance from top of pile to top of layer          =     4.500000 ft
   Distance from top of pile to bottom of layer       =    10.000000 ft
   Effective unit weight at top of layer              =   122.000000 pcf
```

Effective unit weight at bottom of layer = 122.000000 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 3 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 20.000000 ft
Effective unit weight at top of layer = 169.000000 pcf
Effective unit weight at bottom of layer = 169.000000 pcf
Uniaxial compressive strength at top of layer = 2000. psi
Uniaxial compressive strength at bottom of layer = 2000. psi
Initial modulus of rock at top of layer = 9183. psi
Initial modulus of rock at bottom of layer = 9183. psi
RQD of rock at top of layer = 0.0000 %
RQD of rock at bottom of layer = 0.0000 %
k rm of rock at top of layer = 0.0005000
k rm of rock at bottom of layer = 0.0005000

(Depth of the lowest soil layer extends 7.000 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weight of rock were outside the limits of
50 pcf to 150 pcf.

The maximum input value, in layer 1, for effective unit weight = 169.00 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	Uniaxial qu psi	RQD %	E50 or krm	kpy pci	Rock Mass Modulus psi
1	Sand	-5.000	125.0000	34.0000	--	--	--	122.0000	--
	(Reese, et al.)	4.5000	125.0000	34.0000	--	--	--	122.0000	--
2	Sand	4.5000	122.0000	32.0000	--	--	--	83.0000	--
	(Reese, et al.)	10.0000	122.0000	32.0000	--	--	--	83.0000	--
3	Weak	10.0000	169.0000	--	2000.	0.00	5.00E-04	--	9183.
	Rock	20.0000	169.0000	--	2000.	0.00	5.00E-04	--	9183.

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 = 0.470000 in	S = 0.0000 in/in	307300.	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	=	13.000000 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	307.300

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 307.300 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000457	43.1424962	9448389.	97.2589632	12.8690591	
0.00000913	86.2849924	9448389.	52.3044816	13.8325954	
0.00001370	129.4274886	9448389.	37.3196544	14.7961314	
0.00001826	172.5699847	9448389.	29.8272408	15.7596677	
0.00002283	215.7124809	9448389.	25.3317926	16.7232038	
0.00002740	258.8549771	9448389.	22.3348272	17.6867399	
0.00003196	301.9974733	9448389.	20.1941376	18.6502762	
0.00003653	345.1399695	9448389.	18.5886204	19.6138123	
0.00004110	388.2824657	9448389.	17.3398848	20.5773485	
0.00004566	431.4249618	9448389.	16.3408963	21.5408847	
0.00005023	474.5674580	9448389.	15.5235421	22.5044208	
0.00005479	517.7099542	9448389.	14.8424136	23.4679570	
0.00005936	560.8524504	9448389.	14.2660741	24.4314932	
0.00006393	603.9949466	9448389.	13.7720688	25.3950294	
0.00006849	647.1374428	9448389.	13.3439309	26.3585656	
0.00007306	690.2799389	9448389.	12.9693102	27.3221017	
0.00007762	733.4224351	9448389.	12.6387625	28.2856379	
0.00008219	776.5649313	9448389.	12.3449424	29.2491740	
0.00008676	819.7074275	9448389.	12.0820507	30.2127102	
0.00009132	862.8499237	9448389.	11.8454482	31.1762464	
0.00009589	905.9924199	9448389.	11.6313792	32.1397826	
0.0001005	949.1349161	9448389.	11.4367711	33.1033188	
0.0001050	992.2774122	9448389.	11.2590854	34.0668549	
0.0001096	1035.	9448389.	11.0962068	35.0303911	
0.0001142	1079.	9448389.	10.9463585	35.9939273	

0.0001187	1122.	9448389.	10.8080370	36.9574635	
0.0001233	1165.	9448389.	10.6799616	37.9209997	
0.0001279	1208.	9448389.	10.5610344	38.8845358	
0.0001324	1251.	9448389.	10.4503091	39.8480720	
0.0001370	1294.	9448389.	10.3469654	40.8116082	
0.0001415	1337.	9448389.	10.2502891	41.7751444	
0.0001461	1381.	9448389.	10.1596551	42.7386806	
0.0001507	1424.	9448389.	10.0745140	43.7022167	
0.0001552	1467.	9448389.	9.9943813	44.6657529	
0.0001598	1510.	9448389.	9.9188275	45.6292891	
0.0001644	1553.	9448389.	9.8474712	46.5928253	
0.0001689	1596.	9448389.	9.7799720	47.5563614	
0.0001735	1639.	9448389.	9.7160253	48.5198976	
0.0001781	1683.	9448389.	9.6553580	49.4834338	
0.0001872	1766.	9434060.	9.5455487	50.0000000	Y
0.0001963	1844.	9391831.	9.4515105	50.0000000	Y
0.0002055	1917.	9328469.	9.3707943	50.0000000	Y
0.0002146	1985.	9250395.	9.3011456	50.0000000	Y
0.0002237	2050.	9161583.	9.2409113	50.0000000	Y
0.0002329	2111.	9064670.	9.1888301	50.0000000	Y
0.0002420	2169.	8963194.	9.1434935	50.0000000	Y
0.0002511	2225.	8858325.	9.1041015	50.0000000	Y
0.0002603	2278.	8751888.	9.0697528	50.0000000	Y
0.0002694	2329.	8644290.	9.0399488	50.0000000	Y
0.0002785	2378.	8537219.	9.0138881	50.0000000	Y
0.0002877	2425.	8431073.	8.9911425	50.0000000	Y
0.0002968	2471.	8326142.	8.9713484	50.0000000	Y
0.0003059	2515.	8221359.	8.9539116	50.0000000	Y
0.0003151	2556.	8111756.	8.9369099	50.0000000	Y
0.0003242	2593.	7998733.	8.9202772	50.0000000	Y
0.0003333	2628.	7883074.	8.9041028	50.0000000	Y
0.0003425	2660.	7766050.	8.8882578	50.0000000	Y
0.0003516	2689.	7648570.	8.8726835	50.0000000	Y
0.0003607	2717.	7531392.	8.8573294	50.0000000	Y
0.0003699	2742.	7414964.	8.8422319	50.0000000	Y
0.0003790	2766.	7299194.	8.8276533	50.0000000	Y
0.0003881	2789.	7185317.	8.8132270	50.0000000	Y
0.0003973	2810.	7073230.	8.7991545	50.0000000	Y
0.0004064	2829.	6962562.	8.7853675	50.0000000	Y
0.0004155	2848.	6854066.	8.7715398	50.0000000	Y
0.0004246	2865.	6747449.	8.7583674	50.0000000	Y
0.0004338	2882.	6643788.	8.7452589	50.0000000	Y
0.0004429	2897.	6541136.	8.7324025	50.0000000	Y
0.0004520	2912.	6441604.	8.7197281	50.0000000	Y
0.0004612	2926.	6343999.	8.7074504	50.0000000	Y
0.0004703	2939.	6248518.	8.6951991	50.0000000	Y
0.0004794	2951.	6155890.	8.6833297	50.0000000	Y
0.0004886	2963.	6064456.	8.6715403	50.0000000	Y
0.0004977	2974.	5976203.	8.6600952	50.0000000	Y
0.0005068	2985.	5889196.	8.6485949	50.0000000	Y
0.0005160	2995.	5804921.	8.6377330	50.0000000	Y
0.0005251	3005.	5722168.	8.6264808	50.0000000	Y
0.0005342	3014.	5641926.	8.6158763	50.0000000	Y
0.0005434	3023.	5562975.	8.6052420	50.0000000	Y
0.0005799	3055.	5267380.	8.5648117	50.0000000	Y
0.0006164	3081.	4998243.	8.5265198	50.0000000	Y
0.0006530	3104.	4753664.	8.4907866	50.0000000	Y
0.0006895	3124.	4530621.	8.4570775	50.0000000	Y
0.0007260	3141.	4326085.	8.4255629	50.0000000	Y
0.0007625	3156.	4138549.	8.3955833	50.0000000	Y
0.0007991	3169.	3965799.	8.3671818	50.0000000	Y
0.0008356	3181.	3806384.	8.3401968	50.0000000	Y
0.0008721	3191.	3658877.	8.3147635	50.0000000	Y
0.0009087	3201.	3522344.	8.2904370	50.0000000	Y
0.0009452	3209.	3395111.	8.2676955	50.0000000	Y
0.0009817	3217.	3276498.	8.2456696	50.0000000	Y
0.0010182	3224.	3166083.	8.2248477	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	307.3000000000	3224.

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	-5.000	0.00	N.A.	No	0.00	110738.
2	4.5000	10.0862	Yes	No	110738.	257521.
3	10.0000	15.0000	No	Yes	N.A.	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 = 0.470000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 307300.0 lbs

Max moment at pile head

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	0.4700	-2925894.	90195.	0.00	77906.	6.34E+09	-732.843	1216.	0.00
0.1300	0.4694	-2785955.	88998.	-7.02E-04	74749.	6.34E+09	-764.057	2539.	0.00
0.2600	0.4678	-2647547.	87783.	-0.00131	71627.	7.80E+09	-794.459	2649.	0.00
0.3900	0.4654	-2510818.	86520.	-0.00181	68543.	8.23E+09	-824.165	2763.	0.00
0.5200	0.4622	-2375867.	85212.	-0.00227	65499.	8.54E+09	-853.124	2880.	0.00
0.6500	0.4583	-2242784.	83859.	-0.00268	62497.	8.82E+09	-881.275	3000.	0.00
0.7800	0.4538	-2111655.	82463.	-0.00306	59539.	9.06E+09	-908.555	3123.	0.00
0.9100	0.4487	-1982564.	81025.	-0.00341	56627.	9.25E+09	-934.899	3250.	0.00
1.0400	0.4431	-1855587.	79547.	-0.00373	53763.	9.38E+09	-960.240	3380.	0.00
1.1700	0.4371	-1730800.	78029.	-0.00403	50948.	9.44E+09	-985.674	3518.	0.00
1.3000	0.4306	-1608273.	76470.	-0.00431	48184.	9.45E+09	-1012.	3668.	0.00
1.4300	0.4236	-1488084.	74871.	-0.00456	45473.	9.45E+09	-1038.	3823.	0.00
1.5600	0.4163	-1370303.	73232.	-0.00480	42816.	9.45E+09	-1063.	3983.	0.00
1.6900	0.4087	-1255001.	71555.	-0.00501	40215.	9.45E+09	-1087.	4149.	0.00
1.8200	0.4007	-1142245.	69841.	-0.00521	37671.	9.45E+09	-1110.	4321.	0.00
1.9500	0.3924	-1032099.	68092.	-0.00539	35187.	9.45E+09	-1132.	4500.	0.00
2.0800	0.3839	-924627.	66311.	-0.00555	32763.	9.45E+09	-1153.	4684.	0.00
2.2100	0.3751	-819887.	64497.	-0.00570	30400.	9.45E+09	-1172.	4876.	0.00
2.3400	0.3661	-717934.	62654.	-0.00582	28100.	9.45E+09	-1191.	5075.	0.00
2.4700	0.3569	-618824.	60782.	-0.00593	25864.	9.45E+09	-1208.	5282.	0.00
2.6000	0.3476	-522605.	58884.	-0.00603	23694.	9.45E+09	-1225.	5497.	0.00
2.7300	0.3381	-429325.	56962.	-0.00611	21590.	9.45E+09	-1240.	5720.	0.00
2.8600	0.3285	-339028.	55017.	-0.00617	19553.	9.45E+09	-1254.	5954.	0.00
2.9900	0.3189	-251756.	53051.	-0.00622	17584.	9.45E+09	-1267.	6197.	0.00
3.1200	0.3091	-167546.	51066.	-0.00625	15685.	9.45E+09	-1278.	6451.	0.00
3.2500	0.2994	-86435.	49063.	-0.00627	13855.	9.45E+09	-1289.	6716.	0.00
3.3800	0.2896	-8452.	47046.	-0.00628	12096.	9.45E+09	-1298.	6994.	0.00
