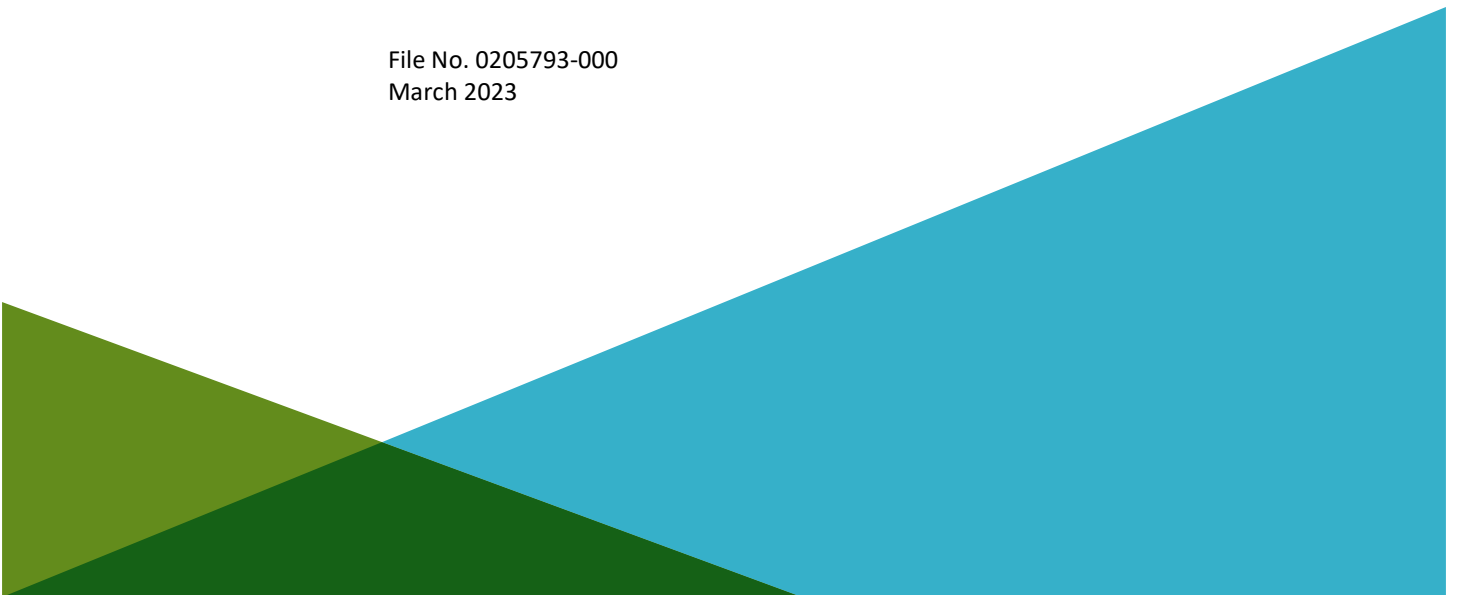


GEOTECHNICAL DESIGN REPORT
MATTAGODUS STREAM BRIDGE OVER
MATTAGODUS STREAM NO. 5116
MAINEDOT WIN 022266.00
ROUTE 170, WEBSTER PLANTATION, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

for
Maine Department of Transportation
Augusta, Maine

File No. 0205793-000
March 2023





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Maine Department of Transportation
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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Geotechnical Design Report
Mattagodus Stream Bridge over Mattagodus Stream No. 5116
MaineDOT WIN 022266.00
Route 170, Webster Plantation, Maine

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit herewith our report entitled, "Geotechnical Design Report, Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00, Route 170, Webster Plantation, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our proposed work scope and fee estimate document, dated 24 June 2022 and executed by you on 11 July 2022 (Bridge Program Assignment Letter #1) and the provisions of our Multi-PIN Project Contract Number 2020062300000000766, dated 27 August 2020.

Introduction

This GDR presents the results of preliminary (Phase I) and final design (Phase II) phase subsurface investigation and laboratory testing programs, technical evaluations, and geotechnical design recommendations completed by Haley & Aldrich on behalf of MaineDOT for the proposed bridge that will carry vehicular traffic on Route 170 (Park Street) over Mattagodus Stream in Webster Plantation, Maine (see Figure 1, Project Locus).

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

Plan locations of test borings (borings) are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 East Zone. The project elevation datum and elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88). The proposed limit work extends from Sta. 2+50 (west) to Sta. 7+25 (east).

PROJECT LOCATION AND EXISTING BRIDGE STRUCTURE

The existing bridge carries eastbound (EB) and westbound (WB) vehicular traffic on Route 170 (Park Street) over Mattagodus Stream in Webster Plantation, Maine, approximately 0.4 miles southeast of the Kingman Township town line.

Based on our review of the historic bridge drawings and the approved Preliminary Design Report (PDR), dated 18 October 2021, it is our understanding that the existing bridge was originally constructed in 1918 and consists of an approximate 40-ft long, 20-ft wide (curb to curb), single-span structure supported on two unreinforced, soil-supported, cast-in-place (CIP) concrete abutments that are presumably bearing on soil (Abutment No. 1; west) and bedrock (Abutment No. 2; east). The superstructure consists of concrete tee beams with a concrete deck and bituminous concrete wearing surface. The bridge and roadway were paved in 2019.

Historic bridge drawings that were provided by MaineDOT are included in Appendix C.

PROPOSED BRIDGE STRUCTURE

Because MaineDOT determined the existing bridge to be structurally deficient, multiple bridge replacement alternatives were developed and evaluated during preliminary design. Each alternative considered several factors including but not limited to overall project cost, maintenance of traffic, and future bridge maintenance. The bridge replacement alternative recommended by MaineDOT in the PDR consists of a 76-ft long, single-span bridge that is supported on two CIP concrete abutments at the stations and elevations summarized below.

Substructure	Station at Centerline of Alignment (ft)	Proposed Bottom of Abutment Elevation (ft, NAVD 88)
Abutment No. 1	Sta. 4+60	El. 318.6
Abutment No. 2	Sta. 5+36	El. 316.6

The bridge superstructure will be constructed using either precast concrete NEXT F beams or a GBeam composite bridge system. The bridge structure will be approximately 28-ft wide (curb to curb) and will consist of two, 11-ft wide travel lanes and two, 3-ft wide shoulders with a 9-in. thick composite, CIP concrete deck and integral concrete wearing surface. Based on our discussions with you and the subsurface conditions present at the site, it is our understanding that Abutment No. 1 will be a pile-supported integral abutment and Abutment No. 2 will be a semi-integral abutment supported by a mass concrete footing bearing on bedrock.

Based on our review of profile and cross section drawings developed by MaineDOT for the recommended bridge alternatives, it is our understanding that the proposed horizontal and vertical alignments will approximately match existing. The vertical curve across the replacement bridge will be slightly higher than the existing and therefore, slight raises in grade are anticipated (i.e., less than 1 ft).

Traffic will be maintained during construction via a one-lane temporary bridge located on the downstream side of the existing bridge.

Existing and proposed site conditions are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.

Geologic Setting

Based on our review of the Maine Geological Survey's (MGS's) Surficial Geologic Map of Maine (1985), near-surface soils that are present in the vicinity of the site consist of stream alluvium (sand, gravel, silt), swamp deposits (peat, muck, clay, silt, and sand) and/or esker deposits consisting of gravel and sand.

According to the MGS's Bedrock Geologic Map of Maine (1985), bedrock at or in the immediate vicinity of the site is primarily mapped as undifferentiated pelite and sandstone of the Allsbury Formation or unnamed limestone.

More specific information on the soil and bedrock conditions present at the site is presented in subsequent sections of this report.

Subsurface Exploration Programs

Preliminary and final design phase subsurface exploration programs (investigations) were completed at the site in support of the subject project. The investigations were developed, planned, coordinated, and executed by MaineDOT. MaineDOT personnel were present on-site during drilling and prepared boring field logs. Upon completion of the borings, Haley & Aldrich coordinated with MaineDOT to obtain and review soil and rock samples and finalized the boring logs. The information summarized in the following sections of this report is based on our review of the boring logs and discussions with MaineDOT.

The soil and bedrock samples collected by MaineDOT during the preliminary and final design phase investigations were preserved in plastic containers and wooden boxes, respectively. The samples that were not submitted for laboratory testing are available for review upon request. The available soil and bedrock samples are currently being stored at the Haley & Aldrich storage facility in Portland, Maine.

"As-drilled" boring locations and ground surface elevations were determined in the field by MaineDOT using GPS survey equipment upon the completion of drilling and were provided to Haley & Aldrich. The as-drilled plan locations and ground surface elevations of the preliminary and final design phase borings are summarized in Table I and are shown on Figure 2.

All drilling and sampling activities were performed in accordance with MaineDOT requirements.

PRELIMINARY DESIGN PHASE SUBSURFACE INVESTIGATION

MaineDOT completed a preliminary design phase subsurface exploration program (investigation) at the site in June 2018. The preliminary investigation consisted of three borings, designated BB-WPMS-101

through BB-WPMS-103. Boring BB-WPMS-101 was drilled near the north end of proposed Abutment No. 1 and borings BB-WPMS-102 and BB-WPMS-103 were drilled near the north and south ends of proposed Abutment No. 2, respectively.

The borings were drilled by MaineDOT using a CME 45C drill rig. Borings were advanced to depths ranging from approximately 21 to 47 ft below ground surface (BGS) using cased-washed drilling methods and a combination of solid-stem augers (SSAs) and 3-in. (NW-size) outside diameter (OD) steel casing.

Soil samples were generally collected at standard, 5-ft intervals by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the boring logs. The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the logs. The uncorrected SPT N-value (N_{uncorrected}) is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The drill rig was equipped with a calibrated automatic hammer per MaineDOT requirements. The energy-corrected SPT N-value (N₆₀), which is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.928; 92.8 percent theoretical hammer efficiency) divided by 0.6, is also provided on the logs.

Each boring was advanced approximately 9 to 10 ft into bedrock using a 2.0-in. (NQ-size) ID, diamond-tipped core barrel.

FINAL DESIGN PHASE SUBSURFACE INVESTIGATION

MaineDOT completed a design phase investigation at the site in July 2022. The investigation consisted of one boring, designated BB-WPMS-201, which was drilled at/near the south side of proposed Abutment No. 1.

The boring was drilled by MaineDOT using the same equipment and drilling and sampling means and methods to those described in the previous section of this report. The boring was advanced to a depth of approximately 47 ft BGS, which includes approximately 11 ft of penetration into bedrock. The hammer efficiency factor for the automatic hammer used to complete the design phase investigation was 0.974 (97.4 percent theoretical hammer efficiency) as shown on the boring logs.

Generalized Subsurface Conditions

The subsurface conditions present at the site generally consist of fill soils overlying naturally-deposited glacial till (Abutment No. 1) or bedrock (Abutment No. 2). Refer to Table II for a detailed summary of the soil units and encountered thicknesses in each boring. A general description of each soil/bedrock unit is provided separately, below. Detailed soil and bedrock descriptions are provided on the boring logs included Appendix A. Refer to Figure 2 for a graphical representation of the subsurface conditions present along the proposed bridge alignment.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Bituminous Concrete	0.4 to 0.6	An approximate 5 to 7-in. thick layer of bituminous concrete <i>(encountered in all borings)</i>
Fill	12 to 15	Very loose to medium dense SAND with varying amounts of silt and gravel (SM) <i>(encountered in all borings)</i>
Glacial Till	20	Medium dense to very dense SAND with varying amounts of silt and gravel; dense sandy GRAVEL with little silt; very stiff SILT with some sand and little gravel (ML) <i>(encountered in borings BB-WPMS-101 and BB-WPMS-201 at Abutment No. 1)</i>
Bedrock		Bedrock was encountered in each boring. At Abutment No. 1 the top of bedrock was encountered at depths ranging from approximately 33 to 35 ft BGS (El. 293 to El. 294). At Abutment No. 2 the top of bedrock was encountered at depths ranging from approximately 12 to 16 ft BGS (El. 313 to El. 317). In general, the top of bedrock and slopes down from east to west.

Please note that soil descriptions provided on the boring logs do not represent actual field conditions other than at the specific boring locations. The actual conditions encountered between boring locations may vary from those described herein.

BEDROCK CONDITIONS

As stated previously, approximately 9 to 11 ft of bedrock was cored in the borings. The sampled and recovered bedrock generally consisted of moderately hard to hard, slightly to severely weathered, brown-grey to black, aphanitic, SHALE. Primary joints were observed dipping at low to vertical angles and are very close to closely spaced, tight to open, and joint surfaces were smooth to rough.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for the bedrock encountered in the borings drilled at the site ranged from 0 to 100 percent (average = 40 percent), indicating variable rock quality; from very poor to excellent in accordance with the MaineDOT Geotechnical Section “Key to Soil and Rock Descriptions and Terms Field Identification Information” document, dated January 2020.

Detailed bedrock core data and descriptions are provided on Table III and on the boring logs in Appendix A. In addition, photographs of the recovered bedrock core samples are provided for reference in Appendix A.

GROUNDWATER CONDITIONS

The water level was measured by MaineDOT in each boring either during or after drilling, as shown on the boring logs provided in Appendix A. The measured water levels ranged from approximately 15 to 20 ft BGS (El. 308 to 313) near Abutment No. 1 (borings BB-WPMS-101 and BB-WPMS-201) and from

approximately 7 to 9 ft BGS (El. 320 to 322) near Abutment No. 2 (borings BB-WPMS-102 and BB-WPMS-103).

In general, water levels may fluctuate with season, precipitation, local soil/bedrock conditions, and excavation means and methods. Therefore, water levels may vary from those summarized above and provided on the testing boring logs included in Appendix A.

Geotechnical Laboratory Testing Programs

Preliminary and design phase laboratory testing programs were conducted by MaineDOT and Haley & Aldrich on representative soil and rock samples collected during the subsurface investigations to aid in soil classification and determination of engineering soil and rock properties. Soil testing completed during the preliminary design phase was performed by the MaineDOT Central Laboratory. Rock testing completed during the final design phase was completed by GeoTesting Express, Inc. of Acton, Massachusetts. All Laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures. A summary of the lab testing results is provided below.

Laboratory Test	ASTM Test Designation	Geologic Unit	No. of Tests	Range in Test Results ¹
Moisture Content	ASTM D2216-98	Fill	4	7.0% < WC < 16.7%
Grain Size	ASTM D422	Fill	4	<u>AASHTO Classification:</u> A-1-b, A-2-4 <u>USCS Classification:</u> SM, SC-SM
Compressive Strength and Elastic Moduli of Rock	ASTM D7012	Bedrock	2	<u>Peak Compressive Stress:</u> 6,308 to 9,861 psi <u>Young's Modulus:</u> 3,190,000 to 7,820,000 psi <u>Poisson's Ratio:</u> 0.07 to 0.28 <u>Bulk Density:</u> 169 to 170 pcf

Notes:

¹ WC = Moisture Content; psi = pounds per square in.; pcf = pounds per cubic foot

All laboratory test results are shown on the boring logs included in Appendix A and complete results are provided in Appendix B.

Geotechnical Evaluations and Design Recommendations

Geotechnical design recommendations, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Ninth Edition, 2020, referred to herein as AASHTO LRFD.
- MaineDOT Bridge Design Guide (BDG), August 2003, with Interim Revisions through June 2018, referred to herein as Bridge Design Guide.

Engineering calculations that support the design recommendations outlined in this report are provided for reference in Appendix D.

APPROACH EMBANKMENTS

As stated above, existing site grades along Route 170 will be raised by no more than approximately 1 ft to meet proposed finish grades. Because of the limited amount of raise in grade along Route 170 and because compressible soils (e.g., organic or marine silt/clay soils) were not encountered in the preliminary and final design phase borings drilled at the site, we anticipate that post-construction settlement of the new approach roadways will be negligible.

Embankment stability evaluations were not completed because proposed raises in grade along Route 170 are minimal (i.e., less than 1 ft). Because of this and because the proposed slope inclinations are less than the soils internal friction angle, it is our opinion that the proposed embankment side slopes will be stable when constructed in accordance with the requirements of the MaineDOT Standard Specifications.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Site class was determined in accordance with AASHTO LRFD Section 3.10.3.1 using Method B. In instances where SPT N-values were equal to 0 (i.e., weight of rod or weight of hammer), were greater than 100 blows per foot (bpf) or where bedrock was present, default values of 1 and 100 bpf were used, respectively.

Based on the nature and thickness of the overburden soils and depth to bedrock at the site, as determined from the borings, we recommend the site be considered "Site Class D." Spectral accelerations were determined based on the geographic site location and the recommended "Site Class D" designation using the United States Geological Survey (USGS) Seismic Design Web Service, which is based on the AASHTO recommended response spectra for a 7 percent probability of exceedance in 75 years (approximately 1,000-year return period). The recommended seismic design parameters are summarized below.

Design Parameter	Design Value
Site factor for short-period range of acceleration response spectrum, $F_a =$	1.600
Site factor for long-period range of acceleration response spectrum, $F_v =$	2.400
Site factor at zero-period on acceleration response spectrum, $F_{pga} =$	1.600
Horizontal response spectral acceleration coeff. at 0.2-s period on rock, $S_s (g) =$	0.150
Horizontal response spectral acceleration coeff. at 1.0-s period on rock, $S_1 (g) =$	0.046
Peak seismic ground acceleration coeff. on rock, PGA (g) =	0.068
Horizontal response spectral acceleration coeff. at 0.2-s period modified by $F_a, S_{DS} (g) =$	0.239
Horizontal response spectral acceleration coeff. at 1.0-s period modified by $F_v, S_{D1} (g) =$	0.110
Peak seismic ground acceleration coefficient modified by $F_{pga}, A_s (g) =$	0.108

In accordance with AASHTO LRFD Section 3.10.6, the site falls within Seismic Zone 1 based on the calculated value of S_{D1} (i.e., $S_{D1} < 0.15 =$ Seismic Zone 1 from AASHTO LRFD Table 3.10.6.1).

Based on our review of the soil conditions encountered in the borings and the results of the laboratory testing, it is our opinion that the potential for saturated granular soils present at the site to liquefy during the design earthquake event is low.

ABUTMENT NO. 1 FOUNDATION SUPPORT

As discussed above and as shown on the interpretive subsurface profile (Figure 2), the subsurface conditions along the proposed bridge alignment consist primarily of fill overlying glacial till and bedrock. We consider the glacial till soils and bedrock suitable to support the new abutment. Based on the depth to the suitable foundation bearing strata, scour considerations and the anticipated magnitude of the design loads, it is our opinion that driven piles are the most practicable and cost-effective foundation alternative.

Based on the information summarized above and our discussions with MaineDOT, it is our understanding that MaineDOT prefers the use of HP14x89 or HP14x117 steel H-piles to support Abutment No. 1. It is also our understanding that the maximum factored Strength Limit State (STR) pile load determined by MaineDOT is 360 kips.

Corrosion and Deterioration

The geotechnical engineering design of the proposed piles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. Based on our visual observation of the soil samples and our experience on similar projects with similar soil conditions, it is our opinion that the in-situ soils have low corrosive potential. Because of this, the net factored pile resistances recommended herein do not include a reduction in pile cross sectional area for steel degradation.

Static Axial Compressive Pile Resistance

As discussed previously, it is our opinion that steel H-piles will need to be driven to/into bedrock to provide the minimum required axial compressive resistance, primarily through end bearing resistance, considering the nature, consistency, and limited thickness of overburden soils (i.e., fill and glacial till).

Structural Resistance

In accordance with AASHTO LRFD Section 10.7.3, a distinction is made between piles driven to “soft” rock and those that are driven to “hard” rock. Based on our experience on similar projects with similar bedrock conditions, the sampling, recovery, and RQD measurements summarized herein, observation of the recovered rock core samples, and the results of laboratory testing, it is our opinion that the bedrock present at Abutment No. 1 be classified as “hard.” The geotechnical resistance of piles driven to end bearing in “hard” rock is controlled by the structural resistance as discussed in AASHTO LRFD Section 10.7.3.2.3. The structural resistance factor (Section 6.5.4.2) for axial resistance of piles in compression and subject to damage due to severe driving (“hard” rock bearing stratum) is 0.5. In addition, resistance factors for Service (SER) and Extreme Event (EE) Limit State loading are 1.0. Therefore, the nominal and factored structural resistances of HP14x89 and HP14x117 steel H-piles at the SER, STR and EE Limit States are summarized below.

Steel H-pile Section	Nominal Structural Resistance (kips)	Factored Structural Resistance (kips)		
		Service Limit State ($\phi=1.0$)	Strength Limit State ($\phi=0.5$)	Extreme Limit State ($\phi=1.0$)
HP14x89	1,305	1,305	653	1,305
HP14x117	1,720	1,720	860	1,720

Drivability Resistance

The design of the proposed abutment piles also included consideration of drivability resistance in accordance with AASHTO LRFD Section 10.7.8. The drivability evaluations were conducted using the computer program GRL WEAP 14 v. 14.1.6.0 (WEAP) developed by GRL Engineers, Inc. The drivability analyses were conducted to determine 1) if the piles could be impact driven through overburden soils to bedrock and 2) what nominal resistance could be achieved using hammer sizes typical of local pile driving contractors without damaging or overstressing the piles while keeping the penetration resistance below 15 blows per inch (bpi), which is the upper limit of penetration resistance typically allowed by the MaineDOT Standard Specifications.

The drivability evaluations were conducted assuming that a Delmag D16-32 single acting diesel hammer with a maximum rated energy equal to 40,130 ft-lbs. will be used to install the HP14x89 piles, and a Delmag D36-32 single acting diesel hammer with a maximum rated energy equal to 90,560 ft-lbs. will be used to install the HP14x117 piles. The Delmag D16-32 and D36-32 hammers have ratchet style fuel pumps with four settings, which are designed to limit the ram

stroke to 5.2 ft (fuel setting 1), 7.0 ft (fuel setting 2), 8.8 ft (fuel setting 3), and 11.4 ft (fuel setting 4; maximum). Drivability evaluations were completed based on the subsurface conditions present at the site. Fuel settings 4 and 3 were used for the HP14x89 and HP14x117 drivability analyses, respectively. The drivability resistances, maximum compressive stresses, and penetration resistances for the HP 14x89 and HP14x117 steel H-piles are summarized below.

Substructure	Steel H-pile Section	Pile Hammer	Pile Hammer Fuel Setting (Stroke, ft)	Drivability Resistance (kips)		Maximum Comp. Stress (ksi)	Penetration Resistance (bpi)
				Nominal	Factored ($\phi_{dyn} = 0.65$)		
Abutment No. 1	HP14x89	Delmag D16-32	4 (11.4)	700	455	44	15 to 16
	HP14x117	Delmag D36-32	3 (8.8)	825	536	46 to 47	8 to 9

The results of the evaluations show the following:

- Drivability resistance for HP14x89 steel H-piles is controlled by the maximum penetration resistance allowed by the MaineDOT Standard Specifications (15 bpi).
- Drivability resistance for HP14x117 steel H-piles is controlled by the maximum compressive driving stress. Based on our experience on similar projects with similar subsurface conditions and pile types and sizes, it is our opinion that WEAP tends to overpredict compressive stress as compared to what is measured in the field during dynamic testing. Because of this, the maximum compressive stress shown above (46 to 47 ksi) is considered acceptable.

Please note that the drivability results summarized above are based on an assumed pile-hammer size, fuel setting and the assumption that the piles penetrate through the entire soil overburden and end bear in/on bedrock. If the actual, factored axial compressive pile loads vary or if the actual pile-hammer system used to install the piles is different than the assumed system, additional evaluations will be required to determine the nominal and factored drivability resistances that can be achieved at a reasonable penetration resistance without overstressing the piles.

Summary of Axial Compressive Pile Resistances

The factored axial compressive pile resistance is controlled by the lesser of the STR Limit State drivability and structural resistances since the piles will be installed using impact hammers and the minimum nominal resistance will be confirmed in the field using dynamic testing. The factored resistances (structural and drivability) are summarized below.

Steel H-pile Section	Factored Structural Resistance (kips)	Factored Drivability Resistance (kips)	Governing Factored Resistance (kips)	Factored Strength Limit State Pile Load (kips)
HP14x89	653	455	455	360
HP14x117	861	536	536	

As summarized above, the factored STR Limit State axial compressive pile resistance is controlled by the drivability resistance. Therefore, we recommend that net factored STR Limit State axial compressive resistances equal to 455 kips and 536 kips for HP14x89 and HP14x117 steel H-piles, respectively, be used in designing Abutment No. 1.

MaineDOT standard practice is to dynamically test (during construction) a minimum of one pile per substructure and not less than 2 percent of the total number of piles anticipated to confirm that the minimum required nominal resistance has been achieved unless variable subsurface or sloping bedrock conditions are present. The top of bedrock surface encountered in borings drilled at Abutment No. 1 (BB-WMPS-101 and BB-WMPS-201) varied from approximately El. 293 to El. 294, which in our opinion does not reflect highly variable or sloping bedrock conditions. In addition, and based on our discussions with MaineDOT, it is our understanding that a total of four piles will be used to support Abutment No. 1. For these reasons, we recommend that a minimum of one production pile be dynamically tested (including 24-hr restrrike) during construction at Abutment No. 1.

We also recommend that minimum pile spacing, clearance and embedment (into the pile cap) meet the requirements of AASHTO LRFD Section 10.7.1.2. The pile tips should also be protected using cast steel driving shoes to prevent damage when driving through the very dense glacial till and to/into bedrock.

Lateral Pile Analysis

As stated above, Abutment No. 1 is being designed as an integral abutment and therefore, lateral pile analyses are required to investigate the effect that thermal displacements have on bending moments and combined stresses in the piles.

Haley & Aldrich completed lateral pile analyses in accordance with the requirements of Bridge Design Guide Section 5.4.2.4 and the recommendations included in the "Integral Abutment Bridge Design Guidelines" by the Vermont Agency of Transportation (VTrans) using the computer program LPile v. 2019.11.09 (LPile) developed by Ensoft, Inc using the soil profiles and soil and rock properties summarized below.

Stratum	Soil Model	Moisture Condition	Stratum Top Elevation	Stratum Thickness (ft)	Unit Weight (pcf)	Friction Angle (deg.)	Modulus (k, pci)
Fill	Sand (Reese)	Dry	318.6	5	120	31	124
Glacial Till 1	Sand (Reese)	Wet	313.6	5	125	34	76
Glacial Till 2	Sand (Reese)	Wet	308.6	15	130	48	184
Bedrock	NA	NA	293.6	NA	NA	NA	NA

Notes:

1. A groundwater level at El. 313.6 (top of glacial till 1 stratum) was used in the lateral pile analyses.

Steel H-pile head boundary conditions included a factored axial compressive pile load of 360 kips (STR Limit State), a pile head deflection corresponding to the thermal deflection of 0.3 in. (expansion and contraction cases) in weak axis bending, and a fixed-head condition (zero rotation condition). The ground surface levels used in the evaluations were El. 328.3 (expansion case; proposed finish grade behind the abutment) and El. 322.3 (contraction case; proposed top of shelf in front of the abutment). HP14x89 and HP14x117 steel H-pile properties were used for weak-axis bending with a 50 kips per square inch (ksi) yield strength. The results of the initial lateral pile analyses are summarized below.

Pile Section	Case	Max. Factored Axial Comp. Pile Load (kips)	Shear Force Resulting in 0.3 in. of Pile Head Displacement (kips)	Max. Moment at Pile Head (kip-ft)	Depth Below Pile Head to Fixity ¹ (ft)	Total Stress at Pile Head (ksi)	Unbraced Length	
							Top Segment (L1, ft)	Bottom Segment (L2, ft)
HP14x89	Exp.	360	98.0	214.6	16.4	72.0	2.9	9.6
	Cont.		46.0	143.4	18.4	52.8	3.8	10.7
HP14x117	Exp.		108.9	260.9	17.7	63.0	3.2	10.4
	Cont.		51.2	170.6	19.7	44.9	4.1	11.3

Notes:

- ¹ – Taken as the depth along the pile to the second point of zero lateral deflection as defined by MassDOT.

The initial lateral pile analysis results presented above were provided to MaineDOT for review and to determine if a plastic hinge analysis would be required. MaineDOT analyzed the initial lateral pile results and determined that a plastic hinge forms for the HP14x89 expansion case with a calculated plastic moment equal to 171.7 kip-ft. Since the resulting moment from the initial evaluation (HP14x89 expansion case) exceeded the calculated plastic hinge moment (214.6 kip-ft > 171.7 kip-ft) an additional lateral pile evaluation was performed using the maximum factored axial compressive pile load (360 kips), pile head displacement (0.3 in.) and the plastic moment (171.7 kips). The results of the additional lateral pile analysis are summarized below.

Pile Section	Case	Max. Factored Axial Comp. Pile Load (kips)	Shear Force Resulting in 0.3 in. of Pile Head Displacement (kips)	Max. Moment at Pile Head (kip-ft)	Depth Below Pile Head to Fixity ¹ (ft)	Total Stress at Pile Head (ksi)	Unbraced Length (ft)	
							Top Segment	Bottom Segment
HP14x89	Exp.	360	87.9	171.7	16.0	60.4	2.6	9.6

Notes:

¹ – Taken as the depth along the pile to the second point of zero lateral deflection as defined by MassDOT.

The additional lateral pile results presented above were provided to MaineDOT for review. It is our understanding that MaineDOT determined that the result of the additional analysis indicates that the HP14x89 steel H-piles oriented for weak-axis bending are suitable to resist the applied loads and displacements.

Elastic Pile Compression and Pile Settlement

Elastic shortening of the steel piles was evaluated based on a maximum Service Limit State pile load equal to 244 kips, which was provided by MaineDOT. Based on that, the recommended minimum delivered pile length (35 ft; see next section) and steel material properties, we estimate the maximum elastic pile compression for an individual HP14x89 steel H-pile to be 0.14 in. The elastic shortening of the piles is anticipated to occur primarily during construction, soon after the superstructure loads are applied.

Please note that the elastic shortening estimate summarized above (0.14 in.) does not include pile tip settlement, which is considered to be negligible for two primary reasons: 1) the relatively small load transmitted to the pile tip and 2) the piles will be bearing in/on bedrock.

Estimated Pile Tip Elevations and Pile Lengths

As discussed previously, the piles are expected to develop most of their axial compressive resistance through end bearing in/on bedrock. For estimating pile length bid quantities, we recommend the following:

Substructure	Bottom of Pile Cap Elev. (ft, NAVD 88)	Top of Pile Elev. (ft, NAVD 88)	Approx. Top of Bedrock Elev. ¹ (ft, NAVD 88)	Est. Pile Tip Elev. ² (ft, NAVD 88)	Approx. In-Place Pile Length (ft)	Recommended Minimum Delivered Pile Length ³ (ft)
Abut. No. 1	318.6	320.6	293.0 to 294.0	288.0	32.6	35

Notes:

¹ - Values shown based on elevation of top of bedrock encountered in boring BB-WPMS-201.

² - Values shown based on the assumption that the piles penetrate 5 ft into bedrock.

³ - Represents the approximate recommended minimum delivered pile length rounded up to the nearest 5-ft increment.

Please note that the recommended minimum delivered pile length summarized above does not account for an additional 5 ft of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and pile driving equipment. We recommend that the Contractor evaluate the subsurface conditions present at the site and estimate actual delivered pile lengths at each substructure location, which will be submitted to the Resident for review prior to ordering in accordance with the requirements of the Standard Specifications.

Based on the recommended pile lengths summarized above, we anticipate that minimal pile splicing may be needed. The actual number of pile splices may vary and is dependent on the actual order lengths proposed by the Contractor and the final pile tip elevations. Please note that Standard Specification Section 501.047 limits the total number of field splices to one for pile lengths ranging between 20 and 35 ft.

ABUTMENT NO. 2 BREASTWALL AND WINGWALL FOUNDATION SUPPORT

As shown on Figure 2 and as discussed herein, the subsurface conditions present at Abutment No. 2 generally consist of approximately 12 to 15 ft of fill soils overlying bedrock. Based on our discussions with you, it is our understanding that mass concrete spread footings bearing on intact bedrock is the preferred alternative to support Abutment No. 2 breastwall and wingwalls. The proposed bottom of footing elevation is El. 316.6, which is coincident with the top of bedrock surface encountered in boring BB-WPMS-103 (drilled on the northern side of the abutment). The top of bedrock surface encountered in boring BB-WPMS-102 (drilled on the southern side of the abutment) is El. 312.9. In areas where the top of bedrock surface is below the proposed bottom of footing after the removal of all soil and loose, weathered and/or fractured rock, we recommend that a concrete seal or sub-footing be constructed up to the proposed bottom of footing level.

Additional foundation recommendations are summarized below. Please note that the design recommendations presented below assume foundation subgrade preparation is completed in accordance with the guidance provided in the Construction Considerations section of this report as well as the requirements of the Contract Documents (CDs; drawings, Standard Specifications, and Special Provisions).

- **Bearing Resistance:**
 - For the SER Limit State, mass concrete footings should be designed such that footing contact pressures do not exceed 20 kips per square foot (ksf). At this pressure, it is estimated that elastic settlement of footings bearing on "fair to very good" bedrock will generally be less than ½ in. as stated in LRFD Section 10.6.2.4.4.
 - For the STR Limit State, footings should be designed for a factored bearing resistance of 47 ksf, using a resistance factor of 0.45.
 - For the EE Limit State, footings should be designed for a factored bearing resistance of 84 ksf, using a resistance factor of 0.8.
 - Please note that the maximum factored applied footing pressure at the STR Limit State calculated by MaineDOT is 6.5 ksf.

- Load Distribution and Eccentricity:
 - Application of permanent and transient loads is specified in AASHTO LRFD Section 11.5.6. We recommend the stress distribution at the base of the footings be assumed to be a triangular or trapezoidal distribution over the effective footing base as shown in AASHTO LRFD Figure 11.6.3.2-2.
 - The eccentricity of loading at the STR, based on factored loads, should not exceed 0.45 of the spread footing dimensions in either direction. The eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the base width and length.
- Sliding Resistance:
 - In accordance with AASHTO LRFD Tables C3.11.5.3-1 and 10.5.5.2.2-1, we recommend that sliding resistance of abutment breastwall and wingwall footings be calculated using the design parameters presented below.

Subgrade Saturation Condition During Construction	Coefficient of Friction ($\tan \delta$)	Interface Friction Angle (δ , degrees)	Strength Limit State Resistance Factor for Sliding (φ_t)	Service/Extreme Limit State Resistance Factor for Sliding (φ_t)
Prepared in-the-dry	0.7	35	0.8	1.0

- Lateral passive soil resistance in front of the footings, if present, should be neglected in accordance with requirements of the Bridge Design Guide. “Anchorage” of footings to bedrock (e.g., rock dowels) may be required to provide additional sliding resistance. If additional lateral load resistance is needed between the footings and bedrock, as determined by MaineDOT, we will provide additional geotechnical recommendations for rock dowels.

ABUTMENT BREASTWALL AND WINGWALL DESIGN

We recommend that the abutment breastwalls and wingwalls be designed for all relevant SER, STR, and EE Limit States and load combinations specified in AASHTO LRFD Sections 3.4.1 and 11.5.5. Additional recommendations are provided below.

- Drainage:
 - The abutment breastwall and wingwall designs should include a drainage system to intercept any water and direct it to a suitable discharge point. Drainage should be provided in accordance with the requirements of Bridge Design Guide Section 5.4.2.13.

- Lateral Earth Pressures:
 - Recommendations summarized below are based on the following:
 - Abutment breastwall and wingwalls are backfilled with a free-draining material (i.e., Soil Type 4, BDG Table 3-3; total unit weight = 125 pcf; internal angle of friction = 32 degrees; interface friction angle = 24 degrees).
 - The abutment breastwall and wingwall backwalls are vertical.
 - Adequate drainage is provided, as recommended herein and in accordance with the requirements of the Bridge Design Guide, to eliminate the potential for unbalanced hydrostatic pressures to develop.
 - Backfill surfaces behind the abutments that are inclined at 2.32 percent (Abutment No. 1) and -0.42 percent (Abutment No. 2).
 - Integral abutments (Abutment No. 1) should be designed using a static passive lateral earth pressure coefficient calculated using the “movement-dependent” methodology developed by MassDOT. For an abutment rotation equal to 0.0025 for the expansion case (provided by MaineDOT) and the soil type and properties presented above, we recommend using a static passive lateral earth pressure coefficient equal to 3.25.
 - Semi-integral abutments (Abutment No. 2) should be designed for Rankine active earth pressures over the rigid abutment height and a uniform distribution due to the height of soil behind the abutment backwall. We recommend using a Rankine static active earth pressure coefficient equal to 0.31.
 - Because the Abutment No. 2 backwall/end diaphragm is relatively independent from the cantilever abutment because of the detailing of the elastomeric bearing and soil plate, the length of the abutment heel has limited to no influence on the earth pressures engaged by the beam ends. Therefore, we recommend that the passive pressure be based on Coulomb Theory and a static passive earth pressure coefficient equal to 8.16.
 - In general, we recommend that walls be designed for a live load surcharge equivalent to the earthfill height summarized in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2. A uniform lateral load equal to the surcharge times the lateral earth pressure coefficient should also be applied to abutments and walls to account for the live load surcharge.

Frost Protection

The minimum depth of embedment/cover for Abutment No. 1 was evaluated in accordance with Section 5.2.1 of the Bridge Design Guide. Based on a design freezing index equal to 1,875 freezing degree-days, we recommend that the abutments bear a minimum of 6.5 ft below the lowest adjacent ground surface exposed to freezing.

The Abutment No. 2 breastwall and wingwall spread footings will bear directly on bedrock. Therefore, it is our opinion that the potential for frost-induced heave for foundations bearing on bedrock is low and therefore, a minimum footing embedment depth requirement is not considered necessary.

Construction Considerations

Based on the subsurface conditions encountered and our understanding of the proposed bridge replacement, we offer the following general geotechnical observations regarding the proposed bridge construction.

ABUTMENT NO. 2 TEMPORARY EARTH SUPPORT

Based on the proposed elevation of the bottom of Abutment No. 2, a temporary earth support system may be needed to construct the substructure (if conventional temporary sloping is not feasible and/or if the water level in the stream is higher than the top of bedrock surface). Based on the subsurface soil and groundwater conditions at the site, we anticipate that the most cost-effective excavation support system for construction of Abutment No. 2 is steel sheet piles.

Design of temporary works, including temporary earth support system(s) are the responsibility of the Contractor and should be completed by a Licensed Professional Engineer in the State of Maine hired by the Contractor. We recommend that temporary earth support system(s) be designed to support all appropriate combinations of earth, water, and surcharge loads (from traffic, construction equipment, material stockpiles and other sources) imposed on the system(s) during all phases of the construction period. The Contractor is responsible for choosing an appropriate factor of safety for the earth support system(s). The Contractor's design should also consider the means and methods and construction sequencing proposed. We recommend that design calculations and shop drawings be prepared by the Contractor, stamped by a Licensed Professional Engineer in the State of Maine, and submitted to the Resident for review prior to construction in accordance with the requirements of the Standard Specifications.

ABUTMENT NO. 1 DYNAMIC PILE LOAD TESTING PROGRAM

The Contractor will be required to confirm the nominal axial compressive pile resistance in the field using dynamic testing methods. The piles should be driven to a minimum nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 in accordance with AASHTO LRFD Table 10.5.5.2.3-1 (360 kips / 0.65 = 554 kips). We recommend that the Contractor perform a minimum of one dynamic pile load test with 24-hour (minimum) restrike to evaluate hammer system efficiencies, driving stresses in the pile, and the nominal resistance of the piles. We also recommend that the dynamic testing be completed prior to production pile driving and that CAPWAP analysis be performed on the test pile. The CAPWAP results will be used to finalize driving criteria for the production piles to assure that the piles achieve the minimum required resistance without being overstressed.

ABUTMENT NO. 2 BEDROCK SUBGRADE PREPARATION

The nature, slope, and degree of fracturing of the bedrock bearing surface will not be evident until the foundation excavation for the abutment breast and wingwalls is completed. Construction activities should not be permitted to disturb the bedrock mass. We recommend that the bedrock surface be

cleared of all loose, fractured and/or weathered/decomposed bedrock and soil (i.e., unsuitable material). If excavation of unsuitable material extends below the bearing level of the footings, we recommend that Class S concrete be placed from the bottom of excavation up to the proposed footing bearing level, after the surface has been inspected in the field by the Resident and/or project Geotechnical Engineer, as discussed below and in accordance with the requirements of the Standard Specifications.

- Foundation bearing areas should be level. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level. Smooth bedrock, if present, should be roughened or serrated prior to placing concrete to enhance sliding resistance.
- In-the-dry or underwater excavation of steeply sloping and/or loose, fractured bedrock may be done using conventional excavation methods. Based on the proposed bearing level of the abutment breast and wingwall footings and the top of bedrock surface encountered in the borings we do not anticipate the need for bedrock removal using drilling and blasting techniques.
- Depending on the actual water level in the stream during foundation construction, portions of the abutment breast and wingwall excavations may be submerged. In accordance with MaineDOT Standard Specifications Section 511, if the foundation excavations are submerged, the Contractor shall prepare and submit a written procedure for cleaning and inspection of the bedrock subgrade to the Resident and project Geotechnical Engineer for review. We recommend that the final bedrock surface be approved by the Resident and/or project Geotechnical Engineer prior to placement of the concrete seals or footing reinforcing steel and concrete.
- It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps so that subgrade preparation and foundation construction is completed in-the-dry. We recommend that dewatering effluent be managed in accordance with all local, state and/or federal regulations.

SUBMITTAL REVIEWS

The Contract Documents (CDs; drawings, Standard Specifications and Special Provisions) should be written so that the requirements of the CDs are consistent with the design intent of the geotechnical recommendations outlined herein. Standard Specifications require that the Contractor and the Contractor's engineer perform analyses and submit results to MaineDOT for review. We recommend that Haley & Aldrich be allowed to review the geotechnical-related submittals to ensure that the Contractor's analyses/submittals are in accordance with the intent of the design as summarized herein. This will enable us to observe compliance with the design concepts, assumptions, and specifications, and to facilitate design changes if subsurface conditions differ from those anticipated prior to the start of construction.

CONSTRUCTION MONITORING

The geotechnical design and earthwork recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation construction activities is required to enable the geotechnical engineer to confirm that procedures and techniques used by the Contractor during construction are appropriate and will not impact the design of the bridge. Therefore, we recommend that an individual representing MaineDOT, qualified by geotechnical training and experience be present at the site to provide monitoring during the foundation construction activities listed below.

- Dynamic testing of test piles at Abutment No. 1.
- Installation of production piles at Abutment No. 1.
- Inspection of bedrock subgrade at Abutment No. 2.

Limitations

This report is prepared for the exclusive use of MaineDOT relative to the Mattagodus Stream Bridge Replacement project in Webster Plantation, Maine. There are no intended beneficiaries other than MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from MaineDOT and from Haley & Aldrich indicating that the report is adequate for such other use. Use of this report by such other person or entity without the written authorization of MaineDOT and Haley & Aldrich shall be at such other person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

The analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report.

We understand that this report will be included as a reference document in the package that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the Contract Documents (CDs; plans, Standard Specifications, Special Provisions) and that the information contained in the CDs takes precedence over the information provided in this report.

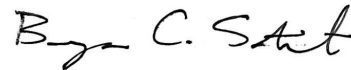
Closure

We appreciate the opportunity to provide geotechnical engineering services on this project. Please do not hesitate to call if you have any questions or comments.

Sincerely yours,
HALEY & ALDRICH, INC.



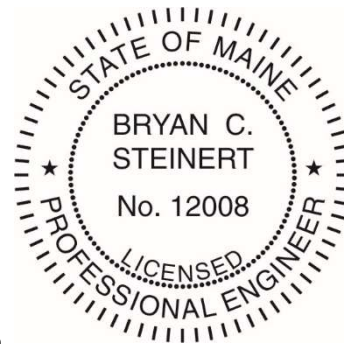
Eric Hunstein, E.I.
Geotechnical Engineer



Bryan C. Steinert, P.E.
Senior Project Manager



Wayne A. Chadbourne, P.E.
Principal | Lead Quality Control Engineer



Enclosures:

- Table I – Subsurface Exploration Location Data
- Table II – Subsurface Exploration Subsurface Data
- Table III – Subsurface Exploration Bedrock Core Data
- Figure 1 – Project Locus
- Figure 2 – Boring Location Plan and Interpretive Subsurface Profile
- Appendix A – Boring Logs and Rock Core Photographs
- Appendix B – Laboratory Test Results
- Appendix C – Historic Bridge Drawings
- Appendix D – Geotechnical Calculations

\\haleyaldrich.com\share\CF\Projects\0205793\Deliverables\Geotechnical Design Report\2023-0331-HAI-22266 Mattagodus Bridge No 5116 GDR-F.docx

TABLE I

Subsurface Exploration Location Data
 Mattagodus Stream Bridge over Mattagodus Stream No. 5116
 MaineDOT WIN 022266.00
 Route 170, Webster Plantation, Maine

Haley & Aldrich, Inc. File No.: 0205793-000

Test Boring No. ¹	Ground Surface Elevation ³ (ft)	Station ⁴	Offset Distance and Direction ^{4,5} (ft)	Horizontal Coordinates (ft) ²	
				Northing (Y)	Easting (X)
BB-WPMS-101	327.8	4+57.2	6.5' LT	616,116	2,223,699
BB-WPMS-102	328.4	5+40.2	7.2' RT	616,098	2,223,782
BB-WPMS-103	328.5	5+38.1	6.4' LT	616,112	2,223,780
BB-WPMS-201	328.2	4+63.0	5.2' RT	616,104	2,223,705

Notes:

- ¹ As-drilled test boring locations are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.
- ² As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet and reference NAD83, Maine 2000 East Zone coordinate system.
- ³ Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
- ⁴ Station and offset information shown is relative to the proposed baseline shown on Figure 2 and was determined by MaineDOT.
- ⁵ LT = offset distance left of centerline looking upstation; RT = offset distance right of centerline looking upstation; ft = feet.

	Individual	Date
Prepared By:	SSM	8/1/2022
Checked By:	BCS	1/24/2023
Reviewed By:	WAC	3/20/2023

TABLE II

Subsurface Exploration Subsurface Data
 Mattagodus Stream Bridge over Mattagodus Stream No. 5116
 MaineDOT WIN 022266.00
 Route 170, Webster Plantation, Maine

Haley & Aldrich, Inc. File No.: 0205793-000

Test Boring No. ¹	Ground Surface Elevation ² (ft)	Stratigraphic Data ^{2,3}									Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration ²
		Bituminous Concrete Thickness (ft)	Fill			Glacial Till			Bedrock			
			Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)		
BB-WPMS-101	327.8	0.4	0.4	327.4	13.6	14.0	313.8	19.4	33.4	294.4	43.4	284.4
BB-WPMS-102	328.4	0.4	0.4	328.0	15.1	NE	NE	NE	15.5	312.9	25.5	302.9
BB-WPMS-103	328.5	0.4	0.4	328.1	11.5	NE	NE	NE	11.9	316.6	20.9	307.6
BB-WPMS-201	328.2	0.6	0.6	327.6	14.4	15.0	313.2	20.2	35.2	293.0	46.8	281.4

Notes:

¹ As-drilled test boring locations are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.

² Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

³ "NE" indicates stratum was not encountered in test boring.

