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GEOTECHNICAL DESIGN REPORT

ROUTE 100 BRIDGE NO. 5957 OVER INTERSTATE 95
MAINEDOT WIN 025315.00
ROUTE 100
PALMYRA, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

for
HNTB Corporation
South Portland, Maine

File No. 0208880-001
June 2026

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HNTB Corporation
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Attention: Dilan Gobeil, P.E.
Project Manager

Subject: Geotechnical Design Report
Route 100 Bridge No. 5957 over Interstate 95
MaineDOT WIN 025315.00
Route 100, Palmyra, Maine

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit herewith our report entitled, "Geotechnical Design Report, Route 100 Bridge No. 5957 over Interstate 95, MaineDOT WIN 25315.00, Palmyra, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our proposal, dated 22 September 2025, which was executed by your Matthew Hearn on 24 October 2025. This GDR supersedes the Preliminary Geotechnical Design Report, which was issued to the Maine Department of Transportation (MaineDOT) and you on 28 February 2025.

Introduction

This GDR presents the results of geotechnical field investigation and laboratory testing programs, technical evaluations, and presents geotechnical design recommendations completed by Haley & Aldrich on behalf of HNTB Corporation (HNTB) for the proposed replacement bridge that will carry vehicular traffic on Route 100 over Interstate 95 (I-95) in Palmyra, Maine (see Figure 1) approximately ½-mile south of the intersection with Route 2.

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

Plan locations of test borings (borings) are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 Central Zone. The project elevation datum and elevations (El.) referenced herein are in feet (ft) and reference the North American Vertical Datum of 1988 (NAVD 88).

The proposed limit of work extends along Route 100 from approximately Station (Sta.) 102+75 (south) to Sta. 121+50 (north). Additional work is also planned along limited lengths of the northbound (NB) and southbound (SB) on and off ramps.

PROJECT LOCATION, EXISTING SITE CONDITIONS, AND EXISTING BRIDGE STRUCTURE

The existing bridge carries SB and NB vehicular traffic on Route 100 over I-95 in Palmyra, Maine. The project site consists of a semi-wooded area. Existing roadway grades within the limit of work are generally flat and range from approximately El. 224 to El. 246.

The existing ground surface along the proposed roadway alignment (east of the existing bridge) ranges from approximately El. 245 (Sta. 108+50) down to El. 231 in the vicinity of proposed Abutment 1 (Sta. 111+10). The existing ground surface elevation on I-95, beneath the existing bridge, is at El. 222 (approximate). On the north side of I-95, the existing ground surface ranges from approximately El. 229 near proposed Abutment 2 (Sta. 113+70) up to El. 234 at Sta. 116+75 and back down to El. 225 (Sta. 121+50; north limit of work).

Based on our review of historic, as-built bridge drawings and the preliminary design report (PDR) that was prepared by HNTB Corporation (HNTB; bridge and highway designer), it is our understanding that the existing bridge was constructed in 1963 and consists of an approximate 250-ft long, 30 to 33-ft wide (curb-to-curb), four-span structure that is supported on two cast-in-place concrete stub abutments and three piers. The existing abutments are supported on 25-ft long plumb and battered 10BP42 steel H-piles with a maximum pile load of 37 tons. The existing piers are supported by 8.5-ft square spread footings with a "maximum foundation pressure" of 2.3 tons per square foot (tsf). The spread footings bear on naturally-deposited, inorganic granular soil between El. 212.5 and El. 213.5. Relevant historic drawings are included in Appendix C.

Bridge inspection reports provided by MaineDOT and information included in the PDR indicate that the existing bridge is structurally deficient with ongoing maintenance problems (e.g., cracking of concrete wearing surface; cracking, spalling, and vehicular damage of concrete parapets; minor to moderate cracking and potholes at joint headers.)

PROPOSED BRIDGE STRUCTURE

Based on our review of the final PDR and the 60 percent plans, it is our understanding that the bridge replacement alternative recommended by HNTB generally consists of the following:

- An approximate 260-ft long, 34-ft wide (curb-to-curb), two-span bridge superstructure constructed using metalized steel plate girders.
- Substructures will consist of two pile-supported integral abutments and one pier footing bearing on naturally-deposited, inorganic granular soil at the locations and elevations shown below.

Substructure	Station at Centerline of Alignment (ft)	Approximate Bottom of Foundation Elevation
Abutment 1	Sta. 111+10	235
Pier 1	Sta. 112+40	211
Abutment 2	Sta. 113+70	232

- A 30-inch (in.) thick approach roadway section (25 in. of aggregate subbase and 5 in. of bituminous concrete) that is a minimum of 40-ft wide, which includes two 11-ft wide travel lanes and two 6-ft wide shoulders.
- A horizontal alignment that will be offset to the east, which will require placement of up to approximately 17 ft of fill to meet proposed finish grades.
- The proposed bridge and approach embankments will be constructed in a single phase with traffic maintained on the existing bridge.

Existing and proposed site conditions are shown in Figures 2 and 3.

Geologic Setting

Based on our review of the Maine Geological Survey's (MGS's) Surficial Geologic Map of Pittsfield Quadrangle, Maine (1986), Open-File No. 86-35 (see Reference 1), near-surface soils that are present in the vicinity of the site consist of glacial till (heterogenous mix of sand, silt, clay, and stones). Swamp and marsh deposits (peat, silt, clay, and sand) are also mapped near the site (i.e., in the vicinity of the SB off ramp).

In addition, according the MGS's Bedrock Geologic Map of the Pittsfield Quadrangle, Maine (1971), Open File No. 71-3 (see Reference 2), bedrock present at the site consists of massive quartzite beds with sequences of interbedded phyllite and metasiltstone of the Vassalboro Formation.

Geotechnical Field Investigations

HISTORIC INVESTIGATIONS

Geotechnical field investigations consisting of "wash borings," "auger borings," and "rod soundings," collectively referred to herein after as "explorations," were completed by MaineDOT in support of the design and construction of the existing bridge structure, Route 100, and the NB and SB on and off ramps in 1961 and 1962. The number and location of the historic explorations as well as information on the exploration depths, and drilling and sampling means and methods that were used are shown on the historic boring logs included in Appendix A and/or are summarized in References 3 and 4.

PRELIMINARY INVESTIGATION

Haley & Aldrich completed a preliminary (Phase I) geotechnical field investigation at the site between 24 May and 6 June 2024. Three borings, designated BB-PI95-101 through BB-PI95-103, were drilled adjacent to and east of the existing Abutment 1, Pier 2, and Abutment 2 substructures.

The boring locations were laid out in the field by Haley & Aldrich by taping distances from existing site features prior to the start of drilling. "As-drilled" boring locations and ground surface elevations were determined in the field by MaineDOT using global positioning system (GPS) survey equipment upon the completion of drilling and were provided to Haley & Aldrich. The "as-drilled" boring locations and ground surface elevations are summarized in Table I and are shown in Figure 2.

The borings were drilled by Northern Test Boring, Inc. (NTB) of Gorham, Maine using a Diedrich D-50 track-mounted drill rig. The borings were advanced to depths ranging from approximately 75 to 99 ft below existing ground surface (BGS) using cased-washed drilling methods and a combination of 3-in. (NW-size), 4-in. (HW-size), and 6-in. (SW-size) inside diameter (ID) steel casing.

Soil samples were generally collected continuously from ground surface to approximately 4 to 6 ft BGS and at standard 5-ft intervals thereafter, by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb automatic hammer dropped from a height of 30 in., as indicated on the boring logs. The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the boring logs. The uncorrected SPT N-value (N-uncorrected) is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The drill rig was equipped with a calibrated automatic hammer (hammer) per MaineDOT requirements and Haley & Aldrich reviewed the hammer calibration report provided by NTB, confirmed that the hammer was calibrated within 12 months of when drilling was completed, and confirmed the hammer efficiency factor. The energy-corrected SPT N-vale (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.949; 94.9 percent theoretical hammer efficiency) divided by 0.6. The energy-corrected SPT N-values are also provided on the boring logs.

Unsaturated soil samples recovered from the borings were screened in the field with a photoionization detector (PID) for the presence of volatile organic compounds (VOCs). The measured PID readings are shown on the boring logs.

Approximately 9 to 12 ft of bedrock was sampled (cored) in each boring. The bedrock was cored using a 2-in. (NQ-size) ID diamond-tipped core barrel.

A Haley & Aldrich geologist was present on-site full time to monitor the drilling and to document the soil, bedrock, and groundwater conditions encountered and to prepare boring logs. Soil and bedrock samples were collected and preserved in plastic containers and wooden boxes, respectively, and are currently being stored in our storage facility in Portland, Maine. The soil and bedrock samples that were not submitted for laboratory testing are available for review upon request.

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

SUPPLEMENTAL INVESTIGATION

A supplemental geotechnical field investigation was completed under the direction of Haley & Aldrich at the site on 29 and 30 October 2024. Three borings, designated BB-PI95-201, BB-PI95-202, and BB-PI95-203, were drilled along and east of the existing approach embankment toe north of I-95 to determine whether compressible and low-strength soils were present as this area is near where MGS maps indicate the presence of swamp and marsh deposits, which were encountered in historic borings drilled along the nearby SB off ramp.

The boring locations were laid out in the field by Haley & Aldrich using GPS survey equipment prior to the start of drilling. Similar to the preliminary investigation, MaineDOT determined the “as-drilled” boring locations and ground surface elevations using GPS survey equipment upon the completion of

drilling, which were provided to Haley & Aldrich, and are summarized in Table I, and are shown in Figure 2.

The supplemental borings were drilled by New England Boring Contractors (NEBC) of Hermon, Maine using a Mobile Drill B-53 track-mounted drill. The borings were advanced to depths ranging from approximately 34 to 36 ft BGS and were terminated in naturally-deposited inorganic soil using similar equipment and means and methods to those that were used by NTB to complete the preliminary borings.

The drill rig was equipped with a calibrated automatic hammer (hammer) per MaineDOT requirements and Haley & Aldrich reviewed the hammer calibration report provided by NEBC, confirmed that the hammer was calibrated within 12 months of when drilling was completed, and confirmed the hammer efficiency factor (0.834; 83.4 percent theoretical hammer efficiency).

A Haley & Aldrich geologist was present on-site full time to monitor the drilling and to document the soil, bedrock, and groundwater conditions encountered and to prepare boring logs. Soil samples were collected and preserved in plastic containers and wooden boxes, respectively, and are currently being stored in our storage facility in Portland, Maine. The soil and bedrock samples that were not submitted for laboratory testing are available for review upon request.

