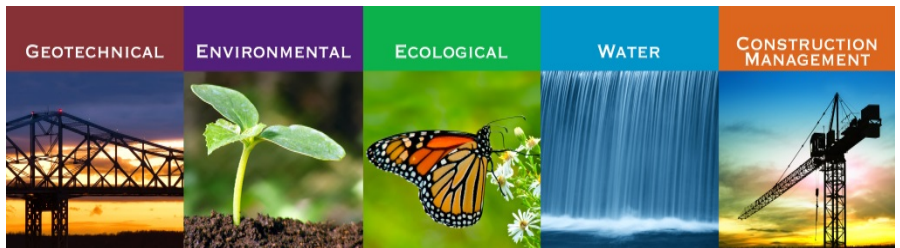




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GEOTECHNICAL DESIGN REPORT LUNT ROAD BRIDGE NO. 5829 OVER INTERSTATE 295 MAINE DOT WIN 21723.00 FALMOUTH, MAINE

Prepared for:
WSP USA
Manchester, New Hampshire

Rev. August 2018
09.0025950.01

Prepared by:
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VIA EMAIL

Rev. August 2, 2018
File No. 09.0025950.01

Mr. Adam Stockin, P.E.
WSP USA
650 Elm Street, 4th Floor
Manchester, New Hampshire 03101

Re: Geotechnical Design Report
Lunt Road Bridge Replacement #5829 over Interstate 295
MaineDOT WIN 21723.00
Falmouth, Maine

Dear Adam:

We are pleased to provide this Geotechnical Design Report (GDR) to WSP USA (WSP) for Bridge No. 5829 Lunt Road over Interstate 295 in Falmouth, Maine. Our work was completed in accordance with the Subconsultant Agreement between WSP and GZA GeoEnvironmental, Inc. (GZA) executed on August 4, 2017, GZA's proposal No. 09.P000036.19, dated May 31, 2018, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to WSP, who is under contract with Maine Department of Transportation for design of the proposed bridge replacement.

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

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Nicholas Williams, E.I.T.
Project Engineer

Andrew R. Blaisdell, P.E.
Consultant Reviewer



Christopher L. Snow, P.E.
Associate Principal

NVW/CLS/ARB:erc

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APPENDIX B	Previous Subsurface Explorations
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APPENDIX D	Laboratory Test Results
APPENDIX E	Rock Core Photographs
APPENDIX F	Calculations



1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the proposed replacement of Maine Department of Transportation (MaineDOT) Lunt Road Bridge No. 5829 over Interstate 295 (I-295) in Falmouth, Maine. Our work was completed in accordance with the Subconsultant Agreement between WSP USA and GZA executed August 4, 2017, which incorporates GZA's proposal No. 09.P000036.19, dated May 31, 2018, and the *Limitations* included in **Appendix A** of this report.

GZA is providing geotechnical engineering services as a Subconsultant to WSP USA, who is under contract with the State of Maine, MaineDOT for design of the proposed bridge.

1.1 BACKGROUND

The Lunt Road Bridge No. 5829 carries Lunt Road over Interstate 295, in Falmouth, Maine, as shown on the *Locus Plan*, **Figure 1**.

The existing bridge was constructed in 1960 and consists of a four-span bridge supported by stub abutments and three central piers. The overall bridge is 216 feet long, with spans of 48, 60, 60, and 48 feet. The abutments are supported on H-Piles and the three piers are supported on spread footings bearing on bedrock. The original ground surface sloped down from west to east along the bridge alignment. The majority of the I-295 section beneath the bridge was excavated between 2 and 25 feet below original grade to construct the roadway. It appears the eastern approach embankment was constructed by excavating approximately 7 feet below the ground surface to remove peat material found in the 1959 borings, and then filling as much as approximately 40 feet above the bottom of the excavation. The western abutment was constructed by excavating approximately 4 to 10 feet deep, and then filling approximately back to grade.

1.2 PROPOSED CONSTRUCTION

The replacement bridge will consist of two 109-foot-long spans extending from approximately Sta. 11+75.12 to Sta. 13+93.12, as shown on **Figure 2, Interpretive Subsurface Profile**. The new bridge is proposed to consist of a concrete superstructure with integral abutment substructures, supported by steel H-piles and a center pier supported by a spread footing. The new abutments will be constructed at the location of the existing abutments, which will be removed. WSP provided an estimated thermal deformation of the bridge superstructure of 1.06 inches at each abutment and a maximum factored axial load of 466 kips per pile.

Proposed pavement grades on Lunt Road will be within approximately 2 feet plus or minus of existing grades. Approach embankment modifications will include widening the top of the embankments by approximately 12 feet and grading side slopes at inclination of 2 horizontal to 1 vertical (2H:1V).

1.3 OBJECTIVES AND SCOPE OF SERVICES

A previous GZA effort was completed in 2017 to support a then-proposed bridge reconstruction. The objectives of our current work are to evaluate subsurface conditions and provide geotechnical design



recommendations and construction considerations for bridge replacement. To meet the current objectives, GZA completed the following Scope of Services:

- Completed geotechnical evaluations for soil and bedrock properties; stability and settlement of approach embankments; frost susceptibility and drainage of approach embankments; AASHTO LRFD load and resistance factors associated with geotechnical design elements; nominal resistance of new pile foundations; and bearing resistance of new spread footing foundations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

2.1 HISTORIC MAINEDOT EXPLORATIONS

Subsurface data from eleven test borings (LR-1 through LR-11) were shown in the 1959 bridge plan set. Two borings were drilled at each abutment, two borings were drilled at each pier location, and one boring was drilled east of the bridge. The plans indicate that subsurface conditions included silty clay and/or silt overlying sand, and bedrock. A surficial layer of organics and peat were encountered at the ground surface in the vicinity of the east abutment.

The subsurface data from the 1959 bridge plans are included in **Appendix B**.

2.2 FINAL DESIGN SUBSURFACE EXPLORATIONS

GZA completed a final design subsurface exploration program consisting of six (6) test borings (BB-LR-101 through -106). The test boring program included four borings drilled with a truck-mounted rig and two drilled with an ATV-mounted rig. One boring was drilled along the I-295 mainline north of each of the existing piers; one on Lunt Road behind each abutment; and one was drilled at the northerly toe of the east approach embankment.

The borings were drilled to depths of approximately 18.0 to 56.2 feet. The borings were cored approximately 10.0 feet into bedrock, except boring BB-LR-106, which was carried to practical refusal. New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. The drilling was completed between August 7, 2017 and August 11, 2017. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix C**.

The borings were drilled using 3- and 4-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with a 140-pound automatic hammer with a 30-inch drop. One thin-wall tube sample was taken for laboratory consolidation testing. Bedrock cores were obtained using NX2 coring equipment. MaineDOT provided calibration data for the automatic hammer No. Truck 83 (Plate # 4368) indicating an efficiency factor of 0.87 and for automatic hammer No. NEBC 2 indicating an efficiency factor of 0.68.

The as-drilled boring locations were taped from existing structures by GZA during the subsurface investigation. These locations are considered accurate to the degree implied by the method used to establish them and are shown on **Figures 2 and 3**. Approximate ground surface elevations were



interpolated between 1-foot contours shown on the plan provided by WSP and should also be considered accurate to the degree implied by the method used to establish them. Elevations referenced in this report are in feet and refer to the North American Vertical Datum of 1988 (NAVD88).

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete a soil and bedrock testing program to assess the gradation and engineering characteristics of the soil and strength of the bedrock. The testing program consisted of eight (8) gradation analysis/AASHTO Classification/Frost Classification assessments, two (2) Atterberg limit evaluations, and six (6) moisture content determinations on split-spoon samples from the borings; and three unconfined compression strength tests with strain measurements on bedrock core samples. Additional testing was performed on silty clay material by R.W. Gillespie and Associates, of Saco Maine, including one (1) one-dimensional consolidation test, one (1) set of Atterberg limits, and one (1) lab vane shear test on soil from the thin-wall tube sample. Results of the testing are included in **Appendix D**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Surficial geologic units mapped in the Lunt Road Bridge area include fine-grained, gray to blackish-gray, silt and clay with minor sand (Presumpscot Formation), overlying bedrock and older glacial sediments.

Based on the Bedrock Geologic Map of Maine (Osberg, et. Al, 1985; on-line GIS version), bedrock at the site is mapped as the Hutchins Corner Formation bedrock unit. The Hutchins Corner Formation is described as fine to medium grained, medium gray quartz-plagioclase-biotite granofels or gneiss, heavily intruded by pegmatite.

4.2 SUBSURFACE PROFILE

Four soil units were encountered in the test borings beneath surficial pavement and overlying bedrock: Fill, Clay, Sand, and Glacial Till. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix C**. The subsurface conditions are also shown in relation to the bridge alignment on the interpretive subsurface profile in **Figure 3**.

Subsurface Unit	Approximate Encountered Thickness (ft)	Generalized Description
Asphalt Pavement	0.5	Asphalt Pavement Encountered in borings BB-LR-101, -102, -104 and -105



Subsurface Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	2.5 to 38.5	Sand Fill: Variable <u>from</u> Brown, medium dense to dense, fine to coarse SAND, some to trace Gravel, trace Silt (SP-SM), <u>to</u> Sandy GRAVEL, trace Silt (GP-GM). • MainedOT Frost Classification = 0, II • Encountered in borings BB-LR-101, -102, -103, -104, -105, and -106 Clay Fill: Gray, stiff to hard, Silty CLAY, little Sand, trace Gravel (CL-ML, CL) • MainedOT Frost Classification = IV • Encountered between approximately El 8.0 and El 22.8 in borings BB-LR-105 and -106
Clay	3 to 15.5	Gray, medium stiff to stiff, Silty CLAY, trace to little Sand, trace organic fibers (CL). • MainedOT Frost Classification = III, IV • Encountered in borings BB-LR-101, -102, -104, -105, and -106
Glacial Till	2.7 to 7.0	Variable <u>from</u> Gray and Brown medium dense Gravelly SAND, little Silt (SM); <u>to</u> Silty fine to medium SAND (SM). • Encountered in borings BB-LR-102, -105, -106
Top of Bedrock Elevation	Encountered Top of Rock: Approx. El. -11.5 to 25.3	

4.2.1 Bedrock

Bedrock consisted of hard, fresh to slightly weathered, fine grained, gray to blue, GNEISS with hard, fresh, coarse grained, gray PEGMATITE intrusions. Joints are very close to moderately spaced, moderately dipping, planar to undulating, rough, open, and fresh to discolored. The Rock Quality Designation (RQD) ranged from 47 to 97 percent, averaging 74 percent, which indicates typically fair quality rock.

Unconfined compressive strength (UCS) testing was conducted on three samples of rock, the results of which are summarized in the following table.

SUMMARY OF BEDROCK STRENGTH TEST RESULTS							
Boring	Depth below Existing Grade (ft)	Depth below Top of Rock (ft)	Elevation (ft NAVD 88)	Unconfined Compressive Strength (psi)	Young's Modulus, E (ksi)	Unit Weight (pcf)	Rock Type
BB-LR-102	11.7	0.7	11.3	5,590	2,760	171	GNEISS
BB-LR-103	8.0	0.0	14.0	6,530	3,170	172	GNEISS
BB-LR-104	19.1	2.9	3.4	3,910	1,870	172	GNEISS

4.2.2 Groundwater

The test borings were drilled using drive-and-wash techniques, which introduce water into the borehole during drilling. As a result, stabilized groundwater levels were not determined at the boring locations. Based on the observed depth where samples transitioned to wet, we estimate that the groundwater was approximately 13 to 20 feet below ground surface at the time the test borings were completed.



The groundwater observations were made at the times and under the conditions stated in the borings logs. Fluctuations in groundwater will occur due to variations in temperature, season, precipitation, and construction activity in the area. Consequently, water levels during and after construction are likely to vary from those encountered at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations based on the 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition, with Interims (AASHTO) and the MaineDOT Bridge Design Guide, 2014 Edition (MaineDOT BDG). Supporting calculations developed by GZA for the project are attached in **Appendix F** of this report.

5.2 APPROACH EMBANKMENTS

The existing bridge approaches were constructed with maximum side slope angles of approximately 2 horizontal to 1 vertical (2H:1V), or flatter. Lunt Road will be raised approximately 1 foot from its existing grade and be widened to the north. The proposed widening will require up to 6 feet of fill to be placed on the north side of existing embankments adjacent to the abutments.

GZA evaluated potential settlement due to consolidation of silty clay deposits beneath the embankments. Based on the test boring and laboratory testing results, the clay is considered heavily overconsolidated. As a result, the deposit will be only moderately compressible under new fill loading. We estimate the post-construction embankment settlements to be approximately ¼ inch. Settlements of this magnitude do not represent a performance issue for the embankment and are considered too small to initiate downdrag¹ in new and/or existing pile foundations. Supporting calculations are provided in **Appendix F**.

The embankments will be constructed per MaineDOT standard specifications and details using engineered fill placed over fill and medium stiff to stiff, heavily overconsolidated, silty clay. In our experience, conventional earth fill embankments constructed over relatively dense and stiff soils meet the minimum required safety factors for global stability.

5.3 EVALUATION OF FOUNDATION TYPES

5.3.1 Abutment Foundations

Based on constructability and cost considerations, WSP selected an integral abutment bridge supported on steel H-piles. Given the shallow depth to rock and the nature of the overburden soils, we anticipate that the piles will be supported primarily by end bearing on or near bedrock.

¹ Hannigan, Goble, Likens and Rausche, FHWA-NHI-05-042, Design and Construction of Driven Pile Foundations, Reference Manual Volume 1, April 2006, Page 9-157.



5.3.2 Pier Foundations

The existing three pier foundations consist of spread footings bearing on bedrock. The outer two pier foundations will be removed to approximately 2 feet below finished grade. The central pier will be completely removed and replaced. The test borings revealed bedrock at a depth of approximately 8 feet below ground surface at the location of the middle pier. Braced excavations of this depth are considered technically feasible in the interstate median, and we expect that it will be preferable to use spread footing foundations bearing on sound, intact bedrock at the pier.

5.4 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the proposed approach fills (including backfill behind and beneath abutments), clay, sand, glacial till, and bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1 using the average SPT N-value and average undrained shear strength (S_u) of the soil materials encountered in the borings. LRFD allows the assumption that rock within the upper 100 feet of the profile has an N-value equal to 100. However, the SPT N-value used to determine the site class was conservatively evaluated by including only the blow counts, undrained shear strength, and thickness of soil above the rock, reducing the effective thickness of the profile and neglecting the bedrock in the upper 100 feet. The average undrained shear strength fell between 1,000 and 2,000 psf for the west abutment, and the average SPT N-value fell between 15 and 50 blows per foot for the east abutment. The pier foundations will bear directly on bedrock. For the sake of simplicity, Site Class D was assigned for design for the entire site. Supporting calculations are provided in **Appendix F**.

5.5 LOAD AND RESISTANCE FACTORS

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

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

GEOTECHNICAL RESISTANCE FACTORS – STRENGTH LIMIT STATE			
Foundation Resistance Type	Method/Condition	Resistance Factor (ϕ)	AASHTO Reference
Bearing	Footing on Rock	0.45	10.5.5.2.2-1
Axial Pile Resistance	Driven Pile, Installed with Dynamic Testing (for Driven Steel H-Pile, Hard Driving)	0.65	10.5.5.2.3-1
Axial Tip Resistance	Drilled Shaft Tip Resistance in Rock (for Steel H-Pile Cast in Concrete-Filled Rock Socket)	0.50	10.5.5.2.4-1

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



5.6 SPREAD FOOTING DESIGN CONSIDERATIONS

A spread footing foundation bearing on sound, intact bedrock is the preferred foundation type at the center pier.

5.6.1 Footing Bearing Resistance

Preliminary nominal and factored bearing resistances were calculated for bedrock-bearing footings using the Rock Mass Rating (RMR)-based empirical correlation presented in “Foundations on Rock,” by Duncan Wyllie. RMR was evaluated in accordance with Table 10.4.6.4-1 of the 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO). The current (8th) Edition of the AASHTO Design Specifications does not include the Rock Mass Rating (RMR) formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.2.6.2 of the 8th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was followed.

GZA used bedrock data obtained in test borings BB-LR-102, BB-LR-103, and BB-LR-104 to develop foundation design parameters at the pier locations. The bedrock properties used in the bearing resistance evaluation are presented below:

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	RQD (percent)	Unconfined Compressive Strength (ksi)	Rock Mass Rating (RMR)	m	s
Gneiss	74	4.0	47	0.57	0.0001537

Based on the available data and the stated methodology, the calculated nominal bearing resistance is 62 kips per square foot (ksf). This will provide a factored bearing resistance of 28 ksf for the strength limit state. Supporting calculations are provided in **Appendix F**.

LRFD Article 10.6.2.4.4 indicates that footings bearing on rock with an RMR-based rock quality of Fair or better and designed using LRFD methods are generally anticipated to experience ½ inch or less of elastic settlement.

5.7 PILE DESIGN CONSIDERATIONS

5.7.1 Pile Design Load

Steel HP14x89 H-section piles are proposed to support both abutment extensions. WSP provided a maximum factored axial load of 466 kips per pile for the strength limit state.

Abutment 1 – West Abutment

West Abutment piles should be installed to a nominal axial resistance of at least 932 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.5 for piles for axial resistance in compression (AASHTO 10.5.5.2.3-1). The resistance factor assumes the static analysis method of end bearing in rock where they will be socketed.



Abutment 2 – East Abutment

Piles should be installed to a nominal axial resistance of at least 717 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.65 for piles installed under hard driving conditions. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted during construction in accordance with AASHTO requirements to assess nominal geotechnical pile resistance.

5.7.2 Axial Pile Resistance and Settlement

Since the piles will gain support in, on or near bedrock, there is no reduction for group interaction in axial compression. As previously noted, downdrag forces will not be considered due to the limited anticipated settlement. By utilizing steel H-piles for support of the abutments, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

Design considerations for socketed piles at Abutment 1 and driven piles at Abutment 2 are presented below.

Abutment 1 – West Abutment

As described in subsequent sections, GZA's lateral pile analysis for the west abutment concludes that a driven pile will not achieve fixity due to the shallow depth to bedrock. A rock socket was evaluated to improve lateral resistance. Installation of H-piles in the rock sockets could be achieved by driving the piles into a socket backfilled with well-graded stone, or by cleaning out the rock socket, adding a plate at the base of the pile, and grouting the pile into the socket. The socket depths anticipated to be necessary to provide adequate fixity are discussed in the subsequent lateral pile analysis section. However, it is apparent that the shallowest required rock socket can be achieved by using the grouted tip detail. As a result, we have developed the appropriate details to support the pile using a grouted tip.

In concept, the grouted rock socket detail will include a steel plate welded to the tip of the pile and designed such that grout can be reliably placed below and around the pile tip and promote full, uniform load transfer to end bearing in bedrock. For this condition, the end bearing resistance may be calculated as it would be for a drilled shaft bearing in bedrock. Using LRFD equation 10.8.3.5.4C-1 and an unconfined compressive strength of 4 ksi yields a nominal axial resistance of 10 ksi. The minimum required bearing area of the pile base is approximately 100 square inches (sq. in.) to provide the required nominal geotechnical axial compression resistance of 932 kips. For practical purposes, it is our opinion that a 14-inch by 14-inch square base plate equipped with a tremie tube extending through and below the base plate is appropriate.

A technically feasible construction sequence to meet the design intent for the drilled and grouted H-pile is as follows:

- Drill through the overburden and seat a temporary casing into the top of rock;
- Drill a 26-inch minimum diameter rock socket;
- Clean all loose material out of the socket using an airlift or vacuum truck;
- Inspect the socket base using camera and sounding techniques;



- Attach centralizers, the base plate and tremie tube to the pile
- Suspend the pile from the top of the temporary casing so that the base plate is approximately 1-foot above the base;
- Grout the socket until the stabilized grout level is 2 feet above the base plate;
- Allow the grout to set for at least 24 hours;
- Tremie stone into the casing; and
- Pull the temporary casing making sure to maintain at least 4 feet of stone above the bottom of the temporary casing as it is withdrawn.

Abutment 2 – East Abutment

Evaluations were conducted for axial compressive geotechnical resistance of driven H-piles. Side friction was estimated for HP 14x89 steel H-Piles using the Nordlund method for granular layers (Fill and Glacial Till) and the α -method for the clay layers in accordance with LRFD Section 10.7. The results indicate that the pile should develop approximately 121 kips in side resistance and that the remainder will be developed in end bearing on or near bedrock. To achieve a nominal resistance of 717 kips, we estimate the side friction to represent approximately 17 percent of the total geotechnical resistance.

5.7.3 Preliminary Pile Drivability Analysis

At Abutment 2, the driven pile installation control will include wave equation and dynamic pile testing with signal matching analysis. Therefore, the piles should be driven to a nominal resistance of 717 kips calculated by dividing the maximum factored pile load of 466 kips by a resistance factor of 0.65, per LRFD Table 10.5.5.2.3-1.

GZA completed preliminary wave equation analyses to assess the drivability of the proposed pile section. Analyses were completed using a pile length of 33 feet, an ASTM A572 Grade 50 (50ksi yield stress) HP14x89 steel H-Pile, and a Delmag D36 diesel hammer with a maximum rated energy of 90,540 foot-pounds (ft-lbs). In order to limit the stroke, the hammer was analyzed on the third lowest of the four available fuel settings. The results are summarized below.

Location	Embedment (ft)	Wave Equation Analysis Results at Rult = 717 kips			
		Percent Side Friction	Final Penetration Resistance (blows/inch)	Driving Stress (ksi)	Stroke (feet)
Abutment 2	33	17	11	44.2	8.1

Since the driving stresses do not exceed the limiting driving stress of 45 ksi (0.9 Fy) for 50 ksi yield stress steel, and the calculated penetration resistance is within the MaineDOT preferred range of 6 to 15 blows per inch, the analyzed hammer system is judged acceptable to install the piles to the required nominal resistance noted. Supporting calculations are provided in **Appendix F**.



5.7.4 Lateral Pile Analysis

GZA developed soil profiles for lateral pile evaluations at each abutment. The profiles reflect the soil conditions encountered in the test borings. At Abutment 1, a 6-foot deep rock socket is assumed to be backfilled with grout 1 foot below and 2 feet above the tip. Since the unconfined compressive strength of the grout will be specified similar to the unconfined compressive strength of the bedrock, the model assumes the bottom 2 feet of the pile is embedded directly in rock with an unconfined compressive strength of 4,000 psi, and the next 3 feet of the socket is backfilled with pea stone.

L-PILE® INPUT PARAMETERS ABUTMENT 1 (WEST), PILE LENGTH = 15 FT (NEAREST BORING BB-LR-101)						
Stratum	Soil Model	Top of Layer Elevation (ft-NAVD 88)	Layer Thickness (ft)	k (pci) / E50 / UC (psi)	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Marine Clay	Reese Stiff Clay	33	9.5	$E_{50} = 0.006$	2000 psf	118
Glacial Till	Reese Sand	23.5	1	$k = 225$	32	125
Pea Stone Infill	Reese Sand	22.5	3	$k = 225$	34	125
Concrete	Hard Rock	19.5	2	UC = 4000	--	152

L-PILE® INPUT PARAMETERS ABUTMENT 2 (EAST), PILE LENGTH = 33 FT (NEAREST BORING BB-LR-105)						
Stratum	Soil Model	Top of Layer Elevation (ft-NAVD 88)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Fill	Reese Sand	30	12	$k = 225$	33	125
Clay Fill	Reese Stiff Clay	18	9	$E_{50} = 0.008$	1000 psf	118
Marine Clay	Reese Stiff Clay	9	10	$E_{50} = 0.008$	1000 psf	55.6
Glacial Till	Reese Sand	-1	2	$k = 60$	32	62.6

GZA's lateral pile analysis for the integral abutments was conducted in accordance with Section 5.4.2.4 of the MaineDOT BDG and the recommendations included in the "Integral Abutment Bridge Design Guidelines" by the Vermont Department of Transportation (VTrans). The analysis used ASTM A572 Grade 50, HP 14x89 piles aligned for weak-axis bending parallel to the bridge centerline.

Lateral pile analysis used the Ensoft, Inc. LPILE version 2016.9.06 software. The ground surface was assumed at the bottom of abutment for thermal contraction.

For initial analysis, the pile head boundary conditions included the factored axial load of 466 kips, the defined pile head deflection corresponding to the thermal deflection of 1.06 inches as provided by WSP, and a fixed-head condition (zero slope condition). The plastic moment resistance of the pile was calculated using unbraced lengths and effective length factors resulting from the first LPILE analysis. The structural capacity of the pile was checked to determine if a plastic hinge formed for the specified displacement and specified axial thrust load based on the provisions of Article 6 of AASHTO and as per the recommendations of Section 5.4.2.4 of the MaineDOT BDG.



For the initial analyses, our results are summarized in the table below.

L-PILE® RESULTS – FIRST ITERATION					
Location	Axial Load (kips)	Lateral Thermal Deflection (in)	LPile Moment at Pile Head (ft-kips)	Plastic Hinge Moment (ft-kips)	Plastic Hinge Forms
Abutment 1 (West)	466	1.06	-307	-137	yes
Abutment 2 (East)	466	1.06	-311	-137	yes

The results indicate that plastic hinges will form at the base of pile cap for the proposed configuration. The hinges allow the piles head to rotate with a constant moment, i.e., the plastic moment. The pile head transforms from a fixed connection to a pinned connection, thereby changing the effective length of the top segment for stability checks. The capacity of the top segment of the pile is enhanced if the plastic moment forms as described.

Since the resulting moment from the first LPILE iterations exceeded the calculated plastic hinge moment, a second LPILE analysis was performed using the specified pile head displacement and moment. The pile head displacement in the second iteration remained unchanged while the specified pile head moment was set equal to the plastic moment. This change results in a reduction of the axial buckling resistance of the upper segment of the pile (Segment 1) where the plastic hinge occurs. The results of the second analyses are summarized below.

L-PILE® RESULTS – SECOND ITERATION					
Location	Axial Load (kips)	Lateral Thermal Deflection (in)	Plastic Hinge Moment at Pile Head (ft-kips)	Structural Resistance Demand Ratio, Segment 1	Combined Bending Demand Ratio, Segment 2
Abutment 1 (West)	466	1.06	-137	0.53	0.93
Abutment 2 (East)	466	1.06	-137	0.53	0.95

The results indicate that the demand ratio for combined bending is less than 1.0 in Segment 2, and therefore also show that with the exception of the plastic hinge location, the pile remains within the elastic range over the remainder of its length and is stable against buckling. The results also indicate that the nominal structural resistance in Segment 1 is still sufficient to support the design loads. The integral abutment pile calculations are included in **Appendix F**.

5.8 ADDITIONAL FOUNDATION CONSIDERATIONS

5.8.1 Frost Protection

Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1300, and with low-moisture content (<20%) soils, the estimated depth of frost penetration is approximately 5 feet. Consequently, new foundation levels should be set at least 5 feet below ground surfaces exposed to freezing temperatures.

Granular fill soils encountered near the surface at the abutments typically were classified as AASHTO A-1-b with MaineDOT Frost Classification from 0 to II, indicating they are considered non-frost-susceptible to



moderately frost-susceptible. Since there was no evidence of significant pavement distress or heave, these materials are judged to be suitable for continued use beneath the approach roadway after reconstruction. In accordance with MaineDOT Standards, new backfill placed behind abutments will consist of non-frost susceptible materials.

5.8.2 Lateral Earth Pressures

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

Based on the estimated thermal bridge expansion of 1.06 inches and the abutment height of approximately 11 feet, the calculated abutment rotation is 0.008 feet/foot. In accordance with the requirements of the BDG Section 5.4.2.11, integral abutment reinforcement is to be designed for full Coloumb passive pressure if the wall rotation is greater than 0.005 feet/foot. Therefore, we conclude that full Coloumb passive earth pressure is appropriate for design.

Lateral earth pressure evaluations for abutments based on the BDG summarized below:

- Passive earth pressure coefficients should be developed using Coloumb theory for Type 4 soil.
- AASHTO Commentary C3.10.9.1 specifies that bridges in Seismic Zone 1 and single-span bridges are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes should be designed with MaineDOT typical slope angles of 2H:1V, and should be provided with loam and seed for permanent erosion protection. We understand that a limited area is proposed to have a 1.75H:1V slope. The proposed erosion protection there consists of riprap placed on a non-woven geotextile consistent with MaineDOT Typical Details, and is considered acceptable.

6.2 RECOMMENDATIONS FOR FOUNDATIONS

6.2.1 Seismic Design

The United States Geological Survey Online Design Maps Tool was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years as follows:



SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1.6
F _a	1.6
F _v	2.4
A _s (Period = 0.0 sec)	0.137 g
S _{Ds} (Period = 0.2 sec)	0.275 g
S _{D1} (Period = 1.0 sec)	0.107 g

6.2.2 Abutment and Wingwall Design

- Backfill behind new abutments and wingwalls should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - Coefficient of At-Rest Earth Pressure, $K_o = 0.47$
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of new abutment and wing wall structures. The drains should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center.

6.2.3 Pile Design

Abutment 1 (West Abutment)

- The proposed abutment may be supported on ASTM A572, Grade 50 steel (50 ksi yield stress) HP14x89 H-piles socketed into the bedrock. The nominal axial pile resistance of 932 kips will be developed through end bearing at the base of the rock socket, providing a factored axial resistance of 466 kips per pile.
- Rock socket holes should be drilled through the overburden using temporary casing that is seated into the top of rock prior to drilling the socket.
- Rock sockets should be at least 26 inches diameter and should be cleaned of all loose material using an airlift or vacuum truck, and the socket base should be inspected using camera and sounding techniques prior to pile placement.
- The piles should be equipped with centralizers, 14-inch by 14-inch base plates and tremie tubes prior to installation.



- The lower portion of the tremie tube (below the base plate) should consist of a 1-inch minimum diameter black steel tremie tube extending through and at least 3-inches below the center of the base plate. The tremie tube extending above the base plate may consist of steel, PVC or plastic tubing.
- The piles should be suspended from the top of the temporary casing so that the base plate is approximately 1 foot above the base, then tremie grouted up to 2 feet above the base plate using 4,000 psi minimum unconfined compressive strength grout.
- The grout should be allowed to set for at least 24 hours before backfilling the hole and removing the temporary casing.
- Stone consisting of MaineDOT 703,22 Underdrain Backfill Material, Type C should be used to backfill the remainder of the socket and the drill hole. Care should be taken to maintain at least 4 feet of stone above the bottom of the temporary casing as it is withdrawn.
- Due to the limited number of explorations, the pile lengths should be assumed to range from approximately 15 to 25 feet.

Abutment 2 (East Abutment)

- The east abutment may be supported on ASTM A572, Grade 50 steel (50 ksi yield stress) HP14x89 H-piles driven to bearing on or near the top of bedrock. The piles should be driven to a nominal resistance of 717 kips, calculated by dividing the maximum factored pile load of 466 kips by the resistance factor 0.65.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- Preliminary wave equation analyses indicate that the piles can be driven to a nominal resistance of 717 kips using a diesel hammer with a rated energy of about 90, 540 ft-lbs for the anticipated 33-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9F_y for 50 ksi steel). The final penetration resistance was 11 blows per inch, which is within the MaineDOT range of 3 to 15 blows per inch.
- In the preliminary wave equation analysis, the pile hammer had to be operated at the third lowest of its four fuel settings in order to control driving stresses. It is anticipated that careful attention will be needed to cease driving when refusal criteria are met in order to prevent overstressing of the piles.
- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings and shown on **Figure 3** (approximately El. 0 at the East Abutment), plus or minus 10 feet to account for possible variability in the top of rock surface.
- We recommend that one pile be dynamically tested at the end of initial drive to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the remaining piles. The plans should also require a restrike test on the tested pile. In the event that the initial driving results are favorable and show sufficient excess resistance to allow for some relaxation, the restrike tests may be waived.



- Pile splices should be avoided, given the short lengths, however, if necessary, piles should be spliced in accordance with MaineDOT Section 501.047.

6.2.4 Spread Footing Design

- The proposed piers should be supported on spread footing foundations bearing on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed for a nominal bearing resistance, q_n , of 62 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 28 ksf. A bearing resistance of 28 ksf should be used for service limit state design.
- LRFD Article 10.6.2.4.4 indicates that footings bearing on rock with an RMR-based rock quality of Fair or better and designed using LRFD methods are generally anticipated to experience ½ inch or less of settlement.
- Existing substructures should be completely removed at the center pier prior to new foundation construction.
- Spread footings founded on bedrock should be checked for eccentricity with LRFD Section 10.6.3.3. Eccentricity of the footing reaction at the strength limit state should be limited such that the resultant reaction on the base of the footing is no further than 0.45 B from the centerline of the footing, where B is the principal dimension of the footing perpendicular to the axis of rotation.
- The base resistance against sliding may be based on NAVFAC DM7.02-63, Table 1, which indicates the sliding resistance coefficient ($\tan \delta$) is equal to 0.7 for cast-in-place concrete on sound rock. Therefore, the nominal sliding resistance between footings and bedrock subgrades is equal to the vertical force multiplied by 0.7. The factored sliding resistance coefficient is 0.56 for Strength Limit State.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction.

7.0 CONSTRUCTION CONSIDERATIONS

This section describes geotechnical-related issues that have the potential to impact design and cost considerations for bridge construction.

7.1 SUPPORT OF EXCAVATION AND DEWATERING

Excavation for bridge pier foundation is anticipated to extend approximately 8 feet below existing grades. Due to the close proximity to the travel lanes of I-295, the excavation will require temporary lateral support. An internally-braced steel sheet pile system is considered suitable for temporary support of the pier excavation.

Excavation for the abutment replacements is anticipated to be accomplished using sloped open cut techniques, in accordance with MaineDOT and OSH requirements.



Dewatering is anticipated to be feasible by pumping from sumps placed within the excavations. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.2 SUBGRADE PREPARATION

GZA anticipates that the footing excavation for the piers will be completed in-the-dry. Existing foundations at the center pier location should be entirely removed prior to new foundation construction. Prior to casting footings, the bedrock surface should be cleaned using high-pressure air or water to remove all soil and loose, decomposed, highly weathered and fractured bedrock.

If the bedrock is found to be sloping more steeply than 4H:1V, the Geotechnical Engineer should be provided the opportunity to review the exposed bedrock surface and participate in developing and reviewing any measures proposed to enhance sliding resistance.

7.3 PILE INSTALLATION CONTROL

Abutment 1 (West Abutment)

The west abutment piles will be installed in pre-drilled holes that extend from the pile cap subgrade to the bottom of the proposed rock socket. We anticipate that it will be necessary to use temporary casing to keep the holes open through the overburden soil. The temporary casings may be installed by driving, vibrating or spinning, assuming that a suitable seal can be achieved to allow preparation of a clean rock socket. Once the casings are seated in the top of the rock the rock sockets may be drilled. Once the socket drilling is completed, the bases should be cleaned out using an auger, cleanout bucket, air lift, vacuum truck or other suitable means such that no loose material remains in the base.

A temporary template or frame may be employed to establish the pile top locations. The piles may then be placed in the prepared and approved sockets and tremie grout placed 1 foot below and 2 feet above the base plates. The plan details should show the tremie grout placement being made from the bottom of the drilled sockets, beneath the pile bearing plates. The grout should be allowed to set up for at least 24 hours, then the frame may be removed, and the remainder of the temporary casing backfilled with stone. The temporary casing may then be removed making sure to maintain the stone level at least 4 feet above the base of the temporary casing throughout the process.

Abutment 2 (East Abutment)

We recommend that the pile installation be controlled using wave equation analysis and field logging and that final penetration resistance be based on dynamic pile testing with signal matching analysis in accordance with the recommendations in **Section 6.2.3**.



FIGURES



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

Data Supplied by :



PROJ. MGR.: BMC
DESIGNED BY: NVW
REVIEWED BY: CLS
OPERATOR: NVW

DATE: 09-28-2017

LOCUS PLAN LUNT ROAD BRIDGE #5829

FALMOUTH, MAINE
MAINEDOT WIN 21723.00

JOB NO.
09.0025950.00

FIGURE NO.
1

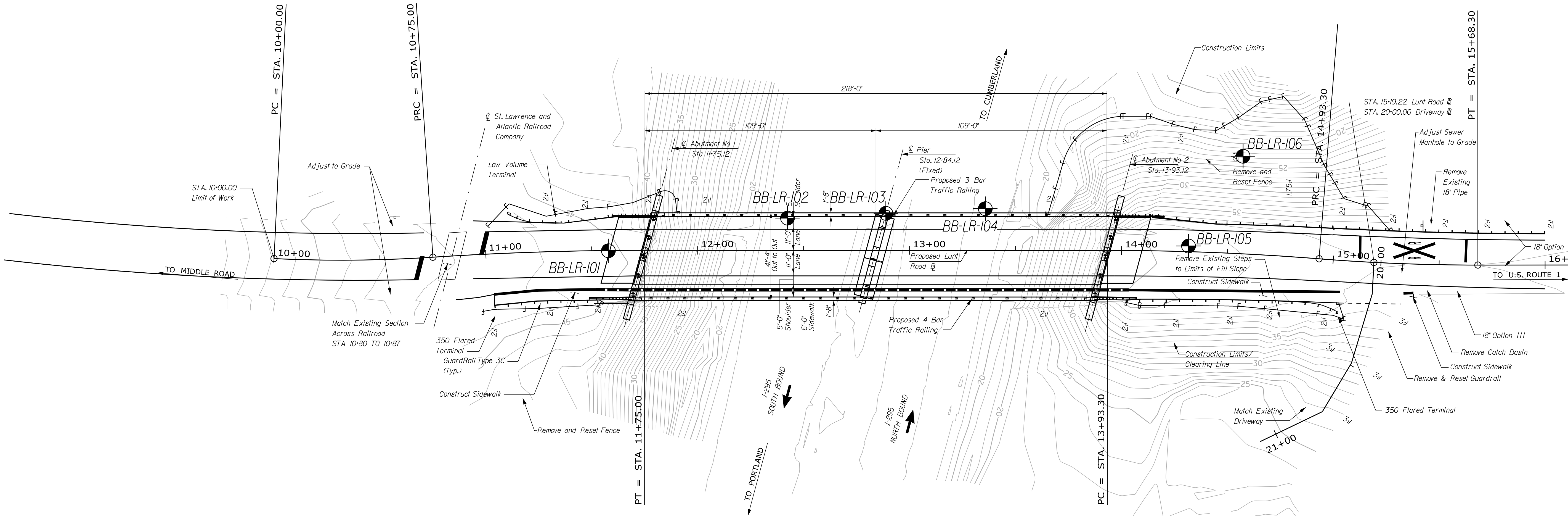
Date:8/2/2018

common

Division: HIGHWAY

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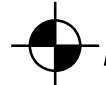
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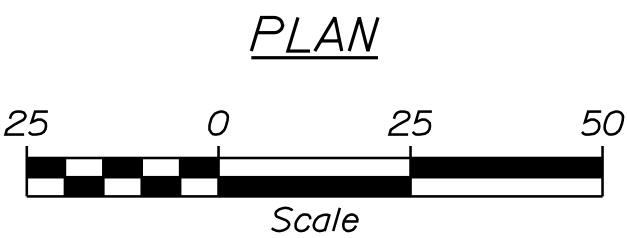


NOTES

- 1) Base map developed from electronic files (004_BDPLAN.dgn, Z_BDBridge.dgn, Z_Align.dgn, RWPLAN.dgn, and 3DContours.dgn) provided by WSP on July 10, 2018.
- 2) The as-drilled locations of the test borings were taped from existing structures by GZA during the subsurface investigation, and are considered approximate.
- 3) BB-LR-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between August 7 and 11, 2017

BORING LOCATION PLAN LEGEND

 BB-LR-106 Location and designation of cased wash boring



PREPARED BY:


PROJ. MANAGER	BY	DATE	SIGNATURE
DESIGN-DETAILED	NW	SEPT 2017	
CHECKED-REVIEWED	NW	SEPT 2017	
DESIGN-DETAILED			
DESIGN-DETAILED			
REVISIONS 1	NW	JULY 2018	P.E. NUMBER
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

FIGURE

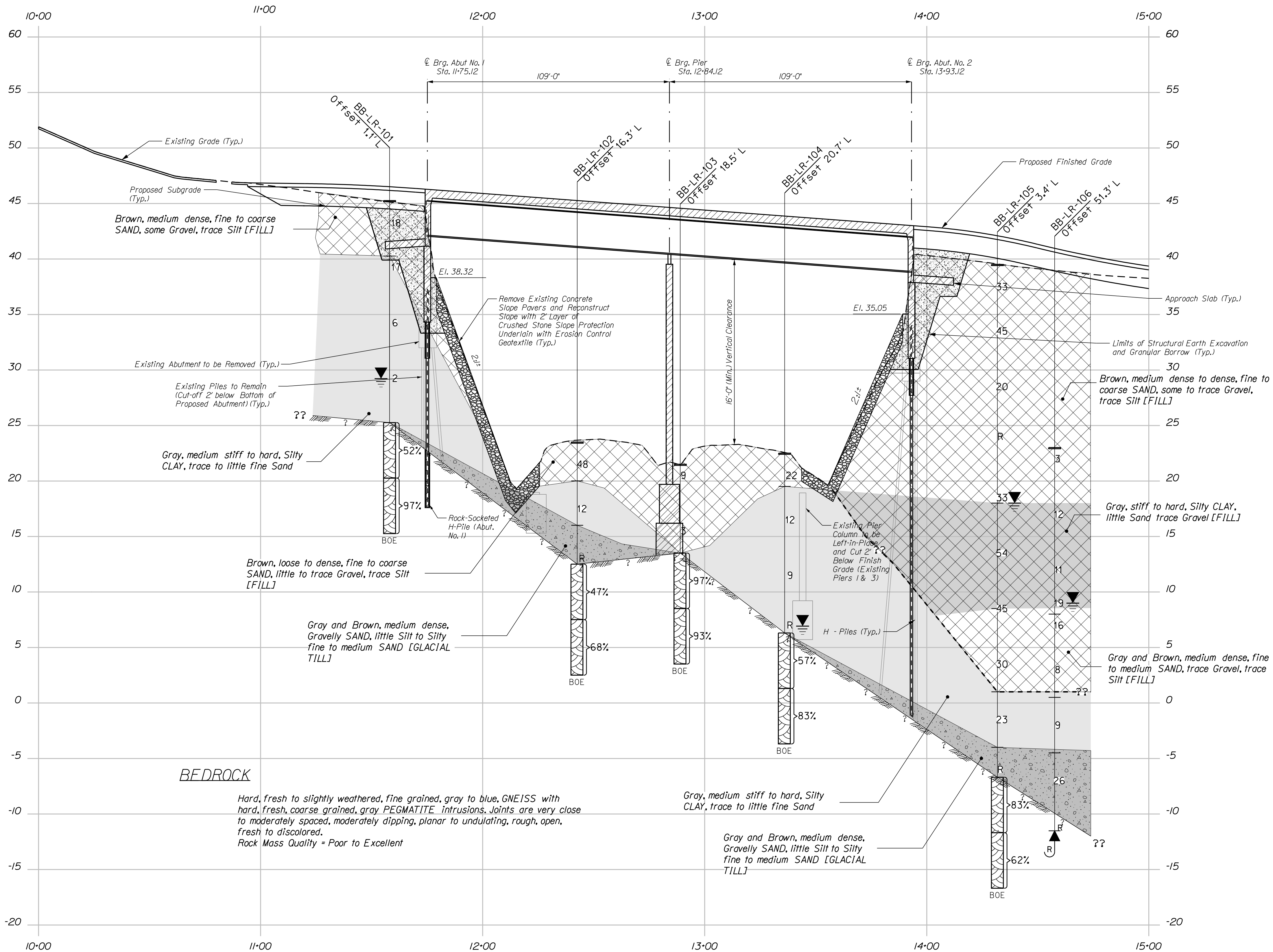
2

Date:8/2/2018

Username: common

Division: HIGHWAY

Filename: ...\\FromWSP071718\\SP FBR.dgn



NOTES

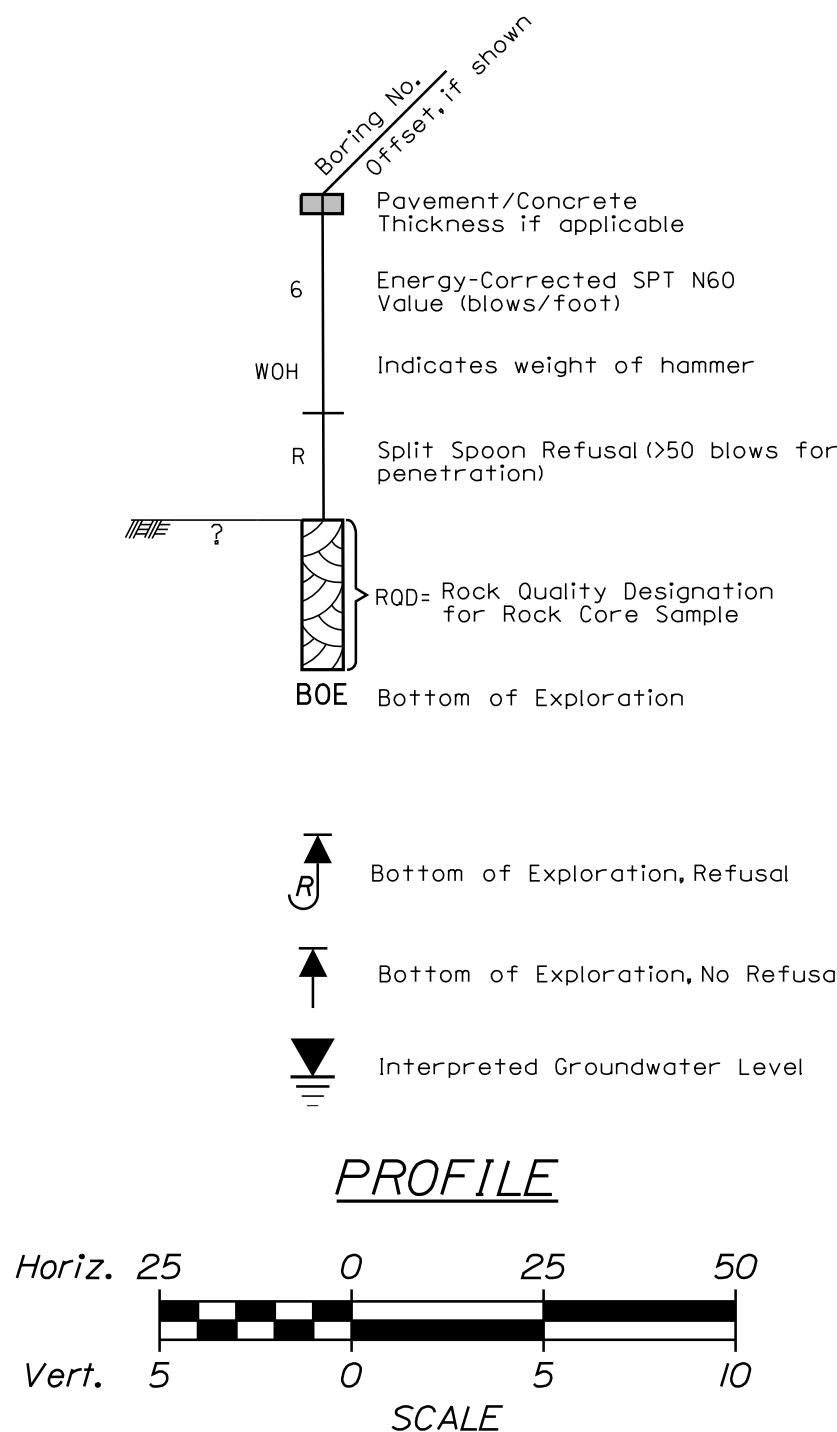
1) Profile developed from an electronic file (005_Profile.dgn) provided by WSP on June 7, 2018 and electronic files (005_Profile.dgn and Profile_Xref.dgn) provided by WSP on July 17, 2018.