	Individual	Date
Prepared By:	SSM	8/1/2022
Checked By:	BCS	1/24/2023
Reviewed By:	WAC	3/20/2023

TABLE III

Subsurface Exploration Bedrock Core Data
 Mattagodus Stream Bridge over Mattagodus Stream No. 5116
 MaineDOT WIN 022266.00
 Route 170, Webster Plantation, Maine

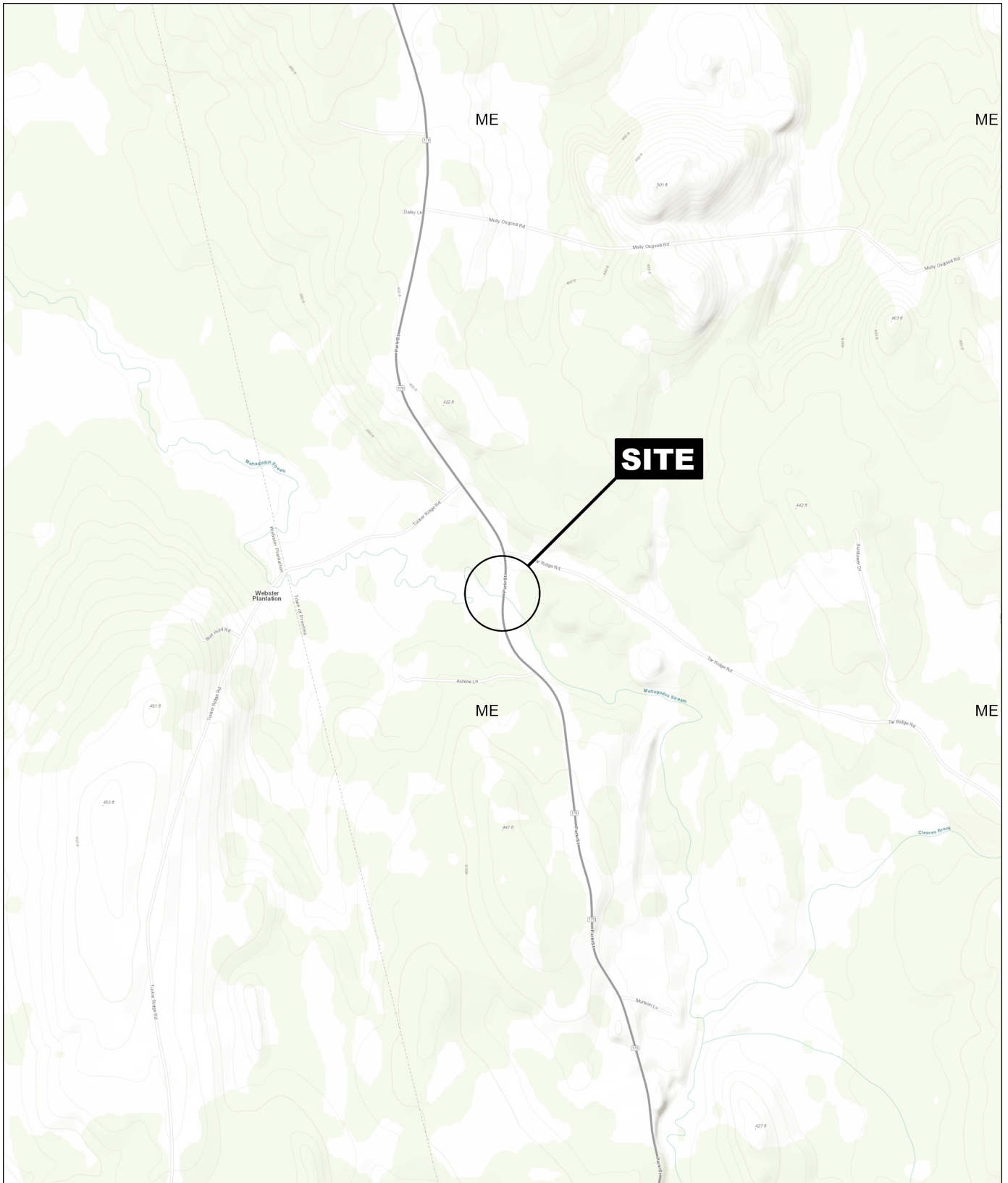
Haley & Aldrich, Inc. File No.: 0205793-000

Test Boring No. ¹	Ground Surface Elevation ² (ft)	Bedrock Core Diameter (in.)	Run				Total Core Recovery ³		Rock Quality Designation ^{4,5}			Physical Rock Parameters		Lithologic, Rock Mass and Discontinuity Description ⁶	
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (in.)	%	Length (in.)	%	Designation	Weathering		Estimated Field Strength
				Top	Bottom	Midpoint									
BB-WPMS-101	327.8	NQ (1.875")	R1	33.4	38.4	35.9	5.0	24	40%	0	0%	very poor	moderate	Brown to grey, aphanitic to fine grained SHALE. Some portions fragmented from extensive iron and manganese oxidation. R2 highly fractured throughout core stem. Discernable joints dipping at steep to vertical angles, very closely spaced, open. Secondary joints dipping at low to moderate angles.	
			R2	38.4	43.4	40.9	5.0	44	73%	0	0%	very poor	moderate		
BB-WPMS-102	328.4	NQ (1.875")	R1	15.5	20.5	18.0	5.0	60	100%	60	100%	excellent	fresh	Black to grey, fine grained SHALE with minor calcite veins. Discernable joints dipping at moderate to steep angles, very close to widely spaced, tight to wide.	
			R2	20.5	25.5	23.0	5.0	60	100%	56	93%	excellent	fresh		
BB-WPMS-103	328.5	NQ (1.875")	R1	11.9	16.4	14.2	4.5	51	94%	21	42%	poor	slight	Grey to black, aphanitic SHALE with dark brown manganese oxide staining. Discernable joints dipping at low to vertical angles, very close to moderately spaced, open. Near vertical foliation and several quartz veins observed.	
			R2	16.4	20.9	18.7	4.5	44	81%	22	50%	poor	fresh to slight		
BB-WPMS-201	328.2	NQ (1.875")	R1	35.6	38.8	37.2	3.2	12	31%	0	0%	very poor	severe	Brown-grey-white, aphanitic to fine grained SHALE with quartz intrusions. Discernable joints dipping at low to vertical angles, very close to closely spaced, tight to open. Portions of the core stem are highly fractured. Very thin, high angle bedding.	
			R2	38.8	43.8	41.3	5.0	43	72%	24	55%	fair	slight to moderate		
			R3	43.8	46.8	45.3	3.0	36	100%	8	22%	very poor	slight to moderate		

Notes:

- ¹ As-drilled test boring locations are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.
- ² Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
- ³ TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the core run.
- ⁴ RQD = rock quality designation. RQD is the total length of intact, full-diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of at least 4 in.) measured along the core axis. The percent RQD is the total length of RQD measured divided by the length of the core run. Note that vertical discontinuities are not included in determination of RQD.
- ⁵ Designation based on RQD in accordance with MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms" Field Identification Information, dated January 2020.
- ⁶ Refer to the test boring logs and rock core photographs in Appendix A for additional information.

	Individual	Date
Prepared By:	SSM	8/1/2022
Checked By:	BCS	1/24/2023
Reviewed By:	WAC	3/20/2023



2025795-J0-LOCUS_HALEYALDRICH.RLG



SITE COORDINATES: 45°29'30"N, 68°7'10"W



MAP SOURCE: USGS

**HALEY
ALDRICH**

MATTAGODUS STREAM BRIDGE
OVER MATTAGODUS STREAM NO. 5116
MAINEDOT WIN 022266.00
ROUTE 170, WEBSTER PLANTATION, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 2,000 FEET
MARCH 2023

FIGURE 1

Date: 3/16/2023

Username: terry.white

Division: GEOTECH

Filename: ... \GEOTECH\STA\005_BLP&ISPl.dgn



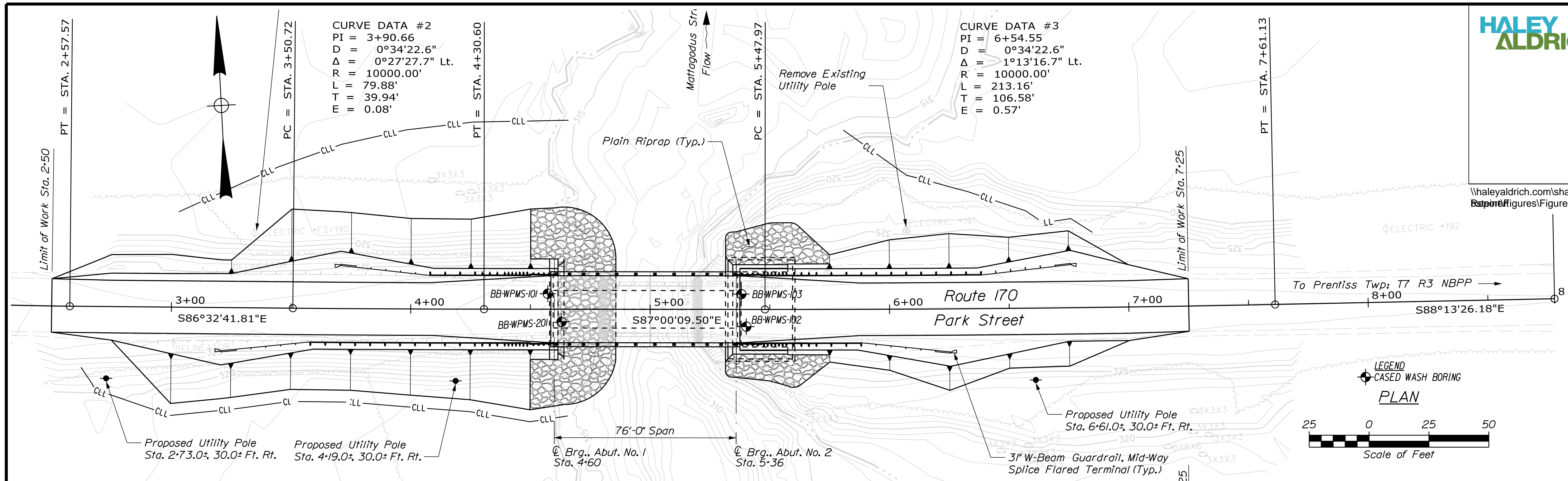
MATTAGODUS STREAM BRIDGE OVER
MATTAGODUS STREAM NO. 5116
MAINEDOT WIN 022266.00
ROUTE 170, WEBSTER PLANTATION, MAINE

BORING LOCATION PLAN AND INTERPRETIVE SUBSURFACE PROFILE

SCALE: AS SHOWN
MARCH 2023

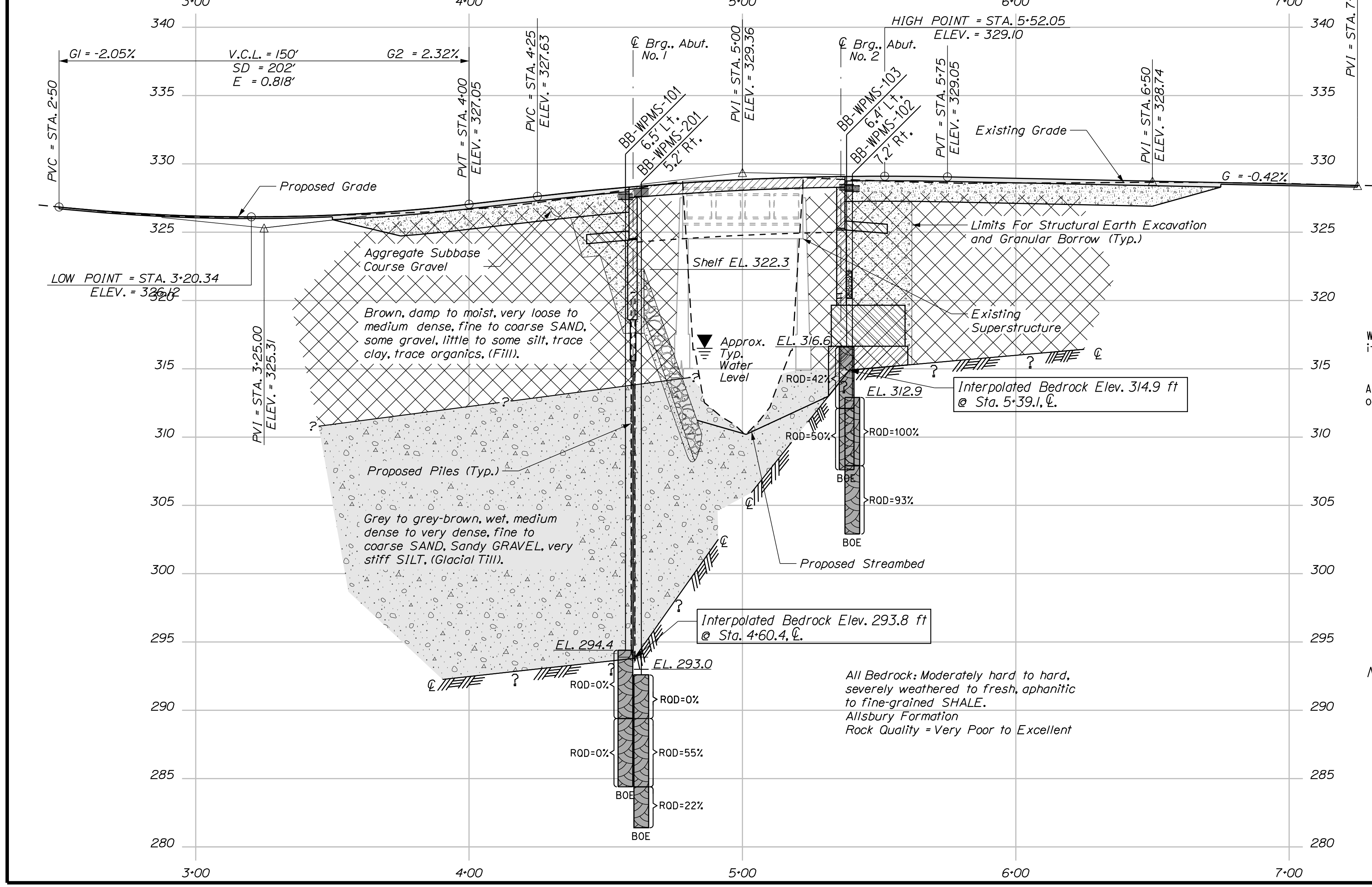
FIGURE 2

\\haleyaldrich.com\share\CF\Projects\0205793\Deliverables\Geotechnical Design
Borings\Figures\Figure 2-F.pdf



NOTES:

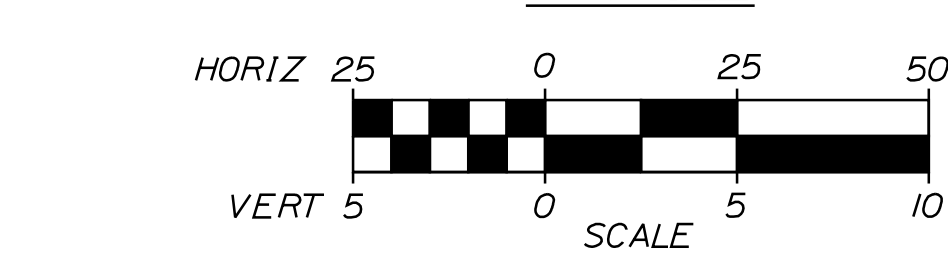
1. THIS PLAN WAS PREPARED BY MAINEDOT BASED ON INPUT AND REVIEW COMMENTS PROVIDED BY HALEY & ALDRICH, INC.
2. THE PLAN LOCATION OF TEST BORINGS SHOWN WERE DETERMINED BY MAINEDOT UPON THE COMPLETION OF DRILLING USING GPS SURVEY EQUIPMENT.
3. TEST BORINGS WERE DRILLED AND MONITORED BY MAINEDOT IN JUNE 2018 (-100 SERIES TEST BORINGS) AND JULY 2022 (-200 SERIES TEST BORINGS).
4. REFER TO APPENDIX A FOR TEST BORING LOGS AND ROCK CORE PHOTOGRAPHS AND APPENDIX B FOR LABORATORY TEST RESULTS.



LEGEND

- Weathered Bedrock, if applicable
- Approximate Top of Bedrock
- Boring No. (offset, if shown)
- Pavement Thickness, if applicable
- ROD: Rock Quality Designation of Bedrock Core Sample
- BOE: Bottom of Exploration
- No Refusal (NR)
- Refusal (R)

PROFILE



Note: 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.

DEPARTMENT		BRIDGE NO. 5116	
STATE OF MAINE		PROFESSIONAL ENGINEER	
BRYAN C. STERNER		No. 12008	
SIGNATURE		P.E. NUMBER	
DATE		DATE	
3/1/2023		3/1/2023	
PROJ. MANAGER	MARK PARLIN	BY	Bryan C. Sterner
DESIGN-DETAILED	N. SHERWOOD	DATE	FEB 2023
CHECKED-REVIEWED	B. STERNER	DATE	FEB 2023
DESIGN-DETAILED	N. SHERWOOD	DATE	FEB 2023
DESIGN-DETAILED	E. HUNSTEIN	DATE	FEB 2023
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			
MATTAGODUS STREAM BRIDGE			
MATTAGODUS STREAM			
WEBSTER PLT PENOBSCOT COUNTY			
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE			

APPENDIX A
Boring Logs and
Rock Core Photographs

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER 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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	
		(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.	
	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	
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):				Desired Rock 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				Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock quality (very poor, poor, etc.) ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12 Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))	
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				Sample Container Labeling Requirements:	
				WIN	Blow Counts
				Bridge Name / Town	Sample Recovery
				Boring Number	Date
				Sample Number	Personnel Initials
				Sample Depth	

MaineDOT Field Vane Undrained Shear Strength Testing Procedures Summary

- There are 5 (five) rectangular vane sizes available (all Geonor vanes):
 - 65 x 130 mm,
 - 55 x 110 mm,
 - 16 x 32 mm,
 - 20 x 40 mm, and
 - 25.4 x 50.8 mm
- Maintain a full water column in the casing as the drill tools are extracted before a vane shear test.
- Turn vanes using a vane rotation rate of about **1 minute per quadrant (90 degrees)**.
- **In soft clay-silt soils:** Use 65 x 130 mm or 55 x 110 mm vanes
 - Take **two** sets of vanes (peak/remolded) at depths of 1 and 2 ft below the casing
 - Then sample with the Standard Penetration Test (SPT) spoon
- **If a Shelby tube is taken in soft clay-silt soils:** Use 65 x 130 mm or 55 x 110 mm vanes
 - **Take tube first** (maintain full water column in the casing before and after tube sampling)
 - Take **two** sets of vanes (peak/remolded) at a depths of 1 and 2 ft below tube interval
- **In stiff clay-silt soils:** Use 16 x 32 mm, 20 x 40 mm or 25.4 x 50.8 mm vanes
 - Take **one** set of vanes (peak/remolded) at a depth of 4 inches below the casing
 - Then sample with the SPT spoon
- **If a Shelby tube is taken in stiff clay-silt soils:** Use 16 x 32 mm, 20 x 40 mm or 25.4 x 50.8 mm vanes
 - Take **one** set of vanes (peak/remolded) at a depth of 4 inches below the casing
 - **Then take tube** (maintain full water column in the casing throughout procedure)
- The raw field vane shear test torque readings is follows:
 - ft-lbs for 65 x 130 mm and 55 x 110 mm vanes
 - in-lbs for 16 x 32 mm, 20 x 40 mm, 25.4 x 50.8 mm vanes
- The maximum torque for each vane will be as follows:

Vane Dimensions (mm)	Maximum Torque (ft-lbs)	Maximum Undrained Shear Strength (psf)
65 x 130	44.3	1217
55 x 110	44.3	1978
Vane Dimensions (mm)	Maximum Torque (in-lbs)	Maximum Undrained Shear Strength (psf)
16 x 32	26.6	4181
20 x 40	26.6	2141
25.4 x 50.8	26.6	1045

- ✓ The drilling inspector will be responsible for determining which vane should be used based on site conditions. Call the project geotechnical engineer or geologist if in doubt about which vane size to use.

Driller: MaineDOT	Elevation (ft.): 327.8	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/21/2018; 07:30-12:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 4+57.2, 6.5 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 19.5 ft bgs.

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	327.4	5" BITUMINOUS CONCRETE.		
										Brown, damp, Gravelly fine to coarse SAND, some silt, (Fill). Note: Sample collected from SSA cuttings.		
									324.8			
5	1D	24/13	5.00 - 7.00	5/7/6/6	13	20				Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#296512 A-1-b, SM WC=8.0%	
10	2D	24/7	10.00 - 12.00	1/1/1/1	2	3	4			Brown, moist, very loose, fine to coarse SAND, some gravel, some silt, trace clay, (Fill).	G#296513 A-2-4, SM WC=16.7%	
15	3D	24/14	15.00 - 17.00	4/5/4/9	9	14	21			Grey, wet, medium dense, Silty fine to coarse SAND, little gravel, (Glacial Till).		
20	4D	24/12	20.00 - 22.00	16/24/36/18	60	93	33			Grey, wet, very dense, Gravelly, fine to coarse SAND, little silt, (Glacial Till).		
										Roller Coned ahead to 25.0 ft bgs.		
25												

Remarks:
 N 616116.4763 E 2223699.4121 - Maine 2000 East Zone
 Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Mattagodus Stream Bridge #5116 carries Route 170 over Mattagodus Stream	Boring No.: BB-WPMS-101
	Location: Webster Plantation, Maine	WIN: 22266.00

Driller: MaineDOT	Elevation (ft.): 327.8	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/21/2018; 07:30-12:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 4+57.2, 6.5 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 19.5 ft bgs.

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>
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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25	5D	24/5	25.00 - 27.00	11/11/18/22	29	45	33	298.8	[Graphic Log]	Similar to above, except dense, (Glacial Till).	
							121				
							117				
							108				
							96	294.4	[Graphic Log]	Grey-brown, wet, very dense, fine to coarse SAND, some gravel, some silt, (Glacial Till). Roller Coned ahead to 33.4 ft bgs.	
30	6D	24/12	30.00 - 32.00	22/26/25/30	51	79	52				
							134				
							210	294.4	[Graphic Log]	a100 blows for 0.4 ft. Top of Bedrock at Elev. 294.4 ft.	
	R1	60/24	33.40 - 38.40	RQD = 0%			a100 NQ-2				
35								284.4	[Graphic Log]	R1: Bedrock: Brown to grey, fine-grained, SHALE, moderately hard, moderately weathered fragmented from extensive iron and manganese oxidation. Joints indiscernible. Allsbury Formation Rock Quality = Very Poor 40% Recovery R1: Core times (min:sec) 33.4-34.4 ft (2:16) 34.4-35.4 ft (2:53) 35.4-36.4 ft (2:05) 36.4-37.4 ft (3:26) 37.4-38.4 ft (4:42)	
								284.4	[Graphic Log]	R2: Bedrock: Brown to grey, aphanitic SHALE, moderately hard, moderately weathered, highly fractured throughout core stem. Discernible joints steeply dipping to vertical, very close, open. Secondary low angle to moderately dipping joints. Allsbury Formation Rock Quality = Very Poor 73% Recovery R2: Core times (min:sec) 38.4-39.4 ft (2:31) 39.4-40.4 ft (2:34) 40.4-41.4 ft (2:49) 41.4-42.4 ft (3:41) 42.4-43.4 ft (3:46)	
	R2	60/44	38.40 - 43.40	RQD = 0%							
40								284.4	[Graphic Log]	Bottom of Exploration at 43.4 feet below ground surface.	
45								284.4	[Graphic Log]		
50								284.4	[Graphic Log]		

Remarks:
 N 616116.4763 E 2223699.4121 - Maine 2000 East Zone
 Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

Driller: MaineDOT	Elevation (ft.): 328.4	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/26/2018; 07:30-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 5+40.2, 7.2 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 8.5 ft bgs.

Hammer Efficiency Factor: 0.928 **Hammer Type:** Automatic Hydraulic Rope & Cathead

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

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows					
0								SSA	328.0	5" BITUMINOUS CONCRETE.		
										Brown, damp, Gravelly fine to coarse SAND, some silt, (Fill). Note: Sample collected from SSA cuttings.		
5	1D	24/9	5.00 - 7.00	3/2/2/2	4	6				Brown, damp, loose, fine to coarse SAND, some silt, some gravel, (Fill).	G#296514 A-1-b, SM WC=7.0%	
10	2D	24/8	10.00 - 12.00	2/2/1/1	3	5	10			Similar to above, except little silt, (Fill).	G#296515 A-1-b, SM WC=8.8%	
							6					
							7					
							57					
							62					
15	R1	60/60	15.50 - 20.50	RQD = 100%				a50 NQ-2	312.9	a50 blows for 0.5 ft.		
										Top of Bedrock at Elev. 312.9 ft.		
										R1: Bedrock: Black to grey, fine- grained, SHALE with minor calcite veins, hard, fresh, steep (80 degrees) foliation, single joint at approximately 19.5 ft, moderately dipping, wide, tight.		
										Allsbury Formation		
										Rock Quality = Excellent		
										100% Recovery		
										R1: Core Times (min:sec)		
										15.5-16.5 ft (3:50)		
										16.5-17.5 ft (3:38)		
										17.5-18.5 ft (3:30)		
										18.5-19.5 ft (3:42)		
										19.5-20.5 ft (3:45)		
										R2: Bedrock: Similar to R1, except joints very close to wide.		
										Allsbury Formation		
										Rock Quality = Excellent		
										100% Recovery		
										R2: Core Times (min:sec)		
										20.5-21.5 ft (3:15)	$q_p=1,420$ ksf (19.7-20.05 ft)	
25											$q_p=908$ ksf (23.6-23.96 ft)	

Remarks:
 N 616098.4769 E 2223781.5284 - Maine 200 East Zone
 Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Mattagodus Stream Bridge #5116 carries Route 170 over Mattagodus Stream Location: Webster Plantation, Maine	Boring No.: BB-WPMS-102 WIN: 22266.00
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Driller: MaineDOT	Elevation (ft.): 328.4	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/26/2018; 07:30-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 5+40.2, 7.2 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 8.5 ft bgs.

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)				
25							▽	302.9		21.5-22.5 ft (3:21) 22.5-23.5 ft (3:08) 23.5-24.5 ft (3:15) 24.5-25.5 ft (3:25)		
30										Bottom of Exploration at 25.5 feet below ground surface.		
35												
40												
45												
50												

Remarks:
 N 616098.4769 E 2223781.5284 - Maine 200 East Zone
 Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Mattagodus Stream Bridge #5116 carries Route 170 over Mattagodus Stream Location: Webster Plantation, Maine	Boring No.: BB-WPMS-103 WIN: 22266.00
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Driller: MaineDOT	Elevation (ft.): 328.5	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/26/2018; 12:00-14:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 5+38.1, 6.4 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 6.5 ft bgs.

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>
<small>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</small>	<small>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</small>
	<small>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 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected</small>
	<small>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</small>

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	328.1	5" BITUMINOUS CONCRETE. Note: Soils encountered between 0.4 and 11.4 ft bgs, similar to those encountered in Test Boring BB-WPMS-102, (Fill).		
0.4												
5										485 blows for 0.9 ft.		
10							21					
	R1	54/51	11.90 - 16.40	RQD = 42%				a85 NQ-2	316.6	Top of Bedrock Elev. 316.6 ft. R1: Bedrock: Grey to black, aphanitic SHALE, moderately hard, slightly weathered, with dark brown manganese oxide staining, joints dipping at moderate to vertical angles, very close to moderately close, open, near vertical foliation, several quartz veins. Allsbury Formation Quality = Poor 94% Recovery Core Times not recorded. R2: Bedrock: Similar to R1, except fresh to slightly weathered, joints dipping at low and high angles, close to moderately close. Allsbury Formation Rock Quality = Poor 81% Recovery Core Times not recorded.		
15												
	R2	54/44	16.40 - 20.90	RQD = 50%						Bottom of Exploration at 20.9 feet below ground surface.		
20									307.6			
25												

Remarks:
N 616112.1661 E 2223780.1437 - Maine 2000 East Zone
Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Mattagodus Stream Bridge #5116 carries Route 170 over Mattagodus Stream Location: Webster Plantation, Maine	Boring No.: BB-WPMS-201 WIN: 22266.00
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Driller: MaineDOT	Elevation (ft.): 328.2	Auger ID/OD: 5" Solid Stem
Operator: Travis/Bruce	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: N. Pukay	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 7/12/2022; 08:00-16:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 4+63, 5.2 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 15.1 ft bgs, after drilling

Hammer Efficiency Factor: 0.974	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	327.6	7" BITUMINOUS CONCRETE.	-0.6	
5	1D	24/22	5.00 - 7.00	6/6/6/6	12	19				Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).		
10	2D	24/21	10.00 - 12.00	6/3/3/6	6	10	14			Brown, moist, loose, fine to coarse SAND, some gravel, some silt, trace organics, wood in tip of spoon, (Fill). Note: Casing refusal at 12.5 ft bgs on wood. Roller coned ahead from 12.5-15.0 ft bgs. Wood chips observed in wash until 12.7 ft bgs. Wood from 10.8-12.7 ft bgs.		
15	3D	24/10	15.00 - 17.00	5/7/8/8	15	24	28		313.2	Grey, wet, very stiff, SILT, some fine to coarse sand, little gravel, well bonded, (Glacial Till).	-15.0	
20	4D	17/9	20.00 - 21.42	15//17/50(5")	---		38		308.6	Brown-grey, wet, very dense, Gravelly fine to coarse SAND, little silt, (Glacial Till). Note: Roller coned ahead from 20.0-23.7, then advanced casing to 24.0, then roller coned ahead to 25.0 ft, cobble at 23.7 ft bgs.	-19.6	
25							67					

Remarks:
 N 616104.4600 E 2223704.5600 - Maine 2000 East Zone
 Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Mattagodus Stream Bridge #5116 carries Route 170 over Mattagodus Stream Location: Webster Plantation, Maine	Boring No.: BB-WPMS-201 WIN: 22266.00
--	--	--

Driller: MaineDOT	Elevation (ft.): 328.2	Auger ID/OD: 5" Solid Stem
Operator: Travis/Bruce	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: N. Pukay	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 7/12/2022; 08:00-16:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 4+63, 5.2 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 15.1 ft bgs, after drilling

Hammer Efficiency Factor: 0.974	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>
--	--

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
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25	5D	24/13	25.00 - 27.00	20/18/19/18	37	60	89		Brown, grey, wet, very dense, fine to coarse Sandy GRAVEL, little silt, (Glacial Till). Note: Roller coned ahead from 30.0-35.0 ft bgs. Note: Cobbles at 31.8 ft bgs.		
							106				
							84				
							130				
							198				
30	MD	6/0	30.00 - 30.50	91(6")	---		116		Note: Cobbles at 31.8 ft bgs.		
							405				
							55				
							97				
							87				
35	6D R1	5/2 38.4/12	35.20 - 35.62 35.60 - 38.80	100(5") RQD = 0%	---		138 NQ-2		Top of Bedrock at Elev. 293.0 ft. Brown, wet, very dense, GRAVEL (Rock Chips), little sand, little silt. R1: Bedrock: Brown-grey, fine-grained to aphanitic SHALE, severely weathered. Few discernible low angle and vertical joints, very close, open, highly fractured throughout, dirty brown wash water throughout run. Allsbury Formation Rock Quality = Very Poor 31% Recovery R1: Core Times (min:sec) 35.6-36.6 ft (3:02) 36.6-37.6 ft (2:44) 37.6-38.6 ft (3:25) 38.6-38.8 ft (3:02) Core Blocked R2: Bedrock: Brown-grey-white, aphanitic SHALE, hard, slightly to moderately weathered, joints low angle to moderately dipping, very close to close, tight to open. Quartz intrusion approximately 38.8-41.0 ft bgs. 0.1 ft drop from 39.7-39.8 ft bgs. Wash light grey from 40.4-41.3 ft bgs., otherwise brown. Allsbury Formation Rock Quality = Fair 72% Recovery R2: Core Times (min:sec) 38.8-39.8 ft (2:57) 39.8-40.8 ft (4:09) 40.8-41.8 ft (2:55) 41.8-42.8 ft (2:20) 42.8-43.8 ft (3:00)		
							293.0				
							292.6				
							289.4				
							284.4				
40	R2	60/43	38.80 - 43.80	RQD = 55%					R3: Bedrock: Brown-grey, aphanitic SHALE, moderately hard,		
							281.4				
45	R3	36/36	43.80 - 46.80	RQD = 22%							
50											

Remarks:
 N 616104.4600 E 2223704.5600 - Maine 2000 East Zone
 Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Mattagodus Stream Bridge #5116 carries Route 170 over Mattagodus Stream Location: Webster Plantation, Maine	Boring No.: BB-WPMS-201 WIN: 22266.00
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Driller: MaineDOT	Elevation (ft.): 328.2	Auger ID/OD: 5" Solid Stem
Operator: Travis/Bruce	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: N. Pukay	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 7/12/2022; 08:00-16:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 4+63, 5.2 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 15.1 ft bgs, after drilling

Hammer Efficiency Factor: 0.974	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>		
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 Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
50									slightly to moderately weathered, joints dipping at low to high angles, very close to close, open. Very thin high angle bedding. Allsbury Formation Rock Quality = Very Poor 100% Recovery R3: Core Times (min:sec) 43.8-44.8 ft (2:27) brown wash water 44.8-45.8 ft (1:57) 45.8-46.8 ft (1:54) Bottom of Exploration at 46.8 feet below ground surface.		
51											
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75											

Remarks:
 N 616104.4600 E 2223704.5600 - Maine 2000 East Zone
 Note: Soil and Rock samples collected by MaineDOT and reviewed by Haley & Aldrich, Inc.

**ROCK CORE PHOTOGRAPHS
MATTAGODUS STREAM BRIDGE OVER MATTAGODUS STREAM NO. 5116
MAINEDOT WIN 022266.00
ROUTE 170, WEBSTER PLANTATION, MAINE**



Top Row: BB-WPMS-101: Run No. R1 33.4' (left) to 38.4' (right)
Top Middle Row: BB-WPMS-101: Run No. R2 38.4' (left) to 43.4' (right)
Bottom Middle Row: BB-WPMS-102: Run No. R1 15.5' (left) to 20.5' (right)
Bottom Row: BB-WPMS-102: Run No. R2 20.5' (left) to 25.5' (right)

**ROCK CORE PHOTOGRAPHS
MATTAGODUS STREAM BRIDGE OVER MATTAGODUS STREAM NO. 5116
MAINEDOT WIN 022266.00
ROUTE 170, WEBSTER PLANTATION, MAINE**



Top Row: BB-WPMS-103: Run No. R1 11.9' (left) to 16.4' (right)
Top Middle Row: BB-WPMS-103: Run No. R2 16.4' (left) to 20.9' (right)

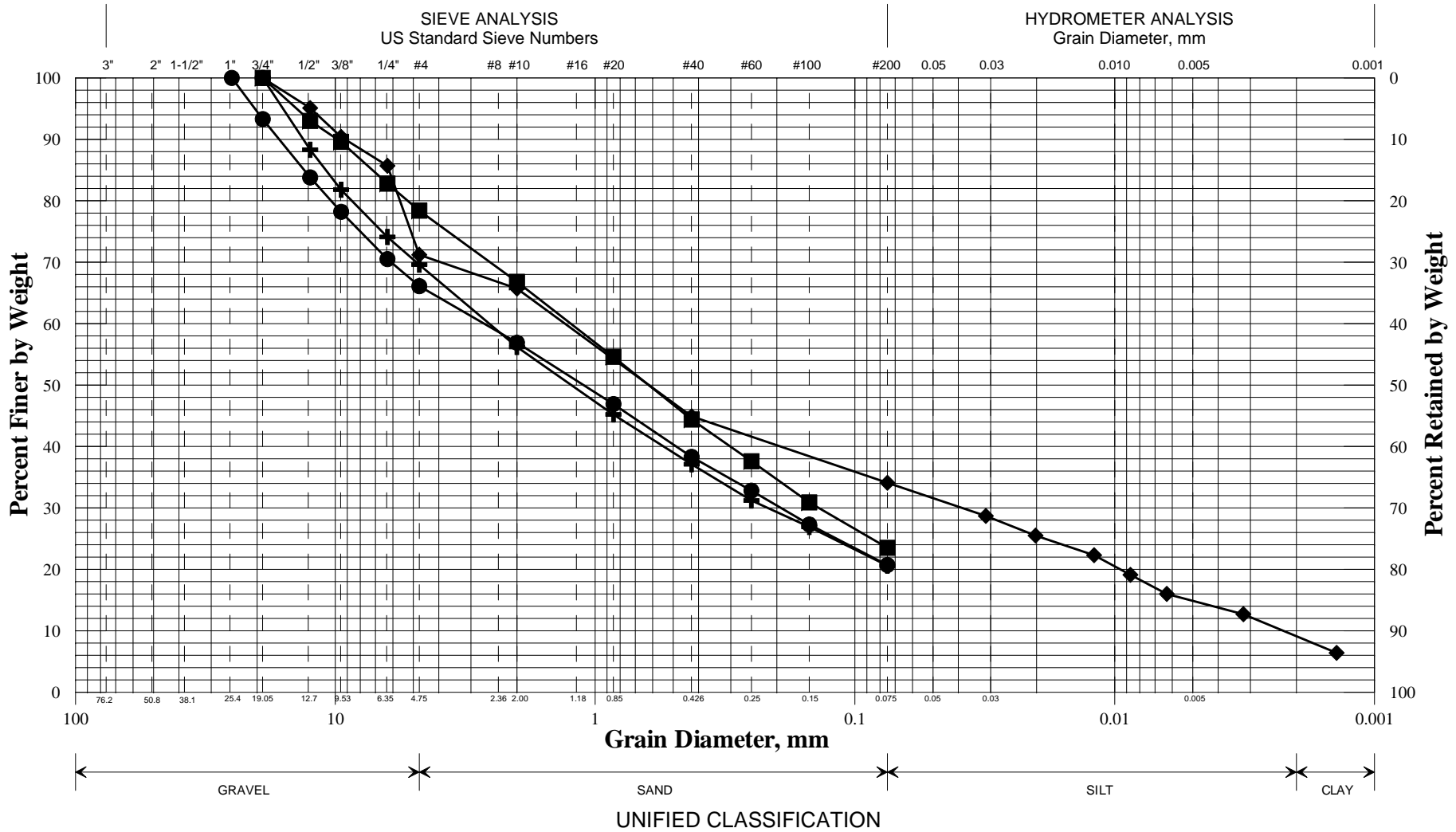
**ROCK CORE PHOTOGRAPHS
MATTAGODUS STREAM BRIDGE OVER MATTAGODUS STREAM NO. 5116
MAINEDOT WIN 022266.00
ROUTE 170, WEBSTER PLANTATION, MAINE**



Top Row: BB-WPMS-201: Run No. R1 35.6' (left) to 38.8' (right)
Middle Row: BB-WPMS-201: Run No. R2 38.8' (left) to 43.8' (right)
Bottom Row: BB-WPMS-201: Run No. R3 43.8' (left) to 46.8' (right)

APPENDIX B
Laboratory Test Results

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WPMS-101/1D	4+57.2	6.5 LT	5.0-7.0	SAND, some gravel, little silt.	8.0			
◆	BB-WPMS-101/2D	4+57.2	6.5 LT	15.0-17.0	SAND, some gravel, some silt, trace clay.	16.7			
■	BB-WPMS-102/1D	5+40.2	7.2 RT	5.0-7.0	SAND, some silt, some gravel.	7.0			
●	BB-WPMS-102/2D	5+40.2	7.2 RT	10.0-12.0	SAND, some gravel, little silt.	8.8			
▲									
×									

WIN
022266.00
Town
Webster Plt
Reported by/Date
WHITE, TERRY A 7/16/2018



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296512	BB-WPMS-101/1D	GEOTECHNICAL (DISTURBED)	6/21/2018	7/3/2018
Sample Type: GEOTECHNICAL Location:		Station: 4+57.2 Offset, ft: 6.5 LT Dbfg, ft: 5.0-7.0	Sampler: BRUCE WILDER	
WIN/Town 022266.00 - WEBSTER PLT				

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]	88.3
⅜ in. [9.5 mm]	81.8
¼ in. [6.3 mm]	74.1
No. 4 [4.75 mm]	69.6
No. 10 [2.00 mm]	56.2
No. 20 [0.850 mm]	45.2
No. 40 [0.425 mm]	37.1
No. 60 [0.250 mm]	31.2
No. 100 [0.150 mm]	26.9
No. 200 [0.075 mm]	20.6

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), %	8.0

Consolidation (T 216)

Trimming, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft ³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft ²	tons/ft ²	tons/ft ²	tons/ft ²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **7/10/2018**

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
296514	BB-WPMS-102/1D	GEOTECHNICAL (DISTURBED)	6/26/2018	7/3/2018
Sample Type: GEOTECHNICAL Location:		Station: 5+40.2 Offset, ft: 7.2 RT Dbfg, ft: 5.0-7.0	Sampler: BRUCE WILDER	
WIN/Town 022266.00 - WEBSTER PLT				

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]	93.0
⅜ in. [9.5 mm]	89.6
¼ in. [6.3 mm]	82.8
No. 4 [4.75 mm]	78.4
No. 10 [2.00 mm]	66.8
No. 20 [0.850 mm]	54.6
No. 40 [0.425 mm]	44.4
No. 60 [0.250 mm]	37.6
No. 100 [0.150 mm]	30.9
No. 200 [0.075 mm]	23.5

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), %	7.0

Consolidation (T 216)					
Trimming, 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.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **7/10/2018**

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
296515	BB-WPMS-102/2D	GEOTECHNICAL (DISTURBED)	6/26/2018	7/3/2018
Sample Type: GEOTECHNICAL Location:		Station: 5+40.2 Offset, ft: 7.2 RT Dbfg, ft: 10.0-12.0	Sampler: BRUCE WILDER	
WIN/Town 022266.00 - WEBSTER PLT				

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]	93.3
½ in. [12.5 mm]	83.8
⅜ in. [9.5 mm]	78.2
¼ in. [6.3 mm]	70.5
No. 4 [4.75 mm]	66.1
No. 10 [2.00 mm]	56.9
No. 20 [0.850 mm]	46.9
No. 40 [0.425 mm]	38.3
No. 60 [0.250 mm]	32.8
No. 100 [0.150 mm]	27.3
No. 200 [0.075 mm]	20.7

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), %	8.8

Consolidation (T 216)					
Trimming, 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.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

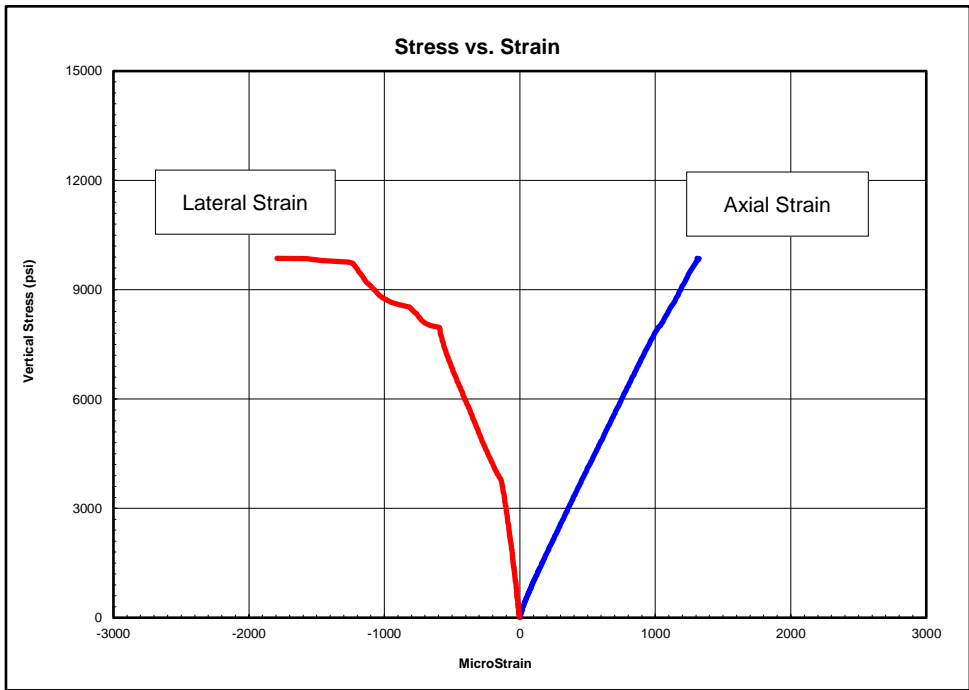
Date Reported: **7/10/2018**

Paper Copy: Lab File; Project File; Geotech File



Client:	Haley & Aldrich, Inc.
Project Name:	Mattagodus Stream Bridge 5116
Project Location:	Webster Plantation, ME
GTX #:	316245
Test Date:	11/4/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-WPMS-102
Sample ID:	R1
Depth, ft:	19.7-20.05
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

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



Peak Compressive Stress: 9,861 psi

The strain values recorded within the second and third stress ranges 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
1000-3600	7,820,000	0.28
3600-6200	7,580,000	---
6200-8900	6,780,000	---

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.



Client:	Haley & Aldrich, Inc.	Test Date:	11/3/2022
Project Name:	Mattagodus Stream Bridge 5116	Tested By:	bp
Project Location:	Webster Plantation, ME	Checked By:	smd
GTX #:	316245		
Boring ID:	BB-WPMS-102		
Sample ID:	R1		
Depth:	19.7-20.05 ft		
Visual Description:	See photographs		

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

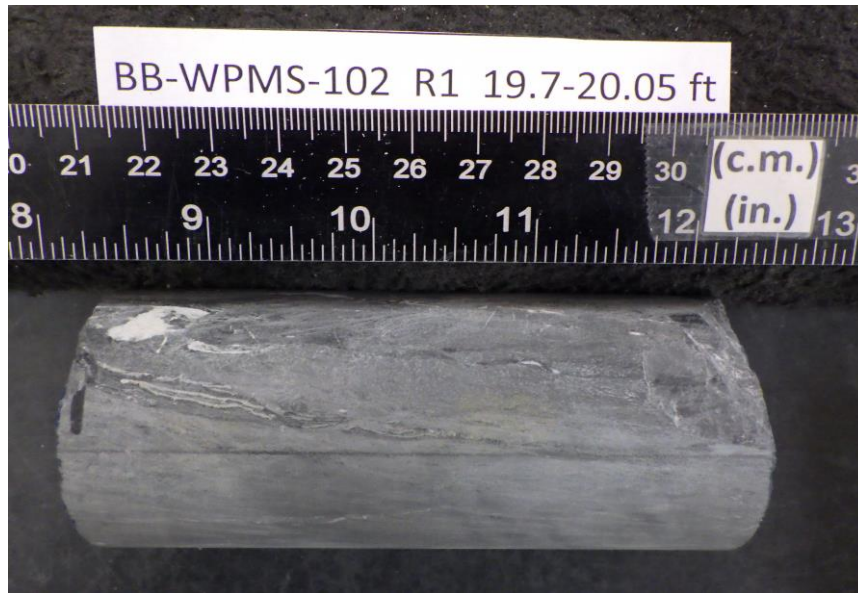
BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)			
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? NO			
Specimen Length, in:	4.23	4.23	4.23	Maximum difference must be < 0.020 in. Straightness Tolerance Met? NO			
Specimen Diameter, in:	1.98	1.98	1.98				
Specimen Mass, g:	578.01						
Bulk Density, lb/ft ³ :	169						
Length to Diameter Ratio:	2.1						
		Minimum Diameter Tolerance Met?	YES				
		Length to Diameter Ratio 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	-0.00010	-0.00010	-0.00010	-0.00020
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	-0.00010	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	0.00000
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	-0.00010
	Difference between max and min readings, in: 0° = 0.0001 90° = 0.0002 Maximum difference must be < 0.0020 in. Difference = \pm 0.00010 Flatness Tolerance Met? YES														

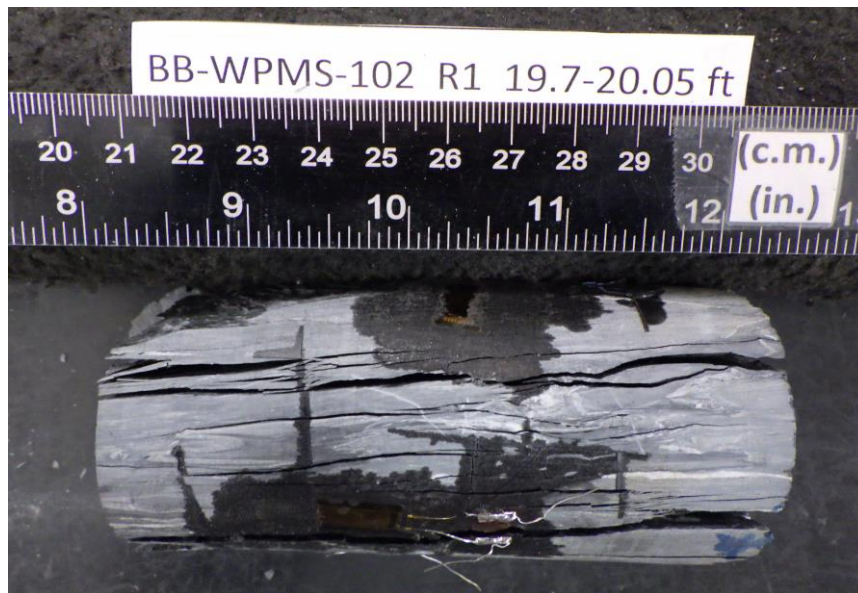
<div style="text-align: center;"> <p>End 1 Diameter 1 $y = -0.00008x - 0.00003$</p> </div> <div style="text-align: center;"> <p>End 2 Diameter 1 $y = 0.00002x - 0.00001$</p> </div>	<div style="text-align: center;"> <p>End 1 Diameter 2 $y = 0.00004x - 0.00001$</p> </div> <div style="text-align: center;"> <p>End 2 Diameter 2 $y = -0.00006x + 0.00001$</p> </div>	<p>DIAMETER 1</p> <p>End 1: Slope of Best Fit Line: 0.00008 Angle of Best Fit Line: 0.00475</p> <p>End 2: Slope of Best Fit Line: 0.00002 Angle of Best Fit Line: 0.00115</p> <p>Maximum Angular Difference: 0.00360</p> <p align="right">Parallelism Tolerance Met? YES Spherically Seated</p> <hr/> <p>DIAMETER 2</p> <p>End 1: Slope of Best Fit Line: 0.00004 Angle of Best Fit Line: 0.00213</p> <p>End 2: Slope of Best Fit Line: 0.00006 Angle of Best Fit Line: 0.00327</p> <p>Maximum Angular Difference: 0.00115</p> <p align="right">Parallelism Tolerance Met? YES Spherically Seated</p>
--	--	---

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)						Maximum angle of departure must be \leq 0.25°	
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	Perpendicularity Tolerance Met? YES	
END 2							
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES		
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	Mattagodus Stream Bridge 5116
Project Location:	Webster Plantation, ME
GTX #:	316245
Test Date:	11/4/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-WPMS-102
Sample ID:	R1
Depth, ft:	19.7-20.05



After cutting and grinding

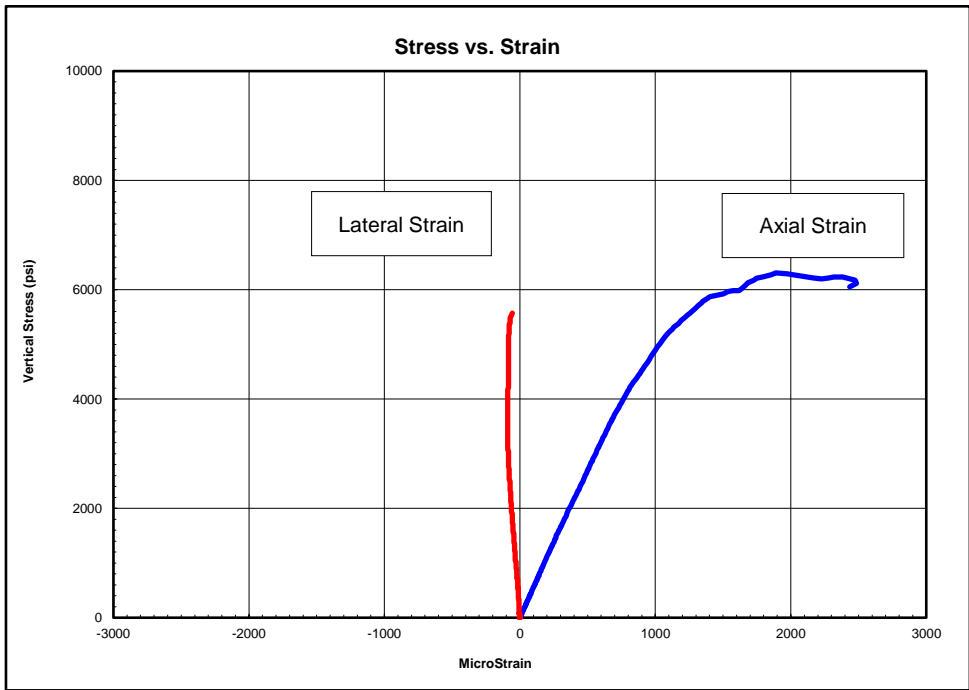


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Mattagodus Stream Bridge 5116
Project Location:	Webster Plantation, ME
GTX #:	316245
Test Date:	11/4/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-WPMS-102
Sample ID:	R2
Depth, ft:	23.6-23.96
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

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



Peak Compressive Stress: 6,308 psi

The lateral strain gauges failed before the peak value was attained. Poisson's Ratio could not be determined within the third stress range.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
600-2300	5,380,000	0.18
2300-4000	5,130,000	0.07
4000-5700	3,190,000	---

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.