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

Generalized Subsurface Conditions

The subsurface conditions encountered in the borings generally consist of fill overlying naturally-deposited glaciolacustrine deposit and/or glacial till soils overlying bedrock. Refer to Table II for a detailed summary of the soil units and thicknesses encountered in each boring. A general description of each soil/bedrock unit is provided separately, below. Detailed soil and bedrock descriptions are provided on the boring logs included Appendix A. Refer to Figure 3 for a graphical representation of the subsurface conditions present along the proposed bridge alignment.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Swamp Deposit	2	Very soft PEAT and SILT, trace fine sand, trace organic material and roots. *(encountered in boring BB-PI95-203)*
Fill	2 to 15	Stiff to hard SILT and lean CLAY with varying amounts of sand and gravel. *(encountered in borings BB-PI95-101 and BB-PI95-103)*
Glaciolacustrine Deposit	10	Very stiff to hard SILT and lean CLAY with varying amounts of sand and gravel. *(encountered in boring BB-PI95-101)*
Glacial Till	57 to 70	Medium dense to very dense SAND and GRAVEL with varying amounts of silt and clay; very stiff to hard SILT and lean CLAY with varying amounts and gravel and sand. *(encountered in each boring)*

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Bedrock	Bedrock was encountered and cored (sampled) in each preliminary (-100 series) boring. The top of the bedrock surface was encountered at depths ranging from approximately 59 to 87 ft BGS (El. 142 to El. 160) and slopes down from north to south.	

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

BEDROCK CONDITIONS

As stated previously, approximately 9 to 12 ft of bedrock was cored (sampled) in the preliminary borings. The sampled and recovered bedrock generally consisted of hard, fresh to slightly weathered, grey to light grey, aphanitic to fine grained, QUARTZITE and PHYLLITE. Primary joints were observed dipping at horizontal to vertical angles and are extremely close to moderately spaced, tight to open, and joint surfaces were rough and smooth.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for the bedrock samples that were recovered from the preliminary borings ranged from 0 to 94 percent (average = 38 percent), indicating variable bedrock quality; from very poor to excellent in accordance with the MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms Field Identification Information" document, dated May 2024.

Detailed bedrock core data and descriptions are provided in Table III and on the boring logs in Appendix A. Photographs of the recovered bedrock core samples are also provided for reference in Appendix A.

GROUNDWATER CONDITIONS

Groundwater observation wells were not installed in any of the preliminary and supplemental borings and therefore, long-term static water levels at the site could not be determined. Because of this, the following general observations were made relative to groundwater conditions during or immediately after drilling:

- Recovered soil samples from each boring were visually observed to be "wet" at approximately 15 to 20 ft BGS (El. 198 to El. 214).
- Water levels measured in each boring during or after drilling ranged from approximately 0 to 11 ft BGS (El. 209 to El. 218).

Please note that the visual observations made during drilling and groundwater levels measured during or after drilling was completed may have been affected by drilling means/methods and may not be representative of actual static water levels at the site. In general, groundwater levels can be expected to fluctuate, subject to boring drilling means/methods, seasonal variation, local soil conditions, topography, and precipitation. Groundwater levels encountered during construction may differ from those observed in the borings.

Geotechnical Laboratory Testing Program

A geotechnical laboratory testing program was undertaken on disturbed soil samples recovered from borings drilled during the preliminary field investigation to assist in soil classification and determination of engineering soil properties. Geotechnical laboratory testing was performed by GeoTesting Express, Inc. of Acton, Massachusetts in accordance with applicable American Society for Testing Materials (ASTM) testing procedures. A summary of the laboratory testing and results is provided below.

Laboratory Test	ASTM Test Designation	Geologic Unit	No. of Tests	Range in Test Results[1]
Atterberg Limits	ASTM D4318-98	Fill	1	LL = 26%; PL = 17%; PI = 9%; WC = 14%
		Glaciolacustrine Deposit	1	LL = 26%; PL = 18%; PI = 8%; WC = 22%
		Glacial Till	5	20% < LL < 31%; 13% < PL < 17% 5% < PI < 16%; 9% < WC < 19%
Grain Size	ASTM D6913	Fill	1	A-4 (AASHTO); CL (USCS)
		Glaciolacustrine Deposit	2	A-4 (AASHTO); ML, CL (USCS)
		Glacial Till	13	A-4, A-1-b, A-2-4, A-6 (AASHTO); SM, ML, CL, SC-SM (USCS)

Note:
[1] LL = liquid limit; PL = plastic limit; PI = plasticity index; WC = water content

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

Geotechnical Evaluations and Design Recommendations

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

- American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Tenth Edition, 2024, referred to herein as AASHTO LRFD.
- MaineDOT Bridge Design Guide (BDG), August 2003, with Interim Revisions through June 2018, referred to herein as BDG.

Engineering calculations that support the design recommendations included herein are provided for reference in Appendix D.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Site class was determined in accordance with AASHTO LRFD Section 3.10.3.1 by correlating SPT N-values in overburden soils to shear wave velocity (V_s), determining the shear wave velocity within 100 ft of ground surface ($\bar{v}_s$), considering a range of shear wave velocity (i.e., multiplied by 1.0 [$\bar{v}_s$], multiplied by 1.3 [$1.3\bar{v}_s$] and divided by 1.3 [$\bar{v}_s/1.33$]), and selecting the corresponding spectral accelerations judged to be most critical for ground motion analyses at each period (e.g., selecting the maximum spectral acceleration at each period).

Based on the nature and consistency of the overburden soil and bedrock present within 100 ft of the ground surface at the site, and the range of correlated shear wave velocities of these materials, Seismic Site Class C/D, D, and D/E were determined using the factored shear wave velocities. Risk-targeted spectral accelerations were determined based on the geographic site location and the different site classifications using the United States Geological Survey (USGS) Seismic Design Web Service. Please note that the AASHTO LRFD design response spectra are 5 percent damped, risk-targeted design response spectra.

We recommend a response spectrum consisting of the maximum acceleration at each period among the different site classes determined from the range of time-averaged shear wave velocities be used for seismic design of the bridge structures. The recommended design response spectrum is provided in Appendix D.

The site falls within Seismic Zone 1 based on the horizontal response spectral acceleration coefficient at 1.0-s period (S_{D1}) being less than 0.10g following AASHTO LRFD Section 3.10.6 (i.e., $S_{D1} < 0.15$ = Seismic Zone 1 from AASHTO LRFD Table 3.10.6-1). Based on our review of the soil conditions encountered in the borings and the results of the laboratory testing, it is our opinion that the potential for saturated granular soils present at the site to liquefy during the design earthquake event is low.

APPROACH EMBANKMENT DESIGN CONSIDERATIONS

Settlement

As stated above, up to approximately 17 ft of fill will be required to construct the proposed bridge approach embankments. As shown in Figure 3, the areas where grade raises are required are underlain by stiff to hard cohesive and/or medium dense to very dense cohesionless soils. In our opinion, these soils are not susceptible to long-term consolidation settlement. However, we do anticipate that these soils will experience some elastic settlement due to proposed raises in grade.

Because similar soil conditions are present along each approach embankment and because the maximum raise in grade is in the vicinity of Abutment 1 (approximately Sta. 111+00; 17 ft), elastic settlement due to the placement of normal weight earthfill was estimated along this approach embankment using the program Settle3 by Rocscience.

Soil descriptions, correlations with SPT N-values, geotechnical laboratory test results, and our experience were used to select soil properties for use in completing the settlement evaluations and are summarized below.

Soil Layer	Property	Value[1]
Existing Fill	Unit Weight	115 pcf
	Modulus of Elasticity	400 ksf
Glacial Deposits (Cohesive)	Unit Weight	125 pcf
	Modulus of Elasticity	300 ksf
Glacial Deposits (Cohesionless)	Unit Weight	125 pcf
	Modulus of Elasticity	1,300 ksf
***Note**:* [1] *pcf = pounds per cubic ft; ksf = kips per square ft*		

The calculated elastic settlement at Sta. 111+00 is approximately 3 in., which we anticipate will occur rapidly during construction and immediately following completion of fill placement, prior to final paving. Post-construction settlement is anticipated to be negligible.

Global Embankment Stability

Embankment construction using normal weight earthfill can cause excessive vertical and lateral strains, potentially resulting in a shear failure of the foundation soil and subsequent failure of the embankments. A series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide2 (version 9.029) to evaluate the likelihood of global stability failures at the site.

Static and pseudo-static (seismic) stability evaluations modeling circular and non-circular failure surfaces were conducted transverse to the proposed baseline (in the west-east/left-right directions) at the approximate locations where maximum raises in grade are anticipated (Sta. 111+00 and 113+75). These locations and directions were judged by Haley & Aldrich to be the critical sections. Global stability evaluations were also completed in the longitudinal direction (i.e., parallel with the proposed baseline in the south-north [Abutment 1] and north-south [Abutment 2] directions).

Typical soils profiles were developed for each analysis location based on the subsurface conditions encountered in the preliminary and supplemental borings. Soil descriptions, correlations with SPT N-values, geotechnical laboratory test results, and our experience were used to select soil properties for use in completing global embankment stability evaluations and are summarized below.

Sta. 111+00 (Abutment 1)			
Material	**Unit Weight (pcf)**	**Friction Angle (degrees)**	**Undrained Shear Strength (psf)**
New Embankment Fill	125	32	0
Fill	115	0	2,000
Glacial Soils (Cohesionless above El. 210)	125	30	0
Glacial Soils (Cohesionless below El. 210)	125	40	0
Glacial Soils (Cohesive above El. 180)	125	0	2,500

Sta. 111+00 (Abutment 1)			
Material	**Unit Weight (pcf)**	**Friction Angle (degrees)**	**Undrained Shear Strength (psf)**
Glacial Soils (Cohesive below El. 180)	125	0	10,000
Riprap *(longitudinal only)*	140	45	0
Abutment Concrete *(longitudinal only)*	Infinite Strength		
HP14x102 steel H-pile *(longitudinal only)* parameters: passive (Method B) force application, 8-ft pile spacing, and lateral capacity per pile of 14 kips			
Note: *"Glacial Soils" refers to Glaciolacustrine Deposits and Glacial Till.*			

Sta. 113+75 (Abutment 2)			
Material	**Unit Weight (pcf)**	**Friction Angle (degrees)**	**Undrained Shear Strength (psf)**
New Embankment Fill	125	32	0
Fill	115	0	1,000 to 2,000
Glacial Soils (Cohesionless above El. 210)	125	30	0
Glacial Soils (Cohesionless below El. 210)	125	40	0
Glacial Soils (Cohesive above El. 180)	125	0	2,500
Glacial Soils (Cohesive below El. 180)	125	0	10,000
Riprap *(longitudinal only)*	140	45	0
Abutment Concrete *(longitudinal only)*	Infinite Strength		
HP14x102 steel H-pile *(longitudinal only)* parameters: passive (Method B) force application, 8-ft pile spacing, and lateral capacity per pile of 14 kips			
Note: *"Glacial Soils" refers to Glaciolacustrine Deposits and Glacial Till.*			

In addition, a 250 pound per square foot (psf) live load surcharge was assumed to act over the width of the embankment travel lanes (transverse) and along the roadway length (longitudinal) in both models (static and pseudo-static). As noted in the tables above, the longitudinal stability evaluations also modeled the presence of the abutment foundation piles.