3.5100	0.2797	66371.	45014.	-0.00628	13403.	9.45E+09	-1307.	7286.	0.00
3.6400	0.2700	138010.	42970.	-0.00626	15019.	9.45E+09	-1314.	7591.	0.00
3.7700	0.2602	206441.	40916.	-0.00623	16562.	9.45E+09	-1320.	7913.	0.00
3.9000	0.2505	271643.	38853.	-0.00619	18033.	9.45E+09	-1325.	8251.	0.00
4.0300	0.2409	333599.	36783.	-0.00614	19431.	9.45E+09	-1329.	8607.	0.00
4.1600	0.2314	392294.	34707.	-0.00608	20755.	9.45E+09	-1332.	8980.	0.00
4.2900	0.2219	447718.	32629.	-0.00601	22005.	9.45E+09	-1333.	9371.	0.00
4.4200	0.2126	499861.	30549.	-0.00594	23181.	9.45E+09	-1333.	9780.	0.00
4.5500	0.2034	548722.	28527.	-0.00585	24283.	9.45E+09	-1260.	9666.	0.00
4.6800	0.1943	594472.	26565.	-0.00575	25315.	9.45E+09	-1255.	10073.	0.00
4.8100	0.1854	637122.	24612.	-0.00565	26277.	9.45E+09	-1248.	10501.	0.00
4.9400	0.1767	676683.	22671.	-0.00554	27170.	9.45E+09	-1240.	10950.	0.00
5.0700	0.1681	713172.	20744.	-0.00543	27993.	9.45E+09	-1231.	11421.	0.00
5.2000	0.1598	746609.	18832.	-0.00531	28747.	9.45E+09	-1220.	11916.	0.00

L1=3.4'

Max moment in segment 2

5.3300	0.1516	777017.	16937.	-0.00518	29433.	9.45E+09	-1208.	12437.	0.00
5.4600	0.1436	804423.	15062.	-0.00505	30051.	9.45E+09	-1195.	12985.	0.00
5.5900	0.1358	828857.	13209.	-0.0[illegible]	[illegible]	[illegible]	-1181.	13562.	0.00
5.7200	0.1282	850351.	11380.	-0.0[illegible]	[illegible]	[illegible]	-1165.	14171.	0.00
5.8500	0.1209	868944.	9576.	-0.0[illegible]	[illegible]	[illegible]	-1148.	14813.	0.00
5.9800	0.1138	884673.	7799.	-0.00449	31861.	9.45E+09	-1130.	15492.	0.00
6.1100	0.1069	897583.	6051.	-0.00435	32153.	9.45E+09	-1111.	16209.	0.00
6.2400	0.1002	907720.	4335.	-0.00420	32381.	9.45E+09	-1090.	16969.	0.00
6.3700	0.09379	915131.	2656.	-0.00405	32548.	9.45E+09	-1062.	17666.	0.00
6.5000	0.08759	919886.	1045.	-0.00389	32656.	9.45E+09	-1003.	17868.	0.00
6.6300	0.08164	922126.	-475.268	-0.00374	32706.	9.45E+09	-945.650	18070.	0.00
6.7600	0.07592	921991.	-1906.	-0.00359	32703.	9.45E+09	-889.227	18272.	0.00
6.8900	0.07044	919620.	-3251.	-0.00344	32650.	9.45E+09	-834.135	18474.	0.00
7.0200	0.06519	915146.	-4510.	-0.00329	32549.	9.45E+09	-780.458	18676.	0.00
7.1500	0.06018	908700.	-5687.	-0.00314	32403.	9.45E+09	-728.276	18878.	0.00
7.2800	0.05541	900409.	-6784.	-0.00299	32216.	9.45E+09	-677.660	19080.	0.00
7.4100	0.05086	890399.	-7802.	-0.00284	31990.	9.45E+09	-628.673	19282.	0.00
7.5400	0.04655	878788.	-8746.	-0.00269	31729.	9.45E+09	-581.374	19484.	0.00
7.6700	0.04246	865692.	-9618.	-0.00255	31433.	9.45E+09	-535.814	19686.	0.00
7.8000	0.03859	851224.	-10419.	-0.00241	31107.	9.45E+09	-492.038	19888.	0.00
7.9300	0.03495	835492.	-11154.	-0.00227	30752.	9.45E+09	-450.085	20090.	0.00
8.0600	0.03152	818598.	-11825.	-0.00213	30371.	9.45E+09	-409.987	20292.	0.00
8.1900	0.02830	800641.	-12435.	-0.00200	29966.	9.45E+09	-371.771	20494.	0.00
8.3200	0.02529	781716.	-12987.	-0.00187	29539.	9.45E+09	-335.456	20696.	0.00
8.4500	0.02247	761913.	-13483.	-0.00174	29092.	9.45E+09	-301.059	20898.	0.00
8.5800	0.01986	741317.	-13927.	-0.00162	28628.	9.45E+09	-268.587	21100.	0.00
8.7100	0.01743	720009.	-14323.	-0.00149	28147.	9.45E+09	-238.046	21302.	0.00
8.8400	0.01519	698064.	-14672.	-0.00138	27652.	9.45E+09	-209.433	21504.	0.00
8.9700	0.01313	675555.	-14977.	-0.00126	27144.	9.45E+09	-182.741	21706.	0.00
9.1000	0.01125	652547.	-15243.	-0.00115	26625.	9.45E+09	-157.960	21908.	0.00
9.2300	0.00953	629103.	-15472.	-0.00105	26096.	9.45E+09	-135.073	22110.	0.00
9.3600	0.00797	605281.	-15666.	-9.47E-04	25559.	9.45E+09	-114.059	22312.	0.00
9.4900	0.00658	581133.	-15829.	-8.49E-04	25014.	9.45E+09	-94.891	22514.	0.00
9.6200	0.00533	556708.	-15964.	-7.55E-04	24463.	9.45E+09	-77.541	22716.	0.00
9.7500	0.00422	532051.	-16072.	-6.65E-04	23907.	9.45E+09	-61.974	22918.	0.00
9.8800	0.00325	507200.	-16158.	-5.80E-04	23347.	9.45E+09	-48.151	23120.	0.00
10.0100	0.00241	482193.	-17941.	-4.98E-04	22782.	9.45E+09	-2237.	1448140.	0.00
10.1400	0.00170	451702.	-21085.	-4.21E-04	22095.	9.45E+09	-1794.	1650841.	0.00
10.2700	0.00110	416810.	-23501.	-3.49E-04	21308.	9.45E+09	-1303.	1853542.	0.00
10.4000	6.06E-04	378713.	-25141.	-2.83E-04	20448.	9.45E+09	-798.722	2056242.	0.00
10.5300	2.12E-04	338642.	-26004.	-2.24E-04	19544.	9.45E+09	-307.707	2258943.	0.00
10.6600	-9.37E-05	297796.	-26129.	-1.72E-04	18623.	9.45E+09	147.9223	2461644.	0.00
10.7900	-3.23E-04	257286.	-25583.	-1.26E-04	17709.	9.45E+09	552.1340	2664344.	0.00

10.9200	-4.87E-04	218099.	-24454.	-8.67E-05	16825.	9.45E+09	894.2050	2867045.	0.00
11.0500	-5.94E-04	181071.	-22846.	-5.37E-05	15990.	9.45E+09	1168.	3069746.	0.00
11.1800	-6.54E-04	146872.	-20864.	-2.66E-05	15219.	9.45E+09	1372.	3272446.	0.00
11.3100	-6.77E-04	116000.	-18618.	-4.93E-06	14522.	9.45E+09	1508.	3475147.	0.00
11.4400	-6.69E-04	88787.	-16211.	1.20E-05	13908.	9.45E+09	1578.	3677848.	0.00
11.5700	-6.39E-04	65409.	-13740.	2.47E-05	13381.	9.45E+09	1590.	3880548.	0.00
11.7000	-5.92E-04	45896.	-11290.	3.39E-05	12941.	9.45E+09	1551.	4083249.	0.00
11.8300	-5.34E-04	30153.	-8937.	4.02E-05	12586.	9.45E+09	1466.	4285950.	0.00
11.9600	-4.67E-04	17975.	-6745.	4.41E-05	12311.	9.45E+09	1344.	4488650.	0.00
12.0900	-3.96E-04	9066.	-4768.	4.64E-05	12110.	9.45E+09	1190.	4691351.	0.00
12.2200	-3.22E-04	3054.	-3051.	4.74E-05	11974.	9.45E+09	1011.	4894052.	0.00
12.3500	-2.48E-04	-497.630	-1630.	4.76E-05	11917.	9.45E+09	810.3793	5096752.	0.00
12.4800	-1.74E-04	-2077.	-537.177	4.74E-05	11952.	9.45E+09	590.6196	5299453.	0.00
12.6100	-1.00E-04	-2219.	199.2133	4.70E-05	11956.	9.45E+09	353.4699	5502154.	0.00
12.7400	-2.71E-05	-1501.	552.3525	4.67E-05	11939.	9.45E+09	99.2727	5704854.	0.00
12.8700	4.55E-05	-540.497	495.2741	4.65E-05	11918.	9.45E+09	-172.450	5907555.	0.00
13.0000	1.18E-04	0.00	0.00	4.65E-05	11906.	9.45E+09	-462.517	3055128.	0.00

L2=12.3'-3.4'=8.9'

* 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 interpolated 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              =     0.47000000 inches
Computed slope at pile head       =       0.000000 radians
Maximum bending moment            =      -2925894. inch-lbs
Maximum shear force               =         90195. 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              =             10
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	0.4700	S, rad	0.00	307300.	0.4700	0.00	90195.	-2925894.

Maximum pile-head deflection = 0.4700000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

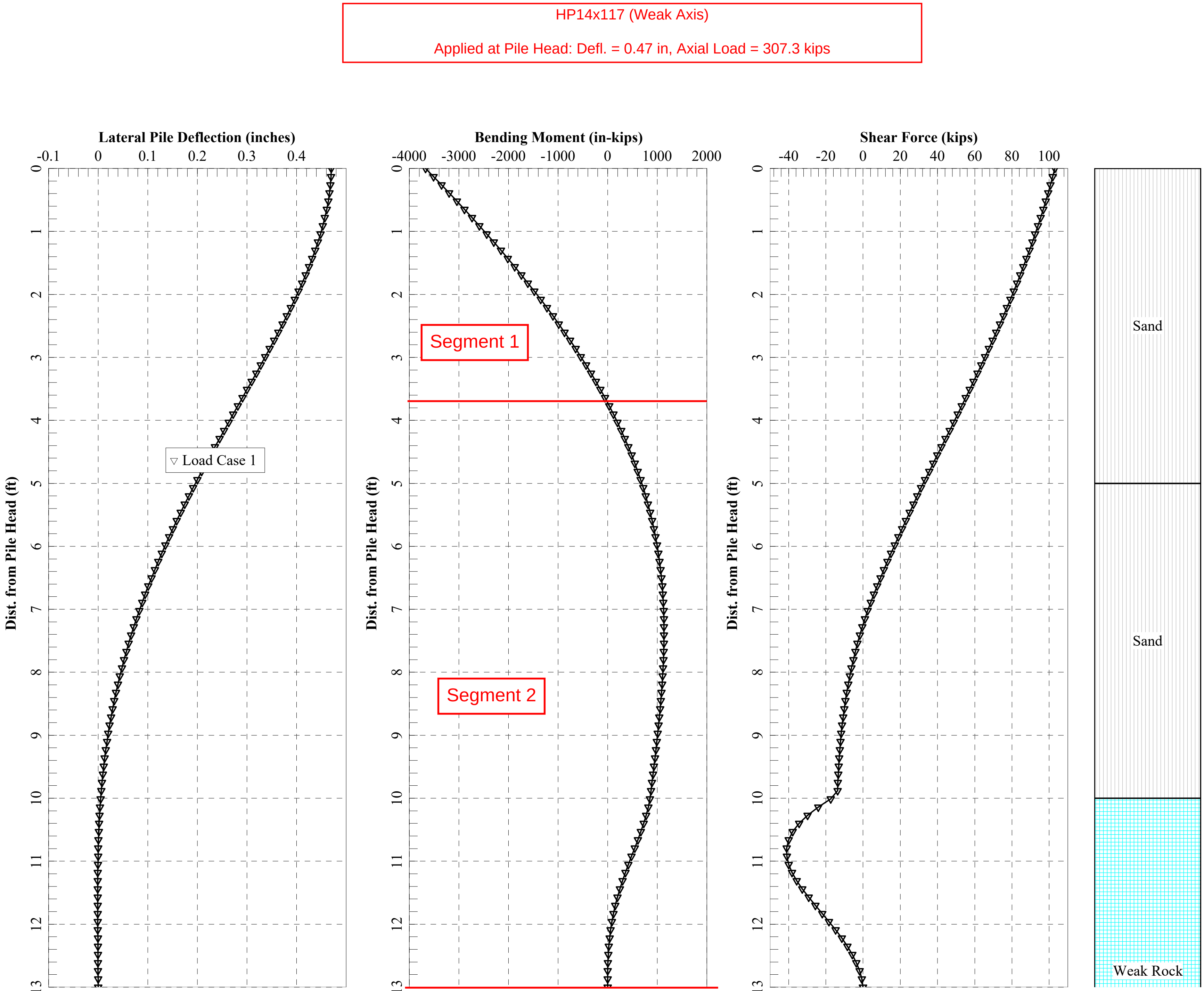
Summary of Warning Messages

The following warning was reported 264 times

**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.


HP14x117 (Weak Axis)
Applied at Pile Head: Defl. = 0.47 in, Axial Load = 307.3 kips
Lateral Pile Deflection (inches)
Bending Moment (in-kips)
Shear Force (kips)
Dist. from Pile Head (ft)
Load Case 1
Segment 1
Segment 2
Sand
Sand
Weak Rock

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

                    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\09.0026189.00 - MEDOT - Puddle Dock Bridge\09.0026189.01\Work\Calcs\LPile\14X117\

Name of input data file:
AB1_Exp_dense fill.lp12d

Name of output report file:
AB1_Exp_dense fill.lp12o

Name of plot output file:
AB1_Exp_dense fill.lp12p

Name of runtime message file:
AB1_Exp_dense fill.lp12r


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

               Date:  March 29, 2024             Time:  13:10:26


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

Project Name: Albion Puddle Dock Bridge


Job Number: 09.0026189.00


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                                   =       13.000 ft
Depth of ground surface below top of pile              =      -5.0000 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           14.9000
  2           13.000           14.9000


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

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                   =    13.000000 ft
   Pile width                                          =    14.200000 in


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

The soil profile is modelled using 3 layers

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

   Distance from top of pile to top of layer           =     -5.00000 ft
   Distance from top of pile to bottom of layer        =     5.000000 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                      =    34.000000 deg.
   Friction angle at bottom of layer                   =    34.000000 deg.
   Subgrade k at top of layer                          =   122.000000 pci
   Subgrade k at bottom of layer                       =   122.000000 pci


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

   Distance from top of pile to top of layer           =     5.000000 ft
   Distance from top of pile to bottom of layer        =    10.000000 ft
   Effective unit weight at top of layer               =   122.000000 pcf
```