2) The as-drilled locations of the test borings were taped from existing structures by GZA during the subsurface investigation. Ground surface elevations at boring locations were interpolated between 1-foot contours in the electronic file 3DContours.dgn provided by WSP on September 5, 2017. Both should be considered accurate to the degree implied by the methods used.

3) BB-LR-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between August 7 and 11, 2017.

4) 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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

INTERPRETIVE SUBSURFACE PROFILE LEGEND



PREPARED BY:



STATE OF MAINE		DEPARTMENT OF TRANSPORTATION		STP-2172(300)		WIN		21723.00		BRIDGE PLANS	
LUNT ROAD BRIDGE		INTERSTATE 295		CUMBERLAND		FALMOUTH		FIGURE		3	
DATE		BY		SIGNATURE		P.E. NUMBER		DATE			
SEPT 2017		NW				JUNE 2018					
DESIGN-REVIEWED		CLS				REVISIONS 1					
DESIGN-DETAILED						REVISIONS 2					
						REVISIONS 3					
						REVISIONS 4					
						FIELD CHANGES					



APPENDIX A – LIMITATIONS



GEOTECHNICAL LIMITATIONS

Use of Report

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

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.
3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.
4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.
6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.



7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.
8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.
9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

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

Cost Estimates

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

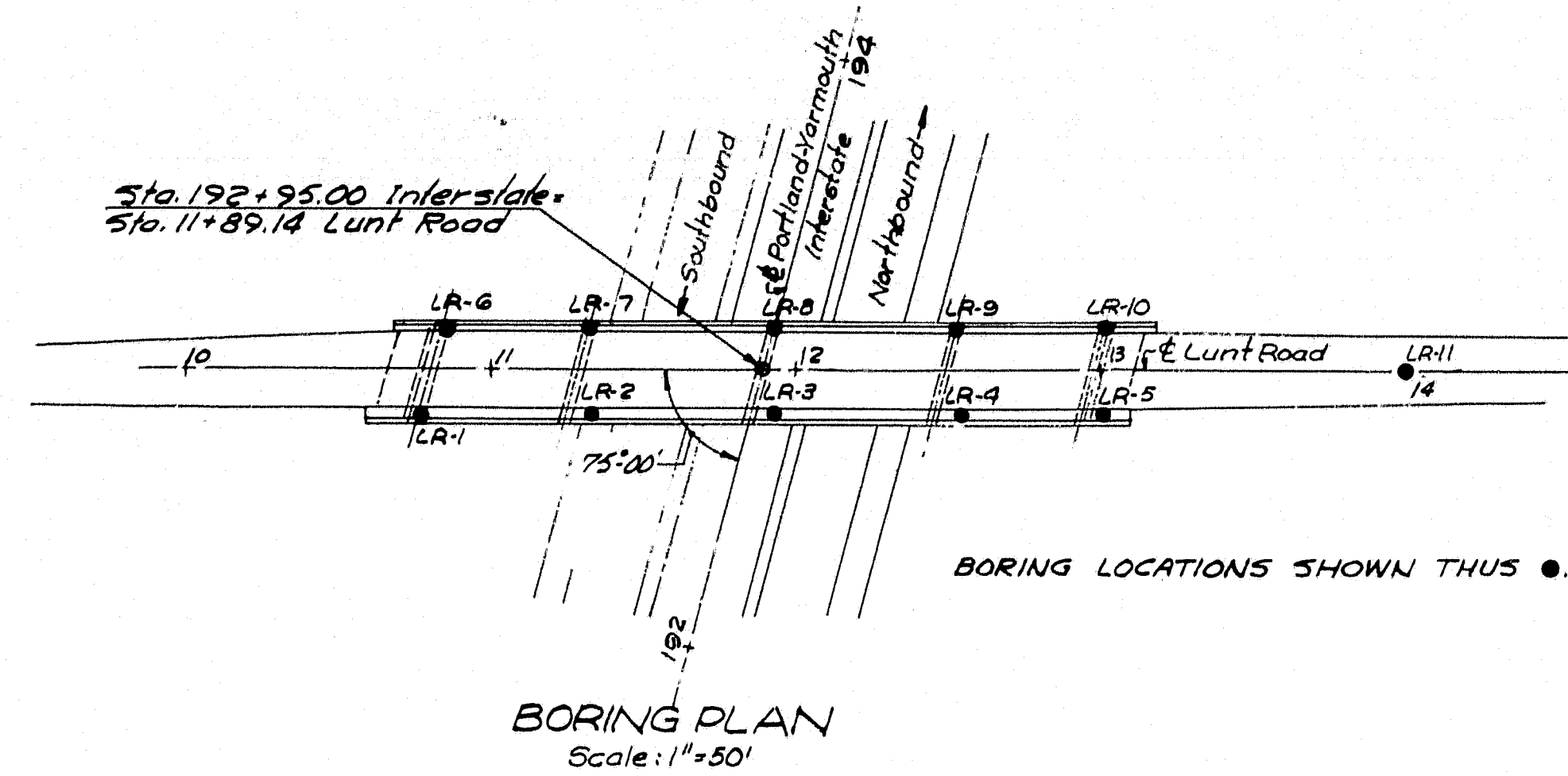
Additional Services

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

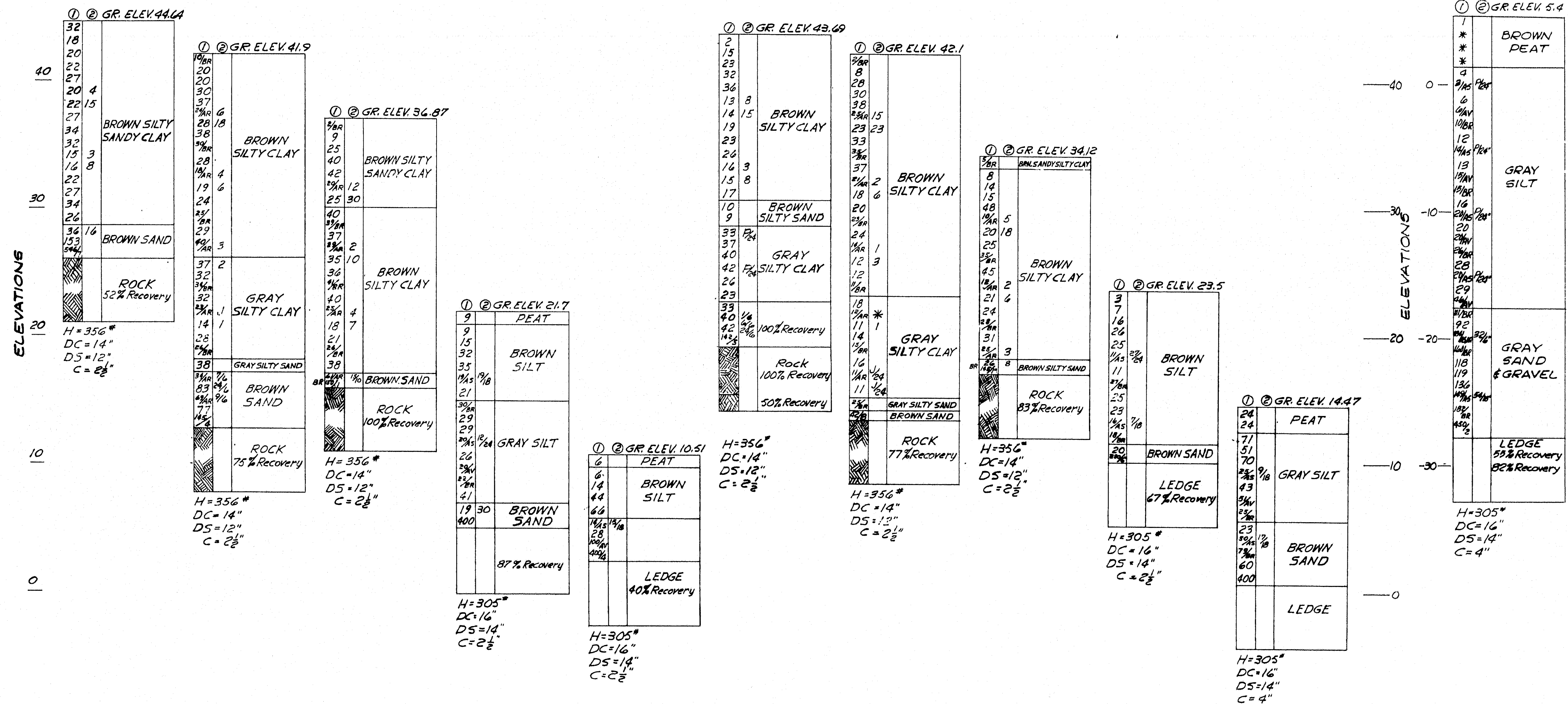


APPENDIX B – 1959 TEST BORING LOGS

FALMOUTH



50 BORING LR-1 BORING LR-2 BORING LR-3 BORING LR-4 BORING LR-5 BORING LR-6 BORING LR-7 BORING LR-8 BORING LR-9 BORING LR-10 50 10 BORING LR-11
STA. 10+74-16'R STA. 11+29-15'R STA. 11+89-15'R STA. 12+49-15'R STA. 12+96-15'R STA. 10+83-15'L STA. 11+29-15'L STA. 11+89-15'L STA. 12+49-15'L STA. 12+96-15'L STA. 14+00



GENERAL NOTES

- BORINGS WERE MADE BY THE MAINE STATE HIGHWAY COMMISSION DECEMBER, 1957 AND JANUARY, 1958.
- FIGURES IN COLUMN ① = BLOWS PER FOOT ON CASING EXCEPT AS NOTED.
- FIGURES IN COLUMN ② = BLOWS PER FOOT ON SAMPLER ROD EXCEPT AS NOTED.
- ELEVATIONS ARE REFERRED TO MEAN SEA LEVEL.
- ADDITIONAL SOIL INFORMATION OBTAINED FROM LABORATORY TESTS IS AVAILABLE FROM THE MAINE STATE HIGHWAY COMMISSION.

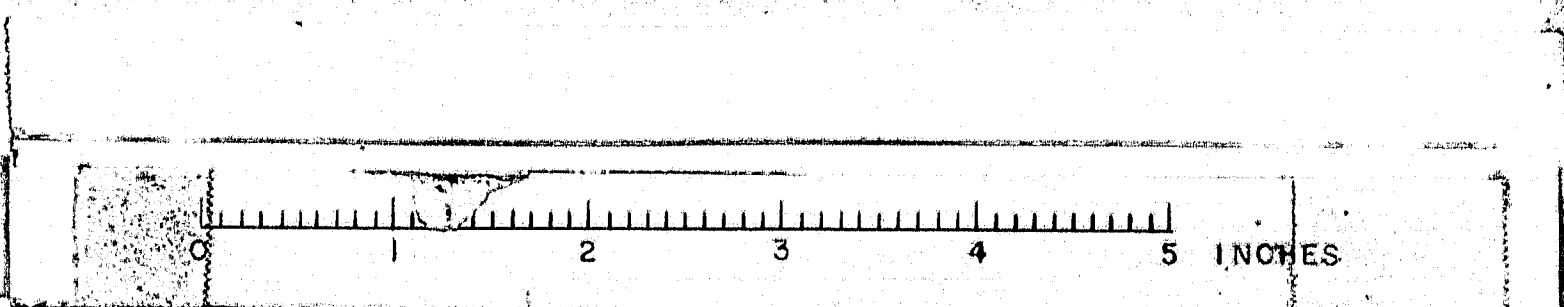
LEGEND

- AS = CASING WAS DRIVEN AFTER SAMPLING.
AR = CASING WAS DRIVEN WASHING AHEAD OF CASING.
H = WEIGHT OF HAMMER IN POUNDS.
J = CASING OR SAMPLER DRIVEN BY STATIC LOAD NOT EXCEEDING 1/2 TON.
P = PISTON
DC = DROP ON CASING IN INCHES.
DS = DROP ON SAMPLER ROD IN INCHES.
AV =
BR =
C = DIAMETER OF CASING IN INCHES
* = DRIVEN BY WEIGHT OF HAMMER

STATE HIGHWAY COMMISSION AUGUSTA, MAINE	
PORTLAND-YARMOUTH INTERSTATE	
LUNT ROAD OVER INTERSTATE	
BORING DATA	
SHEET NO. 28 OF 61.	SCALE: AS NOTED

M-1317 FAY, SPOFFORD & THORNDIKE, INC. BOSTON, MASS. 02114

BORING LOGS
Scale: Vert. 1"=5'





APPENDIX C – GZA TEST BORING LOGS

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Lunt Road Bridge #5829 Location: Falmouth, Maine		Boring No.: BB-LR-101 PIN: 21723.00																																																																																																																																																																																																																																																																																																						
Driller: New England Boring Contractors			Elevation (ft.) 45.3			Auger ID/OD:																																																																																																																																																																																																																																																																																																						
Operator: Brad Enos			Datum: NAVD 88			Sampler: Split Spoon																																																																																																																																																																																																																																																																																																						
Logged By: N. Williams			Rig Type: Mobile Drill - ATV			Hammer Wt./Fall: 140/30																																																																																																																																																																																																																																																																																																						
Date Start/Finish: 8-10-17/8-10-17			Drilling Method: SSA / Drive & Wash			Core Barrel: NX2																																																																																																																																																																																																																																																																																																						
Boring Location: See Boring Location Plan			Casing ID/OD: 3"			Water Level*: NE																																																																																																																																																																																																																																																																																																						
Hammer Efficiency Factor: 0.68			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																																																																																									
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Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th></tr><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>44.8</td><td></td><td>Top 6": Pavement</td><td rowspan="3">G A-1-b, SP</td></tr><tr><td></td><td>1D</td><td>24/10</td><td>1.0 - 3.0</td><td>14-10-6-4</td><td>16</td><td>18</td><td></td><td></td><td></td><td></td><td>Brown, dry, medium dense, fine to coarse SAND, some Gravel, trace Silt. -FILL- (SP)</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>5</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>41.3</td><td></td><td></td><td rowspan="3">LL=44 PL=20 PI=24 WC=25.8</td></tr><tr><td></td><td>2D</td><td>24/24</td><td>5.0 - 7.0</td><td>5-7-8-9</td><td>15</td><td>17</td><td></td><td></td><td></td><td></td><td>Gray, moist, very stiff, Silty CLAY. (CL)</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>10</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="3">WC=26.1</td></tr><tr><td></td><td>3D</td><td>24/24</td><td>10.0 - 12.0</td><td>2-2-3-4</td><td>5</td><td>6</td><td></td><td></td><td></td><td></td><td>Gray, moist, medium stiff, Silty CLAY. (CL)</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="3">C S_{u(lab)}=1840 psf</td></tr><tr><td></td><td>U1</td><td>24/10</td><td>13.0 - 15.0</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Gray, moist, hard, Silty CLAY. (CL)</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>15</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="3">LL=24 PL=15 PI=9 WC=31.1</td></tr><tr><td></td><td>4D</td><td>24/24</td><td>15.0 - 17.0</td><td>WOH-WOH-2-4</td><td>2</td><td>2</td><td></td><td></td><td></td><td></td><td>Gray, wet, medium stiff, Silty CLAY, little fine Sand. 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Joints are very close to close, moderately dipping to high angle, undulating, rough, discolored, open. One joint at 21.6' was very wide. Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min:sec) 20.0-21.0 (2:01)</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>										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									44.8		Top 6": Pavement	G A-1-b, SP		1D	24/10	1.0 - 3.0	14-10-6-4	16	18					Brown, dry, medium dense, fine to coarse SAND, some Gravel, trace Silt. -FILL- (SP)													5									41.3			LL=44 PL=20 PI=24 WC=25.8		2D	24/24	5.0 - 7.0	5-7-8-9	15	17					Gray, moist, very stiff, Silty CLAY. (CL)													10												WC=26.1		3D	24/24	10.0 - 12.0	2-2-3-4	5	6					Gray, moist, medium stiff, Silty CLAY. (CL)																									C S _{u(lab)} =1840 psf		U1	24/10	13.0 - 15.0								Gray, moist, hard, Silty CLAY. (CL)													15												LL=24 PL=15 PI=9 WC=31.1		4D	24/24	15.0 - 17.0	WOH-WOH-2-4	2	2					Gray, wet, medium stiff, Silty CLAY, little fine Sand. (CL)																																																		20														R1	60/60	20.0 - 25.0	RQD = 52%					25.8		R1: 20.0'-25.0' Hard, fresh, fine grained, gray to blue, GNEISS with hard, fresh, coarse grained, gray Pegmatite intrusion at 21.2'-22.7'. 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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS							Project: Lunt Road Bridge #5829 Location: Falmouth, Maine						Boring No.: BB-LR-101 PIN: 21723.00																																																														
Driller: New England Boring Contractors							Elevation (ft.): 45.3						Auger ID/OD:																																																														
Operator: Brad Enos							Datum: NAVD 88						Sampler: Split Spoon																																																														
Logged By: N. Williams							Rig Type: Mobile Drill - ATV						Hammer Wt./Fall: 140/30																																																														
Date Start/Finish: 8-10-17/8-10-17							Drilling Method: SSA / Drive & Wash						Core Barrel: NX2																																																														
Boring Location: See Boring Location Plan							Casing ID/OD: 3"						Water Level*: NE																																																														
Hammer Efficiency Factor: 0.68							Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																				
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Depth (ft.)		Sample No.		Pen./Rec. (in.)		Sample Depth (ft.)		Blows (/6 in.) Shear Strength (psf) or RQD (%)		N-uncorrected		N ₆₀		Casing Blows		Elevation (ft.)		Graphic Log		Visual Description and Remarks																			Laboratory Testing Results/ AASHTO and Unified Class.																																				
25		R2		60/60		25.0 - 30.0		RQD = 97%								15.3				21.0-22.0 (2:15) 22.0-23.0 (2:12) 23.0-24.0 (1:34) R2: 25.0'-30.0' Hard, fresh, fine grained, gray to blue, GNEISS. Joints are close to moderately spaced, moderately dipping, undulating, rough, fresh, open. Rock Mass Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 25.0-26.0 (2:33) 26.0-27.0 (2:40) 27.0-28.0 (2:05) 28.0-29.0 (2:01) 29.0-30.0 (2:19) Bottom of Exploration at 30.00 feet below ground surface.																																																							
30																																																																											
35																																																																											
40																																																																											
45																																																																											
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Soil/Rock Exploration Log
US CUSTOMARY UNITS

Location: Falmouth, Maine

PIN: 21723.00

Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

1. Observed intermittent increase in resistance during auger advancement, consistent with possible cobbles/boulders from approximately 7.0'-10.0'.
2. Spun 4" casing to 8.0' and drove casing to 10.8'. Drove 3" casing to 11.0' prior to core sampling.

Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS										Project: Lunt Road Bridge #5829 Location: Falmouth, Maine					Boring No.: BB-LR-102 PIN: 21723.00									
Driller: New England Boring Contractors					Elevation (ft.) 23.0					Auger ID/OD:														
Operator: Brad Enos					Datum: NAVD 88					Sampler: Split Spoon														
Logged By: N. Williams					Rig Type: Mobile Drill - Truck					Hammer Wt./Fall: 140/30														
Date Start/Finish: 8-7-17/8-7-17					Drilling Method: SSA / Drive & Wash					Core Barrel: NX2														
Boring Location: See Boring Location Plan					Casing ID/OD: 4"/3"					Water Level*: NE														
Hammer Efficiency Factor: 0.87					Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																			
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu 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 = weight of rods WO1P = Weight of one person					S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected					S _u (lab) = Lab Vane 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.)																
25											surface.													
50																								
Remarks: 1. Observed intermittent increase in resistance during auger advancement, consistent with possible cobbles/boulders from approximately 7.0'-10.0'. 2. Spun 4" casing to 8.0' and drove casing to 10.8'. Drove 3" casing to 11.0' prior to core sampling.																								
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											Page 2 of 2 Boring No.: BB-LR-102													

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Lunt Road Bridge #5829 Location: Falmouth, Maine			Boring No.: BB-LR-103 PIN: 21723.00		
Driller: New England Boring Contractors				Elevation (ft.) 22.0			Auger ID/OD:			
Operator: Brad Enos				Datum: NAVD 88			Sampler: Split Spoon			
Logged By: N. Williams				Rig Type: Mobile Drill - Truck			Hammer Wt./Fall: 140/30			
Date Start/Finish: 8-8-17/8-8-17				Drilling Method: SSA / Drive & Wash			Core Barrel: NX2			
Boring Location: See Boring Location Plan				Casing ID/OD: 3"			Water Level*: NE			
Hammer Efficiency Factor: 0.87				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐						
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu 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 = weight of rods WO1P = Weight of one person					S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected	
Sample Information				Graphic Log		Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.		
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)		
0	1D	24/10	0.0 - 2.0	2-2-4-8	6	9	SSA	21.7		
5	2D	24/5	5.0 - 7.0	1-1-1-1	2	3		14.2		
10	R1	60/60	8.0 - 13.0	RQD = 97%				7.8		
15	R2	60/59	13.0 - 18.0	RQD = 93%				4.0		
20										
25										
Remarks: 1. Advanced SSA to 5.0'. Advanced 3" casing from 7.0'-7.8'. 3" casing met refusal at 7.8'. Advanced roller bit to 8.0' prior to core sampling.										
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										
Page 1 of 1							Boring No.: BB-LR-103			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Lunt Road Bridge #5829 Location: Falmouth, Maine		Boring No.: BB-LR-104 PIN: 21723.00																																																																																																																																																																																																																																																																																																																				
Driller: New England Boring Contractors			Elevation (ft.) 22.5		Auger ID/OD:																																																																																																																																																																																																																																																																																																																					
Operator: Brad Enos			Datum: NAVD 88		Sampler: Split Spoon																																																																																																																																																																																																																																																																																																																					
Logged By: N. Williams			Rig Type: Mobile Drill - Truck		Hammer Wt./Fall: 140/30																																																																																																																																																																																																																																																																																																																					
Date Start/Finish: 8-8-17/8-8-17			Drilling Method: SSA / Drive & Wash		Core Barrel: NX2																																																																																																																																																																																																																																																																																																																					
Boring Location: See Boring Location Plan			Casing ID/OD: 3"		Water Level*: NE																																																																																																																																																																																																																																																																																																																					
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows /6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.																																				
50	R2	60/52	51.2 - 56.2	RQD = 62%						Rock Core Times (min:sec): 46.2-47.2 (6:14) 47.2-48.2 (1:33) 48.2-49.2 (1:42) 49.2-50.2 (2:22) 50.2-51.2 (1:40) R2: 51.2'-56.2' Hard, fresh, fine grained, gray to blue, GNEISS with hard, fresh, coarse grained, gray Pegmatite intrusion from 54.8'-55.3'. Joints are very close to close, moderately dipping, planar, rough, fresh to discolored, partially open to open. Rock Mass Quality = Fair Recovery = 87% Rock Core Times (min:sec): 51.2-52.2 (1:55) 52.2-53.2 (1:50) 53.2-54.2 (1:41) 54.2-55.2 (2:09) 55.2-56.2 (3:45)																																					
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Remarks:																																															
1. Intermittent increased resistance during SSA advancement consistent with presence of cobbles from 7.0'-10.0'. 2. Advanced SSA to 10.0'. Advanced 4" casing to 15.0'. 3. During roller cone advancement, observed high resistance from 16.0'-16.9' consistent with possible boulder and intermittent resistance to 20.0' consistent with possible cobbles. 4. Split spoon refusal at 46.1'. Drove casing to 46.2' prior to core sampling.																																															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.												Page 3 of 3 Boring No.: BB-LR-105																																			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Lunt Road Bridge #5829 Location: Falmouth, Maine		Boring No.: BB-LR-106 PIN: 21723.00																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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Definitions: D = Split Spoon SampleMD = Unsuccessful Split Spoon Sample attemptU = Thin Wall Tube SampleMU = Unsuccessful Thin Wall Tube Sample attemptV = Insitu Vane Shear TestMV = Unsuccessful Insitu Vane Shear Test attempt						R = Rock Core SampleSSA = Solid Stem AugerHSA = Hollow Stem AugerRC = Roller ConeWOH = weight of 140lb. hammerWOR = weight of rodsWO1P = Weight of one person							S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected			S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th><th></th></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>-4.5</td><td rowspan="12"></td><td rowspan="12">Brown, wet, medium dense, silty fine to medium SAND. -GLACIAL TILL- (SM)</td><td rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>8D</td><td>24/16</td><td>29.0 - 31.0</td><td>4-12-11-12</td><td>23</td><td>26</td><td></td><td></td><td></td></tr><tr><td>30</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>9D</td><td>6/6</td><td>34.0 - 34.5</td><td>90-50/0"</td><td>R</td><td></td><td></td><td></td><td>-11.5</td></tr><tr><td>35</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12"></td><td rowspan="12">Gray/red, wet, silty fine SAND. Possible decomposed rock. -GLACIAL TILL- (SM) Bottom of Exploration at 34.50 feet below ground surface.</td><td rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>40</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>45</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>50</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>																Sample Information										Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)		25									-4.5		Brown, wet, medium dense, silty fine to medium SAND. -GLACIAL TILL- (SM)																							8D	24/16	29.0 - 31.0	4-12-11-12	23	26				30																																																			9D	6/6	34.0 - 34.5	90-50/0"	R				-11.5	35											Gray/red, wet, silty fine SAND. Possible decomposed rock. -GLACIAL TILL- (SM) Bottom of Exploration at 34.50 feet below ground surface.																																																																																																																40																																																																																																								45																																																																																											50												
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Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 2 of 2
Boring No.: BB-LR-106



APPENDIX D – LABORATORY TESTING RESULTS

LABORATORY TESTING DATA SHEET (Rev 2)



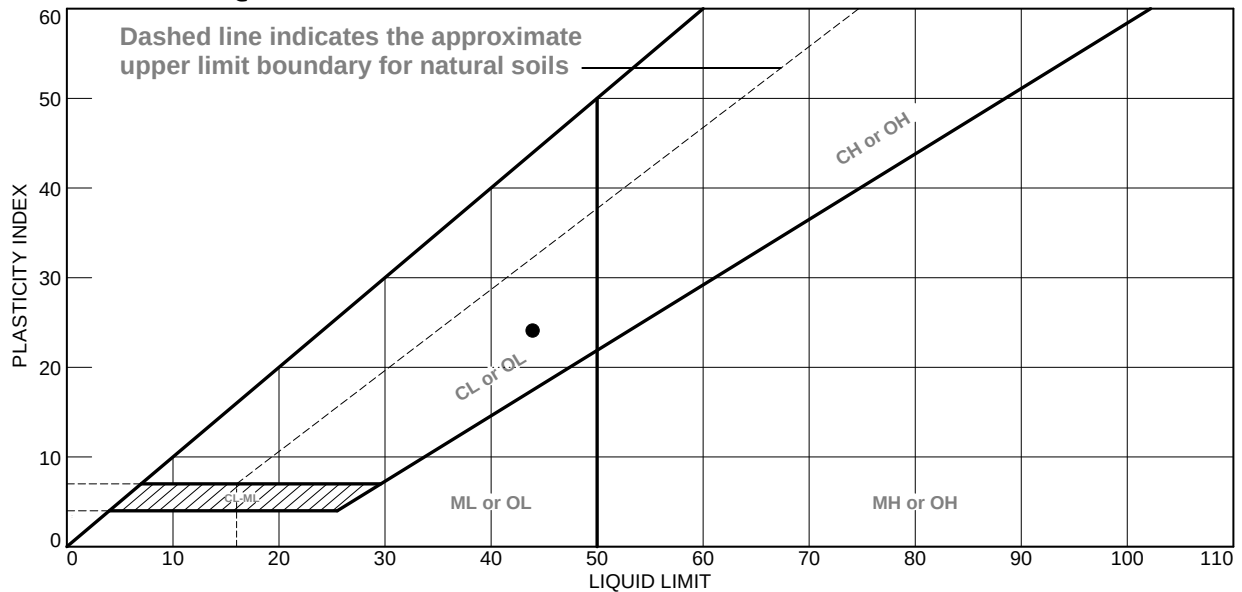
Project Name **Lunt Road**
 Project No. **09.0025950.00**
 Project Manager **B. Cardali**

Location **Falmouth, ME**
 Assigned By **N. Williams**
 Report Date **09.07.17**

Reviewed By _____
 Date Revised **09.13.17**

Boring/ Test Pit No.	Sample No.	Depth ft.	Lab No.	Identification Tests							Corrosivity				Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines (<#200) %	Org. %	Sulfate (mg/kg)	Chloride (mg/kg)	Resistivity (Mohms-cm)	GTL Resist	
BB-LR-101	2D	5-7	1	25.8	44	20									Brownish Gray Silty CLAY
BB-LR-101	3D	10-12	2	26.1											Water Content Only
BB-LR-101	4D	15-17	3	31.3	24	15									Brownish Gray SILT & CLAY
BB-LR-102	2D	5-7	4	15.5											Water Content Only
BB-LR-105	6D	25-27	5	20.7											Water Content Only
BB-LR-105	9D	40-42	6	29.4											Water Content Only
BB-LR-106	4D	13-15	7	33.6											Water Content Only
BB-LR-106	7D	24-26	8	39.6											Water Content Only
BB-LR-101	1D	1-3	9				34.7	64.2	1.1						Light Brown SAND, some gravel, trace silt
BB-LR-103	1D	0-2	10				3.3	93.7	3.0						Light Brown SAND, trace gravel, trace silt
BB-LR-103	2D	5-7	11				6.4	86	7.6						Brown SAND, trace gravel, trace silt
BB-LR-105	1D	1-3	12				6.5	87	6.5						Light Brown SAND, trace gravel, trace silt
BB-LR-105	2D	5-7	13				46.8	45.2	8.0						Dark Brown sandy GRAVEL, little silt
BB-LR-106	1D	0-2	14				0.0	42.7	57.3						Brown sandy Organic SILT
BB-LR-106	3D	10-12	15				9.5	25.1	65.4						Gray SILT & CLAY, little sand, trace gravel
BB-LR-106	6D	19-21	16				2.7	92.7	4.6						Brown SAND, trace silt, trace gravel

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brownish Gray Silty CLAY	44	20	24			
Project No. 09.0025950.00 Client: GZA GeoEnvironmental Project: Lunt Road Falmouth, ME Source of Sample: Borings Depth: 5-7' Sample Number: BB-LR-101 / 2D					Remarks:		
Thielsch Engineering Inc. Cranston, RI							
					Figure L-1		

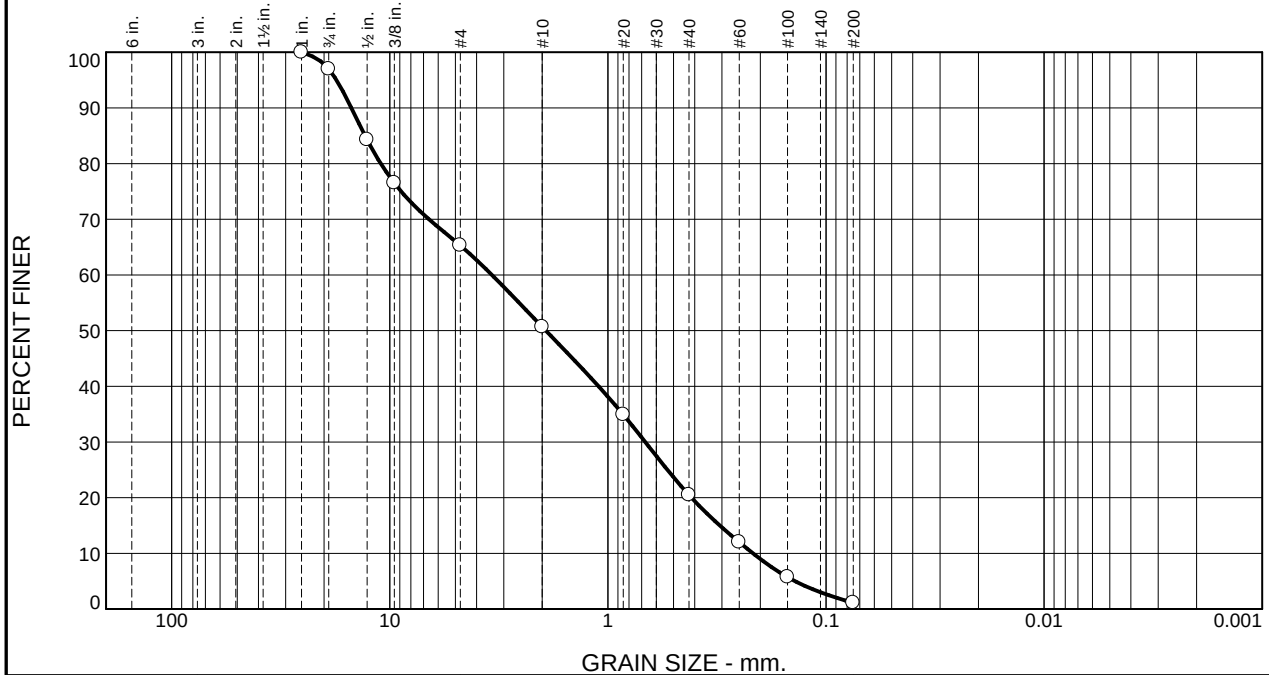
Figure L-1

Tested By: SA _____ Checked By: MJC _____



Tested By: SA **Checked By:** MJC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.0	31.7	14.6	30.2	19.4	1.1	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	97.0		
0.5"	84.3		
0.375"	76.5		
#4	65.3		
#10	50.7		
#20	34.9		
#40	20.5		
#60	12.0		
#100	5.7		
#200	1.1		

* (no specification provided)

Material Description

Light Brown SAND, some gravel, trace silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 15.0014 D₈₅= 12.9832 D₆₀= 3.4016
D₅₀= 1.9264 D₃₀= 0.6742 D₁₅= 0.3068
D₁₀= 0.2160 C_u= 15.75 C_c= 0.62

Remarks

Date Received: 08.31.17 Date Tested: 09.05.17

Tested By: SA

Checked By: Matthew Colman P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 1-3'
Sample Number: BB-LR-101 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

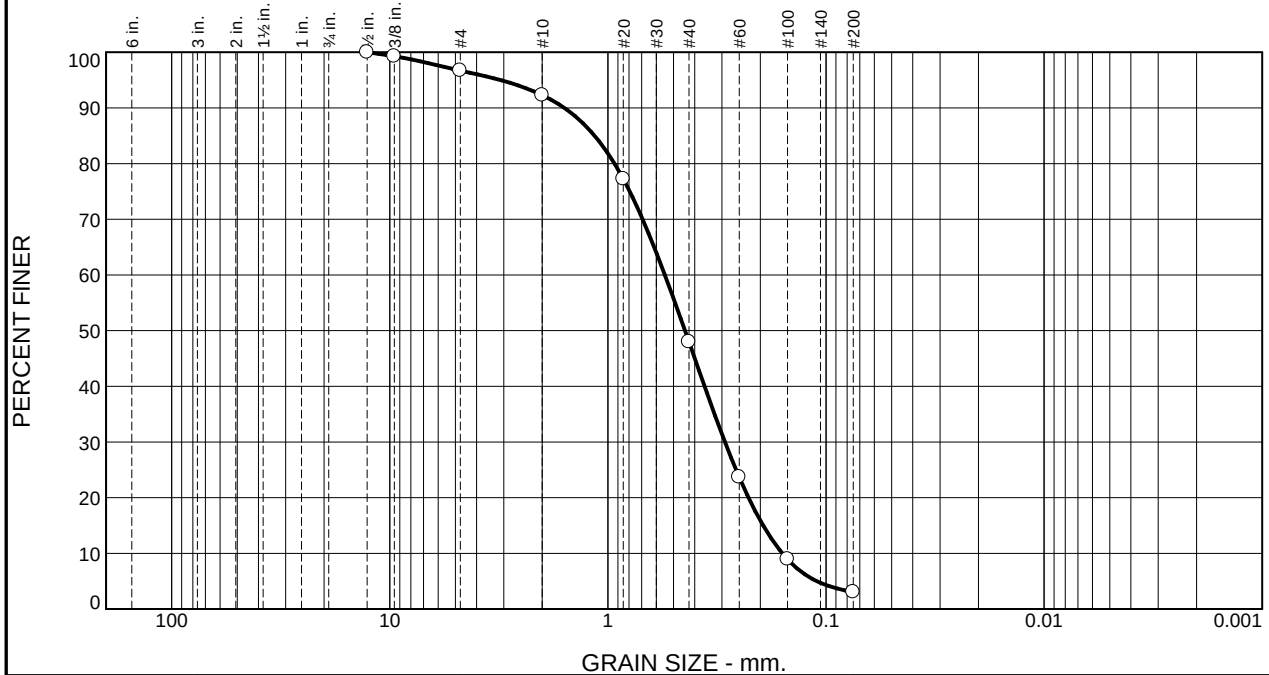
Client: GZA GeoEnvironmental

Project: Lunt Road
Falmouth, ME

Project No: 09.0025950.00

Figure S-9

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.3	4.4	44.3	45.0	3.0	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.5"	100.0		
0.375"	99.3		
#4	96.7		
#10	92.3		
#20	77.3		
#40	48.0		
#60	23.7		
#100	8.9		
#200	3.0		

* (no specification provided)

Material Description

Light Brown SAND, trace gravel, trace silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 1.5835 D₈₅= 1.1507 D₆₀= 0.5482
D₅₀= 0.4430 D₃₀= 0.2907 D₁₅= 0.1939
D₁₀= 0.1586 C_u= 3.46 C_c= 0.97

Remarks

Date Received: 08.31.17 Date Tested: 09.05.17

Tested By: SA

Checked By: Matthew Colman P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 0-2'
Sample Number: BB-LR-103 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

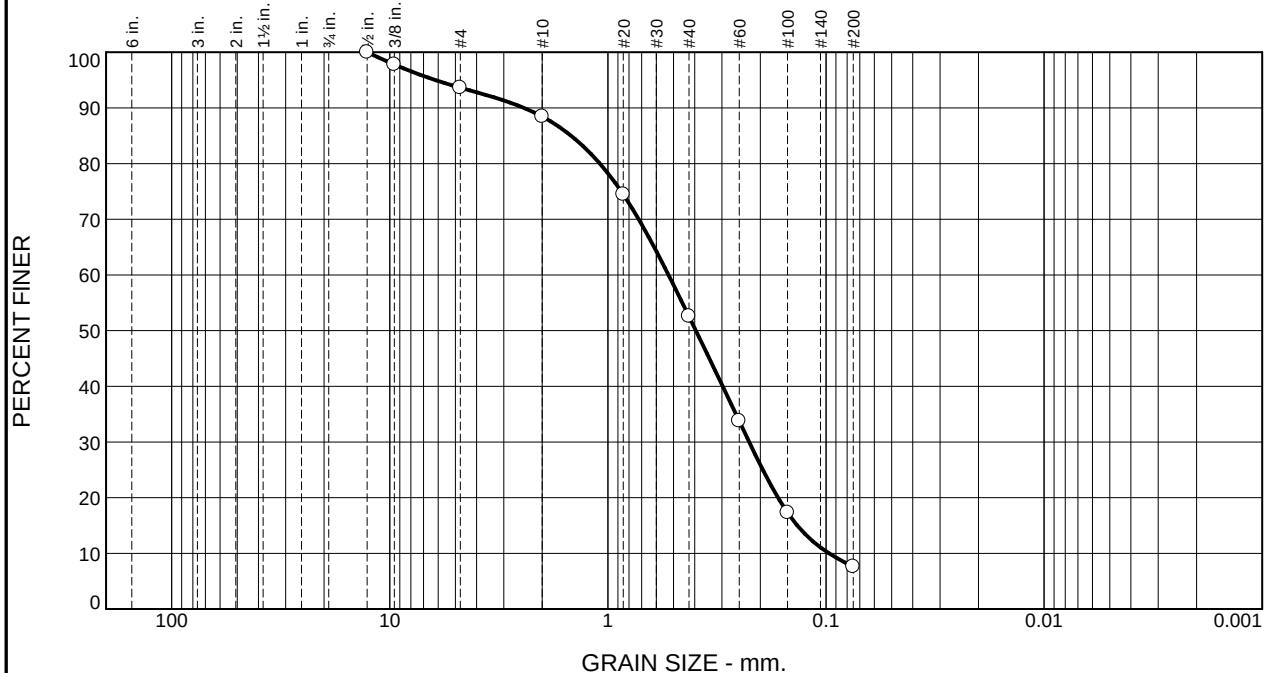
Client: GZA GeoEnvironmental

Project: Lunt Road
Falmouth, ME

Project No: 09.0025950.00

Figure S-10

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.4	5.2	35.9	44.9	7.6	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.5"	100.0		
0.375"	97.8		
#4	93.6		
#10	88.4		
#20	74.5		
#40	52.5		
#60	33.8		
#100	17.3		
#200	7.6		

* (no specification provided)

Material Description

Brown SAND, trace gravel, trace silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP-SM AASHTO (M 145)= A-3

Coefficients

D₉₀= 2.4251 D₈₅= 1.4742 D₆₀= 0.5274
D₅₀= 0.3952 D₃₀= 0.2250 D₁₅= 0.1353
D₁₀= 0.0968 C_u= 5.45 C_c= 0.99

Remarks

Date Received: 08.31.17 Date Tested: 09.05.17

Tested By: SA

Checked By: Matthew Colman P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 5-7'
Sample Number: BB-LR-103 / 2D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

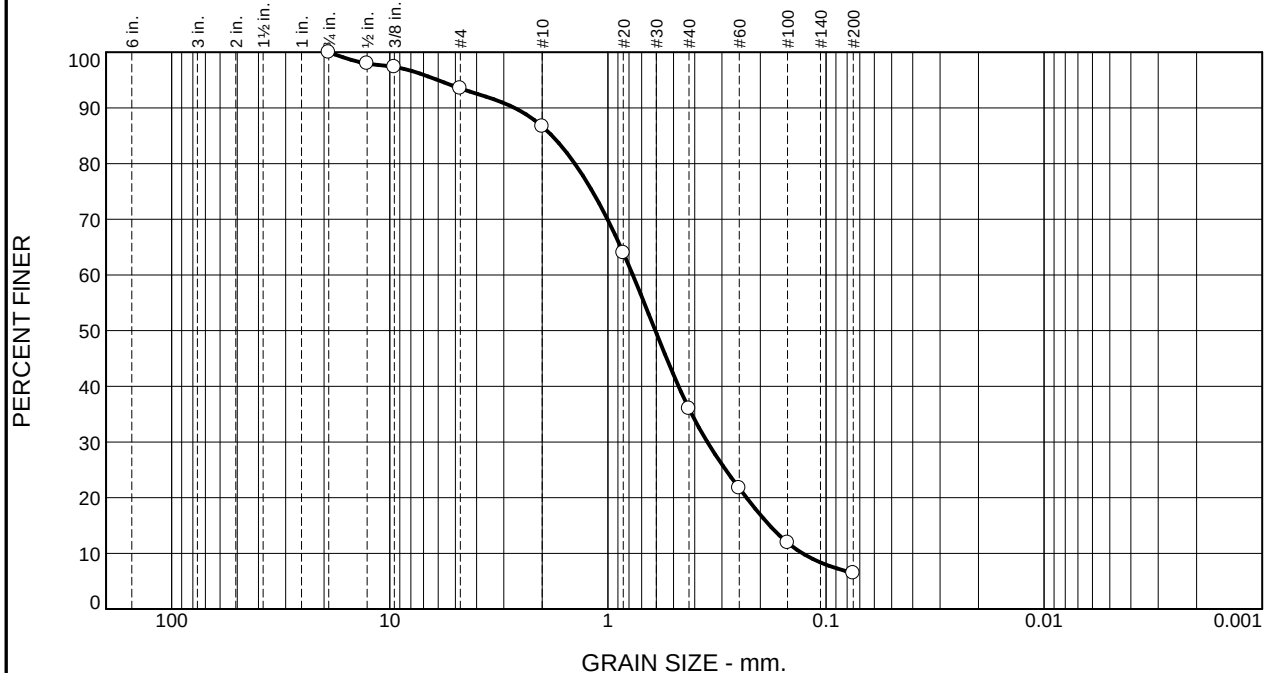
Client: GZA GeoEnvironmental

Project: Lunt Road
Falmouth, ME

Project No: 09.0025950.00

Figure S-11

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.5	6.8	50.7	29.5	6.5	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	98.0		
0.375"	97.4		
#4	93.5		
#10	86.7		
#20	64.0		
#40	36.0		
#60	21.7		
#100	11.9		
#200	6.5		

* (no specification provided)

Material Description

Light Brown SAND, trace gravel, trace silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP-SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 2.6778 D₈₅= 1.8023 D₆₀= 0.7688
D₅₀= 0.6060 D₃₀= 0.3501 D₁₅= 0.1811
D₁₀= 0.1282 C_u= 6.00 C_c= 1.24

Remarks

Date Received: 08.31.17 Date Tested: 09.05.17

Tested By: SA

Checked By: Matthew Colman P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 2-3'
Sample Number: BB-LR-105 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