Please note that the factor of safety for the pseudo-static evaluations was computed using a horizontal acceleration coefficient, k_h, equal to 0.076 g (i.e., one-half of the acceleration coefficient, A_s). A value of $A_s/2$ was selected in accordance with AASHTO LRFD guidance in Section 11.6.5.2.2; the reduction from A_s is due to soil slope flexibility and the fact that the peak ground acceleration during an earthquake lasts only for a very short period of time.

The minimum calculated global stability factors of safety values for static and pseudo-static loading and circular and non-circular, deep-seated failure surfaces are summarized below.

Direction Relative to Roadway	Loading Condition	Minimum Calculated Factor of Safety
Sta. 111+00 (Abutment 1)		
Transverse *(left to right; west to east)*	Static	1.4
	Pseudo-static	1.1
Longitudinal *(left to right; south to north)*	Static	1.4
	Pseudo-static	1.2
Sta. 113+75 (Abutment 2)		
Transverse *(left to right; west to east)*	Static	1.8
	Pseudo-static	1.5
Longitudinal *(right to left; north to south)*	Static	1.5
	Pseudo-static	1.3

The minimum factor of safety required for static stability evaluations is 1.3 where the geotechnical parameters and subsurface stratigraphy are well defined, based on the requirements of LRFD Article 11.6.3.7. It is our opinion that the geotechnical parameters and subsurface stratigraphy are well defined at this site. Per recent MaineDOT guidance, it is our understanding that the minimum required factor of safety for embankments subjected to pseudo-static seismic loading is 1.1.

The results indicate that the proposed embankments meet the required minimum acceptable factors of safety using standard embankment construction and normal weight earthfill and therefore, special embankment construction techniques are not considered necessary.

BRIDGE ABUTMENT AND WINGWALL FOUNDATION SUPPORT

As discussed above and shown in the interpretive subsurface profile (Figure 3), the subsurface conditions at the proposed abutment locations consist of fill overlying glaciolacustrine deposit soils, glacial till, and bedrock. Based on our experience working on similar projects with similar subsurface conditions, it is our opinion that the fill soils are not suitable to support the proposed bridge abutments. We do, however, consider the glaciolacustrine deposit and glacial till soils and bedrock to be suitable abutment foundation bearing strata. Based on the depth to the suitable foundation bearing strata, the anticipated magnitude of the factored compression loads, and the proposed integral nature of the abutments, we consider driven steel H-piles to be the only technically feasible foundation support alternative. It is our understanding, based on our discussions with HNTB, that HP14x102 steel H-piles are being considered for integral abutment foundation support at this time.

Corrosion and Deterioration

The geotechnical engineering design of the proposed abutment piles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. Based on our visual observation of the soil samples recovered from the preliminary and supplemental borings, and our experience on similar projects with

similar soil conditions, it is our opinion that the in-situ soils have low corrosive potential. Because of this, the net factored axial compressive pile resistances recommended in the following sections do not include a reduction in pile cross sectional area for steel degradation.

Static Axial Compressive Pile Resistance

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

Structural Resistance

In accordance with AASHTO LRFD Section 10.7.3, a distinction is made between piles driven to "soft" rock and those that are driven to "hard" rock. Based on our experience on similar projects with similar bedrock conditions, the sampling, recovery, and RQD measurements summarized herein, and observation of the recovered bedrock core samples, it is our opinion that the bedrock present at the abutments be classified as "hard." The geotechnical resistance of piles driven to end bearing in "hard" rock is controlled by the structural resistance as discussed in AASHTO LRFD Section 10.7.3.2.3. The structural resistance factor (Section 6.5.4.2) for axial resistance of piles in compression and subject to damage due to severe driving ("hard" rock bearing stratum) at the Strength Limit State (STR) is 0.5. In addition, resistance factors for Service (SER) and Extreme Event (EE) Limit State loading are 1.0. Therefore, the nominal and factored structural resistances of HP14x102 steel H-piles at the service (SER), strength (STR), and extreme event (EE) limit states are summarized below.

Steel H-pile Section	Nominal Structural Resistance (kips)	Factored Structural Resistance (kips)		
		SER (ϕ = 1.0)	STR (ϕ = 0.5)	EE (ϕ = 1.0)
HP14x102	1,505	1,505	753	1,505

Drivability Resistance

The design of the proposed abutment piles also included consideration of drivability resistance in accordance with AASHTO LRFD Section 10.7.8. The drivability evaluations were conducted using the computer program GRLWEAP 14 version 14.1.20.1 (WEAP) developed by GRL Engineers, Inc. The drivability analyses were conducted to determine what nominal resistance could be achieved assuming the pile can be driven through the overburden soils to the top of bedrock using a hammer size typical of local pile driving contractors without damaging or without overstressing the piles and while maintaining penetration resistance between 3 to 15 blows per in. (bpi), in accordance with MaineDOT Standard Specification Section 501 requirements.

A very dense gravel layer was encountered between El. 179 and El. 189 (approximate) in boring BB-PI95-103, drilled near at Abutment 2. Because a similar layer was not encountered in boring BB-PI95-101,

drilled near Abutment 1, and because driving piles through this layer could be more difficult than driving through other portions of the soil overburden, we judged that drivability would be more critical at Abutment 2 and therefore, drivability analyses were only completed at this substructure location.

The drivability evaluations were conducted assuming that a Delmag D30-32 single acting diesel hammer with a maximum rated energy equal to 75,438 ft-lbs. will be used to install the HP14x102 steel H-piles. The Delmag D 30-32 hammer has a ratchet-style fuel pump with four fuel settings, which are designed to limit the ram stroke to approximately 5.4 ft (fuel setting 1), 7.8 ft (fuel setting 2), 9.5 ft (fuel setting 3), and 11.4 ft (fuel setting 4). Fuel setting 4 was used for the drivability analyses at Abutment 2.

The drivability resistance for HP14x102 steel H-piles are summarized below.

Steel H-pile Section	STR Drivability Resistance (kips)		Maximum Compressive Stress (ksi)	Penetration Resistance (bpi)
	Nominal	Factored (ϕ_{dyn} = 0.65)		
HP14x102	700	455	32	7

As shown above, the maximum compressive stress during driving is below the maximum permissible limit specified in AASHTO LRFD Section 10.7.8, which is 45 ksi (90 percent of F_y for 50 ksi steel piles). The predicted penetration resistance (7 bpi) is between the 3 to 15 bpi range required by the Standard Specifications.

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

Summary of Axial Compressive Pile Resistances

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

Steel H-pile Section	Factored STR Structural Resistance (kips)	Factored STR Drivability Resistance (kips)	Governing STR Factored Resistance (kips)
HP14x102	753	455	455

As summarized above, drivability governs the factored STR axial compressive pile resistance of 455 kips. We understand that HNTB determined the maximum factored STR axial compressive pile load to be 390 kips. Because the factored compressive resistance exceeds the maximum factored pile load, the design requirement is met.

MaineDOT standard practice is to dynamically test (during construction) a minimum of one pile per abutment and not less than 2 percent of the total number of piles anticipated to confirm that the minimum required nominal resistance has been achieved unless variable subsurface or sloping bedrock conditions are present. Based on our review of the available subsurface information, it is our opinion that highly variable or sloping bedrock conditions do not exist. Because of this, we recommend that one pile at each abutment be dynamically tested during construction, prior to the installation of production piles.

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

Lateral Resistance

At the request of HNTB, Haley & Aldrich developed design soil profiles and soil properties at each proposed abutment location for use in completing soil-structure interaction and lateral pile analyses, which are presented below. Please note that the recommended soil properties assume fully saturated soils below El. 217.9 (Abutment 1) and El. 217.1 (Abutment 2).

Abutment 1 (Boring BB-PI95-101)							
Stratum	**Top of Stratum Elevation**	**Moisture Condition**	**Unit Weight (pcf)**	**Friction Angle (ϕ, deg.)**	**Undrained Shear Strength (S_u, psf)**	**Soil Modulus (k, pci)**	**Strain Factor (e_{50}, --)**
Proposed Embankment Fill	235.2	Dry	125	32	Not Applicable (NA)	200	NA
Fill	229.1	Dry	115	NA	2,000	NA	0.007
Fill	217.9	Wet	52.6	NA	2,000	NA	0.007
Glaciolacustrine Deposit	214.1	Wet	62.6	NA	2,500	NA	0.005
Glacial Till	199.1	Wet	62.6	40	NA	125	NA
Glacial Till	169.1	Wet	62.6	NA	10,000	NA	0.004

Abutment 2 (Boring BB-PI95-103)							
Stratum	**Top of Stratum Elevation**	**Moisture Condition**	**Unit Weight (pcf)**	**Friction Angle (ϕ, deg.)**	**Undrained Shear Strength (S_u, psf)**	**Soil Modulus (k, pci)**	**Strain Factor (e_{50}, --)**
Proposed Embankment Fill	231.7	Dry	125	32	NA	200	NA
Fill	219.1	Dry	115	NA	2,000	NA	0.007
Glacial Till	217.1	Wet	62.6	30	NA	60	NA
Glacial Till	209.1	Wet	62.6	40	NA	125	NA
Glacial Till	204.1	Wet	62.6	NA	2,500	NA	0.005
Glacial Till	199.1	Wet	62.6	40	NA	125	NA
Glacial Till	179.1	Wet	62.6	NA	10,000	NA	0.004

Estimated Pile Tip Elevations and Pile Lengths

As stated above, we have assumed that the piles can and need to be driven through the soil overburden and end bear in/on bedrock to develop the minimum required nominal axial compressive pile resistance. Based on this assumption, we recommend the following with respect to pile lengths:

Substructure	Approximate Bottom of Pile Cap Elevation	Approximate Top of Pile Elevation	Approximate Top of Bedrock Elevation[1]	Approximate Pile Tip Elevation[2]	Approximate In-Place Pile Length (ft)
Abutment 1	235	238	142	141	97
Abutment 2	232	235	160	159	76

Notes:
[1] Based on top of bedrock elevations encountered in borings BB-PI95-101 and BB-PI95-103.
[2] Based on the assumption that the piles penetrate 1 ft into bedrock.

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

Based on the recommended in-place pile lengths summarized above, we anticipate that pile splicing will be needed. The actual number of pile splices may vary and is dependent on the actual order lengths proposed by the Contractor and the final pile tip elevations. Please note that Standard Specification

Section 501.047 limits the total number of field splices to one per 40 ft of pile length, for piles over 79-ft long, and requires that the pile piece of lesser length be placed at the tip of the pile (i.e., the first part of the pile that enters the ground).

BRIDGE PIER FOUNDATION SUPPORT

As discussed above and shown in the interpretive subsurface profile (Figure 3), the subsurface conditions at the proposed pier location consist of very stiff to hard cohesive, and medium dense to very dense cohesionless glacial till soils overlying bedrock. Based on our experience working on similar projects with similar subsurface conditions, it is our opinion that the glacial till is a suitable foundation bearing strata and because of that, we recommend that a shallow, soil-supported spread footing be used to support the proposed pier.