Effective unit weight at bottom of layer = 122.000000 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 3 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 20.000000 ft
Effective unit weight at top of layer = 169.000000 pcf
Effective unit weight at bottom of layer = 169.000000 pcf
Uniaxial compressive strength at top of layer = 2000. psi
Uniaxial compressive strength at bottom of layer = 2000. psi
Initial modulus of rock at top of layer = 9183. psi
Initial modulus of rock at bottom of layer = 9183. psi
RQD of rock at top of layer = 0.0000 %
RQD of rock at bottom of layer = 0.0000 %
k rm of rock at top of layer = 0.0005000
k rm of rock at bottom of layer = 0.0005000

(Depth of the lowest soil layer extends 7.000 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weight of rock were outside the limits of
50 pcf to 150 pcf.

The maximum input value, in layer 1, for effective unit weight = 169.00 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	Uniaxial qu psi	RQD %	E50 or krm	kpy pci	Rock Mass Modulus psi
1	Sand	-5.000	125.0000	34.0000	--	--	--	122.0000	--
	(Reese, et al.)	5.0000	125.0000	34.0000	--	--	--	122.0000	--
2	Sand	5.0000	122.0000	32.0000	--	--	--	83.0000	--
	(Reese, et al.)	10.0000	122.0000	32.0000	--	--	--	83.0000	--
3	Weak	10.0000	169.0000	--	2000.	0.00	5.00E-04	--	9183.
	Rock	20.0000	169.0000	--	2000.	0.00	5.00E-04	--	9183.