Client:	Haley & Aldrich, Inc.	Test Date:	11/3/2022
Project Name:	Mattagodus Stream Bridge 5116	Tested By:	bp
Project Location:	Webster Plantation, ME	Checked By:	smd
GTX #:	316245		
Boring ID:	BB-WPMS-102		
Sample ID:	R2		
Depth:	23.6-23.96 ft		
Visual Description:	See photographs		

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

BULK DENSITY				Average		DEVIATION FROM STRAIGHTNESS (Procedure S1)	
Specimen Length, in:	1	2		4.17	4.17	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? NO	
Specimen Diameter, in:				1.98	1.99	Maximum difference must be < 0.020 in. Straightness Tolerance Met? NO	
Specimen Mass, g:				575.6			
Bulk Density, lb/ft ³ :				170			
Length to Diameter Ratio:				2.1			
				Minimum Diameter Tolerance Met? YES	YES		
				Length to Diameter Ratio Tolerance Met? YES	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	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030
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° = 0.00040 90° = 0.00000			
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	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	-0.00020	-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
												Difference between max and min readings, in: 0° = 0.0004 90° = 0.0002 Maximum difference must be < 0.0020 in. Difference = \pm 0.00020 Flatness Tolerance Met? YES			

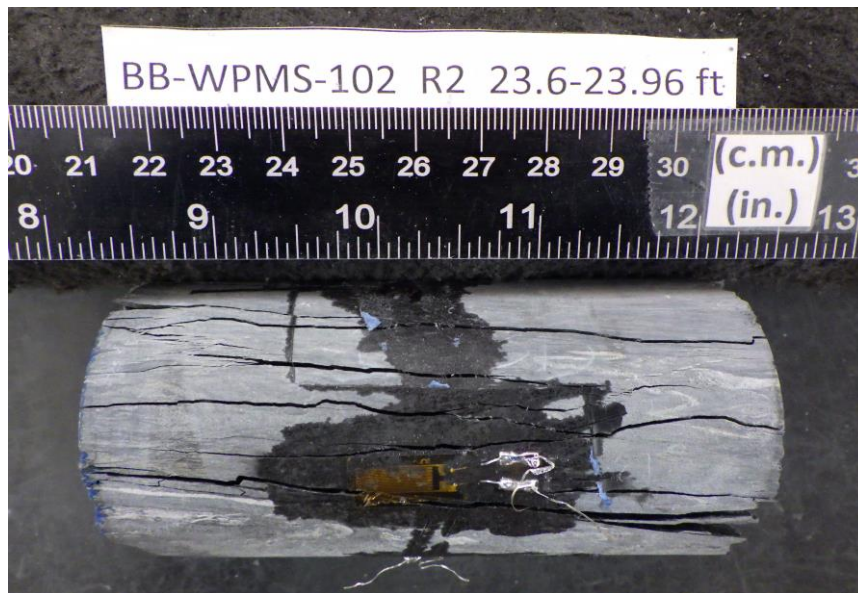
<p align="center">End 1 Diameter 1 $y = -0.00017x - 0.00004$</p>	<p align="center">End 1 Diameter 2 $y = 0.00000$</p>	<p>DIAMETER 1</p> <p>End 1: Slope of Best Fit Line: 0.00017 Angle of Best Fit Line: 0.00982</p> <p>End 2: Slope of Best Fit Line: 0.00015 Angle of Best Fit Line: 0.00884</p> <p>Maximum Angular Difference: 0.00098</p> <p align="center">Parallelism Tolerance Met? YES Spherically Seated</p>
<p align="center">End 2 Diameter 1 $y = -0.00015x - 0.00005$</p>	<p align="center">End 2 Diameter 2 $y = 0.00007x - 0.00003$</p>	

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1	Diameter (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be \leq 0.25°	
Diameter 1, in	0.00040	1.985	0.00020	0.012	YES		
Diameter 2, in (rotated 90°)	0.00000	1.985	0.00000	0.000	YES	Perpendicularity Tolerance Met? YES	
END 2	Diameter (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00040	1.985	0.00020	0.012	YES		
Diameter 2, in (rotated 90°)	0.00020	1.985	0.00010	0.006	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	Mattagodus Stream Bridge 5116
Project Location:	Webster Plantation, ME
GTX #:	316245
Test Date:	11/4/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-WPMS-102
Sample ID:	R2
Depth, ft:	23.6-23.96

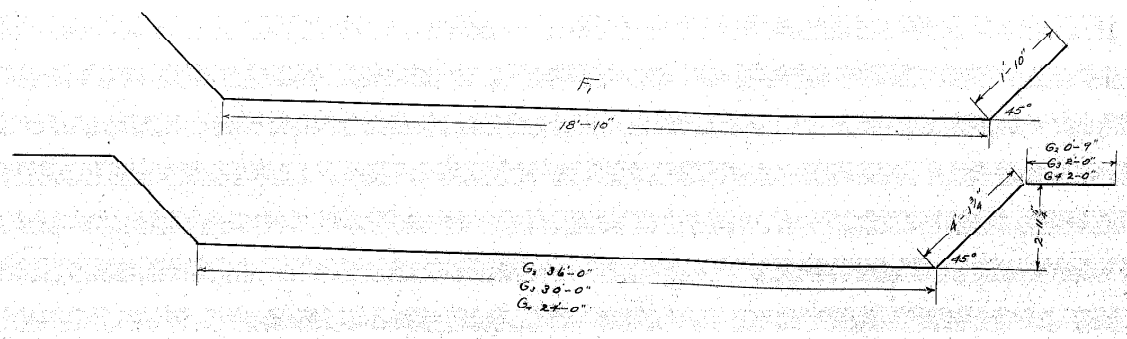
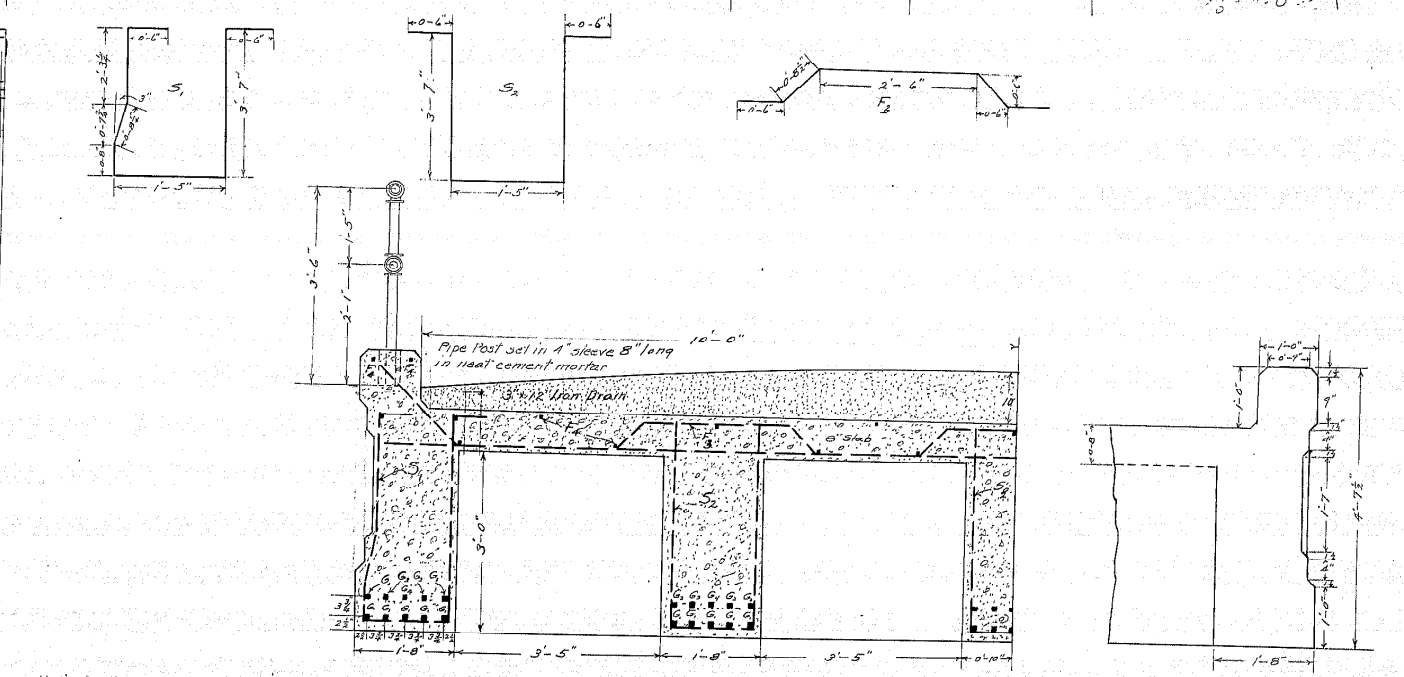
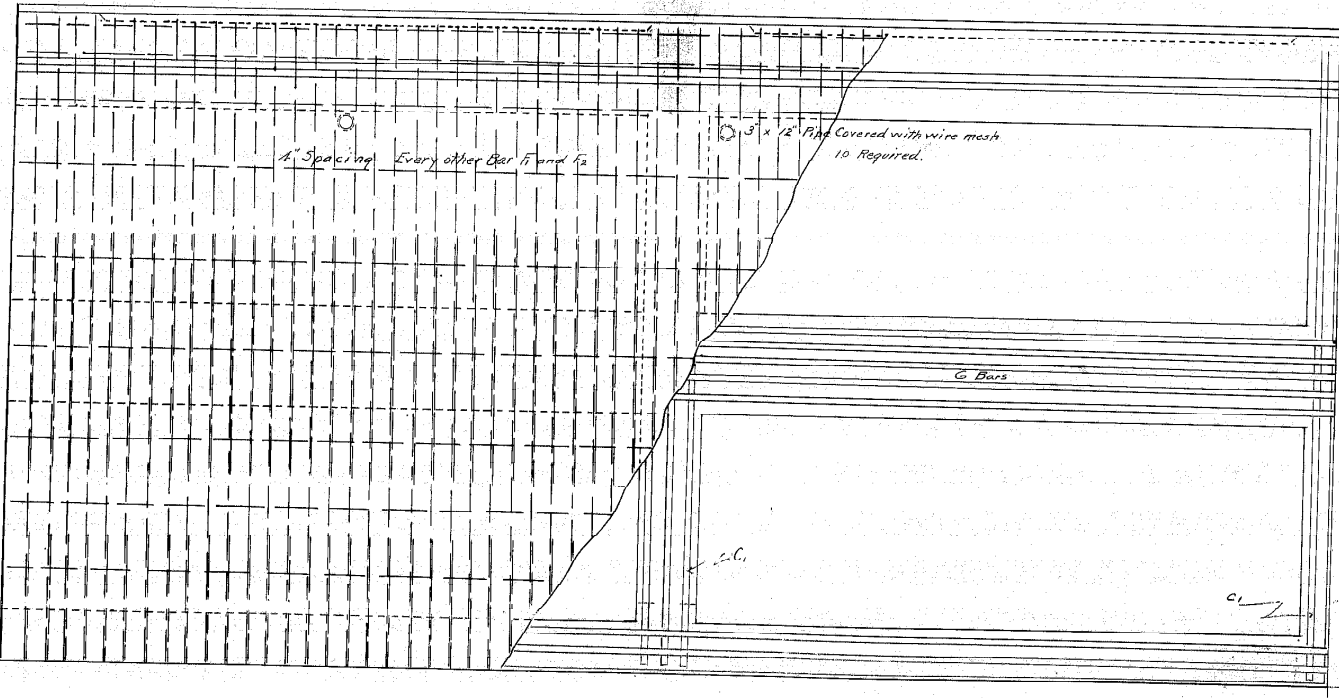
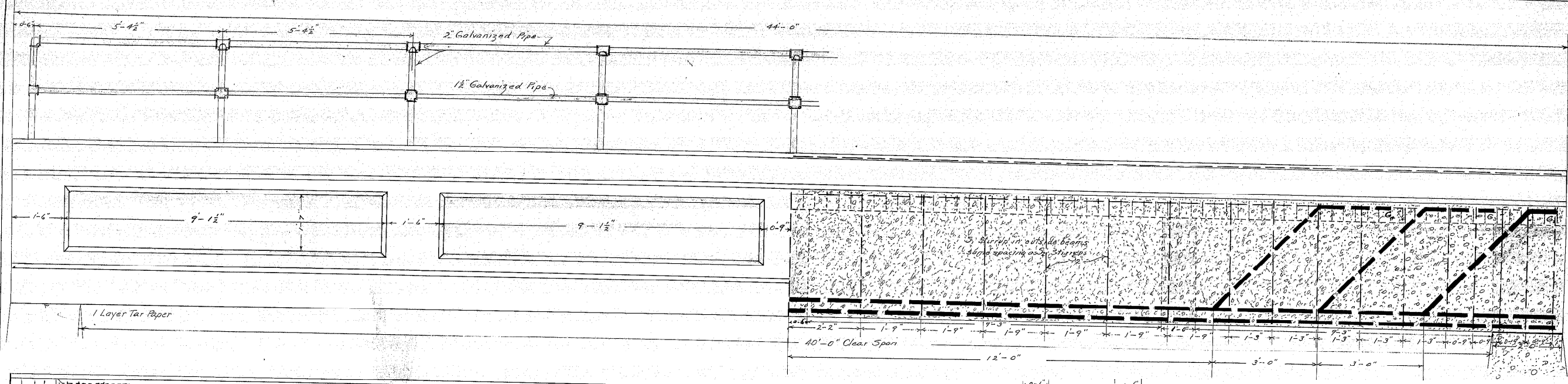


After cutting and grinding



After break

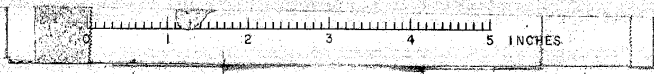
APPENDIX C
Historic Bridge Drawings

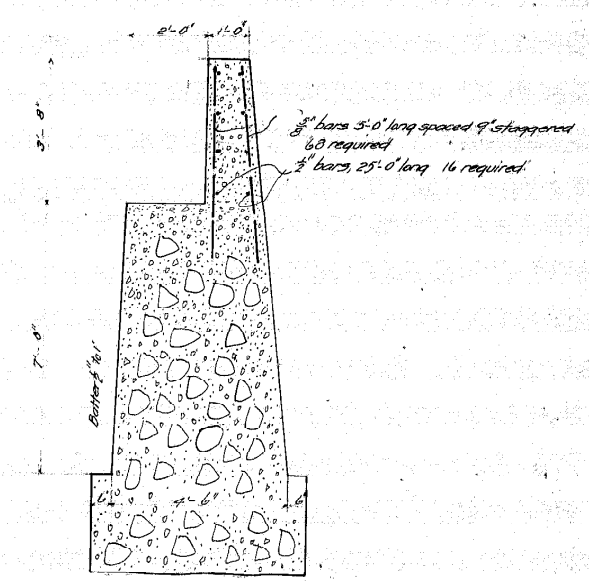
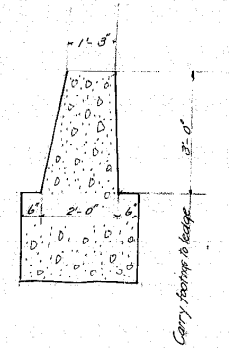
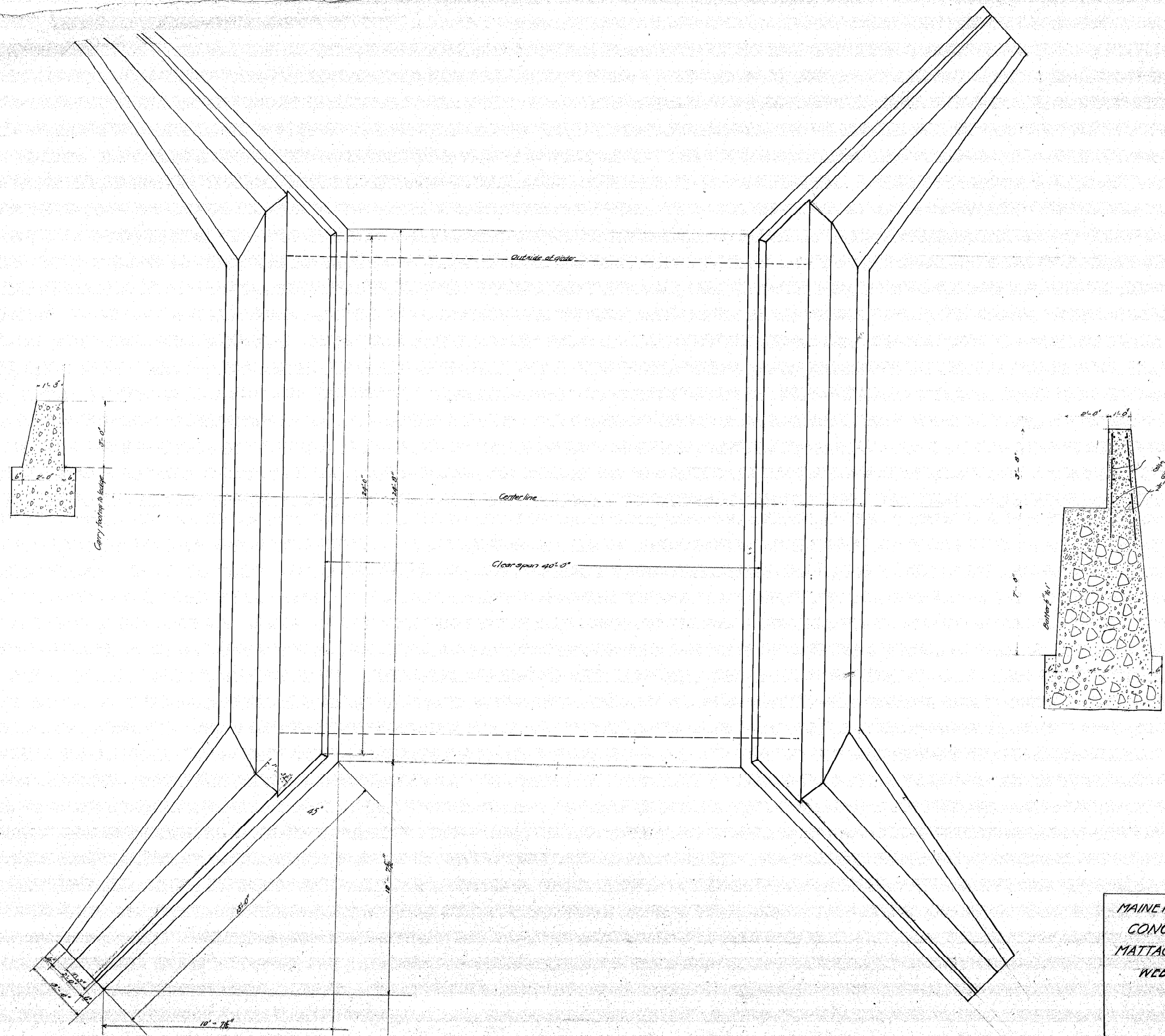


Bar No	Length	Length 1/2	Length 2	Location
G1	13'-6"	1262		Main Bars Slab
G2	16'-8"	280		"
G3	12'-3"	123		"
G4	16'-3"	182		"
F1	22'-6"	1440		Slab Trans
F2	21'-9"	1381		"
F3	21'-11"	1704		Negative
F4	23'-6"	1000		Trans'l
C1	31'-6"	323		Cross Beam
S1	9'-7"	434		Strip
M1	7'-0"	288		Corner Bars
		2758	7307	
		14650	6210	

MAINE HIGHWAY COMMISSION
 PLAN OF
 CONCRETE BRIDGE
 MATTAGODDUS STREAM
 WEBSTER PL. MAINE

109-129





MAINE HIGHWAY COMMISSION
 CONCRETE ABUTMENTS
 MATTAGOODUS STREAM
 WEBSTER PLANTATION
 1918

APPENDIX D
Geotechnical Calculations

**Seismic Site Class and
Design Parameters**

File No.	0205793-000
Sheet	1 of 5
Date	1-Aug-22
Computed by	SSM
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
Subject	Seismic Site Class Evaluation

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using SPT N-values from the test borings drilled approximately near the proposed bridge substructures.

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the four test borings (BB-WPMS-101, BB-WPMS-102, BB-WPMS-103, and BB-WPMS-201), we recommend a **Seismic Site Class D**.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. MaineDOT Bridge Design Guide, August 2003, with June 2018 updates

AVAILABLE INFORMATION

1. Boring logs BB-WPMS-101, BB-WPMS-102, BB-WPMS-103, and BB-WPMS-201 drilled and monitored by MaineDOT.
2. Elevations reference the North American Vertical Datum of 1988 (NAVD 88).

ASSUMPTIONS

1. Where SPT N-values were not available to depths greater than 100 ft, the subsurface profile was extended to 100 ft. The SPT N-values for the extended profile were then assumed based on the available subsurface information.
2. WOH/WOR = SPT N-value of 1.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.10.3.1-1), requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. Categorize the site using one of the following three methods (Method A, B, or C).

Method A

Average shear wave velocity for the upper 100 ft of the soil profile:

$$\bar{V}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of i th soil (ft/s).

d_i = thickness of i th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

File No.	0205793-000
Sheet	2 of 5
Date	1-Aug-22
Computed by	SSM
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
Subject	Seismic Site Class Evaluation

PROCEDURE

Method B

Average standard penetration test (SPT) for the upper 100 ft of the soil profile:

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th soil layer not to exceed 100 ft (blows/ft).

d_i = thickness of i th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

Method C

Average standard penetration test (SPT) for the cohesionless layers in the upper 100 ft of the soil profile:

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th cohesionless soil layer (blows/ft).

d_i = thickness of i th cohesionless soil layer (ft).

m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and m .

Average undrained shear strength for the cohesive layers in the upper 100 ft of the soil profile:

$$\bar{s}_u = \frac{\sum_{i=1}^k d_i}{\sum_{i=1}^k \frac{d_i}{s_{ui}}}$$

where

s_{ui} = undrained shear strength of i th cohesive soil layer (psf), not to exceed 5000 psf

d_i = thickness of i th cohesive soil layer (ft).

k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and k .

Based on the available information, Method A/B/C will be used for the seismic Site Class evaluation.

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
Subject	Seismic Site Class Evaluation

SITE CLASS DEFINITIONS

(Table from AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.)

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5,000$ ft/s
B	Rock with $2,500$ ft/sec $< \bar{v}_s < 5,000$ ft/s
C	Very dense soil and soil rock with $1,200$ ft/sec $< \bar{v}_s < 2,500$ ft/s, or with either $\bar{N} > 50$ blows/ft, or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with 600 ft/s $< \bar{v}_s < 1,200$ ft/s, or with either $15 < \bar{N} < 50$ blows/ft, or $1.0 < \bar{s}_u < 2.0$ ksf
E	Soil profile with $\bar{v}_s < 600$ ft/s or with either $\bar{N} < 15$ blows/ft or $\bar{s}_u < 1.0$ ksf, or any profile with more than 10.0 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u < 0.5$ ksf
F	Soils requiring site-specific evaluations, such as: <ul style="list-style-type: none"> • Peats or highly organic clays ($H > 10.0$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25.0$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

- \bar{v}_s = average shear wave velocity for the upper 100 ft of the soil profile
- \bar{N} = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile
- \bar{s}_u = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile
- PI = plasticity index (ASTM D4318)
- w = moisture content (ASTM D2216)



CALCULATIONS

File No.	0205793-000
Sheet	4 of 5
Date	1-Aug-22
Computed by	SSM
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID:

BB-WPMS-101

Ground Surface El.:

327.8

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N-value (blows/ft)	d/N
1D	5.0	322.8	SAND (Fill)	7.0	13	0.538
2D	10.0	317.8	SAND (Fill)	4.0	2	2.000
3D	15.0	312.8	SAND (Fill)	5.0	9	0.556
4D	20.0	307.8	Silty SAND (Glacial Till)	6.0	60	0.100
5D	25.0	302.8	SAND (Glacial Till)	4.0	29	0.138
6D	30.0	297.8	SAND (Glacial Till)	4.4	51	0.086
R1/R2	33.4	294.4	BEDROCK	69.6	100	0.696
Totals =				100.0		4.114

N-bar (blows/ft) = 24
Seismic Site Class = D

Exploration ID:

BB-WPMS-102

Ground Surface El.:

328.4

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N-value (blows/ft)	d/N
1D	5.0	323.4	SAND (Fill)	10.0	4	2.500
2D	10.0	318.4	SAND (Fill)	5.5	3	1.833
R1/R2	15.5	312.9	BEDROCK	84.5	100	0.845
Totals =				100.0		5.178

N-bar (blows/ft) = 19
Seismic Site Class = D



CALCULATIONS

File No.	0205793-000
Sheet	5 of 5
Date	1-Aug-22
Computed by	SSM
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID:

BB-WPMS-103

Ground Surface El.:

328.5

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N-value* (blows/ft)	d/N
-	-	328.5	SAND (Fill)	11.9	3	3.967
R1/R2	11.9	316.6	BEDROCK	88.1	100	0.881
Totals =				100.0		4.848

N-bar (blows/ft) = 21
Seismic Site Class = D

Note*: No SPT N-values recorded in fill. Assume SPT N-value of 3 (similar to test boring BB-WPMS-102, as note by MaineDOT field representative).

Exploration ID:

BB-WPMS-201

Ground Surface El.:

328.2

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N-value (blows/ft)	d/N
1D	5.0	323.2	SAND (Fill)	10.0	12	0.833
2D	10.0	318.2	SAND (Fill)	5.0	6	0.833
3D	15.0	313.2	SILT (Glacial Till)	4.6	15	0.307
4D	20.0	308.2	SAND (Glacial Till)	5.4	67	0.081
5D	25.0	303.2	SAND (Glacial Till)	10.2	37	0.276
R1/R2/R3	35.2	293.0	BEDROCK	64.8	100	0.648
Totals =				100.0		2.978

N-bar (blows/ft) = 34
Seismic Site Class = D

Abutment No. 1
Static Pile Resistance

Client: Maine Department of Transportation

Date: 11/18/2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116

Computed By: BCS 7/21/22

Subject: Axial Compressive Structural Resistance of Steel H-piles

Checked By: NAS 7/21/22

Objective:

Calculate nominal and factored axial compressive structural resistance of steel H-piles supporting Abutment 1.

References:

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.
2. MaineDOT Bridge Design Guide, 2014.

Evaluation:

Step 1 - Summarize weak axis cross-sectional properties for various steel H-pile

$F_y := 50\text{ksi}$ steel yield strength

$E := 30000\text{ksi}$ Young's modulus of steel

<i>gross steel cross-sectional area, $A_g :=$</i>	$\left(\begin{array}{c} 12.4 \\ 16.7 \\ 15.5 \\ 21.8 \\ 24.6 \\ 21.4 \\ 26.1 \\ 30.1 \\ 34.4 \end{array} \right) \text{ in}^2$	HP10x42	<i>radius of gyration, $r_s :=$</i>	$\left(\begin{array}{c} 2.41 \\ 2.45 \\ 2.86 \\ 2.92 \\ 2.94 \\ 3.49 \\ 3.53 \\ 3.56 \\ 3.59 \end{array} \right) \text{ in}$	HP10x42
		HP10x57			HP10x57
		HP12x53			HP12x53
		HP12x74			HP12x74
		HP12x84			HP12x84
		HP14x73			HP14x73
		HP14x89			HP14x89
		HP14x102			HP14x102
		HP14x117			HP14x117

Step 2 - Calculate nominal (P_n) and factored (P_r) axial compressive pile resistances at the Service (SER; P_{rser}), Strength (STR; P_{rstr}) and Extreme Event (EE; P_{ree}) Limit States

$$\lambda := \left(\frac{K \cdot l}{r_s \cdot \pi} \right)^2 \cdot \frac{F_y}{E} \quad \lambda_{\text{lim}} := 0 \quad \text{AASHTO LRFD Eqn. 6.9.4.1-3; normalized column slenderness factor for fully embedded pile} = 0 \text{ per AASHTO LRFD Section 10.7.3.13-1}$$

nominal axial compressive structural resistance (P_n); AASHTO LRFD Eqn. 6.9.5.1-1

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_g =$	$\left(\begin{array}{c} 620 \\ 835 \\ 775 \\ 1090 \\ 1230 \\ 1070 \\ 1305 \\ 1505 \\ 1720 \end{array} \right) \cdot \text{kip}$	HP10x42
		HP10x57
		HP12x53
		HP12x74
		HP12x84
		HP14x73
		HP14x89
		HP14x102
		HP14x117

Client: Maine Department of Transportation

Date: 11/18/2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116

Computed By: BCS 7/21/22

Subject: Axial Compressive Structural Resistance of Steel H-piles

Checked By: NAS 7/21/22

factored STR Limit State axial compressive structural resistance (Prstr); AASHTO LRFD Eqn. 6.9.2.1-1

$\phi_c := 0.5$ STR Limit State resistance factor from AASHTO LRFD Section 6.5.4.2

$Prstr := \phi_c \cdot P_n =$	· kip	310	HP10x42
		417	HP10x57
		388	HP12x53
		545	HP12x74
		615	HP12x84
		535	HP14x73
		653	HP14x89
		753	HP14x102
		860	HP14x117

factored SER and EE Limit State axial compressive structural resistance (Prser, Pree); AASHTO LRFD Eqn. 6.9.2.1-1

$\phi := 1.0$ SER and EE Limit State resistance factor from AASHTO LRFD Sections 10.5.5.1 and 10.5.5.3

$Prser := \phi \cdot P_n =$	· kip	620
		835
		775
		1090
		1230
		1070
		1305
		1505
		1720

$Pree := \phi \cdot P_n =$	· kip	620
		835
		775
		1090
		1230
		1070
		1305
		1505
		1720

**Abutment No. 1
Pile Drivability Resistance**

Client: Maine Department of Transportation

Date: 29-Jul-2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00

Computed by: SSM

Subject: Abutment 1 Pile Drivability Analysis

Checked by: NAS/BCS

PROBLEM STATEMENT & OBJECTIVE

Determine nominal and factored drivability resistances for HP14x89 and HP14x117 steel H-pile alternatives being considered to support Abutment 1.

EXECUTIVE SUMMARY

HP14x89 steel H-piles can be installed to a nominal resistance equal to 700 kips (factored resist. = 455 kips) with a Delmag D16-32 OED hammer operating on fuel setting 4 (40,130 ft-lbs.) at a penetration resistance equal to 15 to 16 bpi and with a maximum compressive stress during driving equal to 44 ksi.

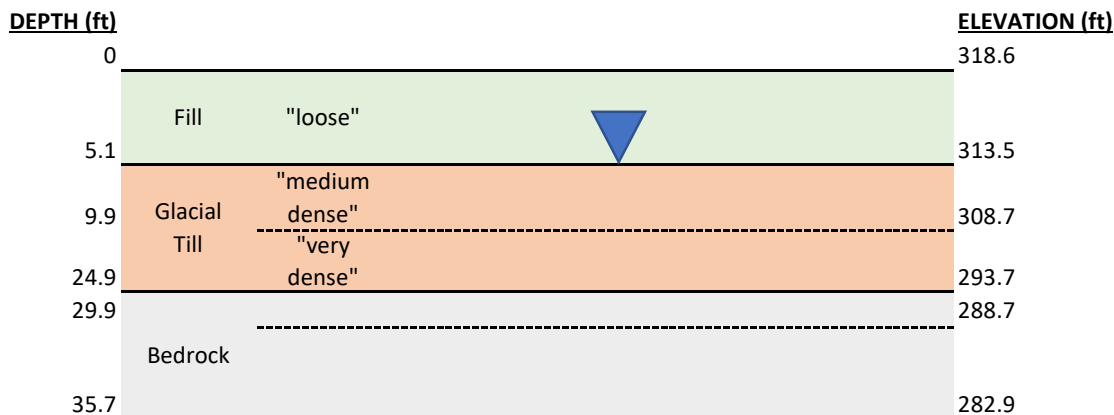
HP14x117 steel H-piles can be installed to a nominal resistance equal to 825 kips (factored resist. = 536 kips) with a Delmag D36-32 OED hammer operating on fuel setting 3 (69,784 ft-lbs.) at a penetration resistance equal to 8 to 9 bpi and with a maximum compressive stress during driving equal to 46 to 47 ksi.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. Maine Department of Transportation Standard Specifications, March 2020, with interim revisions through 9/7/22.
3. GRLWEAP 2014 v14.1.6.0 technical manual.

AVAILABLE INFORMATION

1. Abutment 1 test boring logs (BB-WMPS-101 and BB-WMPS-201).
Soil profile below shows average thicknesses encountered below the bottom of Abutment 1 (El. 318.6)



Groundwater @ Top of Glacial Till = El. 313.5 (depth = 5.1 ft relative to bottom of abutment)

ASSUMPTIONS

1. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD88).
2. Soil and rock conditions based on those encountered in the referenced test borings.
3. Piles will be driven approximately 5 ft into rock based on conditions encountered in the referenced test borings.
4. Two steel H-pile sections will be analyzed (HP14x89 and HP14x117). Piles are assumed to be plumb (vertical).
5. Piles are approx. 35-ft long, which includes 5 ft of pen. into rock and 5 ft of stickup above bottom of abutment.
6. Factored resist. is taken as the nominal resist. multiplied by 0.65 (resistance factor for CAPWAP dynamic testing).
7. Shaft and toe quake and damping factors selected based on experience and default values suggested in WEAP.
8. Penetration resistance is limited to a maximum of 15 bpi in accordance with the MaineDOT Std. Specifications.
9. Limit pile compressive stress during driving to 0.9Fy or 45 ksi for Fy = 50 ksi steel. It is noted that in our experience, WEAP tends to overpredict compressive driving stresses (i.e., actual compressive driving stresses are less than those predicted/calculated by WEAP).
10. Select pile hammer(s) based on those readily available to local pile driving contractors.

File No.:	0205793-00
Sheet:	2 of 2
Date:	29-Jul-2022
Computed by:	SSM
Checked by:	NAS/BCS

Client:	Maine Department of Transportation
Project:	Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
Subject:	Abutment 1 Pile Drivability Analysis

CALCULATIONS AND RESULTS (SEE ATTACHED FOR WEAP OUTPUT)

HP14x89 Steel H-pile Alternative:

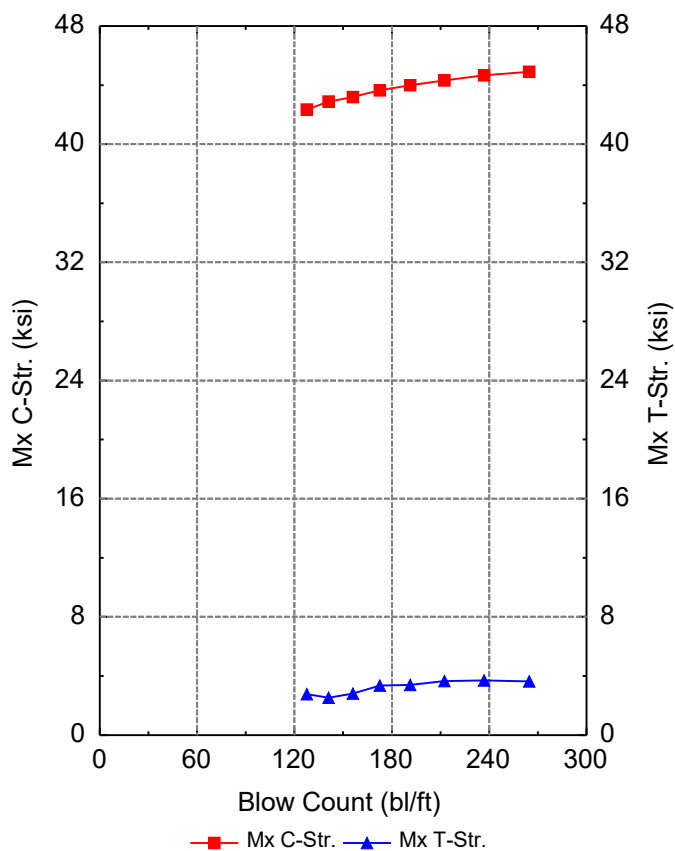
Pile Hammer Make and Model =	Delmag D16-32	
Maximum Rated Energy =	40,130	ft-lbs.
Fuel Setting =	4	
Ram Stroke =	11.4	ft
Rated Energy at Fuel Setting =	40,130	ft-lbs.
Nominal Resistance =	700	kips
Maximum Compressive Stress =	44	ksi
Penetration Resistance =	15 to 16	bpi
Factored Compressive Resistance =	455	kips ($\phi_{dyn} = 0.65$)

Nominal resistance is limited by the maximum permissible penetration resistance (15 bpi). Increasing transferred energy into the pile (i.e., using a larger hammer with a greater maximum rated energy) will reduce the reported penetration resistance but will result in maximum compressive stress that exceeds the maximum permissible. Haley & Aldrich judges the maximum permissible compressive driving stress to be between 45 and 47 ksi because of our experience that suggests WEAP overpredicts compressive stress as compared to what is actually measured with PDA equipment in the field during driving.

HP14x117 Steel H-pile Alternative:

Pile Hammer Make and Model =	Delmag D36-32	
Maximum Rated Energy =	90,402	ft-lbs.
Fuel Setting =	3	
Ram Stroke =	8.8	ft
Rated Energy at Fuel Setting =	69,784	ft-lbs.
Nominal Resistance =	825	kips
Maximum Compressive Stress =	46 to 47	ksi
Penetration Resistance =	8 to 9	bpi
Factored Compressive Resistance =	536	kips ($\phi_{dyn} = 0.65$)

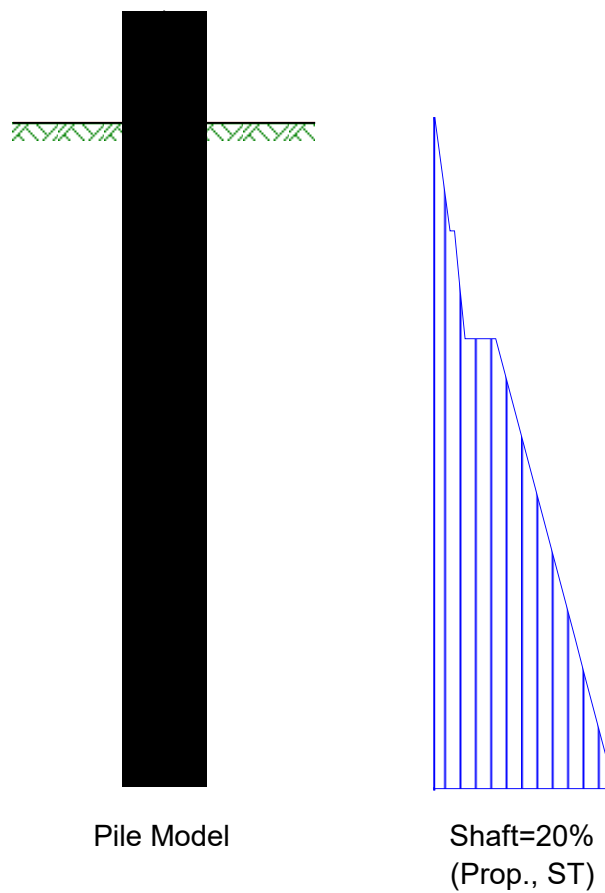
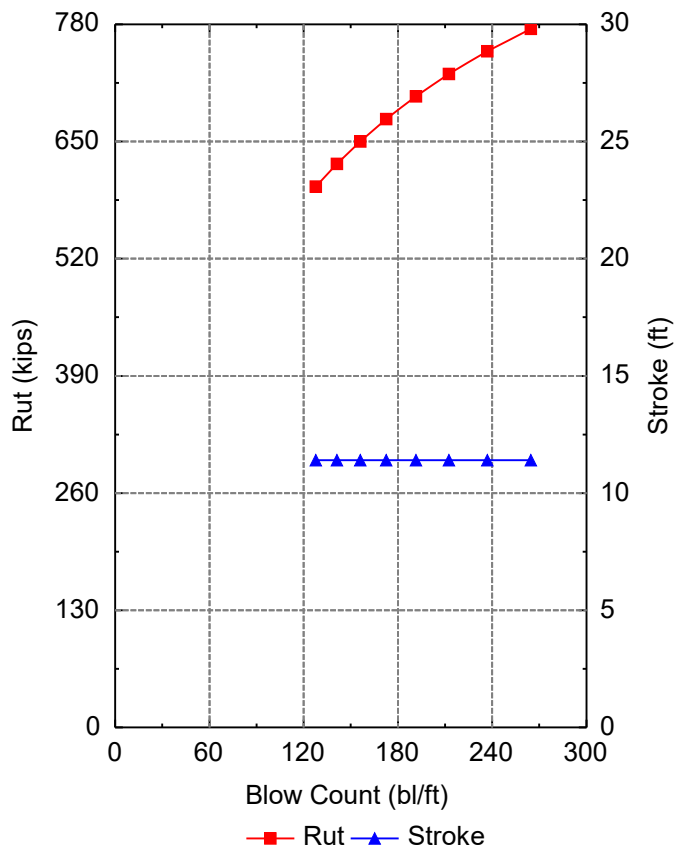
Nominal resistance is limited by the maximum permissible compressive driving stress, which as noted above is judged by Haley & Aldrich to be between 45 and 47 ksi. Reducing the transferred energy into the pile (i.e., using a lower fuel setting with a lesser maximum rated energy) will reduce the reported compressive driving stress but will result in a penetration resistance that exceeds the maximum permissible (15 bpi).



DELMAG D 16-32

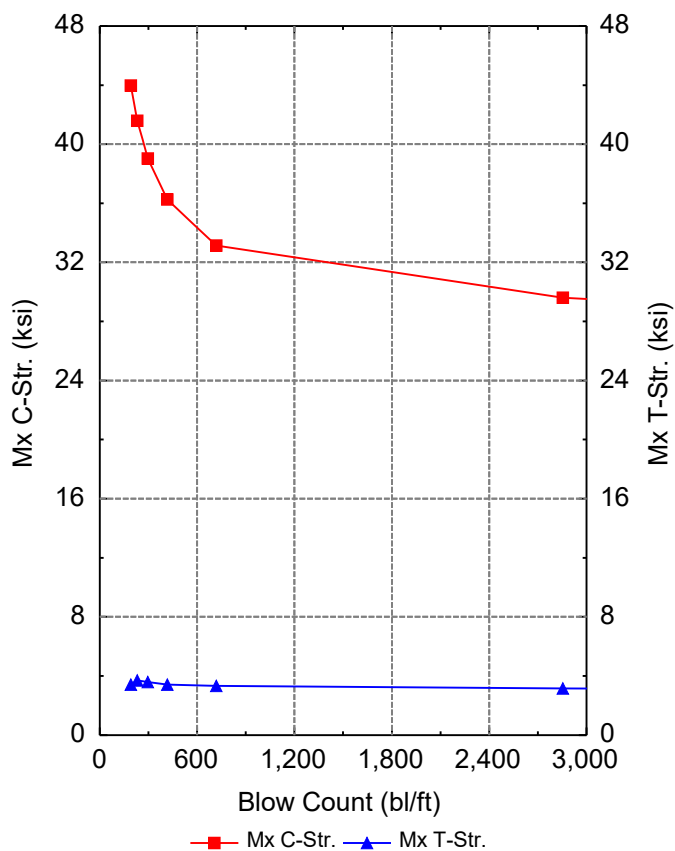
Ram Weight	3.52	kips
Efficiency	0.800	
Pressure	1500.0 (100%)	psi
Helmet Weight	1.900	kips
Hammer Cushion	40103.7	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	35.000	ft
Pile Penetration	30.000	ft
Pile Top Area	26.100	in ²

RSA No



Bearing Graph Summary — DELMAG D 16-32

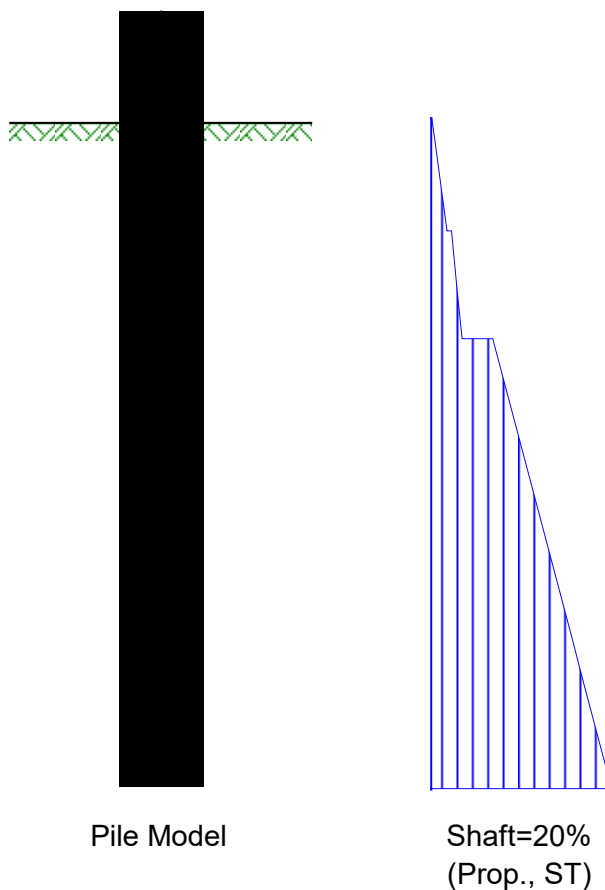
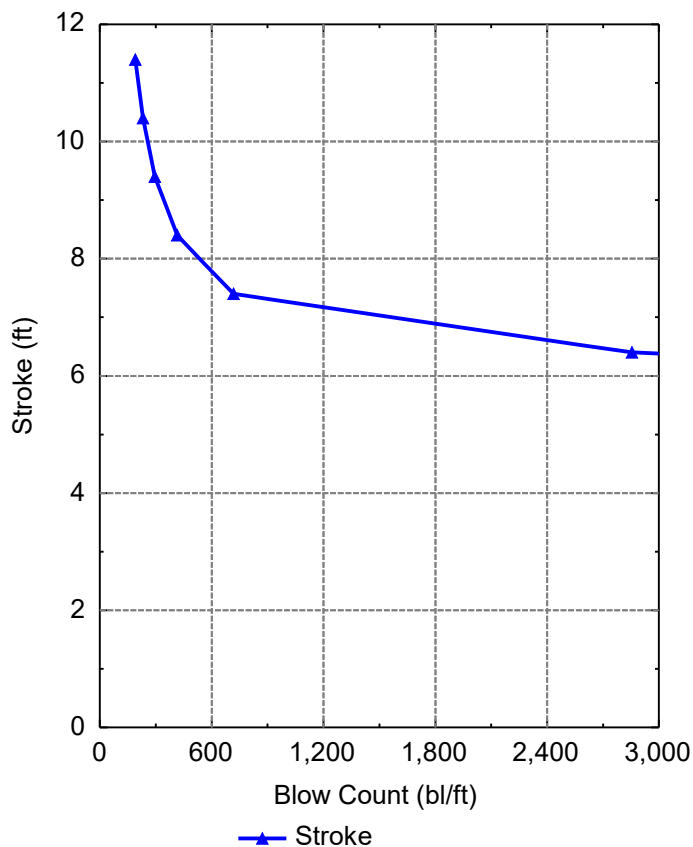
Rut kips	Mx C-Str. ksi	Mx T-Str. ksi	Blow Ct bl/ft	Stroke ft	ENTHRU kip-ft	Hammer -
600.0	42.35	2.76	127.7	11.40	19.83	D 16-32
625.0	42.89	2.53	141.1	11.40	19.84	D 16-32
650.0	43.20	2.81	156.0	11.40	19.80	D 16-32
675.0	43.65	3.35	172.6	11.40	19.78	D 16-32
700.0	43.99	3.38	191.4	11.40	19.76	D 16-32
725.0	44.31	3.66	212.4	11.40	19.75	D 16-32
750.0	44.66	3.71	237.0	11.40	19.70	D 16-32
775.0	44.91	3.62	264.7	11.40	19.70	D 16-32



DELMAG D 16-32

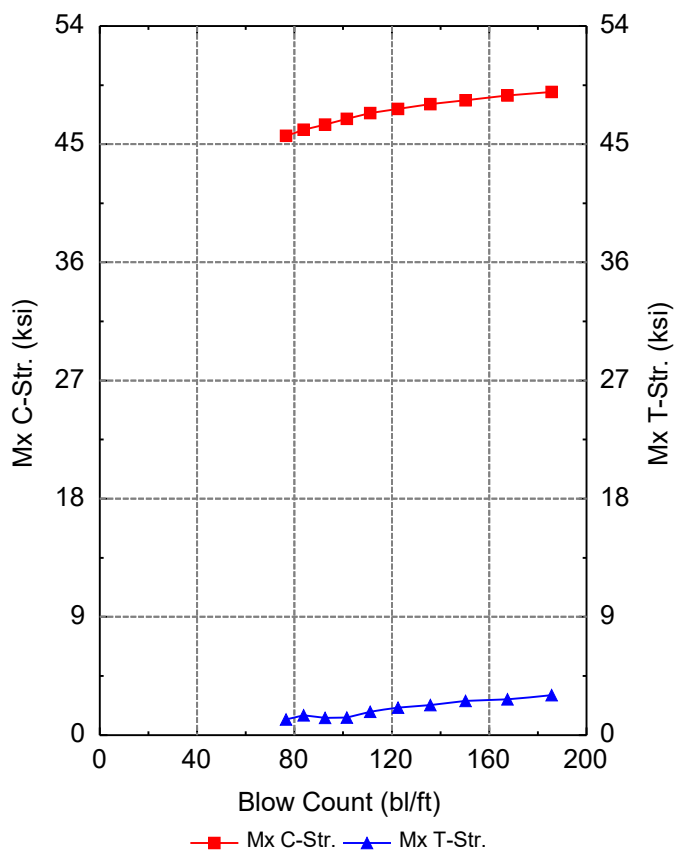
Capacity	700.0	kips
Ram Weight	3.52	kips
Efficiency	0.800	
Pressure	1500.0 (100%)	psi
Helmet Weight	1.900	kips
Hammer Cushion	40103.7	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.149	s/ft
Pile Length	35.000	ft
Pile Penetration	30.000	ft
Pile Top Area	26.100	in ²