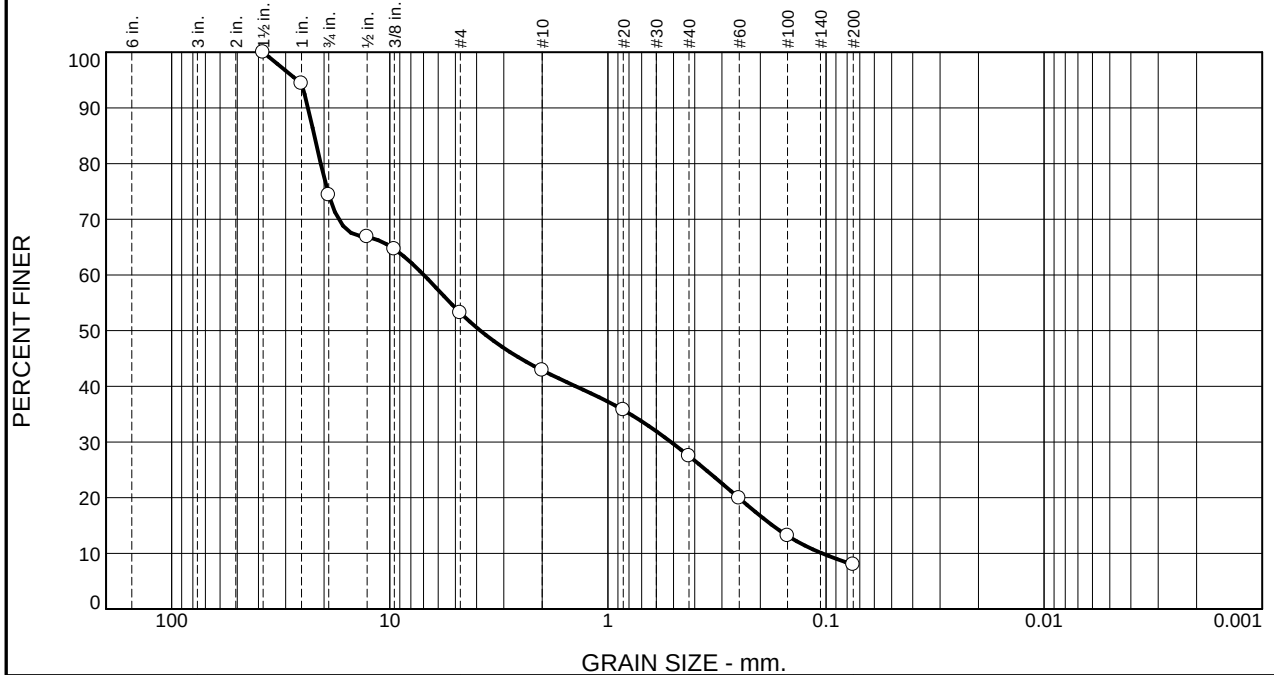
Client: GZA GeoEnvironmental

Project: Lunt Road
Falmouth, ME

Project No: 09.0025950.00

Figure S-12

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	25.6	21.2	10.3	15.4	19.5	8.0	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	94.4		
0.75"	74.4		
0.5"	66.8		
0.375"	64.6		
#4	53.2		
#10	42.9		
#20	35.7		
#40	27.5		
#60	19.9		
#100	13.1		
#200	8.0		

* (no specification provided)

Material Description

Dark Brown sandy GRAVEL, little silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= GP-GM AASHTO (M 145)= A-1-a

Coefficients

D₉₀= 23.7100 D₈₅= 22.1659 D₆₀= 6.9833
D₅₀= 3.8442 D₃₀= 0.5141 D₁₅= 0.1756
D₁₀= 0.1042 C_u= 67.01 C_c= 0.36

Remarks

Date Received: 08.31.17 Date Tested: 09.05.17

Tested By: SA

Checked By: Matthew Colman P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 5-7'
Sample Number: BB-LR-105 / 2D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

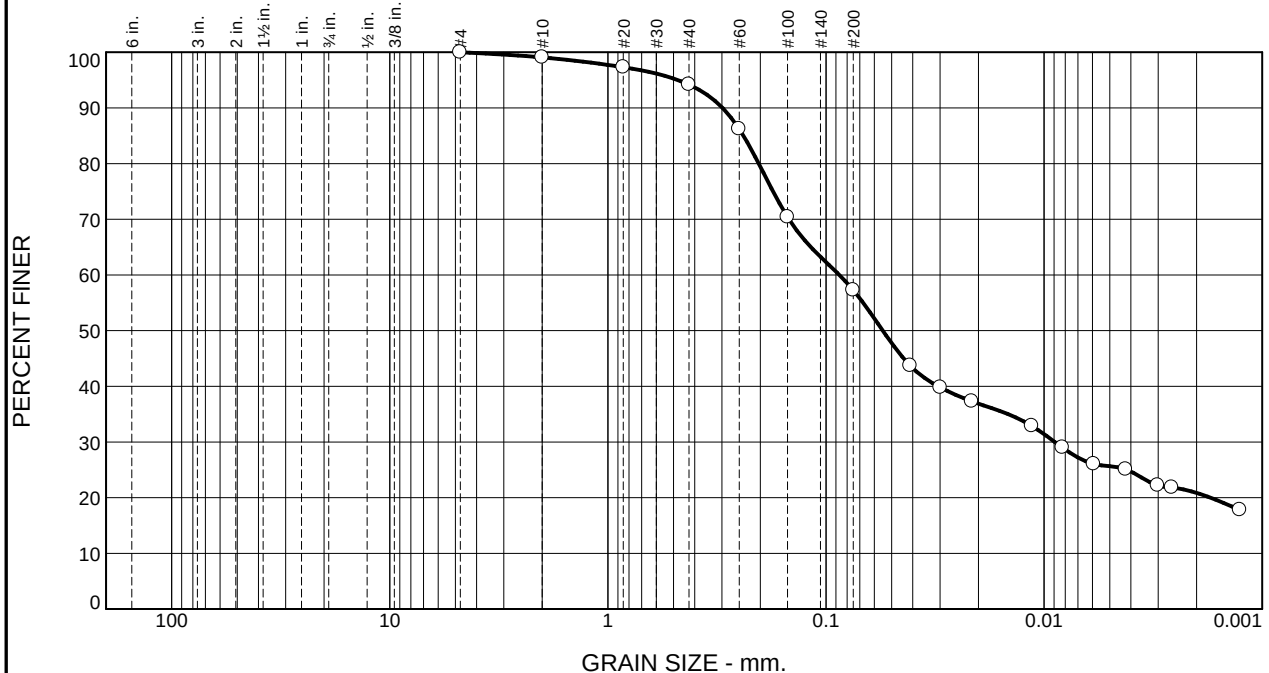
Client: GZA GeoEnvironmental

Project: Lunt Road
Falmouth, ME

Project No: 09.0025950.00

Figure S-13

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.9	4.9	36.9	36.4	20.9

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.1		
#20	97.3		
#40	94.2		
#60	86.2		
#100	70.4		
#200	57.3		
0.0411 mm.	43.7		
0.0298 mm.	39.8		
0.0214 mm.	37.3		
0.0114 mm.	32.9		
0.0082 mm.	29.0		
0.0059 mm.	26.0		
0.0042 mm.	25.1		
0.0030 mm.	22.2		
0.0026 mm.	21.8		
0.0013 mm.	17.8		

* (no specification provided)

Material Description Brown sandy Organic SILT		
Atterberg Limits (ASTM D 4318) PL= LL= PI=		
Classification USCS (D 2487)= AASHTO (M 145)=		
Coefficients D ₉₀ = 0.2982 D ₈₅ = 0.2390 D ₆₀ = 0.0869 D ₅₀ = 0.0549 D ₃₀ = 0.0089 D ₁₅ = D ₁₀ = C _u = C _c =		
Remarks		
Date Received: 08.31.17 Date Tested: 09.06.17 Tested By: SA Checked By: Matthew Colman P.E. Title: Laboratory Manager		

Source of Sample: Borings Depth: 0-2'
 Sample Number: BB-LR-106 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

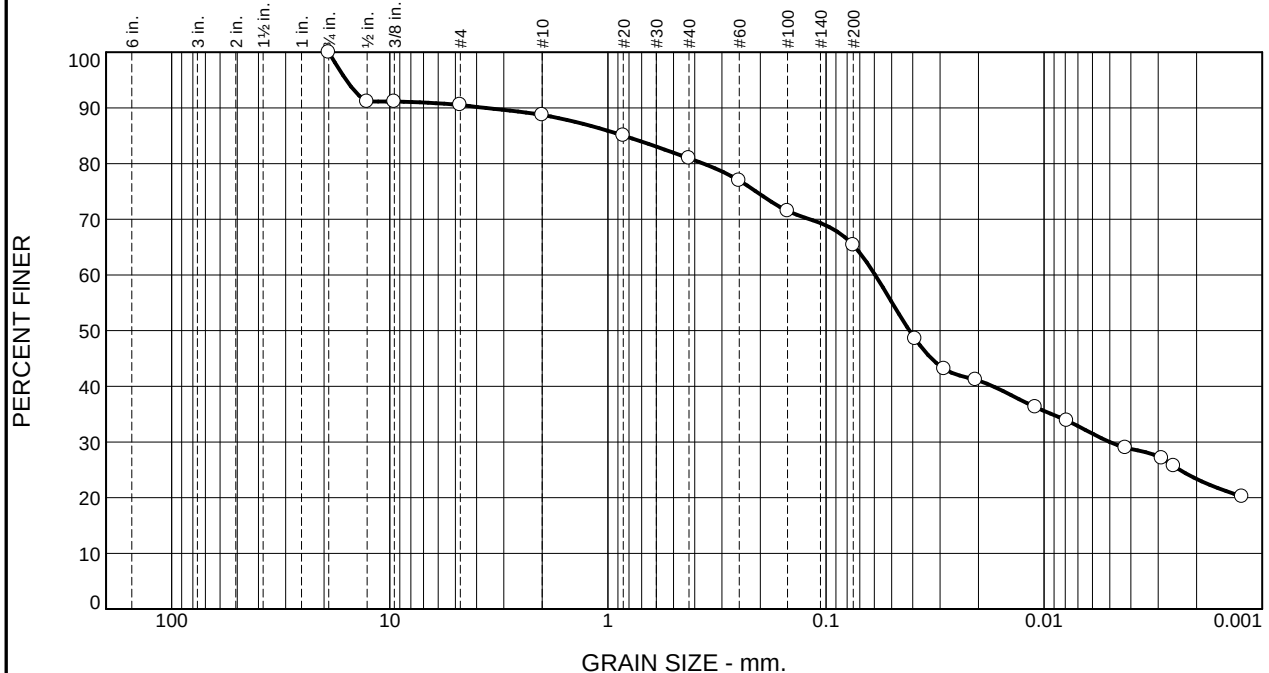
Client: GZA GeoEnvironmental

Project: Lunt Road
 Falmouth, ME

Project No: 09.0025950.00

Figure S-14

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	9.5	1.8	7.7	15.6	42.0	23.4

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	91.1		
0.375"	91.1		
#4	90.5		
#10	88.7		
#20	85.0		
#40	81.0		
#60	77.0		
#100	71.5		
#200	65.4		
0.0390 mm.	48.5		
0.0287 mm.	43.1		
0.0206 mm.	41.2		
0.0110 mm.	36.3		
0.0079 mm.	33.8		
0.0042 mm.	29.0		
0.0029 mm.	27.1		
0.0025 mm.	25.7		
0.0012 mm.	20.2		

* (no specification provided)

Material Description

Gray SILT & CLAY, little sand, trace gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 3.6442 D₈₅= 0.8454 D₆₀= 0.0595
D₅₀= 0.0414 D₃₀= 0.0050 D₁₅=
D₁₀= C_u= C_c=

Remarks

Material plasticity is visual estimate only.

Date Received: 08.31.17

Date Tested: 09.06.17

Tested By: SA

Checked By: Matthew Colman P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 10-12'
Sample Number: BB-LR-106 / 3D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

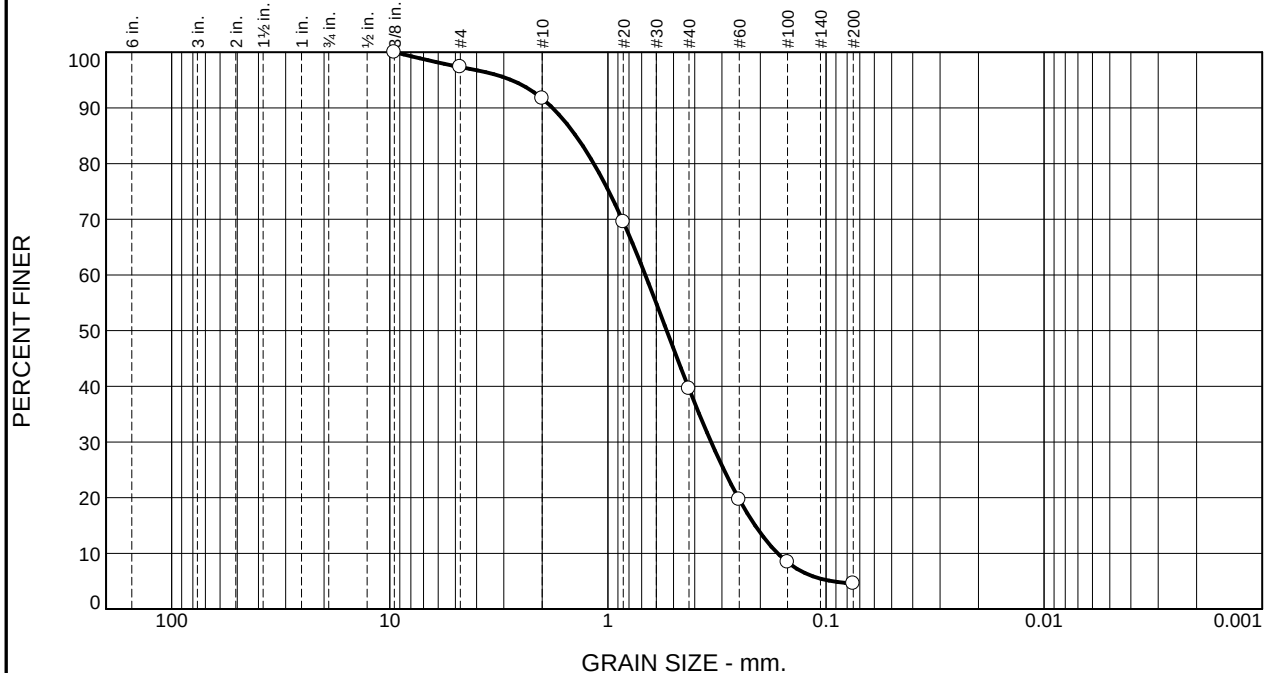
Client: GZA GeoEnvironmental

Project: Lunt Road
Falmouth, ME

Project No: 09.0025950.00

Figure S-15

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.7	5.6	52.1	35.0	4.6	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375"	100.0		
#4	97.3		
#10	91.7		
#20	69.5		
#40	39.6		
#60	19.7		
#100	8.4		
#200	4.6		

* (no specification provided)

Material Description

Brown SAND, trace silt, trace gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 1.7934 D₈₅= 1.4025 D₆₀= 0.6737
D₅₀= 0.5377 D₃₀= 0.3363 D₁₅= 0.2112
D₁₀= 0.1666 C_u= 4.04 C_c= 1.01

Remarks

Sample contained Standing Water

Date Received: 08.31.17 Date Tested: 09.05.17

Tested By: SA

Checked By: Matthew Colman P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 19-21'
Sample Number: BB-LR-106 / 6D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: Lunt Road
Falmouth, ME

Project No: 09.0025950.00

Figure S-16

Matthew Cohen

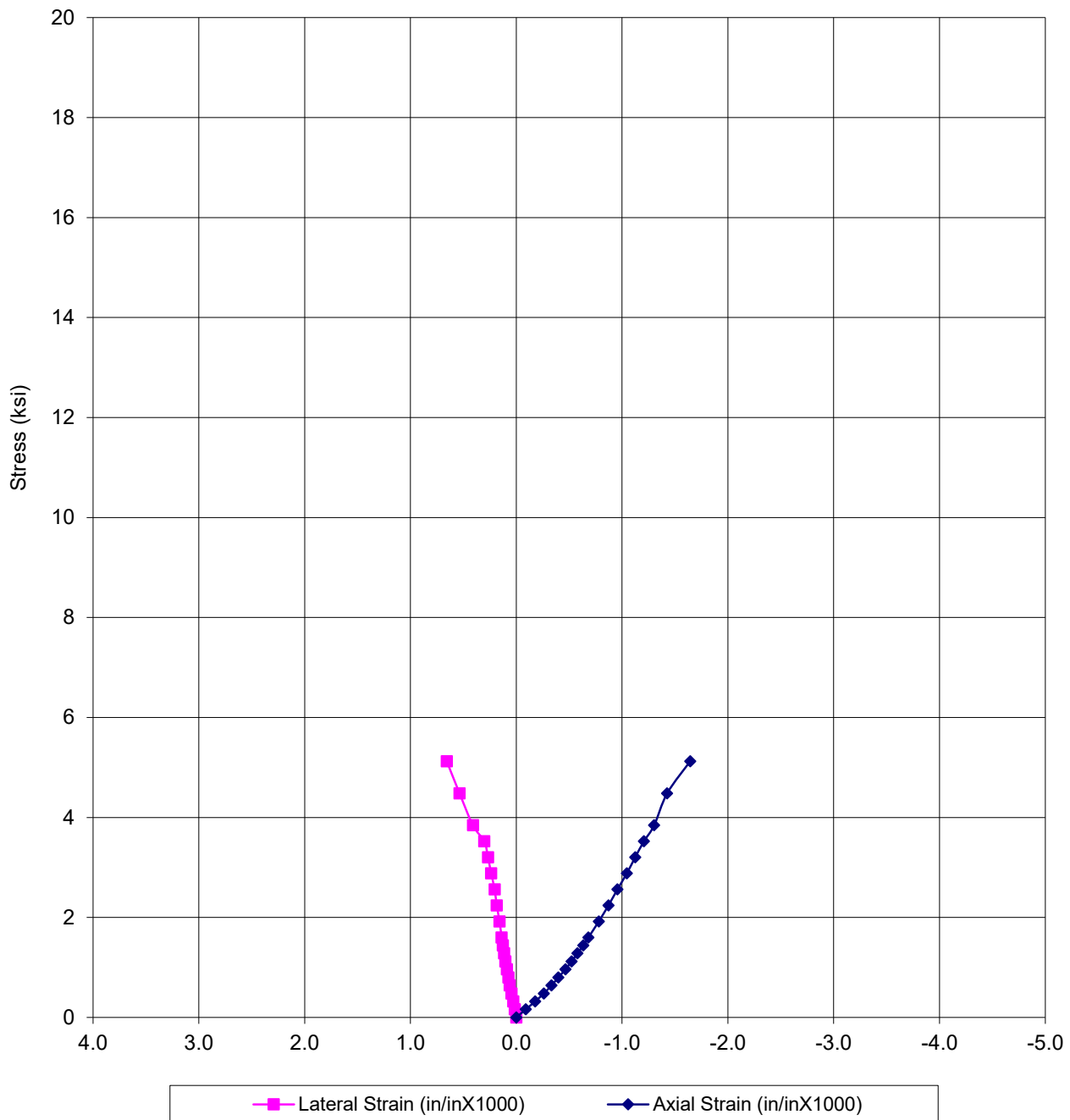
Reviewed By _____

Date Revised **09.13.17**

Notes:

(1) Volume Determined By Measuring Dimensions	(3) P=Petrographic PLD=Point Load (diametrical), PLA= Point Load (Axial) ST= Splitting Tensile U= Unconfined Compressive Strength	(5) Strain at Peak Deviator Stress	(9) Estimated Uniaxial Compressive Strength from Table 1 of ASTM D5731 for NX cores (Is x 24)
(2) Determined by Measuring Dimensions and Weight of Saturated Sample	(4) Taken at Peak Deviator Stress	(6) Cerchar Abrasivity Index (7) Young's Modulus (8) Poisson's Ratio	

Lunt Road ME



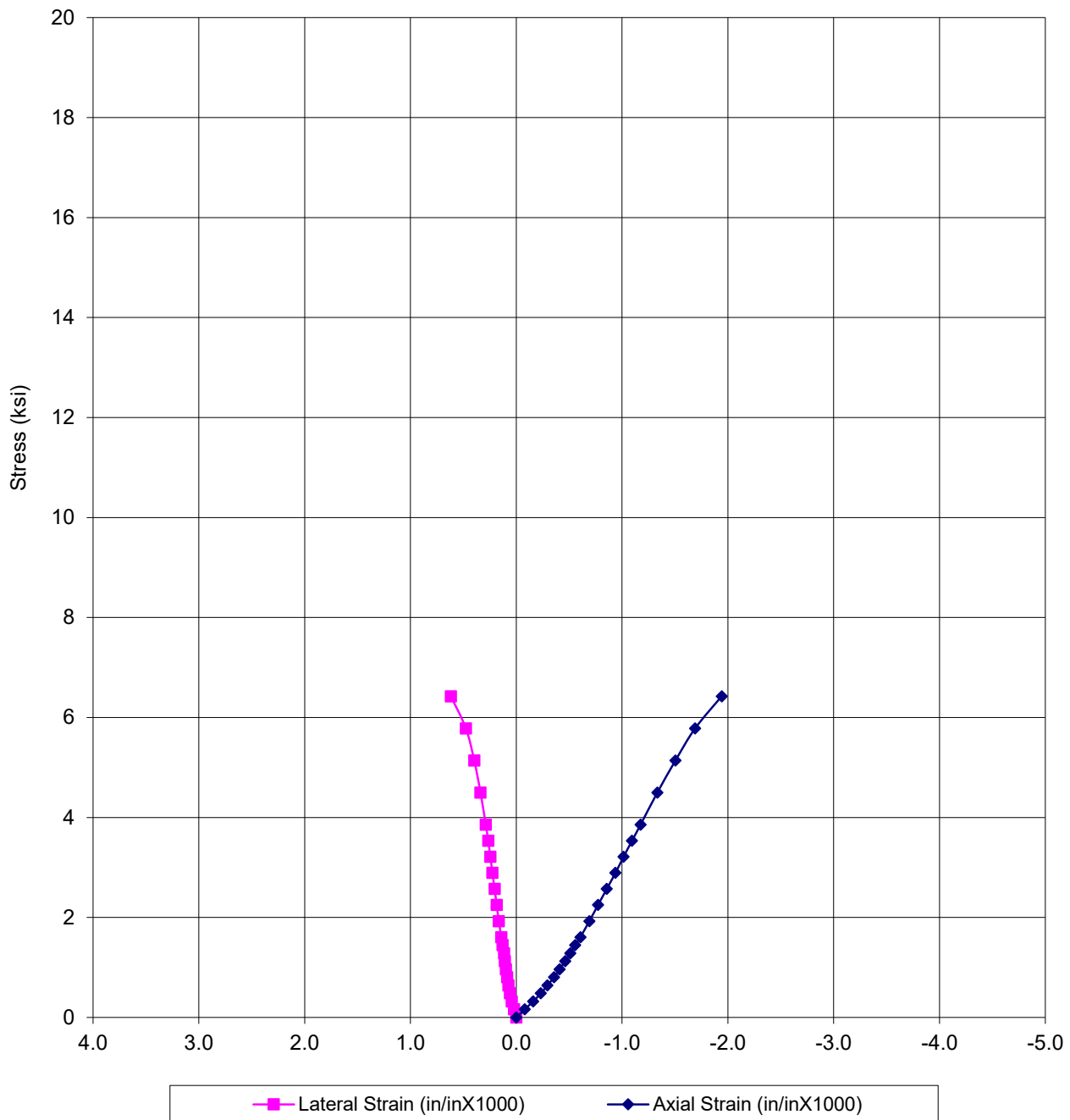
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-LR-102
Sample No. R1
Depth: 11.8-12.2

File No. 09.002595.00
Date: 09/01/17
Test No. R-17



Lunt Road
ME



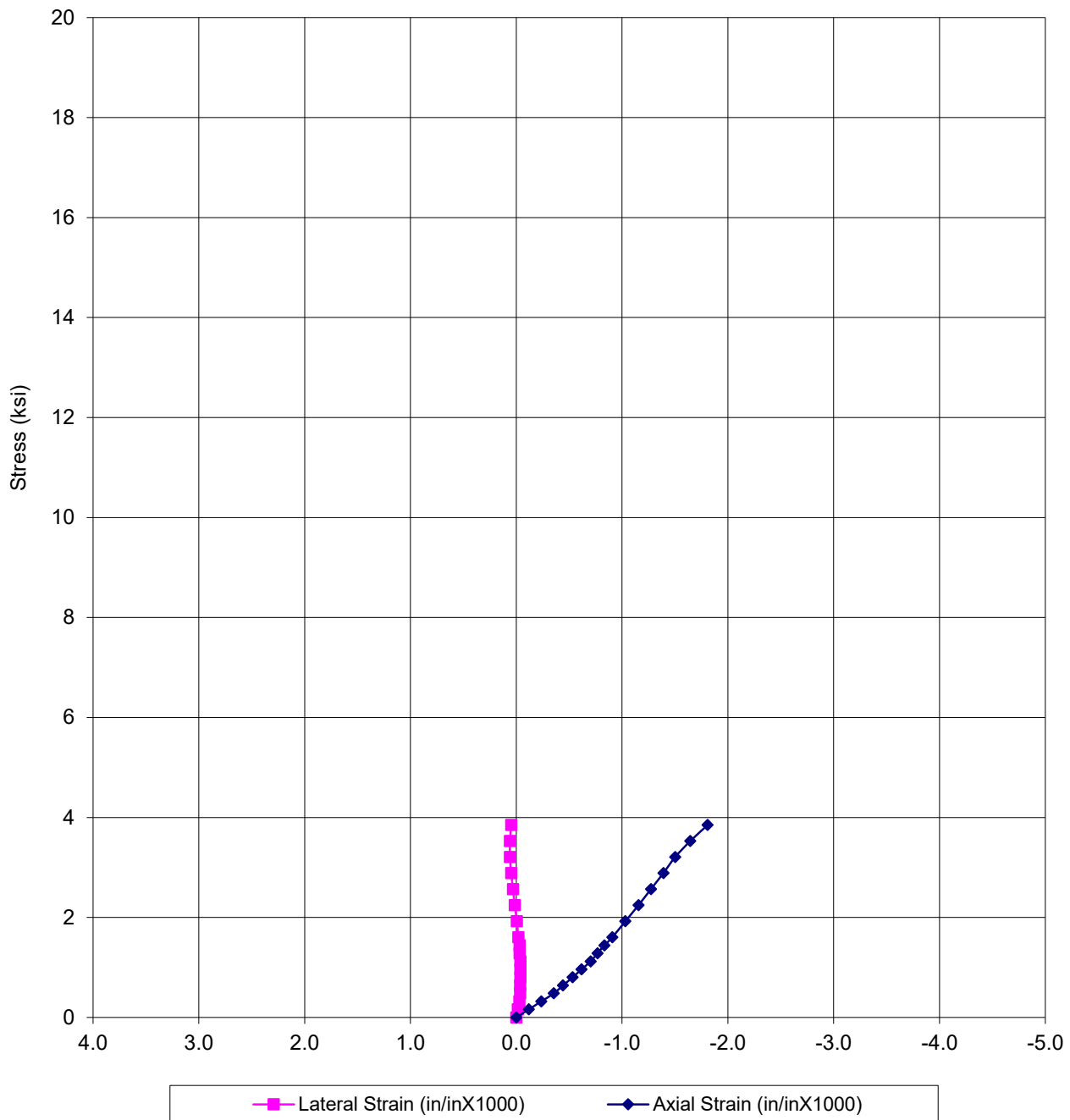
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-LR-103
Sample No. R1
Depth: 8.1-8.5

File No. 09.002595.00
Date: 09/01/17
Test No. R-18



Lunt Road ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-LR-104
Sample No. R1
Depth: 19.3-19.7

File No. 09.002595.00
Date: 09/01/17
Test No. R-19



Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



09.0025950.00
BB-LR-102

Boring No.	Sample No.	Depth
<u>BB-LR-102</u>	<u>R1</u>	<u>11.7-12.3'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.

Sample No.

Depth

BB-LR-102

R1

11.7-12.3'

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.

Sample No.

Depth

BB-LR-102

R1

11.7-12.3'

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



09.0025950
BB-LR-102
R1
11.7

Boring No.	Sample No.	Depth
<u>BB-LR-102</u>	<u>R1</u>	<u>11.7-12.3'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



195 Frances Avenue
Cranston, RI 02910

THIELSCH ENGINEERING, INC.
24 HOUR EMERGENCY SERVICE

Tel. (401) 467-6454
Fax (401) 467-2398

Boring No. Sample No. Depth

BB-LR-102

R1

11.7-12.3'

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.	Sample No.	Depth
<u>BB-LR-103</u>	<u>R1</u>	<u>8-8.75'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.	Sample No.	Depth
<u>BB-LR-103</u>	<u>R1</u>	<u>8-8.75'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.	Sample No.	Depth
<u>BB-LR-103</u>	<u>R1</u>	<u>8-8.75'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.	Sample No.	Depth
<u>BB-LR-103</u>	<u>R1</u>	<u>8-8.75'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.	Sample No.	Depth
<u>BB-LR-104</u>	<u>R1</u>	<u>19.1-19.8'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.	Sample No.	Depth
<u>BB-LR-104</u>	<u>R1</u>	<u>19.1-19.8'</u>

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



Boring No.

Sample No.

Depth

BB-LR-104

R1

19.1-19.8'

Lunt Road
Falmouth, ME

GZA Project # 09.0025950.00



195 Frances Avenue
Cranston, RI 02910

THIELSCH ENGINEERING, INC.
24 HOUR EMERGENCY SERVICE

Tel. (401) 467-6454
Fax (401) 467-2398

BB-LR-104
19.1-19.8
09.0025950.00

Boring No.

Sample No.

Depth

BB-LR-104

R1

19.1-19.8'

**R. W. Gillespie & Associates, Inc.**

86 Industrial Park Road, Suite 4, Saco, ME 04072 207-286-8008
200 Int'l Drive, Suite 170, Portsmouth, NH 03801 603-427-0244
44 Wood Avenue, Suite I, Mansfield, MA 508-623-0101

LETTER OF TRANSMITTAL

Date: 9/18/2017	Project No.: 0876-013
Attention: Christopher Snow (christopher.snow@GZA.com)	
Re: Laboratory Testing BB-Lunt Road-101 Falmouth, ME	

GZA Geoenvironmental

477 Congress Street

Portland, ME 04101

We are sending you attached Laboratory Test Results.

Laboratory No. (s)

Test (s) Performed

14690: Tube Sample 13'-15', Falmouth, ME

Consolidation

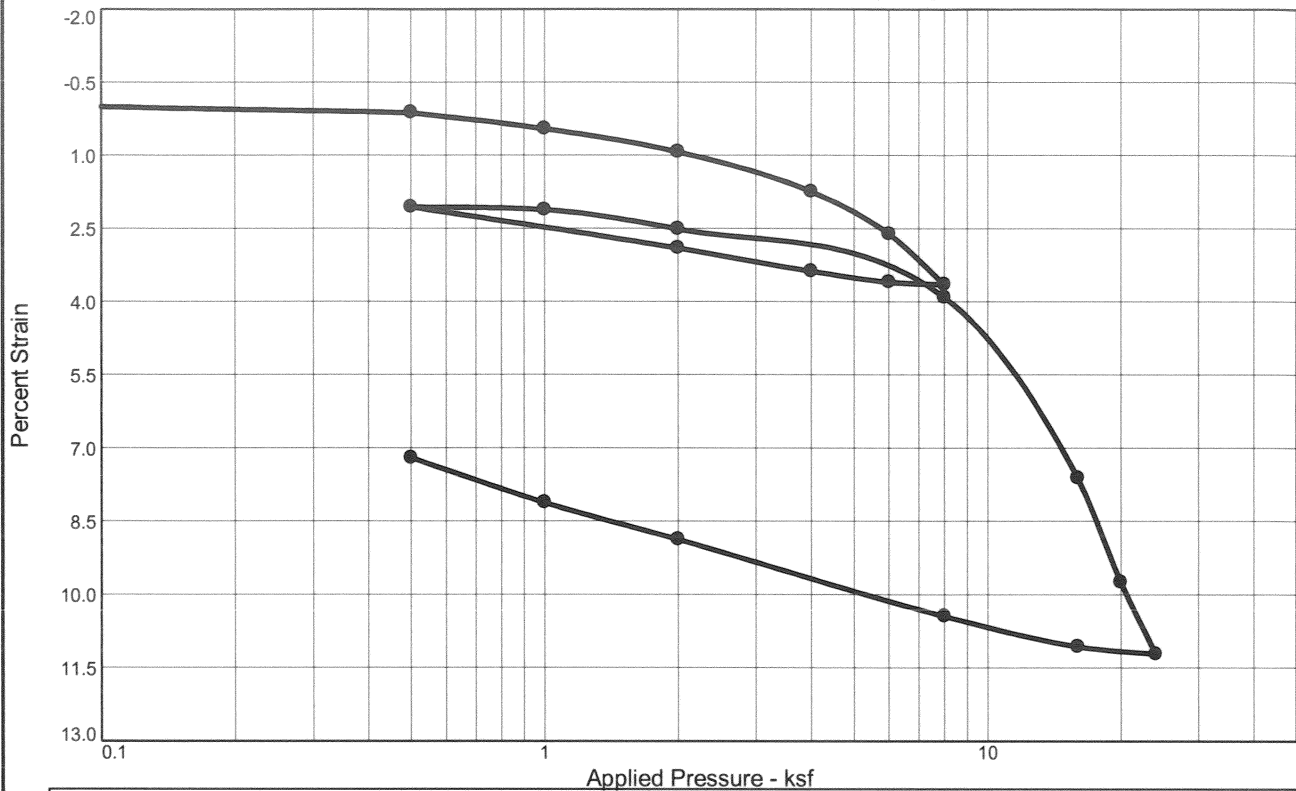
Remarks:

--

Copy to:

If enclosures are not noted, kindly notify us as once.

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.50	2.157		8	4.00	5.601		15	20.00	0.252	
2	1.00	1.914		9	2.00	2.223		16	24.00	0.164	
3	2.00	2.302		10	0.50	0.591		17	16.00	7.007	
4	4.00	2.135	0.002	11	1.00	0.484	0.000	18	8.00	2.715	
5	6.00	0.895	0.004	12	2.00	0.536	0.000	19	2.00	0.427	
6	8.00	0.102	0.005	13	8.00	1.947	0.002	20	1.00	0.213	
7	6.00	6.156		14	16.00	0.567	0.014	21	0.50	0.183	

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
98.0 %	39.8 %	81.5	51.6	25.9	2.75							1.117

MATERIAL DESCRIPTION

Fat Clay

USCS

AASHTO

Project No. 0876-013 **Client:** GZA GeoEnvironmental, Inc.

Project: Laboratory Testing

Loc.: BB-LR-101: Falmouth, ME **Depth:** 13'-15' **Sample No.:** BB-LR-101

R.W. Gillespie & Associates, Inc.

Saco, Maine

Remarks:

Lab No. 14690

Tested By: JRF

Checked By: EJW

✓
EJW

Dial Reading vs. Time

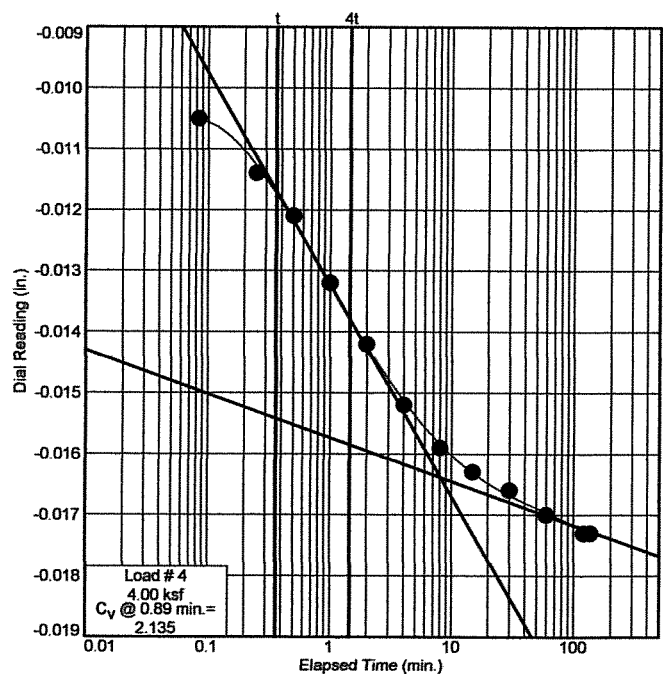
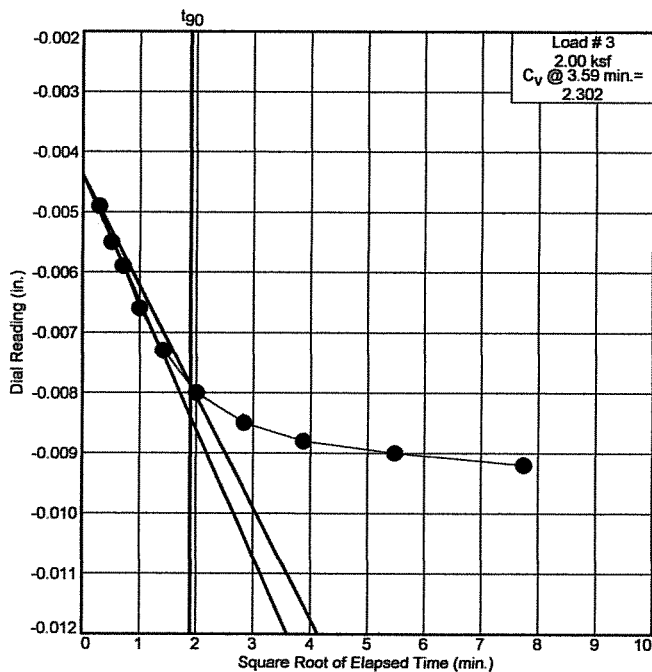
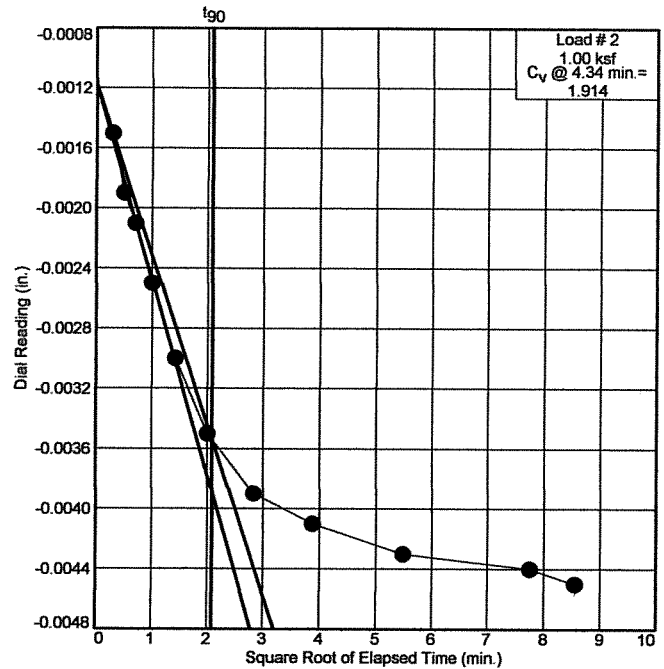
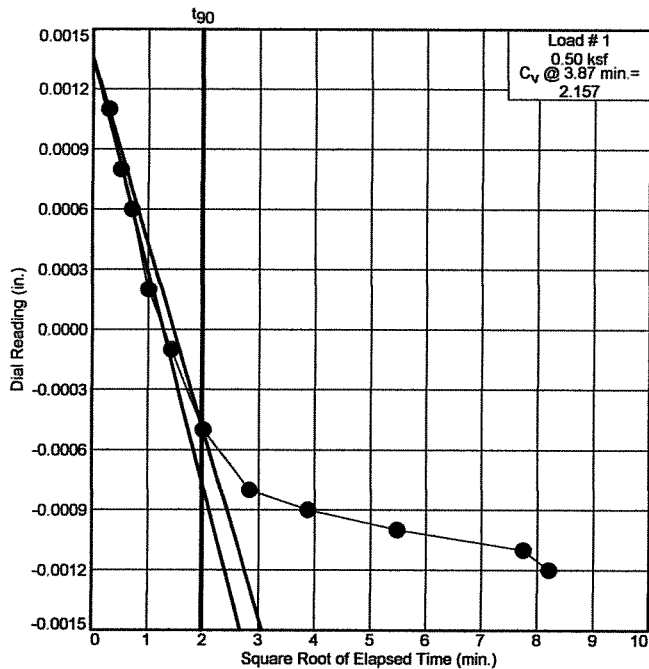
Project No.: 0876-013

Project: Laboratory Testing

Location: BB-LR-101: Falmouth, ME

Depth: 13'-15'

Sample Number: BB-LR-101



R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 14691

msb

Dial Reading vs. Time

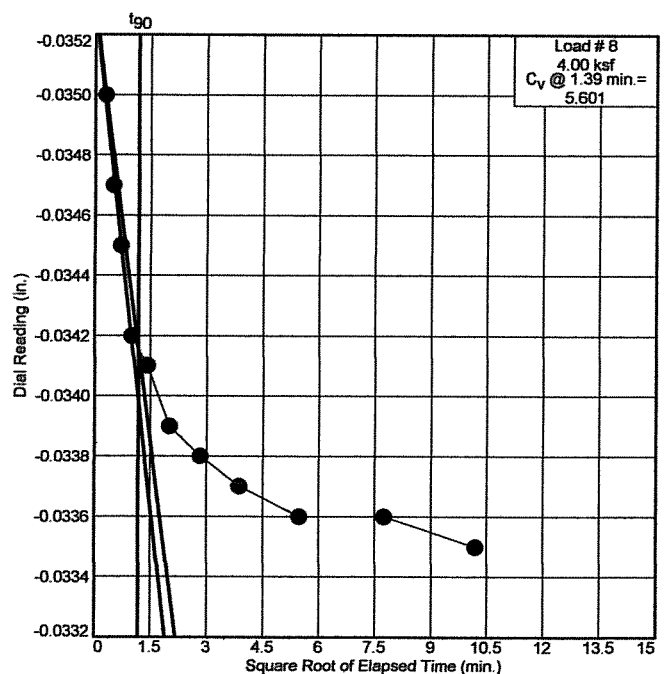
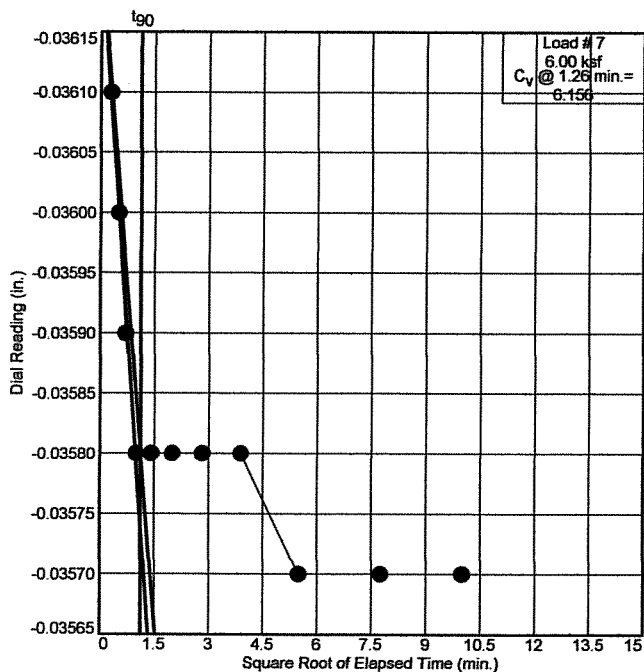
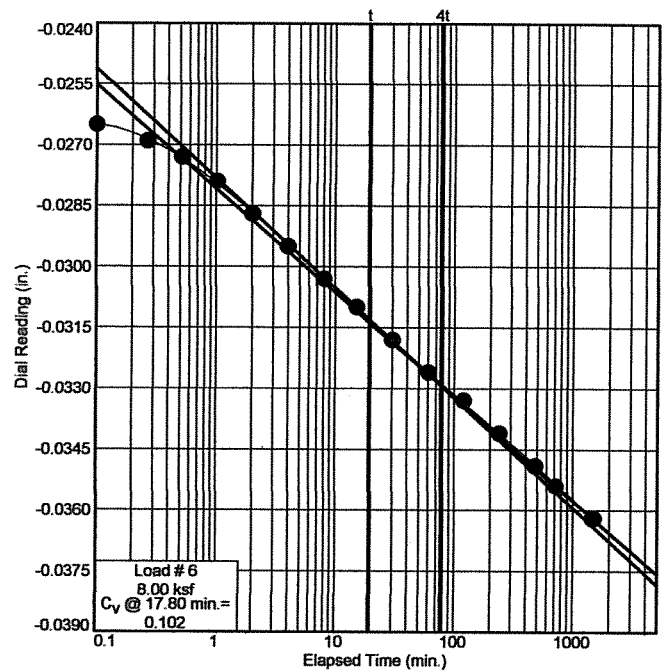
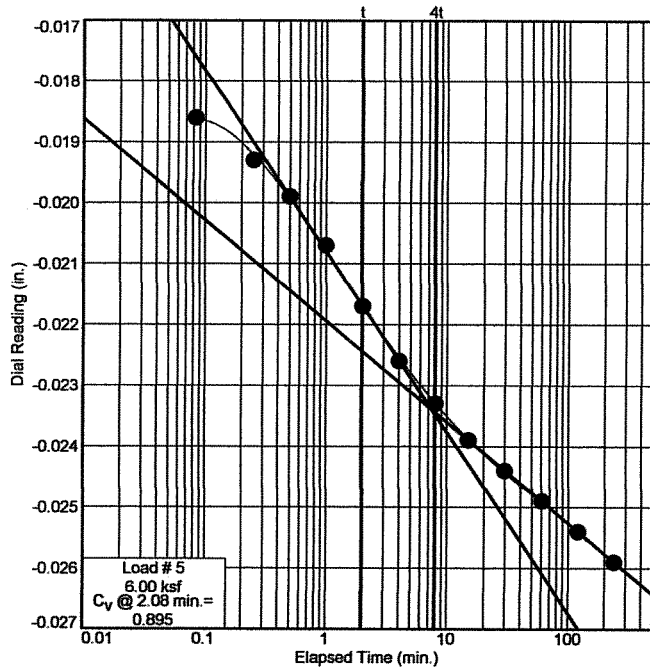
Project No.: 0876-013