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 = 0.470000 in	S = 0.0000 in/in	307300.	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	=	13.000000 ft
Flange Width	=	14.900000 in
Section Depth	=	14.200000 in
Flange Thickness	=	0.805000 in
Web Thickness	=	0.805000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	34.123950 sq. in.
Moment of Inertia	=	444.363799 in^4
Elastic Bending Stiffness	=	12886550. kip-in^2
Plastic Modulus, Z	=	91.398684in^3
Plastic Moment Capacity = Fy Z	=	4570.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1706.197 kips
Nominal Axial Tensile Capacity	=	-1706.197 kips

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

Number	Axial Thrust Force kips
1	307.300

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 307.300 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000461	59.4504450	12886018.	74.7583425	9.9921978	
0.00000923	118.9008901	12886018.	41.1041712	10.9789902	
0.00001384	178.3513351	12886018.	29.8861142	11.9657828	
0.00001845	237.8017802	12886018.	24.2770856	12.9525753	
0.00002307	297.2522252	12886018.	20.9116685	13.9393678	
0.00002768	356.7026702	12886018.	18.6680571	14.9261603	
0.00003229	416.1531153	12886018.	17.0654775	15.9129528	
0.00003691	475.6035603	12886018.	15.8635428	16.8997453	
0.00004152	535.0540054	12886018.	14.9287047	17.8865378	
0.00004614	594.5044504	12886018.	14.1808342	18.8733303	
0.00005075	653.9548955	12886018.	13.5689402	19.8601228	
0.00005536	713.4053405	12886018.	13.0590285	20.8469153	
0.00005998	772.8557855	12886018.	12.6275648	21.8337078	
0.00006459	832.3062306	12886018.	12.2577387	22.8205003	
0.00006920	891.7566756	12886018.	11.9372228	23.8072928	
0.00007382	951.2071207	12886018.	11.6567714	24.7940853	
0.00007843	1011.	12886018.	11.4093143	25.7808778	
0.00008304	1070.	12886018.	11.1893524	26.7676703	
0.00008766	1130.	12886018.	10.9925443	27.7544628	
0.00009227	1189.	12886018.	10.8154171	28.7412553	
0.00009688	1248.	12886018.	10.6551592	29.7280478	
0.0001015	1308.	12886018.	10.5094701	30.7148403	
0.0001061	1367.	12886018.	10.3764497	31.7016328	
0.0001107	1427.	12886018.	10.2545143	32.6884253	
0.0001153	1486.	12886018.	10.1423337	33.6752178	

0.0001200	1546.	12886018.	10.0387824	34.6620103	
0.0001246	1605.	12886018.	9.9429016	35.6488028	
0.0001292	1665.	12886018.	9.8538694	36.6355953	
0.0001338	1724.	12886018.	9.7709773	37.6223878	
0.0001384	1784.	12886018.	9.6936114	38.6091803	
0.0001430	1843.	12886018.	9.6212369	39.5959728	
0.0001476	1902.	12886018.	9.5533857	40.5827653	
0.0001522	1962.	12886018.	9.4896467	41.5695578	
0.0001569	2021.	12886018.	9.4296571	42.5563503	
0.0001615	2081.	12886018.	9.3730955	43.5431428	
0.0001661	2140.	12886018.	9.3196762	44.5299353	
0.0001707	2200.	12886018.	9.2691444	45.5167278	
0.0001753	2259.	12886018.	9.2212722	46.5035203	
0.0001799	2319.	12886018.	9.1758549	47.4903128	
0.0001892	2437.	12886018.	9.0916669	49.4638978	
0.0001984	2553.	12867873.	9.0178110	50.0000000	Y
0.0002076	2661.	12814995.	8.9556668	50.0000000	Y
0.0002168	2762.	12735668.	8.9033976	50.0000000	Y
0.0002261	2857.	12636660.	8.8594472	50.0000000	Y
0.0002353	2947.	12523519.	8.8224835	50.0000000	Y
0.0002445	3032.	12400798.	8.7913544	50.0000000	Y
0.0002537	3114.	12270482.	8.7653722	50.0000000	Y
0.0002630	3191.	12135817.	8.7436612	50.0000000	Y
0.0002722	3266.	11997845.	8.7257583	50.0000000	Y
0.0002814	3337.	11859136.	8.7109318	50.0000000	Y
0.0002907	3403.	11709082.	8.6969210	50.0000000	Y
0.0002999	3463.	11548804.	8.6833988	50.0000000	Y
0.0003091	3518.	11382093.	8.6702164	50.0000000	Y
0.0003183	3569.	11210660.	8.6572587	50.0000000	Y
0.0003276	3615.	11036149.	8.6444551	50.0000000	Y
0.0003368	3658.	10860727.	8.6319164	50.0000000	Y
0.0003460	3697.	10685141.	8.6197235	50.0000000	Y
0.0003552	3734.	10510298.	8.6078814	50.0000000	Y
0.0003645	3768.	10337419.	8.5962505	50.0000000	Y
0.0003737	3799.	10166802.	8.5846780	50.0000000	Y
0.0003829	3829.	9998231.	8.5733578	50.0000000	Y
0.0003922	3856.	9832594.	8.5623611	50.0000000	Y
0.0004014	3882.	9670971.	8.5514425	50.0000000	Y
0.0004106	3906.	9512303.	8.5411134	50.0000000	Y
0.0004198	3928.	9356574.	8.5304926	50.0000000	Y
0.0004291	3949.	9204591.	8.5203154	50.0000000	Y
0.0004383	3969.	9056808.	8.5104358	50.0000000	Y
0.0004475	3988.	8911432.	8.5004647	50.0000000	Y
0.0004567	4006.	8770532.	8.4908224	50.0000000	Y
0.0004660	4022.	8632465.	8.4814065	50.0000000	Y
0.0004752	4038.	8498489.	8.4718989	50.0000000	Y
0.0004844	4053.	8367270.	8.4630005	50.0000000	Y
0.0004937	4067.	8239624.	8.4539033	50.0000000	Y
0.0005029	4081.	8115397.	8.4451225	50.0000000	Y
0.0005121	4094.	7993906.	8.4365097	50.0000000	Y
0.0005213	4106.	7875980.	8.4278905	50.0000000	Y
0.0005306	4118.	7760870.	8.4195961	50.0000000	Y
0.0005398	4128.	7648350.	8.4113726	50.0000000	Y
0.0005490	4139.	7539520.	8.4033822	50.0000000	Y
0.0005859	4177.	7128466.	8.3720674	50.0000000	Y
0.0006228	4208.	6757022.	8.3431710	50.0000000	Y
0.0006597	4235.	6419493.	8.3159095	50.0000000	Y
0.0006966	4258.	6112300.	8.2902421	50.0000000	Y
0.0007336	4278.	5832068.	8.2659541	50.0000000	Y
0.0007705	4296.	5575391.	8.2431594	50.0000000	Y
0.0008074	4311.	5339620.	8.2217589	50.0000000	Y
0.0008443	4325.	5122129.	8.2010259	50.0000000	Y
0.0008812	4337.	4921209.	8.1816261	50.0000000	Y
0.0009181	4348.	4735357.	8.1633868	50.0000000	Y
0.0009550	4357.	4562576.	8.1455641	50.0000000	Y
0.0009919	4366.	4401604.	8.1291032	50.0000000	Y
0.0010288	4374.	4251574.	8.1132197	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	307.3000000000	4374.

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	-5.000	0.00	N.A.	No	0.00	127531.
2	5.0000	10.6120	Yes	No	127531.	243547.
3	10.0000	15.0000	No	Yes	N.A.	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 = 0.470000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 307300.0 lbs

Max moment at pile head

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	0.4700	-3670507.	103121.	0.00	70543.	1.08E+10	-729.235	1210.	0.00
0.1300	0.4696	-3510442.	101931.	-5.19E-04	67860.	1.08E+10	-760.415	2526.	0.00
0.2600	0.4684	-3351985.	100720.	-9.94E-04	65203.	1.18E+10	-791.145	2635.	0.00
0.3900	0.4665	-3195242.	99463.	-0.00142	62575.	1.21E+10	-821.410	2747.	0.00
0.5200	0.4639	-3040300.	98158.	-0.00182	59978.	1.24E+10	-851.139	2862.	0.00
0.6500	0.4608	-2887246.	96808.	-0.00219	57412.	1.26E+10	-880.263	2980.	0.00
0.7800	0.4571	-2736163.	95412.	-0.00253	54879.	1.28E+10	-908.711	3101.	0.00
0.9100	0.4529	-2587131.	93973.	-0.00286	52380.	1.29E+10	-936.409	3225.	0.00
1.0400	0.4482	-2440227.	92491.	-0.00316	49917.	1.29E+10	-963.283	3353.	0.00
1.1700	0.4430	-2295526.	90968.	-0.00345	47491.	1.29E+10	-989.259	3483.	0.00
1.3000	0.4374	-2153099.	89404.	-0.00372	45103.	1.29E+10	-1017.	3626.	0.00
1.4300	0.4314	-2013021.	87796.	-0.00397	42755.	1.29E+10	-1044.	3776.	0.00
1.5600	0.4251	-1875369.	86146.	-0.00421	40447.	1.29E+10	-1071.	3932.	0.00
1.6900	0.4183	-1740214.	84454.	-0.00442	38181.	1.29E+10	-1097.	4093.	0.00
1.8200	0.4113	-1607630.	82722.	-0.00463	35958.	1.29E+10	-1123.	4259.	0.00
1.9500	0.4039	-1477684.	80952.	-0.00481	33780.	1.29E+10	-1147.	4430.	0.00
2.0800	0.3962	-1350444.	79144.	-0.00499	31646.	1.29E+10	-1170.	4608.	0.00
2.2100	0.3883	-1225974.	77301.	-0.00514	29560.	1.29E+10	-1193.	4792.	0.00
2.3400	0.3802	-1104335.	75424.	-0.00528	27520.	1.29E+10	-1214.	4982.	0.00
2.4700	0.3718	-985587.	73514.	-0.00541	25529.	1.29E+10	-1234.	5179.	0.00
2.6000	0.3633	-869786.	71573.	-0.00552	23588.	1.29E+10	-1254.	5383.	0.00
2.7300	0.3546	-756985.	69603.	-0.00562	21697.	1.29E+10	-1272.	5595.	0.00
2.8600	0.3458	-647236.	67605.	-0.00570	19857.	1.29E+10	-1289.	5816.	0.00
2.9900	0.3368	-540586.	65582.	-0.00578	18069.	1.29E+10	-1305.	6045.	0.00
3.1200	0.3278	-437082.	63534.	-0.00584	16333.	1.29E+10	-1320.	6283.	0.00
3.2500	0.3186	-336764.	61464.	-0.00588	14651.	1.29E+10	-1334.	6531.	0.00
3.3800	0.3094	-239672.	59374.	-0.00592	13024.	1.29E+10	-1347.	6789.	0.00
3.5100	0.3001	-145844.	57264.	-0.00594	11451.	1.29E+10	-1358.	7059.	0.00
3.6400	0.2909	-55312.	55137.	-0.00595	9933.	1.29E+10	-1369.	7340.	0.00
3.7700	0.2816	31893.	52995.	-0.00595	9540.	1.29E+10	-1378.	7635.	0.00
3.9000	0.2723	115742.	50839.	-0.00595	10946.	1.29E+10	-1386.	7943.	0.00
4.0300	0.2630	196210.	48670.	-0.00593	12295.	1.29E+10	-1394.	8266.	0.00
4.1600	0.2538	273275.	46491.	-0.00590	13587.	1.29E+10	-1400.	8606.	0.00
4.2900	0.2446	346917.	44303.	-0.00586	14822.	1.29E+10	-1405.	8963.	0.00
4.4200	0.2355	417118.	42107.	-0.00581	15999.	1.29E+10	-1409.	9336.	0.00
4.5500	0.2265	483865.	39906.	-0.00576	17118.	1.29E+10	-1412.	9725.	0.00
4.6800	0.2175	547149.	37703.	-0.00570	18179.	1.29E+10	-1413.	10132.	0.00
4.8100	0.2087	606962.	35499.	-0.00563	19181.	1.29E+10	-1413.	10558.	0.00
4.9400	0.2000	663302.	33297.	-0.00555	20126.	1.29E+10	-1411.	11004.	0.00
5.0700	0.1914	716170.	31159.	-0.00547	21012.	1.29E+10	-1331.	10847.	0.00
5.2000	0.1829	765759.	29088.	-0.00538	21844.	1.29E+10	-1324.	11290.	0.00