RSA No



Bearing Graph Summary — DELMAG D 16-32

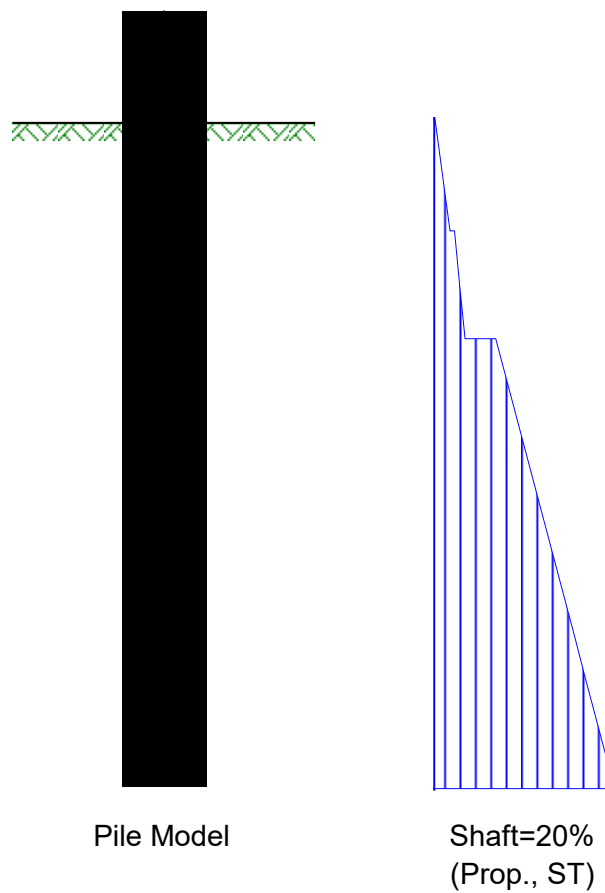
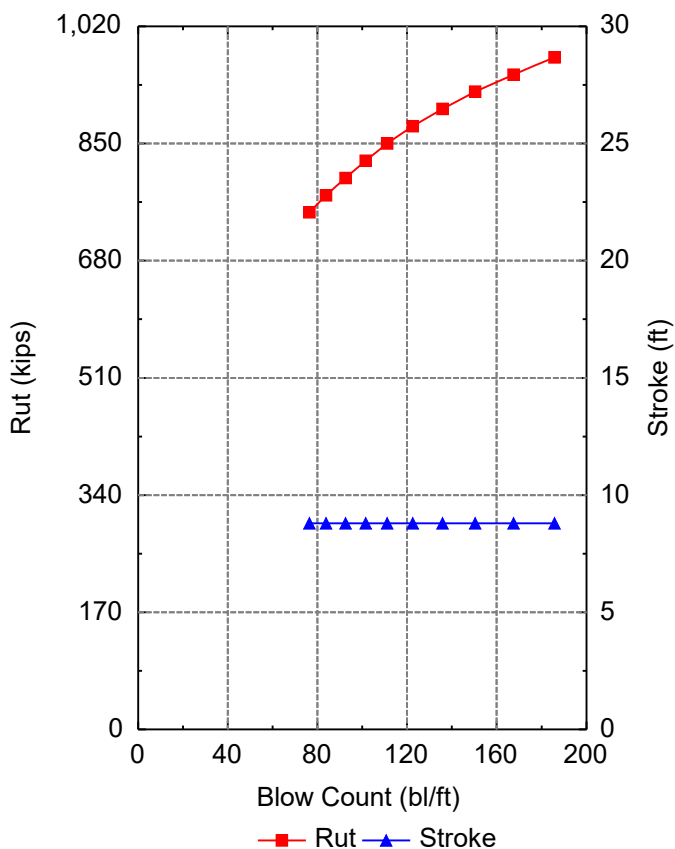
Rut kips	Mx C-Str. ksi	Mx T-Str. ksi	Blow Ct bl/ft	Stroke ft	ENTHRU kip-ft	Hammer -
700.0	25.39	2.86	9999.0	5.40	6.71	D 16-32
700.0	29.61	3.15	2854.9	6.40	8.88	D 16-32
700.0	33.14	3.33	717.1	7.40	11.05	D 16-32
700.0	36.26	3.42	415.6	8.40	13.21	D 16-32
700.0	39.02	3.58	295.2	9.40	15.37	D 16-32
700.0	41.59	3.69	231.3	10.40	17.55	D 16-32
700.0	43.97	3.41	190.5	11.40	19.78	D 16-32



DELMAG D 36-32

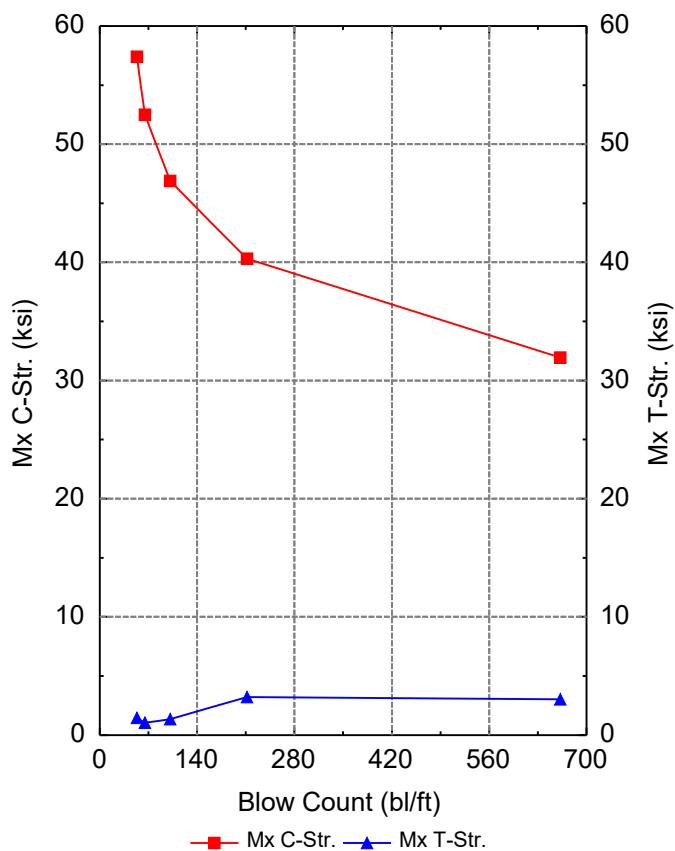
Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500.0 (100%)	psi
Helmet Weight	1.900	kips
Hammer Cushion	40103.7	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	35.000	ft
Pile Penetration	30.000	ft
Pile Top Area	34.400	in ²

RSA No



Bearing Graph Summary — DELMAG D 36-32

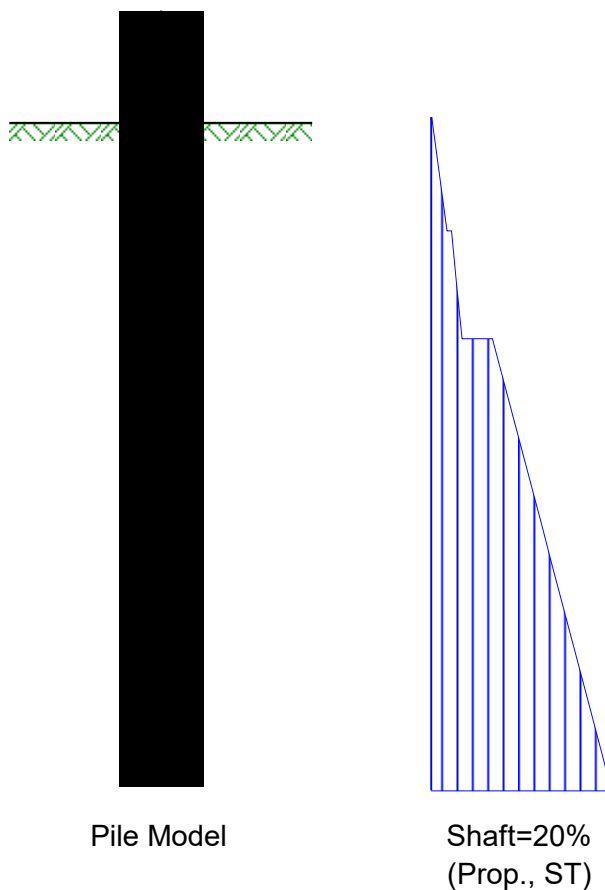
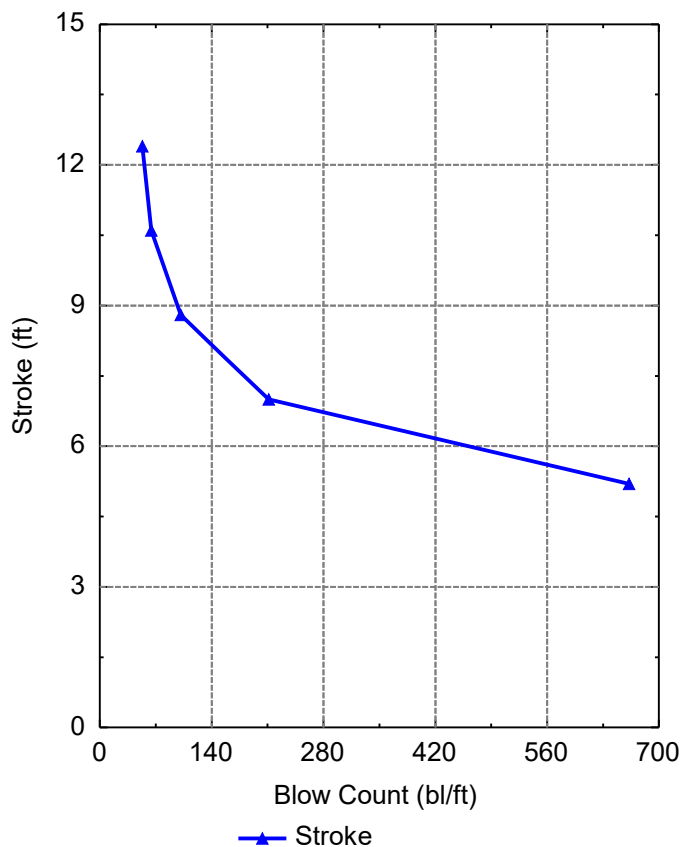
Rut kips	Mx C-Str. ksi	Mx T-Str. ksi	Blow Ct bl/ft	Stroke ft	ENTHRU kip-ft	Hammer -
750.0	45.63	1.19	76.5	8.80	30.34	D 36-32
775.0	46.09	1.51	83.8	8.80	30.00	D 36-32
800.0	46.50	1.31	92.6	8.80	29.84	D 36-32
825.0	46.93	1.34	101.5	8.80	29.78	D 36-32
850.0	47.36	1.78	111.1	8.80	29.81	D 36-32
875.0	47.68	2.09	122.5	8.80	29.75	D 36-32
900.0	48.06	2.29	135.8	8.80	29.66	D 36-32
925.0	48.35	2.59	150.3	8.80	29.60	D 36-32
950.0	48.71	2.72	167.5	8.80	29.49	D 36-32
975.0	48.99	3.03	185.7	8.80	29.47	D 36-32



DELMAG D 36-32

Capacity	825.0	kips
Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500.0 (100%)	psi
Helmet Weight	1.900	kips
Hammer Cushion	40103.7	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	35.000	ft
Pile Penetration	30.000	ft
Pile Top Area	34.400	in ²

RSA No



Bearing Graph Summary — DELMAG D 36-32

Rut kips	Mx C-Str. ksi	Mx T-Str. ksi	Blow Ct bl/ft	Stroke ft	ENTHRU kip-ft	Hammer -
825.0	31.94	3.01	662.6	5.20	12.70	D 36-32
825.0	40.32	3.22	211.5	7.00	21.19	D 36-32
825.0	46.89	1.33	101.1	8.80	29.86	D 36-32
825.0	52.50	1.04	64.6	10.60	39.71	D 36-32
825.0	57.40	1.45	53.3	12.40	47.01	D 36-32

Abutment No. 1
Integral Abutment Pile Design

Client: Maine Department of Transportation

Date: 12/13/2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00

Computed by: BCS

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

Checked by: JLL

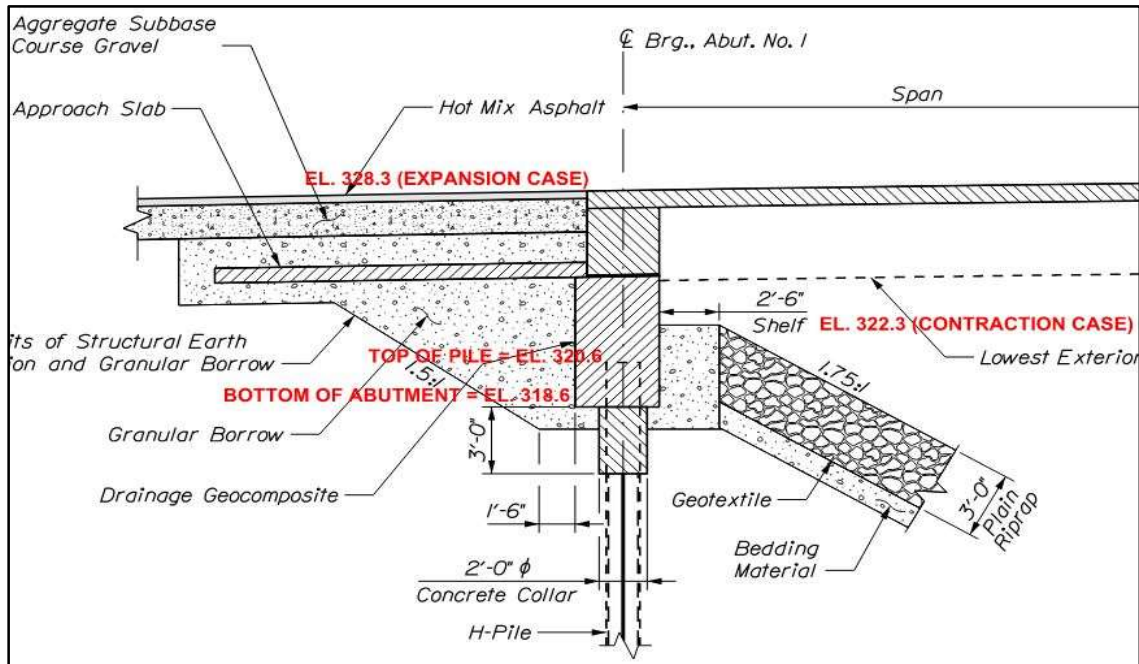
PROBLEM STATEMENT & OBJECTIVE

Determine design soil profile and soil properties for Abutment 1 integral abutment and lateral pile analysis. Conduct lateral pile analysis for HP14x89 and HP14x117 steel H-piles to determine whether combined stresses are above or below the elastic range.

AVAILABLE INFORMATION, REFERENCES AND ASSUMPTIONS

1. Abutment 1 test boring logs BB-WMPS-101 and BB-WMPS-201.
2. Ensoft, Inc. LPILE v2019 Technical Manual, March 2020.
3. Bottom of abutment and top of pile elevations provided by MaineDOT (El. 318.6 and El. 320.6).
4. Maximum factored (STR) axial compressive pile load (360 kips), thermal displacement (0.3 in.) and plastic moment (171.5 kip-ft; HP14x89 expansion case) provided by MaineDOT.
5. Evaluate HP14x89 and HP14x117 steel H-pile alternatives, weak axis orientation.
6. Assume no reduction in cross-sectional area due to corrosion
7. Lateral pile evaluations for a single pile only, group effects not considered.
8. Separate evaluations completed for each pile section and for expansion and contraction conditions because of the differences in proposed finish grade behind (El. 328.3; expansion case) and in front (El. 322.6; contraction case) of the abutment.
9. Concrete pile collar extending below the bottom of the abutment, if required/present, is not considered.

SOIL PROFILE



Parameter	Expan. Case	Cont. Case	
GS Elev.	El. 328.3	El. 322.3	(see above sketch)
Pile Top Elev.	El. 318.6	El. 318.6	(pile top elev. assumed to be bottom of abutment elev.)
Pile Top Depth	9.7 ft	3.7 ft	(difference between gs elev. and pile top elev.)
Top of Fill Elev.	El. 318.6		(coincident with bottom of abutment elev.)
Top of Glac. Till 1 Elev.	El. 313.6		(5 ft below bottom of abutment)
Top of Glac. Till 2 Elev.	El. 308.6		(10 ft below bottom of abutment)
Top of Bedrock Elev.	El. 293.6		(25 ft below bottom of abutment)
Groundwater Elev.	El. 313.6		(5 ft below bottom of abutment)

Client: Maine Department of Transportation

Date: 12/13/2022

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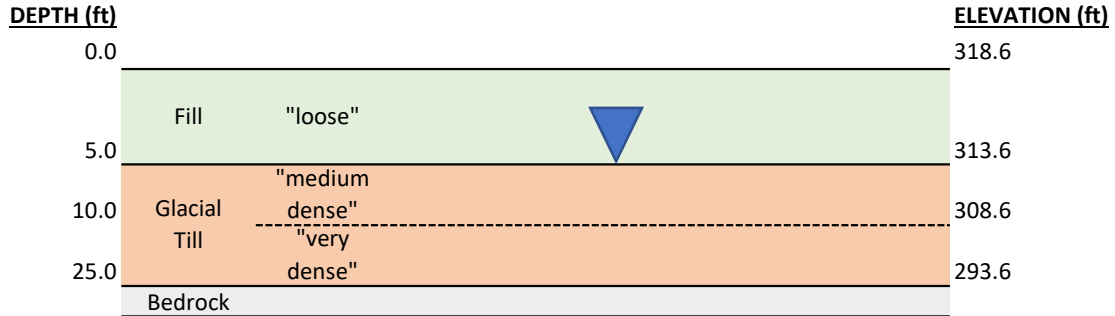
Computed by: BCS

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

Checked by: JLL

SOIL PROFILE (CONT.)

Soil profile below and summarized on the previous sheet shows approximate average thicknesses encountered below the bottom of Abut. 1 (El. 318.6)



SOIL PROPERTIES

Friction angles estimated using SPT N60 values taken from test boring logs and the following correlations:

Eqn. No.	Equation	Reference
E1	$\phi = (12N_{60})^{0.5} + 25$	Dunham (1954)
E2	$\phi = (12N_{60})^{0.5} + 20$	Dunham (1954)
E3	$\phi = (12N_{60})^{0.5} + 15$	Dunham (1954)
E4	$\phi = (20N_{60})^{0.5} + 15$	Ohsaki et al. (1959)
E5	$\phi = 3.5(N_{60})^{0.5} + 20$	Muromachi et al (1974)
E6	$\phi = (15N_{60})^{0.5} + 15 \leq 45 (N_{60} > 5)$	Japan Road Association (1990)
E7	$\phi' = 27.1 + 0.3N_{60} - 0.00054(N_{60})^2$	Peck, Hanson & Thornburn (1974) / Wolff, (1989)

Summarize SPT N60 values for each stratum from test boring logs....

Stratum	SPT N60 (bpf)			
Fill	3	10	19	20
Glac. Till 1	14	24		
Glac. Till 2	93	45	79	60

Calculate friction angles for the **Fill** stratum using the SPT N60 data and correlations presented above....

SPT N60 (bpf)	E1	E2	E3	E4	E5	E6	E7
3	31.0	26.0	21.0	22.7	26.1	23.7	28.0
10	36.0	31.0	26.0	29.1	31.1	27.2	30.0
19	40.1	35.1	30.1	34.5	35.3	31.9	32.6
20	40.5	35.5	30.5	35.0	35.7	32.3	32.9

Fill friction angle for use in lateral pile analyses = 31 deg.

(represents average of the values shown. Underestimate of strength parameters could result in underestimate of moment in the piles)

Calculate friction angles for the **Glacial Till 1** stratum using the SPT N60 data and correlations presented above....

SPT N60 (bpf)	E1	E2	E3	E4	E5	E6	E7
14	38.0	33.0	28.0	31.7	33.1	29.5	31.2
24	42.0	37.0	32.0	36.9	37.1	34.0	34.0

Glacial Till 1 friction angle for use in lateral pile analyses = 34 deg.

(represents average of the values shown. Underestimate of strength parameters could result in underestimate of moment in the piles)

Client: Maine Department of Transportation

Date: 12/13/2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00

Computed by: BCS

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

Checked by: JLL

SOIL PROPERTIES (CONT.)

Calculate friction angles for the **Glacial Till 2** stratum using the SPT N60 data and correlations presented above....

SPT N60 (bpf)	E1	E2	E3	E4	E5	E6	E7
45	48.2	43.2	38.2	45.0	43.5	41.0	39.5
60	51.8	46.8	41.8	49.6	47.1	45.0	43.2
79	55.8	50.8	45.8	54.7	51.1	45.0	47.4
93	58.4	53.4	48.4	58.1	53.8	45.0	50.3

Glacial Till 2 friction angle for use in lateral pile analyses = 48 deg.

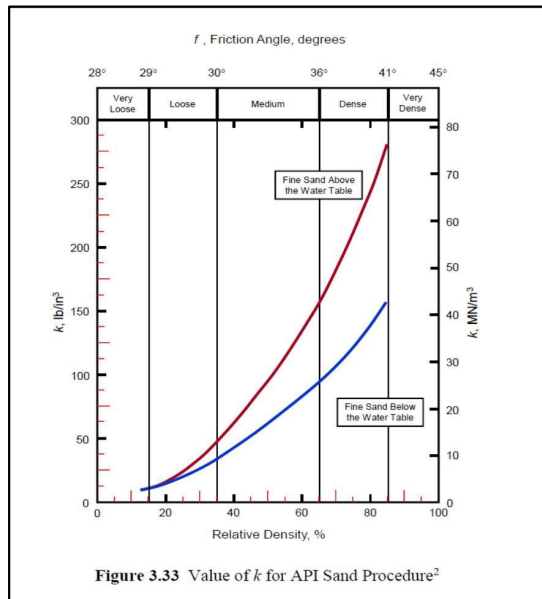
(represents average of the values shown. Underestimate of strength parameters could result in underestimate of moment in the piles)

Estimate horizontal modulus of subgrade reaction (k) using correlation with relative density (D_r) and uncorrected SPT N-values taken from test boring logs....

$$D_r = \left(\frac{N \text{ [#ft]}}{16 + 23\sigma'_{v0} \text{ [ksc]}} \right)^{0.5} \quad \text{(Gibbs+Holtz 1957)}$$

Test Boring No.	Sample No.	Stratum	Samp. Depth (ft)	Samp. Elev. (ft)	Vert. Eff. Stress (psf)	SPT N-value (bpf)	Relative Density (D_r , %)
BB-WPMS-101	1D	Fill	6	321.8	720	13	73
	2D	Fill	11	316.8	1,320	2	25
	3D	Glac. Till 1	16	311.8	1,805	9	50
	4D	Glac. Till 2	21	306.8	2,128	60	100
	5D	Glac. Till 2	26	301.8	2,466	29	81
	6D	Glac. Till 2	31	296.8	2,804	51	100
BB-WPMS-201	1D	Fill	6	322.2	720	12	71
	2D	Fill	11	317.2	1,320	6	44
	3D	Glac. Till 1	16	312.2	1,863	15	64
	5D	Glac. Till 2	26	302.2	2,519	37	91

Estimate horizontal modulus of subgrade reaction (k) using correlation with relative density....



(FBMultiPier, Lpile and Group Technical Manuals)

Client: Maine Department of Transportation

Date: 12/13/2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00

Computed by: BCS

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

Checked by: JLL

SOIL PROPERTIES (CONT.)

Test Boring No.	Sample No.	Stratum	Moisture Condition	Relative Density (D_R , %)	Horizontal Modulus of Subgrade Reaction (k, pci)	
					Dry	Wet
BB-WPMS-101	1D	Fill	Dry	73	205	NA
	2D	Fill	Dry	25	27	NA
	3D	Glac. Till 1	Wet	50	NA	60
	4D	Glac. Till 2	Wet	100	NA	208
	5D	Glac. Till 2	Wet	81	NA	143
	6D	Glac. Till 2	Wet	100	NA	208
BB-WPMS-201	1D	Fill	Dry	71	189	NA
	2D	Fill	Dry	44	74	NA
	3D	Glac. Till 1	Wet	64	NA	92
	5D	Glac. Till 2	Wet	91	NA	176

Summarize horizontal modulus of subgrade reaction values for each stratum....

Stratum	Moisture Condition	Avg. SPT N-value (bpf)	Avg. Relative Density (D_R , %)	Avg. Horiz. Modulus of Subgrade Reaction (k, pci)	
				Dry	Wet
Fill	Dry	8	53	124	NA
Glac. Till 1	Wet	12	57	NA	76
Glac. Till 2	Wet	44	93	NA	184

Summarize LPILE soil property input parameters....

Stratum	Moisture Condition	LPILE Soil Model	Unit Weight (pcf)		Friction Angle (ϕ , deg.)	Soil Modulus (k, pci)
			Total	Bouyant		
Fill	Dry	Sand Reese	120	NA	31	124
Glac. Till 1	Wet	Sand Reese	125	62.6	34	76
Glac. Till 2	Wet	Sand Reese	130	67.6	48	184

Note: Total unit weights shown experientially based and assumed by Haley & Aldrich.

PILE PROPERTIES

Parameter	Steel H-pile Section	
	HP14x89	HP14x117
Yield Strength (F_y , ksi)	50	50
Modulus of Elasticity (E, ksi)	29,000	29,000
Elastic Section Modulus (s_y , in ³)	44.3	59.5
Flange Width (b, in.)	14.7	14.9
Section Depth (d, in.)	13.8	14.2
Flange Thickness (t_f , in.)	0.615	0.805
Web Thickness (t_w , in.)	0.615	0.805
Cross Sectional Area (A_s , in ²)	26.1	34.4

Client: Maine Department of Transportation

Date: 12/13/2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00

Computed by: BCS

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

Checked by: JLL

SUMMARY

Pile Section	Case	Max. Factored Axial Comp. Pile Load ¹ (kip)	Shear Force Resulting in 0.3 in. of Lateral Pile Head Disp. ² (kips)	Max. Moment at Pile Head ² (kip-ft)	Depth Below Pile Head to Fixity ³ (ft)	Total Stress at Pile Head ² (ksi)	Top Unbraced Length Segment ⁴ (L1, ft)	Bottom Unbraced Length Segment ⁴ (L2, ft)
HP14x89	Exp.	360	98.0	214.6	16.4	72.0	2.9	9.6
	Cont.		87.9	171.7	16.0	60.4	2.6	9.6
HP14x117	Exp.		46.0	143.4	18.4	52.8	3.8	10.7
	Cont.		108.9	260.9	17.7	63.0	3.2	10.4
	Cont.		51.2	170.6	19.7	44.9	4.1	11.3

Notes:

- ¹ - Provided by MaineDOT.
- ² - Taken directly from Lpile Summary Reports (attached).
- ³ - Taken as the depth along the pile to the second point of zero lateral deflection as defined by MassDOT.
- ⁴ - Cacluated by linear interpolation (see attached Pile output).

CONCLUSION

Abutment 1 - HP14x89 Cont. Initial

=====
LPILE for Windows, Version 2019-11.009

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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=====
This copy of LPILE is being used by:

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Portland

Serial Number of Security Device: 235486821

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is a violation of the software license agreement.

Files Used for Analysis

Path to file locations:

\\haleyaldrich.com\share\CF\Projects\0205793\Calculations\8. Integral Abutment Pile Design\

Name of input data file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Contraction Case Initial.lp11d

Name of output report file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Contraction Case Initial.lp11o

Name of plot output file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Contraction Case Initial.lp11p

Abutment 1 - HP14x89 Cont. Initial

Name of runtime message file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Contraction Case Initial.lp11r

Date and Time of Analysis

Date: December 20, 2022

Time: 11:20:31

Problem Title

Project Name: Mattagodus Stream Bridge No. 5116

Job Number: 0205793-000

Client: Maine Department of Transportation

Engineer: Bryan C. Steinert, P.E.

Description: Abutment 1 HP14x89 Contraction Case Initial

Program Options and Settings

Computational Options:

- Conventional Analysis

Abutment 1 - HP14x89 Cont. Initial

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
- Input of side resistance moment along pile 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 = 25.000 ft
- Depth of ground surface below top of pile = -3.7000 ft

Abutment 1 - HP14x89 Cont. Initial

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	25.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 25.000000 ft
Pile width = 13.800000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians
Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

Abutment 1 - HP14x89 Cont. Initial

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	=	-3.700000	ft
Distance from top of pile to bottom of layer	=	5.000000	ft
Effective unit weight at top of layer	=	120.000000	pcf
Effective unit weight at bottom of layer	=	120.000000	pcf
Friction angle at top of layer	=	31.000000	deg.
Friction angle at bottom of layer	=	31.000000	deg.
Subgrade k at top of layer	=	124.000000	pci
Subgrade k at bottom of layer	=	124.000000	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	=	62.600000	pcf
Effective unit weight at bottom of layer	=	62.600000	pcf
Friction angle at top of layer	=	38.000000	deg.
Friction angle at bottom of layer	=	38.000000	deg.
Subgrade k at top of layer	=	76.000000	pci
Subgrade k at bottom of layer	=	76.000000	pci

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

Distance from top of pile to top of layer	=	10.000000	ft
Distance from top of pile to bottom of layer	=	25.000000	ft
Effective unit weight at top of layer	=	67.600000	pcf
Effective unit weight at bottom of layer	=	67.600000	pcf
Friction angle at top of layer	=	48.000000	deg.
Friction angle at bottom of layer	=	48.000000	deg.
Subgrade k at top of layer	=	184.000000	pci
Subgrade k at bottom of layer	=	184.000000	pci

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Abutment 1 - HP14x89 Cont. Initial

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.	kpy pci
1	Sand (Reese, et al.)	-3.7000 5.0000	120.0000 120.0000	31.0000 31.0000	124.0000 124.0000
2	Sand (Reese, et al.)	5.0000 10.0000	62.6000 62.6000	38.0000 38.0000	76.0000 76.0000
3	Sand (Reese, et al.)	10.0000 25.0000	67.6000 67.6000	48.0000 48.0000	184.0000 184.0000

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.300000 in	S = 0.0000 in/in	360000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle

Abutment 1 - HP14x89 Cont. Initial

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	=	25.000000 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

Abutment 1 - HP14x89 Cont. Initial

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

Number	Axial Thrust Force kips
----- 1	----- 360.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 360.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in ²	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000444	41.9374124	9448389.	115.7044043	14.8838666	
0.00000888	83.8748248	9448389.	61.5272021	15.8204887	
0.00001332	125.8122373	9448389.	43.4681348	16.7571105	
0.00001775	167.7496497	9448389.	34.4386011	17.6937326	
0.00002219	209.6870621	9448389.	29.0208809	18.6303549	
0.00002663	251.6244745	9448389.	25.4090674	19.5669769	
0.00003107	293.5618869	9448389.	22.8292006	20.5035990	
0.00003551	335.4992993	9448389.	20.8943005	21.4402210	
0.00003995	377.4367118	9448389.	19.3893783	22.3768431	
0.00004439	419.3741242	9448389.	18.1854404	23.3134651	
0.00004882	461.3115366	9448389.	17.2004004	24.2500873	
0.00005326	503.2489490	9448389.	16.3795337	25.1867094	
0.00005770	545.1863614	9448389.	15.6849542	26.1233314	
0.00006214	587.1237739	9448389.	15.0896003	27.0599535	
0.00006658	629.0611863	9448389.	14.5736270	27.9965755	
0.00007102	670.9985987	9448389.	14.1221503	28.9331977	
0.00007546	712.9360111	9448389.	13.7237885	29.8698197	
0.00007989	754.8734235	9448389.	13.3696891	30.8064417	
0.00008433	796.8108359	9448389.	13.0528634	31.7430638	
0.00008877	838.7482484	9448389.	12.7677202	32.6796859	
0.00009321	880.6856608	9448389.	12.5097335	33.6163080	

Abutment 1 - HP14x89 Cont. Initial

0.00009765	922.6230732	9448389.	12.2752002	34.5529301	
0.0001021	964.5604856	9448389.	12.0610611	35.4895521	
0.0001065	1006.	9448389.	11.8647668	36.4261741	
0.0001110	1048.	9448389.	11.6841762	37.3627963	
0.0001154	1090.	9448389.	11.5174771	38.2994183	
0.0001198	1132.	9448389.	11.3631261	39.2360404	
0.0001243	1174.	9448389.	11.2198002	40.1726624	
0.0001287	1216.	9448389.	11.0863588	41.1092845	
0.0001332	1258.	9448389.	10.9618135	42.0459066	
0.0001376	1300.	9448389.	10.8453034	42.9825287	
0.0001420	1342.	9448389.	10.7360751	43.9191507	
0.0001465	1384.	9448389.	10.6334668	44.8557728	
0.0001509	1426.	9448389.	10.5368942	45.7923949	
0.0001554	1468.	9448389.	10.4458401	46.7290169	
0.0001598	1510.	9448389.	10.3598446	47.6656390	
0.0001642	1552.	9448389.	10.2784974	48.6022611	
0.0001687	1594.	9448389.	10.2014317	49.5388831	
0.0001731	1635.	9444750.	10.1289863	50.0000000	Y
0.0001820	1713.	9413633.	9.9992637	50.0000000	Y
0.0001909	1786.	9356714.	9.8872130	50.0000000	Y
0.0001997	1854.	9282029.	9.7898027	50.0000000	Y
0.0002086	1918.	9194494.	9.7048009	50.0000000	Y
0.0002175	1979.	9098100.	9.6303188	50.0000000	Y
0.0002264	2036.	8994945.	9.5650217	50.0000000	Y
0.0002352	2091.	8888017.	9.5074665	50.0000000	Y
0.0002441	2143.	8778986.	9.4565910	50.0000000	Y
0.0002530	2193.	8668920.	9.4115629	50.0000000	Y
0.0002619	2241.	8558522.	9.3717055	50.0000000	Y
0.0002708	2288.	8449149.	9.3362109	50.0000000	Y
0.0002796	2332.	8341141.	9.3045903	50.0000000	Y
0.0002885	2376.	8234786.	9.2764128	50.0000000	Y
0.0002974	2418.	8130322.	9.2512965	50.0000000	Y
0.0003063	2459.	8027951.	9.2289014	50.0000000	Y
0.0003151	2498.	7927838.	9.2089234	50.0000000	Y
0.0003240	2536.	7826452.	9.1901020	50.0000000	Y
0.0003329	2571.	7722229.	9.1716604	50.0000000	Y
0.0003418	2603.	7616213.	9.1535363	50.0000000	Y
0.0003506	2633.	7509294.	9.1356714	50.0000000	Y
0.0003595	2661.	7401927.	9.1181404	50.0000000	Y
0.0003684	2687.	7294361.	9.1010928	50.0000000	Y
0.0003773	2712.	7187800.	9.0842155	50.0000000	Y

Abutment 1 - HP14x89 Cont. Initial

0.0003862	2735.	7082167.	9.0677298	50.0000000	Y
0.0003950	2756.	6977769.	9.0516791	50.0000000	Y
0.0004039	2777.	6875341.	9.0358222	50.0000000	Y
0.0004128	2796.	6773314.	9.0202493	50.0000000	Y
0.0004217	2814.	6673782.	9.0047922	50.0000000	Y
0.0004305	2831.	6575919.	8.9900453	50.0000000	Y
0.0004394	2847.	6479851.	8.9751447	50.0000000	Y
0.0004483	2863.	6385518.	8.9607041	50.0000000	Y
0.0004572	2877.	6293682.	8.9466590	50.0000000	Y
0.0004661	2891.	6203152.	8.9324190	50.0000000	Y
0.0004749	2904.	6114858.	8.9189260	50.0000000	Y
0.0004838	2917.	6028398.	8.9052627	50.0000000	Y
0.0004927	2928.	5943987.	8.8920604	50.0000000	Y
0.0005016	2940.	5861203.	8.8790726	50.0000000	Y
0.0005104	2951.	5780639.	8.8661964	50.0000000	Y
0.0005193	2961.	5701655.	8.8535386	50.0000000	Y
0.0005282	2971.	5624547.	8.8413214	50.0000000	Y
0.0005637	3006.	5333233.	8.7937380	50.0000000	Y
0.0005992	3036.	5067276.	8.7489644	50.0000000	Y
0.0006347	3062.	4824626.	8.7070032	50.0000000	Y
0.0006702	3085.	4602649.	8.6674500	50.0000000	Y
0.0007057	3104.	4398462.	8.6302531	50.0000000	Y
0.0007412	3121.	4210968.	8.5948757	50.0000000	Y
0.0007768	3137.	4038007.	8.5616612	50.0000000	Y
0.0008123	3150.	3878345.	8.5300009	50.0000000	Y
0.0008478	3162.	3730010.	8.5000691	50.0000000	Y
0.0008833	3173.	3592368.	8.4713113	50.0000000	Y
0.0009188	3183.	3464307.	8.4442310	50.0000000	Y
0.0009543	3192.	3344890.	8.4186520	50.0000000	Y
0.0009898	3200.	3232997.	8.3937145	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
-----	-----	-----

1 360.0000000000

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

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

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

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-3.7000	0.00	N.A.	No	0.00	66538.
2	5.0000	7.0114	Yes	No	66538.	189541.
3	10.0000	9.1316	Yes	No	256079.	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

Abutment 1 - HP14x89 Cont. Initial

for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 0.300000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in ²	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3000	-1720256.	45994.	0.00	52752.	9.41E+09	-317.7231	1589.	0.00
0.2500	0.2992	-1583523.	44972.	-5.27E-04	49667.	9.41E+09	-338.0008	3389.	0.00
0.5000	0.2968	-1449286.	43932.	-0.00101	46639.	9.45E+09	-355.5866	3594.	0.00
0.7500	0.2931	-1317753.	42843.	-0.00145	43672.	9.45E+09	-369.9571	3786.	0.00
1.0000	0.2881	-1189097.	41717.	-0.00185	40770.	9.45E+09	-380.5807	3962.	0.00
1.2500	0.2820	-1063459.	40560.	-0.00220	37936.	9.45E+09	-391.0649	4160.	0.00
1.5000	0.2749	-940976.	39350.	-0.00252	35173.	9.45E+09	-415.5860	4535.	0.00
1.7500	0.2669	-821910.	38068.	-0.00280	32487.	9.45E+09	-438.7768	4932.	0.00
2.0000	0.2581	-706512.	36720.	-0.00304	29884.	9.45E+09	-460.3911	5351.	0.00
2.2500	0.2486	-595015.	35309.	-0.00325	27369.	9.45E+09	-480.1965	5794.	0.00
2.5000	0.2386	-487635.	33839.	-0.00342	24947.	9.45E+09	-499.8137	6284.	0.00
2.7500	0.2281	-384587.	32306.	-0.00356	22622.	9.45E+09	-522.0532	6866.	0.00
3.0000	0.2172	-286105.	30709.	-0.00367	20401.	9.45E+09	-542.6860	7495.	0.00
3.2500	0.2061	-192409.	29053.	-0.00374	18287.	9.45E+09	-561.5347	8174.	0.00
3.5000	0.1948	-103702.	27343.	-0.00379	16286.	9.45E+09	-578.4334	8910.	0.00
3.7500	0.1833	-20164.	25585.	-0.00381	14402.	9.45E+09	-593.2293	9707.	0.00
4.0000	0.1719	58041.	23787.	-0.00381	15257.	9.45E+09	-605.7836	10573.	0.00
4.2500	0.1605	130775.	21954.	-0.00378	16897.	9.45E+0			
4.5000	0.1492	197920.	20095.	-0.00372	18412.	9.45E+0			
4.7500	0.1382	259383.	18216.	-0.00365	19798.	9.45E+09	-628.8475	13654.	0.00
5.0000	0.1273	315099.	16424.	-0.00356	21055.	9.45E+09	-565.6089	13325.	0.00
5.2500	0.1168	365615.	14447.	-0.00345	22195.	9.45E+09	-752.6611	19330.	0.00
5.5000	0.1066	409232.	12195.	-0.00333	23178.	9.45E+09	-748.5045	21058.	0.00
5.7500	0.09685	445973.	9961.	-0.00319	24007.	9.45E+09	-741.0227	22955.	0.00
6.0000	0.08748	475891.	7754.	-0.00305	24682.	9.45E+09	-730.2510	25042.	0.00
6.2500	0.07857	499074.	5589.	-0.00289	25205.	9.45E+09	-712.9820	27223.	0.00
6.5000	0.07014	515669.	3541.	-0.00273	25579.	9.45E+09	-652.4244	27907.	0.00
6.7500	0.06219	526215.	1673.	-0.00256	25817.	9.45E+09	-592.7014	28591.	0.00
7.0000	0.05475	531247.	-17.4047	-0.00240	25931.	9.45E+09	-534.2471	29275.	0.00

$$L1 = [20,164 / ((20,164 + 58,041) / (4.0 - 3.75))] + 3.75 = 3.8 \text{ FT}$$

Abutment 1 - HP14x89 Cont. Initial

7.2500	0.04781	531288.	-1535.	-0.00223	25932.	9.45E+09	-477.4522	29959.	0.00
7.5000	0.04138	526849.	-2885.	-0.00206	25832.	9.45E+09	-422.6632	30643.	0.00
7.7500	0.03545	518427.	-4074.	-0.00189	25642.	9.45E+09	-370.1820	31327.	0.00
8.0000	0.03001	506494.	-5110.	-0.00173	25372.	9.45E+09	-320.2665	32011.	0.00
8.2500	0.02506	491506.	-6000.	-0.00157	25034.	9.45E+09	-273.1304	32695.	0.00
8.5000	0.02058	473891.	-6753.	-0.00142	24637.	9.45E+09	-228.9449	33379.	0.00
8.7500	0.01654	454053.	-7378.	-0.00127	24189.	9.45E+09	-187.8397	34063.	0.00
9.0000	0.01294	432369.	-7885.	-0.00113	23700.	9.45E+09	-149.9048	34747.	0.00
9.2500	0.00975	409187.	-8283.	-9.98E-04	23177.	9.45E+09	-115.1919	35431.	0.00
9.5000	0.00695	384828.	-8581.	-8.72E-04	22628.	9.45E+09	-83.7171	36115.	0.00
9.7500	0.00452	359584.	-8790.	-7.54E-04	22058.	9.45E+09	-55.4622	36799.	0.00
10.0000	0.00243	333717.	-8983.	-6.44E-04	21475.	9.45E+09	-73.5458	90749.	0.00
10.2500	6.59E-04	307074.	-9124.	-5.42E-04	20874.	9.45E+09	-20.2983	92405.	0.00
10.5000	-8.21E-04	280143.	-9116.	-4.49E-04	20267.	9.45E+09	25.7348	94061.	0.00
10.7500	-0.00203	253348.	-8980.	-3.64E-04	19662.	9.45E+09	64.8877	95717.	0.00
11.0000	-0.00301	227049.	-8736.	-2.88E-04	19069.	9.45E+09	97.5468	97373.	0.00
11.2500	-0.00376	201551.	-8404.	-2.20E-04	18494.	9.45E+09	124.1395	99029.	0.00
11.5000	-0.00432	177101.	-8000.	-1.60E-04	17942.	9.45E+09	145.1227	100685.	0.00
11.7500	-0.00472	153896.	-7541.	-1.07E-04	17419.	9.45E+09	160.9730	102341.	0.00
12.0000	-0.00497	132088.	-7041.	-6.17E-05	16927.	9.45E+09	172.1773	103997.	0.00
12.2500	-0.00509	111783.	-6514.	-2.30E-05	16469.	9.45E+09	179.2245	105653.	0.00
12.5000	-0.00510	93053.	-5971.	9.51E-06	16046.	9.45E+09	182.5979	107309.	0.00
12.7500	-0.00503	75935.	-5423.	3.63E-05	15660.	9.45E+09	182.7692	108965.	0.00
13.0000	-0.00489	60436.	-4879.	5.80E-05	15311.	9.45E+09	180.1931	110621.	0.00
13.2500	-0.00468	46537.	-4346.	7.50E-05	14997.	9.45E+09	175.3021	112277.	0.00
13.5000	-0.00444	34200.	-3830.	8.78E-05	14719.	9.45E+09	168.5038	113933.	0.00
13.7500	-0.00416	23369.	-3337.	9.69E-05	14474.	9.45E+09	160.1775	115589.	0.00
14.0000	-0.00386	13970.	-2871.	1.03E-04	14262.	9.45E+09	150.6725	117245.	0.00
14.2500	-0.00354	5923.	-2434.	1.06E-04	14081.	9.45E+09	140.3068	118901.	0.00
14.5000	-0.00322	-862.9131	-2030.	1.07E-04	13967.	9.45E+09	129.3663	120557.	0.00
14.7500	-0.00290	-6485.	-1658.	1.06E-04	14094.	9.45E			
15.0000	-0.00259	-11041.	-1321.	1.03E-04	14196.	9.45E			
15.2500	-0.00228	-14633.	-1018.	9.88E-05	14277.	9.45E+09	95.4795	125525.	0.00
15.5000	-0.00199	-17361.	-747.7912	9.37E-05	14339.	9.45E+09	84.4695	127181.	0.00
15.7500	-0.00172	-19322.	-510.3119	8.79E-05	14383.	9.45E+09	73.8501	128837.	0.00
16.0000	-0.00147	-20613.	-303.9417	8.16E-05	14412.	9.45E+09	63.7301	130493.	0.00
16.2500	-0.00123	-21322.	-127.0556	7.49E-05	14428.	9.45E+09	54.1940	132149.	0.00
16.5000	-0.00102	-21537.	22.1921	6.81E-05	14433.	9.45E+09	45.3045	133805.	0.00
16.7500	-8.22E-04	-21336.	145.8055	6.13E-05	14429.	9.45E+09	37.1044	135461.	0.00
17.0000	-6.48E-04	-20794.	245.8908	5.46E-05	14416.	9.45E+09	29.6191	137117.	0.00
17.2500	-4.94E-04	-19979.	324.6068	4.81E-05	14398.	9.45E+09	22.8582	138773.	0.00

$$L2 = [5,923 / ((5,923 + 863) / (14.5 - 14.25))] + 14.25 - 3.8 = 10.7 \text{ FT}$$

Abutment 1 - HP14x89 Cont. Initial

17.5000	-3.59E-04	-18950.	384.1216	4.19E-05	14375.	9.45E+09	16.8183	140429.	0.00
17.7500	-2.42E-04	-17764.	426.5756	3.61E-05	14348.	9.45E+09	11.4844	142085.	0.00
18.0000	-1.43E-04	-16469.	454.0507	3.07E-05	14319.	9.45E+09	6.8323	143741.	0.00
18.2500	-5.84E-05	-15106.	468.5445	2.57E-05	14288.	9.45E+09	2.8302	145397.	0.00
18.5000	1.14E-05	-13713.	471.9508	2.11E-05	14257.	9.45E+09	-0.5594	147053.	0.00
18.7500	6.82E-05	-12320.	466.0436	1.70E-05	14225.	9.45E+09	-3.3787	148709.	0.00
19.0000	1.13E-04	-10954.	452.4670	1.33E-05	14194.	9.45E+09	-5.6724	150365.	0.00
19.2500	1.48E-04	-9634.	432.7279	1.00E-05	14165.	9.45E+09	-7.4870	152021.	0.00
19.5000	1.73E-04	-8379.	408.1927	7.14E-06	14136.	9.45E+09	-8.8698	153677.	0.00
19.7500	1.91E-04	-7200.	380.0871	4.66E-06	14110.	9.45E+09	-9.8673	155333.	0.00
20.0000	2.01E-04	-6108.	349.4983	2.55E-06	14085.	9.45E+09	-10.5252	156989.	0.00
20.2500	2.06E-04	-5109.	317.3800	7.70E-07	14062.	9.45E+09	-10.8870	158645.	0.00
20.5000	2.06E-04	-4206.	284.5584	-7.09E-07	14042.	9.45E+09	-10.9940	160301.	0.00
20.7500	2.02E-04	-3400.	251.7404	-1.92E-06	14024.	9.45E+09	-10.8847	161957.	0.00
21.0000	1.94E-04	-2691.	219.5222	-2.88E-06	14008.	9.45E+09	-10.5941	163613.	0.00
21.2500	1.84E-04	-2077.	188.3998	-3.64E-06	13994.	9.45E+09	-10.1542	165269.	0.00
21.5000	1.72E-04	-1553.	158.7787	-4.22E-06	13982.	9.45E+09	-9.5932	166925.	0.00
21.7500	1.59E-04	-1115.	130.9849	-4.64E-06	13972.	9.45E+09	-8.9360	168581.	0.00
22.0000	1.45E-04	-756.9617	105.2753	-4.94E-06	13964.	9.45E+09	-8.2037	170237.	0.00
22.2500	1.29E-04	-472.5906	81.8484	-5.13E-06	13958.	9.45E+09	-7.4142	171893.	0.00
22.5000	1.14E-04	-254.7852	60.8543	-5.25E-06	13953.	9.45E+09	-6.5819	173549.	0.00
22.7500	9.79E-05	-96.1291	42.4043	-5.30E-06	13949.	9.45E+09	-5.7181	175205.	0.00
23.0000	8.20E-05	11.0968	26.5800	-5.32E-06	13947.	9.45E+09	-4.8315	176861.	0.00
23.2500	6.60E-05	74.8359	13.4411	-5.30E-06	13949.	9.45E+09	-3.9278	178517.	0.00
23.5000	5.01E-05	103.1992	3.0332	-5.28E-06	13950.	9.45E+09	-3.0108	180173.	0.00
23.7500	3.44E-05	104.4296	-4.6065	-5.24E-06	13950.	9.45E+09	-2.0823	181829.	0.00
24.0000	1.87E-05	86.8837	-9.4435	-5.21E-06	13949.	9.45E+09	-1.1424	183485.	0.00
24.2500	3.08E-06	59.0263	-11.4426	-5.19E-06	13949.	9.45E+09	-0.1903	185141.	0.00
24.5000	-1.25E-05	29.4357	-10.5649	-5.17E-06	13948.	9.45E+09	0.7755	186797.	0.00
24.7500	-2.80E-05	6.8141	-6.7668	-5.17E-06	13947.	9.45E+09	1.7566	188453.	0.00
25.0000	-4.35E-05	0.00	0.00	-5.17E-06	13947.	9.45E+09	2.7545	95054.	0.00

DEPTH TO
FIXITY =
18.4 FT

* 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:

Abutment 1 - HP14x89 Cont. Initial

Pile-head deflection = 0.30000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -1720256. inch-lbs
 Maximum shear force = 45994. 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 = 6
 Number of zero deflection points = 3