Project: Laboratory Testing

Location: BB-LR-101: Falmouth, ME

Depth: 13'-15'

Sample Number: BB-LR-101



R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 14692

MR

Dial Reading vs. Time

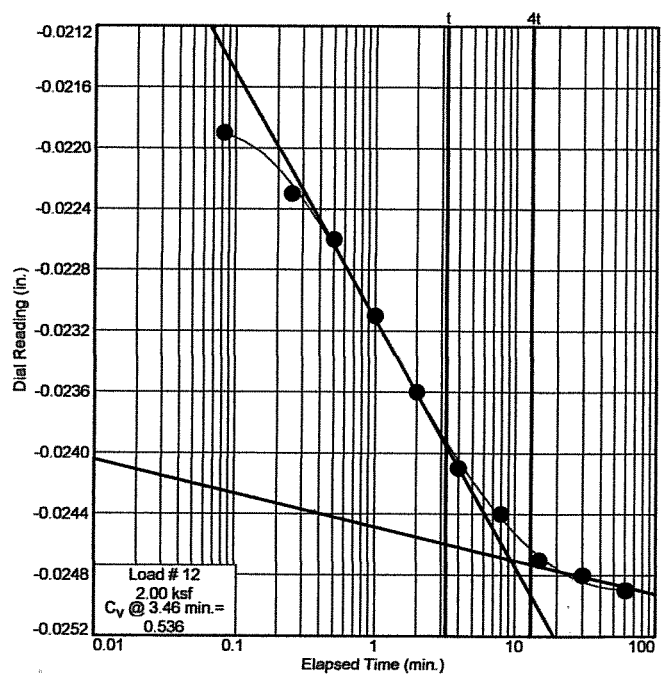
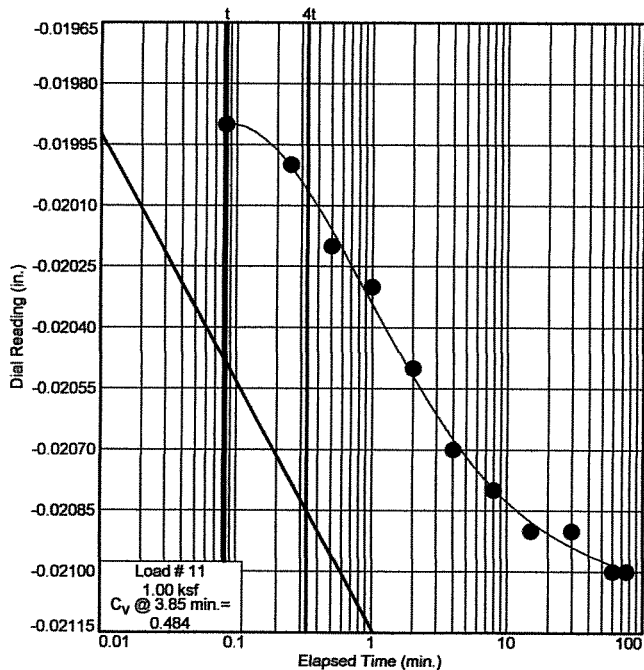
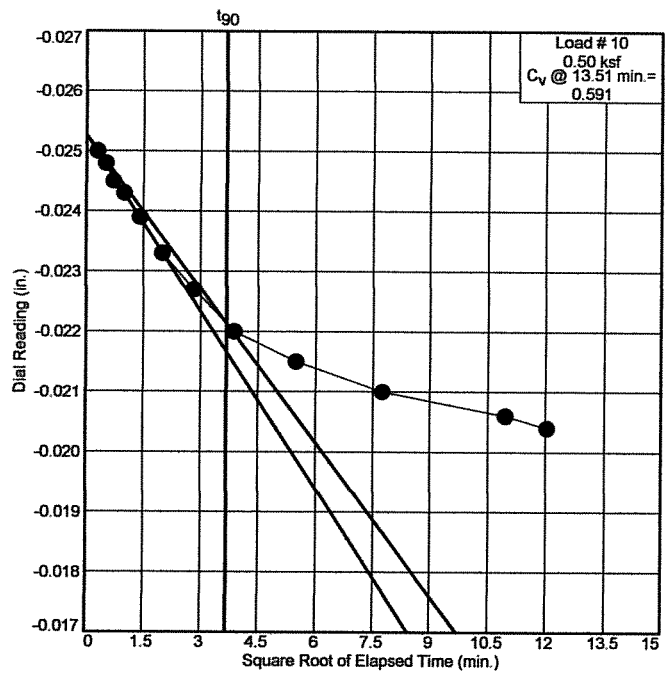
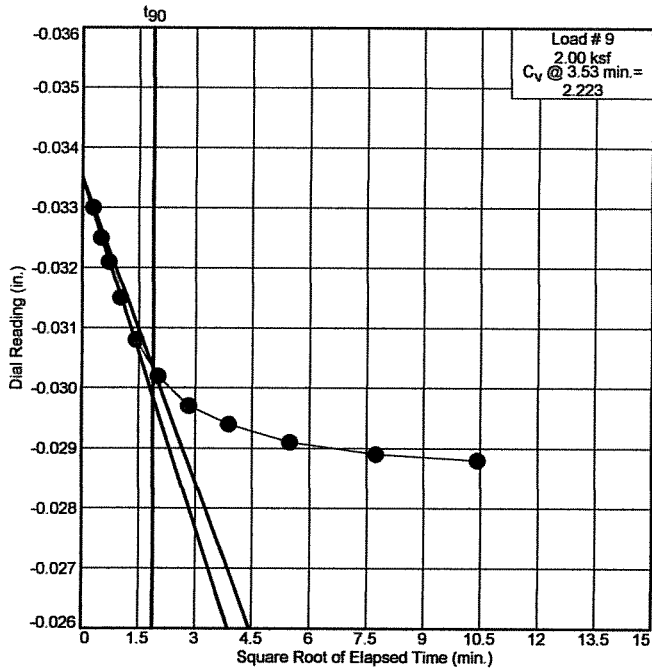
Project No.: 0876-013

Project: Laboratory Testing

Location: BB-LR-101: Falmouth, ME

Depth: 13'-15'

Sample Number: BB-LR-101



R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 14693

MTG

Dial Reading vs. Time

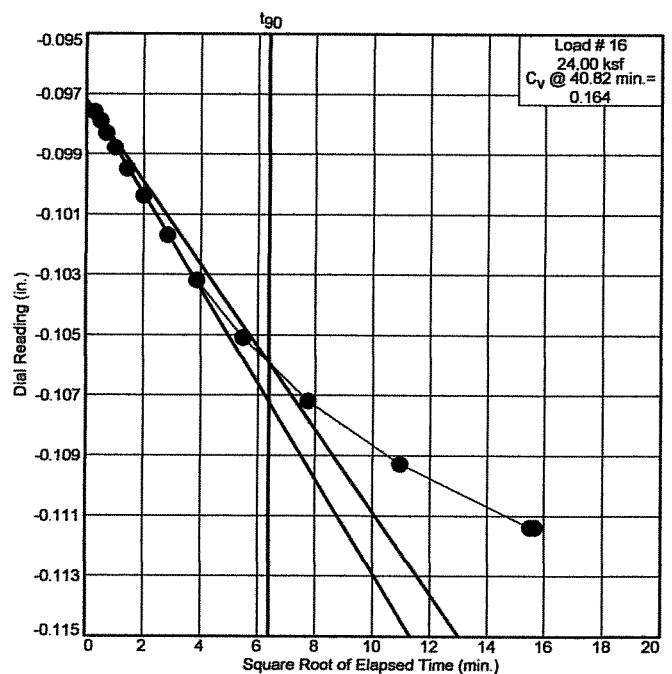
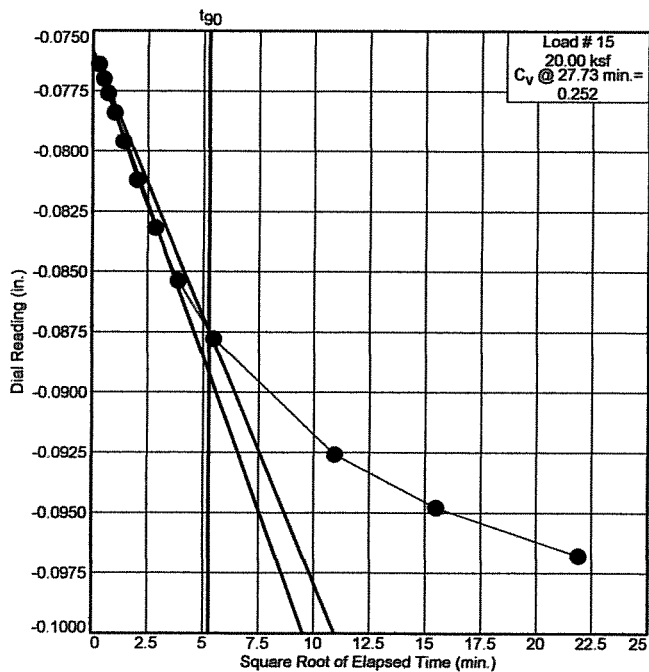
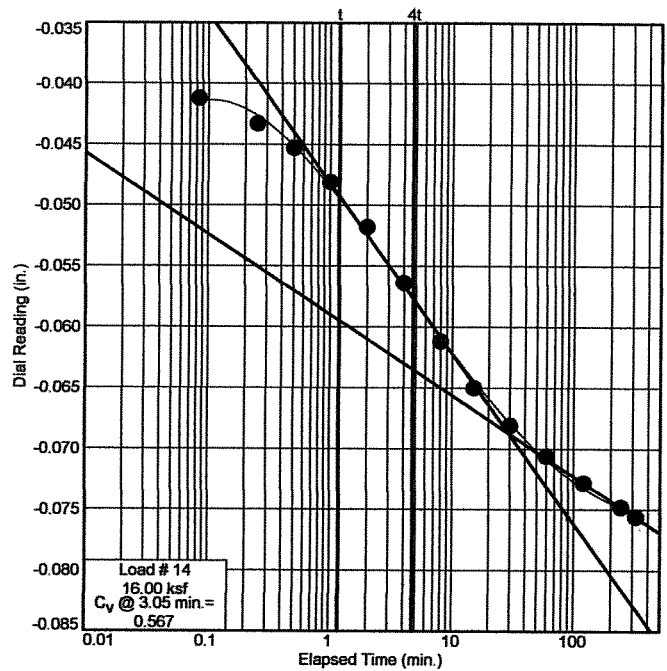
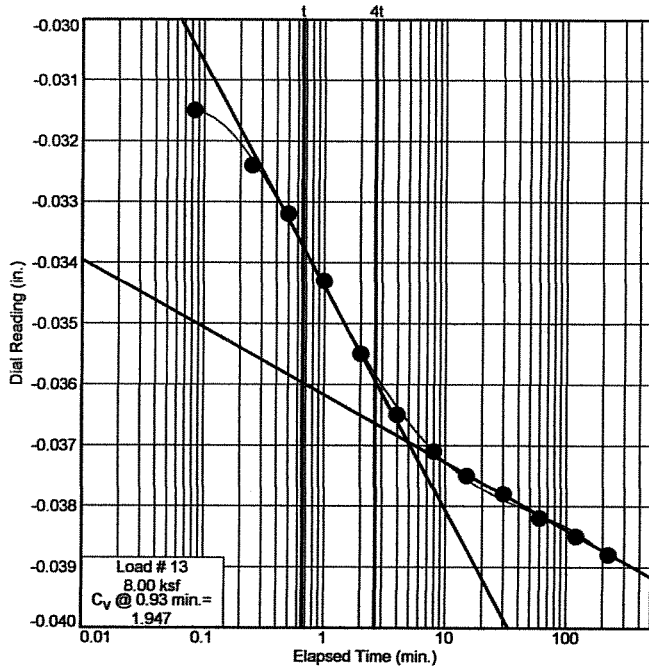
Project No.: 0876-013

Project: Laboratory Testing

Location: BB-LR-101: Falmouth, ME

Depth: 13'-15'

Sample Number: BB-LR-101



R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 14694

MB

Dial Reading vs. Time

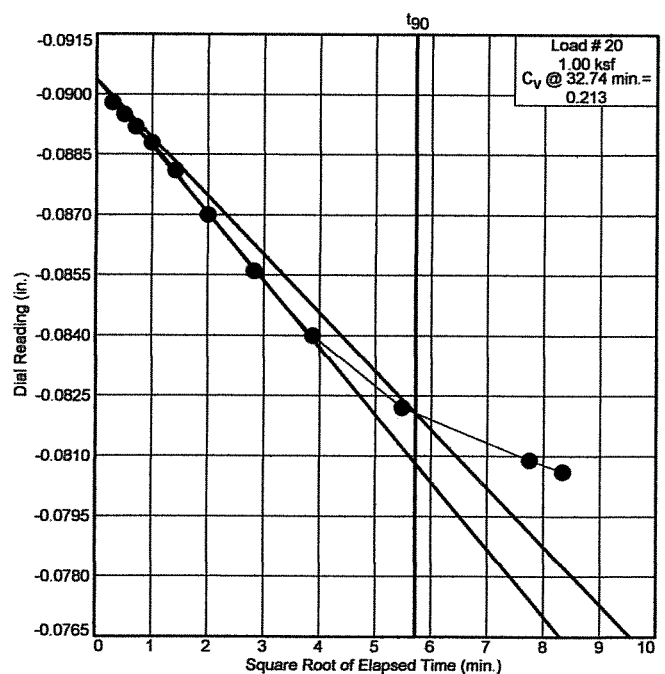
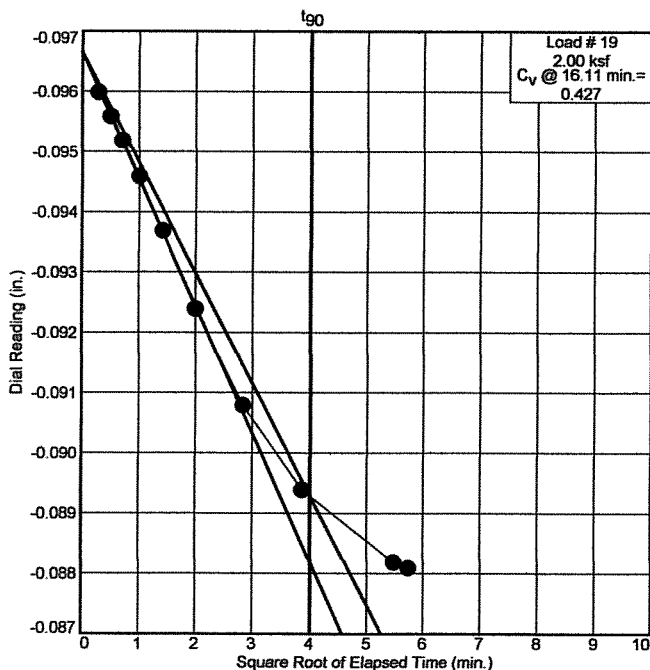
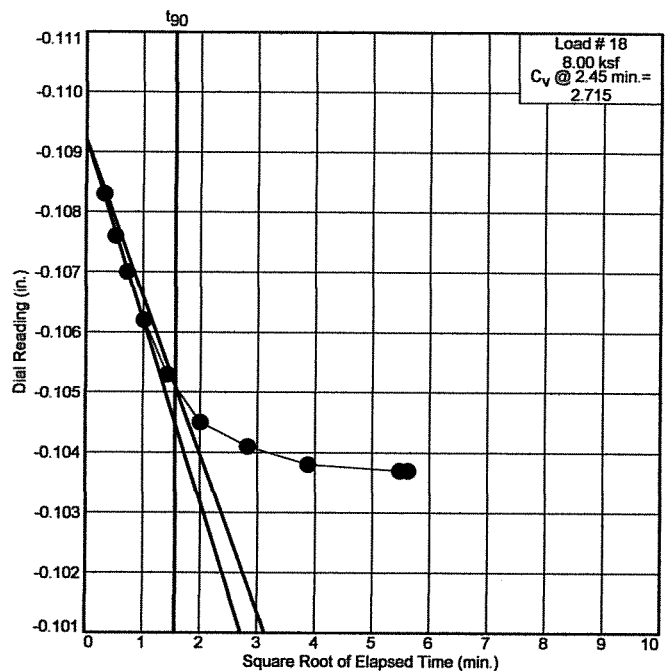
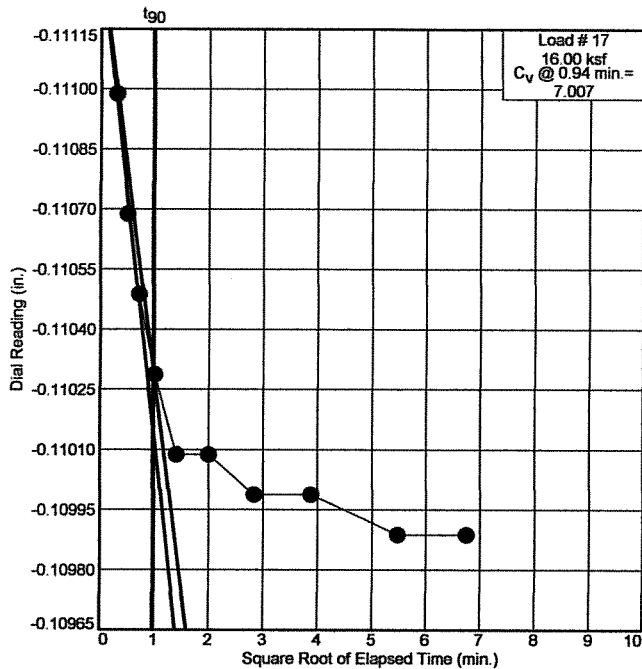
Project No.: 0876-013

Project: Laboratory Testing

Location: BB-LR-101: Falmouth, ME

Depth: 13'-15'

Sample Number: BB-LR-101



R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 14695

MTB

Dial Reading vs. Time

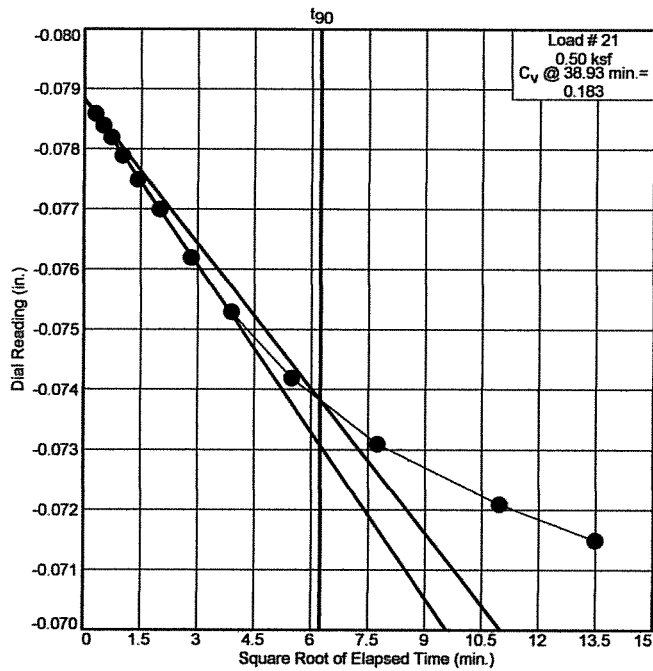
Project No.: 0876-013

Project: Laboratory Testing

Location: BB-LR-101: Falmouth, ME

Depth: 13'-15'

Sample Number: BB-LR-101

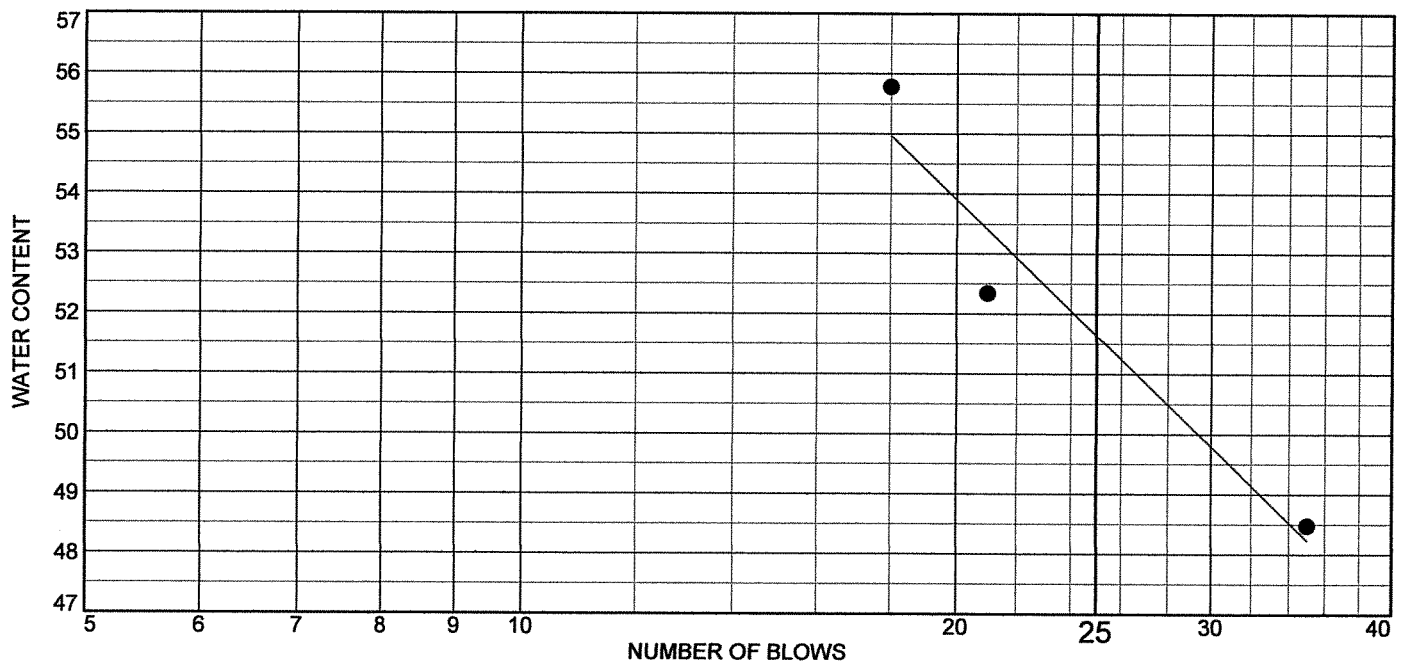
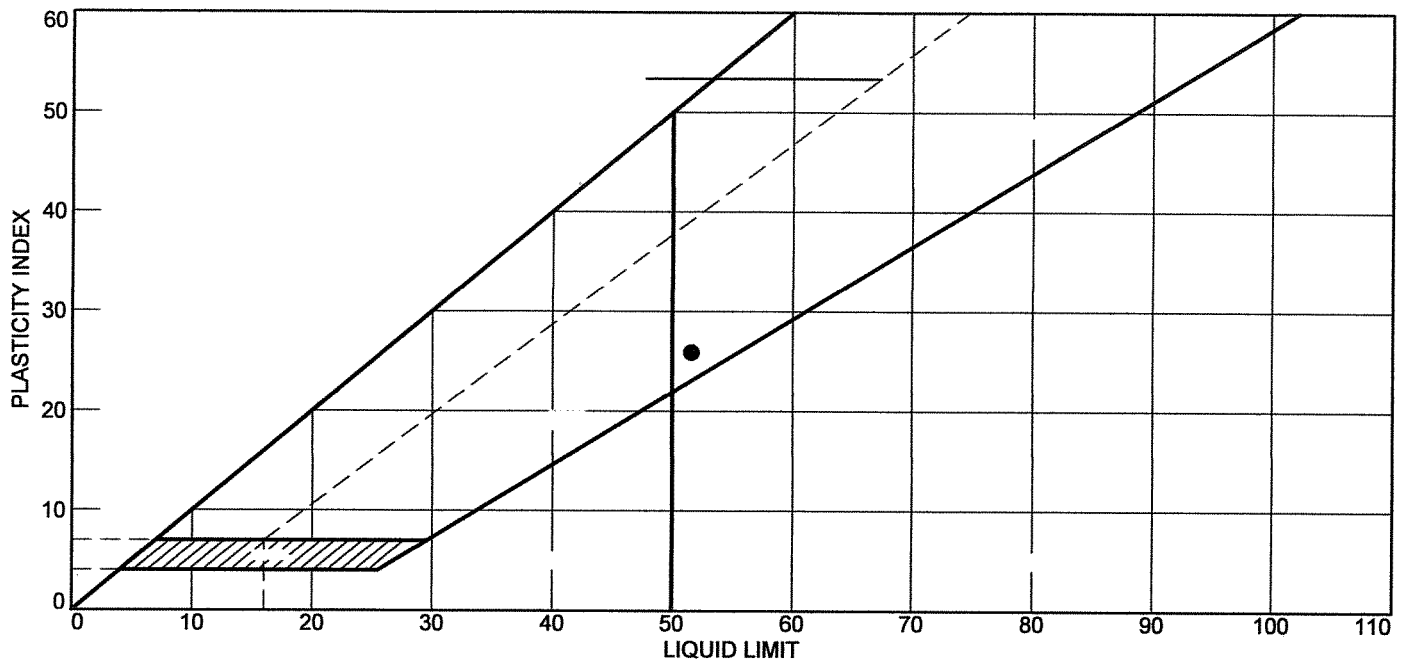


R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 14696

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Fat Clay	51.6	25.7	25.9		98.2	

Project No. 0876-013 Client: GZA GeoEnvironmental, Inc.

Project: Laboratory Testing

Location: BB-LR-101: Falmouth, ME

Sample Number: BB-LR-101 Depth: 13'-15'

R.W. Gillespie & Associates, Inc.

Saco, Maine

Remarks:

● Moisture Content: 41%

Lab No. 14690

Tested By: JRF

Checked By: MTG

MTG

Laboratory Vane Shear Test Results

ASTM D4648 Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil

Project: Laboratory Testing

Location: Falmouth, ME

Client: GZA GeoEnvironmental, Inc.

Date: 8/31/2017

Project No.: 0876-013

Test Depth: 13' to 15'

Boring/Sample No.					Lab No.	14690	
Test No.	Test Depth (ft)	Vane Size	Max. Torque (Undisturbed) (kg-cm)	Max. Torque (Remolded) (kg-cm)	Undrained Shear Strength (psf)	Undrained Shear Strength (psf)	Moisture Content
1	13.5	M	88	9	1838	188	41%

Vane Size	
	(mm)
S	16 x 32
M	20 x 40
L	24.5 x 50.8

Tested By: JRF

Checked By: MTG



R.W. Gillespie & Associates

MTG



APPENDIX E – ROCK CORE PHOTOGRAPHS



**MaineDOT Lunt Road
Bridge #5829 over Interstate 295
Falmouth, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LR-102	R1	11.0 - 16.0	60	100%	28	47%	GNEISS/PEGMATITE	1
BB-LR-102	R2	16.0 - 21.0	60	100%	41	68%	GNEISS	2
BB-LR-104	R1	16.2 - 21.2	60	100%	34	57%	GNEISS/PEGMATITE	3
BB-LR-104	R2	21.2 - 26.2	60	100%	50	83%	GNEISS	4



- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.
 3. Transition between core runs within a row are marked by wood separators.



**MaineDOT Lunt Road
Bridge #5829 over Interstate 295
Falmouth, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LR-103	R1	8.0 - 13.0	60	100%	58	97%	GNEISS	1
BB-LR-103	R2	13.0 - 18.0	59	98%	56	93%	GNEISS/PEGMATITE	2
BB-LR-105	R1	46.2 - 51.2	60	100%	50	83%	GNEISS	3
BB-LR-105	R2	51.2 - 56.2	52	87%	37	62%	GNEISS	4



- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.
 3. Transition between core runs within a row are marked by wood separators.



**MaineDOT Lunt Road
Bridge #5829 over Interstate 295
Falmouth, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LR-101	R1	20.0 - 25.0	60	100%		52%	Masonry/Concrete	1
BB-LR-101	R2	25.0 - 30.0	60	100%		97%	GNEISS	1,2



APPENDIX F – CALCULATIONS



AASHTO Seismic Site Class Summary

Project: Lund Road Bridge	Project No.: 09.0025950.00
Location: Falmouth, ME	
Calculated By: NVW	Date: 9/22/2017
Checked By: CLS	Date: 9/22/2017

Objective: Determine seismic site class in accordance with 2011 AASHTO Guide Specification for LRFD Seismic Bridge Design, 2nd Edition.

Subsurface Information: SPT borings performed by New England Boring Contractors of Hermon, ME for GZA in August 2017 (BB-LR-101 through -105).

- Approach:**
- 1) Assess bridge geometry for location of ground motion determination. For short bridges with few spans, ground motion should be applied at base of abutment per C3.4.2.2 of AASHTO Seismic Guide Specification. Consider pier motions for longer bridges, either at ground surface for pier supported on spread footings, at pile cap for stiff embedded cap, or near point of fixity for deep foundation with no/flexible cap.
 - 2) Assess if site conditions are known in sufficient detail to establish site class in accordance with AASHTO section 3.5.1. If not, an assumption of Site Class D is permitted unless conditions for Site Class E or F are established.
 - 3) Check for the three categories of Site Class F requiring site-specific evaluation:
 - Peats or highly organic clays greater than 10 feet in thickness
 - Thick layers (greater than 25 feet) of highly plastic clay ($PI > 75$)
 - Very thick soft/medium stiff clays (greater than 125 feet)
 - 4) Check for existence of greater than 10 feet of soft clay (where $s_u < 500$ psf, $w > 40\%$, and $PI > 20$). If these conditions are met, classify as Site Class E.
 - 5) Categorize the site using the $\bar{V}_s$, $\bar{N}$, or $\bar{s}_u$ methods.
 - 6) Determine the appropriate Site Class based on the boring-specific results.

Location of Ground Motions: The bridge is relatively short, approximately 200 feet long, with four spans. Therefore, consider ground motion at the base of the abutments, corresponding to borings BB-LR-101 and BB-LR-105. Ignore material in upper 5' for Site Class evaluation.

Site Class Results per Boring:

Exploration	Location	$\bar{N}$	$\bar{S}_u$ (psf)	Site Class	Notes
BB-LR-101	West Abut	--	> 1000	D	Based on $\bar{S}_u > 1000$ psf
BB-LR-105	East Abut	31	--	D	Based on $\bar{N}$, $15 < \bar{N} < 50$

Site Class: Based on results of the N-value and S_u site class evaluations, seismic site class is Site Class D. **Accordingly, we recommend Site Class D for the design.**

Attachments: Refer to the attached tables for further information.

SPT-Based Seismic Site Class Evaluation

Lund Road Bridge
Falmouth, ME

Calculated By: NVW
Checked By: CLS

Date: 9/22/2017
Date: 9/22/2017

INPUT

Exploration ID: **BB-LR-105**

Ground Surface/Mudline Elevation: **39.5**

Depth to Bedrock: **46.2** ft

Hammer ETR: **0.87**

Depth of Boring: **56.2** ft

EQUATIONS

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

where: m = number of layers
SPT borings & d_i = the thickness of any soil or rock layer between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) not to exceed 100 blows/ft corrected to an energy efficiency ratio of 0.6 (N60).
Note: All SPT depths are referenced from the ground surface/mudline.

CALCULATION

$$\bar{N} = 31$$

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	Field SPT N-value	N60	d _i	d _i / N _i
	Top, ft	Bottom, ft					
FILL		5.0	34.5	23	33	--	--
	5.0	7.0	33.5	31	45	3.5	0.078
	10.0	12.0	28.5	14	20	5.0	0.246
	15.0	16.1	24.0	R	#VALUE!	--	--
	20.0	22.0	18.5	23	33	3.7	0.109
CLAY (FILL)	25.0	27.0	13.5	37	54	7.0	0.130
	30.0	32.0	8.5	31	45.0	1.0	0.022
SAND	35.0	37.0	3.5	21	30.5	7.5	0.246
CLAY	40.0	42.0	-1.5	16	23.2	5.0	0.216
GLACIAL TILL	45.0	46.0	-6.0	R	#VALUE!	--	--
ROCK							

Notes:

SPT-Based Seismic Site Class Evaluation

Lund Road Bridge
Falmouth, ME

Calculated By: NVW
Checked By: CLS

Date: 9/22/2017
Date: 9/22/2017

INPUT

Exploration ID:	BB-LR-101	Ground Surface/Mudline Elevation:	45.3
Depth to Bedrock:	19.5 ft	Hammer ETR:	0.677
Depth of Boring:	30.1 ft		

EQUATIONS

$$\bar{S}_u = \frac{d_c}{\sum_{i=1}^k \frac{d_i}{S_{ui}}}$$

where: k = number of layers
SPT borings d_c = Total thickness of cohesive soil layers in the top 100 feet
 S_{ui} = the undrained shear strength in psf, not to exceed 5000 psf
 d_i = the thickness of any soil or rock layer between 0 and 100 feet.
Note: All SPT depths are referenced from the ground surface/mudline.

CALCULATION

$$\bar{S}_u = 1950$$

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	N60	Lab Su or Su (Interated N60 value in psf)	d_i	d_i / S_{ui}
	Top, ft	Bottom, ft					
Fill	1.0	4.0	42.8	18	--	--	--
Clay	5.0	7.0	39.3	17	2200	4.5	0.002
	13.0	15.0	31.3	--	1840	8.5	0.005
Rock							

Notes:

Laboratory testing conducted at 13 - 15' showing Su = 1850 psf, and interpolated N-values within the Clay stratum indicate undrained shear strength values greater than 1000 psf.



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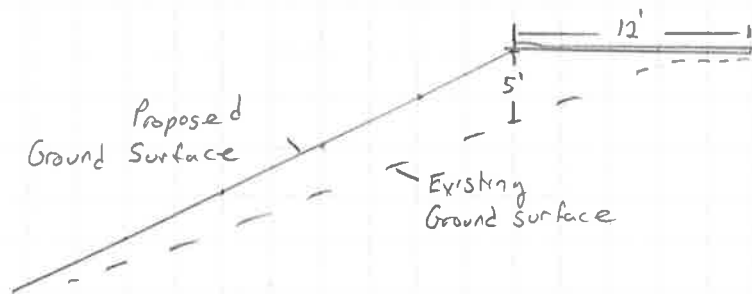
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 Scientists

Updated 05/31/18 - NVW
 CK 06/14/18 - CLS ✓

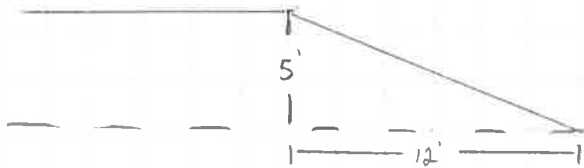
JOB 09.0025950.00 Lunt Road
 SHEET NO 1 OF 3
 Calculated By N. Williams Date 9/22
 Checked By C. Snow Date 10/5/17
 Scale

Settlement - Fill

Based on typical cross sections and x-section at 15+50 and 12+75 the maximum fill height occurs at the east abutment (15+23.06) (see figure)



Using Boussinesq stress under long strip w/ terrace loading, Determine stress increase at the depth of the clay layers.



Conservatively apply factor of 0.621 to the change in pressure for both clay layers at Sta 15+23.06

From Cassagrande Construction from Consolidation testing of the clay:

RR = 0.015 Maximum Past Pressure, $P_c' = 7.5$ ksf

CR = 0.17

Assuming fill with total unit weight of 125 lb/ft³

$$\Delta P = 5'(125 \text{ pcf}) = 625 \text{ psf}$$

Use Delta P of 6', Delta P = 750 psf

$$\begin{aligned} \therefore \text{For each clay layer } \Delta P &= 0.621(625 \text{ psf}) \\ &= 388 \text{ psf} \\ &= 0.39 \text{ ksf} \end{aligned}$$

For each clay layer,
 Delta P = .47 ksf



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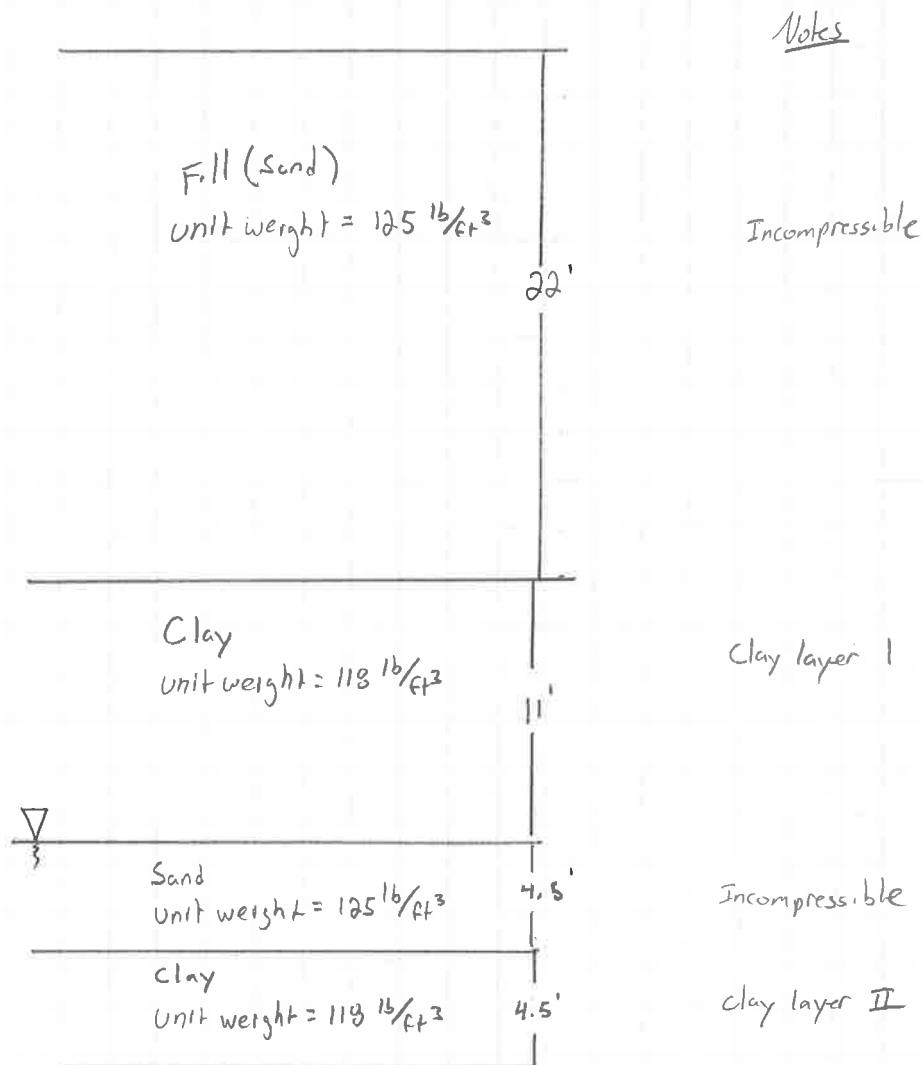
SHEET NO 2 OF 3

Calculated By _____ Date _____

Checked By _____ Date _____

Scale _____

Soil Profile for Settlement calculation ~ Sta. 15+23





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SHEET NO 3 OF 3

Calculated By _____ Date _____

Checked By _____ Date _____

Scale _____

Total settlement

Assuming no settlement in cohesionless layer

$$S_{(n)} = RR(H) \log \left[\frac{P_0 + \Delta P}{P_0} \right], \quad P_0 + \Delta P < P_c' \quad \therefore \text{No Virgin Compression}$$

Layer 1

H = 11 feet

Depth to midpoint = 27.5 feet

$$P_0 = 125 \frac{\text{lb}}{\text{ft}^2} (22') + 118 \frac{\text{lb}}{\text{ft}^2} (5.5')$$

Fill Clay

$$= 2750 + 649$$

$$= 3,399 \text{ psf}$$

$$= 3.4 \text{ ksf}$$

$$S_1 = 0.015 (11) \log \left[\frac{3.4 + 0.39}{3.4} \right]$$

Use 0.47 ksf instead
of 0.39 ksf

$$= 0.008'$$

$$= 0.09''$$

$$S = 0.0093'$$

$$S = 0.11''$$

Layer 2

H = 4.5 feet

Depth to midpoint = 35.0 feet

$$P_0 = 125 \frac{\text{lb}}{\text{ft}^2} (22') + 118 \frac{\text{lb}}{\text{ft}^2} (11') + (125 \frac{\text{lb}}{\text{ft}^2} - 62.4 \frac{\text{lb}}{\text{ft}^2}) 4.5' + (118 \frac{\text{lb}}{\text{ft}^2} - 62.4 \frac{\text{lb}}{\text{ft}^2}) 2.25'$$

Fill clay Sand Clay

$$= 2750 + 1298 + 281.7 + 125.1$$

$$= 4455 \text{ psf}$$

$$= 4.5 \text{ ksf}$$

$$S_2 = 0.015 (4.5) \log \left[\frac{4.5 + 0.39}{4.5} \right]$$

Use 0.47 ksf instead
of 0.39 ksf

$$S = 0.0029'$$

$$S = 0.035''$$

$$= 0.0024'$$

$$= 0.03''$$

$$S_c = 0.11'' + 0.035''$$

$$= 0.15''$$

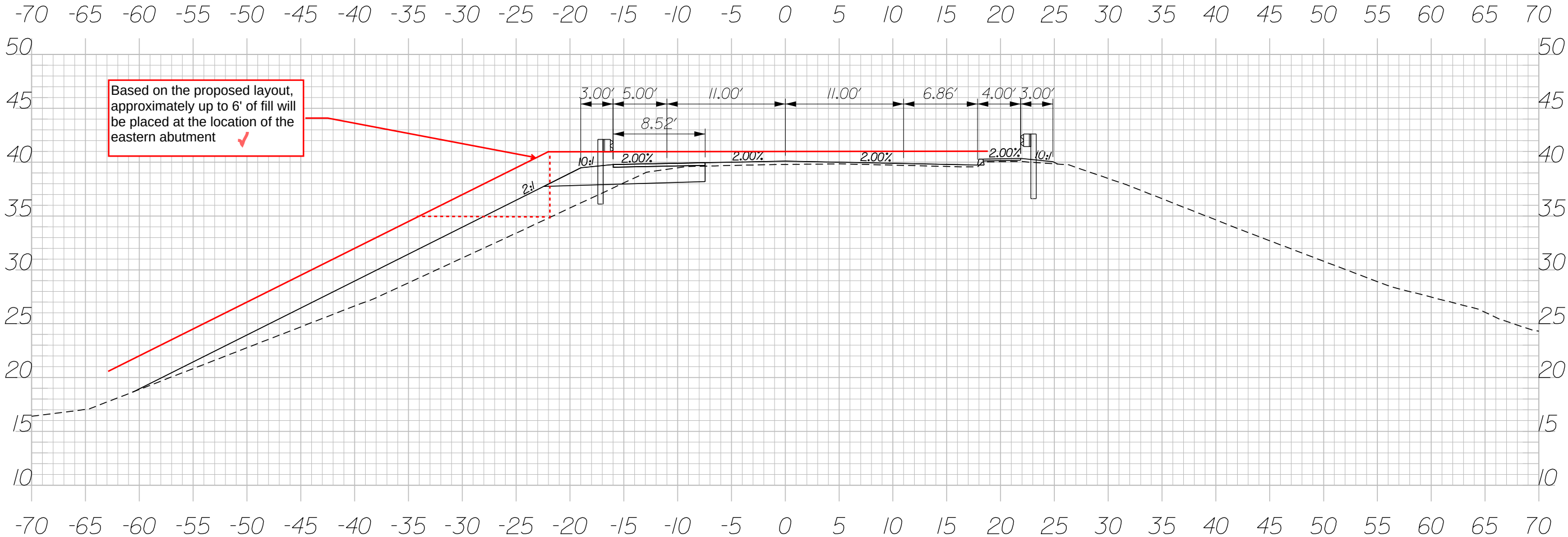
Given the small estimated consolidation settlement, secondary compression is judged to be minor, and estimated settlement is judged to be less than 0.4 inches, downdrag forces will not be considered. ✓

Date:9/13/2017

Username:

Division:

Filename: ... \CADD\012_XSect_01.dgn



Approx Sta. 15+23

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BRIDGE NO. _____ PIN _____
PIN TR2083.10
HIERARCHY PLANS

PROJ. MANAGER	BY	DATE	SIGNATURE
DESIGN-DETAILED	DESIGNER	DATE	
CHECKED-REVIEWED	DESIGNER	DATE	
DESIGN-DETAILED	DESIGNER	DATE	
DESIGN-DETAILED	DESIGNER	DATE	
REVISIONS 1	DESIGNER	DATE	
REVISIONS 2	DESIGNER	DATE	
REVISIONS 3	DESIGNER	DATE	
REVISIONS 4	DESIGNER	DATE	
FIELD CHANGES	DESIGNER	DATE	

LUNT BRIDGE
INTERSTATE 295
FALMOUTH
CUMBERLAND
CROSS SECTIONS 3 OF 7

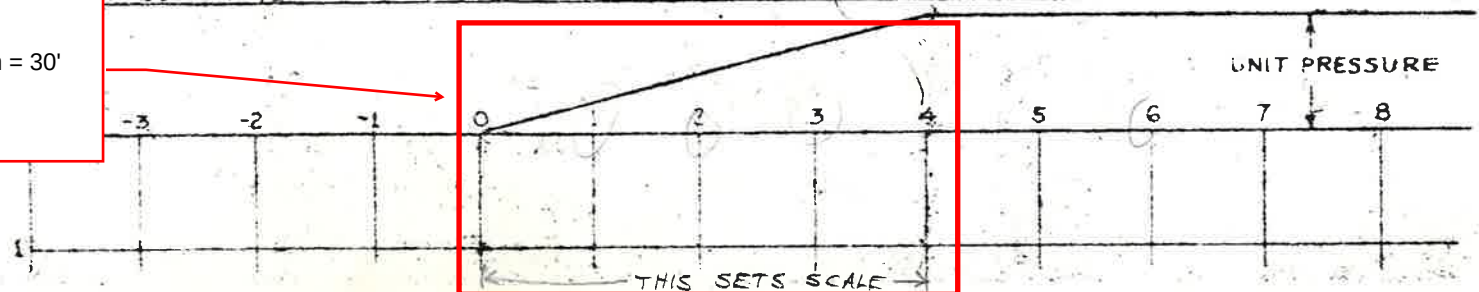
SHEET NUMBER
13
OF XX

Table - Stresses under long Strip with terrace loading.