L1=3.7'

5.3300	0.1746	812081.	27029.	-0.00528	22620.	1.29E+10	-1316.	11755.	0.00
5.4600	0.1664	855154.	24984.	-0.00518	23343.	1.29E+10	-1306.	12242.	0.00
5.5900	0.1584	894999.	22955.	-0.00508	24011.	1.29E+10	-1295.	12753.	0.00
5.7200	0.1506	931640.	20944.	-0.00496	24625.	1.29E+10	-1283.	13290.	0.00
5.8500	0.1430	965104.	18953.	-0.00485	25186.	1.29E+10	-1270.	13854.	0.00
5.9800	0.1355	995423.	16984.	-0.00473	25694.	1.29E+10	-1255.	14448.	0.00
6.1100	0.1282	1022630.	15039.	-0.00461	26150.	1.29E+10	-1239.	15074.	0.00
6.2400	0.1211	1046763.	13120.	-0.00448	26555.	1.29E+10	-1221.	15733.	0.00
6.3700	0.1142	1067863.	11229.	-0.00436	26909.	1.29E+10	-1203.	16428.	0.00
6.5000	0.1075	1085975.	9369.	-0.0[illegible]	[illegible]	[illegible]29E+10	-1183.	17163.	0.00
6.6300	0.1010	1101145.	7540.	-0.0[illegible]	[illegible]	[illegible]29E+10	-1162.	17940.	0.00
6.7600	0.09474	1113424.	5768.	-0.0[illegible]	[illegible]	[illegible]29E+10	-1110.	18272.	0.00
6.8900	0.08867	1122938.	4084.	-0.00382	27832.	1.29E+10	-1050.	18474.	0.00
7.0200	0.08281	1129831.	2491.	-0.00369	27948.	1.29E+10	-991.390	18676.	0.00
7.1500	0.07716	1134246.	989.6936	-0.00355	28022.	1.29E+10	-933.795	18878.	0.00
7.2800	0.07173	1136322.	-423.005	-0.00341	28056.	1.29E+10	-877.358	19080.	0.00
7.4100	0.06652	1136198.	-1749.	-0.00328	28054.	1.29E+10	-822.167	19282.	0.00
7.5400	0.06151	1134007.	-2989.	-0.00314	28018.	1.29E+10	-768.306	19484.	0.00
7.6700	0.05673	1129880.	-4147.	-0.00300	27948.	1.29E+10	-715.851	19686.	0.00
7.8000	0.05215	1123946.	-5224.	-0.00286	27849.	1.29E+10	-664.877	19888.	0.00
7.9300	0.04779	1116328.	-6222.	-0.00273	27721.	1.29E+10	-615.453	20090.	0.00
8.0600	0.04364	1107148.	-7145.	-0.00259	27567.	1.29E+10	-567.641	20292.	0.00
8.1900	0.03970	1096522.	-7995.	-0.00246	27389.	1.29E+10	-521.500	20494.	0.00
8.3200	0.03596	1084563.	-8774.	-0.00233	27189.	1.29E+10	-477.087	20696.	0.00
8.4500	0.03243	1071381.	-9485.	-0.00220	26968.	1.29E+10	-434.450	20898.	0.00
8.5800	0.02910	1057079.	-10131.	-0.00207	26728.	1.29E+10	-393.635	21100.	0.00
8.7100	0.02597	1041757.	-10714.	-0.00194	26471.	1.29E+10	-354.685	21302.	0.00
8.8400	0.02304	1025512.	-11239.	-0.00182	26199.	1.29E+10	-317.636	21504.	0.00
8.9700	0.02030	1008435.	-11707.	-0.00169	25912.	1.29E+10	-282.523	21706.	0.00
9.1000	0.01776	990611.	-12122.	-0.00157	25614.	1.29E+10	-249.376	21908.	0.00
9.2300	0.01540	972123.	-12486.	-0.00145	25304.	1.29E+10	-218.220	22110.	0.00
9.3600	0.01322	953048.	-12804.	-0.00134	24984.	1.29E+10	-189.080	22312.	0.00
9.4900	0.01122	933457.	-13078.	-0.00122	24655.	1.29E+10	-161.973	22514.	0.00
9.6200	0.00940	913418.	-13311.	-0.00111	24319.	1.29E+10	-136.915	22716.	0.00
9.7500	0.00775	892992.	-13507.	-0.00100	23977.	1.29E+10	-113.921	22918.	0.00
9.8800	0.00628	872238.	-13668.	-8.96E-04	23629.	1.29E+10	-93.000	23120.	0.00
10.0100	0.00496	851206.	-17332.	-7.91E-04	23276.	1.29E+10	-4604.	1447931.	0.00
10.1400	0.00381	818921.	-24059.	-6.90E-04	22735.	1.29E+10	-4021.	1647911.	0.00
10.2700	0.00281	776803.	-29789.	-5.94E-04	22029.	1.29E+10	-3325.	1847891.	0.00
10.4000	0.00195	726547.	-34384.	-5.03E-04	21186.	1.29E+10	-2566.	2047871.	0.00
10.5300	0.00124	670005.	-37779.	-4.18E-04	20238.	1.29E+10	-1786.	2247850.	0.00
10.6600	6.50E-04	609079.	-39967.	-3.41E-04	19217.	1.29E+10	-1021.	2447830.	0.00
10.7900	1.77E-04	545634.	-40997.	-2.71E-04	18153.	1.29E+10	-299.743	2647810.	0.00

Max moment in segment 2

10.9200	-1.94E-04	481427.	-40955.	-2.09E-04	17077.	1.29E+10	354.4305	2847790.	0.00
11.0500	-4.74E-04	418056.	-39956.	-1.54E-04	16014.	1.29E+10	926.0284	3047770.	0.00
11.1800	-6.75E-04	356913.	-38138.	-1.07E-04	14989.	1.29E+10	1405.	3247750.	0.00
11.3100	-8.08E-04	299169.	-35648.	-6.75E-05	14021.	1.29E+10	1787.	3447730.	0.00
11.4400	-8.85E-04	245756.	-32640.	-3.45E-05	13126.	1.29E+10	2070.	3647709.	0.00
11.5700	-9.16E-04	197366.	-29263.	-7.64E-06	12314.	1.29E+10	2259.	3847689.	0.00
11.7000	-9.09E-04	154461.	-25661.	1.37E-05	11595.	1.29E+10	2359.	4047669.	0.00
11.8300	-8.73E-04	117289.	-21967.	3.01E-05	10972.	1.29E+10	2378.	4247649.	0.00
11.9600	-8.15E-04	85897.	-18299.	4.24E-05	10446.	1.29E+10	2324.	4447629.	0.00
12.0900	-7.41E-04	60156.	-14764.	5.12E-05	10014.	1.29E+10	2208.	4647609.	0.00
12.2200	-6.55E-04	39784.	-11453.	5.73E-05	9672.	1.29E+10	2037.	4847589.	0.00
12.3500	-5.62E-04	24366.	-8446.	6.12E-05	9414.	1.29E+10	1819.	5047568.	0.00
12.4800	-4.65E-04	13374.	-5808.	6.35E-05	9230.	1.29E+10	1563.	5247548.	0.00
12.6100	-3.64E-04	6184.	-3597.	6.46E-05	9109.	1.29E+10	1272.	5447528.	0.00
12.7400	-2.63E-04	2090.	-1863.	6.51E-05	9040.	1.29E+10	951.5580	5647508.	0.00
12.8700	-1.61E-04	310.3210	-649.660	6.53E-05	9011.	1.29E+10	603.5958	5847488.	0.00
13.0000	-5.92E-05	0.00	0.00	6.53E-05	9005.	1.29E+10	229.3015	3023734.	0.00

L2=13'-3.7'= 9.3'

* 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 interpolated 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 = 0.47000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = -3670507. inch-lbs
Maximum shear force = 103121. 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 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	0.4700	S, rad	0.00	307300.	0.4700	0.00	103121.	-3670507.