Summary of Pile-head Responses for Conventional Analyses

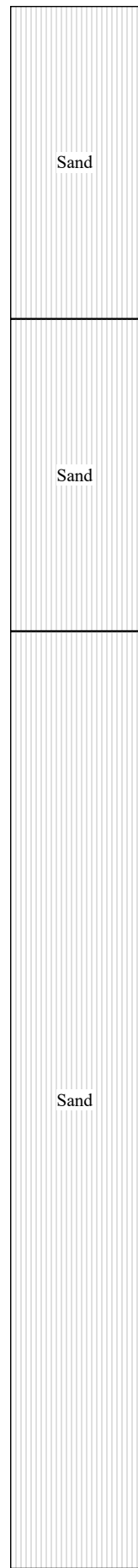
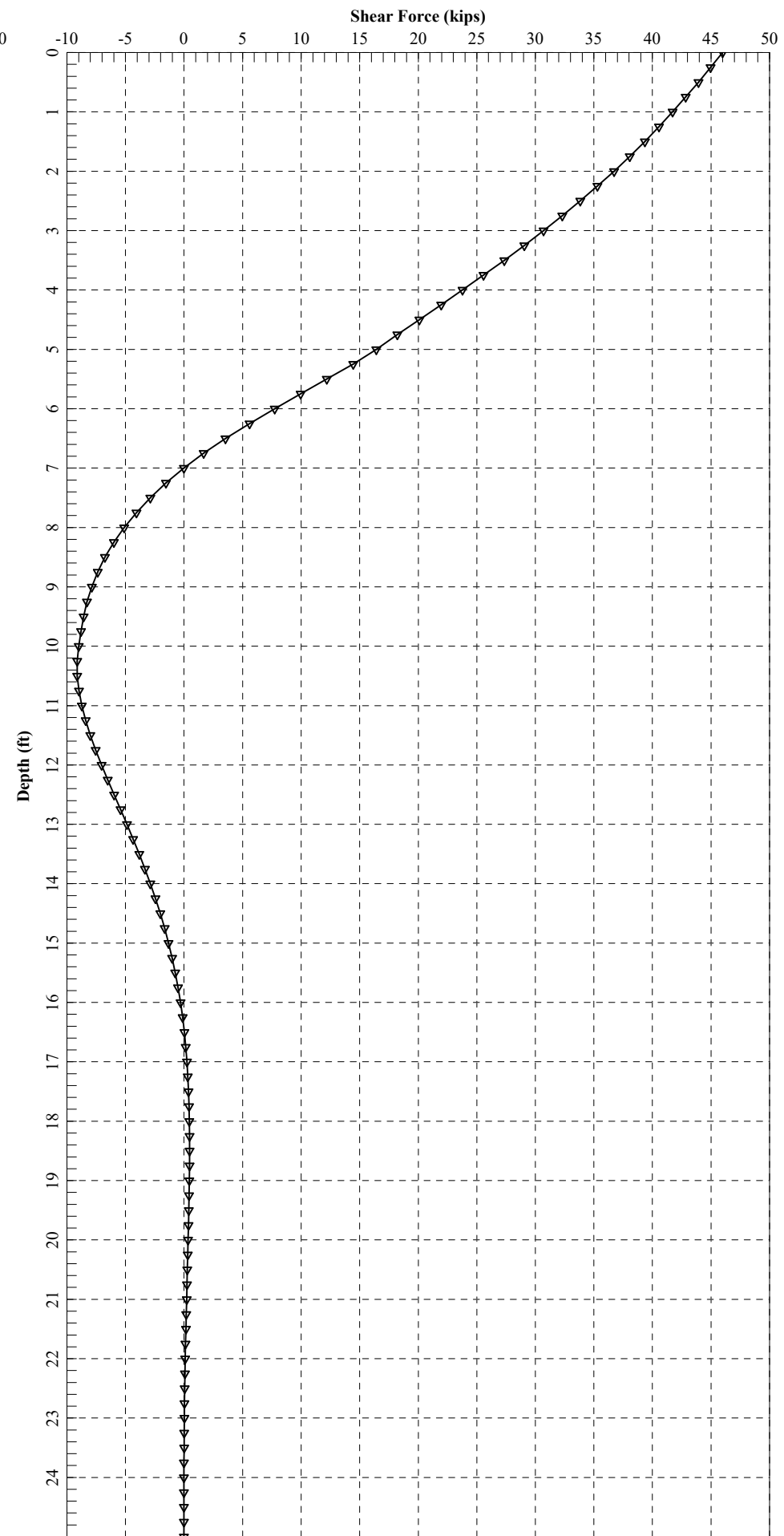
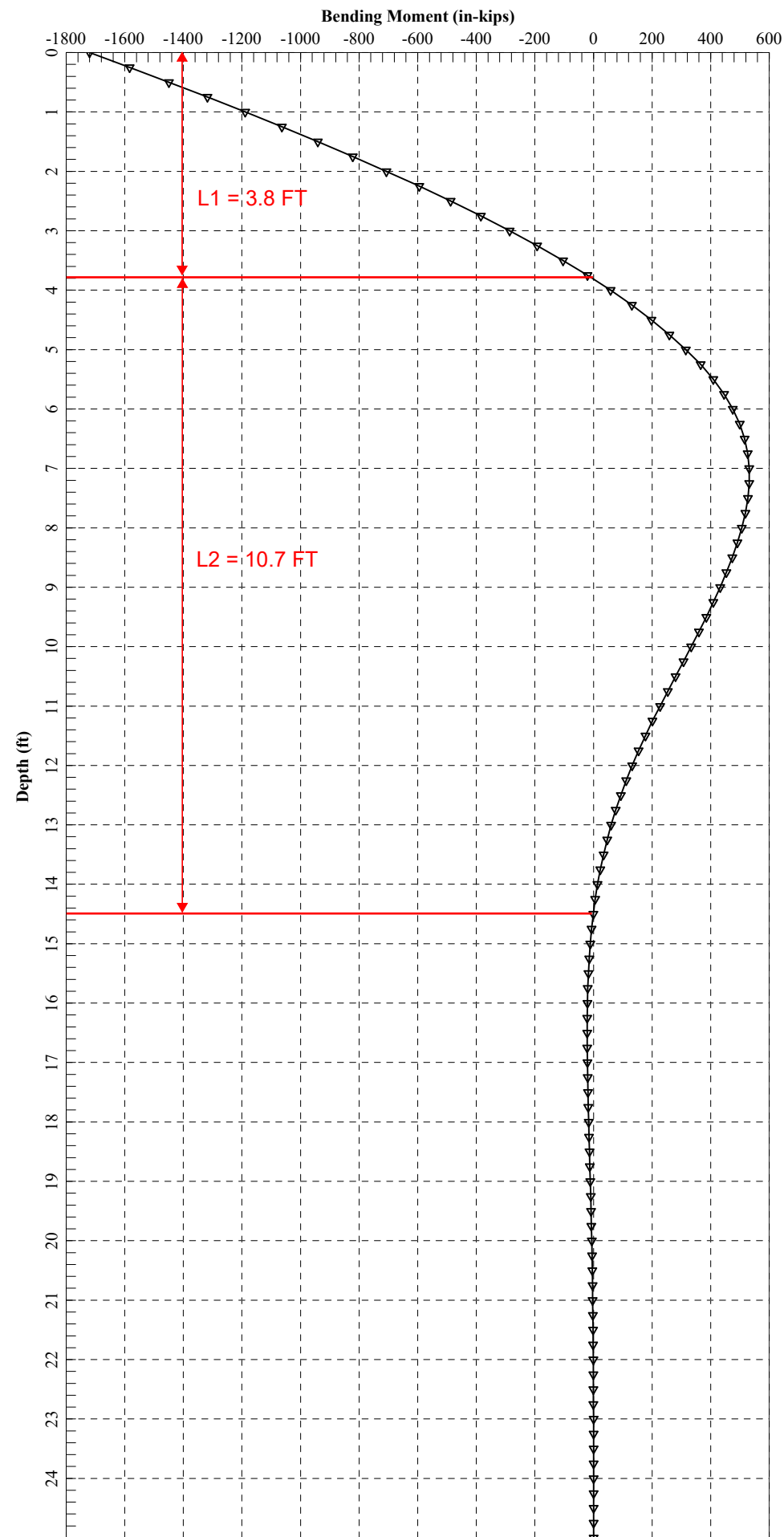
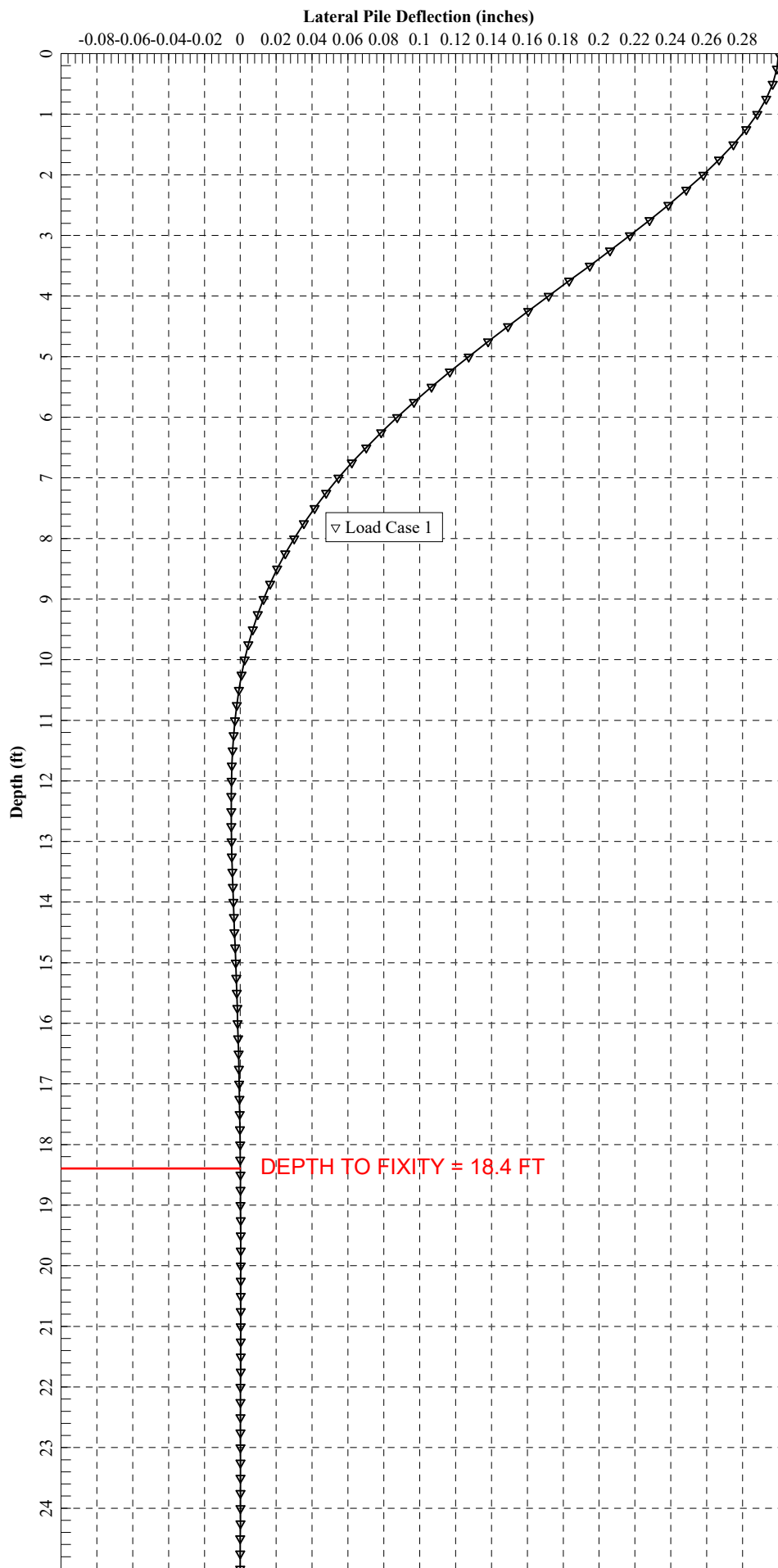
Definitions of Pile-head Loading Conditions:

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	0.3000	S, rad	0.00	360000.	0.3000	0.00	45994.	-1720256.

Maximum pile-head deflection = 0.3000000000 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.



Abutment 1 - HP14x89 Exp Initial

LPILE for Windows, Version 2019-11.009

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:

\\haleyaldrich.com\share\CF\Projects\0205793\Calculations\8. Integral Abutment Pile Design\

Name of input data file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial.lp11d

Name of output report file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial.lp11o

Name of plot output file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial.lp11p

Abutment 1 - HP14x89 Exp Initial

Name of runtime message file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial.lp11r

Date and Time of Analysis

Date: December 20, 2022

Time: 11:13:33

Problem Title

Project Name: Mattagodus Stream Bridge No. 5116

Job Number: 0205793-000

Client: Maine Department of Transportation

Engineer: Bryan C. Steinert, P.E.

Description: Abutment 1 HP14x89 Expansion Case Initial

Program Options and Settings

Computational Options:

- Conventional Analysis

Abutment 1 - HP14x89 Exp Initial

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
- Input of side resistance moment along pile 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 = 25.000 ft
- Depth of ground surface below top of pile = -9.7000 ft

Abutment 1 - HP14x89 Exp Initial

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	25.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile

Length of section = 25.000000 ft

Pile width = 13.800000 in

Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees

= 0.000 radians

Pile Batter Angle = 0.000 degrees

= 0.000 radians

Soil and Rock Layering Information

Abutment 1 - HP14x89 Exp Initial

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	=	-9.700000	ft
Distance from top of pile to bottom of layer	=	5.000000	ft
Effective unit weight at top of layer	=	120.000000	pcf
Effective unit weight at bottom of layer	=	120.000000	pcf
Friction angle at top of layer	=	31.000000	deg.
Friction angle at bottom of layer	=	31.000000	deg.
Subgrade k at top of layer	=	124.000000	pci
Subgrade k at bottom of layer	=	124.000000	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	=	62.600000	pcf
Effective unit weight at bottom of layer	=	62.600000	pcf
Friction angle at top of layer	=	38.000000	deg.
Friction angle at bottom of layer	=	38.000000	deg.
Subgrade k at top of layer	=	76.000000	pci
Subgrade k at bottom of layer	=	76.000000	pci

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

Distance from top of pile to top of layer	=	10.000000	ft
Distance from top of pile to bottom of layer	=	25.000000	ft
Effective unit weight at top of layer	=	67.600000	pcf
Effective unit weight at bottom of layer	=	67.600000	pcf
Friction angle at top of layer	=	48.000000	deg.
Friction angle at bottom of layer	=	48.000000	deg.
Subgrade k at top of layer	=	184.000000	pci
Subgrade k at bottom of layer	=	184.000000	pci

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Abutment 1 - HP14x89 Exp Initial

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.	kpy pci
1	Sand (Reese, et al.)	-9.7000 5.0000	120.0000 120.0000	31.0000 31.0000	124.0000 124.0000
2	Sand (Reese, et al.)	5.0000 10.0000	62.6000 62.6000	38.0000 38.0000	76.0000 76.0000
3	Sand (Reese, et al.)	10.0000 25.0000	67.6000 67.6000	48.0000 48.0000	184.0000 184.0000

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.300000 in	S = 0.0000 in/in	360000.	N.A.	Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

Abutment 1 - HP14x89 Exp Initial

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	=	25.000000 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

Abutment 1 - HP14x89 Exp Initial

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

Number	Axial Thrust Force kips
1	360.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 360.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in ²	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000444	41.9374124	9448389.	115.7044043	14.8838666	
0.00000888	83.8748248	9448389.	61.5272021	15.8204887	
0.00001332	125.8122373	9448389.	43.4681348	16.7571105	
0.00001775	167.7496497	9448389.	34.4386011	17.6937326	
0.00002219	209.6870621	9448389.	29.0208809	18.6303549	
0.00002663	251.6244745	9448389.	25.4090674	19.5669769	
0.00003107	293.5618869	9448389.	22.8292006	20.5035990	
0.00003551	335.4992993	9448389.	20.8943005	21.4402210	
0.00003995	377.4367118	9448389.	19.3893783	22.3768431	
0.00004439	419.3741242	9448389.	18.1854404	23.3134651	
0.00004882	461.3115366	9448389.	17.2004004	24.2500873	
0.00005326	503.2489490	9448389.	16.3795337	25.1867094	
0.00005770	545.1863614	9448389.	15.6849542	26.1233314	
0.00006214	587.1237739	9448389.	15.0896003	27.0599535	
0.00006658	629.0611863	9448389.	14.5736270	27.9965755	
0.00007102	670.9985987	9448389.	14.1221503	28.9331977	
0.00007546	712.9360111	9448389.	13.7237885	29.8698197	
0.00007989	754.8734235	9448389.	13.3696891	30.8064417	
0.00008433	796.8108359	9448389.	13.0528634	31.7430638	
0.00008877	838.7482484	9448389.	12.7677202	32.6796859	
0.00009321	880.6856608	9448389.	12.5097335	33.6163080	

Abutment 1 - HP14x89 Exp Initial

0.00009765	922.6230732	9448389.	12.2752002	34.5529301	
0.0001021	964.5604856	9448389.	12.0610611	35.4895521	
0.0001065	1006.	9448389.	11.8647668	36.4261741	
0.0001110	1048.	9448389.	11.6841762	37.3627963	
0.0001154	1090.	9448389.	11.5174771	38.2994183	
0.0001198	1132.	9448389.	11.3631261	39.2360404	
0.0001243	1174.	9448389.	11.2198002	40.1726624	
0.0001287	1216.	9448389.	11.0863588	41.1092845	
0.0001332	1258.	9448389.	10.9618135	42.0459066	
0.0001376	1300.	9448389.	10.8453034	42.9825287	
0.0001420	1342.	9448389.	10.7360751	43.9191507	
0.0001465	1384.	9448389.	10.6334668	44.8557728	
0.0001509	1426.	9448389.	10.5368942	45.7923949	
0.0001554	1468.	9448389.	10.4458401	46.7290169	
0.0001598	1510.	9448389.	10.3598446	47.6656390	
0.0001642	1552.	9448389.	10.2784974	48.6022611	
0.0001687	1594.	9448389.	10.2014317	49.5388831	
0.0001731	1635.	9444750.	10.1289863	50.0000000	Y
0.0001820	1713.	9413633.	9.9992637	50.0000000	Y
0.0001909	1786.	9356714.	9.8872130	50.0000000	Y
0.0001997	1854.	9282029.	9.7898027	50.0000000	Y
0.0002086	1918.	9194494.	9.7048009	50.0000000	Y
0.0002175	1979.	9098100.	9.6303188	50.0000000	Y
0.0002264	2036.	8994945.	9.5650217	50.0000000	Y
0.0002352	2091.	8888017.	9.5074665	50.0000000	Y
0.0002441	2143.	8778986.	9.4565910	50.0000000	Y
0.0002530	2193.	8668920.	9.4115629	50.0000000	Y
0.0002619	2241.	8558522.	9.3717055	50.0000000	Y
0.0002708	2288.	8449149.	9.3362109	50.0000000	Y
0.0002796	2332.	8341141.	9.3045903	50.0000000	Y
0.0002885	2376.	8234786.	9.2764128	50.0000000	Y
0.0002974	2418.	8130322.	9.2512965	50.0000000	Y
0.0003063	2459.	8027951.	9.2289014	50.0000000	Y
0.0003151	2498.	7927838.	9.2089234	50.0000000	Y
0.0003240	2536.	7826452.	9.1901020	50.0000000	Y
0.0003329	2571.	7722229.	9.1716604	50.0000000	Y
0.0003418	2603.	7616213.	9.1535363	50.0000000	Y
0.0003506	2633.	7509294.	9.1356714	50.0000000	Y
0.0003595	2661.	7401927.	9.1181404	50.0000000	Y
0.0003684	2687.	7294361.	9.1010928	50.0000000	Y
0.0003773	2712.	7187800.	9.0842155	50.0000000	Y

Abutment 1 - HP14x89 Exp Initial

0.0003862	2735.	7082167.	9.0677298	50.0000000	Y
0.0003950	2756.	6977769.	9.0516791	50.0000000	Y
0.0004039	2777.	6875341.	9.0358222	50.0000000	Y
0.0004128	2796.	6773314.	9.0202493	50.0000000	Y
0.0004217	2814.	6673782.	9.0047922	50.0000000	Y
0.0004305	2831.	6575919.	8.9900453	50.0000000	Y
0.0004394	2847.	6479851.	8.9751447	50.0000000	Y
0.0004483	2863.	6385518.	8.9607041	50.0000000	Y
0.0004572	2877.	6293682.	8.9466590	50.0000000	Y
0.0004661	2891.	6203152.	8.9324190	50.0000000	Y
0.0004749	2904.	6114858.	8.9189260	50.0000000	Y
0.0004838	2917.	6028398.	8.9052627	50.0000000	Y
0.0004927	2928.	5943987.	8.8920604	50.0000000	Y
0.0005016	2940.	5861203.	8.8790726	50.0000000	Y
0.0005104	2951.	5780639.	8.8661964	50.0000000	Y
0.0005193	2961.	5701655.	8.8535386	50.0000000	Y
0.0005282	2971.	5624547.	8.8413214	50.0000000	Y
0.0005637	3006.	5333233.	8.7937380	50.0000000	Y
0.0005992	3036.	5067276.	8.7489644	50.0000000	Y
0.0006347	3062.	4824626.	8.7070032	50.0000000	Y
0.0006702	3085.	4602649.	8.6674500	50.0000000	Y
0.0007057	3104.	4398462.	8.6302531	50.0000000	Y
0.0007412	3121.	4210968.	8.5948757	50.0000000	Y
0.0007768	3137.	4038007.	8.5616612	50.0000000	Y
0.0008123	3150.	3878345.	8.5300009	50.0000000	Y
0.0008478	3162.	3730010.	8.5000691	50.0000000	Y
0.0008833	3173.	3592368.	8.4713113	50.0000000	Y
0.0009188	3183.	3464307.	8.4442310	50.0000000	Y
0.0009543	3192.	3344890.	8.4186520	50.0000000	Y
0.0009898	3200.	3232997.	8.3937145	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
----------	-------------------	---------------------------------

1

360.0000000000

Abutment 1 - HP14x89 Exp Initial
3200.

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

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

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

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-9.7000	0.00	N.A.	No	0.00	276935.
2	5.0000	11.9631	Yes	No	276935.	435253.
3	10.0000	12.8622	Yes	No	712188.	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

Abutment 1 - HP14x89 Exp Initial
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 0.300000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3000	-2575709.	97998.	0.00	72048.	7.70E+09	-1306.	6530.	0.00
0.2500	0.2985	-2287375.	93884.	-9.47E-04	65544.	7.70E+09	-1365.	13716.	0.00
0.5000	0.2943	-2010361.	89712.	-0.00173	59296.	9.04E+09	-1416.	14435.	0.00
0.7500	0.2881	-1745373.	85396.	-0.00234	53318.	9.39E+09	-1461.	15214.	0.00
1.0000	0.2803	-1492932.	80954.	-0.00285	47624.	9.45E+09	-1500.	16057.	0.00
1.2500	0.2710	-1253482.	76404.	-0.00329	42222.	9.45E+09	-1533.	16972.	0.00
1.5000	0.2605	-1027401.	71764.	-0.00365	37123.	9.45E+09	-1560.	17966.	0.00
1.7500	0.2491	-815011.	67051.	-0.00395	32332.	9.45E+09	-1582.	19050.	0.00
2.0000	0.2369	-616576.	62282.	-0.00417	27856.	9.45E+09	-1597.	20232.	0.00
2.2500	0.2241	-432307.	57475.	-0.00434	23699.	9.45E+09	-1607.	21515.	0.00
2.5000	0.2108	-262352.	52650.	-0.00445	19865.	9.45E+09	-1610.	22909.	0.00
2.7500	0.1974	-106796.	47825.	-0.00451	16356.	9.45E+09	-1607.	24424.	0.00
3.0000	0.1838	34334.	43018.	-0.00452	14722.	9.45E+09	-1597.	26075.	0.00
3.2500	0.1703	161075.	38249.	-0.00449	17581.	9.45E+09	-1500.	27844.	0.00
3.5000	0.1569	273523.	33535.	-0.00442	20117.	9.45E+09	-1533.	31999.	0.00
3.7500	0.1437	371833.	28895.	-0.00432	22335.	9.45E+09	-1500.	34364.	0.00
4.0000	0.1310	456218.	24345.	-0.00419	24238.	9.45E+09	-1462.	36968.	0.00
4.2500	0.1186	526945.	19902.	-0.00403	25834.	9.45E+09	-1418.	39842.	0.00
4.5000	0.1068	584335.	15582.	-0.00385	27128.	9.45E+09	-1370.	43026.	0.00
4.7500	0.09551	628761.	11400.	-0.00366	28130.	9.45E+09	-1137.	40219.	0.00
5.0000	0.08483	660644.	7640.	-0.00346	28850.	9.45E+09	-1019.	40903.	0.00
5.2500	0.07477	682065.	4405.	-0.00324	29333.	9.45E+09	-906.1990	41587.	0.00
5.5000	0.06537	694077.	1516.	-0.00302	29604.	9.45E+09	-797.9375	42271.	0.00
5.7500	0.05663	697695.	-1040.	-0.00280	29685.	9.45E+09	-695.2058	42955.	0.00
6.0000	0.04855	693893.	-3279.	-0.00258	29600.	9.45E+09	-598.4059	43639.	0.00
6.2500	0.04114	683596.	-5220.	-0.00236	29367.	9.45E+09	-507.8449	44323.	0.00
6.5000	0.03437	667678.	-6879.	-0.00215	29008.	9.45E+09	-423.7407	45007.	0.00
6.7500	0.02824	646961.	-8277.	-0.00194	28541.	9.45E+09	-346.2278	45691.	0.00
7.0000	0.02273	622209.	-9432.	-0.00174	27983.	9.45E+09			0.00

$L1 = [106,796 / ((106,796 + 34,334) / (3.0 - 2.75))] + 2.75 = 2.9 \text{ FT}$

Abutment 1 - HP14x89 Exp Initial

7.2500	0.01781	594127.	-10364.	-0.00155	27349.	9.45E+09	-275.3633	46375.	0.00
7.5000	0.01346	563363.	-11094.	-0.00136	26655.	9.45E+09	-211.1329	47059.	0.00
7.7500	0.00964	530506.	-11641.	-0.00119	25914.	9.45E+09	-153.4574	47743.	0.00
8.0000	0.00633	496086.	-12024.	-0.00103	25138.	9.45E+09	-102.1986	48427.	0.00
8.2500	0.00349	460576.	-12263.	-8.73E-04	24337.	9.45E+09	-57.1654	49111.	0.00
8.5000	0.00109	424393.	-12376.	-7.33E-04	23520.	9.45E+09	-18.1196	49795.	0.00
8.7500	-9.04E-04	387902.	-12380.	-6.04E-04	22697.	9.45E+09	15.2186	50479.	0.00
9.0000	-0.00253	351414.	-12293.	-4.86E-04	21874.	9.45E+09	43.1656	51163.	0.00
9.2500	-0.00382	315195.	-12129.	-3.81E-04	21057.	9.45E+09	66.0692	51847.	0.00
9.5000	-0.00481	279462.	-11903.	-2.86E-04	20251.	9.45E+09	84.3046	52531.	0.00
9.7500	-0.00554	244393.	-11630.	-2.03E-04	19460.	9.45E+09	98.2703	53215.	0.00
10.0000	-0.00603	210123.	-11089.	-1.31E-04	18687.	9.45E+09	262.4039	130493.	0.00
10.2500	-0.00633	178144.	-10277.	-6.92E-05	17966.	9.45E+09	278.6176	132149.	0.00
10.5000	-0.00645	148611.	-9428.	-1.73E-05	17299.	9.45E+09	287.5858	133805.	0.00
10.7500	-0.00643	121615.	-8561.	2.56E-05	16691.	9.45E+09	290.2977	135461.	0.00
11.0000	-0.00629	97190.	-7694.	6.03E-05	16140.	9.45E+09	287.6941	137117.	0.00
11.2500	-0.00607	75321.	-6841.	8.77E-05	15646.	9.45E+09	280.6596	138773.	0.00
11.5000	-0.00577	55952.	-6015.	1.09E-04	15209.	9.45E+09	270.0158	140429.	0.00
11.7500	-0.00542	38994.	-5226.	1.24E-04	14827.	9.45E+09	256.5177	142085.	0.00
12.0000	-0.00503	24332.	-4480.	1.34E-04	14496.	9.45E+09	240.8511	143741.	0.00
12.2500	-0.00461	11828.	-3783.	1.39E-04	14214.	9.45E+09	223.6314	145397.	0.00
12.5000	-0.00419	1334.	-3139.	1.41E-04	13977.	9.45E+09	205.4039	147053.	0.00
12.7500	-0.00377	-7313.	-2551.	1.41E-04	14112.	9.45E+09	186.6455	148709.	0.00
13.0000	-0.00335	-14277.	-2020.	1.37E-04	14269.	9.45E+09	175.1100	150221.	0.00
13.2500	-0.00294	-19726.	-1544.	1.32E-04	14392.	9.45E+09	167.1100	151701.	0.00
13.5000	-0.00256	-23827.	-1124.	1.25E-04	14485.	9.45E+09	130.9809	153677.	0.00
13.7500	-0.00219	-26740.	-757.2265	1.17E-04	14550.	9.45E+09	113.5952	155333.	0.00
14.0000	-0.00186	-28622.	-441.1215	1.08E-04	14593.	9.45E+09	97.1415	156989.	0.00
14.2500	-0.00155	-29620.	-172.7739	9.87E-05	14615.	9.45E+09	81.7569	158645.	0.00
14.5000	-0.00126	-29872.	51.1676	8.93E-05	14621.	9.45E+09	67.5374	160301.	0.00
14.7500	-0.00101	-29506.	234.2876	7.99E-05	14613.	9.45E+09	54.5426	161957.	0.00
15.0000	-7.85E-04	-28639.	380.3023	7.06E-05	14593.	9.45E+09	42.8006	163613.	0.00
15.2500	-5.87E-04	-27377.	492.9718	6.17E-05	14565.	9.45E+09	32.3124	165269.	0.00
15.5000	-4.14E-04	-25815.	576.0251	5.33E-05	14530.	9.45E+09	23.0564	166925.	0.00
15.7500	-2.67E-04	-24036.	633.0979	4.54E-05	14489.	9.45E+09	14.9921	168581.	0.00
16.0000	-1.42E-04	-22114.	667.6822	3.81E-05	14446.	9.45E+09	8.0641	170237.	0.00
16.2500	-3.85E-05	-20112.	683.0864	3.13E-05	14401.	9.45E+09	2.2053	171893.	0.00
16.5000	4.60E-05	-18083.	682.4051	2.53E-05	14355.	9.45E+09	-2.6595	173549.	0.00
16.7500	1.13E-04	-16072.	668.4983	1.99E-05	14310.	9.45E+09	-6.6117	175205.	0.00
17.0000	1.65E-04	-14115.	643.9774	1.51E-05	14266.	9.45E+09	-9.7355	176861.	0.00
17.2500	2.04E-04	-12241.	611.1992	1.09E-05	14223.	9.45E+09	-12.1166	178517.	0.00

$$L2 = [1,334 / ((1,334 + 7,313) / (12.75 - 12.5))] + 12.5 - 2.9 = 9.6 \text{ FT}$$

DEPTH TO
FIXITY =
16.4 FT

Abutment 1 - HP14x89 Exp Initial

17.5000	2.30E-04	-10471.	572.2642	7.28E-06	14183.	9.45E+09	-13.8400	180173.	0.00
17.7500	2.47E-04	-8823.	529.0217	4.22E-06	14146.	9.45E+09	-14.9884	181829.	0.00
18.0000	2.56E-04	-7306.	483.0772	1.65E-06	14112.	9.45E+09	-15.6413	183485.	0.00
18.2500	2.57E-04	-5928.	435.8039	-4.46E-07	14081.	9.45E+09	-15.8741	185141.	0.00
18.5000	2.53E-04	-4691.	388.3573	-2.13E-06	14053.	9.45E+09	-15.7570	186797.	0.00
18.7500	2.44E-04	-3593.	341.6900	-3.45E-06	14028.	9.45E+09	-15.3545	188453.	0.00
19.0000	2.32E-04	-2633.	296.5697	-4.44E-06	14007.	9.45E+09	-14.7257	190109.	0.00
19.2500	2.18E-04	-1804.	253.5963	-5.14E-06	13988.	9.45E+09	-13.9232	191765.	0.00
19.5000	2.02E-04	-1100.	213.2208	-5.60E-06	13972.	9.45E+09	-12.9938	193421.	0.00
19.7500	1.84E-04	-512.8934	175.7625	-5.86E-06	13959.	9.45E+09	-11.9783	195077.	0.00
20.0000	1.66E-04	-33.0946	141.4275	-5.94E-06	13948.	9.45E+09	-10.9117	196733.	0.00
20.2500	1.49E-04	348.5104	110.3250	-5.89E-06	13955.	9.45E+09	-9.8233	198389.	0.00
20.5000	1.31E-04	641.5864	82.4842	-5.74E-06	13962.	9.45E+09	-8.7373	200045.	0.00
20.7500	1.14E-04	855.8069	57.8687	-5.50E-06	13967.	9.45E+09	-7.6731	201701.	0.00
21.0000	9.80E-05	1001.	36.3909	-5.20E-06	13970.	9.45E+09	-6.6454	203357.	0.00
21.2500	8.29E-05	1085.	17.9250	-4.87E-06	13972.	9.45E+09	-5.6652	205013.	0.00
21.5000	6.88E-05	1119.	2.3182	-4.52E-06	13972.	9.45E+09	-4.7394	206669.	0.00
21.7500	5.58E-05	1109.	-10.5991	-4.17E-06	13972.	9.45E+09	-3.8721	208325.	0.00
22.0000	4.38E-05	1064.	-21.0038	-3.82E-06	13971.	9.45E+09	-3.0643	209981.	0.00
22.2500	3.28E-05	991.3107	-29.0726	-3.50E-06	13970.	9.45E+09	-2.3149	211637.	0.00
22.5000	2.28E-05	897.2839	-34.9756	-3.20E-06	13967.	9.45E+09	-1.6204	213293.	0.00
22.7500	1.36E-05	788.3657	-38.8704	-2.93E-06	13965.	9.45E+09	-0.9761	214949.	0.00
23.0000	5.21E-06	670.3918	-40.8986	-2.70E-06	13962.	9.45E+09	-0.3760	216605.	0.00
23.2500	-2.57E-06	548.8043	-41.1820	-2.51E-06	13960.	9.45E+09	0.1870	218261.	0.00
23.5000	-9.83E-06	428.7120	-39.8209	-2.35E-06	13957.	9.45E+09	0.7203	219917.	0.00
23.7500	-1.67E-05	314.9557	-36.8933	-2.23E-06	13954.	9.45E+09	1.2314	221573.	0.00
24.0000	-2.32E-05	212.1743	-32.4544	-2.15E-06	13952.	9.45E+09	1.7278	223229.	0.00
24.2500	-2.96E-05	124.8704	-26.5383	-2.10E-06	13950.	9.45E+09	2.2162	224885.	0.00
24.5000	-3.58E-05	57.4698	-19.1599	-2.07E-06	13949.	9.45E+09	2.7027	226541.	0.00
24.7500	-4.20E-05	14.3739	-10.3180	-2.05E-06	13948.	9.45E+09	3.1919	228197.	0.00
25.0000	-4.81E-05	0.00	0.00	-2.05E-06	13947.	9.45E+09	3.6868	114926.	0.00

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

Abutment 1 - HP14x89 Exp Initial

Pile-head deflection = 0.30000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -2575709. inch-lbs
 Maximum shear force = 97998. 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 = 9
 Number of zero deflection points = 3

Summary of Pile-head Responses for Conventional Analyses

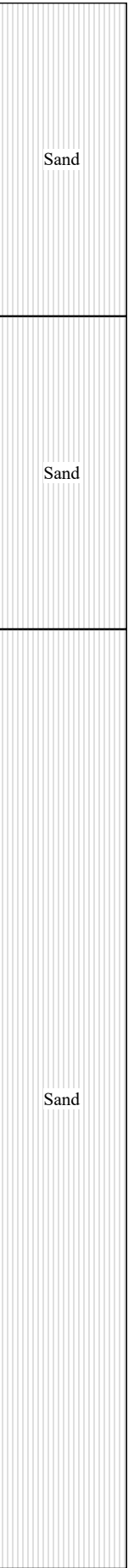
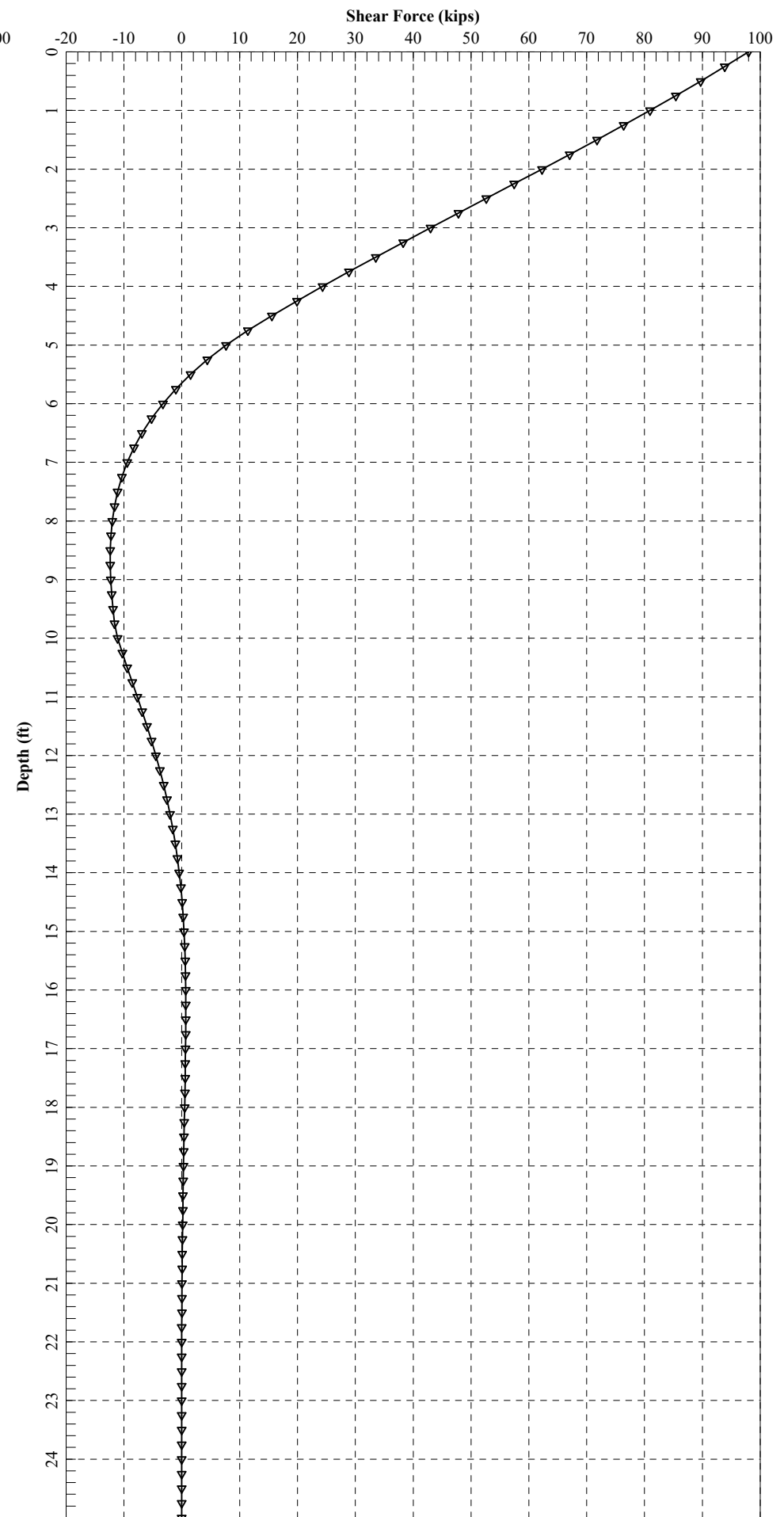
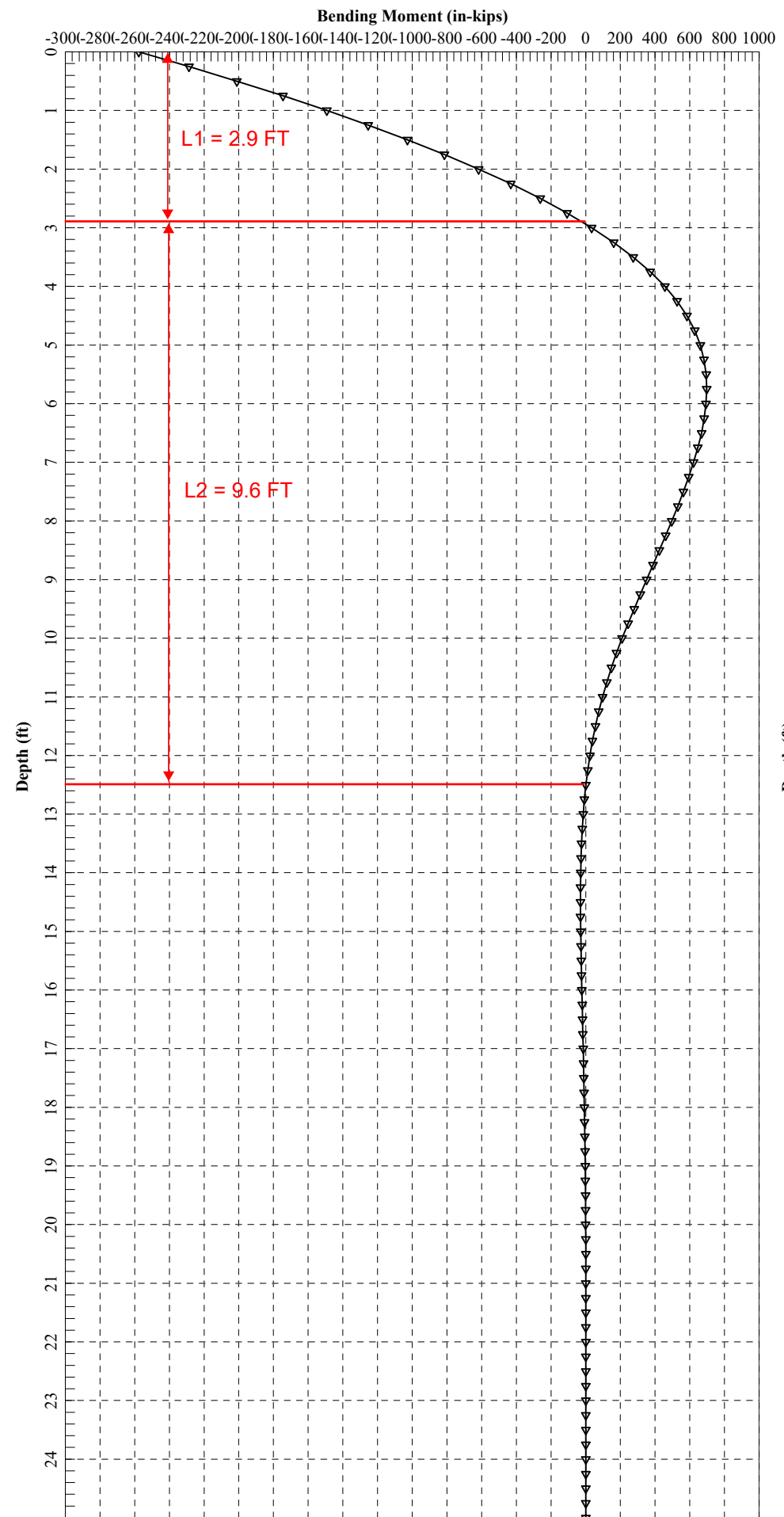
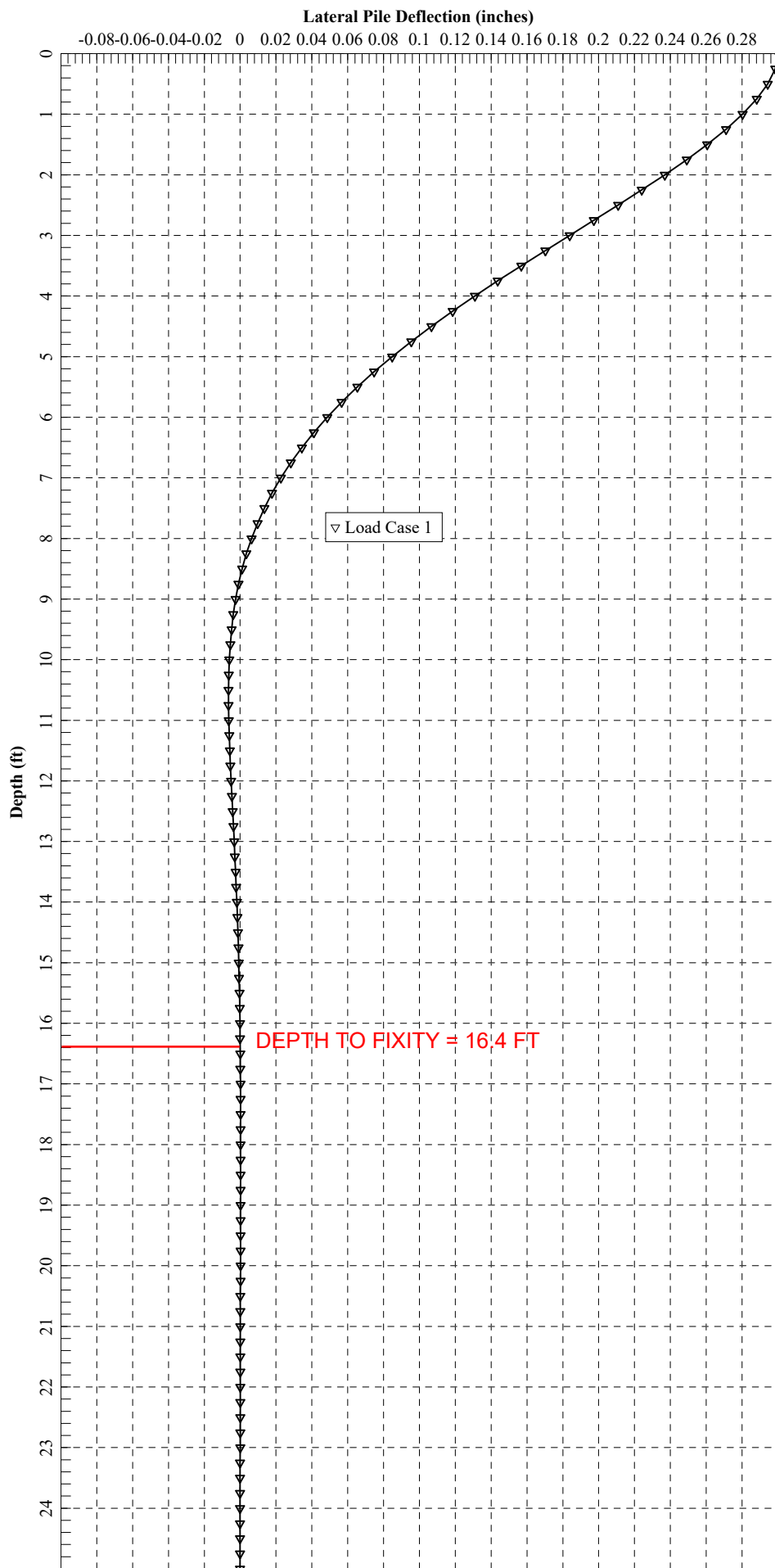
Definitions of Pile-head Loading Conditions:

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	0.3000	S, rad	0.00	360000.	0.3000	0.00	97998.	-2575709.

Maximum pile-head deflection = 0.3000000000 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.



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LPILE for Windows, Version 2019-11.009

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:

\\haleyaldrich.com\share\CF\Projects\0205793\Calculations\8. Integral Abutment Pile Design\

Name of input data file:

2023-0215-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial+Supp.lp11d

Name of output report file:

2023-0215-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial+Supp.lp11o

Name of plot output file:

2023-0215-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial+Supp.lp11p

~~Abutment 1 HP14x117 Con. Initial~~

Name of runtime message file:

2023-0215-HAI-22266 Webster Plt Abutment 1 HP14x89 Expansion Case Initial+Supp.lp11r

Date and Time of Analysis

Date: February 15, 2023

Time: 16:00:44

Problem Title

Project Name: Mattagodus Stream Bridge No. 5116

Job Number: 0205793-000

Client: Maine Department of Transportation

Engineer: Bryan C. Steinert, P.E.