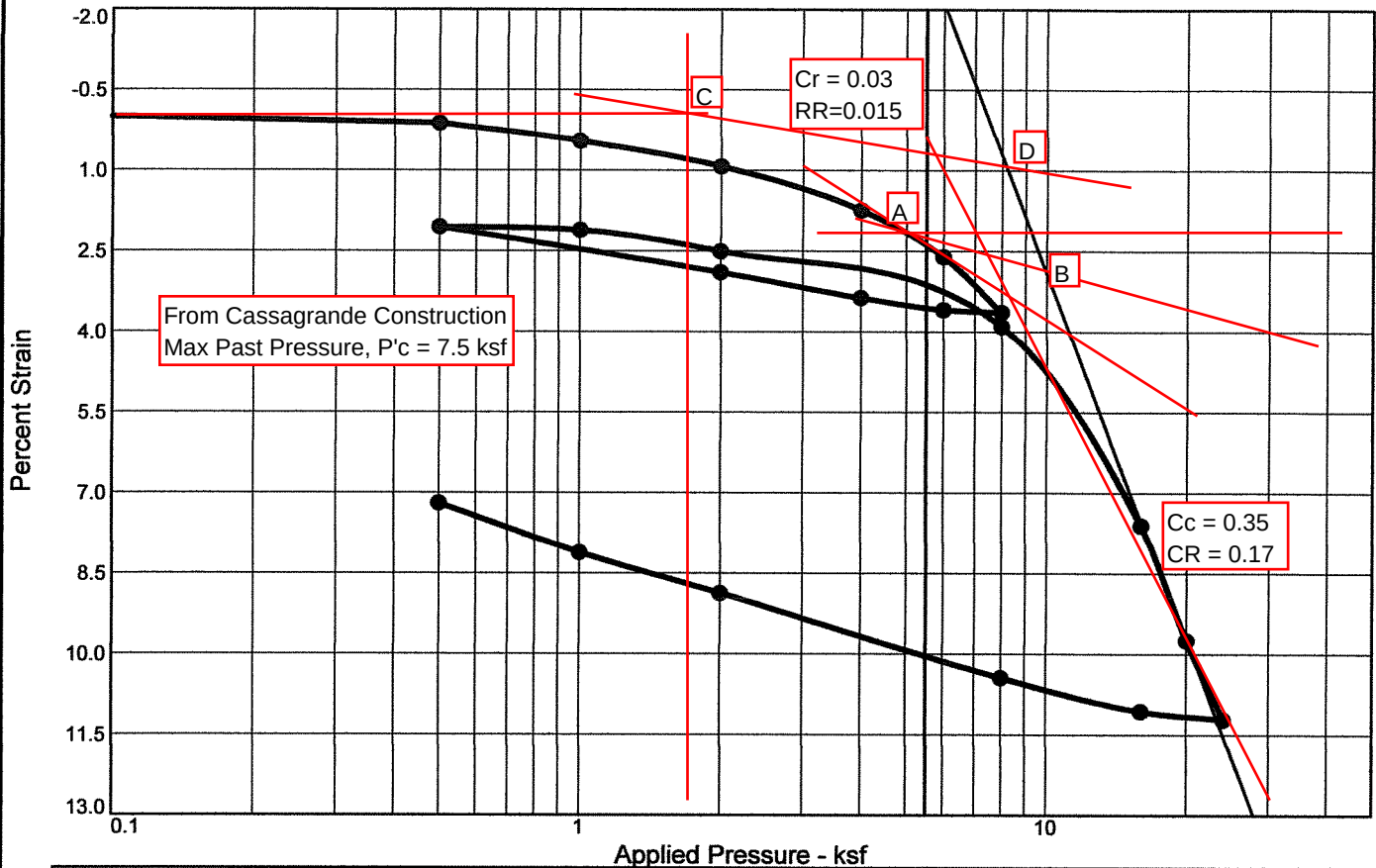
Stress	Coordinates	-4	-3	-2	-1	0	1	2	3	4	5	6	7	8
Vertical	0	0.000	0.000	0.000	0.000	0.000	0.250	0.500	0.750	1.000	1.000	1.000	1.000	1.000
	1	.001	.003	.005	.016	.078	.262	.500	.737	.922	.984	.998	.998	.999
	2	.008	.015	.029	.063	.147	.302	.500	.698	.852	.937	.971	.986	.992
	3	.024	.038	.065	.116	.205	.338	.500	.662	.795	.884	.935	.962	.977
	4	.045	.068	.104	.163	.250	.366	.500	.634	.750	.837	.896	.932	.955
	5	.071	.100	.143	.203	.285	.387	.500	.613	.715	.797	.858	.900	.930
	6	.097	.130	.175	.237	.313	.402	.500	.598	.687	.763	.824	.870	.903
	7	.123	.159	.206	.265	.335	.415	.500	.585	.665	.736	.794	.841	.877
	8	.148	.186	.231	.288	.352	.424	.500	.576	.647	.712	.768	.815	.852
	9	.171	.208	.253	.307	.367	.432	.500	.568	.633	.693	.746	.791	.829
	10	.192	.229	.274	.323	.379	.439	.500	.562	.621	.676	.727	.771	.809
Horizontal	0	0.000	0.000	0.000	0.000	0.000	0.250	0.500	0.750	1.000	1.000	1.000	1.000	1.000
	1	.108	.131	.164	.220	.304	.392	.500	.608	.697	.780	.836	.870	.892
	2	.203	.239	.285	.343	.404	.454	.500	.546	.596	.657	.715	.762	.797
	3	.279	.318	.361	.408	.449	.478	.500	.521	.551	.592	.638	.683	.721
	4	.337	.372	.409	.443	.470	.489	.500	.511	.529	.556	.591	.628	.663
	5	.379	.409	.438	.463	.481	.493	.500	.506	.518	.537	.562	.591	.622
	6	.410	.434	.457	.475	.489	.496	.500	.504	.512	.525	.543	.566	.590
	7	.431	.451	.469	.483	.492	.497	.500	.503	.508	.518	.530	.549	.569
	8	.447	.463	.477	.487	.495	.497	.500	.503	.506	.513	.523	.537	.553
	9	.458	.472	.483	.491	.496	.498	.500	.502	.504	.509	.517	.528	.542
	10	.467	.478	.486	.493	.497	.499	.500	.502	.503	.507	.514	.521	.532
Shear	0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	1	.010	.014	.024	.047	.106	.162	.177	.162	.105	.047	.024	.014	.010
	2	.035	.049	.074	.116	.177	.230	.250	.231	.176	.116	.074	.049	.035
	3	.068	.090	.124	.169	.222	.264	.283	.265	.221	.169	.124	.091	.068
	4	.103	.130	.165	.207	.250	.283	.295	.283	.250	.208	.166	.130	.102
	5	.134	.164	.197	.234	.268	.294	.303	.294	.268	.234	.198	.163	.134
	6	.162	.191	.221	.253	.280	.301	.308	.301	.280	.253	.222	.191	.162
	7	.185	.212	.240	.267	.289	.305	.311	.305	.289	.267	.240	.212	.185
	8	.205	.229	.254	.277	.296	.307	.312	.308	.295	.277	.254	.229	.205
	9	.221	.243	.264	.284	.300	.310	.314	.310	.299	.284	.265	.243	.221
	10	.255	.273	.290	.303	.311	.314	.314	.311	.302	.289	.273	.254	.235

Depth of 30'

4 Blocks = 12'
1 Block = 3'
Therefore Max Depth = 30'



CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C _v (ft.2/day)	C _α	No.	Load (ksf)	C _v (ft.2/day)	C _α	No.	Load (ksf)	C _v (ft.2/day)	C _α
1	0.50	2.157		8	4.00	5.601		15	20.00	0.252	
2	1.00	1.914		9	2.00	2.223		16	24.00	0.164	
3	2.00	2.302		10	0.50	0.591		17	16.00	7.007	
4	4.00	2.135	0.002	11	1.00	0.484	0.000	18	8.00	2.715	
5	6.00	0.895	0.004	12	2.00	0.536	0.000	19	2.00	0.427	
6	8.00	0.102	0.005	13	8.00	1.947	0.002	20	1.00	0.213	
7	6.00	6.156		14	16.00	0.567	0.014	21	0.50	0.183	

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P _c (ksf)	C _c	C _r	Swell Press. (ksf)	Swell %	e _o
Sat.	Moist.											
98.0 %	39.8 %	81.5	51.6	25.9	2.75							1.117

MATERIAL DESCRIPTION

Fat Clay

USCS

AASHTO

Project No. 0876-013 **Client:** GZA GeoEnvironmental, Inc.

Project: Laboratory Testing

Loc.: BB-LR-101: Falmouth, ME **Depth:** 13'-15' **Sample No.:** BB-LR-101

R.W. Gillespie & Associates, Inc.

Saco, Maine

Remarks:

Lab No. 14690

Tested By: JRF

Checked By: MTG

MTG



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Engineers and
Scientists

JOB: 09.0025950.00 Lunt Road Bridge

SUBJECT: Bearing Resistance

SHEET: 1 OF 9

CALCULATED BY: N. Williams 09-13-17

CHECKED BY: C.Snow

REVIEWED BY: _____

Objective

Assess nominal and factored bearing resistance of foundations on rock at the location of each Pier location.

Methodology

Use data from test borings and evaluate the nominal bearing resistance as follows:

1. Bedrock Properties From Test Borings
2. Calculation Of Rock Mass Rating
3. Determine Rock Property Constants s and m
4. Calculate Nominal Bearing Resistance of Bedrock q_n

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012. (AASHTO LRFD).

Note: AASHTO 7th Edition is now in effect, but the coefficients used in the bedrock bearing evaluations are understood to be correlated relative to the older Hoek and Brown 1988 methodology. Therefore, RMR is used for the evaluation per LRFD 6th Edition rather than GSI per LRFD 7th Edition.

2. Wyllie, Duncan C., "Foundations on Rock", Second edition, 1992.

1. Rock Properties

Bedrock properties were obtained from rock core specimens and logs completed for the Lunt Road Bridge Reconstruction Project. The following table presents the data for the Pier test boring where spread footings on bedrock will be utilized (B102, B103, and B104) as well as the data collected at the abutments (B101 and B105). The following bearing resistance is based on the data collected at B102, B103, and B104.

<u>Run</u>	<u>Depth</u>	<u>RQD(%)</u>	<u>Rock Type</u>	<u>Joint Spacing Desc.</u>	<u>Corr. Spacing (in)</u>	<u>Aperture Desc</u>	<u>Corr. Aperture (in)</u>
B102R1	11.0-16.0	47	Gneiss	Very Close to Moderate	0.75-24	Open	0.02-0.1
B102R2	16.0-21.0	68	Gneiss	Very Close to Moderate	0.75-24	Open	0.02-0.1
B103R1	8.0-13.0	97	Gneiss	Close to Moderate	2.5-24	Open	0.02-0.1
B103R2	13.0-18	93	Gneiss	Close to Moderate	2.5-24	Open	0.02-0.1
B104R1	16.2-21.2	57	Gneiss	Very Close to Close	0.75-8	Open	0.02-0.1
B104R2	21.2-26.2	83	Gneiss	Very Close to Moderate	0.75-24	Open	0.02-0.1

Average RQD of rock = 74



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Engineers and
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JOB: 09.0025950.00 Lunt Road Bridge

SUBJECT: Bearing Resistance

SHEET: 2 OF 9

CALCULATED BY: N. Williams 09-13-17

CHECKED BY: C.Snow

REVIEWED BY: _____

2. Calculation of Rock Mass Rating (RMR)

From AASHTO LRFD Table 10.4.6.4-1, determine the RMR.

Parameter 1- Uniaxial Compressive Strength

Uniaxial compressive strength tests were performed on three core specimens, one at each of the pier locations.

Boring	Run	Depth	Rock Type	qp (ksi)
102	R1	11.7-12.3	Gneiss	5.59
103	R1	8.1-8.5	Gneiss	6.53
104	R1	19.3-19.7	Gneiss	3.91

Select design unconfined compressive strength of 4,000 psi.

Representative unconfined compressive strength of intact rock.

$$\sigma_{u,r} := 4\text{ksi}$$

$$\sigma_{u,r} = 576\cdot\text{ksf}$$

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 4$ for $\sigma_{u,r}=520$ to 1080 ksf

Parameter 2- Drill Core Quality

Average RQD from table above (B102, B103, and B104): 74%; choose 50-75 %

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 13$ for RQD = 50% to 75%

Parameter 3- Spacing of Joints

From Boring Logs, generally very close to moderate = 0.75 in to 24 in

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_3 := 10$ For spacing of 2in-1ft

Parameter 4- Condition of Joints

From boring logs, rough, aperture generally .02 inches to 0.1 inches and hard joint walls.



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JOB: 09.0025950.00 Lunt Road Bridge

SUBJECT: Bearing Resistance

SHEET: 3 OF 9

CALCULATED BY: N. Williams 09-13-17

CHECKED BY: C.Snow

REVIEWED BY: _____

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 20$ for slightly rough surfaces, separation <0.05 in, hard joint wall rock

Parameter 5- Ground Water Conditions

Groundwater Conditions

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 7$ for interstitial water. joint water pressure = 0.0 to 0.2 total vertical stress

Adjustment for joint orientation (Parameter 6)

The joint sets are generally moderately dipping and generally rough and open. Considering no rock joints will daylight below bearing level, joint orientation is considered fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$ for foundations - fair conditions

Total RMR Rating

$$RMR := RR_1 + RR_2 + RR_3 + RR_4 + RR_5 + RR_6$$

$$RMR = 47$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 41-60 is indicative of Fair Rock Quality (Class No. 3)

3. Determine Rock Property Constants s and m

From AASHTO LRFD Table 10.4.6.4-4 for Fair Quality Rock Mass

Categorized as rock type E (Gneiss), RMR=45, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 10).

$$m := .57$$

$$s := .0001537$$



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Engineers and
 Scientists

JOB: 09.0025950.00 Lunt Road Bridge

SUBJECT: Bearing Resistance

SHEET: 4 OF 9

CALCULATED BY: N. Williams 09-13-17

CHECKED BY: C.Snow

REVIEWED BY: _____

4. Calculate Nominal and Factored Bearing Resistance of Bedrock q_n and q_R

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

$$q_n := C_{f1} \cdot \sqrt{s} \cdot \sigma_{u,r} \cdot \left[1 + \sqrt{m \cdot \left(\frac{1}{s} \right) + 1} \right]$$

Where

$$C_{f1} := 1.12$$

From Wyllie Table 5.4 Pg. 138 Correction factor for foundation shape for rectangular foundation:

$$s = 0.000154$$

For $L/B > 5$, use factor $C_{f1} = 1.05$,

$$m = 0.57$$

For $L/B = 2$, use factor $C_{f1} = 1.12$,

Estimate footing is roughly $L/B = 2$, use 1.12.

$$\sigma_{u,r} = 4 \text{ ksi}$$

Nominal Bearing Resistance

$$q_n := C_{f1} \cdot \sqrt{s} \cdot \sigma_{u,r} \cdot \left[1 + \sqrt{m \cdot \left(\frac{1}{s} \right) + 1} \right]$$

$$q_n = 62.8 \text{ ksf}$$

Say 62 ksf

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$$\phi_b := 0.45 \quad \text{Footing on rock}$$

$$q_R := \phi_b \cdot q_n$$

$$q_R = 28.3 \text{ ksf}$$

Say 28 ksf



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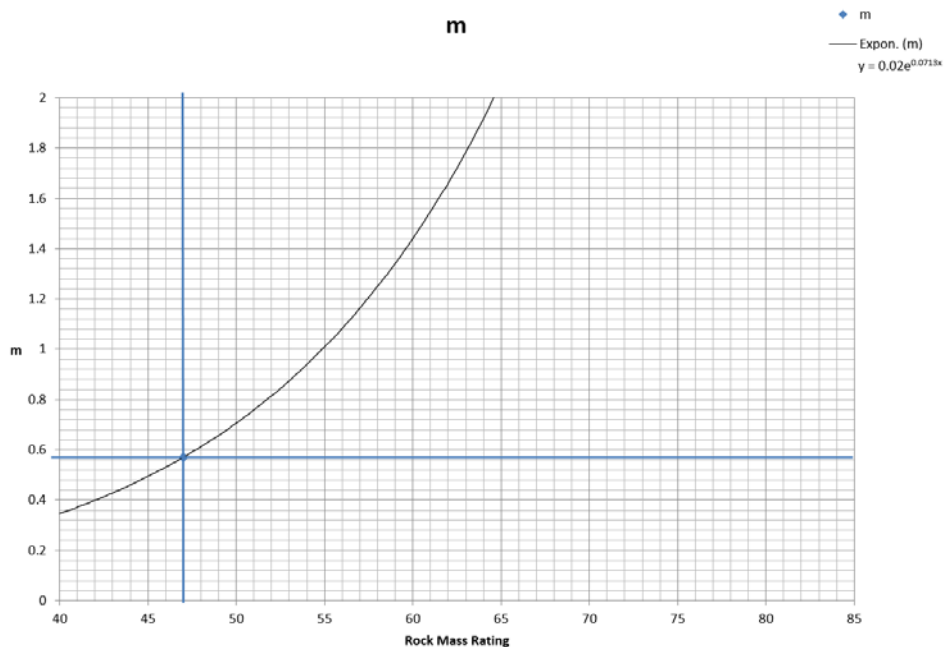
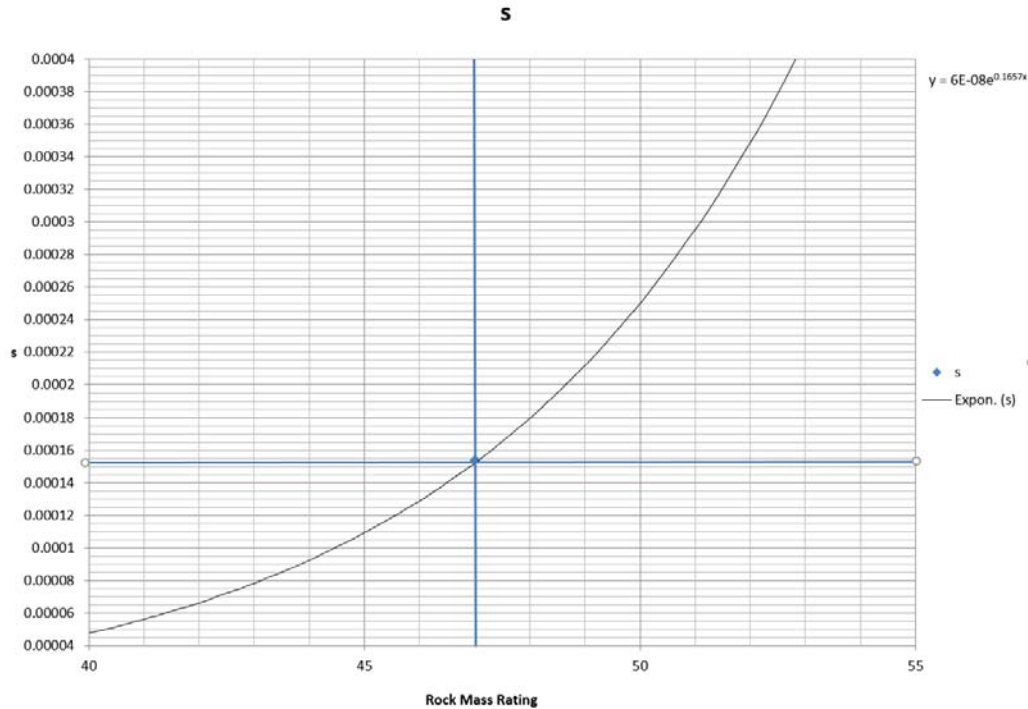
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SHEET: 5 OF 9

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 [Reference:M:\FILES\GEOTECH\Design Calculations\Units v7.xmcd](#)



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AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

Parameter			Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
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		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• 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.	400–2000 gal./hr.	>2000 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			



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Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 10.4.6.4-3 Geomechanics Rock Mass Classes Determined From Total Ratings.

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



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AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-4 Approximate relationship between rock-mass quality and material constants used in defining nonlinear strength (Hoek and Brown, 1988)

Rock Quality	Constants	Rock Type				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities CSIR rating: <i>RMR</i> = 100	<i>m</i>	7.00	10.00	15.00	17.00	25.00
	<i>s</i>	1.00	1.00	1.00	1.00	1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft. CSIR rating: <i>RMR</i> = 85	<i>m</i>	2.40	3.43	5.14	5.82	8.567
	<i>s</i>	0.082	0.082	0.082	0.082	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	<i>m</i>	0.575	0.821	1.231	1.395	2.052
	<i>s</i>	0.00293	0.00293	0.00293	0.00293	0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft. CSIR rating: <i>RMR</i> = 44	<i>m</i>	0.128	0.183	0.275	0.311	0.458
	<i>s</i>	0.00009	0.00009	0.00009	0.00009	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	<i>m</i>	0.029	0.041	0.061	0.069	0.102
	<i>s</i>	3×10^{-6}	3×10^{-6}	3×10^{-6}	3×10^{-6}	3×10^{-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	<i>m</i>	0.007	0.010	0.015	0.017	0.025
	<i>s</i>	1×10^{-7}	1×10^{-7}	1×10^{-7}	1×10^{-7}	1×10^{-7}



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JOB: 09.0025950.00 Lunt Road Bridge

SUBJECT: Bearing Resistance

SHEET: 1 OF 2

CALCULATED BY: N. Williams 06-12-18

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Objective

Assess bearing resistance at the location of the west abutment.

Methodology

Use data from test borings and evaluate the nominal bearing resistance as follows:

1. Bedrock Properties From Test Borings
2. Calculate end bearing capacity of the bedrock q_p

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017. (AASHTO LRFD).

Parameter 1- Uniaxial Compressive Strength

Uniaxial compressive strength tests were performed on three core specimens, one at each of the pier locations.

Boring	Run	Depth	Rock Type	q_u (ksi)
102	R1	11.7-12.3	Gneiss	5.59
103	R1	8.1-8.5	Gneiss	6.53
104	R1	19.3-19.7	Gneiss	3.91

Select design unconfined compressive strength of 4,000 psi.

Representative unconfined compressive strength of intact rock.

$$\sigma_{u,r} := 4 \text{ ksi} \quad \sigma_{u,r} = 576 \text{ ksf}$$

Calculate Bearing Resistance of Bedrock q_p

$$q_p := 2.5 \cdot \sigma_{u,r} \quad \text{AASHTO 10.8.3.5.4c-1}$$

End Bearing Capacity of Bedrock

$$(q_p) = 1440 \text{ ksf}$$

$$\phi_{\text{stat}} := 0.5 \quad \text{Per AASHTO 10.5.5.2.4-1}$$

$$q_r := q_p \cdot \phi_{\text{stat}} = 720 \text{ ksf} \quad \text{Factored unit tip resistance}$$

$$A_p := 1.36 \text{ ft}^2 \quad \text{Area of Pile toe with 14"x14" Bearing Plate}$$

$$R_r := q_r \cdot A_p = 979.2 \text{ kip} \quad \text{Factored end bearing resistance of pile.}$$

$$Q := 466 \text{ kip} \quad \text{Maximum factored Load per pile Provided by WSP}$$

$$\frac{R_r}{Q} = 2.1 \quad R_r/Q \text{ is greater than 1, geotechnical resistance is suitable for a 14" by 14" plate on end of pile.}$$



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 CHECKED BY A. Blaisdell 6/8/2018
C. Snow 6/25/2018

Integral Abutment LRFD Pile Design

Subject: Pile Design for the Lunt Road Bridge Replacement in Falmouth, Maine.

- Reference:**
- AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017
 - Maine BDG Chapter 5 - Substructures 2014
 - VTRANS Integral Abutment Bridge Design Guidelines 2008

Design Steps - Maine BDG

Step 1 - Determine the foundation displacements and the load effects (P_u and M_u) from the superstructure and substructure designs.

$P_u := 466 \cdot \text{kip}$ Maximum Factored Axial Load

1.06" thermal movement given by WSP at each abutment.

Step 2 - If applicable, determine the magnitude of scour.

N/A

Step 3 - Select preliminary pile size.

HP14 x 89, Weak Axis Properties

Steel yield strength	$F_{y50} := 50 \text{ksi}$
Modulus of elasticity for steel	$E := 29000 \text{ksi}$
Cross sectional area of pile	$A_g := 26.1 \text{in}^2$
Radius of gyration	$r_y := 3.53 \text{in}$
Width of Flange	$b_f := 14.7 \cdot \text{in}$
Thickness of Flange	$t_f := .615 \cdot \text{in}$
Elastic Section Modulus	$S_y := 44.3 \cdot \text{in}^3$
Plastic Section Modulus	$Z_y := 67.7 \cdot \text{in}^3$

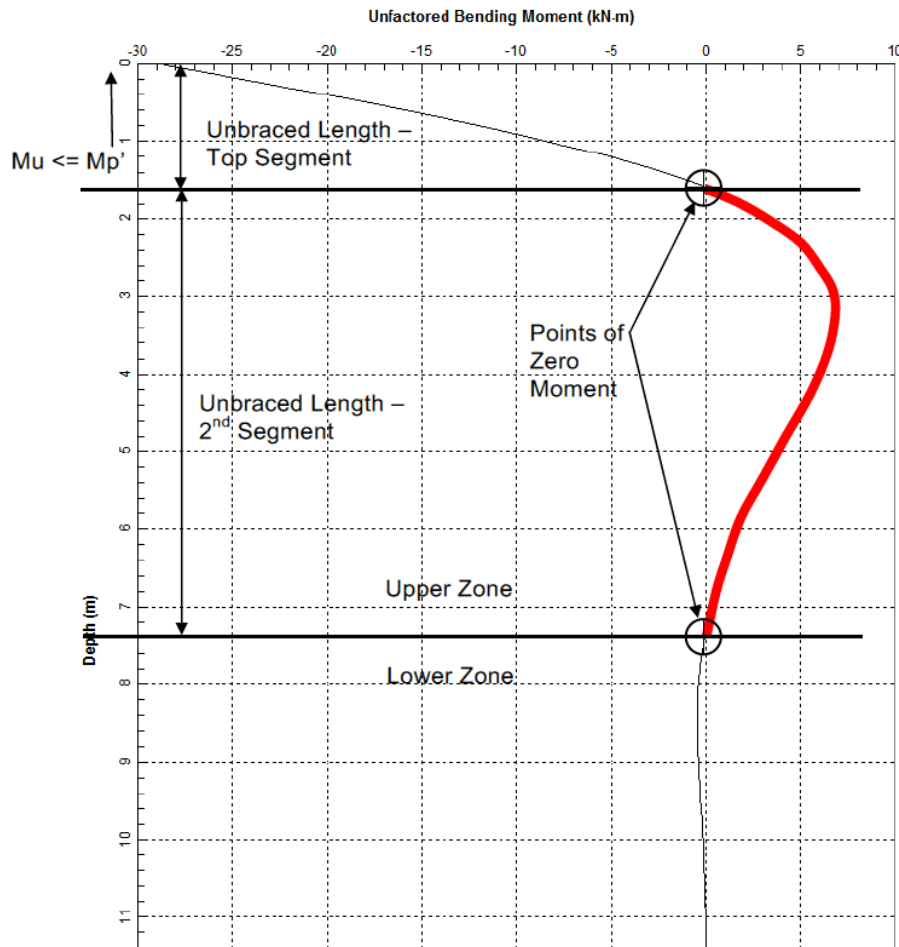
Step 4 - Determine the pile unbraced length and maximum moment at the top of the pile by running LPILE software for top translation = 1.06 inches, $P_u = 466 \text{ kip}$, and Live Load Rotation = 0 (Fixed against rotation)



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Maximum moment from LPile output

$$M_u := 3680 \text{ kip} \cdot \text{in} = 306.67 \cdot \text{kip} \cdot \text{ft}$$

Unbraced lengths from LPile output

Upper segment

$$L_1 := 5.00 \cdot \text{ft}$$

Lower segment

$$L_2 := 10.25 \cdot \text{ft}$$

Step 5 - Determine if the applied moment on the pile will cause pile head plastic deformation considering the interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment.

See above for unbraced lengths (critical lengths L_1 and L_2).

Upper segment slenderness factor

$$\lambda_1 := \frac{(L_1)}{r_y} = 16.997$$



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Lower segment slenderness factor

$$\lambda_2 := \frac{(L_2)}{r_y} = 34.844$$

b. Determine K values for top and bottom of the pile per LRFD Table C4.6.2.5-1

Buckled shape of column is shown by dashed line	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of K when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0
End condition code		Rotation fixed Rotation free Rotation fixed Rotation free		Translation fixed Translation fixed Translation free Translation free		

Upper segment K value (Type d)

$$K_1 := 1.2$$

Lower segment K value (Type c)

$$K_2 := 1$$

c. Calculate the nominal and factored structural pile resistance P_n , per LRFD 6.9.4.1 using the λ values

Elastic critical buckling resistance, P_e , based on flexural buckling

Upper P_e

$$P_{e1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_1 \cdot \lambda_1)^2} = 17956.5 \cdot \text{kip}$$



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Lower P_e

$$P_{e2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_2 \cdot \lambda_2)^2} = 6152.9 \cdot \text{kip}$$

Nominal yield resistance, P_o

$$P_o := F_{y50} \cdot A_g = 1305 \cdot \text{kip}$$

Check that the ratio of P_e to P_o is > 0.44

If $P_e/P_o > 0.44$ then use equation 6.9.4.1.1-1

If $P_e/P_o < 0.44$ then use equation 6.9.4.1.1-2

$$P_n = \left[0.658 \left(\frac{P_o}{P_e} \right) \right] P_o \quad (6.9.4.1.1-1)$$

$$P_n = 0.877 P_e \quad (6.9.4.1.1-2)$$

$$\frac{P_{e1}}{P_o} = 13.76 \quad \frac{P_{e2}}{P_o} = 4.715$$

Use eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{n1} := \left[0.658 \left(\frac{P_o}{P_{e1}} \right) \right] P_o = 1265.9 \cdot \text{kip}$$

$$P_{n2} := \left[0.658 \left(\frac{P_o}{P_{e2}} \right) \right] P_o = 1194.1 \cdot \text{kip}$$

Factored structural Pile Resistance, $P_r = \phi_c (P_n)$

$$\phi_c := .7$$

$$P_{r1} := \phi_c \cdot P_{n1} = 886.13 \cdot \text{kip}$$

$$P_{r2} := \phi_c \cdot P_{n2} = 835.901 \cdot \text{kip}$$

d. Compare the ratio of P_u , the maximum factored axial load, to P_r , the structural resistance in the specified portion of the pile - the pile size should be such that the ratio is not less than 0.20.



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Check for both segments

$$\frac{P_u}{P_{r1}} = 0.526$$

$$\frac{P_u}{P_{r2}} = 0.557$$

e. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)

Check slenderness ratio for flange, limiting slenderness ratio for compact flange, and limiting slenderness ratio for a noncompact flange.

slenderness ratio for flange eq. 6.12.2.2.1-3

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$$

$$\lambda_{pf} := 0.38 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 9.152$$

$$\lambda_{rf} := 0.83 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 19.989$$

If $\lambda_{pf} < \lambda_f < \lambda_{rf}$ Use equation 6.12.2.2.1-2 to find the nominal flexural resistance

$$M_n := \left[1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left[\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \left(\frac{E}{F_{y50}} \right)^{.5}} \right] \right] \cdot F_{y50} \cdot Z_y = 256.9 \cdot \text{ft} \cdot \text{kip}$$

$$\phi_f := 1.0 \quad \text{for flexural resistance according to Maine BDG}$$

$$M_r := M_n \cdot \phi_f = 256.9 \cdot \text{ft} \cdot \text{kip}$$

$$M_u = 306.7 \cdot \text{ft} \cdot \text{kip} \quad M_u \text{ from LPile} > \text{than } M_r, \text{ exceeding the flexural resistance}$$



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$$\frac{P_u}{P_{r1}} + \frac{8}{9} \cdot \frac{M_u}{M_r} = 1.6$$

If less than 1, remains in elastic zone. Since it exceeds 1, pile is expected to yield at the base of the pile cap.

j. Calculate the moment that will cause a plastic hinge at the top of the pile, M_p' ,

Note: M_p' will be lower than M_n due to the inclusion of the axial load in the interaction equation for pile over stresses

$$\frac{P_u}{P_r} + \frac{8.0}{9.0} \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0 \quad \text{LRFD 6.9.2.2 Interaction equation}$$

Use the interaction equation to find the moment that will cause a plastic hinge at the top of the pile. Assume M_{ux} and $M_{rx} = 0$ (out-of-plane), $M_{ry} = M_r$ and $M_u = M_p'$, solve for M_p'

$$M_p' := \left(\frac{9}{8} \right) \cdot \left[1 - \left(\frac{P_u}{P_{r1}} \right) \right] \cdot M_r = 137 \cdot \text{ft} \cdot \text{kip}$$

k. If the calculated moment from LPile Run 1 exceeds the moment that would cause a plastic hinge, a plastic hinge forms, and the moment (M_p') represents the limiting moment reaction at the pile top for subsequent analysis.

Step 6 - For fixed head piles, run a second LPILE analysis with end conditions 1) Top moment = M_p' , top translation = 1.06 in; and axial load equal to P_u . Recalculate unbraced lengths from the moment vs. depth curve.

New unbraced lengths were determined from the second LPile analysis

Upper segment

$$L_{1p} := 2.75 \cdot \text{ft}$$

Lower segment

$$L_{2p} := 11.25 \cdot \text{ft}$$

6a. Repeat steps 5a through 5d above.

6b- If the pile size is such that the ratio of P_u to structural resistance exceeds 0.2, check the upper segment of the



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pile with the interaction equation of LRFD 6.9.2.2. If a plastic hinge forms at the top of the pile, the K value of the upper segment changes from 1.2, for a rotation fixed head condition, to 2.1, for a rotation free head condition. With the new K value and lengths repeat step 5.

5a

Upper segment slenderness factor

$$\lambda_{1p} := \frac{(L_{1p})}{r_y} = 9.348$$

Lower segment slenderness factor

$$\lambda_{2p} := \frac{(L_{2p})}{r_y} = 38.244$$

5b

Upper segment K value (Type e)

$$K_{1p} := 2.1$$

Lower segment K value (Type c)

$$K_{2p} := 1$$

5c

Elastic critical buckling resistance, P_e based on flexural buckling

Upper P_e

$$P_{ep1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_{1p} \cdot \lambda_{1p})^2} = 19383 \cdot \text{kip}$$

Lower P_e

$$P_{ep2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_{2p} \cdot \lambda_{2p})^2} = 5107.6 \cdot \text{kip}$$

Nominal yield resistance, P_o

$$P_o := F_{y50} \cdot A_g = 1305 \cdot \text{kip}$$

Check that the ratio of P_e to P_o is > 0.44

If $P_e/P_o > 0.44$ then use equation 6.9.4.1.1-1

If $P_e/P_o < 0.44$ then use equation 6.9.4.1.1-2

$$\frac{P_{ep1}}{P_o} = 14.853$$

$$\frac{P_{ep2}}{P_o} = 3.914$$



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Use eq. 6.9.4.1.1-2: Nominal structural Pile resistance, P_n for both segments

$$P_{np1} := \left[0.658 \left(\frac{P_o}{P_{ep1}} \right) \right] \cdot P_o = 1268.74 \cdot \text{kip}$$

$$P_{np2} := \left[0.658 \left(\frac{P_o}{P_{ep2}} \right) \right] \cdot P_o = 1172.65 \cdot \text{kip}$$

Factored structural Pile Resistance, $P_r = \phi_c (P_n)$

$$P_{rp1} := \phi_c \cdot P_{np1} = 888.1 \cdot \text{kip}$$

$$P_{rp2} := \phi_c \cdot P_{np2} = 820.9 \cdot \text{kip}$$

d. Compare the ratio of P_u to the structural resistance in the upper portion of the pile - the pile size should be such that the ratio is not less than 0.20.

Check for both segments

$$\frac{P_u}{P_{rp1}} = 0.5247$$

$$\frac{P_u}{P_{rp2}} = 0.568$$

From VTrans Integral Abutment Design Section 4.5.2 - Check the axial capacity of the upper segment and the interaction equation for the second segment to assess suitability of pile section.

Upper Segment

Check that $P_u/P_{rp1} < 1$

$$\frac{P_u}{P_{rp1}} = 0.525 \quad < 1, \text{ OK}$$

Lower Segment

Ultimate moment along the lower segment from LPILE output

$$M_{\max 2} := 104 \cdot \text{ft} \cdot \text{kip}$$

$$\frac{P_u}{P_{rp2}} + \frac{8}{9} \cdot \frac{M_{\max 2}}{M_r} = 0.928 \quad < 1, \text{ OK}$$

Initial LPILE Run

WestAbut 14x89.lp9o

LPILE for Windows, Version 2016-09.010

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

Path to file locations:

\09 Jobs\0025900s\09.0025950.00 - WSP Lunt Rd Bridge\09.0025950.01 - Full Replacement\Calcs\Lpile\West\14x89\Weak\466
kips\

Name of input data file:

WestAbut 14x89.lp9d

Name of output report file:

WestAbut 14x89.lp9o

Name of plot output file:

WestAbut 14x89.lp9p

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WestAbut 14x89.lp9o

Name of runtime message file:

WestAbut 14x89.lp9r

Date and Time of Analysis

Date: June 21, 2018

Time: 9:06:30

Problem Title

Project Name:

Job Number:

Client:

Engineer:

Description:

Program Options and Settings

Computational Options:

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- Use unfactored loads in computations (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
- 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
- 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 = 15.500 ft
 Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.
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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	13.8000
2	15.500	13.8000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

- Cross-sectional Shape = Weak H-Pile
- Length of section = 15.500000 ft
- Flange Width = 14.700000 in
- Section Depth = 13.800000 in
- Flange Thickness = 0.615000 in
- Web Thickness = 0.615000 in
- Section Area = 25.811550 sq. in
- Moment of Inertia = 325.837265 in^4
- Elastic Modulus = 29000000. psi

 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 4 layers

Layer 1 is stiff clay without free water

Distance from top of pile to top of layer = 0.0000 ft
 Distance from top of pile to bottom of layer = 9.500000 ft
 Effective unit weight at top of layer = 118.000000 pcf
 Effective unit weight at bottom of layer = 118.000000 pcf
 Undrained cohesion at top of layer = 2000. psf
 Undrained cohesion at bottom of layer = 2000. psf
 Epsilon-50 at top of layer = 0.006000
 Epsilon-50 at bottom of layer = 0.006000

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

Distance from top of pile to top of layer = 9.500000 ft
 Distance from top of pile to bottom of layer = 10.500000 ft
 Effective unit weight at top of layer = 125.000000 pcf
 Effective unit weight at bottom of layer = 125.000000 pcf
 Friction angle at top of layer = 32.000000 deg.
 Friction angle at bottom of layer = 32.000000 deg.
 Subgrade k at top of layer = 225.000000 pci
 Subgrade k at bottom of layer = 225.000000 pci

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

Distance from top of pile to top of layer = 10.500000 ft
 Distance from top of pile to bottom of layer = 13.500000 ft
 Effective unit weight at top of layer = 125.000000 pcf
 Effective unit weight at bottom of layer = 125.000000 pcf
 Friction angle at top of layer = 34.000000 deg.
 Friction angle at bottom of layer = 34.000000 deg.
 Subgrade k at top of layer = 225.000000 pci
 Subgrade k at bottom of layer = 225.000000 pci

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Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 13.500000 ft
 Distance from top of pile to bottom of layer = 15.500000 ft
 Effective unit weight at top of layer = 152.000000 pcf
 Effective unit weight at bottom of layer = 152.000000 pcf
 Uniaxial compressive strength at top of layer = 4000. psi
 Uniaxial compressive strength at bottom of layer = 4000. psi

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

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

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

The maximum input value, in layer 1, for effective unit weight = 152.00 pcf

This data may be erroneous. Please check your data.

 Summary of Input Soil Properties

Layer	Soil Type	Layer	Effective	Undrained	Angle of	Uniaxial	E50	
Layer	Name	Depth	Unit Wt.	Cohesion	Friction	qu	or	kpy
Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Stiff Clay	0.00	118.0000	2000.	--	--	0.00600	--
	w/o Free Water	9.5000	118.0000	2000.	--	--	0.00600	--

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2	Sand	9.5000	125.0000	--	32.0000	--	225.0000
	(Reese, et al.)	10.5000	125.0000	--	32.0000	--	225.0000
3	Sand	10.5000	125.0000	--	34.0000	--	225.0000
	(Reese, et al.)	13.5000	125.0000	--	34.0000	--	225.0000
4	Strong Rock	13.5000	152.0000	--	--	4000.	--
	(Vuggy Limestone)	15.5000	152.0000	--	--	4000.	--

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
1	5	y = 1.060000 in	S = 0.0000 in/in	466000.	N.A.

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).

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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:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	116798.
2	9.5000	10.5770	No	No	116798.	29879.
3	10.5000	11.5770	Yes	No	146678.	149434.
4	13.5000	13.5000	No	Yes	N.A.	N.A.