Maximum pile-head deflection = 0.4700000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

Summary of Warning Messages

The following warning was reported 288 times

**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.

Lateral Pile Analyses
(Plastic Moment Applied)

Table 3 - LPile Output Plastic Hinge Summary
MaineDOT - Puddle Dock Bridge
GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0026189.01
CALCULATED BY E. Tome 7/3/2024
CHECKED BY A. Blaisdell 7/11/2024

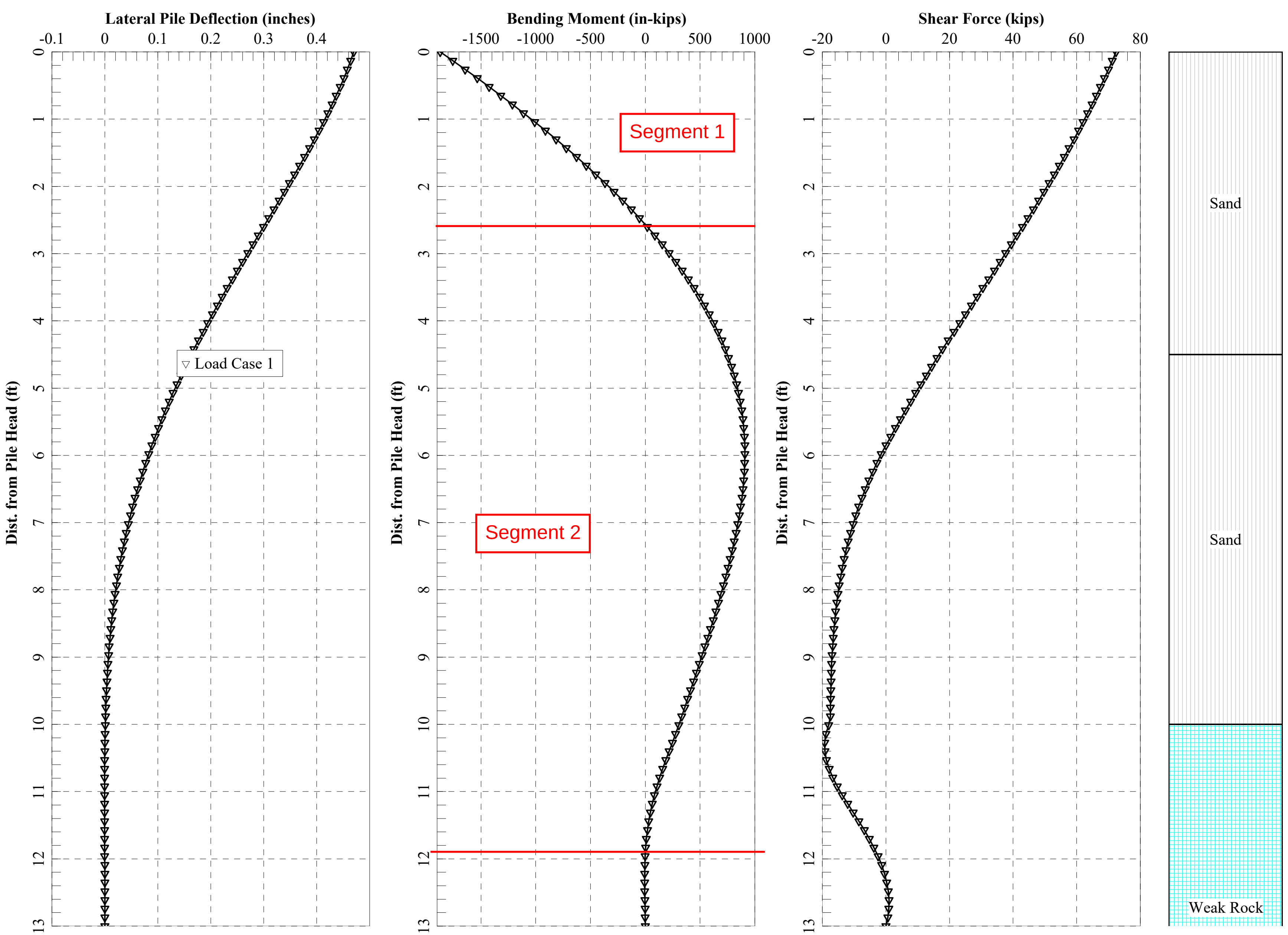
Abutment 1									
Pile Section	**Pile Length (ft)**	**Axial Load[2] (kips)**	**Deflection at Pile Head (in)**	**Shear Force at Pile Head (kips)**	**Total Stress at Pile Head (ksi)**	**Max Moment in Upper Section (in-kip)**	**Upper Section Length[3] (ft)**	**Max Moment in Lower Section (in-kip)**	**Lower Section Length[3] (ft)**
HP 14x89 (Weak axis) Plastic Hinge	13	357.2	0.47	72.4	56.1	-1871.4	2.6	909.1	9.3

Notes:

1. Soil layering and properties are presented in Table 1.
2. The axial load is the maximum Factored Axial Load provided by MaineDOT.
3. The upper section length is measured from the top of pile to first moment inflection point. The lower section length is measured between first and second moment inflection points.
4. Plastic hinge case included applied moment of 1,871 in-kips provided by MaineDOT and applied deflection at pile head.

HP14x89 (Weak Axis) Plastic Hinge Check

Applied at Pile Head: Mp= 1,871,400 in-lbs, Defl. = 0.47 in, Axial Load = 357.2 kips



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

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

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

Path to file locations:
\09 Jobs\0026100s\09.0026189.00 - MEDOT - Puddle Dock Bridge\09.0026189.01\Work\Calcs\LPile\14X89 Plastic Hinge\

Name of input data file:
AB1_Exp_dense fill.lp12d

Name of output report file:
AB1_Exp_dense fill.lp12o

Name of plot output file:
AB1_Exp_dense fill.lp12p

Name of runtime message file:
AB1_Exp_dense fill.lp12r


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

               Date:  July 3, 2024               Time:  12:43:48


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

Project Name: Albion Puddle Dock Bridge


Job Number: 09.0026189.00


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                                   =       13.000 ft
Depth of ground surface below top of pile              =      -5.0000 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.
```

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.7000
2	13.000	14.7000

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

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                   =    13.000000 ft
   Pile width                                          =    13.800000 in


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

The soil profile is modelled using 3 layers

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

   Distance from top of pile to top of layer           =     -5.00000 ft
   Distance from top of pile to bottom of layer        =     4.500000 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                      =    34.000000 deg.
   Friction angle at bottom of layer                   =    34.000000 deg.
   Subgrade k at top of layer                          =   122.000000 pci
   Subgrade k at bottom of layer                       =   122.000000 pci


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

   Distance from top of pile to top of layer           =     4.500000 ft
   Distance from top of pile to bottom of layer        =    10.000000 ft
   Effective unit weight at top of layer               =   122.000000 pcf
```

Effective unit weight at bottom of layer = 122.000000 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 3 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 20.000000 ft
Effective unit weight at top of layer = 169.000000 pcf
Effective unit weight at bottom of layer = 169.000000 pcf
Uniaxial compressive strength at top of layer = 2000. psi
Uniaxial compressive strength at bottom of layer = 2000. psi
Initial modulus of rock at top of layer = 9183. psi
Initial modulus of rock at bottom of layer = 9183. psi
RQD of rock at top of layer = 0.0000 %
RQD of rock at bottom of layer = 0.0000 %
k rm of rock at top of layer = 0.0005000
k rm of rock at bottom of layer = 0.0005000

(Depth of the lowest soil layer extends 7.000 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weight of rock were outside the limits of 50 pcf to 150 pcf.

The maximum input value, in layer 1, for effective unit weight = 169.00 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	Uniaxial qu psi	RQD %	E50 or krm	kpy pci	Rock Mass Modulus psi
1	Sand	-5.000	125.0000	34.0000	--	--	--	122.0000	--
	(Reese, et al.)	4.5000	125.0000	34.0000	--	--	--	122.0000	--
2	Sand	4.5000	122.0000	32.0000	--	--	--	83.0000	--
	(Reese, et al.)	10.0000	122.0000	32.0000	--	--	--	83.0000	--
3	Weak	10.0000	169.0000	--	2000.	0.00	5.00E-04	--	9183.
	Rock	20.0000	169.0000	--	2000.	0.00	5.00E-04	--	9183.