Description: Abutment 1 HP14x89 Expansion Case Initial+Supplemental

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
- Input of side resistance moment along pile 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 = 25.000 ft
- Depth of ground surface below top of pile = -9.7000 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	25.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 25.000000 ft
Pile width = 13.800000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

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	=	-9.700000 ft
Distance from top of pile to bottom of layer	=	5.000000 ft
Effective unit weight at top of layer	=	120.000000 pcf
Effective unit weight at bottom of layer	=	120.000000 pcf
Friction angle at top of layer	=	31.000000 deg.
Friction angle at bottom of layer	=	31.000000 deg.
Subgrade k at top of layer	=	124.000000 pci
Subgrade k at bottom of layer	=	124.000000 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	=	62.600000 pcf
Effective unit weight at bottom of layer	=	62.600000 pcf
Friction angle at top of layer	=	38.000000 deg.
Friction angle at bottom of layer	=	38.000000 deg.
Subgrade k at top of layer	=	76.000000 pci
Subgrade k at bottom of layer	=	76.000000 pci

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

Distance from top of pile to top of layer	=	10.000000 ft
Distance from top of pile to bottom of layer	=	25.000000 ft
Effective unit weight at top of layer	=	67.600000 pcf
Effective unit weight at bottom of layer	=	67.600000 pcf
Friction angle at top of layer	=	48.000000 deg.
Friction angle at bottom of layer	=	48.000000 deg.
Subgrade k at top of layer	=	184.000000 pci
Subgrade k at bottom of layer	=	184.000000 pci

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	kpy pci
1	Sand (Reese, et al.)	-9.7000 5.0000	120.0000 120.0000	31.0000 31.0000	124.0000 124.0000
2	Sand (Reese, et al.)	5.0000 10.0000	62.6000 62.6000	38.0000 38.0000	76.0000 76.0000
3	Sand (Reese, et al.)	10.0000 25.0000	67.6000 67.6000	48.0000 48.0000	184.0000 184.0000

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.300000 in	S = 0.0000 in/in	360000.	N.A.	Yes
2	4	y = 0.300000 in	M = -2060422. in-lbs	360000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

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

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	25.000000 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	360.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 360.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in ²	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000444	41.9374124	9448389.	115.7044043	14.8838666	
0.00000888	83.8748248	9448389.	61.5272021	15.8204887	
0.00001332	125.8122373	9448389.	43.4681348	16.7571105	
0.00001775	167.7496497	9448389.	34.4386011	17.6937326	
0.00002219	209.6870621	9448389.	29.0208809	18.6303549	
0.00002663	251.6244745	9448389.	25.4090674	19.5669769	
0.00003107	293.5618869	9448389.	22.8292006	20.5035990	
0.00003551	335.4992993	9448389.	20.8943005	21.4402210	
0.00003995	377.4367118	9448389.	19.3893783	22.3768431	
0.00004439	419.3741242	9448389.	18.1854404	23.3134651	
0.00004882	461.3115366	9448389.	17.2004004	24.2500873	
0.00005326	503.2489490	9448389.	16.3795337	25.1867094	
0.00005770	545.1863614	9448389.	15.6849542	26.1233314	
0.00006214	587.1237739	9448389.	15.0896003	27.0599535	
0.00006658	629.0611863	9448389.	14.5736270	27.9965755	
0.00007102	670.9985987	9448389.	14.1221503	28.9331977	
0.00007546	712.9360111	9448389.	13.7237885	29.8698197	
0.00007989	754.8734235	9448389.	13.3696891	30.8064417	
0.00008433	796.8108359	9448389.	13.0528634	31.7430638	
0.00008877	838.7482484	9448389.	12.7677202	32.6796859	

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0.00009321	880.6856608	9448389.	12.5097335	33.6163080	
0.00009765	922.6230732	9448389.	12.2752002	34.5529301	
0.0001021	964.5604856	9448389.	12.0610611	35.4895521	
0.0001065	1006.	9448389.	11.8647668	36.4261741	
0.0001110	1048.	9448389.	11.6841762	37.3627963	
0.0001154	1090.	9448389.	11.5174771	38.2994183	
0.0001198	1132.	9448389.	11.3631261	39.2360404	
0.0001243	1174.	9448389.	11.2198002	40.1726624	
0.0001287	1216.	9448389.	11.0863588	41.1092845	
0.0001332	1258.	9448389.	10.9618135	42.0459066	
0.0001376	1300.	9448389.	10.8453034	42.9825287	
0.0001420	1342.	9448389.	10.7360751	43.9191507	
0.0001465	1384.	9448389.	10.6334668	44.8557728	
0.0001509	1426.	9448389.	10.5368942	45.7923949	
0.0001554	1468.	9448389.	10.4458401	46.7290169	
0.0001598	1510.	9448389.	10.3598446	47.6656390	
0.0001642	1552.	9448389.	10.2784974	48.6022611	
0.0001687	1594.	9448389.	10.2014317	49.5388831	
0.0001731	1635.	9444750.	10.1289863	50.0000000	Y
0.0001820	1713.	9413633.	9.9992637	50.0000000	Y
0.0001909	1786.	9356714.	9.8872130	50.0000000	Y
0.0001997	1854.	9282029.	9.7898027	50.0000000	Y
0.0002086	1918.	9194494.	9.7048009	50.0000000	Y
0.0002175	1979.	9098100.	9.6303188	50.0000000	Y
0.0002264	2036.	8994945.	9.5650217	50.0000000	Y
0.0002352	2091.	8888017.	9.5074665	50.0000000	Y
0.0002441	2143.	8778986.	9.4565910	50.0000000	Y
0.0002530	2193.	8668920.	9.4115629	50.0000000	Y
0.0002619	2241.	8558522.	9.3717055	50.0000000	Y
0.0002708	2288.	8449149.	9.3362109	50.0000000	Y
0.0002796	2332.	8341141.	9.3045903	50.0000000	Y
0.0002885	2376.	8234786.	9.2764128	50.0000000	Y
0.0002974	2418.	8130322.	9.2512965	50.0000000	Y
0.0003063	2459.	8027951.	9.2289014	50.0000000	Y
0.0003151	2498.	7927838.	9.2089234	50.0000000	Y
0.0003240	2536.	7826452.	9.1901020	50.0000000	Y
0.0003329	2571.	7722229.	9.1716604	50.0000000	Y
0.0003418	2603.	7616213.	9.1535363	50.0000000	Y
0.0003506	2633.	7509294.	9.1356714	50.0000000	Y
0.0003595	2661.	7401927.	9.1181404	50.0000000	Y
0.0003684	2687.	7294361.	9.1010928	50.0000000	Y

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0.0003773	2712.	7187800.	9.0842155	50.0000000	Y
0.0003862	2735.	7082167.	9.0677298	50.0000000	Y
0.0003950	2756.	6977769.	9.0516791	50.0000000	Y
0.0004039	2777.	6875341.	9.0358222	50.0000000	Y
0.0004128	2796.	6773314.	9.0202493	50.0000000	Y
0.0004217	2814.	6673782.	9.0047922	50.0000000	Y
0.0004305	2831.	6575919.	8.9900453	50.0000000	Y
0.0004394	2847.	6479851.	8.9751447	50.0000000	Y
0.0004483	2863.	6385518.	8.9607041	50.0000000	Y
0.0004572	2877.	6293682.	8.9466590	50.0000000	Y
0.0004661	2891.	6203152.	8.9324190	50.0000000	Y
0.0004749	2904.	6114858.	8.9189260	50.0000000	Y
0.0004838	2917.	6028398.	8.9052627	50.0000000	Y
0.0004927	2928.	5943987.	8.8920604	50.0000000	Y
0.0005016	2940.	5861203.	8.8790726	50.0000000	Y
0.0005104	2951.	5780639.	8.8661964	50.0000000	Y
0.0005193	2961.	5701655.	8.8535386	50.0000000	Y
0.0005282	2971.	5624547.	8.8413214	50.0000000	Y
0.0005637	3006.	5333233.	8.7937380	50.0000000	Y
0.0005992	3036.	5067276.	8.7489644	50.0000000	Y
0.0006347	3062.	4824626.	8.7070032	50.0000000	Y
0.0006702	3085.	4602649.	8.6674500	50.0000000	Y
0.0007057	3104.	4398462.	8.6302531	50.0000000	Y
0.0007412	3121.	4210968.	8.5948757	50.0000000	Y
0.0007768	3137.	4038007.	8.5616612	50.0000000	Y
0.0008123	3150.	3878345.	8.5300009	50.0000000	Y
0.0008478	3162.	3730010.	8.5000691	50.0000000	Y
0.0008833	3173.	3592368.	8.4713113	50.0000000	Y
0.0009188	3183.	3464307.	8.4442310	50.0000000	Y
0.0009543	3192.	3344890.	8.4186520	50.0000000	Y
0.0009898	3200.	3232997.	8.3937145	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
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~~Abutment 1 - HP14x117 Con. Initial~~

1	360.0000000000	3200.
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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	-9.7000	0.00	N.A.	No	0.00	276935.
2	5.0000	11.9631	Yes	No	276935.	435253.
3	10.0000	12.8622	Yes	No	712188.	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 = 0.300000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3000	-2575709.	97998.	0.00	72048.	7.70E+09	-1306.	6530.	0.00
0.2500	0.2985	-2287375.	93884.	-9.47E-04	65544.	7.70E+09	-1365.	13716.	0.00
0.5000	0.2943	-2010361.	89712.	-0.00173	59296.	9.04E+09	-1416.	14435.	0.00
0.7500	0.2881	-1745373.	85396.	-0.00234	53318.	9.39E+09	-1461.	15214.	0.00
1.0000	0.2803	-1492932.	80954.	-0.00285	47624.	9.45E+09	-1500.	16057.	0.00
1.2500	0.2710	-1253482.	76404.	-0.00329	42222.	9.45E+09	-1533.	16972.	0.00
1.5000	0.2605	-1027401.	71764.	-0.00365	37123.	9.45E+09	-1560.	17966.	0.00
1.7500	0.2491	-815011.	67051.	-0.00395	32332.	9.45E+09	-1582.	19050.	0.00
2.0000	0.2369	-616576.	62282.	-0.00417	27856.	9.45E+09	-1597.	20232.	0.00
2.2500	0.2241	-432307.	57475.	-0.00434	23699.	9.45E+09	-1607.	21515.	0.00
2.5000	0.2108	-262352.	52650.	-0.00445	19865.	9.45E+09	-1610.	22909.	0.00
2.7500	0.1974	-106796.	47825.	-0.00451	16356.	9.45E+09	-1607.	24424.	0.00
3.0000	0.1838	34334.	43018.	-0.00452	14722.	9.45E+09	-1597.	26075.	0.00
3.2500	0.1703	161075.	38249.	-0.00449					
3.5000	0.1569	273523.	33535.	-0.00442					
3.7500	0.1437	371833.	28895.	-0.00432	22335.	9.45E+09	-1533.	31999.	0.00
4.0000	0.1310	456218.	24345.	-0.00419	24238.	9.45E+09	-1500.	34364.	0.00
4.2500	0.1186	526945.	19902.	-0.00403	25834.	9.45E+09	-1462.	36968.	0.00
4.5000	0.1068	584335.	15582.	-0.00385	27128.	9.45E+09	-1418.	39842.	0.00
4.7500	0.09551	628761.	11400.	-0.00366	28130.	9.45E+09	-1370.	43026.	0.00
5.0000	0.08483	660644.	7640.	-0.00346	28850.	9.45E+09	-1137.	40219.	0.00
5.2500	0.07477	682065.	4405.	-0.00324	29333.	9.45E+09	-1019.	40903.	0.00
5.5000	0.06537	694077.	1516.	-0.00302	29604.	9.45E+09	-906.1990	41587.	0.00
5.7500	0.05663	697695.	-1040.	-0.00280	29685.	9.45E+09	-797.9375	42271.	0.00
6.0000	0.04855	693893.	-3279.	-0.00258	29600.	9.45E+09	-695.2058	42955.	0.00
6.2500	0.04114	683596.	-5220.	-0.00236	29367.	9.45E+09	-598.4059	43639.	0.00
6.5000	0.03437	667678.	-6879.	-0.00215	29008.	9.45E+09	-507.8449	44323.	0.00
6.7500	0.02824	646961.	-8277.	-0.00194	28541.	9.45E+09	-423.7407	45007.	0.00

$$L1 = [106,796 / ((106,796 + 34,334) / (3.0 - 2.75))] + 2.75 = 2.9 \text{ FT}$$

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7.0000	0.02273	622209.	-9432.	-0.00174	27983.	9.45E+09	-346.2278	45691.	0.00
7.2500	0.01781	594127.	-10364.	-0.00155	27349.	9.45E+09	-275.3633	46375.	0.00
7.5000	0.01346	563363.	-11094.	-0.00136	26655.	9.45E+09	-211.1329	47059.	0.00
7.7500	0.00964	530506.	-11641.	-0.00119	25914.	9.45E+09	-153.4574	47743.	0.00
8.0000	0.00633	496086.	-12024.	-0.00103	25138.	9.45E+09	-102.1986	48427.	0.00
8.2500	0.00349	460576.	-12263.	-8.73E-04	24337.	9.45E+09	-57.1654	49111.	0.00
8.5000	0.00109	424393.	-12376.	-7.33E-04	23520.	9.45E+09	-18.1196	49795.	0.00
8.7500	-9.04E-04	387902.	-12380.	-6.04E-04	22697.	9.45E+09	15.2186	50479.	0.00
9.0000	-0.00253	351414.	-12293.	-4.86E-04	21874.	9.45E+09	43.1656	51163.	0.00
9.2500	-0.00382	315195.	-12129.	-3.81E-04	21057.	9.45E+09	66.0692	51847.	0.00
9.5000	-0.00481	279462.	-11903.	-2.86E-04	20251.	9.45E+09	84.3046	52531.	0.00
9.7500	-0.00554	244393.	-11630.	-2.03E-04	19460.	9.45E+09	98.2703	53215.	0.00
10.0000	-0.00603	210123.	-11089.	-1.31E-04	18687.	9.45E+09	262.4039	130493.	0.00
10.2500	-0.00633	178144.	-10277.	-6.92E-05	17966.	9.45E+09	278.6176	132149.	0.00
10.5000	-0.00645	148611.	-9428.	-1.73E-05	17299.	9.45E+09	287.5858	133805.	0.00
10.7500	-0.00643	121615.	-8561.	2.56E-05	16691.	9.45E+09	290.2977	135461.	0.00
11.0000	-0.00629	97190.	-7694.	6.03E-05	16140.	9.45E+09	287.6941	137117.	0.00
11.2500	-0.00607	75321.	-6841.	8.77E-05	15646.	9.45E+09	280.6596	138773.	0.00
11.5000	-0.00577	55952.	-6015.	1.09E-04	15209.	9.45E+09	270.0158	140429.	0.00
11.7500	-0.00542	38994.	-5226.	1.24E-04	14827.	9.45E+09	256.5177	142085.	0.00
12.0000	-0.00503	24332.	-4480.	1.34E-04	14496.	9.45E+09	240.8511	143741.	0.00
12.2500	-0.00461	11828.	-3783.	1.39E-04	14214.	9.45E+09	223.6314	145397.	0.00
12.5000	-0.00419	1334.	-3139.	1.41E-04	13977.	9.45E+09	205.4039	147053.	0.00
12.7500	-0.00377	-7313.	-2551.	1.41E-04	14112.	9.45E+09	186.6455	148709.	0.00
13.0000	-0.00335	-14277.	-2020.	1.37E-04					
13.2500	-0.00294	-19726.	-1544.	1.32E-04					
13.5000	-0.00256	-23827.	-1124.	1.25E-04	14485.	9.45E+09	130.9809	153677.	0.00
13.7500	-0.00219	-26740.	-757.2265	1.17E-04	14550.	9.45E+09	113.5952	155333.	0.00
14.0000	-0.00186	-28622.	-441.1215	1.08E-04	14593.	9.45E+09	97.1415	156989.	0.00
14.2500	-0.00155	-29620.	-172.7739	9.87E-05	14615.	9.45E+09	81.7569	158645.	0.00
14.5000	-0.00126	-29872.	51.1676	8.93E-05	14621.	9.45E+09	67.5374	160301.	0.00
14.7500	-0.00101	-29506.	234.2876	7.99E-05	14613.	9.45E+09	54.5426	161957.	0.00
15.0000	-7.85E-04	-28639.	380.3023	7.06E-05	14593.	9.45E+09	42.8006	163613.	0.00
15.2500	-5.87E-04	-27377.	492.9718	6.17E-05	14565.	9.45E+09	32.3124	165269.	0.00
15.5000	-4.14E-04	-25815.	576.0251	5.33E-05	14530.	9.45E+09	23.0564	166925.	0.00
15.7500	-2.67E-04	-24036.	633.0979	4.54E-05	14489.	9.45E+09	14.9921	168581.	0.00
16.0000	-1.42E-04	-22114.	667.6822	3.81E-05	14446.	9.45E+09	8.0641	170237.	0.00
16.2500	-3.85E-05	-20112.	683.0864	3.13E-05	14401.	9.45E+09	2.2053	171893.	0.00
16.5000	4.60E-05	-18083.	682.4051	2.53E-05	14355.	9.45E+09	-2.6595	173549.	0.00
16.7500	1.13E-04	-16072.	668.4983	1.99E-05	14310.	9.45E+09	-6.61		
17.0000	1.65E-04	-14115.	643.9774	1.51E-05	14266.	9.45E+09	-9.73		

$L2 = [1,334 / ((1,334 + 7,313) / (12.75 - 12.5))] + 12.5 - 2.9 = 9.6 \text{ FT}$

DEPTH TO FIXITY = 16.4 FT

~~Abutment 1 HP14x117 Con. Initial~~

17.2500	2.04E-04	-12241.	611.1992	1.09E-05	14223.	9.45E+09	-12.1166	178517.	0.00
17.5000	2.30E-04	-10471.	572.2642	7.28E-06	14183.	9.45E+09	-13.8400	180173.	0.00
17.7500	2.47E-04	-8823.	529.0217	4.22E-06	14146.	9.45E+09	-14.9884	181829.	0.00
18.0000	2.56E-04	-7306.	483.0772	1.65E-06	14112.	9.45E+09	-15.6413	183485.	0.00
18.2500	2.57E-04	-5928.	435.8039	-4.46E-07	14081.	9.45E+09	-15.8741	185141.	0.00
18.5000	2.53E-04	-4691.	388.3573	-2.13E-06	14053.	9.45E+09	-15.7570	186797.	0.00
18.7500	2.44E-04	-3593.	341.6900	-3.45E-06	14028.	9.45E+09	-15.3545	188453.	0.00
19.0000	2.32E-04	-2633.	296.5697	-4.44E-06	14007.	9.45E+09	-14.7257	190109.	0.00
19.2500	2.18E-04	-1804.	253.5963	-5.14E-06	13988.	9.45E+09	-13.9232	191765.	0.00
19.5000	2.02E-04	-1100.	213.2208	-5.60E-06	13972.	9.45E+09	-12.9938	193421.	0.00
19.7500	1.84E-04	-512.8934	175.7625	-5.86E-06	13959.	9.45E+09	-11.9783	195077.	0.00
20.0000	1.66E-04	-33.0946	141.4275	-5.94E-06	13948.	9.45E+09	-10.9117	196733.	0.00
20.2500	1.49E-04	348.5104	110.3250	-5.89E-06	13955.	9.45E+09	-9.8233	198389.	0.00
20.5000	1.31E-04	641.5864	82.4842	-5.74E-06	13962.	9.45E+09	-8.7373	200045.	0.00
20.7500	1.14E-04	855.8069	57.8687	-5.50E-06	13967.	9.45E+09	-7.6731	201701.	0.00
21.0000	9.80E-05	1001.	36.3909	-5.20E-06	13970.	9.45E+09	-6.6454	203357.	0.00
21.2500	8.29E-05	1085.	17.9250	-4.87E-06	13972.	9.45E+09	-5.6652	205013.	0.00
21.5000	6.88E-05	1119.	2.3182	-4.52E-06	13972.	9.45E+09	-4.7394	206669.	0.00
21.7500	5.58E-05	1109.	-10.5991	-4.17E-06	13972.	9.45E+09	-3.8721	208325.	0.00
22.0000	4.38E-05	1064.	-21.0038	-3.82E-06	13971.	9.45E+09	-3.0643	209981.	0.00
22.2500	3.28E-05	991.3107	-29.0726	-3.50E-06	13970.	9.45E+09	-2.3149	211637.	0.00
22.5000	2.28E-05	897.2839	-34.9756	-3.20E-06	13967.	9.45E+09	-1.6204	213293.	0.00
22.7500	1.36E-05	788.3657	-38.8704	-2.93E-06	13965.	9.45E+09	-0.9761	214949.	0.00
23.0000	5.21E-06	670.3918	-40.8986	-2.70E-06	13962.	9.45E+09	-0.3760	216605.	0.00
23.2500	-2.57E-06	548.8043	-41.1820	-2.51E-06	13960.	9.45E+09	0.1870	218261.	0.00
23.5000	-9.83E-06	428.7120	-39.8209	-2.35E-06	13957.	9.45E+09	0.7203	219917.	0.00
23.7500	-1.67E-05	314.9557	-36.8933	-2.23E-06	13954.	9.45E+09	1.2314	221573.	0.00
24.0000	-2.32E-05	212.1743	-32.4544	-2.15E-06	13952.	9.45E+09	1.7278	223229.	0.00
24.2500	-2.96E-05	124.8704	-26.5383	-2.10E-06	13950.	9.45E+09	2.2162	224885.	0.00
24.5000	-3.58E-05	57.4698	-19.1599	-2.07E-06	13949.	9.45E+09	2.7027	226541.	0.00
24.7500	-4.20E-05	14.3739	-10.3180	-2.05E-06	13948.	9.45E+09	3.1919	228197.	0.00
25.0000	-4.81E-05	0.00	0.00	-2.05E-06	13947.	9.45E+09	3.6868	114926.	0.00

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

~~Abutment 1 - HP14x117 Con. Initial~~

Output Summary for Load Case No. 1:

Pile-head deflection = 0.30000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -2575709. inch-lbs
 Maximum shear force = 97998. 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 = 9
 Number of zero deflection points = 3

 Computed Values of Pile Loading and Deflection
 for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Moment (Loading Type 4)

Displacement of pile head = 0.300000 inches
 Moment at pile head = -2060421.5 in-lbs
 Axial load at pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3000	-2060422.	87897.	-0.00152	60425.	8.95E+09	-1306.	6530.	0.00
0.2500	0.2944	-1800588.	83909.	-0.00217	54564.	8.95E+09	-1352.	13782.	0.00
0.5000	0.2870	-1552274.	79791.	-0.00272	48962.	9.45E+09	-1393.	14564.	0.00
0.7500	0.2781	-1315966.	75559.	-0.00318	43632.	9.45E+09	-1428.	15409.	0.00
1.0000	0.2679	-1092060.	71230.	-0.00356	38581.	9.45E+09	-1458.	16325.	0.00
1.2500	0.2567	-880901.	66820.	-0.00387	33818.	9.45E+09	-1482.	17320.	0.00
1.5000	0.2447	-682778.	62345.	-0.00412	29349.	9.45E+09	-1501.	18406.	0.00
1.7500	0.2320	-497932.	57821.	-0.00431	25179.	9.45E+09	-1515.	19587.	0.00
2.0000	0.2188	-326549.	53265.	-0.00444	21313.	9.45E+09	-1522.	20869.	0.00
2.2500	0.2054	-168754.	48696.	-0.00452	17754.	9.45E+09	-1524.	22262.	0.00
2.5000	0.1917	-24617.	44130.	-0.00455	14503.	9.45E+09	-1520.	23777.	0.00
2.7500	0.1781	105851.	39587.	-0.00453	16335.	9.45E+09	-1510.	25428.	0.00
3.0000	0.1645	222697.	35082.	-0.00448	18071.	9.45E+09	-1492.	27221.	0.00
3.2500	0.1512	326025.	30634.	-0.00440	19707.	9.45E+09	-1469.	29056.	0.00

L1 = [24,617 / ((24,617 + 105,851) / (2.75 - 2.5))] + 2.5 = 2.55 FT

~~Abutment 1 - HP14x117 Con. Initial~~

3.5000	0.1382	415995.	26260.	-0.00428	23331.	9.45E+09	-1445.	31365.	0.00
3.7500	0.1255	492821.	21975.	-0.00413	25064.	9.45E+09	-1412.	33742.	0.00
4.0000	0.1134	556771.	17796.	-0.00397	26506.	9.45E+09	-1374.	36363.	0.00
4.2500	0.1017	608163.	13737.	-0.00378	27666.	9.45E+09	-1331.	39262.	0.00
4.5000	0.09068	647363.	9814.	-0.00358	28550.	9.45E+09	-1284.	42479.	0.00
4.7500	0.08024	674786.	6040.	-0.00337	29169.	9.45E+09	-1232.	46065.	0.00
5.0000	0.07045	690888.	2775.	-0.00316	29532.	9.45E+09	-944.4474	40219.	0.00
5.2500	0.06131	698252.	104.6374	-0.00293	29698.	9.45E+09	-835.9310	40903.	0.00
5.5000	0.05284	697855.	-2248.	-0.00271	29689.	9.45E+09	-732.4682	41587.	0.00
5.7500	0.04503	690625.	-4298.	-0.00249	29526.	9.45E+09	-634.5086	42271.	0.00
6.0000	0.03788	677448.	-6064.	-0.00228	29229.	9.45E+09	-542.4083	42955.	0.00
6.2500	0.03138	659158.	-7562.	-0.00206	28816.	9.45E+09	-456.4347	43639.	0.00
6.5000	0.02550	636533.	-8812.	-0.00186	28306.	9.45E+09	-376.7717	44323.	0.00
6.7500	0.02023	610299.	-9832.	-0.00166	27714.	9.45E+09	-303.5255	45007.	0.00
7.0000	0.01554	581124.	-10643.	-0.00147	27056.	9.45E+09	-236.7302	45691.	0.00
7.2500	0.01141	549619.	-11262.	-0.00129	26345.	9.45E+09	-176.3539	46375.	0.00
7.5000	0.00780	516339.	-11710.	-0.00112	25594.	9.45E+09	-122.3044	47059.	0.00
7.7500	0.00468	481780.	-12005.	-9.63E-04	24815.	9.45E+09	-74.4354	47743.	0.00
8.0000	0.00202	446387.	-12166.	-8.16E-04	24017.	9.45E+09	-32.5518	48427.	0.00
8.2500	-2.19E-04	410548.	-12209.	-6.80E-04	23208.	9.45E+09	3.5843	49111.	0.00
8.5000	-0.00206	374600.	-12153.	-5.55E-04	22397.	9.45E+09	34.2487	49795.	0.00
8.7500	-0.00355	338831.	-12012.	-4.42E-04	21590.	9.45E+09	59.7502	50479.	0.00
9.0000	-0.00472	303485.	-11801.	-3.40E-04	20793.	9.45E+09	80.4256	51163.	0.00
9.2500	-0.00559	268758.	-11536.	-2.49E-04	20010.	9.45E+09	96.6362	51847.	0.00
9.5000	-0.00621	234809.	-11228.	-1.69E-04	19244.	9.45E+09	108.7635	52531.	0.00
9.7500	-0.00661	201758.	-10889.	-1.00E-04	18498.	9.45E+09	117.2058	53215.	0.00
10.0000	-0.00681	169693.	-10268.	-4.10E-05	17775.	9.45E+09	296.2787	130493.	0.00
10.2500	-0.00685	140236.	-9371.	8.17E-06	17111.	9.45E+09	301.9009	132149.	0.00
10.5000	-0.00676	113448.	-8466.	4.84E-05	16506.	9.45E+09	301.6118	133805.	0.00
10.7500	-0.00656	89335.	-7569.	8.06E-05	15962.	9.45E+09	296.3425	135461.	0.00
11.0000	-0.00628	67859.	-6694.	1.06E-04	15478.	9.45E+09	286.9637	137117.	0.00
11.2500	-0.00593	48943.	-5852.	1.24E-04	15051.	9.45E+09	274.2809	138773.	0.00
11.5000	-0.00553	32478.	-5052.	1.37E-04	14680.	9.45E+09	259.0303	140429.	0.00
11.7500	-0.00511	18334.	-4301.	1.45E-04	14361.	9.45E+09	241.8777	142085.	0.00
12.0000	-0.00466	6360.	-3603.	1.49E-04	14091.	9.45E+09	223.4172	143741.	0.00
12.2500	-0.00421	-3606.	-2962.	1.49E-04	14029.	9.45E+09	204.1729	145397.	0.00
12.5000	-0.00377	-11732.	-2378.	1.47E-04					
12.7500	-0.00333	-18194.	-1854.	1.42E-04					
13.0000	-0.00291	-23163.	-1387.	1.36E-04					
13.2500	-0.00252	-26810.	-977.0683	1.28E-04	14552.	9.45E+09	127.4951	152021.	0.00
13.5000	-0.00215	-29301.	-620.9790	1.19E-04	14608.	9.45E+09	109.8978	153677.	0.00

$L2 = [6,360 / ((6,360 + 3,606) / (12.25 - 12.0))] + 12.0 - 2.55 = 9.60 \text{ FT}$

~~Abutment 1 - HP14x117 Con. Initial~~

13.7500	-0.00180	-30793.	-316.1278	1.09E-04	14642.	9.45E+09	93.3364	155333.	0.00
14.0000	-0.00149	-31434.	-59.2259	9.95E-05	14656.	9.45E+09	77.9315	156989.	0.00
14.2500	-0.00121	-31363.	153.3176	8.95E-05	14655.	9.45E+09	63.7641	158645.	0.00
14.5000	-9.52E-04	-30708.	325.2839	7.97E-05	14640.	9.45E+09	50.8801	160301.	0.00
14.7500	-7.28E-04	-29584.	460.5468	7.01E-05	14615.	9.45E+09	39.2952	161957.	0.00
15.0000	-5.32E-04	-28096.	562.9889	6.09E-05	14581.	9.45E+09	28.9995	163613.	0.00
15.2500	-3.62E-04	-26337.	636.4306	5.23E-05	14541.	9.45E+09	19.9616	165269.	0.00
15.5000	-2.18E-04	-24390.	684.5718	4.42E-05	14497.	9.45E+09	12.1326	166925.	0.00
15.7500	-9.70E-05	-22325.	710.9453	3.68E-05	14451.	9.45E+09	5.4498	168581.	0.00
16.0000	2.82E-06	-20204.	718.8801	3.01E-05	14403.	9.45E+09	-0.1599	170237.	0.00
16.2500	8.34E-05	-18077.	711.4745	2.40E-05	14355.	9.45E+09	-4.77		
16.5000	1.47E-04	-15987.	691.5780	1.86E-05	14308.	9.45E+09	-8.48		DEPTH TO FIXITY = 16.0 FT
16.7500	1.95E-04	-13968.	661.7807	1.38E-05	14262.	9.45E+09	-11.3777	175205.	0.00
17.0000	2.30E-04	-12046.	624.4085	9.69E-06	14219.	9.45E+09	-13.5371	176861.	0.00
17.2500	2.53E-04	-10242.	581.5248	6.15E-06	14178.	9.45E+09	-15.0520	178517.	0.00
17.5000	2.67E-04	-8570.	534.9365	3.16E-06	14141.	9.45E+09	-16.0068	180173.	0.00
17.7500	2.72E-04	-7039.	486.2034	6.86E-07	14106.	9.45E+09	-16.4818	181829.	0.00
18.0000	2.71E-04	-5654.	436.6516	-1.33E-06	14075.	9.45E+09	-16.5527	183485.	0.00
18.2500	2.64E-04	-4417.	387.3879	-2.93E-06	14047.	9.45E+09	-16.2898	185141.	0.00
18.5000	2.53E-04	-3324.	339.3170	-4.16E-06	14022.	9.45E+09	-15.7575	186797.	0.00
18.7500	2.39E-04	-2372.	293.1593	-5.06E-06	14001.	9.45E+09	-15.0143	188453.	0.00
19.0000	2.23E-04	-1554.	249.4692	-5.68E-06	13982.	9.45E+09	-14.1124	190109.	0.00
19.2500	2.05E-04	-862.5635	208.6536	-6.07E-06	13967.	9.45E+09	-13.0980	191765.	0.00
19.5000	1.86E-04	-288.8421	170.9903	-6.25E-06	13954.	9.45E+09	-12.0109	193421.	0.00
19.7500	1.67E-04	176.8805	136.6460	-6.27E-06	13951.	9.45E+09	-10.8853	195077.	0.00
20.0000	1.49E-04	544.5744	105.6930	-6.15E-06	13960.	9.45E+09	-9.7500	196733.	0.00
20.2500	1.30E-04	824.3317	78.1256	-5.94E-06	13966.	9.45E+09	-8.6283	198389.	0.00
20.5000	1.13E-04	1026.	53.8749	-5.64E-06	13970.	9.45E+09	-7.5388	200045.	0.00
20.7500	9.66E-05	1160.	32.8229	-5.30E-06	13973.	9.45E+09	-6.4959	201701.	0.00
21.0000	8.13E-05	1235.	14.8146	-4.92E-06	13975.	9.45E+09	-5.5096	203357.	0.00
21.2500	6.71E-05	1259.	-0.3301	-4.52E-06	13976.	9.45E+09	-4.5868	205013.	0.00
21.5000	5.42E-05	1242.	-12.8070	-4.12E-06	13975.	9.45E+09	-3.7311	206669.	0.00
21.7500	4.24E-05	1191.	-22.8183	-3.74E-06	13974.	9.45E+09	-2.9431	208325.	0.00
22.0000	3.17E-05	1113.	-30.5654	-3.37E-06	13972.	9.45E+09	-2.2216	209981.	0.00
22.2500	2.22E-05	1015.	-36.2426	-3.03E-06	13970.	9.45E+09	-1.5632	211637.	0.00
22.5000	1.35E-05	902.5679	-40.0318	-2.73E-06	13968.	9.45E+09	-0.9629	213293.	0.00
22.7500	5.79E-06	780.9309	-42.0984	-2.46E-06	13965.	9.45E+09	-0.4148	214949.	0.00
23.0000	-1.22E-06	655.2930	-42.5883	-2.23E-06	13962.	9.45E+09	0.08823	216605.	0.00
23.2500	-7.61E-06	530.2245	-41.6256	-2.04E-06	13959.	9.45E+09	0.5536	218261.	0.00
23.5000	-1.35E-05	409.9564	-39.3118	-1.90E-06	13956.	9.45E+09	0.9889	219917.	0.00
23.7500	-1.90E-05	298.4481	-35.7254	-1.78E-06	13954.	9.45E+09	1.4020	221573.	0.00

Abutment 1 HP14x117 Con. Initial									
24.0000	-2.42E-05	199.4552	-30.9226	-1.70E-06	13952.	9.45E+09	1.7999	223229.	0.00
24.2500	-2.92E-05	116.5929	-24.9388	-1.65E-06	13950.	9.45E+09	2.1893	224885.	0.00
24.5000	-3.41E-05	53.3946	-17.7909	-1.63E-06	13948.	9.45E+09	2.5759	226541.	0.00
24.7500	-3.90E-05	13.3614	-9.4810	-1.62E-06	13948.	9.45E+09	2.9641	228197.	0.00
25.0000	-4.38E-05	0.00	0.00	-1.61E-06	13947.	9.45E+09	3.3566	114926.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be 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. 2:

Pile-head deflection = 0.30000000 inches
 Computed slope at pile head = -0.00152482 radians
 Maximum bending moment = -2060422. inch-lbs
 Maximum shear force = 87897. 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 = 3

 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

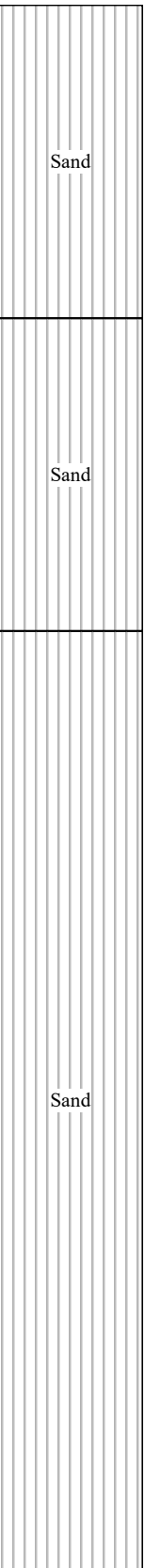
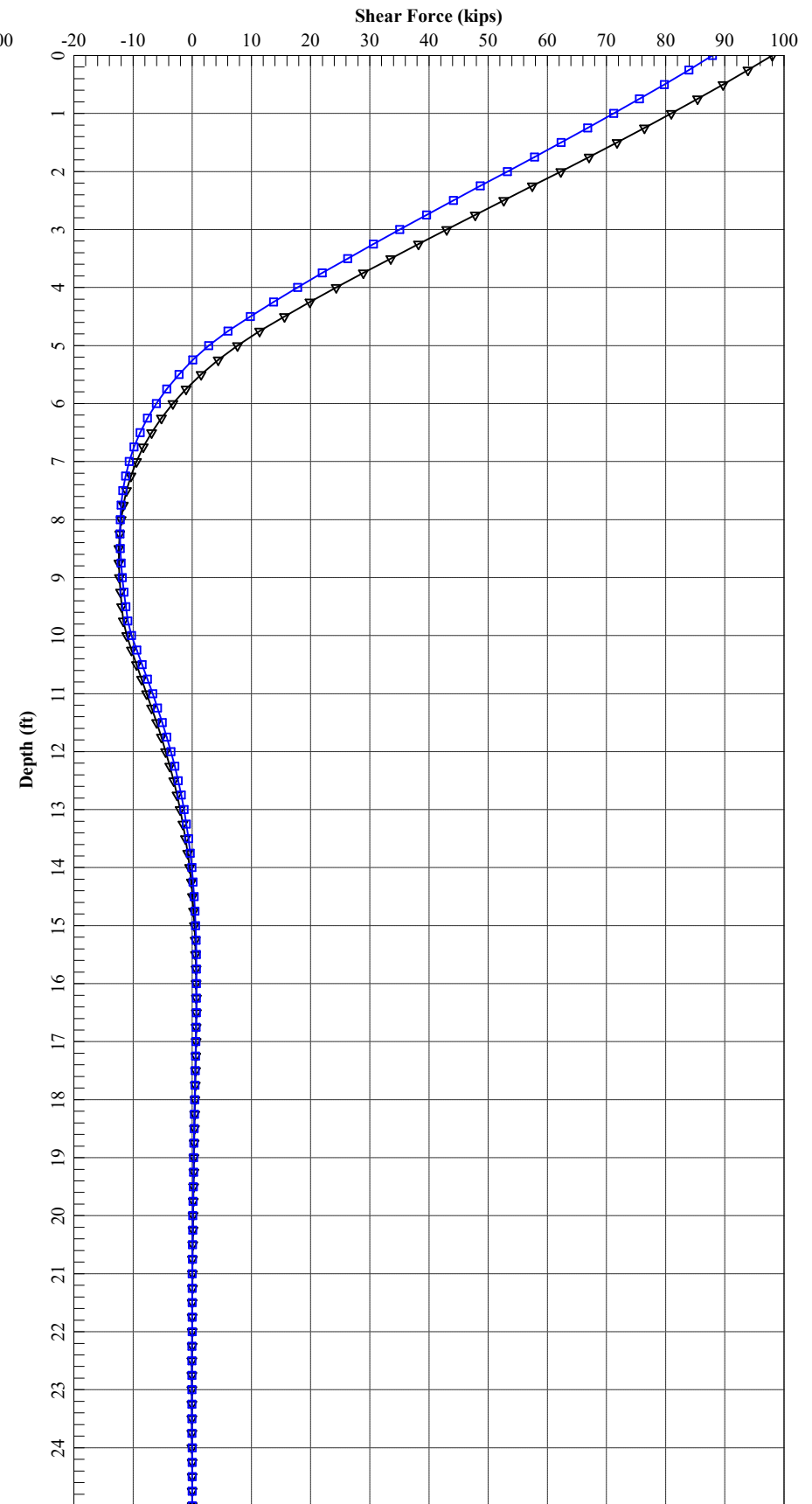
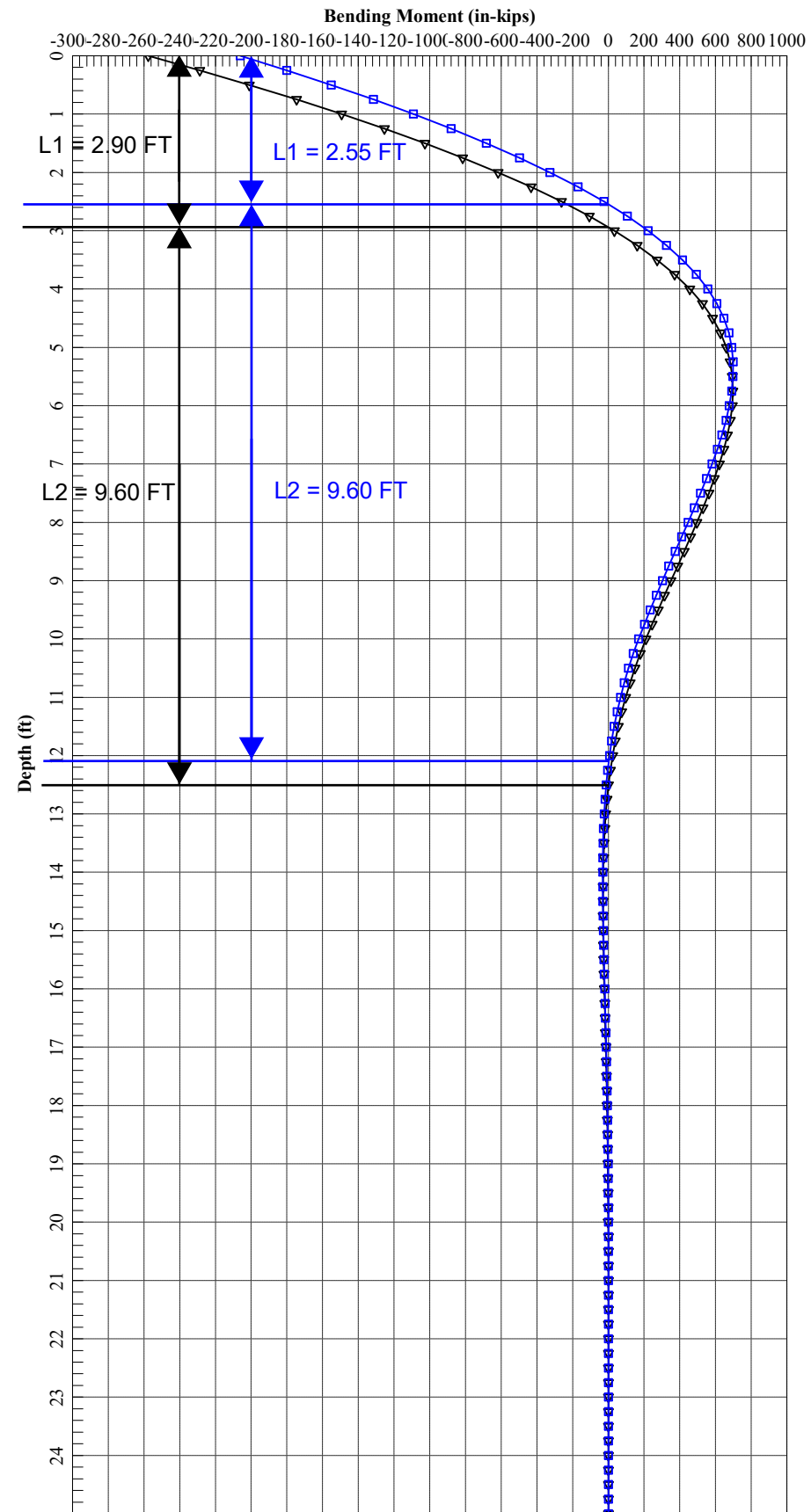
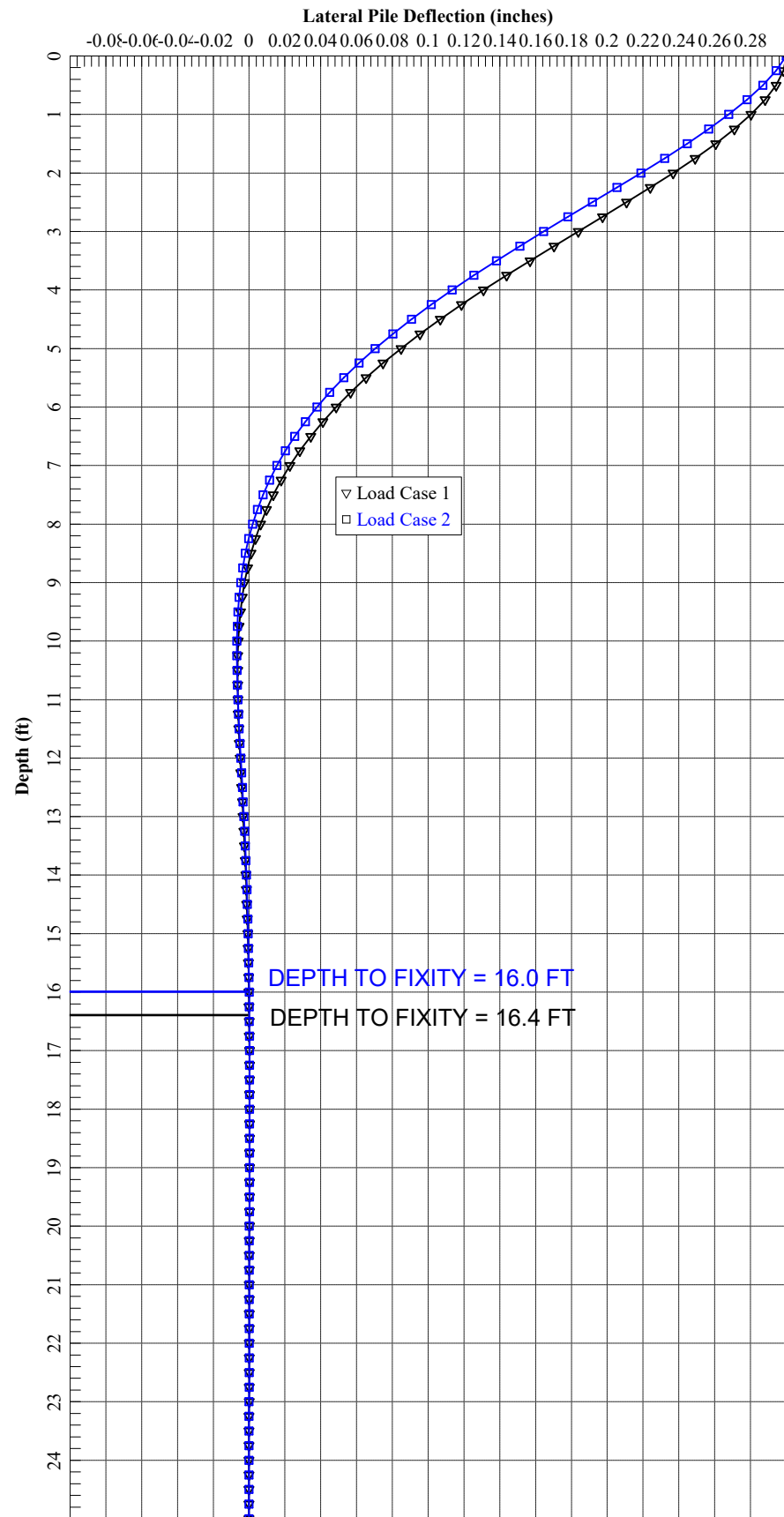
Load Case	Load Type	Load Type	Load Type	Axial Loading	Pile-head Deflection	Pile-head Rotation	Max Shear in Pile	Max Moment in Pile
-----------	-----------	-----------	-----------	---------------	----------------------	--------------------	-------------------	--------------------

Abutment 1 - HP14x117 Con. Initial									
No.	1	Load 1	2	Load 2	lbs	inches	radians	lbs	in-lbs
1	y, in	0.3000	S, rad	0.00	360000.	0.3000	0.00	97998.	-2575709.
2	y, in	0.3000	M, in-lb	-2060422.	360000.	0.3000	-0.00152	87897.	-2060422.

Maximum pile-head deflection = 0.3000000000 inches

Maximum pile-head rotation = -0.0015248157 radians = -0.087366 deg.

The analysis ended normally.



Abutment 1 - HP14x117 Con. Initial

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LPILE for Windows, Version 2019-11.009

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:

\\haleyaldrich.com\share\CF\Projects\0205793\Calculations\8. Integral Abutment Pile Design\

Name of input data file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Contraction Case Initial.lp11d

Name of output report file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Contraction Case Initial.lp11o

Name of plot output file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Contraction Case Initial.lp11p

Abutment 1 - HP14x117 Con. Initial

Name of runtime message file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Contraction Case Initial.lp11r

Date and Time of Analysis

Date: December 20, 2022

Time: 11:32:13

Problem Title

Project Name: Mattagodus Stream Bridge No. 5116

Job Number: 0205793-000

Client: Maine Department of Transportation

Engineer: Bryan C. Steinert, P.E.