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

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

Compared to yield
moment in MathCAD
sheet

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 1.06000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 466000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb*2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.0607	-3680076.	74072.	0.00	95984.	9.45E+09	-432.5544	379.2642	0.00
0.1550	1.0600	-3542766.	73241.	-7.11E-04	93076.	9.45E+09	-443.5193	778.2508	0.00
0.3100	1.0580	-3406387.	72406.	-0.00139	90188.	9.45E+09	-454.3415	798.7257	0.00
0.4650	1.0548	-3270998.	71551.	-0.00205	87321.	9.45E+09	-465.0158	819.9849	0.00
0.6200	1.0504	-3136660.	70676.	-0.00268	84477.	9.45E+09	-475.5369	842.0623	0.00
0.7750	1.0448	-3003431.	69782.	-0.00329	81655.	9.45E+09	-485.8999	864.9945	0.00
0.9300	1.0382	-2871372.	68869.	-0.00387	78859.	9.45E+09	-496.0997	888.8207	0.00
1.0850	1.0305	-2740538.	67937.	-0.00442	76088.	9.45E+09	-506.1315	913.5827	0.00
1.2400	1.0217	-2610988.	66986.	-0.00494	73345.	9.45E+09	-515.9903	939.3253	0.00
1.3950	1.0121	-2482778.	66018.	-0.00545	70630.	9.45E+09	-525.6713	966.0963	0.00
1.5500	1.0015	-2355963.	65031.	-0.00592	67944.	9.45E+09	-535.1698	993.9467	0.00
1.7050	0.9900	-2230597.	64027.	-0.00637	65290.	9.45E+09	-544.4810	1023.	0.00
1.8600	0.9778	-2106735.	63006.	-0.00680	62667.	9.45E+09	-553.6004	1053.	0.00
2.0150	0.9647	-1984428.	61968.	-0.00720	60077.	9.45E+09	-562.5233	1085.	0.00
2.1700	0.9510	-1863729.	60913.	-0.00758	57521.	9.45E+09	-571.2450	1117.	0.00
2.3250	0.9365	-1744688.	59843.	-0.00794	55000.	9.45E+09	-579.7610	1151.	0.00
2.4800	0.9215	-1627355.	58757.	-0.00827	52515.	9.45E+09	-588.0669	1187.	0.00
2.6350	0.9058	-1511779.	57655.	-0.00858	50068.	9.45E+09	-596.1580	1224.	0.00
2.7900	0.8895	-1398008.	56539.	-0.00886	47658.	9.45E+09	-604.0298	1263.	0.00
2.9450	0.8728	-1286087.	55409.	-0.00913	45288.	9.45E+09	-611.6780	1304.	0.00
3.1000	0.8556	-1176064.	54264.	-0.00937	42959.	9.45E+09	-619.0980	1346.	0.00
3.2550	0.8379	-1067981.	53106.	-0.00959	40670.	9.45E+09	-626.2854	1390.	0.00
3.4100	0.8199	-961884.	51935.	-0.00979	38423.	9.45E+09	-633.2356	1437.	0.00
3.5650	0.8015	-857812.	50750.	-0.00997	36219.	9.45E+09	-639.9444	1485.	0.00
3.7200	0.7828	-755809.	49554.	-0.01013	34059.	9.45E+09	-646.4071	1536.	0.00
3.8750	0.7638	-655912.	48346.	-0.01027	31944.	9.45E+09	-652.6194	1589.	0.00

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4.0300	0.7446	-558162.	47127.	-0.01039	29874.	9.45E+09	-658.5768	1645.	0.00
4.1850	0.7252	-462595.	45896.	-0.01049	27850.	9.45E+09	-664.2747	1704.	0.00
4.3400	0.7056	-369247.	44656.	-0.01057	25873.	9.45E+09	-669.7087	1765.	0.00
4.4950	0.6859	-278153.	43405.	-0.01063	23944.	9.45E+09	-674.8743	1830.	0.00
4.6500	0.6661	-189346.	42146.	-0.01068	22064.	9.45E+09	-679.7668	1898.	0.00
4.8050	0.6462	-102859.	40877.	-0.01071	20232.	9.45E+09	-684.3818	1970.	0.00
4.9600	0.6262	-18721.	39600.	-0.01072	18450.	9.45E+09	-688.7145	2046.	0.00
5.1150	0.6063	63036.	38315.	-0.01072	19389.	9.45E+09	-692.7604	2125.	0.00
5.2700	0.5864	142387.	37023.	-0.01070	21069.	9.45E+09	-696.5147	2209.	0.00
5.4250	0.5665	219303.	35724.	-0.01066	22698.	9.45E+09	-699.9727	2298.	0.00
5.5800	0.5467	293760.	34419.	-0.01061	24275.	9.45E+09	-703.1295	2392.	0.00
5.7350	0.5270	365735.	33109.	-0.01054	25799.	9.45E+09	-705.9803	2492.	0.00
5.8900	0.5075	435205.	31794.	-0.01047	27270.	9.45E+09	-708.5202	2597.	0.00
6.0450	0.4881	502149.	30474.	-0.01037	28688.	9.45E+09	-710.7441	2708.	0.00
6.2000	0.4689	566549.	29150.	-0.01027	30051.	9.45E+09	-712.6470	2827.	0.00
6.3550	0.4499	628387.	27823.	-0.01015	31361.	9.45E+09	-714.2236	2953.	0.00
6.5100	0.4311	687646.	26493.	-0.01002	32616.	9.45E+09	-715.4688	3087.	0.00
6.6650	0.4126	744313.	25162.	-0.00988	33816.	9.45E+09	-716.3771	3229.	0.00
6.8200	0.3944	798375.	23829.	-0.00973	34960.	9.45E+09	-716.9432	3381.	0.00
6.9750	0.3764	849820.	22495.	-0.00957	36050.	9.45E+09	-717.1613	3544.	0.00
7.1300	0.3588	898639.	21161.	-0.00939	37084.	9.45E+09	-717.0258	3717.	0.00
7.2850	0.3415	944824.	19828.	-0.00921	38062.	9.45E+09	-716.5308	3903.	0.00
7.4400	0.3245	988369.	18496.	-0.00902	38984.	9.45E+09	-715.6703	4102.	0.00
7.5950	0.3079	1029269.	17166.	-0.00882	39850.	9.45E+09	-714.4383	4316.	0.00
7.7500	0.2917	1067522.	15839.	-0.00862	40660.	9.45E+09	-712.8283	4545.	0.00
7.9050	0.2759	1103127.	14515.	-0.00840	41414.	9.45E+09	-710.8339	4793.	0.00
8.0600	0.2604	1136084.	13195.	-0.00818	42112.	9.45E+09	-708.4484	5060.	0.00
8.2150	0.2454	1166397.	11880.	-0.00796	42754.	9.45E+09	-705.6650	5348.	0.00
8.3700	0.2308	1194069.	10570.	-0.00772	43340.	9.45E+09	-702.4766	5660.	0.00
8.5250	0.2167	1219108.	9267.	-0.00749	43870.	9.45E+09	-698.8759	5999.	0.00
8.6800	0.2030	1241520.	7971.	-0.00724	44345.	9.45E+09	-694.8554	6367.	0.00
8.8350	0.1897	1261317.	6682.	-0.00700	44764.	9.45E+09	-690.4074	6768.	0.00
8.9900	0.1769	1278510.	5403.	-0.00675	45128.	9.45E+09	-685.5240	7206.	0.00
9.1450	0.1646	1293113.	4133.	-0.00650	45437.	9.45E+09	-680.1970	7685.	0.00
9.3000	0.1528	1305143.	2873.	-0.00624	45692.	9.45E+09	-674.4181	8210.	0.00
9.4550	0.1414	1314616.	1624.	-0.00598	45893.	9.45E+09	-668.1787	8788.	0.00
9.6100	0.1305	1321554.	137.3476	-0.00572	46039.	9.45E+09	-930.4777	13259.	0.00
9.7650	0.1201	1325047.	-1575.	-0.00546	46113.	9.45E+09	-911.2295	14109.	0.00
9.9200	0.1102	1325161.	-3251.	-0.00520	46116.	9.45E+09	-890.2999	15025.	0.00
10.0750	0.1008	1321970.	-4886.	-0.00494	46048.	9.45E+09	-867.7359	16014.	0.00
10.2300	0.09184	1315551.	-6477.	-0.00468	45912.	9.45E+09	-843.5906	17085.	0.00

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10.3850	0.08337	1305988.	-8023.	-0.00442	45710.	9.45E+09	-817.9238	18248.	0.00
10.5400	0.07538	1293374.	-9653.	-0.00417	45443.	9.45E+09	-935.0734	23072.	0.00
10.6950	0.06787	1277303.	-11361.	-0.00391	45102.	9.45E+09	-901.4596	24705.	0.00
10.8500	0.06082	1257896.	-13005.	-0.00366	44691.	9.45E+09	-866.2977	26491.	0.00
11.0050	0.05424	1235278.	-14582.	-0.00342	44212.	9.45E+09	-829.6885	28453.	0.00
11.1600	0.04810	1209578.	-16090.	-0.00318	43668.	9.45E+09	-791.7400	30613.	0.00
11.3150	0.04241	1180933.	-17526.	-0.00294	43062.	9.45E+09	-752.5669	33002.	0.00
11.4700	0.03716	1149482.	-18889.	-0.00271	42396.	9.45E+09	-712.2905	35657.	0.00
11.6250	0.03232	1115372.	-20175.	-0.00249	41673.	9.45E+09	-671.0384	38619.	0.00
11.7800	0.02789	1078749.	-21384.	-0.00228	40898.	9.45E+09	-628.9443	41945.	0.00
11.9350	0.02386	1039767.	-22514.	-0.00207	40072.	9.45E+09	-586.1473	45701.	0.00
12.0900	0.02020	998579.	-23564.	-0.00187	39200.	9.45E+09	-542.7916	49974.	0.00
12.2450	0.01691	955343.	-24533.	-0.00167	38284.	9.45E+09	-499.0252	54875.	0.00
12.4000	0.01398	910218.	-25420.	-0.00149	37329.	9.45E+09	-454.9984	60552.	0.00
12.5550	0.01137	863364.	-26202.	-0.00132	36337.	9.45E+09	-385.4822	63051.	0.00
12.7100	0.00908	815028.	-26850.	-0.00115	35313.	9.45E+09	-311.7013	63830.	0.00
12.8650	0.00709	765475.	-27369.	-9.95E-04	34264.	9.45E+09	-246.3698	64608.	0.00
13.0200	0.00538	714939.	-27774.	-8.49E-04	33194.	9.45E+09	-189.2247	65386.	0.00
13.1750	0.00393	663626.	-28080.	-7.13E-04	32107.	9.45E+09	-139.9596	66165.	0.00
13.3300	0.00273	611716.	-28302.	-5.88E-04	31008.	9.45E+09	-98.2271	66943.	0.00
13.4850	0.00175	559362.	-28452.	-4.73E-04	29899.	9.45E+09	-63.6401	67722.	0.00
13.6400	9.71E-04	506692.	-32125.	-3.68E-04	28784.	9.45E+09	-3886.	7440000.	0.00
13.7950	3.80E-04	440494.	-37154.	-2.74E-04	27382.	9.45E+09	-1521.	7440000.	0.00
13.9500	-4.94E-05	368956.	-38385.	-1.95E-04	25867.	9.45E+09	197.4416	7440000.	0.00
14.1050	-3.44E-04	298039.	-36922.	-1.29E-04	24365.	9.45E+09	1376.	7440000.	0.00
14.2600	-5.30E-04	231831.	-33672.	-7.69E-05	22963.	9.45E+09	2118.	7440000.	0.00
14.4150	-6.30E-04	172912.	-29358.	-3.71E-05	21716.	9.45E+09	2521.	7440000.	0.00
14.5700	-6.68E-04	122684.	-24530.	-8.00E-06	20652.	9.45E+09	2670.	7440000.	0.00
14.7250	-6.60E-04	81673.	-19592.	1.21E-05	19783.	9.45E+09	2640.	7440000.	0.00
14.8800	-6.22E-04	49780.	-14821.	2.50E-05	19108.	9.45E+09	2490.	7440000.	0.00
15.0350	-5.67E-04	26494.	-10397.	3.26E-05	18615.	9.45E+09	2267.	7440000.	0.00
15.1900	-5.01E-04	11046.	-6424.	3.62E-05	18288.	9.45E+09	2006.	7440000.	0.00
15.3450	-4.32E-04	2535.	-2952.	3.76E-05	18108.	9.45E+09	1728.	7440000.	0.00
15.5000	-3.62E-04	0.00	0.00	3.78E-05	18054.	9.45E+09	1446.	3720000.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.06067368 inches

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WestAbut 14x89.lp9o
Computed slope at pile head = 0.000000 radians
Maximum bending moment = -3680076. inch-lbs
Maximum shear force = 74072. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 11
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

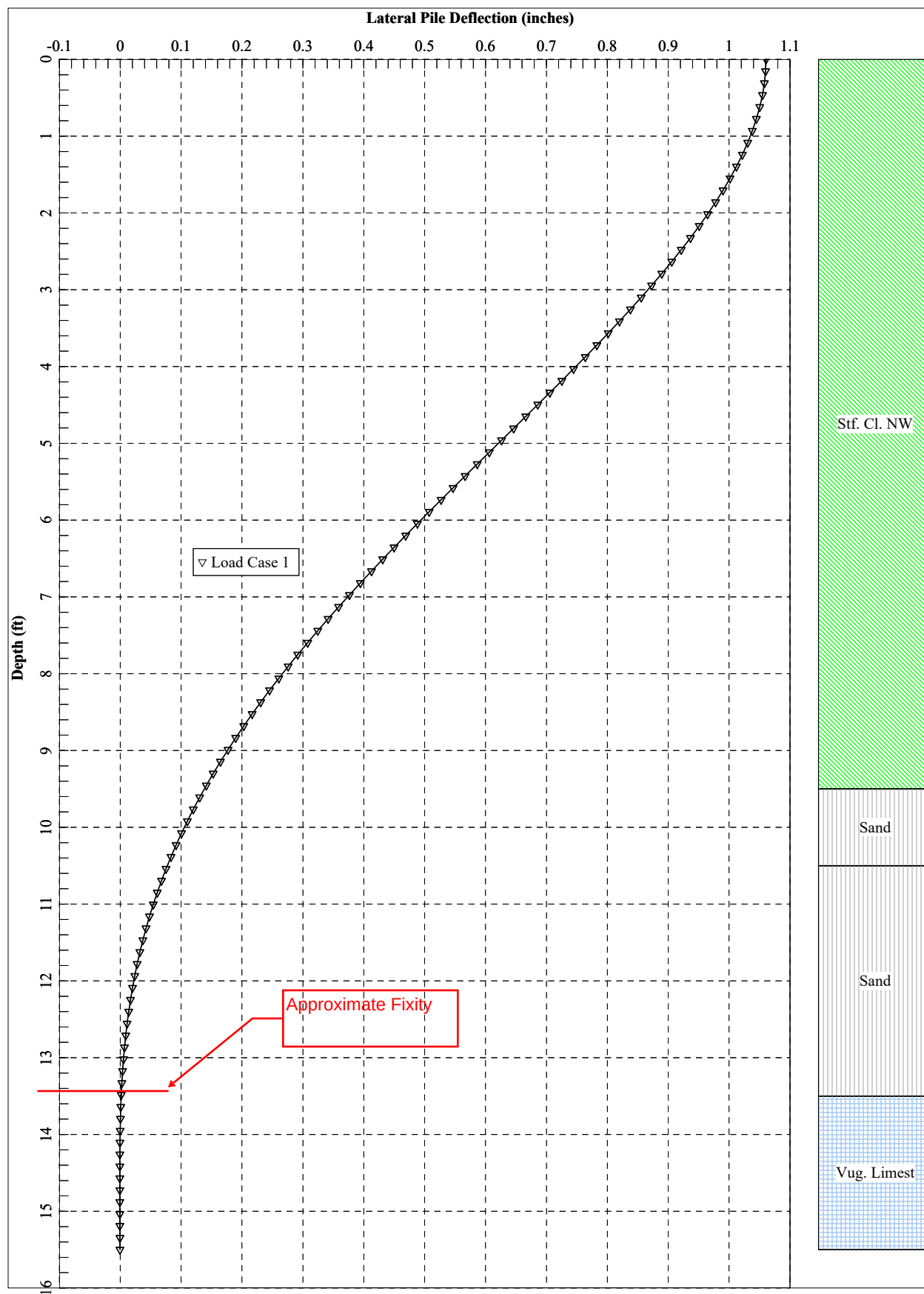
Definitions of Pile-head Loading Conditions:

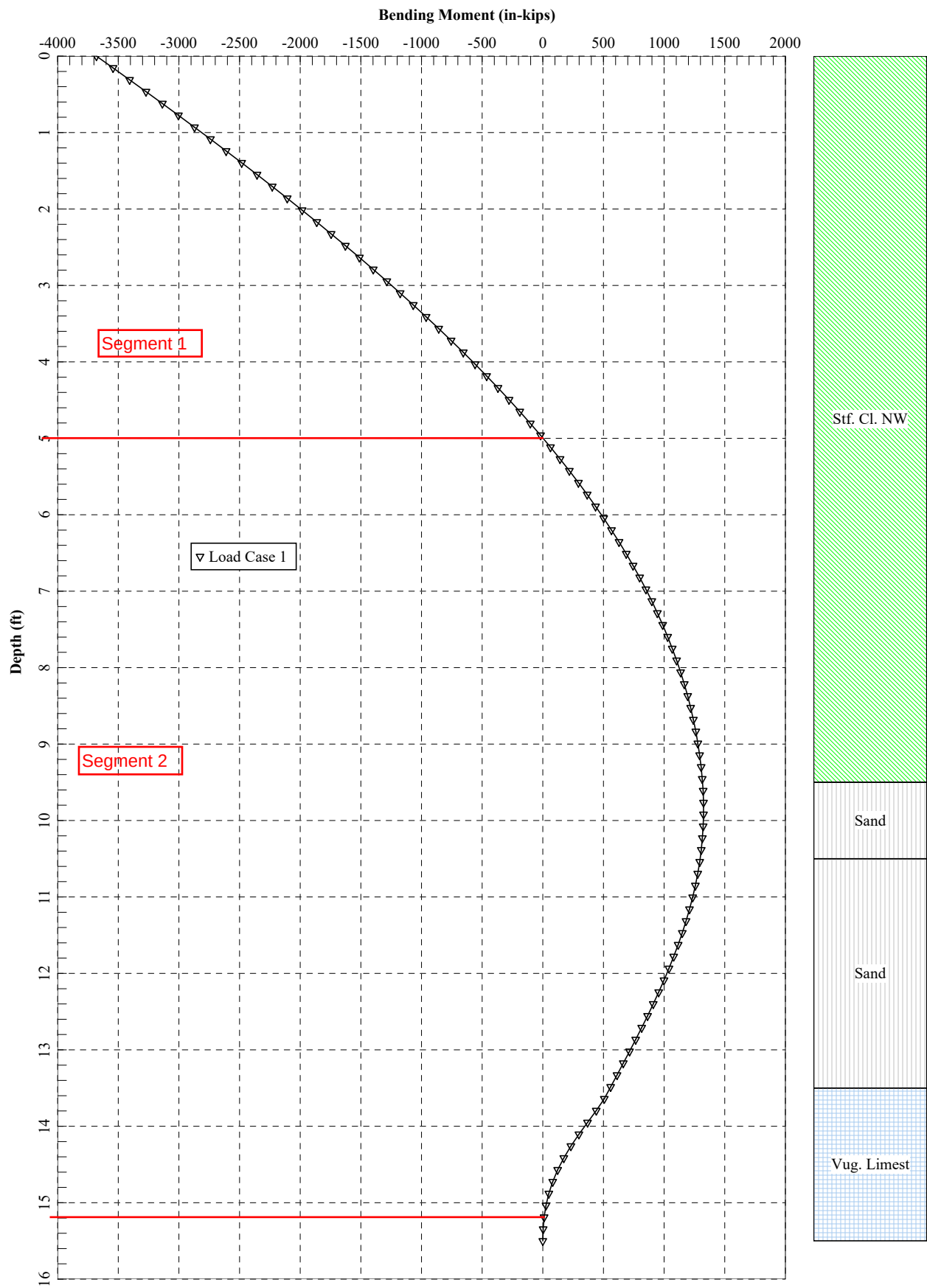
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load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

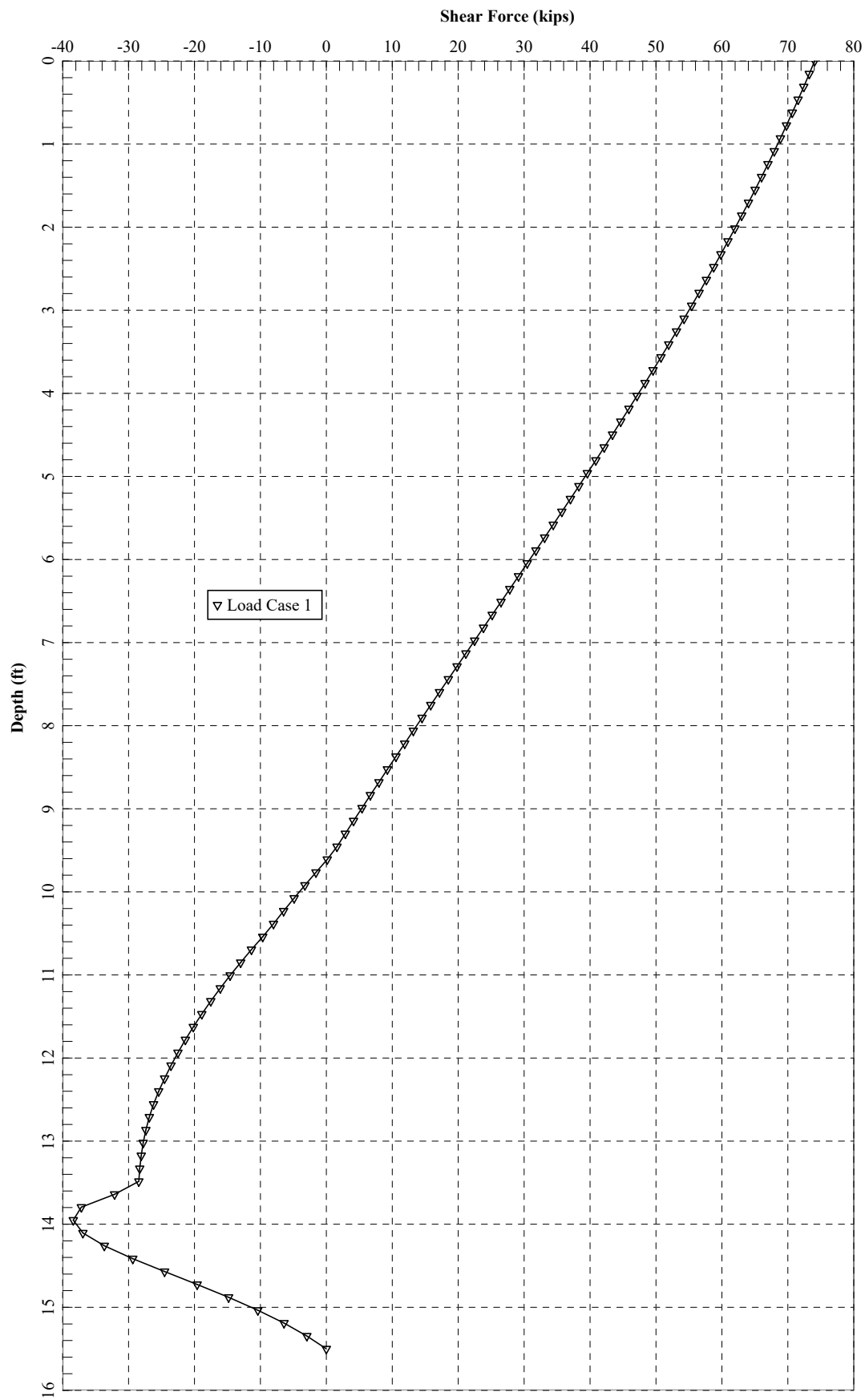
Load Case No.	Load Type	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.0600	S, rad	0.00	466000.	1.0607	0.00	74072.	-3680076.

Maximum pile-head deflection = 1.0606736804 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.







Stf. Cl. NW

Sand

Sand

Vug. Limest

Plastic Hinge Formed

Inputs: Deflection, Moment from 1st analysis, Axial load.

WestAbut 14x89.lp9o

LPILE for Windows, Version 2016-09.010

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

Path to file locations:
\\09 Jobs\0025900s\09.0025950.00 - WSP Lunt Rd Bridge\09.0025950.01 - Full Replacement\Calcs\Lpile\West\14x89\Weak\466
Kips\Plastic Hing Form\

Name of input data file:
WestAbut 14x89.lp9d

Name of output report file:
WestAbut 14x89.lp9o

Name of plot output file:
WestAbut 14x89.lp9p

Page 1

WestAbut 14x89.lp9o

Name of runtime message file:
WestAbut 14x89.lp9r

Date and Time of Analysis

Date: June 21, 2018 Time: 9:12:18

Problem Title

Project Name:

Job Number:

Client:

Engineer:

Description:

Program Options and Settings

Computational Options:

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WestAbut 14x89.1p9o
- Use unfactored loads in computations (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
- 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
- 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 = 15.500 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.
Page 3

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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	13.8000
2	15.500	13.8000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Weak H-Pile
Length of section = 15.500000 ft
Flange Width = 14.700000 in
Section Depth = 13.800000 in
Flange Thickness = 0.615000 in
Web Thickness = 0.615000 in
Section Area = 25.811550 sq. in
Moment of Inertia = 325.837265 in^4
Elastic Modulus = 29000000. psi

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 4 layers

Layer 1 is stiff clay without free water

Distance from top of pile to top of layer = 0.0000 ft
 Distance from top of pile to bottom of layer = 9.500000 ft
 Effective unit weight at top of layer = 118.000000 pcf
 Effective unit weight at bottom of layer = 118.000000 pcf
 Undrained cohesion at top of layer = 2000. psf
 Undrained cohesion at bottom of layer = 2000. psf
 Epsilon-50 at top of layer = 0.006000
 Epsilon-50 at bottom of layer = 0.006000

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

Distance from top of pile to top of layer = 9.500000 ft
 Distance from top of pile to bottom of layer = 10.500000 ft
 Effective unit weight at top of layer = 125.000000 pcf
 Effective unit weight at bottom of layer = 125.000000 pcf
 Friction angle at top of layer = 32.000000 deg.
 Friction angle at bottom of layer = 32.000000 deg.
 Subgrade k at top of layer = 225.000000 pci
 Subgrade k at bottom of layer = 225.000000 pci

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

Distance from top of pile to top of layer = 10.500000 ft
 Distance from top of pile to bottom of layer = 13.500000 ft
 Effective unit weight at top of layer = 125.000000 pcf
 Effective unit weight at bottom of layer = 125.000000 pcf
 Friction angle at top of layer = 34.000000 deg.
 Friction angle at bottom of layer = 34.000000 deg.
 Subgrade k at top of layer = 225.000000 pci
 Subgrade k at bottom of layer = 225.000000 pci

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Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 13.500000 ft
 Distance from top of pile to bottom of layer = 15.500000 ft
 Effective unit weight at top of layer = 152.000000 pcf
 Effective unit weight at bottom of layer = 152.000000 pcf
 Uniaxial compressive strength at top of layer = 4000. psi
 Uniaxial compressive strength at bottom of layer = 4000. psi

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

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

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

The maximum input value, in layer 1, for effective unit weight = 152.00 pcf

This data may be erroneous. Please check your data.

 Summary of Input Soil Properties

Layer	Soil Type	Layer	Effective	Undrained	Angle of	Uniaxial	E50	
Layer	Name	Depth	Unit Wt.	Cohesion	Friction	qu	or	kpy
Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Stiff Clay	0.00	118.0000	2000.	--	--	0.00600	--
	w/o Free Water	9.5000	118.0000	2000.	--	--	0.00600	--

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2	Sand	9.5000	125.0000	--	32.0000	--	225.0000
	(Reese, et al.)	10.5000	125.0000	--	32.0000	--	225.0000
3	Sand	10.5000	125.0000	--	34.0000	--	225.0000
	(Reese, et al.)	13.5000	125.0000	--	34.0000	--	225.0000
4	Strong Rock	13.5000	152.0000	--	--	4000.	--
	(Vuggy Limestone)	15.5000	152.0000	--	--	4000.	--

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
1	4	y = 1.060000 in	M = -1644291. in-lbs	466000.	N.A.

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).

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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:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	116798.
2	9.5000	10.5770	No	No	116798.	29879.
3	10.5000	11.5770	Yes	No	146678.	149434.
4	13.5000	13.5000	No	Yes	N.A.	N.A.

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

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

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

Displacement of pile head = 1.060000 inches

Moment at pile head = -1644291.1 in-lbs

Axial load at pile head = 466000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb*2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.0600	-1644291.	52102.	-0.00849	52874.	9.45E+09	-432.4857	379.4450	0.00
0.1550	1.0439	-1540631.	51289.	-0.00880	50679.	9.45E+09	-441.8265	787.2313	0.00
0.3100	1.0273	-1438237.	50458.	-0.00910	48510.	9.45E+09	-451.0009	816.6073	0.00
0.4650	1.0101	-1337158.	49611.	-0.00937	46370.	9.45E+09	-460.0045	847.0784	0.00
0.6200	0.9924	-1237442.	48747.	-0.00962	44258.	9.45E+09	-468.8326	878.7086	0.00
0.7750	0.9743	-1139137.	47867.	-0.00986	42177.	9.45E+09	-477.4806	911.5659	0.00
0.9300	0.9557	-1042289.	46971.	-0.01007	40126.	9.45E+09	-485.9442	945.7226	0.00
1.0850	0.9368	-946945.	46060.	-0.01027	38107.	9.45E+09	-494.2188	981.2555	0.00
1.2400	0.9175	-853149.	45133.	-0.01044	36120.	9.45E+09	-502.3002	1018.	0.00
1.3950	0.8980	-760946.	44191.	-0.01060	34168.	9.45E+09	-510.1838	1057.	0.00
1.5500	0.8781	-670377.	43235.	-0.01074	32250.	9.45E+09	-517.8655	1097.	0.00
1.7050	0.8580	-581486.	42265.	-0.01087	30368.	9.45E+09	-525.3409	1139.	0.00
1.8600	0.8377	-494313.	41281.	-0.01097	28522.	9.45E+09	-532.6056	1183.	0.00
2.0150	0.8172	-408898.	40284.	-0.01106	26713.	9.45E+09	-539.6556	1228.	0.00
2.1700	0.7965	-325280.	39274.	-0.01113	24942.	9.45E+09	-546.4864	1276.	0.00
2.3250	0.7757	-243498.	38251.	-0.01119	23210.	9.45E+09	-553.0940	1326.	0.00
2.4800	0.7549	-163587.	37216.	-0.01123	21518.	9.45E+09	-559.4741	1379.	0.00
2.6350	0.7340	-85585.	36170.	-0.01125	19866.	9.45E+09	-565.6225	1433.	0.00
2.7900	0.7130	-9524.	35113.	-0.01126	18256.	9.45E+09	-571.5349	1491.	0.00
2.9450	0.6921	64561.	34044.	-0.01126	19421.	9.45E+09	-577.2073	1551.	0.00
3.1000	0.6711	136638.	32966.	-0.01124	20947.	9.45E+09	-582.6354	1615.	0.00
3.2550	0.6503	206676.	31877.	-0.01121	22431.	9.45E+09	-587.8151	1681.	0.00
3.4100	0.6295	274645.	30779.	-0.01116	23870.	9.45E+09	-592.7421	1752.	0.00
3.5650	0.6087	340517.	29672.	-0.01110	25265.	9.45E+09	-597.4122	1825.	0.00
3.7200	0.5882	404264.	28557.	-0.01102	26615.	9.45E+09	-601.8212	1903.	0.00
3.8750	0.5677	465859.	27434.	-0.01094	27919.	9.45E+09	-605.9649	1985.	0.00

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4.0300	0.5475	525279.	26303.	-0.01084	29177.	9.45E+09	-609.8390	2072.	0.00
4.1850	0.5274	582500.	25165.	-0.01073	30389.	9.45E+09	-613.4393	2163.	0.00
4.3400	0.5076	637499.	24021.	-0.01061	31554.	9.45E+09	-616.7614	2260.	0.00
4.4950	0.4879	690255.	22871.	-0.01048	32671.	9.45E+09	-619.8010	2363.	0.00
4.6500	0.4686	740749.	21716.	-0.01034	33740.	9.45E+09	-622.5537	2471.	0.00
4.8050	0.4495	788963.	20556.	-0.01019	34761.	9.45E+09	-625.0152	2586.	0.00
4.9600	0.4307	834880.	19391.	-0.01003	35734.	9.45E+09	-627.1809	2709.	0.00
5.1150	0.4122	878485.	18223.	-0.00986	36657.	9.45E+09	-629.0464	2839.	0.00
5.2700	0.3940	919764.	17051.	-0.00968	37531.	9.45E+09	-630.6071	2977.	0.00
5.4250	0.3761	958704.	15877.	-0.00950	38356.	9.45E+09	-631.8585	3125.	0.00
5.5800	0.3586	995295.	14701.	-0.00931	39131.	9.45E+09	-632.7957	3282.	0.00
5.7350	0.3415	1029527.	13524.	-0.00911	39855.	9.45E+09	-633.4141	3450.	0.00
5.8900	0.3248	1061391.	12345.	-0.00890	40530.	9.45E+09	-633.7088	3629.	0.00
6.0450	0.3084	1090882.	11166.	-0.00869	41155.	9.45E+09	-633.6749	3822.	0.00
6.2000	0.2924	1117995.	9988.	-0.00847	41729.	9.45E+09	-633.3074	4028.	0.00
6.3550	0.2769	1142726.	8811.	-0.00825	42253.	9.45E+09	-632.6011	4250.	0.00
6.5100	0.2617	1165074.	7635.	-0.00802	42726.	9.45E+09	-631.5509	4488.	0.00
6.6650	0.2470	1185038.	6462.	-0.00779	43149.	9.45E+09	-630.1512	4745.	0.00
6.8200	0.2328	1202619.	5291.	-0.00756	43521.	9.45E+09	-628.3967	5022.	0.00
6.9750	0.2189	1217822.	4125.	-0.00732	43843.	9.45E+09	-626.2816	5321.	0.00
7.1300	0.2055	1230650.	2962.	-0.00708	44114.	9.45E+09	-623.8001	5645.	0.00
7.2850	0.1926	1241110.	1804.	-0.00683	44336.	9.45E+09	-620.9462	5997.	0.00
7.4400	0.1801	1249210.	652.4142	-0.00659	44507.	9.45E+09	-617.7135	6379.	0.00
7.5950	0.1681	1254959.	-493.1684	-0.00634	44629.	9.45E+09	-614.0957	6796.	0.00
7.7500	0.1565	1258370.	-1632.	-0.00610	44701.	9.45E+09	-610.0859	7250.	0.00
7.9050	0.1454	1259456.	-2762.	-0.00585	44724.	9.45E+09	-605.6772	7748.	0.00
8.0600	0.1348	1258232.	-3884.	-0.00560	44699.	9.45E+09	-600.8622	8294.	0.00
8.2150	0.1246	1254714.	-4997.	-0.00535	44624.	9.45E+09	-595.6330	8894.	0.00
8.3700	0.1148	1248921.	-6100.	-0.00511	44501.	9.45E+09	-589.9817	9555.	0.00
8.5250	0.1056	1240874.	-7191.	-0.00486	44331.	9.45E+09	-583.8995	10287.	0.00
8.6800	0.09676	1230596.	-8271.	-0.00462	44113.	9.45E+09	-577.3773	11099.	0.00
8.8350	0.08840	1218110.	-9339.	-0.00438	43849.	9.45E+09	-570.4052	12002.	0.00
8.9900	0.08048	1203442.	-10393.	-0.00414	43538.	9.45E+09	-562.9727	13011.	0.00
9.1450	0.07300	1186622.	-11433.	-0.00390	43182.	9.45E+09	-555.0685	14143.	0.00
9.3000	0.06596	1167679.	-12457.	-0.00367	42781.	9.45E+09	-546.6801	15416.	0.00
9.4550	0.05934	1146646.	-13466.	-0.00344	42336.	9.45E+09	-537.7940	16856.	0.00
9.6100	0.05315	1123556.	-14467.	-0.00322	41847.	9.45E+09	-538.8184	18857.	0.00
9.7650	0.04736	1098410.	-15449.	-0.00300	41314.	9.45E+09	-517.4471	20321.	0.00
9.9200	0.04198	1071287.	-16391.	-0.00279	40740.	9.45E+09	-495.0765	21935.	0.00
10.0750	0.03699	1042269.	-17269.	-0.00258	40125.	9.45E+09	-471.7720	23722.	0.00
10.2300	0.03238	1011440.	-18145.	-0.00238	39472.	9.45E+09	-447.6032	25710.	0.00

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10.3850	0.02814	978890.	-18955.	-0.00218	38783.	9.45E+09	-422.6442	27931.	0.00
10.5400	0.02427	944712.	-19784.	-0.00199	38059.	9.45E+09	-469.3947	35981.	0.00
10.6950	0.02073	908748.	-20628.	-0.00181	37298.	9.45E+09	-438.3358	39327.	0.00
10.8500	0.01753	871112.	-21414.	-0.00164	36501.	9.45E+09	-406.6204	43143.	0.00
11.0050	0.01465	831921.	-22141.	-0.00147	35671.	9.45E+09	-374.3465	47533.	0.00
11.1600	0.01207	791293.	-22806.	-0.00131	34811.	9.45E+09	-341.6099	52638.	0.00
11.3150	0.00978	749349.	-23402.	-0.00116	33922.	9.45E+09	-298.8846	56824.	0.00
11.4700	0.00777	706242.	-23904.	-0.00101	33009.	9.45E+09	-240.6269	57602.	0.00
11.6250	0.00602	662183.	-24303.	-8.78E-04	32076.	9.45E+09	-188.7999	58381.	0.00
11.7800	0.00450	617357.	-24612.	-7.52E-04	31127.	9.45E+09	-143.2152	59159.	0.00
11.9350	0.00322	571930.	-24842.	-6.35E-04	30165.	9.45E+09	-103.6484	59938.	0.00
12.0900	0.00214	526048.	-25003.	-5.27E-04	29194.	9.45E+09	-69.8402	60716.	0.00
12.2450	0.00126	479834.	-25106.	-4.28E-04	28215.	9.45E+09	-41.4982	61494.	0.00
12.4000	5.47E-04	433394.	-25162.	-3.38E-04	27232.	9.45E+09	-18.2977	62273.	0.00
12.5550	-3.45E-06	386818.	-25179.	-2.58E-04	26245.	9.45E+09	0.1170	63051.	0.00
12.7100	-4.12E-04	340175.	-25166.	-1.86E-04	25258.	9.45E+09	14.1320	63830.	0.00
12.8650	-6.96E-04	293524.	-25130.	-1.24E-04	24270.	9.45E+09	24.1626	64608.	0.00
13.0200	-8.72E-04	246906.	-25079.	-7.05E-05	23282.	9.45E+09	30.6529	65386.	0.00
13.1750	-9.58E-04	200352.	-25019.	-2.65E-05	22297.	9.45E+09	34.0752	66165.	0.00
13.3300	-9.71E-04	153882.	-24955.	8.37E-06	21313.	9.45E+09	34.9293	66943.	0.00
13.4850	-9.27E-04	107506.	-24891.	3.41E-05	20331.	9.45E+09	33.7426	67722.	0.00
13.6400	-8.44E-04	61229.	-21721.	5.07E-05	19351.	9.45E+09	3375.	7440000.	0.00
13.7950	-7.38E-04	26616.	-15837.	5.94E-05	18618.	9.45E+09	2952.	7440000.	0.00
13.9500	-6.23E-04	2212.	-10774.	6.22E-05	18101.	9.45E+09	2491.	7440000.	0.00
14.1050	-5.07E-04	-13572.	-6572.	6.11E-05	18341.	9.45E+09	2027.	7440000.	0.00
14.2600	-3.96E-04	-22341.	-3215.	5.75E-05	18527.	9.45E+09	1583.	7440000.	0.00
14.4150	-2.93E-04	-25631.	-654.0092	5.28E-05	18597.	9.45E+09	1171.	7440000.	0.00
14.5700	-1.99E-04	-24865.	1176.	4.78E-05	18580.	9.45E+09	796.7385	7440000.	0.00
14.7250	-1.15E-04	-21339.	2344.	4.33E-05	18506.	9.45E+09	458.9661	7440000.	0.00
14.8800	-3.81E-05	-16222.	2912.	3.96E-05	18397.	9.45E+09	152.4449	7440000.	0.00
15.0350	3.26E-05	-10574.	2933.	3.70E-05	18278.	9.45E+09	-130.3197	7440000.	0.00
15.1900	9.94E-05	-5375.	2442.	3.54E-05	18168.	9.45E+09	-397.5988	7440000.	0.00
15.3450	1.64E-04	-1551.	1461.	3.47E-05	18087.	9.45E+09	-657.0057	7440000.	0.00
15.5000	2.29E-04	0.00	0.00	3.46E-05	18054.	9.45E+09	-914.1407	3720000.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.06000000 inches

WestAbut 14x89.1p9o

Computed slope at pile head = -0.00848967 radians

Maximum bending moment = -1644291. inch-lbs

Maximum shear force = 52102. 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 = 14

Number of zero deflection points = 2

Summary of Pile-head Responses for Conventional Analyses

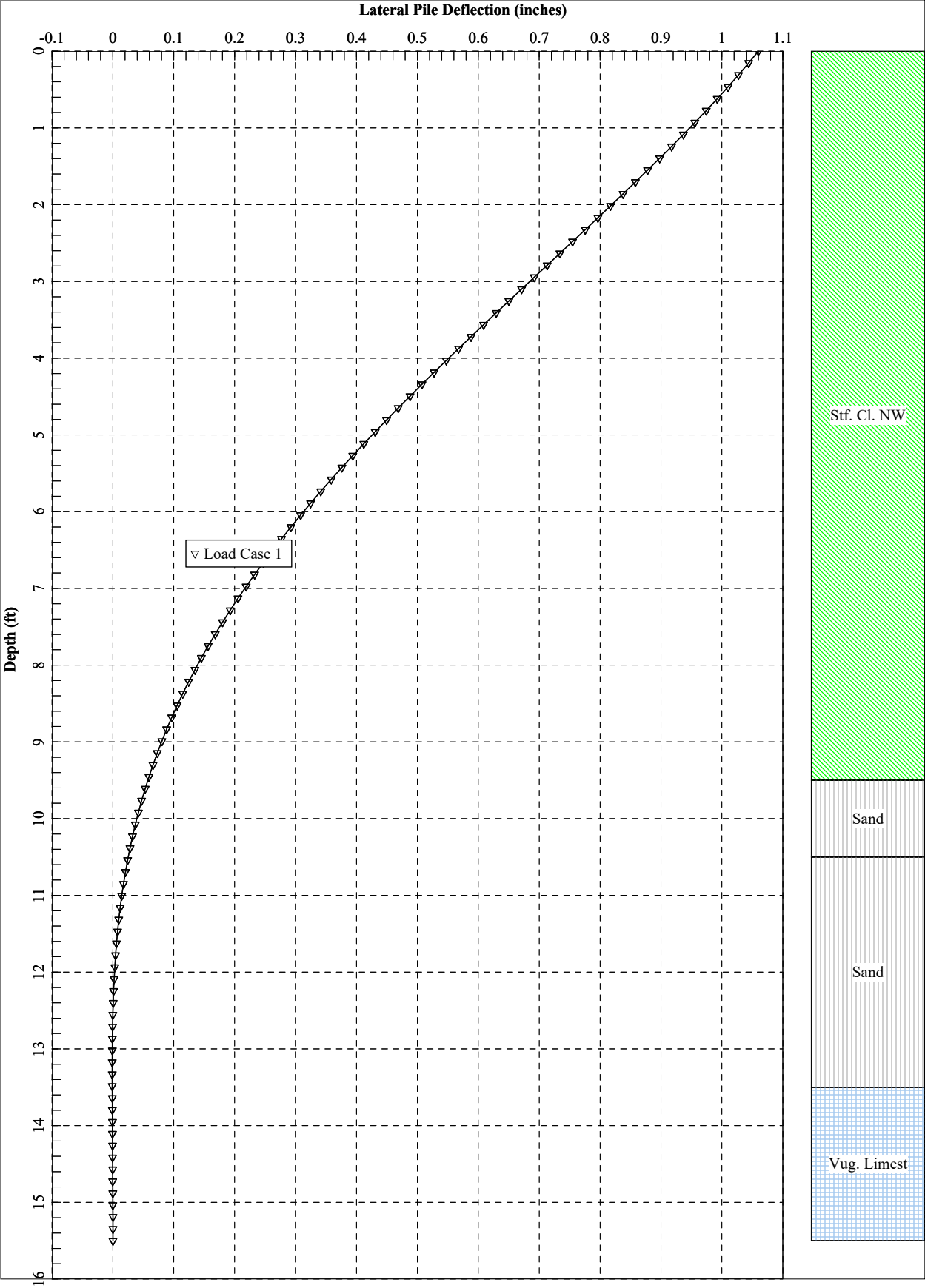
Definitions of Pile-head Loading Conditions:

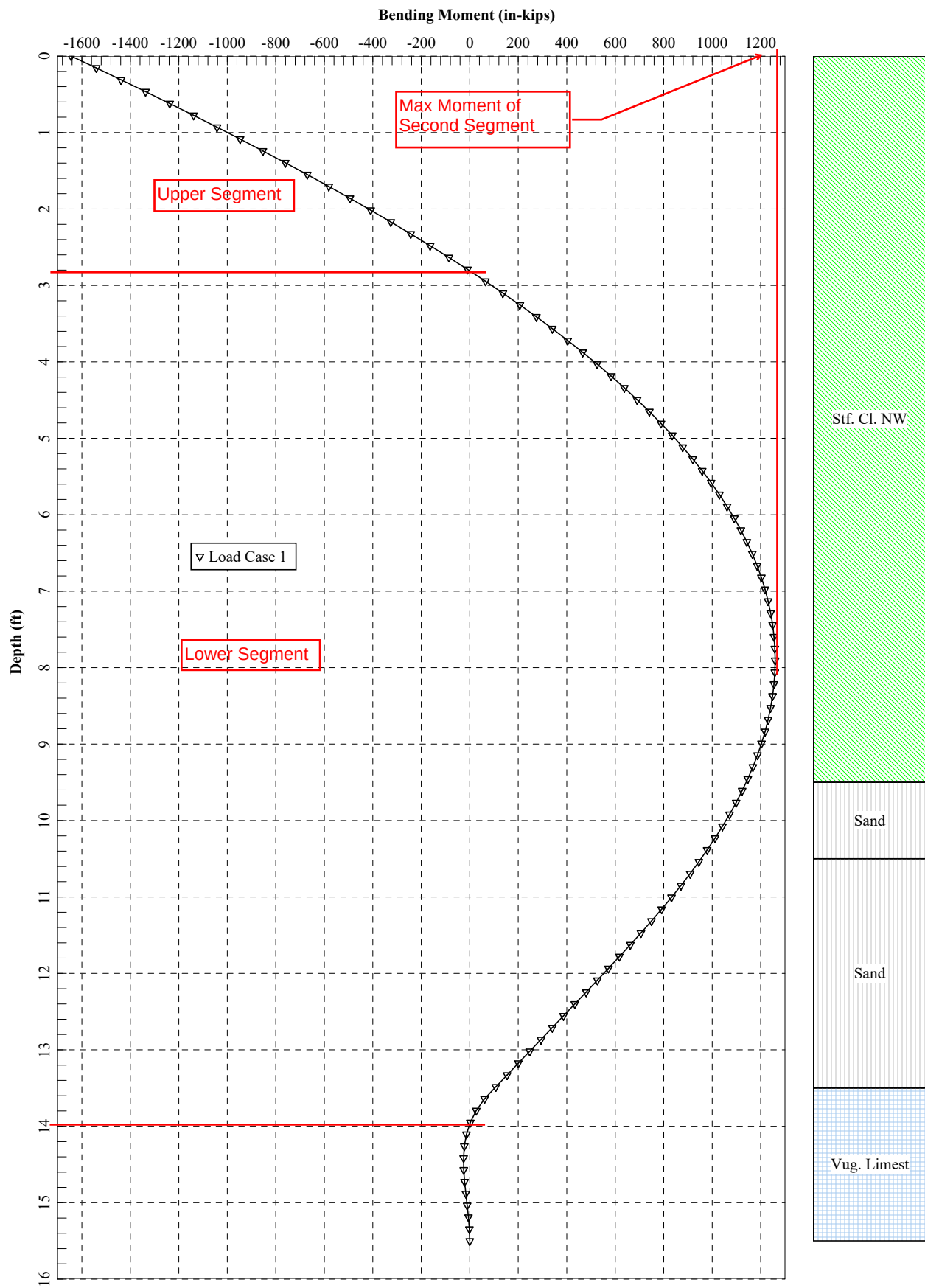
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load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

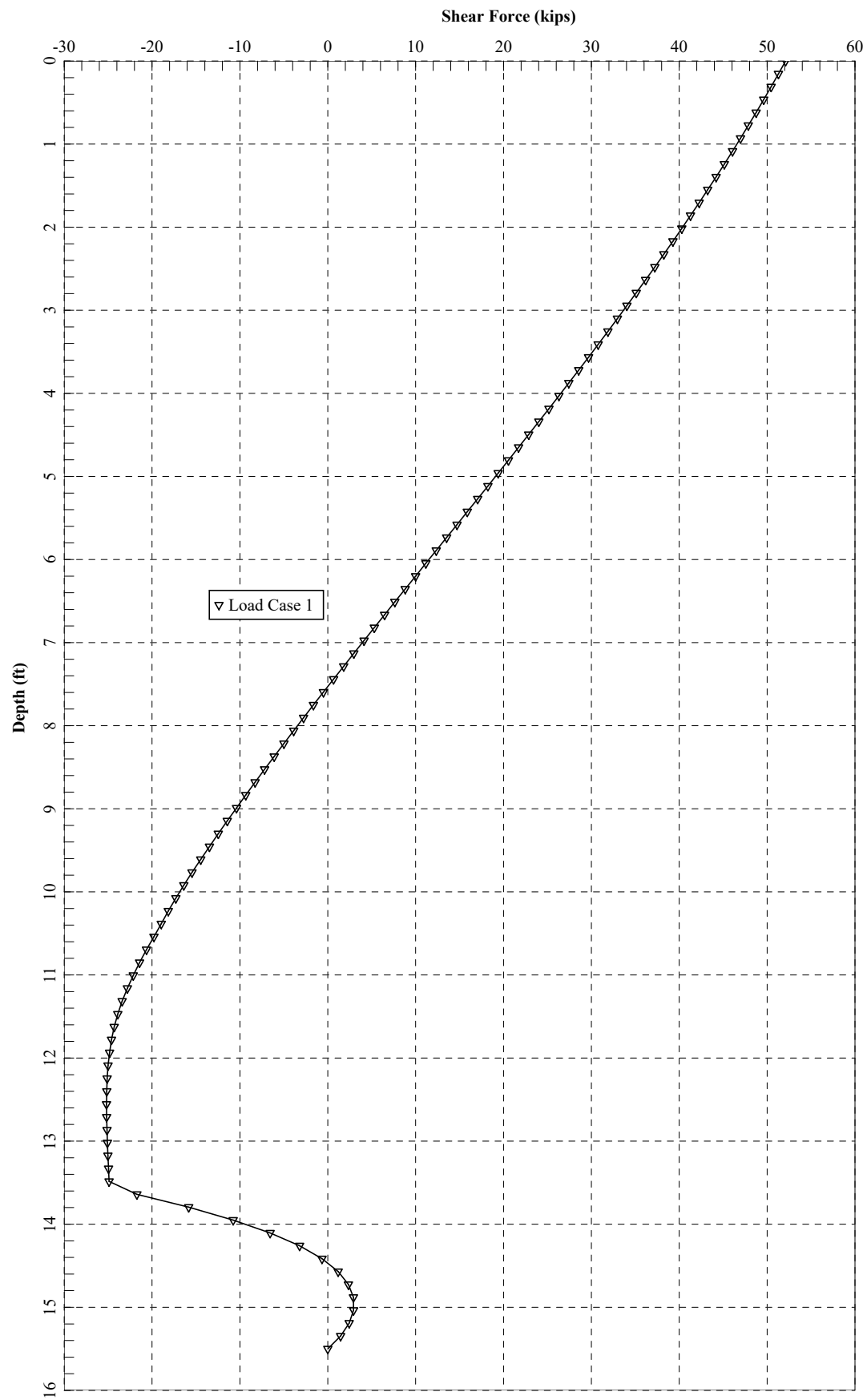
Load Case No.	Load Type	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Load lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.0600	M, in-lb	-1644291.	466000.	1.0600	-0.00849	52102.	-1644291.