Pier foundation design recommendations based on footing dimensions of 14.5 ft x 32.83 ft that were provided by HNTB are provided below.

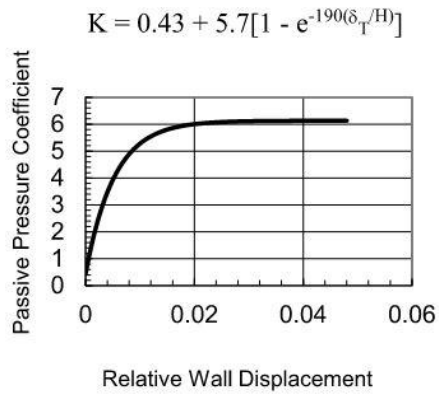
- Bearing Resistance:
  - At the STR, the footing should be designed for a factored bearing resistance equal to 11 ksf, using a resistance factor of 0.45. HNTB reported a STR contact pressure of 8.4 ksf, which is less than the STR factored bearing resistance.
  - At the EE, the footing should be designed for a factored bearing resistance of 20 ksf, using a resistance factor of 0.8. HNTB reported a EE contact pressure of 4.8 ksf, which is less than the EE factored bearing resistance.
- Settlement:
  - HNTB provided a SER contact pressure of 6.2 ksf for the pier footing. At this pressure, we anticipate that up to approximately 1.5 in. of elastic settlement will occur.
- Load Distribution and Eccentricity:
  - Application of permanent and transient loads is specified in AASHTO LRFD Section 11.5.6. We recommend the stress distribution at the base of the footing be assumed to be a uniform distribution over the effective footing base as shown in AASHTO LRFD Figure 11.6.3.2-1.
  - The eccentricity of loading at the STR, based on factored loads, should not exceed one-third of the spread footing dimensions in either direction in accordance with AASHTO LRFD Section 11.6.3.3. This eccentricity corresponds to the resultant of reaction forces falling within the middle two-thirds of the base width and length.
- Sliding Resistance:
  - In accordance with AASHTO LRFD Tables C3.11.5.3-1 and 10.5.5.2.2-1, we recommend that sliding resistance of the pier footing be calculated using the design parameters presented below.

Subgrade Saturation Condition During Construction	Coefficient of Friction (tan δ)	Interface Friction Angle (δ, degrees)	STR Resistance Factor for Sliding (ϕ_τ)	SER/EE Resistance Factor for Sliding (ϕ_τ)
Prepared in-the-dry	0.4	22	0.85	1.0

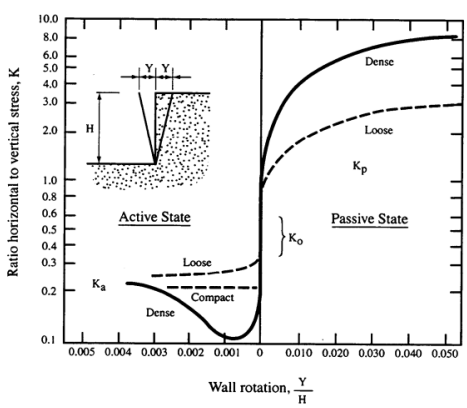
  - Lateral passive soil resistance in front of the footings, if present, should be neglected in accordance with requirements of the BDG.

ABUTMENT BREASTWALL AND WINGWALL DESIGN

- Drainage:
  - The abutment and wingwall design should include a drainage system to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the structure. We recommend that drainage be provided in accordance with BDG Section 5.4.2.13.
- Lateral Earth Pressures:
  - Recommendations summarized below are based on the following:
    - Abutment breastwall and wingwalls are backfilled with a free-draining material (i.e., Soil Type 4, BDG Table 3-3; total unit weight = 125 pcf; internal angle of friction = 32 degrees; interface friction angle = 24 degrees).
    - The abutment breastwall and wingwall backwalls are vertical.
    - Adequate drainage is provided, as recommended herein and in accordance with the requirements of the BDG, to eliminate the potential for unbalanced hydrostatic pressures to develop.
  - Integral abutments should be designed using a static passive lateral earth pressure coefficient (k_p) calculated using the "movement-dependent" coefficient in accordance with AASHTO LRFD Article C3.11.5.4 and Federal Highway Administration (FHWA) NHI-06-089. We recommend using the more stringent of the methodology developed by the Massachusetts Department of Transportation (MassDOT) or FHWA HNI-06-089, Figure 10-4 as outlined below. We also recommend that the k_p value used be no less than 3.25.



Plot of Passive Pressure Coefficient, K. vs. Relative Wall Displacement, δ_T/H (MassDOT).



Magnitude of Wall Rotation to Reach Failure

Soil type and condition	Rotation, Y/H	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

Plot of Passive Pressure Coefficient, K. vs. Relative Wall Displacement, γ/H (FHWA).

- We recommend a load factor for passive earth pressure equal to 1.5 be used to calculate factored passive lateral earth pressures consistent with MaineDOT standard practice.
- In general, we recommend that walls be designed for a live load surcharge equivalent to the earthfill height summarized in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2. A uniform lateral load equal to the surcharge times the lateral earth pressure coefficient should also be applied to abutments and walls to account for the live load surcharge.

FROST PROTECTION

The minimum depth of embedment/cover for both abutments were evaluated in accordance with Section 5.2.1 of the BDG. Based on a design freezing index equal to 1,800 freezing degree-days, we recommend that the pier footing and abutments bear a minimum of 5.5 ft below the lowest adjacent ground surface exposed to freezing.

Construction Considerations

The primary purpose of this section of the GDR is to comment and provide recommendations on items related to excavation, earthwork, driven pile installation and dynamic testing, and other geotechnical aspects of the proposed construction. Since it identifies potential construction issues related to foundations and earthwork, the information in this section is intended to aid personnel who monitor the construction activities. Prospective Contractors for this project should evaluate construction issues based on their own knowledge and experience in the Palmyra, Maine area taking into consideration their proposed construction methods, procedures, equipment, and personnel.

EXCAVATION

Excavation will be required to construct the mass concrete footing supporting Pier 1 as well as other project features. Based on existing ground surface and proposed bottom of footing elevations, excavation depths ranging up to approximately 7 ft BGS will likely be required to construct the footing. We anticipate that excavation of the in-situ fill, glaciolacustrine deposit, and glacial till soils can be accomplished using normal earth-excavating equipment (i.e., hydraulic backhoes and excavators).

We anticipate that excavations may be made using sloped, open-cut techniques. We recommend that the Contractor be responsible for the design, stability, and safety of all excavations in accordance with local, state, and federal regulations.

We also recommend that the Contractor evaluate the need for installing a temporary support of excavation (SOE) system (e.g., steel sheet piles, soldier piles and lagging) to construct the mass concrete footing supporting Pier 1. A temporary SOE system may be needed to limit the amount of excavation and/or limit potential impacts to adjacent infrastructure (e.g., I-95 NB and SB travel lanes, existing Pier 2 footing). If used and if the temporary SOE system extends below the proposed bearing level of the footing, we recommend that this portion (i.e., the portion below the bearing level of the footing) be cutoff and abandoned in place.

CONSTRUCTION DEWATERING

Based on the water levels measured in boreholes during drilling and the proposed bottom of the Pier 1 footing, we anticipate groundwater may be encountered during construction since the bottom of excavation will be within approximately 1 ft of the measured water levels. Because of this, we anticipate that temporary dewatering may be needed, which can likely be accomplished by open pumping using sumps. We recommend that the Contractor be made responsible for controlling all infiltration from groundwater and surface runoff to allow excavation, subgrade preparation, and placement and compaction of backfill soils associated with the Pier 1 footing occur in-the-dry.

Excavation and control of water should be done by methods that prevent disturbance to foundation bearing (subgrade) soils. Sumps should be provided with filters suitable to prevent pumping of fine-grained soil particles.

Dewatering and discharge of dewatering effluent should be performed in accordance with all applicable local, state, and federal regulations. Dewatering effluent should be recharged on-site, if possible.

PIER 1 FOOTING SUBGRADE PREPARATION

As discussed in previous sections of this GDR, we anticipate that soils present at the bottom of excavation will likely consist of fine-grained glacial till deposit soils that contain varying amounts of granular (i.e., sand/gravel) soil. These soils can easily be disturbed and subject to strength loss and increased compressibility by construction activities if care is not taken in excavating within a few feet of the proposed subgrade level and in protecting the subgrade surface after preparation and prior to footing construction and placement and compaction of backfill soils. The following guidelines are recommended to protect the subgrade soils beneath the footing:

- Make final excavations (e.g., within 5 ft of final footing bearing level) into bearing soils in-the-dry using smooth-bladed equipment to limit disturbance.
- Prevent water from accumulating on soil surfaces to reduce the possibility of soil disturbance, strength loss, and increased compressibility. All excavation, subgrade preparation, footing construction, and backfilling should be performed in-the-dry. Subgrades that become disturbed due to water infiltration should be re-excavated and stabilized. Subgrade stabilization methods could include placement of a concrete mudmat or geosynthetic encapsulated crushed stone. Proposed subgrade stabilization methods should be submitted to and reviewed by the Resident and Geotechnical Engineer. Exposed subgrades should be examined in the field by the Resident and/or Geotechnical Engineer prior to rebar cage construction to verify strength and bearing resistance. Over-excavation may be necessary to remove weak, disturbed, or otherwise unacceptable soils. All unsuitable material, if present as determined by the Resident and/or Geotechnical Engineer, shall be over-excavated and replaced with suitable granular soil or geosynthetic encapsulated crushed stone.
- Exposed granular soils at bearing strata could be proofrolled with self-propelled, static/vibratory compaction equipment until firm and prior to placement of footing reinforcement if the soil appears dry and no “free” water is observed as determined by the Resident and/or Geotechnical Engineer. To minimize disturbance, we recommend that wet/saturated granular or cohesive soils exposed at subgrade level not be proofrolled.
- Fill or backfill should be placed in lifts not exceeding 12 in. in loose measure and compacted using self-propelled vibratory equipment. In confined areas, the maximum loose layer should be reduced to 9 in., and compaction performed by hand-guided equipment. Cobbles or boulders having a size exceeding two-thirds of the loose lift thickness should be removed prior to compaction.
- Disturbance due to water infiltration and adverse weather could be reduced by maintaining footing excavations at least 12 in. above the final bearing level until immediately before exposing and preparing the subgrade. It may be desirable to protect the exposed soil subgrade areas as soon as possible after acceptance by the Resident and/or Geotechnical Engineer.
- Limit equipment traffic on exposed soil-bearing surfaces.

- If disturbance and rutting occur, the disturbed materials should be removed and replaced to the satisfaction of the Resident and/or Geotechnical Engineer.
- Soil-bearing surfaces below completed foundations should be protected against freezing, before and after foundation construction. If construction is performed during freezing weather, footings should be backfilled to a sufficient depth (up to 6 ft) as soon as possible after they are constructed. Alternatively, insulating blankets or other means may be used for protection against freezing.
- We recommend that the Contractor be made responsible for protecting subgrade surfaces. Any damage to the subgrade surface resulting from Contractor means and methods should be repaired to the satisfaction of the Resident and/or Geotechnical Engineer at no additional expense to MaineDOT.