Description: Abutment 1 HP14x117 Contraction Case Initial

Program Options and Settings

Computational Options:

- Conventional Analysis

Abutment 1 - HP14x117 Con. Initial

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
- Input of side resistance moment along pile 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 = 25.000 ft
- Depth of ground surface below top of pile = -3.7000 ft

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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	14.9000
2	25.000	14.9000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 25.000000 ft
Pile width = 14.200000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians
Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

Abutment 1 - HP14x117 Con. Initial

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	=	-3.700000 ft
Distance from top of pile to bottom of layer	=	5.000000 ft
Effective unit weight at top of layer	=	120.000000 pcf
Effective unit weight at bottom of layer	=	120.000000 pcf
Friction angle at top of layer	=	31.000000 deg.
Friction angle at bottom of layer	=	31.000000 deg.
Subgrade k at top of layer	=	124.000000 pci
Subgrade k at bottom of layer	=	124.000000 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	=	62.600000 pcf
Effective unit weight at bottom of layer	=	62.600000 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	=	76.000000 pci
Subgrade k at bottom of layer	=	76.000000 pci

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

Distance from top of pile to top of layer	=	10.000000 ft
Distance from top of pile to bottom of layer	=	25.000000 ft
Effective unit weight at top of layer	=	67.600000 pcf
Effective unit weight at bottom of layer	=	67.600000 pcf
Friction angle at top of layer	=	48.000000 deg.
Friction angle at bottom of layer	=	48.000000 deg.
Subgrade k at top of layer	=	184.000000 pci
Subgrade k at bottom of layer	=	184.000000 pci

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Abutment 1 - HP14x117 Con. Initial

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.	kpy pci
1	Sand (Reese, et al.)	-3.7000 5.0000	120.0000 120.0000	31.0000 31.0000	124.0000 124.0000
2	Sand (Reese, et al.)	5.0000 10.0000	62.6000 62.6000	34.0000 34.0000	76.0000 76.0000
3	Sand (Reese, et al.)	10.0000 25.0000	67.6000 67.6000	48.0000 48.0000	184.0000 184.0000

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.300000 in	S = 0.0000 in/in	360000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle

Abutment 1 - HP14x117 Con. Initial

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	=	25.000000 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

Abutment 1 - HP14x117 Con. Initial

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

Number	Axial Thrust Force kips
1	360.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 360.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in ²	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000452	58.2422871	12886018.	87.9369603	11.5165140	
0.00000904	116.4845741	12886018.	47.6934801	12.4832529	
0.00001356	174.7268612	12886018.	34.2789868	13.4499915	
0.00001808	232.9691483	12886018.	27.5717401	14.4167303	
0.00002260	291.2114353	12886018.	23.5473921	15.3834692	
0.00002712	349.4537224	12886018.	20.8644934	16.3502080	
0.00003164	407.6960095	12886018.	18.9481372	17.3169468	
0.00003616	465.9382965	12886018.	17.5108700	18.2836856	
0.00004068	524.1805836	12886018.	16.3929956	19.2504244	
0.00004520	582.4228707	12886018.	15.4986960	20.2171632	
0.00004972	640.6651577	12886018.	14.7669964	21.1839020	
0.00005424	698.9074448	12886018.	14.1572467	22.1506408	
0.00005876	757.1497319	12886018.	13.6413046	23.1173795	
0.00006328	815.3920189	12886018.	13.1990686	24.0841184	
0.00006780	873.6343060	12886018.	12.8157974	25.0508572	
0.00007232	931.8765931	12886018.	12.4804350	26.0175960	
0.00007684	990.1188801	12886018.	12.1845271	26.9843348	
0.00008136	1048.	12886018.	11.9214978	27.9510736	
0.00008588	1107.	12886018.	11.6861558	28.9178124	
0.00009040	1165.	12886018.	11.4743480	29.8845512	
0.00009492	1223.	12886018.	11.2827124	30.8512900	

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0.00009944	1281.	12886018.	11.1084982	31.8180288	
0.0001040	1340.	12886018.	10.9494331	32.7847676	
0.0001085	1398.	12886018.	10.8036233	33.7515064	
0.0001130	1456.	12886018.	10.6694784	34.7182452	
0.0001175	1514.	12886018.	10.5456523	35.6849840	
0.0001220	1573.	12886018.	10.4309985	36.6517228	
0.0001266	1631.	12886018.	10.3245343	37.6184616	
0.0001311	1689.	12886018.	10.2254124	38.5852005	
0.0001356	1747.	12886018.	10.1328987	39.5519393	
0.0001401	1806.	12886018.	10.0463536	40.5186780	
0.0001446	1864.	12886018.	9.9652175	41.4854169	
0.0001492	1922.	12886018.	9.8889988	42.4521557	
0.0001537	1980.	12886018.	9.8172635	43.4188945	
0.0001582	2038.	12886018.	9.7496274	44.3856333	
0.0001627	2097.	12886018.	9.6857489	45.3523721	
0.0001672	2155.	12886018.	9.6253233	46.3191109	
0.0001718	2213.	12886018.	9.5680779	47.2858497	
0.0001763	2271.	12886018.	9.5137682	48.2525885	
0.0001853	2388.	12884229.	9.4133417	50.0000000	Y
0.0001944	2498.	12850726.	9.3266816	50.0000000	Y
0.0002034	2600.	12785182.	9.2527758	50.0000000	Y
0.0002124	2697.	12695476.	9.1896481	50.0000000	Y
0.0002215	2788.	12588038.	9.1356250	50.0000000	Y
0.0002305	2874.	12468129.	9.0892745	50.0000000	Y
0.0002395	2956.	12339598.	9.0494406	50.0000000	Y
0.0002486	3034.	12203939.	9.0154352	50.0000000	Y
0.0002576	3108.	12065491.	8.9860874	50.0000000	Y
0.0002667	3180.	11924166.	8.9610749	50.0000000	Y
0.0002757	3248.	11782141.	8.9396773	50.0000000	Y
0.0002847	3315.	11640692.	8.9213663	50.0000000	Y
0.0002938	3378.	11498062.	8.9053644	50.0000000	Y
0.0003028	3436.	11346846.	8.8898531	50.0000000	Y
0.0003119	3490.	11189286.	8.8747665	50.0000000	Y
0.0003209	3539.	11027301.	8.8600448	50.0000000	Y
0.0003299	3584.	10862153.	8.8455497	50.0000000	Y
0.0003390	3626.	10695826.	8.8313547	50.0000000	Y
0.0003480	3665.	10529594.	8.8174384	50.0000000	Y
0.0003571	3701.	10364416.	8.8037546	50.0000000	Y
0.0003661	3734.	10200064.	8.7902735	50.0000000	Y
0.0003751	3765.	10037159.	8.7771819	50.0000000	Y
0.0003842	3795.	9877146.	8.7642057	50.0000000	Y

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0.0003932	3822.	9719758.	8.7515922	50.0000000	Y
0.0004023	3848.	9565338.	8.7393796	50.0000000	Y
0.0004113	3872.	9413797.	8.7270278	50.0000000	Y
0.0004203	3894.	9264686.	8.7151807	50.0000000	Y
0.0004294	3916.	9120011.	8.7034122	50.0000000	Y
0.0004384	3936.	8977345.	8.6920428	50.0000000	Y
0.0004475	3955.	8838565.	8.6805923	50.0000000	Y
0.0004565	3973.	8703114.	8.6697822	50.0000000	Y
0.0004655	3990.	8570219.	8.6586971	50.0000000	Y
0.0004746	4006.	8441393.	8.6481131	50.0000000	Y
0.0004836	4021.	8314667.	8.6375687	50.0000000	Y
0.0004927	4036.	8192121.	8.6272271	50.0000000	Y
0.0005017	4049.	8071558.	8.6170671	50.0000000	Y
0.0005107	4063.	7954696.	8.6072785	50.0000000	Y
0.0005198	4075.	7840322.	8.5971758	50.0000000	Y
0.0005288	4087.	7729135.	8.5878539	50.0000000	Y
0.0005379	4098.	7619885.	8.5782352	50.0000000	Y
0.0005740	4139.	7211330.	8.5420544	50.0000000	Y
0.0006102	4174.	6839857.	8.5077578	50.0000000	Y
0.0006463	4203.	6502548.	8.4757838	50.0000000	Y
0.0006825	4228.	6195187.	8.4456503	50.0000000	Y
0.0007186	4250.	5914065.	8.4171823	50.0000000	Y
0.0007548	4269.	5655959.	8.3904513	50.0000000	Y
0.0007910	4286.	5418870.	8.3647287	50.0000000	Y
0.0008271	4301.	5199854.	8.3408628	50.0000000	Y
0.0008633	4314.	4997779.	8.3178990	50.0000000	Y
0.0008994	4327.	4810325.	8.2965344	50.0000000	Y
0.0009356	4337.	4635770.	8.2756913	50.0000000	Y
0.0009718	4347.	4473657.	8.2558994	50.0000000	Y
0.0010079	4356.	4321867.	8.2373514	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
-----	-----	-----

1

360.0000000000

Abutment 1 - HP14x117 Con. Initial
4356.

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

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

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

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-3.7000	0.00	N.A.	No	0.00	66781.
2	5.0000	7.9387	Yes	No	66781.	161811.
3	10.0000	8.7466	Yes	No	228593.	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

Abutment 1 - HP14x117 Con. Initial

for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 0.300000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3000	-2047353.	51193.	0.00	44875.	1.29E+10	-321.1985	1606.	0.00
0.2500	0.2993	-1895071.	50164.	-4.59E-04	42322.	1.29E+10	-340.7999	3416.	0.00
0.5000	0.2972	-1745380.	49114.	-8.83E-04	39812.	1.29E+10	-358.9772	3623.	0.00
0.7500	0.2940	-1598480.	48014.	-0.00127	37349.	1.29E+10	-374.1504	3818.	0.00
1.0000	0.2896	-1454546.	46874.	-0.00163	34936.	1.29E+10	-385.8187	3997.	0.00
1.2500	0.2842	-1313718.	45706.	-0.00195	32575.	1.29E+10	-393.4775	4153.	0.00
1.5000	0.2779	-1176102.	44489.	-0.00224	30268.	1.29E+10	-417.5104	4507.	0.00
1.7500	0.2708	-1041947.	43200.	-0.00250	28019.	1.29E+10	-441.8198	4895.	0.00
2.0000	0.2629	-911507.	41840.	-0.00272	25832.	1.29E+10	-464.8046	5303.	0.00
2.2500	0.2544	-785021.	40414.	-0.00292	23711.	1.29E+10	-486.2402	5733.	0.00
2.5000	0.2454	-662713.	38925.	-0.00309	21661.	1.29E+10	-505.9052	6185.	0.00
2.7500	0.2359	-544792.	37372.	-0.00323	19684.	1.29E+10	-529.9282	6739.	0.00
3.0000	0.2260	-431504.	35747.	-0.00335	17784.	1.29E+10	-552.8450	7338.	0.00
3.2500	0.2158	-323082.	34057.	-0.00343	15966.	1.29E+10	-574.2786	7983.	0.00
3.5000	0.2054	-219748.	32304.	-0.00350	14234.	1.29E+10	-594.0667	8676.	0.00
3.7500	0.1948	-121705.	30495.	-0.00354	12590.	1.29E+10	-612.0561	9424.	0.00
4.0000	0.1842	-29141.	28635.	-0.00355	11038.	1.29E+10	-628.1044	10230.	0.00
4.2500	0.1735	57779.	26729.	-0.00355	11518.	1.29E+10	-642.0809	11101.	0.00
4.5000	0.1629	138905.	24786.	-0.00353	12879.	1.29E+10	-654.7522	12000.	0.00
4.7500	0.1524	214111.	22810.	-0.00349	14139.	1.29E+10	-664.7522	13539.	0.00
5.0000	0.1420	283293.	20854.	-0.00343	15299.	1.29E+10	-673.2533	14664.	0.00
5.2500	0.1318	346638.	18808.	-0.00335	16361.	1.29E+10	-679.6468	15359.	0.00
5.5000	0.1218	403385.	16643.	-0.00327	17313.	1.29E+10	-684.5422	15719.	0.00
5.7500	0.1122	453555.	14493.	-0.00317	18154.	1.29E+10	-688.9547	15881.	0.00
6.0000	0.1028	497189.	12366.	-0.00306	18885.	1.29E+10	-693.9115	15955.	0.00
6.2500	0.09384	534353.	10267.	-0.00294	19508.	1.29E+10	-698.4510	15955.	0.00
6.5000	0.08522	565137.	8206.	-0.00281	20025.	1.29E+10	-702.6218	15899.	0.00
6.7500	0.07699	589656.	6188.	-0.00267	20436.	1.29E+10	-706.4815	15800.	0.00
7.0000	0.06917	608044.	4222.	-0.00254	20744.	1.29E+10	-710.0000	15660.	0.00

$$L1 = [29,141 / ((29,141 + 57,779) / (4.25 - 4.0))] + 4.0 = 4.1 \text{ FT}$$

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7.2500	0.06177	620461.	2327.	-0.00239	20952.	1.29E+10	-616.8954	29959.	0.00
7.5000	0.05481	627170.	561.3348	-0.00225	21065.	1.29E+10	-559.8844	30643.	0.00
7.7500	0.04829	628683.	-1035.	-0.00210	21090.	1.29E+10	-504.2737	31327.	0.00
8.0000	0.04221	625499.	-2467.	-0.00195	21037.	1.29E+10	-450.3740	32011.	0.00
8.2500	0.03656	618104.	-3740.	-0.00181	20913.	1.29E+10	-398.4616	32695.	0.00
8.5000	0.03135	606968.	-4861.	-0.00167	20726.	1.29E+10	-348.7778	33379.	0.00
8.7500	0.02656	592540.	-5836.	-0.00153	20484.	1.29E+10	-301.5296	34063.	0.00
9.0000	0.02218	575250.	-6674.	-0.00139	20194.	1.29E+10	-256.8901	34747.	0.00
9.2500	0.01820	555503.	-7382.	-0.00126	19863.	1.29E+10	-214.9999	35431.	0.00
9.5000	0.01462	533681.	-7968.	-0.00113	19497.	1.29E+10	-175.9678	36115.	0.00
9.7500	0.01140	510141.	-8442.	-0.00101	19103.	1.29E+10	-139.8721	36799.	0.00
10.0000	0.00854	485214.	-9001.	-8.96E-04	18685.	1.29E+10	-232.4934	81626.	0.00
10.2500	0.00603	458073.	-9628.	-7.86E-04	18230.	1.29E+10	-185.6001	92405.	0.00
10.5000	0.00383	429146.	-10086.	-6.83E-04	17745.	1.29E+10	-119.9727	94061.	0.00
10.7500	0.00193	399031.	-10358.	-5.87E-04	17240.	1.29E+10	-61.4804	95717.	0.00
11.0000	3.06E-04	368263.	-10465.	-4.97E-04	16724.	1.29E+10	-9.9368	97373.	0.00
11.2500	-0.00106	337313.	-10428.	-4.15E-04	16205.	1.29E+10	34.9058	99029.	0.00
11.5000	-0.00219	306592.	-10266.	-3.40E-04	15690.	1.29E+10	73.3471	100685.	0.00
11.7500	-0.00310	276454.	-9997.	-2.72E-04	15185.	1.29E+10	105.7288	102341.	0.00
12.0000	-0.00382	247198.	-9640.	-2.11E-04	14694.	1.29E+10	132.4261	103997.	0.00
12.2500	-0.00437	219072.	-9210.	-1.57E-04	14223.	1.29E+10	153.8388	105653.	0.00
12.5000	-0.00476	192275.	-8724.	-1.09E-04	13773.	1.29E+10	170.3836	107309.	0.00
12.7500	-0.00502	166964.	-8195.	-6.75E-05	13349.	1.29E+10	182.4870	108965.	0.00
13.0000	-0.00517	143253.	-7635.	-3.14E-05	12951.	1.29E+10	190.5784	110621.	0.00
13.2500	-0.00521	121221.	-7057.	-6.13E-07	12582.	1.29E+10	195.0845	112277.	0.00
13.5000	-0.00517	100914.	-6469.	2.52E-05	12242.	1.29E+10	196.4240	113933.	0.00
13.7500	-0.00506	82350.	-5882.	4.66E-05	11930.	1.29E+10	195.0032	115589.	0.00
14.0000	-0.00489	65520.	-5303.	6.38E-05	11648.	1.29E+10	191.2121	117245.	0.00
14.2500	-0.00468	50394.	-4738.	7.73E-05	11395.	1.29E+10	185.4213	118901.	0.00
14.5000	-0.00443	36925.	-4193.	8.74E-05	11169.	1.29E+10	177.9796	120557.	0.00
14.7500	-0.00415	25048.	-3672.	9.47E-05	10970.	1.29E+10	169.2119	122213.	0.00
15.0000	-0.00386	14688.	-3179.	9.93E-05	10796.	1.29E+10	159.4180	123869.	0.00
15.2500	-0.00356	5759.	-2717.	1.02E-04	10646.	1.29E+10	148.8717	125525.	0.00
15.5000	-0.00325	-1832.	-2287.	1.02E-04	10580.	1.29E+10	137.8204	127181.	0.00
15.7500	-0.00295	-8182.	-1890.	1.01E-04	10687.	1.29E+10			
16.0000	-0.00265	-13392.	-1528.	9.84E-05	10774.	1.29E+10			
16.2500	-0.00235	-17562.	-1200.	9.48E-05	10844.	1.29E+10			
16.5000	-0.00208	-20795.	-905.2644	9.04E-05	10898.	1.29E+10	92.6001	133805.	0.00
16.7500	-0.00181	-23189.	-643.6172	8.53E-05	10939.	1.29E+10	81.8313	135461.	0.00
17.0000	-0.00156	-24841.	-413.6031	7.97E-05	10966.	1.29E+10	71.5114	137117.	0.00
17.2500	-0.00133	-25843.	-213.7549	7.38E-05	10983.	1.29E+10	61.7207	138773.	0.00

$$L2 = [5,759 / ((5,759 + 1,832) / (15.5 - 15.25))] + 15.25 - 4.1 = 11.3 \text{ FT}$$

Abutment 1 - HP14x117 Con. Initial

17.5000	-0.00112	-26283.	-42.3932	6.77E-05	10990.	1.29E+10	52.5205	140429.	0.00
17.7500	-9.28E-04	-26243.	102.3207	6.16E-05	10990.	1.29E+10	43.9554	142085.	0.00
18.0000	-7.52E-04	-25802.	222.3354	5.55E-05	10982.	1.29E+10	36.0544	143741.	0.00
18.2500	-5.95E-04	-25029.	319.6664	4.96E-05	10969.	1.29E+10	28.8330	145397.	0.00
18.5000	-4.55E-04	-23991.	396.3575	4.39E-05	10952.	1.29E+10	22.2944	147053.	0.00
18.7500	-3.31E-04	-22746.	454.4469	3.85E-05	10931.	1.29E+10	16.4318	148709.	0.00
19.0000	-2.24E-04	-21347.	495.9387	3.33E-05	10908.	1.29E+10	11.2294	150365.	0.00
19.2500	-1.32E-04	-19842.	522.7784	2.85E-05	10882.	1.29E+10	6.6638	152021.	0.00
19.5000	-5.28E-05	-18272.	536.8330	2.41E-05	10856.	1.29E+10	2.7059	153677.	0.00
19.7500	1.31E-05	-16673.	539.8748	2.00E-05	10829.	1.29E+10	-0.6780	155333.	0.00
20.0000	6.74E-05	-15076.	533.5699	1.63E-05	10803.	1.29E+10	-3.5253	156989.	0.00
20.2500	1.11E-04	-13507.	519.4685	1.30E-05	10776.	1.29E+10	-5.8757	158645.	0.00
20.5000	1.45E-04	-11988.	498.9995	1.00E-05	10751.	1.29E+10	-7.7703	160301.	0.00
20.7500	1.71E-04	-10535.	473.4680	7.42E-06	10726.	1.29E+10	-9.2507	161957.	0.00
21.0000	1.90E-04	-9163.	444.0541	5.13E-06	10703.	1.29E+10	-10.3585	163613.	0.00
21.2500	2.02E-04	-7882.	411.8148	3.14E-06	10682.	1.29E+10	-11.1343	165269.	0.00
21.5000	2.09E-04	-6699.	377.6875	1.44E-06	10662.	1.29E+10	-11.6172	166925.	0.00
21.7500	2.11E-04	-5619.	342.4947	1.11E-08	10644.	1.29E+10	-11.8446	168581.	0.00
22.0000	2.09E-04	-4644.	306.9505	-1.18E-06	10628.	1.29E+10	-11.8515	170237.	0.00
22.2500	2.04E-04	-3774.	271.6676	-2.16E-06	10613.	1.29E+10	-11.6704	171893.	0.00
22.5000	1.96E-04	-3009.	237.1652	-2.95E-06	10600.	1.29E+10	-11.3312	173549.	0.00
22.7500	1.86E-04	-2345.	203.8777	-3.58E-06	10589.	1.29E+10	-10.8605	175205.	0.00
23.0000	1.74E-04	-1778.	172.1634	-4.06E-06	10580.	1.29E+10	-10.2823	176861.	0.00
23.2500	1.62E-04	-1303.	142.3136	-4.42E-06	10572.	1.29E+10	-9.6175	178517.	0.00
23.5000	1.48E-04	-914.7537	114.5613	-4.67E-06	10565.	1.29E+10	-8.8840	180173.	0.00
23.7500	1.34E-04	-605.8854	89.0906	-4.85E-06	10560.	1.29E+10	-8.0965	181829.	0.00
24.0000	1.19E-04	-369.7335	66.0448	-4.96E-06	10556.	1.29E+10	-7.2674	183485.	0.00
24.2500	1.04E-04	-198.8948	45.5348	-5.03E-06	10553.	1.29E+10	-6.4060	185141.	0.00
24.5000	8.86E-05	-85.6598	27.6468	-5.06E-06	10551.	1.29E+10	-5.5194	186797.	0.00
24.7500	7.34E-05	-22.0776	12.4494	-5.08E-06	10550.	1.29E+10	-4.6122	188453.	0.00
25.0000	5.82E-05	0.00	0.00	-5.08E-06	10550.	1.29E+10	-3.6874	95054.	0.00

DEPTH TO
FIXITY =
19.7 FT

* 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.

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Pile-head deflection = 0.30000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -2047353. inch-lbs
 Maximum shear force = 51193. 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 = 6
 Number of zero deflection points = 2

Summary of Pile-head Responses for Conventional Analyses

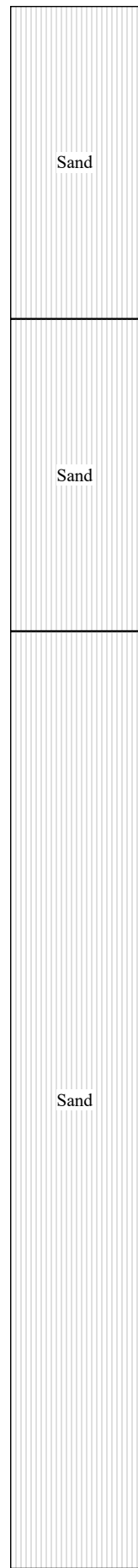
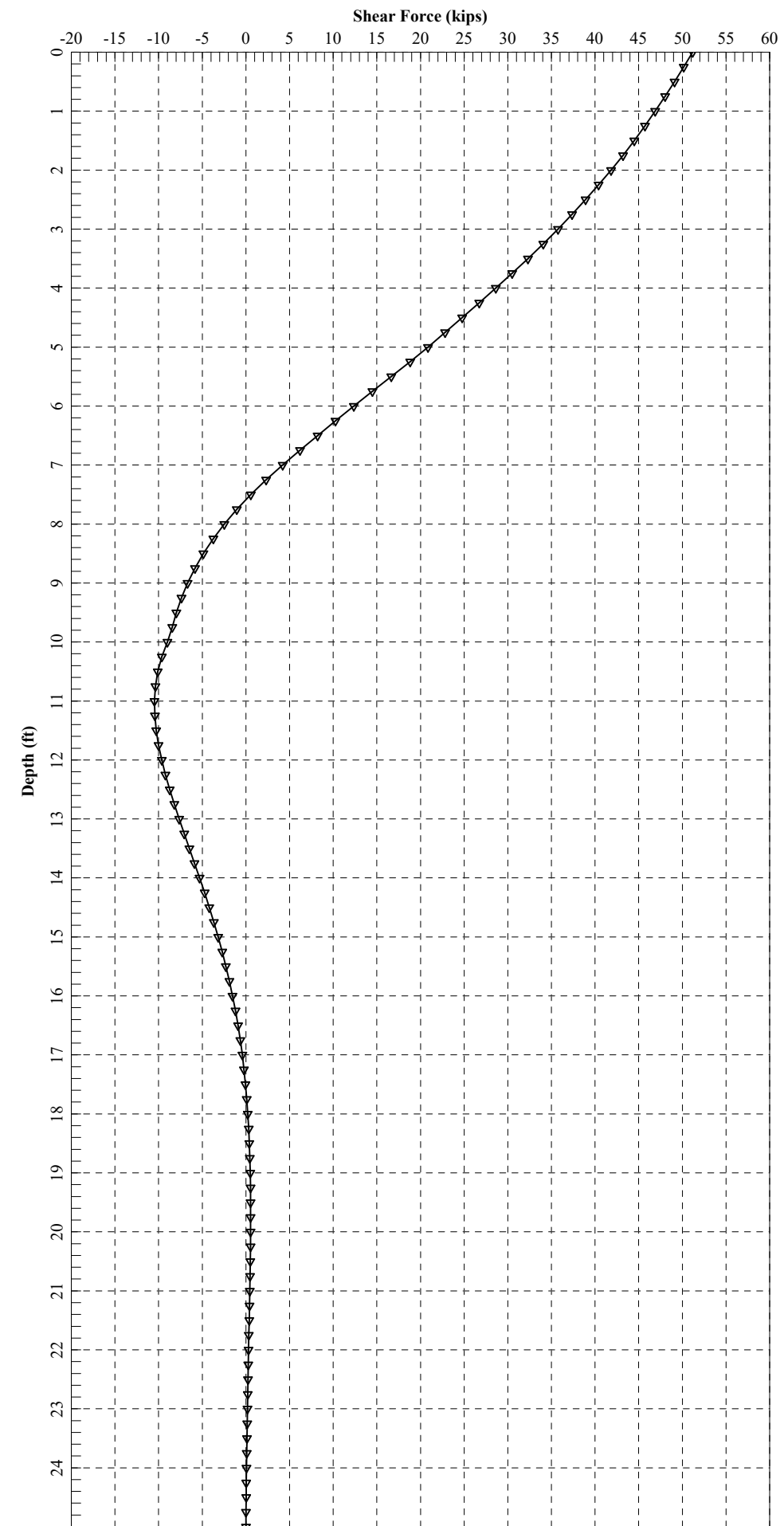
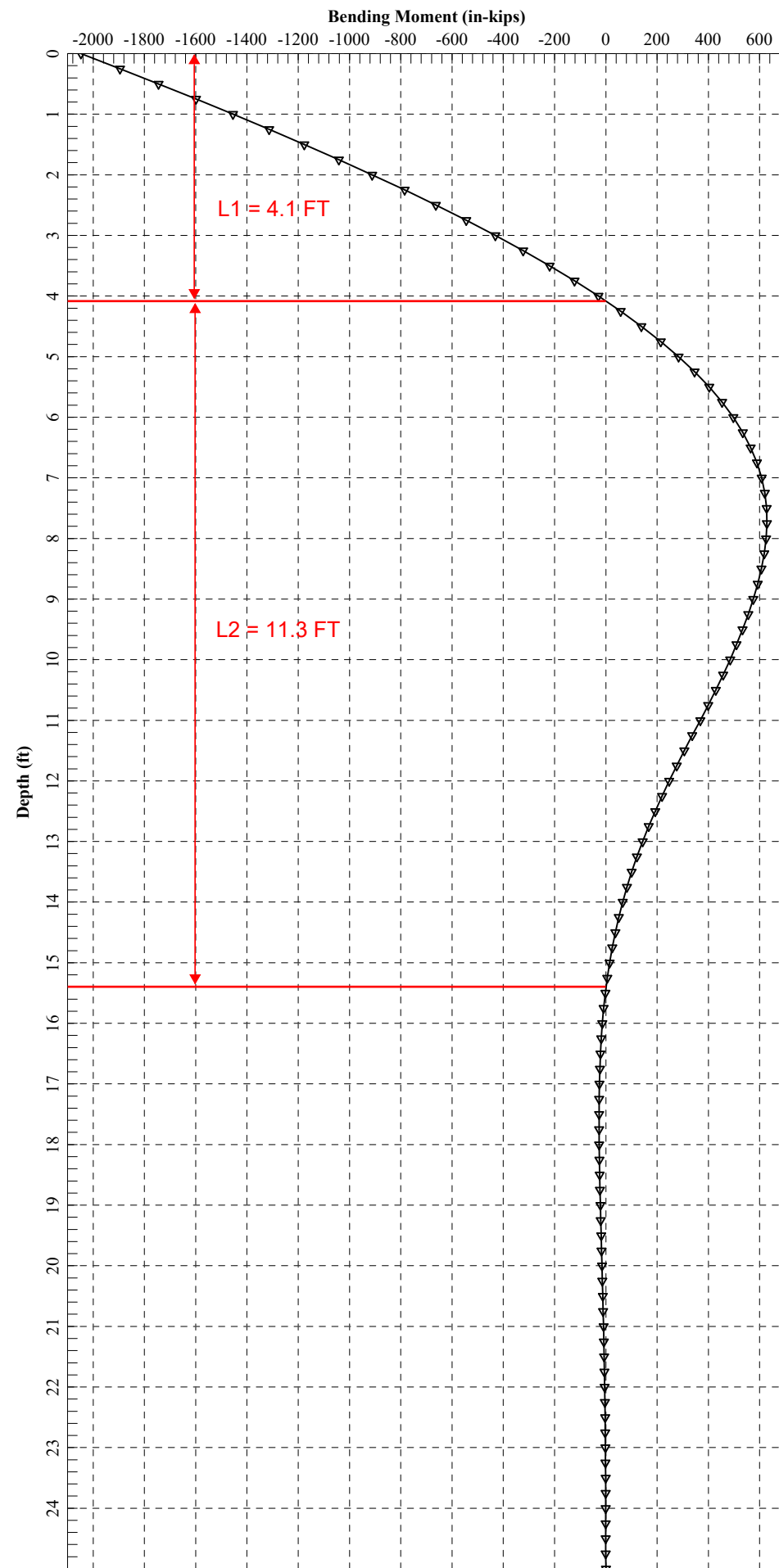
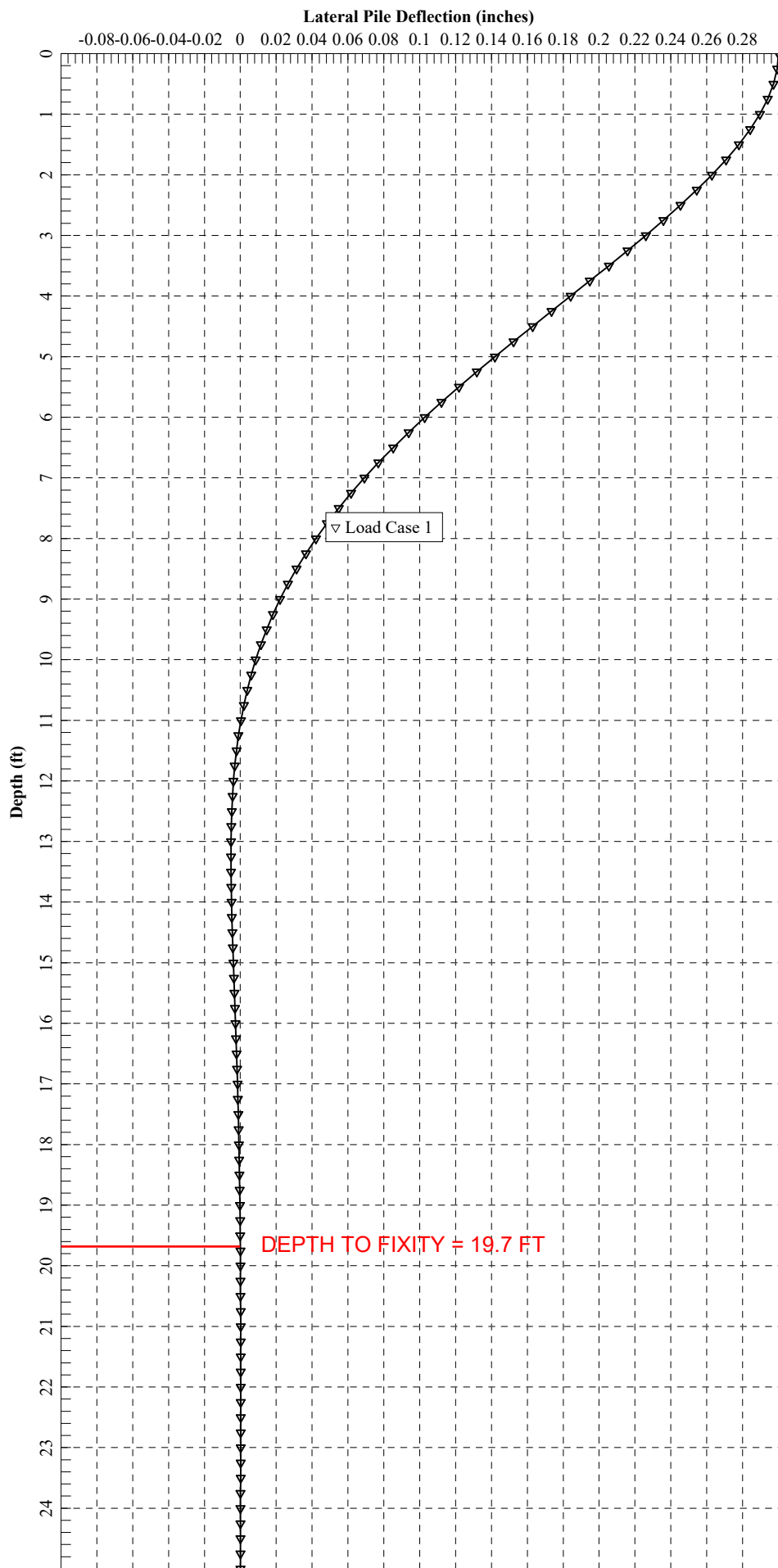
Definitions of Pile-head Loading Conditions:

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	0.3000	S, rad	0.00	360000.	0.3000	0.00	51193.	-2047353.

Maximum pile-head deflection = 0.3000000000 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.



Abutment 1 - HP14x117 Exp. Initial

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LPILE for Windows, Version 2019-11.009

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:

\\haleyaldrich.com\share\CF\Projects\0205793\Calculations\8. Integral Abutment Pile Design\

Name of input data file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Expansion Case Initial.lp11d

Name of output report file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Expansion Case Initial.lp11o

Name of plot output file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Expansion Case Initial.lp11p

Abutment 1 - HP14x117 Exp. Initial

Name of runtime message file:

2022-1213-HAI-22266 Webster Plt Abutment 1 HP14x117 Expansion Case Initial.lp11r

Date and Time of Analysis

Date: December 20, 2022

Time: 11:25:19

Problem Title

Project Name: Mattagodus Stream Bridge No. 5116

Job Number: 0205793-000

Client: Maine Department of Transportation

Engineer: Bryan C. Steinert, P.E.

Description: Abutment 1 HP14x117 Expansion Case Initial

Program Options and Settings

Computational Options:

- Conventional Analysis

Abutment 1 - HP14x117 Exp. Initial

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
- Input of side resistance moment along pile 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 = 25.000 ft
- Depth of ground surface below top of pile = -9.7000 ft

Abutment 1 - HP14x117 Exp. Initial

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	14.9000
2	25.000	14.9000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 25.000000 ft
Pile width = 14.200000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians
Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

Abutment 1 - HP14x117 Exp. Initial

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	=	-9.700000	ft
Distance from top of pile to bottom of layer	=	5.000000	ft
Effective unit weight at top of layer	=	120.000000	pcf
Effective unit weight at bottom of layer	=	120.000000	pcf
Friction angle at top of layer	=	31.000000	deg.
Friction angle at bottom of layer	=	31.000000	deg.
Subgrade k at top of layer	=	124.000000	pci
Subgrade k at bottom of layer	=	124.000000	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	=	62.600000	pcf
Effective unit weight at bottom of layer	=	62.600000	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	=	76.000000	pci
Subgrade k at bottom of layer	=	76.000000	pci

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

Distance from top of pile to top of layer	=	10.000000	ft
Distance from top of pile to bottom of layer	=	25.000000	ft
Effective unit weight at top of layer	=	67.600000	pcf
Effective unit weight at bottom of layer	=	67.600000	pcf
Friction angle at top of layer	=	48.000000	deg.
Friction angle at bottom of layer	=	48.000000	deg.
Subgrade k at top of layer	=	184.000000	pci
Subgrade k at bottom of layer	=	184.000000	pci

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Abutment 1 - HP14x117 Exp. Initial

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.	kpy pci
1	Sand (Reese, et al.)	-9.7000 5.0000	120.0000 120.0000	31.0000 31.0000	124.0000 124.0000
2	Sand (Reese, et al.)	5.0000 10.0000	62.6000 62.6000	34.0000 34.0000	76.0000 76.0000
3	Sand (Reese, et al.)	10.0000 25.0000	67.6000 67.6000	48.0000 48.0000	184.0000 184.0000

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.300000 in	S = 0.0000 in/in	360000.	N.A.	Yes

V = shear force applied normal to pile axis
 M = bending moment applied to pile head
 y = lateral deflection normal to pile axis
 S = pile slope relative to original pile batter angle

Abutment 1 - HP14x117 Exp. Initial

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	=	25.000000 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

Abutment 1 - HP14x117 Exp. Initial

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

Number	Axial Thrust Force kips
1	360.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 360.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in ²	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000452	58.2422871	12886018.	87.9369603	11.5165140	
0.00000904	116.4845741	12886018.	47.6934801	12.4832529	
0.00001356	174.7268612	12886018.	34.2789868	13.4499915	
0.00001808	232.9691483	12886018.	27.5717401	14.4167303	
0.00002260	291.2114353	12886018.	23.5473921	15.3834692	
0.00002712	349.4537224	12886018.	20.8644934	16.3502080	
0.00003164	407.6960095	12886018.	18.9481372	17.3169468	
0.00003616	465.9382965	12886018.	17.5108700	18.2836856	
0.00004068	524.1805836	12886018.	16.3929956	19.2504244	
0.00004520	582.4228707	12886018.	15.4986960	20.2171632	
0.00004972	640.6651577	12886018.	14.7669964	21.1839020	
0.00005424	698.9074448	12886018.	14.1572467	22.1506408	
0.00005876	757.1497319	12886018.	13.6413046	23.1173795	
0.00006328	815.3920189	12886018.	13.1990686	24.0841184	
0.00006780	873.6343060	12886018.	12.8157974	25.0508572	
0.00007232	931.8765931	12886018.	12.4804350	26.0175960	
0.00007684	990.1188801	12886018.	12.1845271	26.9843348	
0.00008136	1048.	12886018.	11.9214978	27.9510736	
0.00008588	1107.	12886018.	11.6861558	28.9178124	
0.00009040	1165.	12886018.	11.4743480	29.8845512	
0.00009492	1223.	12886018.	11.2827124	30.8512900	

Abutment 1 - HP14x117 Exp. Initial

0.00009944	1281.	12886018.	11.1084982	31.8180288	
0.0001040	1340.	12886018.	10.9494331	32.7847676	
0.0001085	1398.	12886018.	10.8036233	33.7515064	
0.0001130	1456.	12886018.	10.6694784	34.7182452	
0.0001175	1514.	12886018.	10.5456523	35.6849840	
0.0001220	1573.	12886018.	10.4309985	36.6517228	
0.0001266	1631.	12886018.	10.3245343	37.6184616	
0.0001311	1689.	12886018.	10.2254124	38.5852005	
0.0001356	1747.	12886018.	10.1328987	39.5519393	
0.0001401	1806.	12886018.	10.0463536	40.5186780	
0.0001446	1864.	12886018.	9.9652175	41.4854169	
0.0001492	1922.	12886018.	9.8889988	42.4521557	
0.0001537	1980.	12886018.	9.8172635	43.4188945	
0.0001582	2038.	12886018.	9.7496274	44.3856333	
0.0001627	2097.	12886018.	9.6857489	45.3523721	
0.0001672	2155.	12886018.	9.6253233	46.3191109	
0.0001718	2213.	12886018.	9.5680779	47.2858497	
0.0001763	2271.	12886018.	9.5137682	48.2525885	
0.0001853	2388.	12884229.	9.4133417	50.0000000	Y
0.0001944	2498.	12850726.	9.3266816	50.0000000	Y
0.0002034	2600.	12785182.	9.2527758	50.0000000	Y
0.0002124	2697.	12695476.	9.1896481	50.0000000	Y
0.0002215	2788.	12588038.	9.1356250	50.0000000	Y
0.0002305	2874.	12468129.	9.0892745	50.0000000	Y
0.0002395	2956.	12339598.	9.0494406	50.0000000	Y
0.0002486	3034.	12203939.	9.0154352	50.0000000	Y
0.0002576	3108.	12065491.	8.9860874	50.0000000	Y
0.0002667	3180.	11924166.	8.9610749	50.0000000	Y
0.0002757	3248.	11782141.	8.9396773	50.0000000	Y
0.0002847	3315.	11640692.	8.9213663	50.0000000	Y
0.0002938	3378.	11498062.	8.9053644	50.0000000	Y
0.0003028	3436.	11346846.	8.8898531	50.0000000	Y
0.0003119	3490.	11189286.	8.8747665	50.0000000	Y
0.0003209	3539.	11027301.	8.8600448	50.0000000	Y
0.0003299	3584.	10862153.	8.8455497	50.0000000	Y
0.0003390	3626.	10695826.	8.8313547	50.0000000	Y
0.0003480	3665.	10529594.	8.8174384	50.0000000	Y
0.0003571	3701.	10364416.	8.8037546	50.0000000	Y
0.0003661	3734.	10200064.	8.7902735	50.0000000	Y
0.0003751	3765.	10037159.	8.7771819	50.0000000	Y
0.0003842	3795.	9877146.	8.7642057	50.0000000	Y

Abutment 1 - HP14x117 Exp. Initial

0.0003932	3822.	9719758.	8.7515922	50.0000000	Y
0.0004023	3848.	9565338.	8.7393796	50.0000000	Y
0.0004113	3872.	9413797.	8.7270278	50.0000000	Y
0.0004203	3894.	9264686.	8.7151807	50.0000000	Y
0.0004294	3916.	9120011.	8.7034122	50.0000000	Y
0.0004384	3936.	8977345.	8.6920428	50.0000000	Y
0.0004475	3955.	8838565.	8.6805923	50.0000000	Y
0.0004565	3973.	8703114.	8.6697822	50.0000000	Y
0.0004655	3990.	8570219.	8.6586971	50.0000000	Y
0.0004746	4006.	8441393.	8.6481131	50.0000000	Y
0.0004836	4021.	8314667.	8.6375687	50.0000000	Y
0.0004927	4036.	8192121.	8.6272271	50.0000000	Y
0.0005017	4049.	8071558.	8.6170671	50.0000000	Y
0.0005107	4063.	7954696.	8.6072785	50.0000000	Y
0.0005198	4075.	7840322.	8.5971758	50.0000000	Y
0.0005288	4087.	7729135.	8.5878539	50.0000000	Y
0.0005379	4098.	7619885.	8.5782352	50.0000000	Y
0.0005740	4139.	7211330.	8.5420544	50.0000000	Y
0.0006102	4174.	6839857.	8.5077578	50.0000000	Y
0.0006463	4203.	6502548.	8.4757838	50.0000000	Y
0.0006825	4228.	6195187.	8.4456503	50.0000000	Y
0.0007186	4250.	5914065.	8.4171823	50.0000000	Y
0.0007548	4269.	5655959.	8.3904513	50.0000000	Y
0.0007910	4286.	5418870.	8.3647287	50.0000000	Y
0.0008271	4301.	5199854.	8.3408628	50.0000000	Y
0.0008633	4314.	4997779.	8.3178990	50.0000000	Y
0.0008994	4327.	4810325.	8.2965344	50.0000000	Y
0.0009356	4337.	4635770.	8.2756913	50.0000000	Y
0.0009718	4347.	4473657.	8.2558994	50.0000000	Y
0.0010079	4356.	4321867.	8.2373514	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
----------	-------------------	---------------------------------

1

360.0000000000

Abutment 1 - HP14x117 Exp. Initial
4356.

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	-9.7000	0.00	N.A.	No	0.00	277643.
2	5.0000	13.4609	Yes	No	277643.	374883.
3	10.0000	12.4645	Yes	No	652526.	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

Abutment 1 - HP14x117 Exp. Initial
 for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 0.300000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 360000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3000	-3130733.	108940.	0.00	63038.	1.20E+10	-1297.	6486.	0.00
0.2500	0.2988	-2809637.	104857.	-7.41E-04	57655.	1.20E+10	-1356.	13617.	0.00
0.5000	0.2956	-2499990.	100707.	-0.00138	52463.	1.28E+10	-1410.	14314.	0.00
0.7500	0.2905	-2202405.	96403.	-0.00193	47474.	1.29E+10	-1459.	15065.	0.00
1.0000	0.2840	-1917397.	91961.	-0.00241	42696.	1.29E+10	-1502.	15873.	0.00
1.2500	0.2761	-1645428.	87396.	-0.00283	38136.	1.29E+10	-1541.	16744.	0.00
1.5000	0.2670	-1386913.	82725.	-0.00318	33802.	1.29E+10	-1574.	17683.	0.00
1.7500	0.2570	-1142212.	77962.	-0.00347	29700.	1.29E+10	-1602.	18698.	0.00
2.0000	0.2462	-911640.	73122.	-0.00371	25834.	1.29E+10	-1625.	19799.	0.00
2.2500	0.2347	-695459.	68222.	-0.00390	22210.	1.29E+10	-1642.	20989.	0.00
2.5000	0.2228	-493881.	63279.	-0.00404	18830.	1.29E+10	-1654.	22272.	0.00
2.7500	0.2105	-307064.	58308.	-0.00413	15698.	1.29E+10	-1660.	23659.	0.00
3.0000	0.1980	-135108.	53328.	-0.00418	12815.	1.29E+10	-1660.	25158.	0.00
3.2500	0.1854	21941.	48356.	-0.00420	10918.	1.29E+10	-1655.	26781.	0.00
3.5000	0.1728	164090.	43408.	-0.00417	13301.	1.29E+10	-1655.	28451.	0.00
3.7500	0.1603	291403.	38501.	-0.00412	15435.	1.29E+10	-1655.	30171.	0.00
4.0000	0.1481	403997.	33651.	-0.00404	17323.	1.29E+10	-1605.	32529.	0.00
4.2500	0.1361	502040.	28876.	-0.00394	18967.	1.29E+10	-1578.	34794.	0.00
4.5000	0.1245	585752.	24190.	-0.00381	20370.	1.29E+10	-1546.	37267.	0.00
4.7500	0.1132	655404.	19607.	-0.00366	21538.	1.29E+10	-1509.	39974.	0.00
5.0000	0.1025	711311.	15284.	-0.00350	22475.	1.29E+10	-1374.	40219.	0.00
5.2500	0.09220	754677.	11338.	-0.00333	23202.	1.29E+10	-1257.	40903.	0.00
5.5000	0.08246	786539.	7737.	-0.00315	23737.	1.29E+10	-1143.	41587.	0.00
5.7500	0.07327	807915.	4474.	-0.00297	24095.	1.29E+10	-1032.	42271.	0.00
6.0000	0.06465	819796.	1537.	-0.00278	24294.	1.29E+10	-925.6155	42955.	0.00
6.2500	0.05659	823140.	-1086.	-0.00259	24350.	1.29E+10	-823.2094	43639.	0.00
6.5000	0.04911	818869.	-3410.	-0.00240	24279.	1.29E+10	-725.6250	44323.	0.00
6.7500	0.04221	807861.	-5448.	-0.00221	24094.	1.29E+10	-633.2107	45007.	0.00
7.0000	0.03587	790951.	-7217.	-0.00202	23811.	1.29E+10	-546.2406	45691.	0.00

$L1 = [135,108 / ((135,108 + 21,941) / (3.25 - 3.0))] + 3.0 = 3.2 \text{ FT}$

Abutment 1 - HP14x117 Exp. Initial

7.2500	0.03008	768926.	-8734.	-0.00184	23441.	1.29E+10	-464.9181	46375.	0.00
7.5000	0.02482	742524.	-10015.	-0.00166	22999.	1.29E+10	-389.3798	47059.	0.00
7.7500	0.02009	712430.	-11079.	-0.00150	22494.	1.29E+10	-319.6995	47743.	0.00
8.0000	0.01585	679280.	-11942.	-0.00133	21938.	1.29E+10	-255.8926	48427.	0.00
8.2500	0.01209	643656.	-12623.	-0.00118	21341.	1.29E+10	-197.9206	49111.	0.00
8.5000	0.00878	606089.	-13138.	-0.00103	20711.	1.29E+10	-145.6948	49795.	0.00
8.7500	0.00589	567059.	-13506.	-8.97E-04	20057.	1.29E+10	-99.0813	50479.	0.00
9.0000	0.00340	526994.	-13741.	-7.70E-04	19385.	1.29E+10	-57.9048	51163.	0.00
9.2500	0.00127	486275.	-13861.	-6.52E-04	18702.	1.29E+10	-21.9526	51847.	0.00
9.5000	-5.15E-04	445237.	-13880.	-5.43E-04	18014.	1.29E+10	9.0217	52531.	0.00
9.7500	-0.00199	404167.	-13814.	-4.44E-04	17326.	1.29E+10	35.2940	53215.	0.00
10.0000	-0.00318	363314.	-13553.	-3.55E-04	16641.	1.29E+10	138.4047	130493.	0.00
10.2500	-0.00412	323615.	-13073.	-2.75E-04	15975.	1.29E+10	181.4992	132149.	0.00
10.5000	-0.00483	285469.	-12478.	-2.04E-04	15336.	1.29E+10	215.5487	133805.	0.00
10.7500	-0.00535	249190.	-11792.	-1.42E-04	14728.	1.29E+10	241.3820	135461.	0.00
11.0000	-0.00568	215021.	-11041.	-8.80E-05	14155.	1.29E+10	259.8271	137117.	0.00
11.2500	-0.00587	183137.	-10243.	-4.16E-05	13620.	1.29E+10	271.6995	138773.	0.00
11.5000	-0.00593	153652.	-9419.	-2.42E-06	13126.	1.29E+10	277.7930	140429.	0.00
11.7500	-0.00589	126628.	-8584.	3.02E-05	12673.	1.29E+10	278.8712	142085.	0.00
12.0000	-0.00575	102082.	-7752.	5.68E-05	12261.	1.29E+10	275.6606	143741.	0.00
12.2500	-0.00555	79992.	-6935.	7.80E-05	11891.	1.29E+10	268.8457	145397.	0.00
12.5000	-0.00529	60301.	-6144.	9.44E-05	11561.	1.29E+10	259.0646	147053.	0.00
12.7500	-0.00498	42927.	-5385.	1.06E-04	11269.	1.29E+10	246.9066	148709.	0.00
13.0000	-0.00465	27763.	-4665.	1.15E-04	11015.	1.29E+10	232.9102	150365.	0.00
13.2500	-0.00429	14690.	-3989.	1.20E-04	10796.	1.29E+10	217.5622	152021.	0.00
13.5000	-0.00393	3570.	-3361.	1.22E-04	10610.	1.29E+10	201.2985	153677.	0.00
13.7500	-0.00356	-5739.	-2782.	1.21E-04	10646.	1.29E+10	184.5041	155333.	0.00
14.0000	-0.00320	-13386.	-2254.	1.19E-04	10774.	1.29			
14.2500	-0.00285	-19521.	-1777.	1.15E-04	10877.	1.29			
14.5000	-0.00251	-24297.	-1350.	1.10E-04	10957.	1.29E+10	154.0047	100501.	0.00
14.7500	-0.00219	-27859.	-971.7753	1.04E-04	11017.	1.29E+10	118.0508	161957.	0.00
15.0000	-0.00188	-30352.	-640.5859	9.74E-05	11059.	1.29E+10	102.7421	163613.	0.00
15.2500	-0.00160	-31913.	-354.0723	9.02E-05	11085.	1.29E+10	88.2669	165269.	0.00
15.5000	-0.00134	-32671.	-109.5903	8.26E-05	11098.	1.29E+10	74.7210	166925.	0.00
15.7500	-0.00111	-32749.	95.7479	7.50E-05	11099.	1.29E+10	62.1711	168581.	0.00
16.0000	-8.93E-04	-32259.	264.9915	6.75E-05	11091.	1.29E+10	50.6580	170237.	0.00
16.2500	-7.02E-04	-31305.	401.2784	6.01E-05	11075.	1.29E+10	40.1999	171893.	0.00
16.5000	-5.32E-04	-29981.	507.7719	5.29E-05	11052.	1.29E+10	30.7957	173549.	0.00
16.7500	-3.84E-04	-28373.	587.6069	4.61E-05	11025.	1.29E+10	22.4276	175205.	0.00
17.0000	-2.56E-04	-26555.	643.8442	3.97E-05	10995.	1.29E+10	15.0639	176861.	0.00
17.2500	-1.46E-04	-24595.	679.4332	3.38E-05	10962.	1.29E+10	8.6621	178517.	0.00

$$L2 = [3,570 / ((3,570 + 5,739) / (13.75 - 13.5))] + 13.5 - 3.2 = 10.4 \text{ FT}$$