Maximum pile-head deflection = 1.060000000 inches
Maximum pile-head rotation = -0.0084896694 radians = -0.486422 deg.

The analysis ended normally.









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JOB: 09.0025950.01
 SUBJECT: Pile Evaluation for Integral Abutment
 SHEET: 1 OF 8
 CALCULATED BY N. Williams 6/8/2018
 CHECKED BY A. Blaisdell 6/8/2018
C. Snow 6/25/2018

Integral Abutment LRFD Pile Design

Subject: Pile Design for the Lunt Road Bridge Replacement in Falmouth, Maine.

- Reference:**
- AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017
 - Maine BDG Chapter 5 - Substructures 2014
 - VTRANS Integral Abutment Bridge Design Guidelines 2008

Design Steps - Maine BDG

Step 1 - Determine the foundation displacements and the load effects (P_u and M_u) from the superstructure and substructure designs.

$P_u := 466 \text{ kip}$ Maximum Factored Axial Load

1.06" thermal movement given by WSP at each abutment.

Step 2 - If applicable, determine the magnitude of scour.

N/A

Step 3 - Select preliminary pile size.

HP14 x 89, Weak Axis Properties

Steel yield strength	$F_{y50} := 50 \text{ ksi}$
Modulus of elasticity for steel	$E := 29000 \text{ ksi}$
Cross sectional area of pile	$A_g := 26.1 \text{ in}^2$
Radius of gyration	$r_y := 3.53 \text{ in}$
Width of Flange	$b_f := 14.7 \text{ in}$
Thickness of Flange	$t_f := .615 \text{ in}$
Elastic Section Modulus	$S_y := 44.3 \text{ in}^3$
Plastic Section Modulus	$Z_y := 67.7 \text{ in}^3$

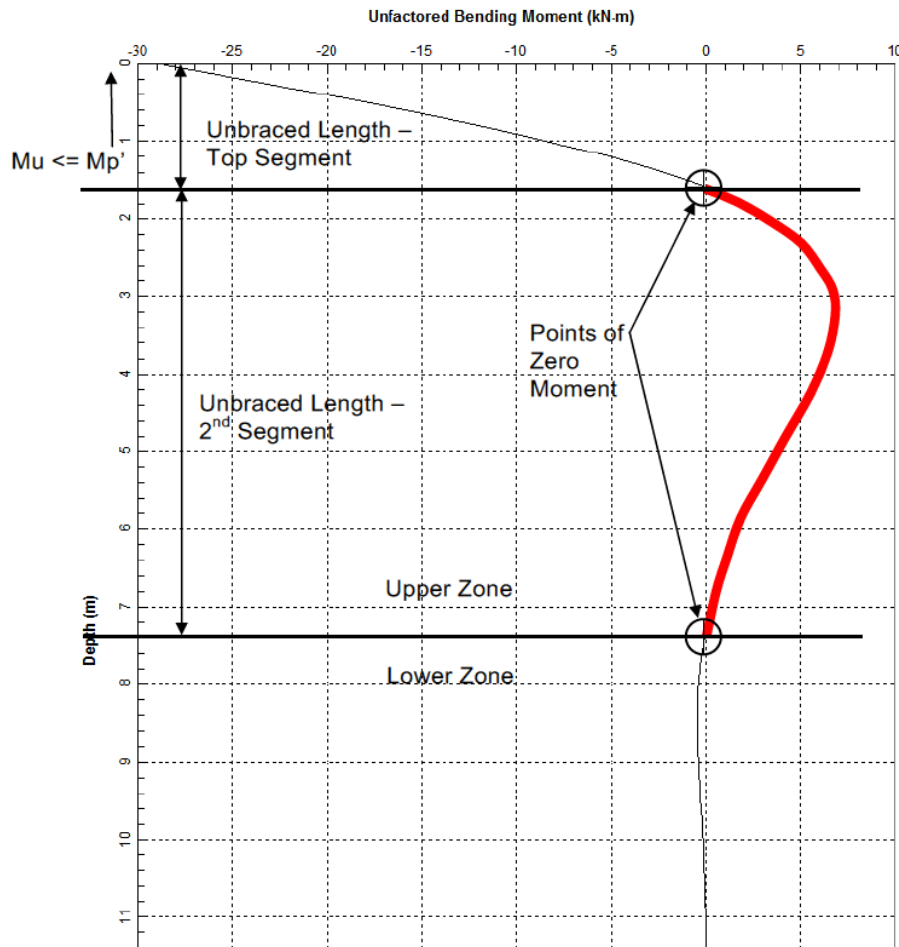
Step 4 - Determine the pile unbraced length and maximum moment at the top of the pile by running LPile software for top translation = 1.06 inches, $P_u = 466 \text{ kip}$, and Live Load Rotation = 0 (Fixed against rotation)



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Maximum moment from LPile output

$$M_u := 3729.36 \text{ kip}\cdot\text{in} = 310.78 \cdot \text{kip}\cdot\text{ft}$$

Unbraced lengths from LPile output

Upper segment

$$L_1 := 5 \cdot \text{ft}$$

Lower segment

$$L_2 := 12.5 \cdot \text{ft}$$

Step 5 - Determine if the applied moment on the pile will cause pile head plastic deformation considering the interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment.

See above for unbraced lengths (critical lengths L_1 and L_2).

Upper segment slenderness factor

$$\lambda_1 := \frac{(L_1)}{r_y} = 16.997$$



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Lower segment slenderness factor

$$\lambda_2 := \frac{(L_2)}{r_y} = 42.493$$

b. Determine K values for top and bottom of the pile per LRFD Table C4.6.2.5-1

Buckled shape of column is shown by dashed line	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical <i>K</i> value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of <i>K</i> when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0
End condition code		Rotation fixed Rotation free Rotation fixed Rotation free		Translation fixed Translation fixed Translation free Translation free		

Upper segment K value (Type d)

$$K_1 := 1.2$$

Lower segment K value (Type c)

$$K_2 := 1$$

c. Calculate the nominal and factored structural pile resistance P_n , per LRFD 6.9.4.1 using the λ values

Elastic critical buckling resistance, P_e , based on flexural buckling

Upper P_e

$$P_{e1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_1 \cdot \lambda_1)^2} = 17956.5 \cdot \text{kip}$$



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Lower P_e

$$P_{e2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_2 \cdot \lambda_2)^2} = 4137.2 \cdot \text{kip}$$

Nominal yield resistance, P_o

$$P_o := F_{y50} \cdot A_g = 1305 \cdot \text{kip}$$

Check that the ratio of P_e to P_o is > 0.44

If $P_e/P_o > 0.44$ then use equation 6.9.4.1.1-1

If $P_e/P_o < 0.44$ then use equation 6.9.4.1.1-2

$$P_n = \left[0.658 \left(\frac{P_o}{P_e} \right) \right] P_o \quad (6.9.4.1.1-1)$$

$$P_n = 0.877 P_e \quad (6.9.4.1.1-2)$$

$$\frac{P_{e1}}{P_o} = 13.76$$

$$\frac{P_{e2}}{P_o} = 3.17$$

Use eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{n1} := \left[0.658 \left(\frac{P_o}{P_{e1}} \right) \right] P_o = 1265.9 \cdot \text{kip}$$

$$P_{n2} := \left[0.658 \left(\frac{P_o}{P_{e2}} \right) \right] P_o = 1143.6 \cdot \text{kip}$$

Factored structural Pile Resistance, $P_r = \phi_c (P_n)$

$$\phi_c := .7$$

$$P_{r1} := \phi_c \cdot P_{n1} = 886.1 \cdot \text{kip}$$

$$P_{r2} := \phi_c \cdot P_{n2} = 800.5 \cdot \text{kip}$$

d. Compare the ratio of P_u , the maximum factored axial load, to P_r , the structural resistance in the specified portion of the pile - the pile size should be such that the ratio is not less than 0.20.



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Check for both segments

$$\frac{P_u}{P_{r1}} = 0.526$$

$$\frac{P_u}{P_{r2}} = 0.582$$

e. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)

Check slenderness ratio for flange, limiting slenderness ratio for compact flange, and limiting slenderness ratio for a noncompact flange.

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$$

slenderness ratio for flange eq. 6.12.2.2.1-3

$$\lambda_{pf} := 0.38 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 9.152$$

$$\lambda_{rf} := 0.83 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 19.989$$

If $\lambda_{pf} < \lambda_f < \lambda_{rf}$ Use equation 6.12.2.2.1-2 to find the nominal flexural resistance

$$M_n := \left[1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left[\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \left(\frac{E}{F_{y50}} \right)^{.5}} \right] \right] \cdot F_{y50} \cdot Z_y = 256.9 \cdot \text{ft} \cdot \text{kip}$$

$$\phi_f := 1.0 \quad \text{for flexural resistance according to Maine BDG}$$

$$M_r := M_n \cdot \phi_f = 256.9 \cdot \text{ft} \cdot \text{kip}$$

$$M_u = 310.8 \cdot \text{ft} \cdot \text{kip} \quad M_u \text{ from LPile} > \text{than } M_r, \text{ exceeding the flexural resistance}$$



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$$\frac{P_u}{P_{rl}} + \frac{8}{9} \cdot \frac{M_u}{M_r} = 1.6$$

If less than 1, remains in elastic zone. Since it exceeds 1, pile is expected to yield at the base of the pile cap.

j. Calculate the moment that will cause a plastic hinge at the top of the pile, M_p' ,

Note: M_p' will be lower than M_n due to the inclusion of the axial load in the interaction equation for pile over stresses

$$\frac{P_u}{P_r} + \frac{8.0}{9.0} \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0 \quad \text{LRFD 6.9.2.2 Interaction equation}$$

Use the interaction equation to find the moment that will cause a plastic hinge at the top of the pile. Assume M_{ux} and $M_{rx} = 0$ (out-of-plane), $M_{ry} = M_r$ and $M_u = M_p'$, solve for M_p'

$$M_p' := \left(\frac{9}{8} \right) \cdot \left[1 - \left(\frac{P_u}{P_{rl}} \right) \right] \cdot M_r = 137 \cdot \text{ft} \cdot \text{kip} \quad M_p = 1644291 \cdot \text{in} \cdot \text{lbf}$$

k. If the calculated moment from LPILE Run 1 exceeds the moment that would cause a plastic hinge, a plastic hinge forms, and the moment (M_p') represents the limiting moment reaction at the pile top for subsequent analysis.

Step 6 - For fixed head piles, run a second LPILE analysis with end conditions 1) Top moment = M_p' , top translation = 1.06 in; and axial load equal to P_u . Recalculate unbraced lengths from the moment vs. depth curve.

New unbraced lengths were determined from the second LPILE analysis

Upper segment	$L_{1p} := 3.0 \cdot \text{ft}$
Lower segment	$L_{2p} := 13 \cdot \text{ft}$

6a. Repeat steps 5a through 5d above.

6b - If the pile size is such that the ratio of P_u to structural resistance exceeds 0.2, check the upper segment of the



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pile with the interaction equation of LRFD 6.9.2.2. If a plastic hinge forms at the top of the pile, the K value of the upper segment changes from 1.2, for a rotation fixed head condition, to 2.1, for a rotation free head condition. With the new K value and lengths repeat step 5.

5a

Upper segment slenderness factor

$$\lambda_{1p} := \frac{(L_{1p})}{r_y} = 10.198$$

Lower segment slenderness factor

$$\lambda_{2p} := \frac{(L_{2p})}{r_y} = 44.193$$

5b

Upper segment K value (Type e)

$$K_{1p} := 2.1$$

Lower segment K value (Type c)

$$K_{2p} := 1$$

5c

Elastic critical buckling resistance, P_e based on flexural buckling

Upper P_e

$$P_{ep1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_{1p} \cdot \lambda_{1p})^2} = 16287.11 \cdot \text{kip}$$

Lower P_e

$$P_{ep2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_{2p} \cdot \lambda_{2p})^2} = 3825.062 \cdot \text{kip}$$

Nominal yield resistance, P_o

$$P_o := F_{y50} \cdot A_g = 1305 \cdot \text{kip}$$

Check that the ratio of P_e to P_o is > 0.44

If $P_e/P_o > 0.44$ then use equation 6.9.4.1.1-1

If $P_e/P_o < 0.44$ then use equation 6.9.4.1.1-2

$$\frac{P_{ep1}}{P_o} = 12.481$$

$$\frac{P_{ep2}}{P_o} = 2.931$$



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Use eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{np1} := \left[0.658 \left(\frac{P_o}{P_{ep1}} \right) \right] \cdot P_o = 1261.96 \cdot \text{kip}$$

$$P_{np2} := \left[0.658 \left(\frac{P_o}{P_{ep2}} \right) \right] \cdot P_o = 1131.34 \cdot \text{kip}$$

Factored structural Pile Resistance, $P_r = \phi_c (P_n)$

$$P_{rp1} := \phi_c \cdot P_{np1} = 883.37 \cdot \text{kip}$$

$$P_{rp2} := \phi_c \cdot P_{np2} = 791.94 \cdot \text{kip}$$

d. Compare the ratio of P_u to the structural resistance in the upper portion of the pile - the pile size should be such that the ratio is not less than 0.20.

Check for both segments

$$\frac{P_u}{P_{rp1}} = 0.5275$$

$$\frac{P_u}{P_{rp2}} = 0.588$$

From VTrans Integral Abutment Design Section 4.5.2 - Check the axial capacity of the upper segment and the interaction equation for the second segment to assess suitability of pile section.

Upper Segment

Check that $P_u/P_{rp1} < 1$

$$\frac{P_u}{P_{rp1}} = 0.528 \quad < 1, \text{ OK}$$

Lower Segment

Ultimate moment along the lower segment from LPile output

$$M_{maxp2} := 103.3 \cdot \text{ft} \cdot \text{kip}$$

$$\frac{P_u}{P_{rp2}} + \frac{8}{9} \cdot \frac{M_{maxp2}}{M_r} = 0.946 \quad < 1, \text{ OK}$$

Initial LPILE Run

EastAbut 14x89.lp9o

LPILE for Windows, Version 2016-09.010

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

Path to file locations:

\09 Jobs\0025900s\09.0025950.00 - WSP Lunt Rd Bridge\09.0025950.01 - Full Replacement\Calcs\Lpile\East\14x89\Weak Axis\

Name of input data file:

EastAbut 14x89.lp9d

Name of output report file:

EastAbut 14x89.lp9o

Name of plot output file:

EastAbut 14x89.lp9p

Page 1

EastAbut 14x89.lp9o

Name of runtime message file:
EastAbut 14x89.lp9r

Date and Time of Analysis

Date: June 25, 2018

Time: 11:04:27

Problem Title

Project Name:

Job Number:

Client:

Engineer:

Description:

Program Options and Settings

Computational Options:

- Use unfactored loads in computations (conventional analysis)

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EastAbut 14x89.1p9o
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
- 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
- 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	=	33.000 ft
Depth of ground surface below top of pile	=	0.0000 ft

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

Page 3

EastAbut 14x89.1p9o
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	13.8000
2	33.000	13.8000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile	
Cross-sectional Shape	= Weak H-Pile
Length of section	= 33.000000 ft
Flange Width	= 14.700000 in
Section Depth	= 13.800000 in
Flange Thickness	= 0.615000 in
Web Thickness	= 0.615000 in
Section Area	= 25.811550 sq. in
Moment of Inertia	= 325.837265 in^4
Elastic Modulus	= 29000000. psi

Ground Slope and Pile Batter Angles

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

Page 4

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

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

Distance from top of pile to top of layer	=	0.0000 ft
Distance from top of pile to bottom of layer	=	12.000000 ft
Effective unit weight at top of layer	=	125.000000 pcf
Effective unit weight at bottom of layer	=	125.000000 pcf
Friction angle at top of layer	=	33.000000 deg.
Friction angle at bottom of layer	=	33.000000 deg.
Subgrade k at top of layer	=	225.000000 pci
Subgrade k at bottom of layer	=	225.000000 pci

Layer 2 is stiff clay without free water

Distance from top of pile to top of layer	=	12.000000 ft
Distance from top of pile to bottom of layer	=	21.000000 ft
Effective unit weight at top of layer	=	118.000000 pcf
Effective unit weight at bottom of layer	=	118.000000 pcf
Undrained cohesion at top of layer	=	1000.000000 psf
Undrained cohesion at bottom of layer	=	1000.000000 psf
Epsilon-50 at top of layer	=	0.008000
Epsilon-50 at bottom of layer	=	0.008000

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	21.000000 ft
Distance from top of pile to bottom of layer	=	31.000000 ft
Effective unit weight at top of layer	=	55.600000 pcf
Effective unit weight at bottom of layer	=	55.600000 pcf
Undrained cohesion at top of layer	=	1000.000000 psf
Undrained cohesion at bottom of layer	=	1000.000000 psf
Epsilon-50 at top of layer	=	0.008000
Epsilon-50 at bottom of layer	=	0.008000

Page 5

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

Distance from top of pile to top of layer	=	31.000000 ft
Distance from top of pile to bottom of layer	=	33.000000 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	=	32.000000 deg.
Friction angle at bottom of layer	=	32.000000 deg.
Subgrade k at top of layer	=	60.000000 pci
Subgrade k at bottom of layer	=	60.000000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer	=	33.000000 ft
Distance from top of pile to bottom of layer	=	38.000000 ft
Effective unit weight at top of layer	=	170.000000 pcf
Effective unit weight at bottom of layer	=	170.000000 pcf
Uniaxial compressive strength at top of layer	=	4000. psi
Uniaxial compressive strength at bottom of layer	=	4000. psi

(Depth of the lowest soil layer extends 5.000 ft below the pile tip)

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

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

The maximum input value, in layer 1, for effective unit weight = 170.00 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties

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Layer	Soil Type	Layer	Effective	Undrained	Angle of	Uniaxial	E50	
Layer	Name	Depth	Unit Wt.	Cohesion	Friction	qu	or	kpy
Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	225.0000
	(Reese, et al.)	12.0000	125.0000	--	33.0000	--	--	225.0000
2	Stiff Clay	12.0000	118.0000	1000.0000	--	--	0.00800	--
	w/o Free Water	21.0000	118.0000	1000.0000	--	--	0.00800	--
3	Stiff Clay	21.0000	55.6000	1000.0000	--	--	0.00800	--
	w/o Free Water	31.0000	55.6000	1000.0000	--	--	0.00800	--
4	Sand	31.0000	62.6000	--	32.0000	--	--	60.0000
	(Reese, et al.)	33.0000	62.6000	--	32.0000	--	--	60.0000
5	Strong Rock	33.0000	170.0000	--	--	4000.	--	--
	(Vuggy Limestone)	38.0000	170.0000	--	--	4000.	--	--

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

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EastAbut 14x89.1p9o

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
1	5	y = 1.060000 in	S = 0.0000 in/in	466000.	N.A.

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:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head	Equivalent Top Depth Below Grnd Surf	Same Layer Type As Layer	Layer is Rock or is Below	F0 Integral for Layer	F1 Integral for Layer
-----------	------------------------------	--------------------------------------	--------------------------	---------------------------	-----------------------	-----------------------

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	ft	ft	Above	Rock Layer	EastAbut 14x89.1p9o lbs	lbs
1	0.00	0.00	N.A.	No	0.00	190243.
2	12.0000	12.0000	No	No	190243.	93150.
3	21.0000	21.0000	Yes	No	283393.	103500.
4	31.0000	31.0000	No	No	386893.	236991.
5	33.0000	33.0000	No	Yes	N.A.	N.A.

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

Compared to yield moment in MathCAD sheet

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 = 1.060000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 466000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.0631	-3728748.	66326.	0.00	97015.	9.45E+09	0.00	0.00	0.00
0.3300	1.0600	-3465066.	66144.	-0.00151	91431.	9.45E+09	-39.6215	148.0199	0.00
0.6600	1.0512	-3199325.	65893.	-0.00290	85804.	9.45E+09	-86.8079	327.0298	0.00
0.9900	1.0370	-2932472.	65449.	-0.00419	80153.	9.45E+09	-137.5732	525.3510	0.00
1.3200	1.0180	-2665509.	64803.	-0.00536	74499.	9.45E+09	-189.0021	735.2283	0.00
1.6500	0.9945	-2399447.	63957.	-0.00642	68865.	9.45E+09	-238.1165	948.1212	0.00
1.9800	0.9671	-2135265.	62918.	-0.00737	63271.	9.45E+09	-286.6033	1174.	0.00
2.3100	0.9361	-1873925.	61695.	-0.00821	57737.	9.45E+09	-330.8226	1399.	0.00
2.6400	0.9021	-1616324.	60305.	-0.00894	52282.	9.45E+09	-371.4406	1631.	0.00
2.9700	0.8653	-1363298.	58762.	-0.00957	46923.	9.45E+09	-407.5815	1865.	0.00

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3.3000	0.8263	-1115609.	57070.	-0.01009	41678.	9.45E+09	-447.3167	2144.	0.00
3.6300	0.7854	-874072.	55207.	-0.01051	36563.	9.45E+09	-493.3045	2487.	0.00
3.9600	0.7431	-639594.	53146.	-0.01082	31598.	9.45E+09	-547.8183	2919.	0.00
4.2900	0.6997	-413213.	50872.	-0.01104	26804.	9.45E+09	-600.5698	3399.	0.00
4.6200	0.6556	-195930.	48391.	-0.01117	22203.	9.45E+09	-652.5466	3941.	0.00
4.9500	0.6112	11271.	45648.	-0.01121	18293.	9.45E+09	-732.5911	4746.	0.00
5.2800	0.5668	206976.	42581.	-0.01116	22437.	9.45E+09	-816.5744	5705.	0.00
5.6100	0.5228	389715.	39173.	-0.01104	26307.	9.45E+09	-904.3992	6851.	0.00
5.9400	0.4794	557970.	35517.	-0.01084	29870.	9.45E+09	-942.5225	7785.	0.00
6.2700	0.4369	711014.	31732.	-0.01057	33111.	9.45E+09	-969.1154	8783.	0.00
6.6000	0.3957	848311.	27855.	-0.01025	36018.	9.45E+09	-988.7959	9896.	0.00
6.9300	0.3558	969445.	23914.	-0.00987	38583.	9.45E+09	-1002.	11149.	0.00
7.2600	0.3175	1074122.	19935.	-0.00944	40800.	9.45E+09	-1008.	12571.	0.00
7.5900	0.2810	1162162.	15943.	-0.00897	42664.	9.45E+09	-1008.	14206.	0.00
7.9200	0.2465	1233494.	11961.	-0.00847	44175.	9.45E+09	-1003.	16110.	0.00
8.2500	0.2140	1288148.	8013.	-0.00794	45332.	9.45E+09	-991.6613	18353.	0.00
8.5800	0.1836	1326255.	4125.	-0.00739	46139.	9.45E+09	-971.7013	20959.	0.00
8.9100	0.1554	1348099.	336.0359	-0.00683	46602.	9.45E+09	-941.9465	24000.	0.00
9.2400	0.1295	1354129.	-3315.	-0.00626	46729.	9.45E+09	-902.1167	27588.	0.00
9.5700	0.1058	1344965.	-6788.	-0.00570	46535.	9.45E+09	-851.9008	31884.	0.00
9.9000	0.08435	1321401.	-10041.	-0.00514	46036.	9.45E+09	-790.8836	37129.	0.00
10.2300	0.06509	1284414.	-13029.	-0.00459	45253.	9.45E+09	-718.4144	43707.	0.00
10.5600	0.04796	1235167.	-15706.	-0.00407	44210.	9.45E+09	-633.3503	52293.	0.00
10.8900	0.03288	1175033.	-18016.	-0.00356	42937.	9.45E+09	-533.4848	64247.	0.00
11.2200	0.01975	1105625.	-19892.	-0.00308	41467.	9.45E+09	-413.9839	82992.	0.00
11.5500	0.00846	1028869.	-21229.	-0.00264	39841.	9.45E+09	-261.0684	122218.	0.00
11.8800	-0.00113	947224.	-21674.	-0.00222	38113.	9.45E+09	36.1832	127021.	0.00
12.2100	-0.00914	865414.	-21238.	-0.00184	36380.	9.45E+09	184.0694	79723.	0.00
12.5400	-0.01572	785821.	-20456.	-0.00150	34695.	9.45E+09	210.7448	53082.	0.00
12.8700	-0.02100	708925.	-19590.	-0.00118	33066.	9.45E+09	226.5385	42726.	0.00
13.2000	-0.02509	635033.	-18673.	-9.02E-04	31502.	9.45E+09	236.8586	37377.	0.00
13.5300	-0.02814	564365.	-17721.	-6.51E-04	30005.	9.45E+09	243.7336	34301.	0.00
13.8600	-0.03025	497082.	-16747.	-4.28E-04	28580.	9.45E+09	248.1736	32492.	0.00
14.1900	-0.03153	433306.	-15759.	-2.33E-04	27230.	9.45E+09	250.7641	31495.	0.00
14.5200	-0.03209	373128.	-14764.	-6.42E-05	25955.	9.45E+09	251.8782	31079.	0.00
14.8500	-0.03204	316611.	-13767.	8.04E-05	24759.	9.45E+09	251.7707	31120.	0.00
15.1800	-0.03146	263798.	-12772.	2.02E-04	23640.	9.45E+09	250.6236	31550.	0.00
15.5100	-0.03044	214710.	-11784.	3.02E-04	22601.	9.45E+09	248.5719	32339.	0.00
15.8400	-0.02906	169355.	-10805.	3.83E-04	21640.	9.45E+09	245.7189	33481.	0.00
16.1700	-0.02741	127722.	-9839.	4.45E-04	20759.	9.45E+09	242.1446	34987.	0.00
16.5000	-0.02554	89787.	-8889.	4.91E-04	19955.	9.45E+09	237.9127	36890.	0.00

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16.8300	-0.02352	55514.	-7956.	5.21E-04	19230.	9.45E+09	233.0738	39239.	0.00
17.1600	-0.02141	24853.	-7044.	5.38E-04	18580.	9.45E+09	227.6685	42105.	0.00
17.4900	-0.01926	-2258.	-6154.	5.43E-04	18102.	9.45E+09	221.7294	45585.	0.00
17.8200	-0.01712	-25889.	-5289.	5.37E-04	18602.	9.45E+09	215.2817	49810.	0.00
18.1500	-0.01501	-46124.	-4450.	5.22E-04	19031.	9.45E+09	208.3449	54961.	0.00
18.4800	-0.01298	-63057.	-3640.	4.99E-04	19389.	9.45E+09	200.9321	61281.	0.00
18.8100	-0.01106	-76790.	-2859.	4.69E-04	19680.	9.45E+09	193.0507	69111.	0.00
19.1400	-0.00927	-87436.	-2111.	4.35E-04	19905.	9.45E+09	184.7016	78931.	0.00
19.4700	-0.00762	-95118.	-1398.	3.97E-04	20068.	9.45E+09	175.8775	91443.	0.00
19.8000	-0.00612	-99969.	-719.4997	3.56E-04	20171.	9.45E+09	166.5607	107698.	0.00
20.1300	-0.00480	-102130.	-79.4069	3.14E-04	20217.	9.45E+09	156.7184	129344.	0.00
20.4600	-0.00364	-101755.	520.5574	2.71E-04	20209.	9.45E+09	146.2938	159097.	0.00
20.7900	-0.00265	-99007.	1078.	2.29E-04	20151.	9.45E+09	135.1888	201758.	0.00
21.1200	-0.00183	-94062.	1590.	1.88E-04	20046.	9.45E+09	123.2265	266682.	0.00
21.4500	-0.00116	-87112.	2051.	1.50E-04	19899.	9.45E+09	110.0561	374963.	0.00
21.7800	-6.39E-04	-78369.	2457.	1.16E-04	19713.	9.45E+09	94.8594	587516.	0.00
22.1100	-2.47E-04	-68078.	2781.	8.49E-05	19496.	9.45E+09	68.4919	1100310.	0.00
22.4400	3.34E-05	-56660.	2898.	5.88E-05	19254.	9.45E+09	-9.2787	1100310.	0.00
22.7700	2.19E-04	-45344.	2759.	3.74E-05	19014.	9.45E+09	-60.9223	1100310.	0.00
23.1000	3.30E-04	-34948.	2480.	2.06E-05	18794.	9.45E+09	-80.0710	961227.	0.00
23.4300	3.82E-04	-25781.	2157.	7.88E-06	18600.	9.45E+09	-83.1433	860808.	0.00
23.7600	3.92E-04	-17898.	1826.	-1.27E-06	18433.	9.45E+09	-83.7029	844889.	0.00
24.0900	3.72E-04	-11313.	1497.	-7.39E-06	18293.	9.45E+09	-82.6428	878697.	0.00
24.4200	3.34E-04	-6015.	1174.	-1.10E-05	18181.	9.45E+09	-80.4259	954131.	0.00
24.7500	2.85E-04	-1975.	861.5737	-1.27E-05	18096.	9.45E+09	-77.3347	1073912.	0.00
25.0800	2.33E-04	855.0204	580.1208	-1.29E-05	18072.	9.45E+09	-64.8132	1100310.	0.00
25.4100	1.83E-04	2668.	351.2365	-1.22E-05	18110.	9.45E+09	-50.7849	1100310.	0.00
25.7400	1.37E-04	3682.	175.4685	-1.09E-05	18132.	9.45E+09	-37.9867	1100310.	0.00
26.0700	9.68E-05	4097.	47.0199	-9.23E-06	18141.	9.45E+09	-26.8863	1100310.	0.00
26.4000	6.36E-05	4088.	-41.2121	-7.51E-06	18141.	9.45E+09	-17.6753	1100310.	0.00
26.7300	3.72E-05	3799.	-96.7009	-5.86E-06	18134.	9.45E+09	-10.3494	1100310.	0.00
27.0600	1.72E-05	3344.	-126.6477	-4.37E-06	18125.	9.45E+09	-4.7752	1100310.	0.00
27.3900	2.76E-06	2812.	-137.5738	-3.08E-06	18113.	9.45E+09	-0.7430	1100310.	0.00
27.7200	-7.17E-06	2266.	-135.0996	-2.01E-06	18102.	9.45E+09	1.9926	1100310.	0.00
28.0500	-1.33E-05	1749.	-123.8611	-1.17E-06	18091.	9.45E+09	3.6834	1100310.	0.00
28.3800	-1.64E-05	1289.	-107.5241	-5.33E-07	18081.	9.45E+09	4.5676	1100310.	0.00
28.7100	-1.75E-05	899.6397	-88.8626	-7.48E-08	18073.	9.45E+09	4.8574	1100310.	0.00
29.0400	-1.70E-05	585.6199	-69.8750	2.36E-07	18066.	9.45E+09	4.7323	1100310.	0.00
29.3700	-1.56E-05	345.3572	-51.9174	4.31E-07	18061.	9.45E+09	4.3372	1100310.	0.00
29.7000	-1.36E-05	172.8414	-35.8399	5.40E-07	18058.	9.45E+09	3.7828	1100310.	0.00
30.0300	-1.13E-05	59.5124	-22.1154	5.89E-07	18055.	9.45E+09	3.1487	1100310.	0.00

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30.3600	-8.95E-06	-4.4853	-10.9562	6.00E-07	18054.	9.45E+09	2.4872	1100310.	0.00
30.6900	-6.58E-06	-29.4759	-2.4124	5.93E-07	18055.	9.45E+09	1.8278	1100310.	0.00
31.0200	-4.25E-06	-25.7810	1.3947	5.82E-07	18054.	9.45E+09	0.09501	88444.	0.00
31.3500	-1.97E-06	-20.5764	1.6709	5.72E-07	18054.	9.45E+09	0.04451	89385.	0.00
31.6800	2.75E-07	-14.6578	1.7466	5.64E-07	18054.	9.45E+09	-0.00628	90326.	0.00
32.0100	2.50E-06	-8.8263	1.6202	5.60E-07	18054.	9.45E+09	-0.05758	91267.	0.00
32.3400	4.71E-06	-3.8911	1.2892	5.57E-07	18054.	9.45E+09	-0.1096	92208.	0.00
32.6700	6.91E-06	-0.6715	0.7504	5.56E-07	18054.	9.45E+09	-0.1625	93149.	0.00
33.0000	9.11E-06	0.00	0.00	5.56E-07	18054.	9.45E+09	-0.2165	47045.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.06309403 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -3728748. inch-lbs
 Maximum shear force = 66326. 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 = 18
 Number of zero deflection points = 4

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	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
---------------	-----------	------------------	-------------	------------------	-------------------	-----------------------------	----------------------------	-----------------------	---------------------------

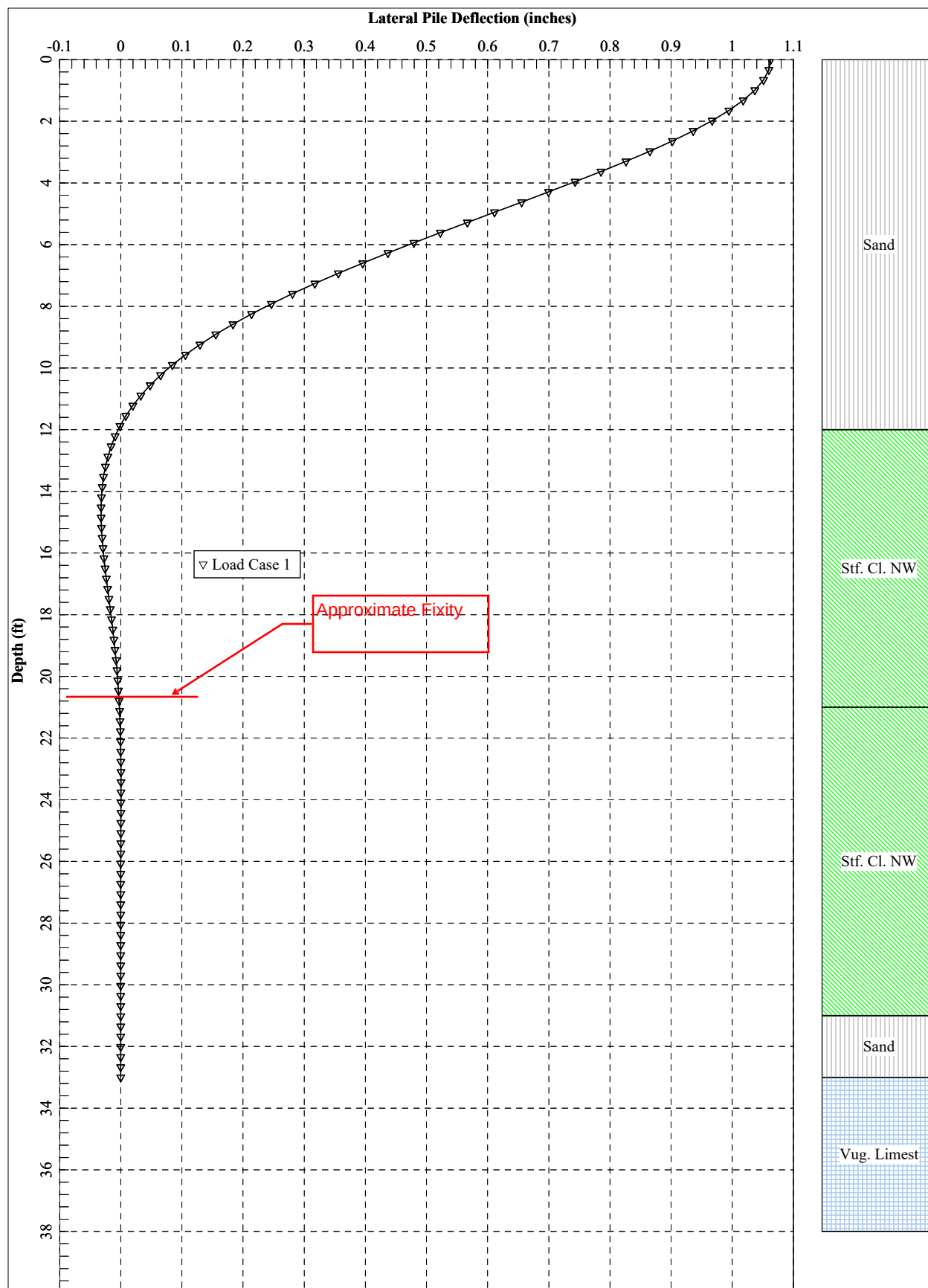
Page 12

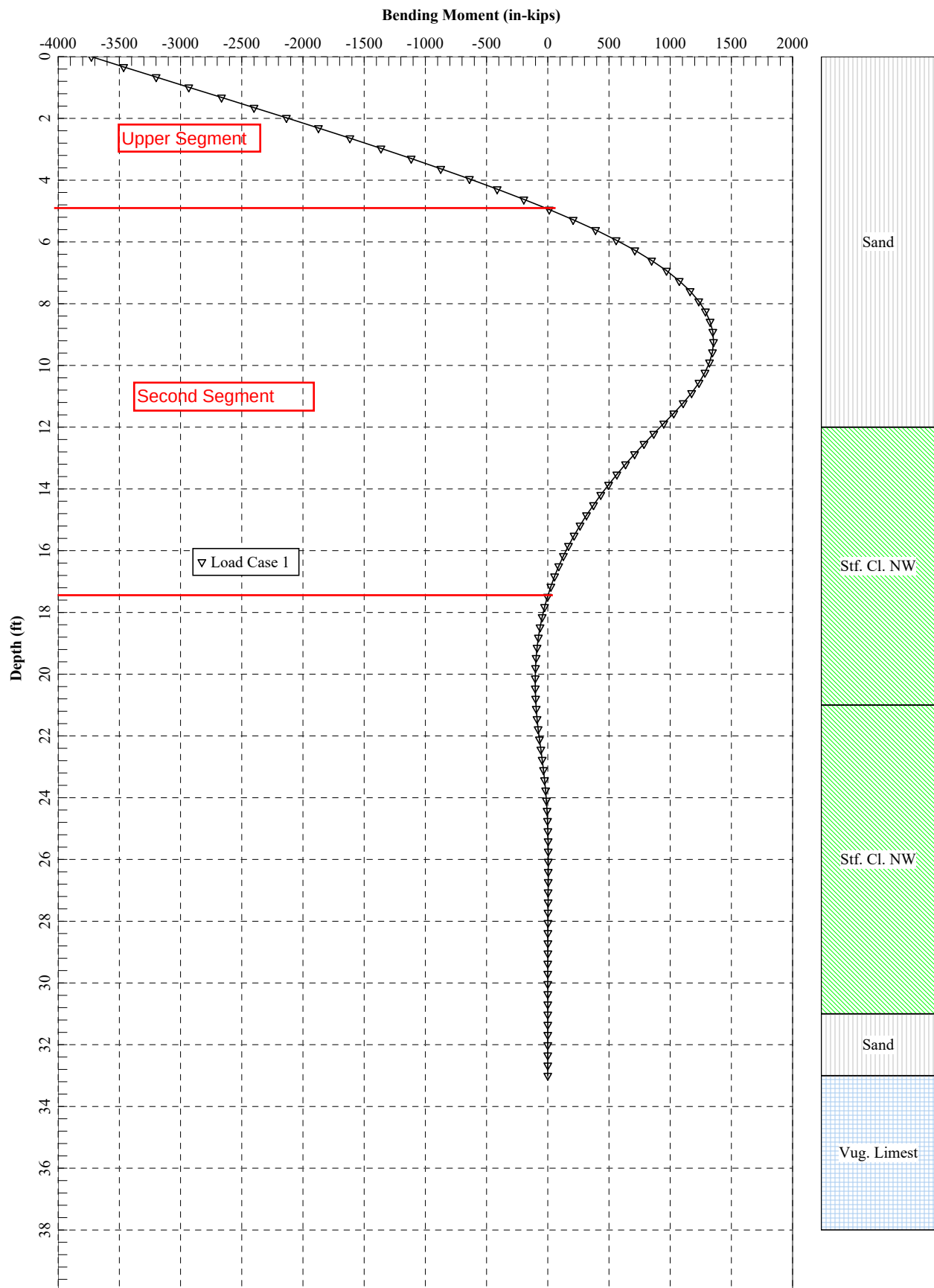
EastAbut 14x89.1p9o

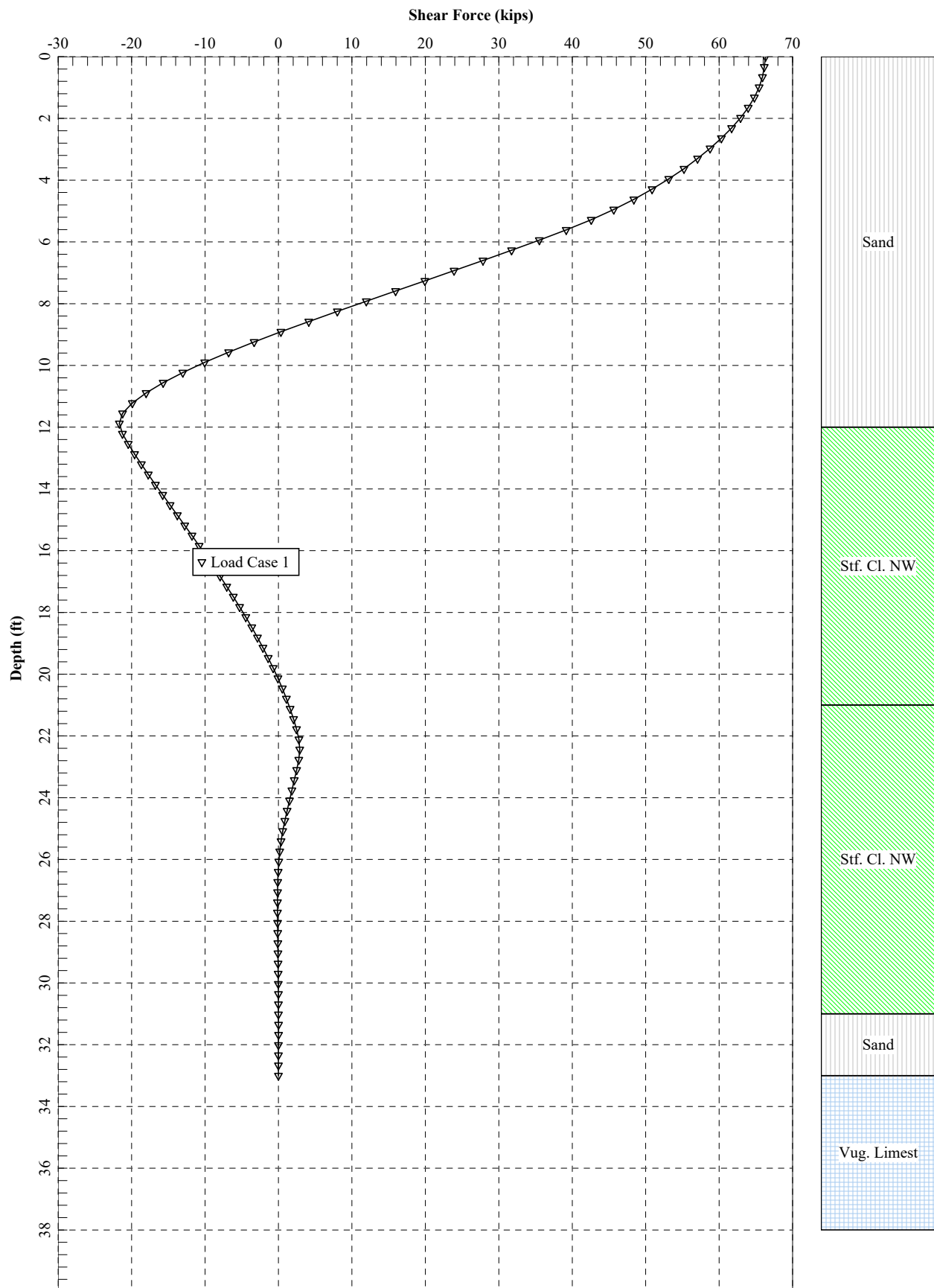
1 y, in 1.0600 S, rad 0.00 466000. 1.0631 0.00 66326. -3728748.

Maximum pile-head deflection = 1.0630940306 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.