ABUTMENT DYNAMIC PILE LOAD TESTING PROGRAM

The Contractor will be required to confirm the nominal axial compressive pile resistance in the field using dynamic testing methods. The piles should be driven to a minimum nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 in accordance with AASHTO LRFD Table 10.5.5.2.3-1 (390 kips/0.65 = 600 kips). We recommend that the Contractor perform a minimum of one dynamic pile load test for each abutment with 24-hour (minimum) restrike to evaluate hammer system efficiencies, driving stresses in the pile, and the nominal resistance of the piles. We also recommend that the dynamic testing be completed prior to production pile driving and that CAPWAP analysis be performed on the test piles. The CAPWAP results will be used to finalize driving criteria for the production piles to assure that the piles achieve the minimum required resistance without being overstressed.

SUBMITTAL REVIEWS

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

CONSTRUCTION MONITORING

The geotechnical design and earthwork recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation construction activities is required to enable the geotechnical engineer to confirm that procedures and techniques used by the Contractor during construction are appropriate and will not impact the design of the bridge. Therefore, we recommend that an individual representing HNTB and

MaineDOT, qualified by geotechnical training and experience, be present at the site to provide monitoring during the foundation construction activities listed below:

- Determination of over-excavation limits of unsuitable soils beneath the pier footing bearing level.
- Preparation of the pier footing bearing surface.
- Dynamic testing of test piles at Abutments 1 and 2.

Limitations

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

Use of this report by any person or entity, including by HNTB and MaineDOT, for a purpose other than relative to the subject project is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the report is adequate for such other use. Use of this report by any other person or entity for such other purpose without written authorization by Haley & Aldrich shall be at such person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

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

It is our understanding that this report will be included as a reference document in the package that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the Contract Documents and that the information contained in the Contract Documents takes precedence over the information provided in this report.

Closure

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

Sincerely yours,
HALEY & ALDRICH, INC.

Tucker P. Jones, P.E.
Assistant Project Manager

Nathan A. Sherwood, P.E.
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STATE OF MAINE
BRYAN C. STEINERT
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LICENSED
PROFESSIONAL ENGINEER

Enclosures:

- Table I – Subsurface Exploration Location Data
- Table II – Subsurface Exploration Subsurface Data
- Table III – Subsurface Exploration Bedrock Core Data
- Figure 1 – Project Locus
- Figure 2 – Boring Location Plan
- Figure 3 – Interpretive Subsurface Profile
- Appendix A – Boring Logs and Bedrock Core Photographs
- Appendix B – Geotechnical Laboratory Test Results
- Appendix C – Historic Drawings
- Appendix D – Geotechnical Calculations

References

1. Map entitled, “Reconnaissance Surficial Geology of the Pittsfield Quadrangle, Maine,” Open-File No. 86-35, prepared by Lindley S. Hanson and Dabney W. Caldwell, Maine Geological Survey, Department of Conservation, dated 1986.

2. Map entitled, “Reconnaissance Bedrock Geology of the Pittsfield Quadrangle, Maine,” Open-File No. 71-3, prepared by John R. Griffin, Maine Geological Survey, Department of Conservation, dated 1971.

3. Report entitled, “Subsurface Investigation for the Route 100 Structure over the Interstate Highway, Palmyra, Maine,” prepared by the Maine State Highway Commission Soils Division, dated June 1962.

4. Report entitled, “Subsurface Investigation for the Proposed Interchange at Interstate 95 and Route 100 in Palmyra, Maine, Project I-95-7(29),” prepared by the Maine State Highway Commission Soils Division, dated June 1962.

TABLES

TABLE I
Subsurface Exploration Location Data
Route 100 Bridge No. 5957 over Interstate 95
MaineDOT WIN 025315.00
Route 100, Palmyra, Maine

Haley & Aldrich, Inc. File No.: 0208880-000

Boring No.[1]	Ground Surface Elevation[2] (ft)	Station[4]	Offset Distance & Direction[4,5] (ft)	Horizontal Coordinates[3]	
				Northing (Y)	Easting (X)
BB-PI95-101	229.1	111 + 7.7	6.9 RT	486,194	1,597,079
BB-PI95-102	217.6	112 + 44.3	9.0 LT	486,310	1,597,153
BB-PI95-103	219.1	113 + 89.7	29.4 LT	486,398	1,597,275
BB-PI95-201	211.0	114 + 45.2	138.7 RT	486,374	1,597,399
BB-PI95-202	210.9	115 + 48.7	114.2 RT	486,467	1,597,443
BB-PI95-203	208.9	116 + 52.3	97.0 RT	486,558	1,597,490

Notes:
[1] Boring locations are shown on Figure 2, Boring Location Plan.
[2] Ground surface elevations at boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet, and reference the North American Vertical Datum of 1988 (NAVD 88).
[3] Horizontal coordinates of borings were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the NAD83, Maine 2000 Central Zone coordinate system.
[4] Station and offset information shown is considered approximate, is relative to project baselines, and was determined by Haley & Aldrich, Inc. using the horizontal coordinate information shown and project baselines provided by HNTB Corporation.
[5] LT = offset distance left of baseline (looking upstation); RT = offset distance right of baseline (looking upstation).

	Individual	Date
Prepared By:	T. Jones	6/11/2024
Checked By:	N. Sherwood	2/12/2025
Reviewed By:	B. Steinert	10/17/2025

TABLE II
Subsurface Exploration Subsurface Data
Route 100 Bridge No. 5957 over Interstate 95
MaineDOT WIN 025315.00
Route 100, Palmyra, Maine

Haley & Aldrich, Inc. File No.: 0208880-000

Boring No.[1]	Ground Surface Elevation[2] (ft)	Stratigraphic Data[3] (ft)														Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration (ft)
		Swamp Deposit			Fill			Glaciolacustrine Deposit			Glacial Till			Bedrock[4]			
		Depth to Top (ft)	Elevation of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elevation of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elevation of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elevation of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elevation of Top (ft)		
BB-PI95-101	229.1	NE	NE	NE	0.0	229.1	15.0	15.0	214.1	10.0	25.0	204.1	61.8	86.8	142.3	98.5	130.6
BB-PI95-102	217.6	NE	NE	NE	NE	NE	NE	NE	NE	NE	0.0	217.6	70.3	70.3	147.3	84.2	133.4
BB-PI95-103	219.1	NE	NE	NE	0.0	219.1	2.0	NE	NE	NE	2.0	217.1	57.0	59.0	160.1	74.5	144.6
BB-PI95-201	211.0	NE	NE	NE	NE	NE	NE	NE	NE	NE	0.0	211.0	>36.0	--	--	36.0	175.0
BB-PI95-202	210.9	NE	NE	NE	NE	NE	NE	NE	NE	NE	0.0	210.9	>36.0	--	--	36.0	174.9
BB-PI95-203	208.9	0.0	208.9	2.3	NE	NE	NE	NE	NE	NE	2.3	206.6	>31.8	--	--	34.1	174.8

Notes:
[1] Boring locations are shown on Figure 2, Boring Location Plan.
[2] Ground surface elevations at boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet, and reference the North American Vertical Datum of 1988 (NAVD 88).
[3] "NE" indicates stratum was not encountered in boring; "--" indicates boring not drilled deep enough or sampling not completed to determine presence of stratum; ">35.5" indicates boring not drilled deep enough to determine full thickness of stratum; actual thickness could be greater than value shown.
[4] The top of possible weathered bedrock was encountered at approximately 31.8 ft BGS (El. 177.1) in boring BB-PI95-203.

	Individual	Date
Prepared By:	T. Jones	6/11/2024
Checked By:	N. Sherwood	2/12/2025
Reviewed By:	B. Steinert	10/17/2025

TABLE III
Subsurface Exploration Bedrock Core Data
Route 100 Bridge No. 5957 over Interstate 95
MaineDOT WIN 025315.00
Route 100, Palmyra, Maine

Haley & Aldrich, Inc. File No.: 0208880-000

Boring No.[1]	Ground Surface Elevation[2] (ft)	Bedrock Core Diameter (in.)	Core Run					Total Core Recovery[3]		Rock Quality Designation[4,5]			Physical Rock Parameters		Lithologic, Rock Mass, and Discontinuity Description[6]
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (in.)	%	Length (in.)	%	Rock Quality	Weathering	Estimated Field Strength	
				Top	Bottom	Midpoint									
BB-PI95-101	229.1	NQ (2")	R1	89.0	91.6	90.3	2.6	19.0	61%	12	38%	Poor	Fresh	Hard	Grey, aphanitic to fine grained, QUARTZITE. Primary joints low to vertical angles, spaced very close to close, tight to open, fresh, quartz stringers throughout.
			R2	91.6	92.7	92.2	1.1	10.2	77%	0	0%	Very Poor			
			R3	92.7	93.5	93.1	0.8	5.8	60%	0	0%	Very Poor			
			R4	93.5	98.5	96.0	5.0	54.6	91%	49	81%	Good			
BB-PI95-102	217.6	NQ (2")	R1	75.2	77.4	76.3	2.2	23.4	88%	8	31%	Poor	Fresh to slightly weathered	Hard	Grey, aphanitic, PHYLLITE. Primary joints dipping horizontal to high angles, spaced extremely close to moderately close, smooth and planar or rough and stepped, fresh to discolored, tight to open. Few calcite stringers.
			R2	77.4	78.8	78.1	1.4	15.3	91%	4	24%	Very Poor			
			R3	78.8	81.0	79.9	2.2	26.4	100%	11	42%	Poor			
			R4	81.0	84.2	82.6	3.2	38.4	100%	11	29%	Poor			
BB-PI95-103	219.1	NQ (2")	R1	63.0	65.7	64.4	2.7	28.4	88%	10	31%	Poor	Fresh to slightly weathered	Hard	Light grey, aphanitic, QUARTZITE. Primary joints dipping at horizontal to moderate angles, spaced very close to close, smooth and planar to rough and undulating, mostly fresh, open.
			R2	65.7	68.7	67.2	3.0	36.0	100%	17	47%	Poor			
			R3	68.7	71.0	69.9	2.3	27.6	100%	0	0%	Very Poor			
			R4	71.0	73.0	72.0	2.0	22.0	92%	18	75%	Fair			
			R5	73.0	74.5	73.8	1.5	20.0	111%	17	94%	Excellent			

Notes:
[1] Boring locations are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.
[2] Ground surface elevations at boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[3] Total core recovery (TCR) is the length of core recovered divided by the length of the run.
[4] Rock Quality Designation (RQD) is the total length of intact, full-diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of at least 4 in.) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in the determination of RQD.
[5] Designation based on RQD in accordance with MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms" Field Identification Information, dated May 2024.
[6] Refer to the boring logs and rock core photographs in Appendix A for additional information.