DEPTH TO
FIXITY =
17.7 FT

Abutment 1 - HP14x117 Exp. Initial

17.5000	-5.28E-05	-22551.	697.1821	2.83E-05	10928.	1.29E+10	3.1705	180173.	0.00
17.7500	2.42E-05	-20473.	699.7346	2.33E-05	10893.	1.29E+10	-1.4689	181829.	0.00
18.0000	8.70E-05	-18403.	689.5533	1.88E-05	10858.	1.29E+10	-5.3187	183485.	0.00
18.2500	1.37E-04	-16377.	668.9085	1.47E-05	10824.	1.29E+10	-8.4445	185141.	0.00
18.5000	1.75E-04	-14422.	639.8718	1.11E-05	10792.	1.29E+10	-10.9132	186797.	0.00
18.7500	2.04E-04	-12561.	604.3145	7.99E-06	10760.	1.29E+10	-12.7916	188453.	0.00
19.0000	2.23E-04	-10813.	563.9090	5.27E-06	10731.	1.29E+10	-14.1454	190109.	0.00
19.2500	2.35E-04	-9189.	520.1338	2.94E-06	10704.	1.29E+10	-15.0380	191765.	0.00
19.5000	2.41E-04	-7699.	474.2815	9.77E-07	10679.	1.29E+10	-15.5302	193421.	0.00
19.7500	2.41E-04	-6346.	427.4680	-6.58E-07	10656.	1.29E+10	-15.6788	195077.	0.00
20.0000	2.37E-04	-5132.	380.6440	-1.99E-06	10636.	1.29E+10	-15.5372	196733.	0.00
20.2500	2.29E-04	-4058.	334.6076	-3.06E-06	10618.	1.29E+10	-15.1538	198389.	0.00
20.5000	2.19E-04	-3118.	290.0177	-3.90E-06	10602.	1.29E+10	-14.5729	200045.	0.00
20.7500	2.06E-04	-2309.	247.4077	-4.53E-06	10588.	1.29E+10	-13.8338	201701.	0.00
21.0000	1.91E-04	-1624.	207.2001	-4.99E-06	10577.	1.29E+10	-12.9713	203357.	0.00
21.2500	1.76E-04	-1055.	169.7201	-5.30E-06	10567.	1.29E+10	-12.0154	205013.	0.00
21.5000	1.60E-04	-594.1641	135.2097	-5.49E-06	10560.	1.29E+10	-10.9916	206669.	0.00
21.7500	1.43E-04	-231.9901	103.8410	-5.59E-06	10554.	1.29E+10	-9.9209	208325.	0.00
22.0000	1.26E-04	40.9536	75.7287	-5.61E-06	10550.	1.29E+10	-8.8206	209981.	0.00
22.2500	1.09E-04	234.5018	50.9423	-5.58E-06	10554.	1.29E+10	-7.7036	211637.	0.00
22.5000	9.25E-05	358.6584	29.5172	-5.51E-06	10556.	1.29E+10	-6.5798	213293.	0.00
22.7500	7.61E-05	423.5068	11.4644	-5.42E-06	10557.	1.29E+10	-5.4555	214949.	0.00
23.0000	6.00E-05	439.1496	-3.2204	-5.32E-06	10557.	1.29E+10	-4.3344	216605.	0.00
23.2500	4.42E-05	415.6725	-14.5488	-5.22E-06	10557.	1.29E+10	-3.2179	218261.	0.00
23.5000	2.87E-05	363.1302	-22.5333	-5.13E-06	10556.	1.29E+10	-2.1052	219917.	0.00
23.7500	1.35E-05	291.5500	-27.1821	-5.05E-06	10555.	1.29E+10	-0.9941	221573.	0.00
24.0000	-1.60E-06	210.9500	-28.4952	-4.99E-06	10553.	1.29E+10	0.1187	223229.	0.00
24.2500	-1.65E-05	131.3654	-26.4615	-4.95E-06	10552.	1.29E+10	1.2371	224885.	0.00
24.5000	-3.13E-05	62.8814	-21.0584	-4.93E-06	10551.	1.29E+10	2.3650	226541.	0.00
24.7500	-4.61E-05	15.6662	-12.2522	-4.92E-06	10550.	1.29E+10	3.5059	228197.	0.00
25.0000	-6.09E-05	0.00	0.00	-4.92E-06	10550.	1.29E+10	4.6622	114926.	0.00

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

Abutment 1 - HP14x117 Exp. Initial

Pile-head deflection = 0.30000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -3130733. inch-lbs
 Maximum shear force = 108940. 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 = 8
 Number of zero deflection points = 3

Summary of Pile-head Responses for Conventional Analyses

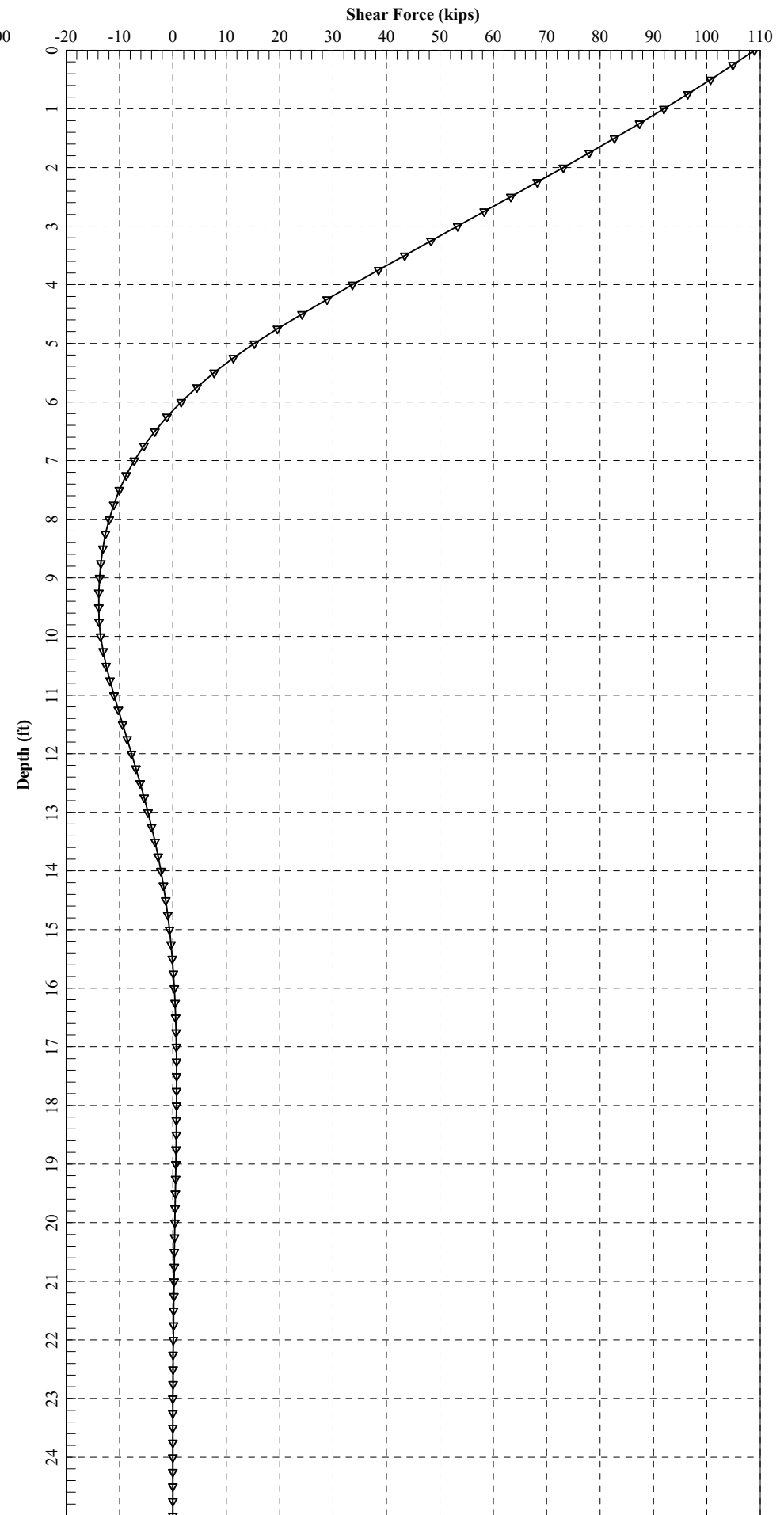
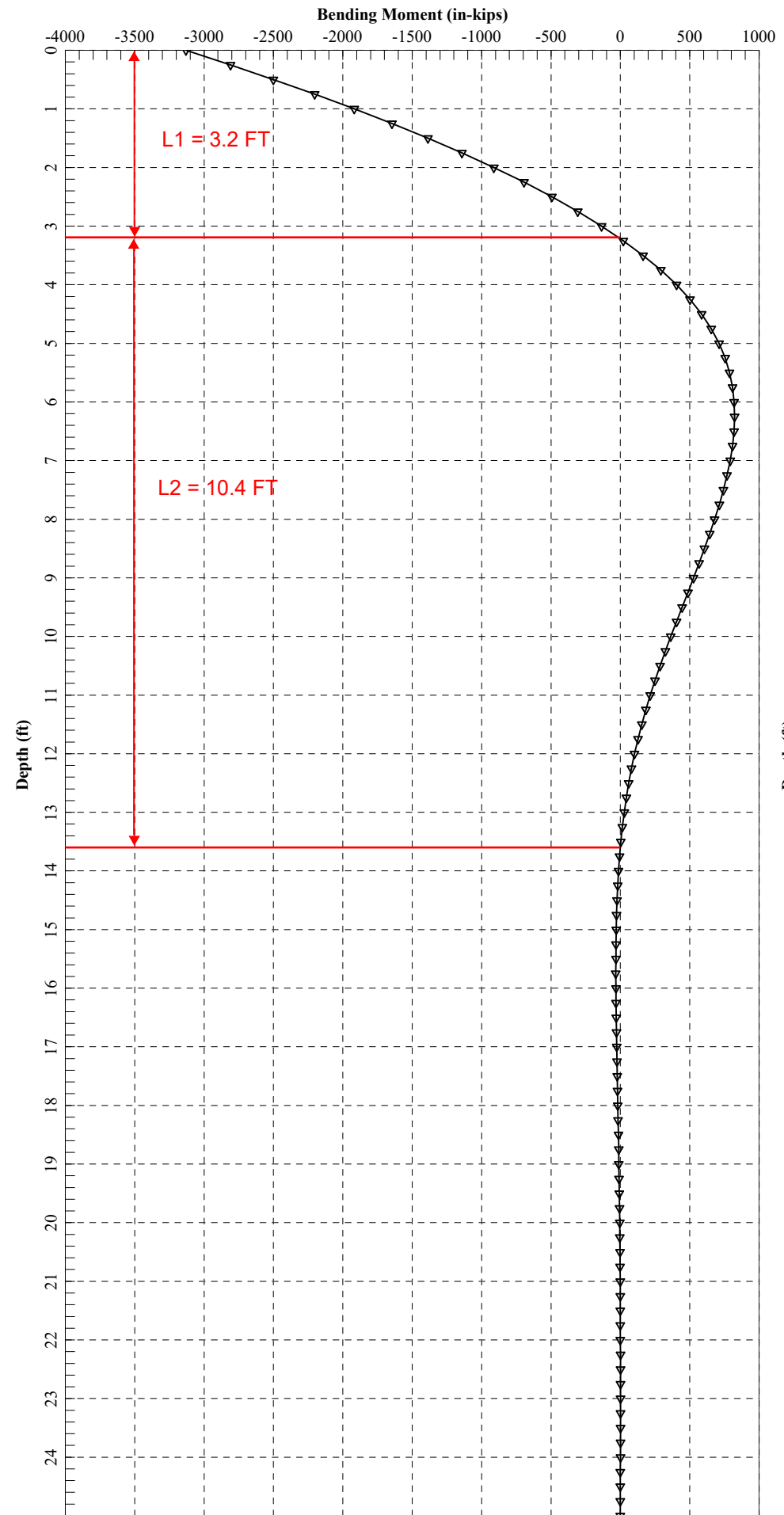
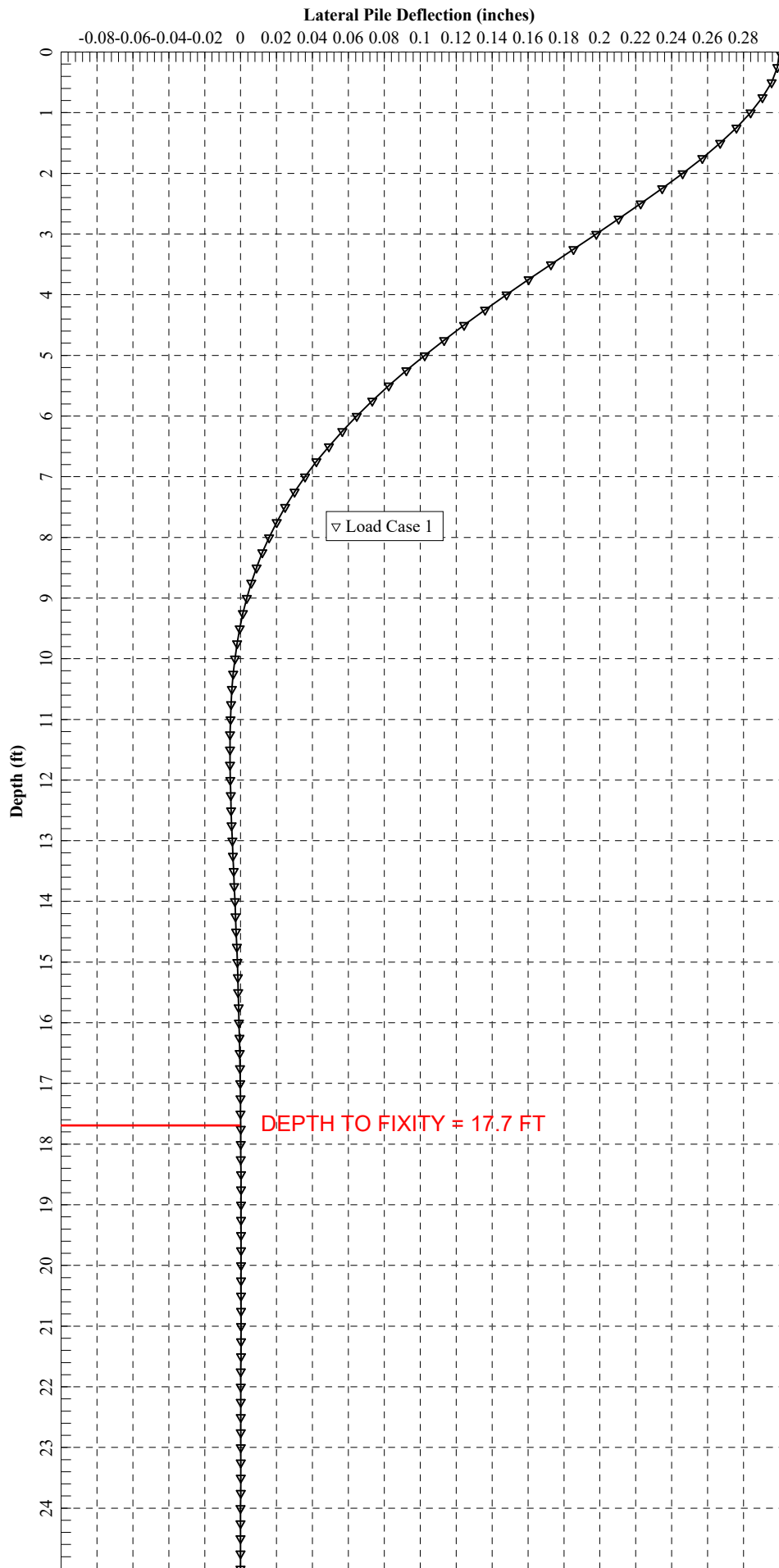
Definitions of Pile-head Loading Conditions:

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	0.3000	S, rad	0.00	360000.	0.3000	0.00	108940.	-3130733.

Maximum pile-head deflection = 0.3000000000 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.



Abutment No. 1
Elastic Pile Compression

File No.	205793-000
Sheet	1 of 1
Date	3-Mar-23
Computed by	NAS
Checked by	EMH

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
Subject	Elastic Compression for Piles

PROBLEM STATEMENT & OBJECTIVE

Estimate settlement under service vertical loads for substructure foundation piles.

EXECUTIVE SUMMARY

The total elastic settlement for a single HP14x89 pile at Abutment No. 1 is = 0.14 in.
 The total elastic settlement for a single HP14x117 pile at Abutment No. 1 is = 0.10 in.

ASSUMPTIONS

1. The H-Pile section are HP14x89 and HP14x117 with no corrosion loss.
2. Top of pile elevation will be taken as the elevation at the bottom of the pile cap at each abutment.
3. Top of bedrock depth per test boring BB-WPMS-201 drilled at Abutment No. 1.
4. Assume pile does elastically compress below top of bedrock.

AVAILABLE INFORMATION:

1. Haley & Aldrich test boring logs BB-WPMS-101 and BB-WPMS-201.
2. Emails from MaineDOT with maximum (unfactored) service axial load of 244 kips.

PROCEDURE

$$\delta = PL/AE$$

δ = settlement
 P = vertical service load on pile
 L = length of pile
 A = cross sectional area of pile
 E = elastic modulus of pile

CALCULATIONS:

	HP14x89	HP14x117	
Max Vertical Service Load, P:	244.0	244.0	kips
Elevation of Top of Pile:	318.6	318.6	ft
Elevation of Tip of Pile:	288.0	288.0	ft
Length, L:	35.0	35.0	ft
Area of Pile, A:	26.1	34.4	in ²
Elastic Modulus of Pile, E:	29,000	29,000	ksi
Total Settlement of Pile, δ_{total}:	0.14	0.10	in.

**Abutment No. 2
Bearing Resistance**

File No.	205793-000
Sheet	1 of 8
Date	17-Nov-22
Computed by	BCS
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00
Subject	Bearing Resistance of Bedrock for Abutment 2 Footing

PROBLEM STATEMENT & OBJECTIVE

Calculate the factored bearing resistance at the service, strength and extreme limit states for the proposed Abutment 2 footing bearing on bedrock.

EXECUTIVE SUMMARY

A factored bearing resistance of	47	ksf for the strength limit state is recommended.
A factored bearing resistance of	20	ksf for the service limit state is recommended.
A factored bearing resistance of	84	ksf for the extreme event limit state is recommended.

AVAILABLE INFORMATION

1. Phase I (-100 series) test boring logs dated 26 June 2018 drilled by MaineDOT.
2. Bottom of Abutment 2 elevation (El. 316.6) provided by MaineDOT.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. NCHRP Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, 2010.
3. The Hoek-Brown Failure Criterion, 1988.

ASSUMPTIONS

1. Peak compressive strength of bedrock from published data for Shale bedrock present at Abutment 2.

Bowles, "Foundation Analysis and Design," Table 4-11, 5th Edition, 1996 (see attached)

Low End of Range =	1,015	psi
High End of Range =	5,802	psi

Hoek, "Rock Engineering", Table 11.2 (see attached)

Low End of Range =	3,626	psi
High End of Range =	7,252	psi

Average Low End of Range =	2,321	psi
Average High End of Range =	6,527	psi

Average of Low and High End Averages =	4,424	psi
	637	ksf

File No.	205793-000
Sheet	2 of 8
Date	17-Nov-22
Computed by	BCS
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00
Subject	Bearing Resistance of Bedrock for Abutment 2 Footing

PROCEDURE FOR STRENGTH LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2020:

10.6.3.2 - Bearing Resistance of Rock

10.6.3.2.1 - General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profile, and other similar profiles as they apply at a particular site. For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock, and the orientation and condition of the discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating.

10.6.3.2.2 Semiempirical Procedures

The nominal bearing resistance of rock should be determined using empirical correlation with the Geometrics Rock Mass Rating system. Local experience shall be considered in the use of these semi-empirical procedures. The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

C10.6.3.2.2

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on the rock mass quality measured in terms of RMR system, the nominal bearing resistance of a rock mass varies from small fraction to six times the unconfined compressive strength of intact rock core samples.

2. See the nominal bearing resistance equation based on Carter and Kulhawy (1988) From NCHRP Report 651:

= + Equation 82b; an errata to Carter and Kulhawy 1988

3. Determine the Rock Mass Ratio (RMR) and strength parameters s and m from NCHRP Report 651 to be used in Equation 82b:

RMR from Table 15 and Table 16

m and s from Hoek-Brown Failure Criterion

4. Apply resistance factor ϕ from Table 10.5.5.2.2-1 in AASHTO LRFD 2020 for bearing resistance of footings on rock

File No.	205793-000
Sheet	3 of 8
Date	17-Nov-22
Computed by	BCS
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00
Subject	Bearing Resistance of Bedrock for Abutment 2 Footing

PROCEDURE FOR SERVICE LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2020:

10.6.2.5 - Bearing Resistance at the Service Limit State

10.6.2.5.1 - Presumptive Values for Bearing Resistance

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

See Table C10.6.2.5.1-1 Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

2. Use AASHTO LRFD 2020 presumptive bearing resistance for service limit state for settlement stated.

PROCEDURE FOR EXTREME EVENT LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2020:

11.5.8 - Resistance Factors for Extreme Event Limit state

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state. For overall stability of the retaining wall when earthquake loading is included, a resistance factor, ϕ , of 0.9 shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSE Walls.

2. Use nominal resistance calculated bearing resistance for the Strength Limit State and apply a resistance factor of 0.8 from AASHTO LRFD 2020 Section 11.5.8 to obtain the factored resistance.

File No.	205793-000
Sheet	4 of 8
Date	17-Nov-22
Computed by	BCS
Checked by	NAS

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00
Subject	Bearing Resistance of Bedrock for Abutment 2 Footing

SUMMARY OF BEDROCK DATA AT SITE: ABUTMENT 2

Abutment No.	Test Boring No.	Ground Surface Elevation	Rock Core No.	Avg. Rock Core Depth (BGS)	Avg. Rock Core Elevation	Depth of Rock Core Below Ftg. Bearing Level (ft)	Rock Core Run Recovery (%)	Rock Quality Designation (RQD, %)
2	BB-WPMS-102	328.4	R1	18.0	310.4	6.2	100	100
			R2	23.0	305.4	11.2	100	93
	BB-WPMS-103	328.5	R1	14.2	314.3	2.3	94	42
			R2	18.7	309.8	6.8	81	50

Because of the difference in measured RQD between test borings BB-WPMS-102 and BB-WPMS-103, calculate the average RQD from each test boring for use in calculating the nominal bearing resistance.

Use average RQD from each test boring:

BB-WPMS-102 =	97	%
BB-WPMS-103 =	46	%

AVAILABLE LABORATORY TEST DATA

Abutment No.	Test Boring No.	Ground Surface Elevation	Rock Core No.	Avg. Rock Specimen Depth (ft BGS)	Avg. Rock Specimen Elevation	Depth of Specimen Below Footing (ft)	Peak Compressive Strength (psi)	Failure Type
2	BB-WPMS-102	328.4	R1	19.9	308.5	8.1	9,861	intact & discontinuity
			R2	23.8	304.6	12.0	6,308	

Average peak compressive strength from laboratory test results =	8,085	psi
Average of "Low and High End Averages" from published data =	4,424	psi

Use average peak compressive strength from laboratory test results =	8,085	psi
	1,164	ksf

Client Maine Department of Transportation

Date 17-Nov-22

Project Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00

Computed by BCS

Subject Bearing Resistance of Bedrock for Abutment 2 Footing

Checked by NAS

Strength Limit State

Similar to the RQD, determine RMR for rock core samples collected from each test boring (BB-WPMS-102 and BB-WPMS-103).

Table 15 from NCHRP Report 651:

PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	>175 ksf	85-175 ksf	45-85 ksf	20-45 ksf	For this low range, unconfined compressive test is preferred		
	Unconfined compressive strength	>4,320 ksf	2,160-4,320 ksf	1,080-2,160 ksf	520-1,080 ksf	215-520 ksf	70-215 ksf	
	Relative Rating	15	12	7	4	2	1	
2	Drill core quality RQD	90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating	20	17	13	8	3		
3	Spacing of joints	>10 ft	3-10 ft	1-3 ft	2 in-1 ft	<2 in		
	Relative Rating	30	25	20	10	5		
4	Condition of joints	<ul style="list-style-type: none"> Very rough surfaces Not continuous No separation Hard joint wall rock 	<ul style="list-style-type: none"> Slightly rough surfaces Separation <0.05 in Hard joint wall rock 	<ul style="list-style-type: none"> Slightly rough surfaces Separation <0.05 in Soft joint wall rock 	<ul style="list-style-type: none"> Slickensided surfaces or gouge <0.2 in thick or joints open 0.05-0.2 in Continuous joints 	<ul style="list-style-type: none"> Soft gouge >0.2 in thick or joints open >0.2 in Continuous joints 		
	Relative Rating	25	20	12	6	0		
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal/hr	400-2,000 gal/hr	>2,000 gal/hr		
		Ratio = joint water pressure/major principal stress	0	0.0-0.2	0.2-0.5	>0.5		
	General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating	10	7	4	0			

Table 16 from NCHRP Report 651:

Strike and dip orientations of joints	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Tunnels	0	-2	-5	-10	-12
Foundations	0	-2	-7	-15	-25
Slopes	0	-5	-25	-50	-60

Total RMR Value

Parameter	Design Value (-102)	Relative Rating	Design Value (-103)	Relative Rating
Intact Rock Strength	1164 ksf	7	1164 ksf	7
RQD	97%	20	46%	8
Joint Spacing	1 ft to 3 ft (observed in photos)	20	2 in to 1 ft (observed in photos)	10
Joint Condition	Slightly rough surfaces separation <0.05 in (observed in photos)	20	Slightly rough surfaces separation <0.05 in (observed in photos)	20
Groundwater Condition	Moist only (interstitial water)	7	Moist only (interstitial water)	7
Joint Strike and Dip	Fair	-7	Fair	-7
Total Rating =		67	Total Rating =	45
		BB-WPMS-102		BB-WPMS-103

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00
Subject	Bearing Resistance of Bedrock for Abutment 2 Footing

Strength Limit State Continued

Determine s and m for each test boring assuming rock type B.

Table 17 from NCHRP Report 651:

RMR rating	100-81	80-61	60-41	40-21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

BB-WPMS-102 BB-WPMS-103

Table 19 from NCHRP Report 651:

Rock quality	Constants	Rock type				
		A	B	C	D	E
INTACT ROCK LABORATORY size specimens free from discontinuities. CSIR rating: <i>RMR</i> = 100	m s	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3-10 ft. CSIR rating: <i>RMR</i> = 85	m s	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3-10 ft. CSIR rating: <i>RMR</i> = 65	m s	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1-3 ft. CSIR rating: <i>RMR</i> = 44	m s	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in; some gouge. Clean compacted waste rock. CSIR rating: <i>RMR</i> = 23	m s	0.029 3 x 10 ⁻⁶	0.041 3 x 10 ⁻⁶	0.061 3 x 10 ⁻⁶	0.069 3 x 10 ⁻⁶	0.102 3 x 10 ⁻⁶
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced < 2 in with gouge. Waste rock with fines. CSIR rating: <i>RMR</i> = 3	m s	0.007 1 x 10 ⁻⁷	0.010 1 x 10 ⁻⁷	0.015 1 x 10 ⁻⁷	0.017 1 x 10 ⁻⁷	0.025 1 x 10 ⁻⁷

Values of m and s from Hoek-Brown 1988:

$$\frac{m}{m_1} = e^{\left(\frac{RMR-100}{14}\right)} \quad \text{Equation 18}$$

m₁ is the value of m for *intact* rock

$$s = e^{\left(\frac{RMR-100}{6}\right)} \quad \text{Equation 19}$$

Test Boring BB-WPMS-102:

Rock Quality	Rock Type	RMR	m ₁	m	s
Good	B	67	0.82	7.77E-02	4.09E-03

Test Boring BB-WPMS-103:

Rock Quality	Rock Type	RMR	m ₁	m	s
Fair	B	45	0.18	3.60E-03	1.04E-04

Client	Maine Department of Transportation
Project	Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00
Subject	Bearing Resistance of Bedrock for Abutment 2 Footing

Strength Limit State Continued

Semi-empirical method by Carter and Kulhawy 1988:

	BB-WPMS-102		BB-WPMS-103		
$q_u =$	8,085	psi	8,085	psi	
$m =$	0.078		3.60E-03		
$s =$	4.09E-03		1.04E-04		
$q_{ult} =$	185.2	ksf	25.7	ksf	Equation 82b
$\phi =$	0.45		0.45		from AASHTO LRFD 2020 Table 10.5.5.2.2-1
$q_R =$	83	ksf	12	ksf	Equation 82b

Use the avg. factored bearing resist. from test borings BB-WPMS-102 and BB-WPMS-103... 47 ksf

Service Limit State

Based on AASHTO LRFD 2020 Table C10.6.2.6.1-1 and engineering judgement the service limit state for bearing resistance on bedrock is 20 ksf

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120-200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60-80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30-50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16-24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16-24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16-24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12-20	14
	Medium dense to dense	8-14	10
	Loose	4-12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8-12	8
	Medium dense to dense	4-8	6
	Loose	2-6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6-10	6
	Medium dense to dense	4-8	5
	Loose	2-4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6-10	6
	Medium dense to dense	4-8	5
	Loose	2-4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6-12	8
	Medium dense to dense	2-6	4
	Loose	1-2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4-8	6
	Medium stiff to stiff	2-6	3
	Soft	1-2	1

Extreme Event Limit State

From the Strength Limit State calculations, the nominal bearing resistance is the following:

$q_{ult} =$ 105 ksf (average of BB-WPMS-102 and BB-WPMS-103)

Using a resistance factor of 0.8 from AASHTO LRFD 2020 Section 11.5.8, the factored bearing resistance is the following:

$q_R =$ 84 ksf

Client Maine Department of Transportation

Date 17-Nov-22

Project Mattagodus Stream Bridge over Mattagodus Stream No. 5116, MaineDOT WIN 022266.00

Computed by BCS

Subject Bearing Resistance of Bedrock for Abutment 2 Footing

Checked by NAS

CONCLUSIONS AND RECOMMENDATIONS**Strength Limit State**The recommended factored bearing resistance for the strength limit state is **47** ksf**Service Limit State**The recommended presumptive bearing resistance for the service limit state is **20** ksf**Extreme Event Limit State**The recommended factored bearing resistance for the extreme event limit state is **84** ksf

BOWLES
"FOUNDATION ANALYSIS & DESIGN"
5TH EDITION 1996

TABLE 4-11
Range of properties for selected rock groups; data from several sources

Type of rock	Typical unit wt., kN/m ³	Modulus of elasticity E , MPa $\times 10^3$	Poisson's ratio, μ	Compressive strength, MPa
Basalt	28	17-103	0.27-0.32	170-415
Granite	26.4	14-83	0.26-0.30	70-276
Schist	26	7-83	0.18-0.22	35-105
Limestone	26	21-103	0.24-0.45	35-170
Porous limestone		3-83	0.35-0.45	7-35
Sandstone	22.8-23.6	3-42	0.20-0.45	28-138
Shale	15.7-22	3-21	0.25-0.45	7-40
Concrete	15.7-23.6	Variable	0.15	15-40

*Depends heavily on confining pressure and how determined; E = tangent modulus at approximately 50 percent of ultimate compression strength.

the bearing-capacity factors for sound rock are approximately

1,015 - 5,802 psi

$$N_q = \tan^6 \left(45^\circ + \frac{\phi}{2} \right) \quad N_c = 5 \tan^4 \left(45^\circ + \frac{\phi}{2} \right) \quad N_\gamma = N_q + 1 \quad (4-27)$$

Use the Terzaghi shape factors of Table 4-1 with these bearing-capacity factors. The rock angle of internal friction is seldom less than 40° (often 45° to 55°) and rock cohesion ranges from about 3.5 to 17.5 MPa (500 to 2500 psi). It is evident from Eq. (4-27) that very high values of ultimate bearing capacity can be computed. The upper limit on allowable bearing capacity is, as previously stated, taken as f'_c of the base concrete or not more than the allowable bearing pressure of metal piles.

The angle of internal friction of rock is pressure-dependent, similar to soil. Also, inspection of rock parameters from a number of sources indicates that, similar to sand, we could estimate $\phi = 45^\circ$ for most rock except limestone or shale where values between 38° and 45° should be used. Similarly we could in most cases estimate $s_u = 5$ MPa as a conservative value. Finally we may reduce the ultimate bearing capacity based on RQD as

$$q'_{ult} = q_{ult}(RQD)^2$$

In many cases the allowable rock-bearing pressure is taken in the range of one-third to one-tenth the unconfined compression strength obtained from intact rock samples and using RQD as a guide, for example, as one-tenth for a small RQD. Others simply use an allowable bearing pressure from the local building code (as in Table 4-8) based on rock type from a visual inspection of the rock cores.

Few building foundations such as mats or spread bases are placed directly on rock. Most situations involving rock-bearing capacity require large-diameter drilled shafts (termed drilled piers as in Chap. 19), which are socketed 2 to 3 shaft diameters into the rock. Recent load tests on this type of foundation [see Rowe and Armitage (1987)] indicate the allowable bearing pressure is on the order of

$$q_a = q_u \text{ to } 2.5q_u$$

where q_u = unconfined compression strength of intact rock core samples. This value is substantially larger than the values of one-third and one-tenth previously cited. The large increase

Table 11.2: Field estimates of uniaxial compressive strength.

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field estimate of strength	Examples
R6	Extremely Strong	> 250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 - 250	4 - 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, phyllite, sandstone, schist, shale
					<i>7,252 - 14,504 psi</i>
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Claystone, coal, concrete, schist, shale, siltstone
					<i>3,626 - 7,252 psi</i>
R2	Weak	5 - 25	**	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, rocksalt, potash
R1	Very weak	1 - 5	**	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock
R0	Extremely weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

* Grade according to Brown (1981).

** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

Abutment No. 2
Sliding Resistance

File No.	205793-000
Sheet	1 of 1
Date	3-Aug-22
Computed by	SSM
Checked by	NAS/BCS

Client: Maine Department of Transportation
 Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
 Subject: Sliding Resistance for Footing Bearing on Bedrock (Abutment 2)

PROBLEM STATEMENT AND OBJECTIVE

Determine the coefficient of friction between the semi-integral Abutment 2 footing and bedrock, resistance factor for sliding for the Strength Limit State, and resistance factor for sliding for the Extreme Event Limit State.

EXECUTIVE SUMMARY

The coefficient of friction between the footing and bedrock is = **0.70**
 The resistance factor for sliding at the Strength Limit State is = **0.8**
 The resistance factor for sliding at the Service and Extreme Event Limit States is = **1.0**

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.
2. MaineDOT Bridge Design Guide, August 2003, with interim revisions through June 2018.

AVAILABLE INFORMATION

1. Test borings BB-WPMS-102 and BB-WPMS-103 drilled near the southern and northern ends of Abutment 2, respectively.
2. Test borings BB-WPMS-102 and BB-WPMS-103 encountered bedrock at El. 312.9 and EL. 316.6, respectively.

ASSUMPTIONS

1. Preliminary design drawings show a proposed bottom of Abutment 2 footing at El. 316.6. The abutment footing will bear on a sub-footing (concrete fill) bearing directly on bedrock.
2. The rock surface will be prepared in-the-dry and in accordance with Section 5.3.4.2 of the BDG and the Std. Spec's.

CALCULATIONS

Coefficient of Friction Between Concrete and Bedrock

Nominal sliding resistance between the cast-in-place concrete footing/sub-footing and bedrock is dependent on the coefficient of friction ($\tan\delta$) at the interface between the two materials.

Estimated footing-rock interface friction angle (δ):

35 deg., friction angle for mass concrete on clean sound rock (AASHTO LRFD Table C3.11.5.3-1)

Recommended δ = **35** deg., friction angle between footing/seal and bedrock
 Recommended $\tan\delta$ = **0.7** coefficient of friction

Resistance Factors

Service Limit State

AASHTO LRFD Section 10.5.5.1 prescribes a Service Limit State resistance factor equal to = **1.0**

Strength Limit State

AASHTO LRFD does not prescribe a sliding resistance factor for shallow foundations on bedrock.
 For cast-in-place concrete on sand, the sliding resistance factor is = **0.8** (AASHTO LRFD Table 10.5.5.2.2-1)

Extreme Event Limit State

Section 10.5.5.3.3 of AASHTO LRFD prescribes a resistance factor of foundations in the Extreme Event Limit State. **1.0**

Table C3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, $\tan \delta$ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f .

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

practical to design the foundation system to accommodate the lateral loads.

The primary ground improvement techniques to mitigate liquefaction fall into five general categories, namely removal and replacement, densification, reinforcement, altering the soil composition, and enhanced drainage. Any one or a combination of methods can be used. However, drainage improvement is not currently considered adequately reliable to prevent liquefaction-induced, excess pore-water pressure build-up due to (1) the time required for excess pore-water pressures to dissipate through the drainage paths, and (2) the potential for drainage materials to become clogged during installation and in service. In addition, with drainage enhancements some settlement is still likely. Therefore, drainage enhancements should not be used as a means to fully mitigate liquefaction. For further discussion of ground improvement methods, see FHWA-SA-98-086, *Ground Improvement Technical Summaries* (Elias et al., 2000); FHWA-SA-95-037; Geotechnical Engineering Circular No. 1, *Dynamic Compaction* (Lukas, 1995); and FHWA/RD-83/O2C, *Design and Construction of Stone Columns* (Barkdale and Bachus, 1983).

The use of large diameter shafts in lieu of the conventional pile cap foundation type may be considered in order to achieve the lateral strength and stiffness required to sustain the column demand while minimizing the foundation exposed surface area normal to the lateral flow direction.

10.5.5—Resistance Factors

10.5.5.1—Service Limit States

Resistance factors for the service limit states shall be taken as 1.0.

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after scour due to the design flood.

10.5.5.2—Strength Limit States

10.5.5.2.1—General

Resistance factors for different types of foundation systems at the strength limit state shall be taken as specified in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5, unless regionally specific values or substantial successful experience is available to justify higher values.

For overall stability, resistance factors shall be as specified in Article 11.6.3.7. Overall stability of foundations shall be investigated using Strength I Load Combination and the provisions of Article 3.4.1.

C10.5.5.2.1

Regionally specific values should be determined based on substantial statistical data combined with calibration or substantial successful experience to justify higher values. Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that have not been mitigated through conservative selection of design parameters.

Certain resistance factors in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5 are presented as a function of soil type, e.g., sand or clay. Naturally occurring soils do not fall neatly into these two classifications. In general, the terms “sand” and “cohesionless soil” may be connoted to mean drained conditions during loading, while “clay” or “cohesive soil”

10.5.5.2.2—Spread Footings

C10.5.5.2.2

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site-specific considerations in Article 10.5.5.2.

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

		Method/Soil/Condition	Resistance Factor
Bearing Resistance	ϕ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
		Precast concrete placed on sand	0.90
Sliding	ϕ_τ	Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
		Passive earth pressure component of sliding resistance	0.50
	ϕ_{ep}		

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (Barker et al., 1991). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in Table 10.5.5.2.2-1 is provided in Allen (2005).

The resistance factors for plate load tests and passive resistance were based on engineering judgment and past ASD practice.

10.5.5.2.3—Driven Piles

C10.5.5.2.3

Resistance factors shall be selected from Table 10.5.5.2.3-1 based on the method used for determining the driving criterion necessary to achieve the required nominal pile bearing resistance.

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the design resistance factors selected should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A

Where nominal pile bearing resistance is determined by static load test, dynamic testing, wave equation, or dynamic formulas, the uncertainty in the nominal resistance is strictly due to the reliability of the resistance determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each production pile is field-verified based on compliance with a driving criterion developed using a dynamic method

10.5.5.3.3—Other Extreme Limit States

Resistance factors for extreme limit state, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

C10.5.5.3.3

The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see Article 10.5.5.2.3).

10.6—SPREAD FOOTINGS**10.6.1—General Considerations****10.6.1.1—General**

Provisions of this Article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and other substructure and superstructure elements. Special attention shall be given to footings on fill, to make sure that the quality of the fill placed below the footing is well controlled and of adequate quality in terms of shear strength and compressibility to support the footing loads.

Spread footings shall be proportioned and designed such that the supporting soil or rock provides adequate nominal resistance, considering both the potential for adequate bearing strength and the potential for settlement, under all applicable limit states in accordance with the provisions of this Section.

Spread footings shall be proportioned and located to maintain stability under all applicable limit states, considering the potential for, but not necessarily limited to, overturning (eccentricity), sliding, uplift, overall stability and loss of lateral support.

10.6.1.2—Bearing Depth

Where the potential for scour, erosion or undermining exists, spread footings shall be located to bear below the maximum anticipated depth of scour, erosion, or undermining as specified in Article 2.6.4.4.

C10.6.1.1

Problems with insufficient bearing and/or excessive settlements in fill can be significant, particularly if poor, e.g., soft, wet, frozen, or nondurable, material is used, or if the material is not properly compacted.

Spread footings should not be used on soil or rock conditions that are determined to be too soft or weak to support the design loads without excessive movement or loss of stability. Alternatively, the unsuitable material can be removed and replaced with suitable and properly compacted engineered fill material, or improved in place, at reasonable cost as compared to other foundation support alternatives.

Footings should be proportioned so that the stress under the footing is as nearly uniform as practicable at the service limit state. The distribution of soil stress should be consistent with properties of the soil or rock and the structure and with established principles of soil and rock mechanics.

C10.6.1.2

Consideration should be given to the use of either a geotextile or graded granular filter material to reduce the susceptibility of fine grained material piping into rip rap or open-graded granular foundation material.

For spread footings founded on excavated or blasted rock, attention should be paid to the effect of excavation and/or blasting. Blasting of highly resistant competent rock formations may result in overbreak and fracturing of the rock to some depth below the bearing elevation. Blasting may reduce the resistance to scour within the zone of overbreak or fracturing.

Evaluation of seepage forces and hydraulic gradients should be performed as part of the design of foundations that will extend below the groundwater table. Upward seepage forces in the bottom of excavations can result in piping loss of soil and/or heaving and loss of stability in the base of foundation excavations. Dewatering with wells or wellpoints can control these problems. Dewatering can result in settlement of adjacent ground or structures. If adjacent structures may be damaged by settlement induced by dewatering, seepage cut-off methods such as sheet piling or slurry walls may be necessary.

Abutment No. 2
Lateral Earth Pressures

File No.	205793-000
Sheet	1 of 1
Date	15-Sep-22
Computed by	BCS
Checked by	NAS

Client: Maine Department of Transportation
 Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00
 Subject: Active and Passive Lateral Earth Pressure Coefficients for Abutment 2

PROBLEM STATEMENT & OBJECTIVE

Calculate static active and passive earth pressure coefficients for Abutment 2 breastwall, backwall and wingwalls. Per MaineDOT BDG Section 3.6.4, for walls with a total height greater than 5 ft, the horizontal movement of the top of the wall due to structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions. In addition, MaineDOT BDG Section 5.4.3 requires the use of Rankine Theory when calculating the static active earth pressure coefficients for semi-integral abutments.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.
2. MaineDOT Bridge Design Guide, August 2003, with interim revisions through June 2018.
3. Preliminary design drawings prepared by MaineDOT.

ASSUMPTIONS

1. Abutment breastwalls and wingwalls are backfilled with Granular Borrow.
2. Free draining material used to backfill walls and drains are provided (i.e., no hydrostatic pressures develop).

SOIL PROPERTIES AND FILL/ABUTMENT BREASTWALL GEOMETRY (RANKINE ACTIVE)

designates input cell

Total Unit Weight, γ (pcf) =	125	pcf	Soil Type 4, MaineDOT BDG Table 3-3
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, MaineDOT BDG Table 3-3
Backslope Angle, β =	-0.42	degrees	estimate from MaineDOT roadway profile
Backface of Wall Angle to Horizontal, α =	90	degrees	per preliminary MaineDOT design drawings
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, MaineDOT BDG Table 3-3

Rankine Static Active Lateral Earth Pressure Coefficient, K_a

$$K_a = \cos \beta \cdot \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

MaineDOT BDG Section 3.6.5.2

$K_{a, Rankine} =$ **0.31**

SOIL PROPERTIES AND FILL/ABUTMENT BACKWALL GEOMETRY (COULOMB PASSIVE)

For the proposed abutment system, the backwall/end diaphragm is relatively independent from the cantilever abutment because of the detailing of the elastomeric bearing and sole plate. Because of this, the length of the abutment heel has limited to no influence on the earth pressures engaged by the beam ends and the passive pressure should be based on Coulomb Theory.

Coulomb Static Passive Lateral Earth Pressure Coefficient, K_p

$$K_p = \frac{\sin(\alpha - \phi)^2}{\sin \alpha^2 \cdot \sin(\alpha + \delta) \cdot \left(1 - \frac{\sin(\phi + \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \delta) \cdot \sin(\alpha + \beta)} \right)^2}$$

MaineDOT BDG Section 3.6.6

$K_{p, Coulomb} =$ **8.16**

Frost Penetration

Client: Maine Department of Transportation

Date: 3-Aug-2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00

Computed by: SSM

Subject: Frost Penetration Depth Evaluation

Checked by: NAS

OBJECTIVE:

Evaluate maximum depth of frost penetration based on soil and groundwater conditions, as well as geographic site location.

REFERENCES:

1. MaineDOT Bridge Design Guide, August 2003, with June 2018 updates
2. MaineDOT test borings BB-WPMS-101, BB-WPMS-102, BB-WPMS-103, and BB-WPMS-201.
3. Preliminary Plan set prepared by MaineDOT dated May 2022.

EVALUATION:

1. Gather relevant information from test borings performed near proposed bridge abutment locations:

SUBSTRUCTURE	APPROX. WINGWALL BEARING ELEVATION	TEST BORING NO./GS EL.	GROUND WATER EL.	SAMPLE NO./ ELEVATION	LAB USCS	MOISTURE CONTENT %
ABUTMENT NO. 1	El. 319	BB-WPMS-101/ El. 327.8	308.3	1D/ El. 322.8 - El. 320.8	SM	8.0
		BB-WPMS-101/ El. 327.8	308.3	3D/ El. 312.8 - El. 310.8	SC-SM	16.7
		BB-WPMS-201/ El. 328.2	313.1	No index laboratory testing available.		
ABUTMENT NO. 2	El. 318.5	BB-WPMS-102/ El. 328.4	319.9	1D/ El. 323.4 - El. 321.4	SM	7.0
		BB-WPMS-102/ El. 328.4	319.9	2D/ El. 318.4 - El. 316.4	SM	8.8
		BB-WPMS-103/ El. 328.5	322.0	No index laboratory testing available.		

2. Portions of the wingwall footings will bear in new embankment fill. Assume the new embankment fill consists of granular borrow.

3. Per MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1875°F - days.

4. Estimate range in frost penetration depth using MaineDOT Bridge Design Guide Table 5-1, the design freezing index above, and a "design" moisture content of the coarse- and (relatively) fine-grained soil (soil types ranged from SM to SC-SM) (i.e., assume average of available four moisture content results).

5. For coarse-grained soil with water content of 10 percent (average of four moisture content results from above) , per Table 5-1, maximum depth of frost is approximately 7.7 ft (91.9 in.).
 For fine-grained soil with water content of 10 percent (average of four moisture content results from above) , per Table 5-1, maximum depth of frost is approximately 5.5 ft (65.4 in.).
 Therefore, the average between the two frost (min. and max.) depths above is approximately 6.5 ft.

Recommend wingwall footings be founded at least 6.5 ft below the lowest adjacent ground surface exposed to freezing.

Client: Maine Department of Transportation

Date: 3-Aug-2022

Project: Mattagodus Stream Bridge over Mattagodus Stream No. 5116 - WIN 022266.00

Computed by: SSM

Subject: Frost Penetration Depth Evaluation

Checked by: BCS

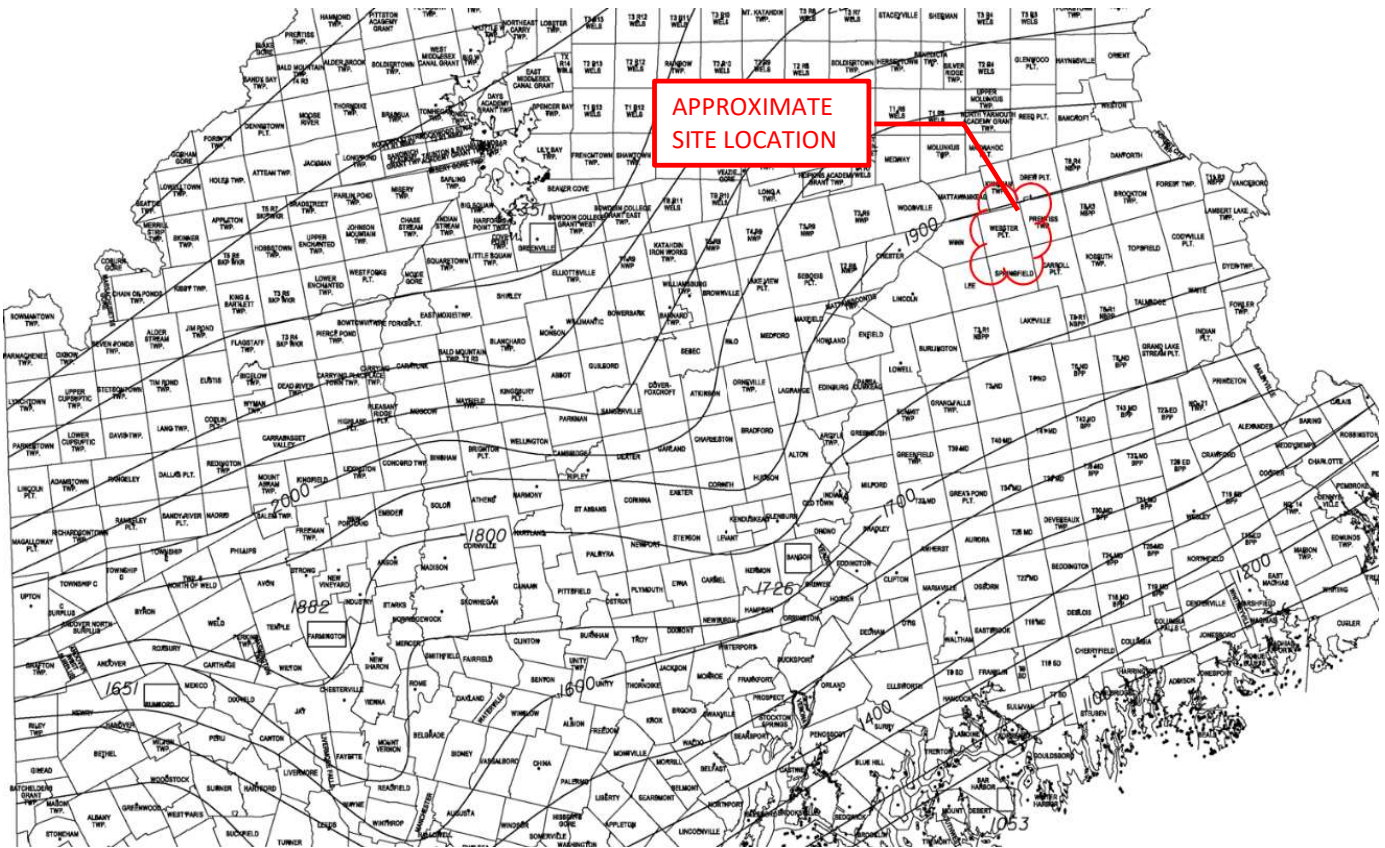


Table 5-1 Depth of Frost Penetration

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	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0