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	4	y = 0.470000 in	M = -1871400. in-lbs	357200.	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	=	13.000000 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	357.200

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 357.200 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000445	42.0014396	9448389.	114.6977565	14.7768180	
0.00000889	84.0028793	9448389.	61.0238782	15.7148700	
0.00001334	126.0043189	9448389.	43.1325855	16.6529221	
0.00001778	168.0057586	9448389.	34.1869391	17.5909741	
0.00002223	210.0071982	9448389.	28.8195513	18.5290261	
0.00002667	252.0086378	9448389.	25.2412927	19.4670782	
0.00003112	294.0100775	9448389.	22.6853938	20.4051302	
0.00003556	336.0115171	9448389.	20.7684696	21.3431823	
0.00004001	378.0129568	9448389.	19.2775285	22.2812343	
0.00004445	420.0143964	9448389.	18.0847756	23.2192863	
0.00004890	462.0158360	9448389.	17.1088870	24.1573384	
0.00005334	504.0172757	9448389.	16.2956464	25.0953904	
0.00005779	546.0187153	9448389.	15.6075197	26.0334425	
0.00006223	588.0201549	9448389.	15.0176969	26.9714945	
0.00006668	630.0215946	9448389.	14.5065171	27.9095466	
0.00007113	672.0230342	9448389.	14.0592348	28.8475985	
0.00007557	714.0244739	9448389.	13.6645739	29.7856506	
0.00008002	756.0259135	9448389.	13.3137642	30.7237027	
0.00008446	798.0273531	9448389.	12.9998819	31.6617547	
0.00008891	840.0287928	9448389.	12.7173878	32.5998068	
0.00009335	882.0302324	9448389.	12.4617979	33.5378588	
0.00009780	924.0316721	9448389.	12.2294435	34.4759108	
0.0001022	966.0331117	9448389.	12.0172938	35.4139629	
0.0001067	1008.	9448389.	11.8228232	36.3520149	
0.0001111	1050.	9448389.	11.6439103	37.2900670	

0.0001156	1092.	9448389.	11.4787599	38.2281190	
0.0001200	1134.	9448389.	11.3258428	39.1661710	
0.0001245	1176.	9448389.	11.1838484	40.1042231	
0.0001289	1218.	9448389.	11.0516468	41.0422751	
0.0001334	1260.	9448389.	10.9282585	41.9803272	
0.0001378	1302.	9448389.	10.8128309	42.9183792	
0.0001423	1344.	9448389.	10.7046174	43.8564312	
0.0001467	1386.	9448389.	10.6029623	44.7944833	
0.0001511	1428.	9448389.	10.5072870	45.7325353	
0.0001556	1470.	9448389.	10.4170788	46.6705874	
0.0001600	1512.	9448389.	10.3318821	47.6086394	
0.0001645	1554.	9448389.	10.2512907	48.5466914	
0.0001689	1596.	9448389.	10.1749410	49.4847435	
0.0001734	1637.	9445158.	10.1030998	50.0000000	Y
0.0001823	1716.	9415172.	9.9744199	50.0000000	Y
0.0001912	1789.	9358901.	9.8633797	50.0000000	Y
0.0002000	1857.	9284796.	9.7668859	50.0000000	Y
0.0002089	1922.	9197783.	9.6827186	50.0000000	Y
0.0002178	1983.	9101858.	9.6089987	50.0000000	Y
0.0002267	2040.	8998843.	9.5444692	50.0000000	Y
0.0002356	2095.	8892311.	9.4875549	50.0000000	Y
0.0002445	2147.	8783382.	9.4373370	50.0000000	Y
0.0002534	2198.	8673653.	9.3928532	50.0000000	Y
0.0002623	2246.	8563329.	9.3535651	50.0000000	Y
0.0002712	2292.	8454022.	9.3186014	50.0000000	Y
0.0002801	2337.	8346074.	9.2874770	50.0000000	Y
0.0002889	2381.	8239771.	9.2597641	50.0000000	Y
0.0002978	2423.	8135356.	9.2350837	50.0000000	Y
0.0003067	2464.	8033028.	9.2130982	50.0000000	Y
0.0003156	2504.	7932952.	9.1935058	50.0000000	Y
0.0003245	2541.	7830857.	9.1748477	50.0000000	Y
0.0003334	2576.	7725829.	9.1565111	50.0000000	Y
0.0003423	2608.	7619035.	9.1384673	50.0000000	Y
0.0003512	2638.	7511592.	9.1207299	50.0000000	Y
0.0003601	2666.	7403761.	9.1034407	50.0000000	Y
0.0003690	2692.	7295687.	9.0864785	50.0000000	Y
0.0003779	2716.	7188650.	9.0696726	50.0000000	Y
0.0003867	2739.	7082417.	9.0533187	50.0000000	Y
0.0003956	2761.	6977602.	9.0373149	50.0000000	Y
0.0004045	2781.	6874660.	9.0215790	50.0000000	Y
0.0004134	2800.	6772765.	9.0062274	50.0000000	Y
0.0004223	2818.	6672733.	8.9908633	50.0000000	Y
0.0004312	2835.	6574530.	8.9761217	50.0000000	Y
0.0004401	2851.	6478490.	8.9615023	50.0000000	Y
0.0004490	2866.	6383842.	8.9470496	50.0000000	Y
0.0004579	2881.	6291643.	8.9330917	50.0000000	Y
0.0004668	2894.	6201216.	8.9190015	50.0000000	Y
0.0004757	2907.	6112527.	8.9055555	50.0000000	Y
0.0004845	2920.	6026109.	8.8921375	50.0000000	Y
0.0004934	2932.	5941443.	8.8788845	50.0000000	Y
0.0005023	2943.	5858704.	8.8661299	50.0000000	Y
0.0005112	2954.	5777794.	8.8532775	50.0000000	Y
0.0005201	2964.	5698964.	8.8407535	50.0000000	Y
0.0005290	2974.	5621530.	8.8285489	50.0000000	Y
0.0005646	3009.	5330159.	8.7814877	50.0000000	Y
0.0006001	3039.	5063842.	8.7369635	50.0000000	Y
0.0006357	3065.	4821173.	8.6953896	50.0000000	Y
0.0006712	3087.	4598847.	8.6560679	50.0000000	Y
0.0007068	3106.	4394674.	8.6191583	50.0000000	Y
0.0007424	3123.	4207193.	8.5840260	50.0000000	Y
0.0007779	3138.	4034191.	8.5511186	50.0000000	Y
0.0008135	3152.	3874549.	8.5196342	50.0000000	Y
0.0008491	3164.	3726372.	8.4901366	50.0000000	Y
0.0008846	3175.	3588697.	8.4615988	50.0000000	Y
0.0009202	3184.	3460653.	8.4346183	50.0000000	Y
0.0009558	3193.	3341330.	8.4093499	50.0000000	Y
0.0009913	3201.	3229453.	8.3846180	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	357.2000000000	3201.

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	-5.000	0.00	N.A.	No	0.00	110738.
2	4.5000	10.0862	Yes	No	110738.	257521.
3	10.0000	15.0000	No	Yes	N.A.	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 Moment (Loading Type 4)
Displacement of pile head = 0.470000 inches
Moment at pile head = -1871400.0 in-lbs
Axial load at pile head = 357200.0 lbs

Max moment at pile head
(provided by MaineDOT)

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	0.4700	-1871400.	72355.	-0.00353	56052.	9.27E+09	-732.843	1216.	0.00
0.1300	0.4643	-1757365.	71192.	-0.00383	53480.	9.27E+09	-758.017	2547.	0.00
0.2600	0.4580	-1645010.	69990.	-0.00412	50946.	9.44E+09	-782.534	2665.	0.00
0.3900	0.4514	-1534408.	68751.	-0.00438	48451.	9.45E+09	-806.329	2787.	0.00
0.5200	0.4444	-1425626.	67475.	-0.00462	45997.	9.45E+09	-829.332	2911.	0.00
0.6500	0.4370	-1318732.	66164.	-0.00485	43586.	9.45E+09	-851.471	3040.	0.00
0.7800	0.4292	-1213789.	64819.	-0.00506	41219.	9.45E+09	-872.680	3172.	0.00
0.9100	0.4212	-1110858.	63442.	-0.00525	38897.	9.45E+09	-892.893	3307.	0.00
1.0400	0.4129	-1009998.	62034.	-0.00543	36622.	9.45E+09	-912.051	3446.	0.00
1.1700	0.4043	-911264.	60596.	-0.00559	34394.	9.45E+09	-931.308	3594.	0.00
1.3000	0.3954	-814713.	59127.	-0.00573	32216.	9.45E+09	-951.931	3755.	0.00
1.4300	0.3864	-720403.	57627.	-0.00585	30089.	9.45E+09	-971.702	3923.	0.00
1.5600	0.3772	-628392.	56096.	-0.00597	28014.	9.45E+09	-990.592	4097.	0.00
1.6900	0.3678	-538734.	54537.	-0.00606	25991.	9.45E+09	-1009.	4278.	0.00
1.8200	0.3582	-451481.	52950.	-0.00614	24023.	9.45E+09	-1026.	4466.	0.00
1.9500	0.3486	-366682.	51338.	-0.00621	22110.	9.45E+09	-1042.	4662.	0.00
2.0800	0.3389	-284385.	49701.	-0.00627	20254.	9.45E+09	-1057.	4866.	0.00
2.2100	0.3291	-204633.	48041.	-0.00631	18455.	9.45E+09	-1071.	5078.	0.00
2.3400	0.3192	-127470.	46360.	-0.00633	16714.	9.45E+09	-1084.	5300.	0.00
2.4700	0.3093	-52933.	44658.	-0.00635	15033.	9.45E+09	-1097.	5531.	0.00
2.6000	0.2994	18939.	42939.	-0.00635	14266.	9.45E+09	-1108.	5773.	0.00
2.7300	0.2895	88113.	41202.	-0.00634	15826.	9.45E+09	-1118.	6026.	0.00
2.8600	0.2796	154558.	39450.	-0.00632	17325.	9.45E+09	-1128.	6292.	0.00
2.9900	0.2698	218244.	37685.	-0.00629	18762.	9.45E+09	-1136.	6571.	0.00
3.1200	0.2600	279145.	35906.	-0.00625	20136.	9.45E+09	-1144.	6863.	0.00
3.2500	0.2503	337237.	34117.	-0.00620	21446.	9.45E+09	-1150.	7172.	0.00
3.3800	0.2406	392498.	32317.	-0.00614	22692.	9.45E+09	-1156.	7496.	0.00
3.5100	0.2311	444909.	30510.	-0.00607	23875.	9.45E+09	-1161.	7836.	0.00
3.6400	0.2217	494454.	28696.	-0.00599	24992.	9.45E+09	-1164.	8192.	0.00
3.7700	0.2124	541120.	26879.	-0.00591	26045.	9.45E+09	-1166.	8564.	0.00
3.9000	0.2033	584898.	25059.	-0.00581	27032.	9.45E+09	-1167.	8955.	0.00
4.0300	0.1943	625784.	23239.	-0.00571	27955.	9.45E+09	-1166.	9364.	0.00
4.1600	0.1854	663773.	21422.	-0.00561	28812.	9.45E+09	-1164.	9794.	0.00
4.2900	0.1768	698869.	19608.	-0.00549	29603.	9.45E+09	-1161.	10245.	0.00
4.4200	0.1683	731075.	17801.	-0.00538	30330.	9.45E+09	-1156.	10718.	0.00
4.5500	0.1600	760400.	16049.	-0.00525	30991.	9.45E+09	-1089.	10619.	0.00
4.6800	0.1519	787004.	14357.	-0.00513	31591.	9.45E+09	-1080.	11095.	0.00
4.8100	0.1440	810906.	12680.	-0.00499	32131.	9.45E+09	-1070.	11595.	0.00
4.9400	0.1363	832129.	11018.	-0.00486	32609.	9.45E+09	-1059.	12122.	0.00
5.0700	0.1289	850698.	9375.	-0.00472	33028.	9.45E+09	-1047.	12677.	0.00
5.2000	0.1216	866641.	7752.	-0.00458	33388.	9.45E+09	-1034.	13261.	0.00