	Individual	Date
Prepared By:	T. Jones	6/11/2024
Checked By:	N. Sherwood	2/12/2025
Reviewed By:	B. Steinert	10/17/2025

FIGURES

SITE COORDINATES: 44°50'02"N, 69°17'31"W

MAP SOURCE: USGS

HALEY ALDRICH

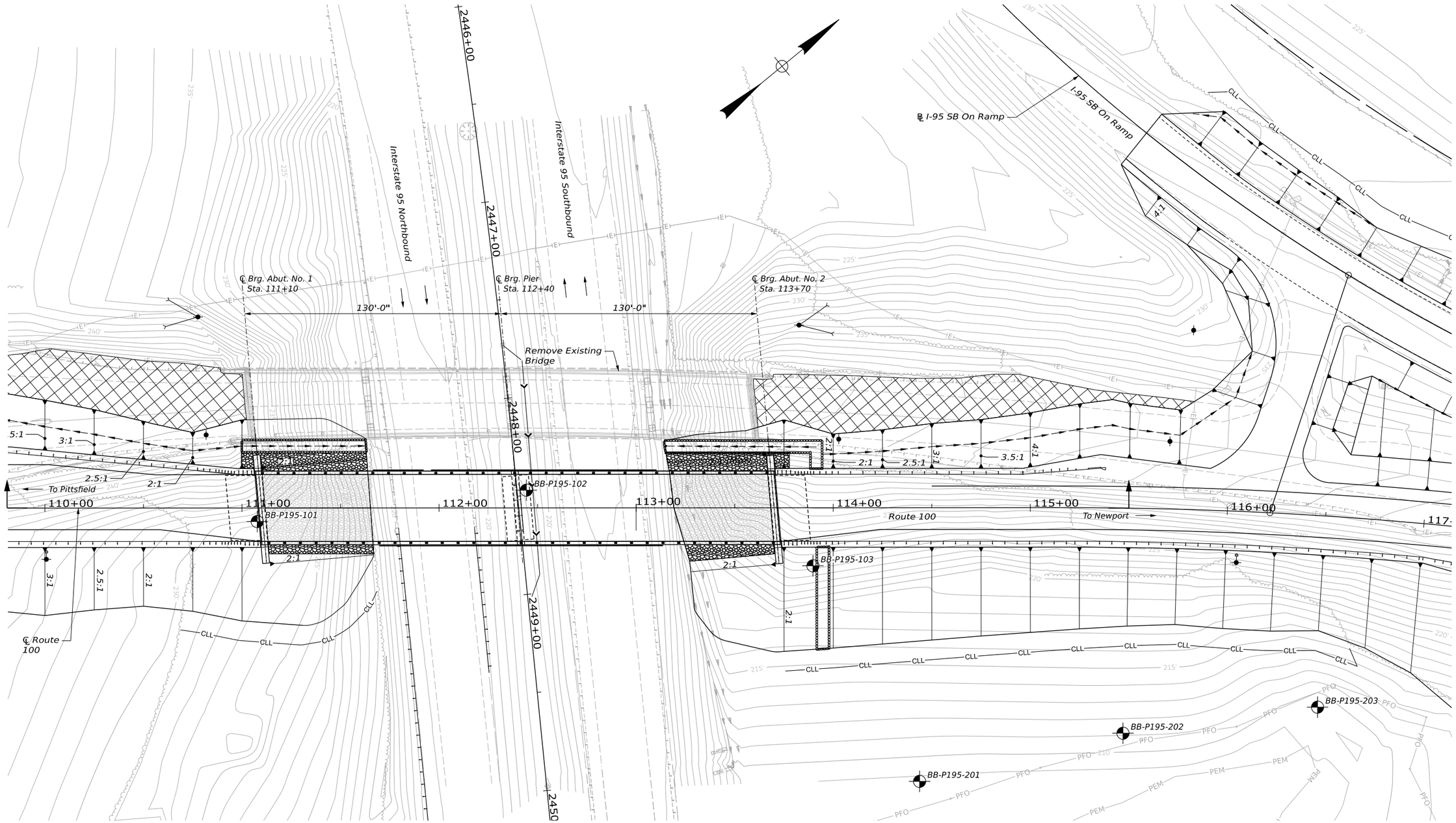
ROUTE 100 BRIDGE NO. 5957 OVER INTERSTATE 95
MAINEDOT WIN 025315.00
ROUTE 100, PALMYRA, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 2,000 FEET
JUNE 2026

FIGURE 1

0208880_000_LOCUS_HALEYALDRICHINSHERWOOD



LEGEND:

BB-P195-101 — Designation and as-drilled location of preliminary phase test boring drilled by Northern Test Boring Inc. and monitored by Haley & Aldrich, Inc. in May and August 2024

BB-P195-201 — Designation and as-drilled location of design phase bridge boring drilled by New England Boring Contractors and monitored by Haley & Aldrich, Inc. in October 2025

Approximate location and orientation of interpretive subsurface profile

Key

BB = Bridge boring

PLAN

25 0 25 50
Scale of Feet

NOTES:

1. Existing site and topographic information and project stationing were taken from electronic files provided by HNTB on 16 February 2026.
2. As-drilled locations of -100 and -200 series test borings were determined in the field by MaineDOT using GPS survey equipment.
3. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

HALEY ALDRICH

ROUTE 100 BRIDGE NO. 5957 OVER INTERSTATE 95
MAINEDOT WIN 025315.00
ROUTE 100, PALMYRA, MAINE

BORING LOCATION PLAN

SCALE: AS SHOWN
JUNE 2026

FIGURE 2

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

Federal Project No. 2531500

WIN 025315.00

SIGNATURE

P.E. NUMBER

DATE

		BY	DATE
PROJ MANAGER	M PARLIN		
DESIGN-DETAILED	T JONES	K POST	FEB 2026
CHECKED-REVIEWED	N SHERWOOD	B STEINERT	APR 2026
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

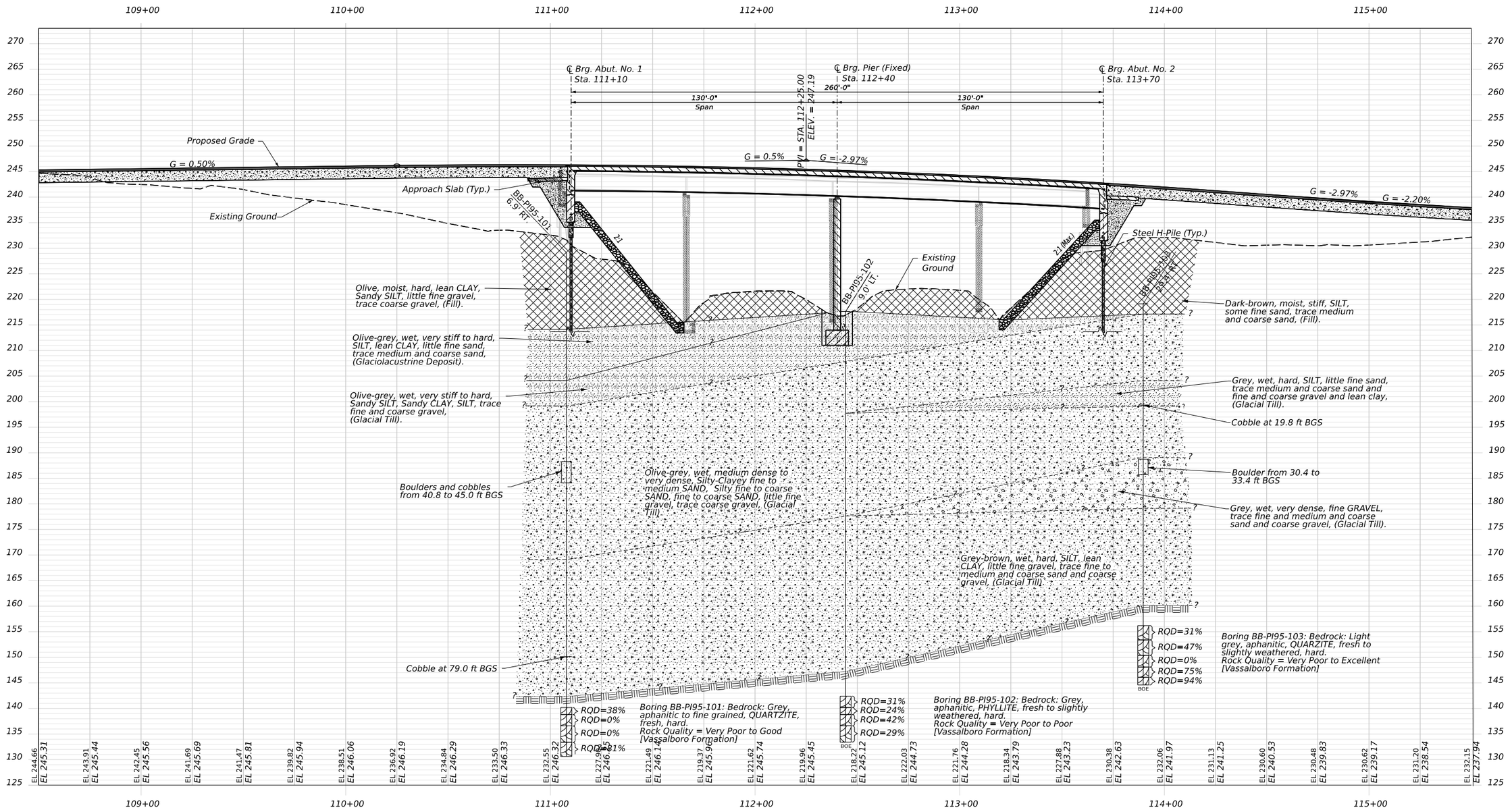
ROUTE 100/I-95 BRIDGE NO. 5957
CROSSING ROUTE 100 / INTERSTATE 95
PALMYRA

BORING LOCATION PLAN

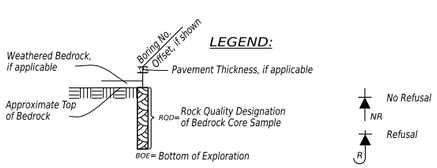
SHEET NUMBER
14
OF 100

Date: 4/27/2026
Username: kpostolowski

Filename: \\haleyaldrich.com\share\CF\Projects\0208880\CAD\252315.00\dgn\0208880_003_Plan_(Route 100).dgn



ROUTE 100 PROFILE



NOTES:

1. This generalized interpretive subsurface 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 information refer to the exploration logs.
2. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

HALEY ALDRICH

ROUTE 100 BRIDGE NO. 5957 OVER INTERSTATE 95
MAINEDOT WIN 025315.00
ROUTE 100, PALMYRA, MAINE

INTERPRETIVE SUBSURFACE PROFILE

SCALE: AS SHOWN
JUNE 2026

FIGURE 3

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

Federal Project No. 2531500

WIN 025315.00

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	T. JONES	K. POST	FEB 2026
CHECKED-REVIEWED	N. SHERWOOD	B. STEINERT	APR 2026
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