EastAbut 14x89 Plastic Moment.lp9o

LPILE for Windows, Version 2016-09.010

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

Path to file locations:

\09 Jobs\0025900s\09.0025950.00 - WSP Lunt Rd Bridge\09.0025950.01 - Full Replacement\Calcs\Lpile\East\14x89\Weak
Axis\Plastic Hinge Formed\

Name of input data file:

EastAbut 14x89 Plastic Moment.lp9d

Name of output report file:

EastAbut 14x89 Plastic Moment.lp9o

Name of plot output file:

EastAbut 14x89 Plastic Moment.lp9p

Page 1

EastAbut 14x89 Plastic Moment.lp9o

Name of runtime message file:

EastAbut 14x89 Plastic Moment.lp9r

Date and Time of Analysis

Date: June 25, 2018

Time: 11:14:00

Problem Title

Project Name:

Job Number:

Client:

Engineer:

Description:

Program Options and Settings

Computational Options:

Page 2

EastAbut 14x89 Plastic Moment.lp9o
- Use unfactored loads in computations (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
- 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
- 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	=	33.000 ft
Depth of ground surface below top of pile	=	0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.
Page 3

EastAbut 14x89 Plastic Moment.lp9o

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	13.8000
2	33.000	13.8000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile	
Cross-sectional Shape	= Weak H-Pile
Length of section	= 33.000000 ft
Flange Width	= 14.700000 in
Section Depth	= 13.800000 in
Flange Thickness	= 0.615000 in
Web Thickness	= 0.615000 in
Section Area	= 25.811550 sq. in
Moment of Inertia	= 325.837265 in^4
Elastic Modulus	= 29000000. psi

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 5 layers

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

Distance from top of pile to top of layer	=	0.0000	ft
Distance from top of pile to bottom of layer	=	12.000000	ft
Effective unit weight at top of layer	=	125.000000	pcf
Effective unit weight at bottom of layer	=	125.000000	pcf
Friction angle at top of layer	=	33.000000	deg.
Friction angle at bottom of layer	=	33.000000	deg.
Subgrade k at top of layer	=	225.000000	pci
Subgrade k at bottom of layer	=	225.000000	pci

Layer 2 is stiff clay without free water

Distance from top of pile to top of layer	=	12.000000	ft
Distance from top of pile to bottom of layer	=	21.000000	ft
Effective unit weight at top of layer	=	118.000000	pcf
Effective unit weight at bottom of layer	=	118.000000	pcf
Undrained cohesion at top of layer	=	1000.000000	psf
Undrained cohesion at bottom of layer	=	1000.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	21.000000	ft
Distance from top of pile to bottom of layer	=	31.000000	ft
Effective unit weight at top of layer	=	55.600000	pcf
Effective unit weight at bottom of layer	=	55.600000	pcf
Undrained cohesion at top of layer	=	1000.000000	psf
Undrained cohesion at bottom of layer	=	1000.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

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

Distance from top of pile to top of layer	=	31.000000	ft
Distance from top of pile to bottom of layer	=	33.000000	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	=	32.000000	deg.
Friction angle at bottom of layer	=	32.000000	deg.
Subgrade k at top of layer	=	60.000000	pci
Subgrade k at bottom of layer	=	60.000000	pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer	=	33.000000	ft
Distance from top of pile to bottom of layer	=	38.000000	ft
Effective unit weight at top of layer	=	170.000000	pcf
Effective unit weight at bottom of layer	=	170.000000	pcf
Uniaxial compressive strength at top of layer	=	4000.	psi
Uniaxial compressive strength at bottom of layer	=	4000.	psi

(Depth of the lowest soil layer extends 5.000 ft below the pile tip)

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

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

The maximum input value, in layer 1, for effective unit weight = 170.00 pcf

This data may be erroneous. Please check your data.

 Summary of Input Soil Properties

EastAbut 14x89 Plastic Moment.lp9o								
Layer	Soil Type	Layer	Effective	Undrained	Angle of	Uniaxial	E50	
Layer	Name	Depth	Unit Wt.	Cohesion	Friction	qu	or	kpy
Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	225.0000
	(Reese, et al.)	12.0000	125.0000	--	33.0000	--	--	225.0000
2	Stiff Clay	12.0000	118.0000	1000.0000	--	--	0.00800	--
	w/o Free Water	21.0000	118.0000	1000.0000	--	--	0.00800	--
3	Stiff Clay	21.0000	55.6000	1000.0000	--	--	0.00800	--
	w/o Free Water	31.0000	55.6000	1000.0000	--	--	0.00800	--
4	Sand	31.0000	62.6000	--	32.0000	--	--	60.0000
	(Reese, et al.)	33.0000	62.6000	--	32.0000	--	--	60.0000
5	Strong Rock	33.0000	170.0000	--	--	4000.	--	--
	(Vuggy Limestone)	38.0000	170.0000	--	--	4000.	--	--

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

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EastAbut 14x89 Plastic Moment.lp9o						
Number of loads specified = 1						
Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	
1	4	y = 1.060000 in	M = -1644291. in-lbs	466000.	N.A.	

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:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer	Top of Layer Below	Equivalent Top Depth Below	Same Layer Type As	Layer is Rock or	F0 Integral	F1 Integral
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No.	Pile Head ft	Grnd Surf ft	Layer Above	EastAbut 14x89 is Below Rock Layer	Plastic for Layer lbs	Moment.lp9o for Layer lbs
1	0.00	0.00	N.A.	No	0.00	190243.
2	12.0000	12.0000	No	No	190243.	93150.
3	21.0000	21.0000	Yes	No	283393.	103500.
4	31.0000	31.0000	No	No	386893.	236991.
5	33.0000	33.0000	No	Yes	N.A.	N.A.

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

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

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head = 1.060000 inches
Moment at pile head = -1644291.0 in-lbs
Axial load at pile head = 466000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb*2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.0600	-1644291.	42577.	-0.00811	52874.	9.45E+09	0.00	0.00	0.00
0.3300	1.0265	-1460078.	42498.	-0.00876	48973.	9.45E+09	-39.6215	152.8500	0.00
0.6600	0.9906	-1275358.	42248.	-0.00934	45061.	9.45E+09	-86.8079	347.0265	0.00
0.9900	0.9525	-1091013.	41804.	-0.00983	41157.	9.45E+09	-137.5732	571.9282	0.00
1.3200	0.9127	-907981.	41157.	-0.01025	37282.	9.45E+09	-189.0022	820.0350	0.00
1.6500	0.8714	-727211.	40311.	-0.01060	33454.	9.45E+09	-238.1166	1082.	0.00
1.9800	0.8288	-549613.	39272.	-0.01086	29693.	9.45E+09	-286.6034	1369.	0.00
2.3100	0.7853	-376084.	38050.	-0.01106	26018.	9.45E+09	-330.8227	1668.	0.00
2.6400	0.7412	-207451.	36659.	-0.01118	22447.	9.45E+09	-371.4408	1984.	0.00

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				EastAbut 14x89	Plastic	Moment.lp9o			
2.9700	0.6968	-44484.	35117.	-0.01123	18996.	9.45E+09	-407.5818	2316.	0.00
3.3000	0.6523	112127.	33424.	-0.01122	20428.	9.45E+09	-447.3170	2716.	0.00
3.6300	0.6079	261636.	31562.	-0.01114	23594.	9.45E+09	-493.3048	3213.	0.00
3.9600	0.5640	403207.	29500.	-0.01100	26592.	9.45E+09	-547.8188	3846.	0.00
4.2900	0.5208	535876.	27226.	-0.01080	29402.	9.45E+09	-600.5704	4566.	0.00
4.6200	0.4785	658712.	24817.	-0.01055	32003.	9.45E+09	-616.1109	5099.	0.00
4.9500	0.4372	771377.	22316.	-0.01025	34389.	9.45E+09	-647.2482	5862.	0.00
5.2800	0.3973	873296.	19704.	-0.00991	36547.	9.45E+09	-672.0576	6699.	0.00
5.6100	0.3588	964001.	17007.	-0.00952	38468.	9.45E+09	-690.0647	7617.	0.00
5.9400	0.3219	1043138.	14243.	-0.00910	40144.	9.45E+09	-705.9508	8686.	0.00
6.2700	0.2867	1110399.	11419.	-0.00865	41568.	9.45E+09	-720.2778	9950.	0.00
6.6000	0.2533	1165505.	8547.	-0.00817	42735.	9.45E+09	-730.0775	11412.	0.00
6.9300	0.2219	1208262.	5645.	-0.00768	43640.	9.45E+09	-735.5673	13125.	0.00
7.2600	0.1925	1238549.	2734.	-0.00716	44282.	9.45E+09	-734.7095	15112.	0.00
7.5900	0.1652	1256357.	-159.0382	-0.00664	44659.	9.45E+09	-726.3735	17413.	0.00
7.9200	0.1399	1261803.	-3004.	-0.00611	44774.	9.45E+09	-710.2778	20101.	0.00
8.2500	0.1168	1255134.	-5769.	-0.00559	44633.	9.45E+09	-686.1688	23272.	0.00
8.5800	0.09568	1236735.	-8422.	-0.00506	44243.	9.45E+09	-653.7946	27060.	0.00
8.9100	0.07665	1207127.	-10930.	-0.00455	43616.	9.45E+09	-612.8597	31663.	0.00
9.2400	0.05962	1166974.	-13258.	-0.00406	42766.	9.45E+09	-562.9460	37390.	0.00
9.5700	0.04453	1117092.	-15369.	-0.00358	41710.	9.45E+09	-503.3601	44761.	0.00
9.9000	0.03130	1058451.	-17223.	-0.00312	40468.	9.45E+09	-432.8122	54766.	0.00
10.2300	0.01982	992205.	-18770.	-0.00269	39065.	9.45E+09	-348.6063	69664.	0.00
10.5600	0.00998	919725.	-19943.	-0.00229	37530.	9.45E+09	-243.8903	96740.	0.00
10.8900	0.00168	842710.	-20524.	-0.00192	35899.	9.45E+09	-49.3076	116436.	0.00
11.2200	-0.00523	764269.	-20307.	-0.00158	34238.	9.45E+09	158.4697	119964.	0.00
11.5500	-0.01087	687722.	-19391.	-0.00128	32617.	9.45E+09	304.1904	110811.	0.00
11.8800	-0.01537	615414.	-18005.	-0.00101	31086.	9.45E+09	396.0122	102037.	0.00
12.2100	-0.01885	548839.	-16784.	-7.63E-04	29676.	9.45E+09	220.4853	46329.	0.00
12.5400	-0.02141	485298.	-15897.	-5.46E-04	28331.	9.45E+09	227.6339	42099.	0.00
12.8700	-0.02317	424951.	-14987.	-3.56E-04	27053.	9.45E+09	232.1758	39676.	0.00
13.2000	-0.02423	367917.	-14062.	-1.89E-04	25845.	9.45E+09	234.7767	38372.	0.00
13.5300	-0.02467	314280.	-13130.	-4.65E-05	24709.	9.45E+09	235.8490	37852.	0.00
13.8600	-0.02460	264098.	-12197.	7.47E-05	23647.	9.45E+09	235.6680	37941.	0.00
14.1900	-0.02408	217407.	-11266.	1.76E-04	22658.	9.45E+09	234.4278	38548.	0.00
14.5200	-0.02321	174225.	-10342.	2.58E-04	21743.	9.45E+09	232.2704	39634.	0.00
14.8500	-0.02204	134550.	-9428.	3.22E-04	20903.	9.45E+09	229.3032	41196.	0.00
15.1800	-0.02065	98367.	-8527.	3.71E-04	20137.	9.45E+09	225.6090	43256.	0.00
15.5100	-0.01910	65645.	-7642.	4.05E-04	19444.	9.45E+09	221.2528	45866.	0.00
15.8400	-0.01744	36343.	-6776.	4.27E-04	18824.	9.45E+09	216.2859	49104.	0.00
16.1700	-0.01572	10404.	-5930.	4.37E-04	18274.	9.45E+09	210.7490	53083.	0.00

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16.5000	-0.01398	-12238.	-5108.	4.36E-04	18313.	9.45E+09	204.6740	57959.	0.00
16.8300	-0.01227	-31660.	-4310.	4.27E-04	18724.	9.45E+09	198.0855	63947.	0.00
17.1600	-0.01060	-47953.	-3540.	4.10E-04	19069.	9.45E+09	191.0009	71343.	0.00
17.4900	-0.00902	-61212.	-2799.	3.88E-04	19350.	9.45E+09	183.4312	80563.	0.00
17.8200	-0.00753	-71548.	-2088.	3.60E-04	19569.	9.45E+09	175.3800	92200.	0.00
18.1500	-0.00617	-79079.	-1411.	3.28E-04	19729.	9.45E+09	166.8427	107126.	0.00
18.4800	-0.00493	-83932.	-767.8307	2.94E-04	19831.	9.45E+09	157.8034	126660.	0.00
18.8100	-0.00384	-86245.	-161.8834	2.58E-04	19880.	9.45E+09	148.2306	152897.	0.00
19.1400	-0.00289	-86167.	404.9872	2.22E-04	19879.	9.45E+09	138.0677	189334.	0.00
19.4700	-0.00208	-83858.	930.2446	1.87E-04	19830.	9.45E+09	127.2138	242272.	0.00
19.8000	-0.00141	-79488.	1411.	1.52E-04	19737.	9.45E+09	115.4822	324309.	0.00
20.1300	-8.73E-04	-73246.	1842.	1.20E-04	19605.	9.45E+09	102.4895	465018.	0.00
20.4600	-4.57E-04	-65341.	2218.	9.13E-05	19438.	9.45E+09	87.2906	756371.	0.00
20.7900	-1.50E-04	-56016.	2473.	6.59E-05	19240.	9.45E+09	41.5900	1100310.	0.00
21.1200	6.47E-05	-45995.	2520.	4.45E-05	19028.	9.45E+09	-17.9738	1100310.	0.00
21.4500	2.03E-04	-36221.	2373.	2.73E-05	18821.	9.45E+09	-56.3284	1100310.	0.00
21.7800	2.81E-04	-27302.	2109.	1.40E-05	18632.	9.45E+09	-76.8725	1084671.	0.00
22.1100	3.13E-04	-19567.	1800.	4.14E-06	18468.	9.45E+09	-79.0699	999512.	0.00
22.4400	3.13E-04	-13058.	1487.	-2.70E-06	18330.	9.45E+09	-79.1112	999573.	0.00
22.7700	2.92E-04	-7778.	1177.	-7.07E-06	18219.	9.45E+09	-77.7386	1054667.	0.00
23.1000	2.57E-04	-3712.	881.1294	-9.47E-06	18133.	9.45E+09	-71.5351	1100310.	0.00
23.4300	2.17E-04	-764.9573	620.1838	-1.04E-05	18070.	9.45E+09	-60.2555	1100310.	0.00
23.7600	1.75E-04	1238.	404.6039	-1.03E-05	18080.	9.45E+09	-48.6232	1100310.	0.00
24.0900	1.35E-04	2478.	233.9578	-9.53E-06	18106.	9.45E+09	-37.5617	1100310.	0.00
24.4200	9.95E-05	3126.	104.8530	-8.36E-06	18120.	9.45E+09	-27.6427	1100310.	0.00
24.7500	6.90E-05	3339.	12.1732	-7.01E-06	18125.	9.45E+09	-19.1652	1100310.	0.00
25.0800	4.40E-05	3248.	-49.9840	-5.62E-06	18123.	9.45E+09	-12.2273	1100310.	0.00
25.4100	2.44E-05	2964.	-87.6332	-4.32E-06	18117.	9.45E+09	-6.7874	1100310.	0.00
25.7400	9.77E-06	2570.	-106.4459	-3.16E-06	18108.	9.45E+09	-2.7140	1100310.	0.00
26.0700	-6.27E-07	2132.	-111.4749	-2.18E-06	18099.	9.45E+09	0.1741	1100310.	0.00
26.4000	-7.48E-06	1695.	-107.0139	-1.38E-06	18090.	9.45E+09	2.0790	1100310.	0.00
26.7300	-1.15E-05	1290.	-96.5576	-7.50E-07	18081.	9.45E+09	3.2020	1100310.	0.00
27.0600	-1.34E-05	933.5170	-82.8318	-2.84E-07	18074.	9.45E+09	3.7302	1100310.	0.00
27.3900	-1.38E-05	634.9152	-67.8664	4.42E-08	18067.	9.45E+09	3.8280	1100310.	0.00
27.7200	-1.31E-05	395.8517	-53.0936	2.60E-07	18062.	9.45E+09	3.6330	1100310.	0.00
28.0500	-1.17E-05	213.4539	-39.4543	3.88E-07	18058.	9.45E+09	3.2555	1100310.	0.00
28.3800	-1.00E-05	81.9424	-27.5049	4.50E-07	18056.	9.45E+09	2.7796	1100310.	0.00
28.7100	-8.15E-06	-6.0446	-17.5150	4.66E-07	18054.	9.45E+09	2.2658	1100310.	0.00
29.0400	-6.32E-06	-58.4953	-9.5541	4.52E-07	18055.	9.45E+09	1.7549	1100310.	0.00
29.3700	-4.57E-06	-83.3816	-3.5631	4.22E-07	18056.	9.45E+09	1.2709	1100310.	0.00
29.7000	-2.97E-06	-88.2738	0.5875	3.86E-07	18056.	9.45E+09	0.8254	1100310.	0.00

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30.0300	-1.51E-06	-80.1547	3.0544	3.51E-07	18056.	9.45E+09	0.4205	1100310.	0.00
30.3600	-1.90E-07	-65.3787	3.9914	3.21E-07	18055.	9.45E+09	0.05268	1100310.	0.00
30.6900	1.03E-06	-49.7261	3.5314	2.97E-07	18055.	9.45E+09	-0.2850	1100310.	0.00
31.0200	2.16E-06	-38.5047	2.8715	2.78E-07	18055.	9.45E+09	-0.04821	88444.	0.00
31.3500	3.23E-06	-28.0095	2.6318	2.64E-07	18055.	9.45E+09	-0.07286	89385.	0.00
31.6800	4.25E-06	-18.6353	2.2956	2.54E-07	18054.	9.45E+09	-0.09695	90326.	0.00
32.0100	5.24E-06	-10.7669	1.8645	2.48E-07	18054.	9.45E+09	-0.1208	91267.	0.00
32.3400	6.22E-06	-4.7846	1.3387	2.45E-07	18054.	9.45E+09	-0.1447	92208.	0.00
32.6700	7.18E-06	-1.0682	0.7177	2.44E-07	18054.	9.45E+09	-0.1689	93149.	0.00
33.0000	8.15E-06	0.00	0.00	2.43E-07	18054.	9.45E+09	-0.1935	47845.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.06000000 inches
 Computed slope at pile head = -0.00811403 radians
 Maximum bending moment = -1644291. inch-lbs
 Maximum shear force = 42577. 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 = 17
 Number of zero deflection points = 4

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

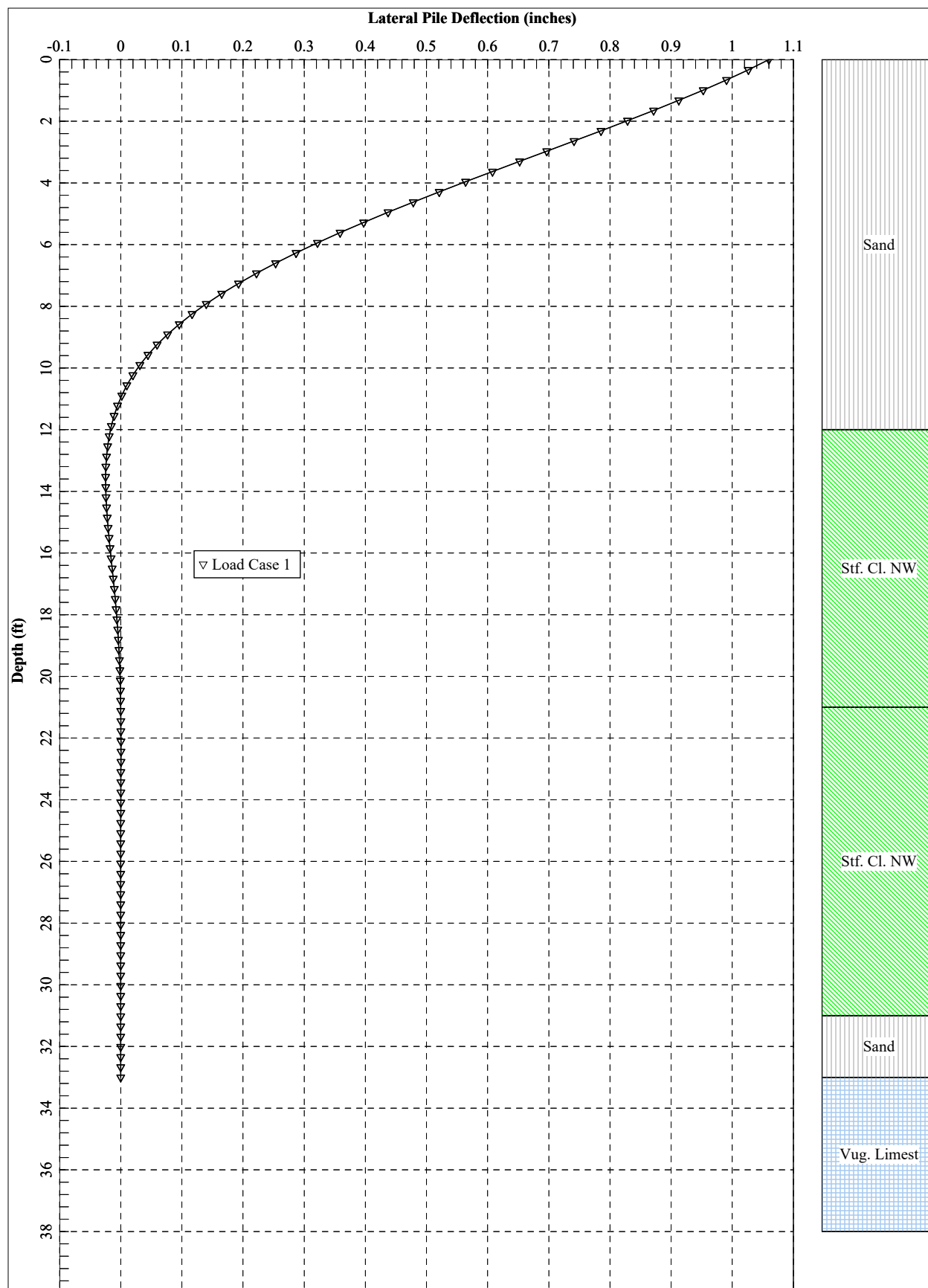
Load Case	Load Type	Load Type	Axial Loading	Pile-head Deflection	Pile-head Rotation	Max Shear in Pile	Max Moment in Pile
-----------	-----------	-----------	---------------	----------------------	--------------------	-------------------	--------------------

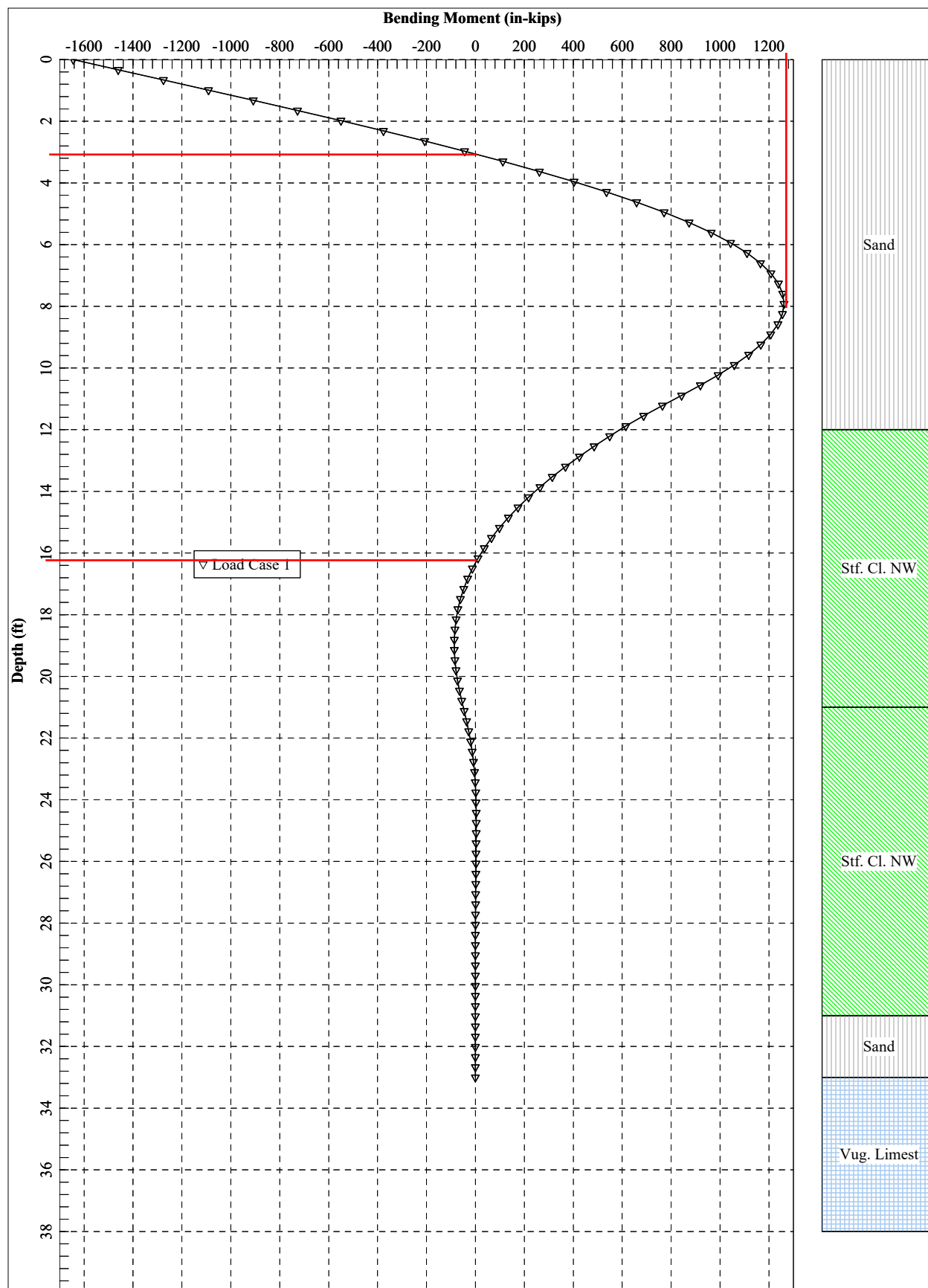
EastAbut 14x89 Plastic Moment.lp9o								
No. 1	Load 1	2	Load 2	lbs	inches	radians	lbs	in-lbs
1	y, in	1.0600 M, in-lb	-1644291.	466000.	1.0600	-0.00811	42577.	-1644291.

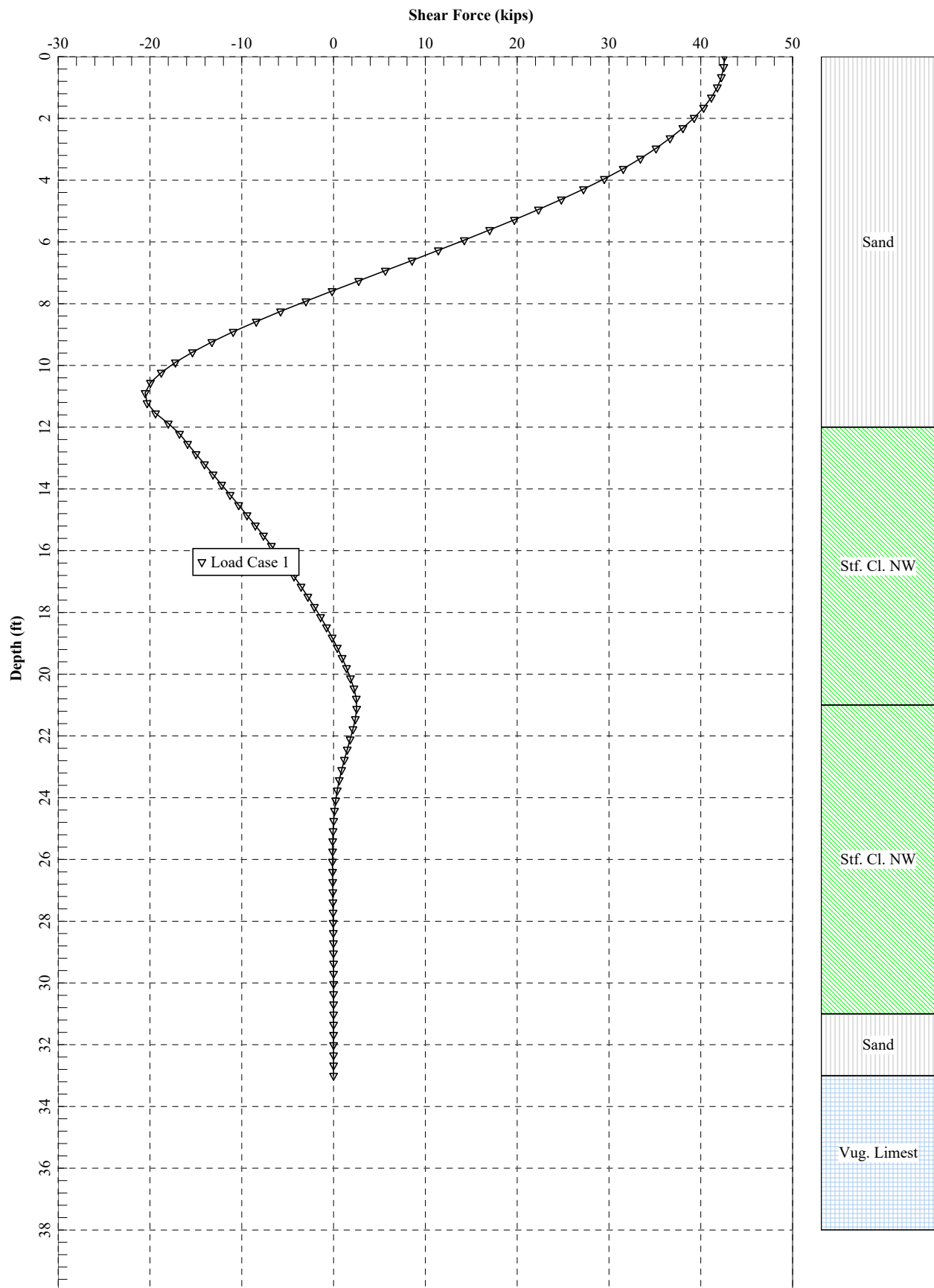
Maximum pile-head deflection = 1.0600000000 inches

Maximum pile-head rotation = -0.0081140307 radians = -0.464900 deg.

The analysis ended normally.







APILE for Windows, Version 2015.7.8

Serial Number : 653550831

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.
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GZA GeoEnvironmental, Inc.
Portland, OR

Path to file locations : P:\09 Jobs\0025900s\09.0025950.00 - WSP Lunt Rd Bridge\09.0025950.01 - Full
Replacement\Calcs\Apile\East\
Name of input data file : HP 14x89.ap7d
Name of output file : HP 14x89.ap7o
Name of plot output file : HP 14x89.ap7p

Time and Date of Analysis

Date: June 28, 2018 Time: 10:27:09

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* INPUT INFORMATION *

Lunt Road Bridge Replacement - Falmouth, ME

DESIGNER : N. Williams

JOB NUMBER : 09.0025950.01

METHOD FOR UNIT LOAD TRANSFERS :

- USACE (U.S. Army Corps of Engineers)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)
- USACE (U.S. Army Corps of Engineers)
Critical Depth Method for Sand:
10 to 20 Pile Diameter based on the Density
- Revised Lambda
- API RP 2A (American Petroleum Institute)
- Load-transfer Units specified by User

TYPE OF LOADING :
- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 26.10 IN²

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 38.00 FT.
 - PILE STICKUP LENGTH, PSL = 5.00 FT.
 - ZERO FRICTION LENGTH, ZFL = 0.00 FT.
 - PERIMETER OF PILE = 57.00 IN.
 - TIP AREA OF PILE = 26.10 IN²
 - INCREMENT OF PILE LENGTH
 USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.00	125.00	33.00	0.00
12.00	SAND	0.00	125.00	33.00	0.00
12.00	CLAY	0.00	118.00	0.00	0.00
21.00	CLAY	0.00	118.00	0.00	0.00
21.00	CLAY	0.00	55.60	0.00	0.00
31.00	CLAY	0.00	55.60	0.00	0.00
31.00	SAND	0.00	62.60	33.00	0.00
33.00	SAND	0.00	62.60	33.00	0.00
33.00	SAND	0.00	150.00	45.00	0.00
38.00	SAND	0.00	150.00	45.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOVED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00

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0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
 WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
 PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
12.00	1.000	1.000
12.00	1.000	1.000
21.00	1.000	1.000
21.00	1.000	1.000
31.00	1.000	1.000
31.00	1.000	1.000
33.00	1.000	1.000
33.00	1.000	1.000
38.00	1.000	1.000

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 * COMPUTATION RESULT *

 * FED. HWY. METHOD *

PILE	TOTAL SKIN	END	ULTIMATE
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HP 14x89.ap7o			
PENETRATION	FRICTION	BEARING	CAPACITY
FT.	KIP	KIP	KIP
0.00	0.0	0.4	0.4
1.00	0.1	0.7	0.9
2.00	0.5	1.3	1.8
3.00	1.2	2.1	3.3
4.00	2.1	2.8	4.9
5.00	3.3	3.5	6.8
6.00	4.8	4.1	8.9
7.00	6.5	4.8	11.4
8.00	8.5	5.5	14.1
9.00	10.8	6.2	17.0
10.00	13.4	6.5	19.9
11.00	16.2	5.6	21.8
12.00	19.2	4.6	23.8
13.00	23.2	3.4	26.6
14.00	28.0	2.0	30.0
15.00	32.7	1.6	34.3
16.00	37.5	1.6	39.1
17.00	42.2	1.6	43.8
18.00	47.0	1.6	48.6
19.00	51.7	1.6	53.3
20.00	56.5	1.6	58.1
21.00	61.2	1.6	62.8
22.00	66.0	1.6	67.6
23.00	70.7	1.6	72.3
24.00	75.5	1.6	77.1
25.00	80.2	1.6	81.8
26.00	85.0	1.6	86.6
27.00	89.7	1.6	91.3
28.00	94.5	1.6	96.1
29.00	99.2	2.1	101.3
30.00	104.0	3.7	107.7
31.00	108.7	12.7	121.4
32.00	114.5	41.8	156.3
33.00	121.3	70.9	192.3

 * ARMY CORPS METHOD *

HP 14x89.ap7o			

PILE	TOTAL SKIN	END	ULTIMATE
PENETRATION	FRICTION	BEARING	CAPACITY
FT.	KIP	KIP	KIP
0.00	0.0	0.5	0.5
1.00	0.2	0.9	1.1
2.00	0.8	1.6	2.4
3.00	1.8	2.5	4.3
4.00	3.3	3.3	6.6
5.00	5.1	4.1	9.3
6.00	7.4	5.0	12.3
7.00	10.0	5.8	15.8
8.00	13.1	6.6	19.7
9.00	16.6	7.4	24.0
10.00	20.5	7.8	28.2
11.00	24.8	6.6	31.4
12.00	29.5	5.3	34.8
13.00	33.7	3.8	37.5
14.00	37.3	2.1	39.4
15.00	40.8	1.6	42.5
16.00	44.4	1.6	46.0
17.00	48.0	1.6	49.6
18.00	51.5	1.6	53.1
19.00	55.1	1.6	56.7
20.00	58.6	1.6	60.3
21.00	62.2	1.6	63.8
22.00	65.8	1.6	67.4
23.00	69.3	1.6	71.0
24.00	72.9	1.6	74.5
25.00	76.5	1.6	78.1
26.00	80.0	1.6	81.6
27.00	83.6	1.6	85.2
28.00	87.1	1.6	88.8
29.00	90.7	2.6	93.3
30.00	94.3	6.1	100.4
31.00	97.8	15.1	113.0
32.00	104.0	39.4	143.3
33.00	112.7	63.6	176.2

 * LAMBDA 2 METHOD *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.0	0.0
1.00	0.3	0.0	0.3
2.00	0.9	0.0	0.9
3.00	1.8	0.0	1.8
4.00	3.0	0.0	3.0
5.00	4.3	0.0	4.3
6.00	5.7	0.0	5.7
7.00	7.3	0.0	7.3
8.00	9.0	0.0	9.0
9.00	10.8	0.0	10.8
10.00	12.6	0.1	12.7
11.00	14.5	0.5	14.9
12.00	16.4	0.8	17.2
13.00	18.4	1.2	19.5
14.00	20.3	1.5	21.8
15.00	22.1	1.6	23.7
16.00	23.9	1.6	25.5
17.00	26.4	1.6	28.0
18.00	29.1	1.6	30.8
19.00	32.0	1.6	33.6
20.00	34.9	1.6	36.5
21.00	37.9	1.6	39.5
22.00	40.9	1.6	42.6
23.00	44.0	1.6	45.7
24.00	47.2	1.6	48.8
25.00	50.3	1.6	51.9
26.00	53.5	1.6	55.1
27.00	56.8	1.6	58.4
28.00	60.0	1.6	61.7
29.00	63.3	1.5	64.9
30.00	66.7	1.2	67.9
31.00	70.1	0.8	70.9
32.00	74.9	0.5	75.3

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33.00 79.8 0.1 79.9

 * API RP-2A (1994) *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.0	0.0
1.00	0.0	0.0	0.0
2.00	0.0	0.0	0.0
3.00	0.0	0.0	0.0
4.00	0.0	0.0	0.0
5.00	0.0	0.0	0.0
6.00	0.0	0.0	0.0
7.00	0.0	0.0	0.0
8.00	0.0	0.0	0.0
9.00	0.0	0.0	0.0
10.00	0.0	0.1	0.1
11.00	0.0	0.5	0.5
12.00	0.0	0.8	0.8
13.00	1.5	1.2	2.7
14.00	4.6	1.5	6.1
15.00	7.8	1.6	9.4
16.00	11.1	1.6	12.7
17.00	14.4	1.6	16.1
18.00	17.9	1.6	19.5
19.00	21.5	1.6	23.1
20.00	25.2	1.6	26.8
21.00	28.9	1.6	30.5
22.00	32.7	1.6	34.4
23.00	36.6	1.6	38.2
24.00	40.5	1.6	42.1
25.00	44.4	1.6	46.1
26.00	48.4	1.6	50.1
27.00	52.5	1.6	54.1
28.00	56.5	1.6	58.1
29.00	60.6	1.5	62.1

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			HP 14x89.ap7o
30.00	64.8	1.2	65.9
31.00	68.9	0.8	69.7
32.00	71.0	0.5	71.5
33.00	71.0	0.1	71.1

NOTES:

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

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

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	-0.5000E+01	0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00	0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01 0.6000E-01 0.8000E-01 0.1200E+00 0.1600E+00 0.5000E+00 0.1000E+02
2	10	-0.2500E+01	0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00	0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01 0.6000E-01 0.8000E-01 0.1200E+00 0.1600E+00 0.5000E+00 0.1000E+02

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			HP 14x89.ap7o
			0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00
3	10	-0.8333E-02	0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00
			0.1600E+00 0.5000E+00 0.1000E+02 0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01 0.6000E-01 0.8000E-01 0.1200E+00 0.1600E+00 0.5000E+00 0.1000E+02
4	10	0.0000E+00	0.0000E+00 0.4193E+00 0.5716E+00 0.6984E+00 0.7542E+00 0.7856E+00 0.8197E+00 0.8378E+00 0.8775E+00 0.8965E+00
			0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01 0.6000E-01 0.8000E-01 0.1200E+00 0.1600E+00 0.5000E+00 0.1000E+02
5	10	0.6025E+01	0.0000E+00 0.6625E+00 0.1144E+01 0.1797E+01 0.2220E+01 0.2515E+01 0.2902E+01 0.3143E+01 0.3786E+01 0.4166E+01
			0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01 0.6000E-01 0.8000E-01 0.1200E+00 0.1600E+00 0.5000E+00 0.1000E+02
6	10	0.1196E+02	0.0000E+00 0.7025E+00 0.1269E+01 0.2125E+01
			0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01

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HP 14x89.ap7o				
7	10	0.1200E+02	0.2742E+01	0.6000E-01
			0.3207E+01	0.8000E-01
			0.3863E+01	0.1200E+00
			0.4304E+01	0.1600E+00
			0.5606E+01	0.5000E+00
			0.6484E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.1026E+01	0.1000E-01
			0.2167E+01	0.2000E-01
			0.4504E+01	0.4000E-01
8	10	0.1653E+02	0.5530E+01	0.6000E-01
			0.5701E+01	0.8000E-01
			0.5530E+01	0.1200E+00
			0.5302E+01	0.1600E+00
			0.5302E+01	0.5000E+00
			0.5302E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.9375E+00	0.1000E-01
			0.1979E+01	0.2000E-01
			0.4115E+01	0.4000E-01
9	10	0.2096E+02	0.5052E+01	0.6000E-01
			0.5208E+01	0.8000E-01
			0.5052E+01	0.1200E+00
			0.4844E+01	0.1600E+00
			0.4844E+01	0.5000E+00
			0.4844E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.9375E+00	0.1000E-01
			0.1979E+01	0.2000E-01
			0.4115E+01	0.4000E-01
10	10	0.2100E+02	0.5052E+01	0.6000E-01
			0.5208E+01	0.8000E-01
			0.5052E+01	0.1200E+00
			0.4844E+01	0.1600E+00
			0.4844E+01	0.5000E+00
			0.4844E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.9375E+00	0.1000E-01
			0.1979E+01	0.2000E-01
			0.4115E+01	0.4000E-01

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HP 14x89.ap7o				
11	10	0.2603E+02	0.9375E+00	0.1000E-01
			0.1979E+01	0.2000E-01
			0.4115E+01	0.4000E-01
			0.5052E+01	0.6000E-01
			0.5208E+01	0.8000E-01
			0.5052E+01	0.1200E+00
			0.4844E+01	0.1600E+00
			0.4844E+01	0.5000E+00
			0.4844E+01	0.1000E+02
			0.0000E+00	0.0000E+00
12	10	0.3096E+02	0.9375E+00	0.1000E-01
			0.1979E+01	0.2000E-01
			0.4115E+01	0.4000E-01
			0.5052E+01	0.6000E-01
			0.5208E+01	0.8000E-01
			0.5052E+01	0.1200E+00
			0.4844E+01	0.1600E+00
			0.4844E+01	0.5000E+00
			0.4844E+01	0.1000E+02
			0.0000E+00	0.0000E+00
13	10	0.3100E+02	0.1275E+01	0.1000E-01
			0.2692E+01	0.2000E-01
			0.5596E+01	0.4000E-01
			0.6872E+01	0.6000E-01
			0.7084E+01	0.8000E-01
			0.6872E+01	0.1200E+00
			0.6588E+01	0.1600E+00
			0.6588E+01	0.5000E+00
			0.6588E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.7337E+00	0.1000E-01
			0.1374E+01	0.2000E-01
			0.2439E+01	0.4000E-01
			0.3289E+01	0.6000E-01
			0.3982E+01	0.8000E-01
			0.5046E+01	0.1200E+00
			0.5824E+01	0.1600E+00
			0.8496E+01	0.5000E+00

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HP 14x89.ap7o				
			0.1069E+02	0.1000E+02
14	10	0.3203E+02		
			0.0000E+00	0.0000E+00
			0.7411E+00	0.1000E-01
			0.1401E+01	0.2000E-01
			0.2523E+01	0.4000E-01
			0.3443E+01	0.6000E-01
			0.4211E+01	0.8000E-01
			0.5418E+01	0.1200E+00
			0.6326E+01	0.1600E+00
			0.9608E+01	0.5000E+00
			0.1251E+02	0.1000E+02
15	10	0.3296E+02		
			0.0000E+00	0.0000E+00
			0.7411E+00	0.1000E-01
			0.1401E+01	0.2000E-01
			0.2523E+01	0.4000E-01
			0.3443E+01	0.6000E-01
			0.4211E+01	0.8000E-01
			0.5418E+01	0.1200E+00
			0.6326E+01	0.1600E+00
			0.9608E+01	0.5000E+00
			0.1251E+02	0.1000E+02
16	10	0.3300E+02		
			0.0000E+00	0.0000E+00
			0.1138E+01	0.1000E-01
			0.2089E+01	0.2000E-01
			0.3588E+01	0.4000E-01
			0.4717E+01	0.6000E-01
			0.5597E+01	0.8000E-01
			0.6881E+01	0.1200E+00
			0.7772E+01	0.1600E+00
			0.1056E+02	0.5000E+00
			0.1258E+02	0.1000E+02
17	10	0.3553E+02		
			0.0000E+00	0.0000E+00
			0.1138E+01	0.1000E-01
			0.2089E+01	0.2000E-01
			0.3588E+01	0.4000E-01
			0.4717E+01	0.6000E-01
			0.5597E+01	0.8000E-01

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HP 14x89.ap7o				
			0.6881E+01	0.1200E+00
			0.7772E+01	0.1600E+00
			0.1056E+02	0.5000E+00
			0.1258E+02	0.1000E+02
18	10	0.3796E+02		
			0.0000E+00	0.0000E+00
			0.1138E+01	0.1000E-01
			0.2089E+01	0.2000E-01
			0.3588E+01	0.4000E-01
			0.4717E+01	0.6000E-01
			0.5597E+01	0.8000E-01
			0.6881E+01	0.1200E+00
			0.7772E+01	0.1600E+00
			0.1056E+02	0.5000E+00
			0.1258E+02	0.1000E+02
TIP	LOAD	TIP MOVEMENT		
	KIP	IN.		
	0.0000E+00	0.0000E+00		
	0.6516E+01	0.1000E-03		
	0.1733E+02	0.5000E-02		
	0.2060E+02	0.1000E-01		
	0.3081E+02	0.5000E-01		
	0.3664E+02	0.1000E+00		
	0.4357E+02	0.2000E+00		
	0.5479E+02	0.5000E+00		
	0.6516E+02	0.1000E+01		
	0.7749E+02	0.2000E+01		

LOAD VERSUS SETTLEMENT CURVE

TOP LOAD TOP MOVEMENT TIP LOAD TIP MOVEMENT
KIP IN. KIP IN.
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			HP 14x89.ap7o
0.9782E+01	0.4924E-02	0.6516E+01	0.1000E-03
0.1475E+02	0.7972E-02	0.8501E+01	0.1000E-02
0.3596E+02	0.2147E-01	0.1733E+02	0.5000E-02
0.5071E+02	0.3268E-01	0.2060E+02	0.1000E-01
0.1212E+03	0.1029E+00	0.3081E+02	0.5000E-01
0.1354E+03	0.1597E+00	0.3664E+02	0.1000E+00
0.1636E+03	0.5746E+00	0.5479E+02	0.5000E+00
0.1743E+03	0.1081E+01	0.6516E+02	0.1000E+01
0.1874E+03	0.2089E+01	0.7749E+02	0.2000E+01

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
200.0	28.66	0.00	1.4	5.52	24.91
400.0	35.86	0.00	3.5	6.95	24.94
500.0	39.25	0.51	4.9	7.26	25.91
600.0	41.93	0.93	6.9	7.68	27.46
717.0	44.16	1.57	11.0	8.14	29.34
800.0	45.40	1.82	16.3	8.47	30.81
900.0	46.57	1.97	31.3	8.82	32.45
1000.0	47.38	2.04	107.1	9.13	33.88
1200.0	46.65	2.10	9999.0	9.21	34.14