L1=2.6'

Max moment in segment 2

5.3300	0.1146	879988.	6151.	-0.0			-1019.	13879.	0.00
5.4600	0.1078	890773.	4573.	-0.0			-1004.	14531.	0.00
5.5900	0.1012	899034.	3020.	-0.0			-987.279	15220.	0.00
5.7200	0.09485	904809.	1494.	-0.00399	34249.	9.45E+09	-969.753	15949.	0.00
5.8500	0.08874	908141.	-4.777	-0.00384	34324.	9.45E+09	-951.251	16722.	0.00
5.9800	0.08287	909075.	-1454.	-0.00369	34345.	9.45E+09	-906.235	17060.	0.00
6.1100	0.07723	907720.	-2827.	-0.00354	34314.	9.45E+09	-854.545	17262.	0.00
6.2400	0.07182	904201.	-4121.	-0.00339	34235.	9.45E+09	-804.011	17464.	0.00
6.3700	0.06664	898643.	-5337.	-0.00324	34110.	9.45E+09	-754.715	17666.	0.00
6.5000	0.06170	891165.	-6476.	-0.00309	33941.	9.45E+09	-706.731	17868.	0.00
6.6300	0.05699	881885.	-7543.	-0.00295	33732.	9.45E+09	-660.125	18070.	0.00
6.7600	0.05250	870918.	-8537.	-0.00280	33484.	9.45E+09	-614.959	18272.	0.00
6.8900	0.04824	858374.	-9462.	-0.00266	33201.	9.45E+09	-571.287	18474.	0.00
7.0200	0.04420	844361.	-10321.	-0.00252	32885.	9.45E+09	-529.159	18676.	0.00
7.1500	0.04038	828982.	-11115.	-0.00238	32538.	9.45E+09	-488.617	18878.	0.00
7.2800	0.03677	812338.	-11847.	-0.00225	32163.	9.45E+09	-449.696	19080.	0.00
7.4100	0.03337	794525.	-12519.	-0.00211	31761.	9.45E+09	-412.426	19282.	0.00
7.5400	0.03017	775635.	-13135.	-0.00198	31335.	9.45E+09	-376.832	19484.	0.00
7.6700	0.02718	755757.	-13696.	-0.00186	30887.	9.45E+09	-342.931	19686.	0.00
7.8000	0.02437	734974.	-14206.	-0.00174	30418.	9.45E+09	-310.737	19888.	0.00
7.9300	0.02176	713368.	-14667.	-0.00162	29930.	9.45E+09	-280.254	20090.	0.00
8.0600	0.01933	691014.	-15082.	-0.00150	29426.	9.45E+09	-251.486	20292.	0.00
8.1900	0.01708	667985.	-15453.	-0.00139	28907.	9.45E+09	-224.426	20494.	0.00
8.3200	0.01500	644348.	-15783.	-0.00128	28373.	9.45E+09	-199.067	20696.	0.00
8.4500	0.01309	620167.	-16075.	-0.00117	27828.	9.45E+09	-175.392	20898.	0.00
8.5800	0.01134	595502.	-16332.	-0.00107	27272.	9.45E+09	-153.383	21100.	0.00
8.7100	0.00974	570409.	-16555.	-9.78E-04	26706.	9.45E+09	-133.015	21302.	0.00
8.8400	0.00829	544940.	-16748.	-8.86E-04	26131.	9.45E+09	-114.257	21504.	0.00
8.9700	0.00698	519143.	-16913.	-7.98E-04	25549.	9.45E+09	-97.077	21706.	0.00
9.1000	0.00580	493062.	-17052.	-7.15E-04	24961.	9.45E+09	-81.434	21908.	0.00
9.2300	0.00475	466737.	-17168.	-6.35E-04	24367.	9.45E+09	-67.287	22110.	0.00
9.3600	0.00382	440206.	-17263.	-5.60E-04	23769.	9.45E+09	-54.587	22312.	0.00
9.4900	0.00300	413501.	-17339.	-4.90E-04	23166.	9.45E+09	-43.282	22514.	0.00
9.6200	0.00229	386653.	-17399.	-4.24E-04	22561.	9.45E+09	-33.316	22716.	0.00
9.7500	0.00168	359688.	-17444.	-3.62E-04	21952.	9.45E+09	-24.629	22918.	0.00
9.8800	0.00116	332630.	-17477.	-3.05E-04	21342.	9.45E+09	-17.156	23120.	0.00
10.0100	7.24E-04	305500.	-18015.	-2.52E-04	20730.	9.45E+09	-672.505	1448140.	0.00
10.1400	3.70E-04	276705.	-18845.	-2.04E-04	20080.	9.45E+09	-391.523	1650841.	0.00
10.2700	8.68E-05	246932.	-19231.	-1.61E-04	19409.	9.45E+09	-103.105	1853542.	0.00
10.4000	-1.33E-04	216885.	-19174.	-1.23E-04	18731.	9.45E+09	175.0761	2056242.	0.00
10.5300	-2.97E-04	187245.	-18703.	-8.95E-05	18062.	9.45E+09	429.4339	2258943.	0.00
10.6600	-4.12E-04	158632.	-17861.	-6.09E-05	17417.	9.45E+09	650.2398	2461644.	0.00
10.7900	-4.87E-04	131587.	-16705.	-3.70E-05	16807.	9.45E+09	831.2809	2664344.	0.00
10.9200	-5.27E-04	106553.	-15301.	-1.73E-05	16242.	9.45E+09	969.4325	2867045.	0.00
11.0500	-5.41E-04	83868.	-13714.	-1.61E-06	15731.	9.45E+09	1064.	3069746.	0.00
11.1800	-5.33E-04	63765.	-12013.	1.06E-05	15277.	9.45E+09	1117.	3272446.	0.00
11.3100	-5.08E-04	46375.	-10259.	1.97E-05	14885.	9.45E+09	1131.	3475147.	0.00
11.4400	-4.71E-04	31734.	-8511.	2.61E-05	14555.	9.45E+09	1111.	3677848.	0.00
11.5700	-4.26E-04	19793.	-6817.	3.04E-05	14285.	9.45E+09	1060.	3880548.	0.00
11.7000	-3.76E-04	10430.	-5222.	3.29E-05	14074.	9.45E+09	985.1732	4083249.	0.00
11.8300	-3.24E-04	3465.	-3759.	3.40E-05	13917.	9.45E+09	889.5267	4285950.	0.00
11.9600	-2.70E-04	-1337.	-2459.	3.42E-05	13869.	9.45E+09	777.6394	4488650.	0.00
12.0900	-2.17E-04	-4245.	-1343.	3.37E-05	13935.	9.45E+09	652.8826	4691351.	0.00
12.2200	-1.65E-04	-5565.	-430.041	3.29E-05	13964.	9.45E+09	517.7406	4894052.	0.00
12.3500	-1.14E-04	-5624.	265.3222	3.20E-05	13966.	9.45E+09	373.7501	5096752.	0.00
12.4800	-6.52E-05	-4773.	729.6341	3.11E-05	13946.	9.45E+09	221.5214	5299453.	0.00
12.6100	-1.73E-05	-3382.	949.8807	3.05E-05	13915.	9.45E+09	60.8460	5502154.	0.00
12.7400	2.98E-05	-1843.	912.2371	3.00E-05	13880.	9.45E+09	-109.107	5704854.	0.00
12.8700	7.64E-05	-569.206	601.3245	2.98E-05	13852.	9.45E+09	-289.499	5907555.	0.00
13.0000	1.23E-04	0.00	0.00	2.98E-05	13839.	9.45E+09	-481.430	3055128.	0.00

L2=11.9'-2.6'= 9.3'

* 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 interpolated 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            =     0.47000000 inches
Computed slope at pile head     =    -0.0035278 radians
Maximum bending moment          =     -1871400. inch-lbs
Maximum shear force             =        72355. 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            =             7
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	0.4700	M, in-lb	-1871400.	357200.	0.4700	-0.00353	72355.	-1871400.

Maximum pile-head deflection = 0.4700000000 inches
Maximum pile-head rotation = -0.0035278284 radians = -0.202130 deg.

Summary of Warning Messages

The following warning was reported 192 times

**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.