ROUTE 100/I-95 BRIDGE NO. 5957
CROSSING ROUTE 100 / INTERSTATE 95
PALMYRA

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
15
OF 100

Date: 6/18/2026
Username: kpostolowski

Filename: \\haleyaldrich.com\share\CF\Projects\0208880\CAD\252315.00\dgn\0208880_004_Profile_(Route 100).dgn

APPENDIX A
Boring Logs and Bedrock Core Photographs

Preliminary and Supplemental Boring Logs and Bedrock Core Photographs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, Gravelly sands, little or no fines
			SP	Poorly-graded sands, Gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.
			OL	Organic silts and organic Silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Density of Cohesionless Soils	Standard Penetration Resistance N_{60}-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

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

Consistency of Cohesive soils	SPT N_{60}-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-101
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	229.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/24/2024-5/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	111+7.7, 6.9 RT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in. ID	**Water Level*:**	11.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.0 - 2.0	15/18/12/14	30	47	HSA		Dark grey-brown, moist, hard, Sandy SILT, trace fine and coarse gravel and lean clay, non-plastic, (Fill). PID Reading = 0.0 ppm	
	2D	24/18	2.0 - 4.0	11/12/14/14	26	41			Olive, moist, hard, lean CLAY, little fine and medium sand and fine gravel, trace coarse sand and coarse gravel, moderately bonded, (Fill). PID Reading = 0.0 ppm	G#779699 WC=14% LL=26 PL=17 A-4, CL
5	3D	24/7	5.0 - 7.0	10/14/17/19	31	49			Olive, moist, hard, lean CLAY, little fine and medium sand and fine gravel, trace coarse sand and coarse gravel, trace wood fibers, non-plastic, moderately bonded, organic odor, (Fill). PID Reading = 0.0 ppm	G#779699 WC=14% LL=26 PL=17 A-4, CL
10	4D	14/14	10.0 - 11.2	10/15/50(2")			117		Olive-brown, moist, hard, lean CLAY, little fine and medium sand and fine gravel, trace coarse sand and coarse gravel, moderately bonded, 1-inch thick gravel layer at 10.5 ft, (Fill). PID Reading = 0.0 ppm Note: Drilling action indicates possible boulder at 11.2 ft.	G#779699 WC=14% LL=26 PL=17 A-4, CL
							188			
							91			
							177			
							56			
15	5D	24/19	15.0 - 17.0	18/15/15/15	30	47	65	214.1	15.0 — Olive, wet, hard, SILT, little fine sand, trace medium and coarse sand, non-plastic, loosely bonded, 1-inch thick gravel layer at 15.5 ft, alternating silt/fine sand layers, (Glaciolacustrine Deposit).	G#779700 A-4, ML
							43			
							40			
							58			
							40			
20	6D	24/18	20.0 - 22.0	3/6/8/11	14	22	57	209.1	20.0 — Grey to dark grey and black, wet, very stiff, lean CLAY, trace fine and medium sand, little organic material, varved, marine odor, (Glaciolacustrine Deposit).	G#779701 WC=22% LL=26 PL=18 A-4, CL
							56			
							44			
							51			
25							68			

Remarks:

1. Photoionization Detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm)
2. Hammer serial number 377

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-101
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	229.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/24/2024-5/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	111+7.7, 6.9 RT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in. ID	**Water Level*:**	11.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/18	25.0 - 27.0	11/12/14/17	26	41	40		Olive-grey, wet, hard, SILT, some fine sand, trace medium and coarse sand and fine and coarse gravel and lean clay, non-plastic, well bonded, (Glacial Till).	
							37			
							45			
							63			
							48			
30	8D	24/24	30.0 - 32.0	21/29/33/32	62	98	62	199.1	30.0 — Olive-grey, wet, very dense, Silty fine to coarse SAND, little fine gravel, trace coarse gravel, non-plastic, well bonded, (Glacial Till).	G#779702 A-4, SM
							135			
							97			
							80			
							193			
35	9D	14/11	35.0 - 36.2	20/50/50(2")			155		Olive-grey, wet, very dense, Silty fine to coarse SAND, little fine gravel, trace coarse gravel, non-plastic, well bonded, (Glacial Till).	G#779702 A-4, SM
							130			
							323			
							360/6 RC			
40	10D	10/8	40.0 - 40.8	43/50(4")					Olive-grey, wet, very dense, fine to coarse SAND, some silt, little fine gravel, trace lean clay, non-plastic, well bonded, (Glacial Till). Note: Probable boulders/cobbles 40.8-45.0 ft. Roller cone advanced 40.8-43.0 ft. NQ core 43.0-45.0 ft.	
							NQ		Note: Telescoped NW casing to 43 ft for coring. Removed to continue sampling.	
45	11D	21/10	45.0 - 46.8	18/33/49/50(3")	82	130	84		Olive-grey, wet, very dense, fine to coarse SAND, some silt and fine gravel, trace coarse gravel, non-plastic, well bonded, (Glacial Till).	G#779703 A-1-b, SM
							62			
							83			
							82			
50							78			

Remarks:

1. Photoionization Detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm)
2. Hammer serial number 377

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-101
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	229.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/24/2024-5/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	111+7.7, 6.9 RT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in. ID	**Water Level*:**	11.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	12D	24/10	50.0 - 52.0	17/39/48/50	87	138	79			Sample 12D similar in description to Sample 11D	G#779703 A-1-b, SM
							64			Note: Material too dense to drive HW casing. Driller drove SW to 10 ft. Telescope 4 in. after cleanout.	
							80				
							120				
							112				
55							RC			Note: Driller miscounted length of rods and mistakenly cleaned out to 65 ft. No samples collected from 52-65 ft. Driller noted drilling conditions changed at 60 ft. Less gravel noted in wash.	
60								169.1		60.0	
65	13D	24/24	65.0 - 67.0	18/20/35/39	55	87				Grey, wet, hard, SILT, some lean clay, trace fine gravel and fine sand, slightly-plastic, very well bonded, (Glacial Till).	
70	14D	24/24	70.0 - 72.0	25/27/44/43	71	112	60			Grey, wet, hard, SILT, trace fine and medium and coarse sand and fine gravel, very well bonded, (Glacial Till).	G#779704 A-4, ML
							71				
							105				
							75				
75							81				

Remarks:

1. Photoionization Detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm)
2. Hammer serial number 377

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-101
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	229.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/24/2024-5/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	111+7.7, 6.9 RT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in. ID	**Water Level*:**	11.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	15D	24/24	75.0 - 77.0	17/27/40/40	67	106	77			Brown and grey, wet, hard, SILT, trace fine and medium and coarse sand and fine gravel, non-plastic, well bonded, (Glacial Till).	G#779704 A-4, ML
							80				
							71				
							74				
							78			Note: Roller cone advanced through probable cobble at 79 ft.	
80	16D	17/14	80.0 - 81.4	26/39/50(5")			75			Grey-brown, wet, hard, SILT, trace fine and medium and coarse sand and fine gravel, non-plastic, very well bonded, (Glacial Till).	G#779704 A-4, ML
							68				
							70				
							74				
							69				
85	17D	16/12	85.0 - 86.3	39/49/50(4")			84			Brown and grey, wet, hard, SILT, little fine gravel, trace fine and medium and coarse sand and coarse gravel and lean clay, non-plastic, very well bonded, (Glacial Till).	
							78	142.3		Note: Telescoped NW casing to 90 ft to keep borehole open. HW casing from 0-70 ft. NW casing blows from 70-90 ft were in pre-drilled borehole.	
							80			86.8	
							71			Top of Bedrock at Elev. 142.3	
	R1	31/19	89.0 - 91.6	RQD = 38%			75			Note: Roller cone advanced used through probable bedrock from 86.8-89.0 ft.	
90							NQ			R1: Bedrock: Grey, aphanitic to fine grained, QUARTZITE, fresh, hard, primary joints low angle to moderate dipping, spaced very close to close, tight to open, fresh, one horizontal secondary joint, open, quartz stringers throughout.	
	R2	13/10	91.6 - 92.7	RQD = 0%						VASSALBORO FORMATION	
	R3	10/6	92.7 - 93.5	RQD = 0%						Rock Quality = Poor	
	R4	60/55	93.5 - 98.5	RQD = 81%						Recovery = 61%	
										R1: Core Times (min:sec):	
										89-90 ft (0:55)	
										90-91 ft (1:36)	
										R2: Similar to R1, except highly fractured at 91.9-92.4 ft (siltstone), no horizontal joints.	
95										VASSALBORO FORMATION	
										Rock Quality = Very Poor	
										Recovery = 77%	
										R2: Core Times (min:sec):	
										91-92 ft (1:09)	
										R3: Bedrock: Grey, aphanitic to fine grained, QUARTZITE, fresh, hard, primary joints high angle to vertical, spaced very close, tight to open, one horizontal secondary joint open, quartz stringers throughout.	
								130.6		VASSALBORO FORMATION	
100											

Remarks:

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2. Hammer serial number 377

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-101
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	229.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/24/2024-5/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	111+7.7, 6.9 RT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in. ID	**Water Level*:**	11.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140 lb. Hammer
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WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100										Rock Quality = Very Poor Recovery = 60% R3: Core Times (min:sec): 92-93 ft (0:55) 93-94 ft (1:15); R4: Bedrock: Grey, aphanitic to fine grained, QUARTZITE, fresh, hard, primary joints low angle, close, tight, one horizontal secondary joint, open, one vertical fracture from 93.5-93.8 ft, quartz stringers throughout. VASSALBORO FORMATION Rock Quality = Good Recovery = 91% R4: Core Times (min:sec): 94-95 ft (2:17) 95-96 ft (2:28) 96-97 ft (1:55) 97-98 ft (1:55) 98-98.5 ft (1:06)	
105											
110											
115											
120											
125											

98.5

Bottom of Exploration at 98.5 feet below ground surface.

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-102
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	217.6	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/2024-6/5/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	112+44.3, 9.0 LT	**Casing ID/OD:**	HW/NW-4.0/3.0 in. ID	**Water Level*:**	5.8
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/18	0.0 - 2.0	2/5/7/7	12	19	HSA		Grey-brown, moist, very stiff, Sandy SILT, trace fine gravel, non-plastic, no sheen, no staining, no odor, moderately bonded, (Glacial Till). PID Reading = 13.4 ppm	
								215.6	2.0	
	2D	24/14	2.0 - 4.0	7/8/14/13	22	35			Grey-brown, moist, hard, SILT, trace fine and medium and coarse sand and fine and coarse gravel, non-plastic, no sheen, no staining, no odor, moderately bonded, (Glacial Till). PID Reading = 10.3 ppm	
5								212.6	5.0	
	3D	24/14	5.0 - 7.0	10/12/17/19	29	46			Grey-brown, moist, hard, Sandy CLAY, little fine gravel, no sheen, no staining, no odor, moderately bonded, (Glacial Till). PID Reading = 27.2 ppm	G#779705 WC=10% LL=23 PL=15 A-4, CL
10								207.6	10.0	
	4D	24/20	10.0 - 12.0	12/19/20/22	39	62			Grey-brown, wet, very dense, Silty-Clayey fine to medium SAND, little fine gravel, trace coarse sand and coarse gravel, non-plastic, no sheen, no staining, no odor, moderately bonded, (Glacial Till). PID Reading = 11.1 ppm	G#779706 WC=9% LL=20 PL=13 A-4, SC-SM
15	5D	24/12	15.0 - 17.0	8/12/19/24	31	49	72		Sample 5D similar in description to Sample 4D, except dense, (Glacial Till).	G#779706 WC=9% LL=20 PL=13 A-4, SC-SM
							69			
							74			
							80			
							84			
20								197.7	19.9	
	6D	5/5	20.0 - 20.4	50(5")			78	197.2	Drilling change at 19.9 ft. Difficult drilling noted by driller. Grey-brown, wet, very dense, fine to coarse SAND, some fine gravel, little silt, trace coarse gravel, non-plastic, well-graded, cobble in spoon tip, (Glacial Till). 20.4	
							89			
							69			
							70			
25							64			

Remarks:

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2. Hammer serial number 377

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-102
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	217.6	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/2024-6/5/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	112+44.3, 9.0 LT	**Casing ID/OD:**	HW/NW-4.0/3.0 in. ID	**Water Level*:**	5.8
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/14	25.0 - 27.0	19/17/19/10	36	57	55		Olive-brown, wet, very dense, fine to coarse SAND, some clay/ silt and fine gravel, trace coarse gravel, well bonded, (Glacial Till).	G#779707 WC=9% LL=21 PL=16 A-2-4, SC-SM
							62			
							65			
							69			
							80			
30								187.6	30.0	
	8D	11/8	30.0 - 30.9	67/50(5")			320		Olive-brown, wet, very dense, fine to coarse SAND, some silt and fine gravel, trace coarse gravel, well-graded, well bonded, (Glacial Till).	G#779708 A-2-4, SM
							215			
							203			
							182			
							150			
35	9D	10/10	35.0 - 35.8	40/50(4")			RC		Olive-brown, wet, very dense, fine to coarse SAND, some silt and fine gravel, trace coarse gravel, no structure, well bonded, (Glacial Till).	G#779708 A-2-4, SM
40								177.6	40.0	
	10D	24/16	40.0 - 42.0	20/25/28/22	53	84			Grey, wet, hard, SILT, little fine gravel, trace fine and medium and coarse sand and coarse gravel, non-plastic, well bonded, (Glacial Till).	G#779709 A-4, ML
45	11D	24/18	45.0 - 47.0	12/21/25/33	46	73			Grey, wet, hard, SILT, little fine gravel, trace fine and medium and coarse sand and coarse gravel, non-plastic, well bonded, (Glacial Till).	G#779709 A-4, ML
50										

Remarks:

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Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-102
WIN: 025315.00

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Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/2024-6/5/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	112+44.3, 9.0 LT	**Casing ID/OD:**	HW/NW-4.0/3.0 in. ID	**Water Level*:**	5.8
Hammer Efficiency Factor:	0.949	**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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	12D	24/22	50.0 - 52.0	32/17/32/31	49	78	RC		Grey, wet, hard, SILT, little fine gravel, trace fine and medium and coarse sand and coarse gravel, non-plastic, well bonded, (Glacial Till).	G#779709 A-4, ML
55	13D	24/22	55.0 - 57.0	25/20/27/31	47	74			Sample 13D similar in description to Sample 12D	
60	14D	24/20	60.0 - 62.0	29/21/25/25	46	73			Grey, wet, hard, SILT, trace fine and medium sand, slightly plastic, well bonded, (Glacial Till).	
65	15D	24/24	65.0 - 67.0	22/20/26/32	46	73			Sample 15D similar in description to Sample 14D	
								148.6	69.0 — Difficult drilling noted by driller from 69.0-70.3 ft.	
70	16D	3/0	70.0 - 70.3	50(3")			NQ	147.3	No Recovery — 70.3 — Top of Bedrock at Elev. 147.3 Note: Roller cone advanced through probable bedrock from 70.3-75.2 ft. Drilling action and wash water indicated fractured bedrock with soil filled fractures from 70.3-73.3 ft. NW casing telescoped 35.0-75.2 ft prior to coring.	
75										

Remarks:

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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.
75	R1	26/23	75.2 - 77.4	RQD = 31%			NQ			R1: Bedrock: Grey, aphanitic, PHYLLITE, fresh to slightly weathered, hard. Primary joints dipping horizontal to low angles, spaced extremely close to moderately close, smooth and planar or rough and stepped, fresh to discolored, tight to open. Secondary joints dipping at moderate to high angles, spaced close to moderately close, smooth and planar or rough and stepped, tight to open. Few calcite stringers. Slight iron staining on few joint surfaces. VASSALBORO FORMATION Rock Quality = Poor Recovery = 88% R1: Core Times (min:sec); 75-76 ft (2:00) 76-77 ft (2:10)	
	R2	17/17	77.4 - 78.8	RQD = 24%						R2: Bedrock: Grey, aphanitic, PHYLLITE, fresh to slightly weathered, hard. Primary joint dipping at moderate to high angles, spaced extremely close to close, smooth and planar, fresh to discolored, tight to open. One low angle secondary joint at 77.8 ft, smooth and planar, open. Few calcite stringers.	
	R3	26/26	78.8 - 81.0	RQD = 42%						VASSALBORO FORMATION Rock Quality = Very Poor Recovery = 91% R2: Core Times (min:sec): 77-78 ft (1:30) 78-79 ft (1:30) R3: Bedrock: Grey, aphanitic, PHYLLITE, fresh to slightly weathered, hard. Primary joints dipping at moderate to high angles, spaced very close, smooth and planar, fresh to discolored, tight. Drilling breaks 80.2-81.0 ft. VASSALBORO FORMATION Rock Quality = Poor Recovery = 100% R3: Core Times (min:sec): 79-80 ft (2:00) 80-81 ft (2:00)	
80											
	R4	38/38	81.0 - 84.2	RQD = 29%						R4: Bedrock: Grey, aphanitic, PHYLLITE, fresh to slightly weathered, hard. Primary joints dipping at horizontal to low angles, spaced extremely close to close, smooth and stepped to rough and undulating, fresh to discolored. Secondary joints dipping at moderate to high angles, spaced close to moderately close, smooth and stepped, fresh to discolored. Few calcite stringers. Iron staining on few joint surfaces. Weathered fractured zone at 82.6 ft with pits present. White quartzite inclusion 84.0-84.2 ft. VASSALBORO FORMATION Rock Quality = Poor Recovery = 100% R4: Core Times (min:sec): 81-82 ft (2:00) 82-83 ft (2:30)	
								133.4			
85											
90											
95											
100										84.2 **Bottom of Exploration at 84.2 feet below ground surface.**	

Remarks:

1. Photoionization Detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm)
2. Hammer serial number 377

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-103
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	219.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey/H. Hollauer	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/31/2024-6/6/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	113+89.7, 29.4 LT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in.	**Water Level*:**	2.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.0 - 2.0	1/1/5/6	6	9	HSA		Dark brown, moist, stiff, SILT, some fine sand, trace medium and coarse sand, roots throughout, organic odor, (Fill). PID Reading = 0.0 ppm	
								217.1	2.0	
	2D	24/10	2.0 - 4.0	4/6/9/6	15	24			Grey-brown, moist, medium dense, Silty-Clayey fine to medium SAND, little coarse gravel, trace coarse sand and coarse gravel, non-plastic, moderately bonded, (Glacial Till). PID Reading = 0.0 ppm	G#779710 WC=13% LL=24 PL=17 A-4, SC-SM
	3D	24/13	4.0 - 6.0	4/5/5/7	10	16			Grey-brown, moist, medium dense, Silty-Clayey fine to medium SAND, little coarse gravel, trace coarse sand and coarse gravel, non-plastic, moderately bonded, (Glacial Till). PID Reading = 0.0 ppm	G#779710 WC=13% LL=24 PL=17 A-4, SC-SM
5										
10								209.1	10.0	
	4D	24/17	10.0 - 12.0	5/20/19/22	39	62			Grey-brown, wet, very dense, Silty fine to medium SAND, little fine and coarse gravel, trace coarse sand, non-plastic, loose to moderately bonded, poorly-graded, (Glacial Till). PID Reading = 0.0 ppm	G#779711 A-4, SM
							90			
							110			
15								204.1	15.0	
	5D	24/14	15.0 - 17.0	7/9/20/21	29	46	95		Grey, wet, hard, SILT, little fine sand, trace medium and coarse sand and fine and coarse gravel and lean clay, moderately bonded, (Glacial Till).	
							80			
							65			
							92			
							78			
20								199.1	Note: Probable cobble at 19.8 ft. 20.0	
	6D	24/12	20.0 - 22.0	18/21/16/18	37	59	106		Grey-brown, wet, very dense, fine to coarse SAND, some silt, little fine and coarse gravel, loose to moderately bonded, poorly-graded, (Glacial Till).	G#779712 A-2-4, SM
							94			
							72			
							91			
25							84			

Remarks:

1. Photoionization Detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm)
2. HW casing broken at 30.0 ft. HW casing from 30.0-45.0 ft unable to be removed upon completion of drilling.
3. Hammer serial number 377

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-103
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	219.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey/H. Hollauer	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/31/2024-6/6/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	113+89.7, 29.4 LT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in.	**Water Level*:**	2.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/12	25.0 - 27.0	20/25/43/38	68	108	80		Grey-brown, wet, very dense, fine to coarse SAND, some silt, little fine and coarse gravel, loosely bonded, poorly-graded, (Glacial Till).	G#779712 A-2-4, SM
							65			
							120			
							128			
							150			
30	8D	5/3	30.0 - 30.4	50(5")			RC	189.1	30.0 — Grey, wet, very dense, fine GRAVEL, trace medium and coarse sand and coarse gravel, poorly-graded, (Glacial Till). Note: Probable boulder from 30.4-33.4 ft.	
35	9D	2/2	35.0 - 35.2	50(2")			30		Grey, wet, very dense, fine GRAVEL, little coarse gravel, trace fine and medium and coarse sand, poorly graded, (Glacial Till).	
							103			
							77			
							55			
							60			
40	10D	24/9	40.0 - 42.0	15/28/35/43	63	100	67	179.1	40.0 — Olive-grey, wet, hard, SILT, little fine sand and fine gravel, trace medium and coarse sand, non-plastic, moderately bonded, (Glacial Till).	G#779713 A-4, ML
							89			
							74			
							88			
							80			
45	11D	24/18	45.0 - 47.0	26/37/48/44	85	134	RC		Olive-grey, wet, hard, SILT, little fine sand and fine gravel, trace medium and coarse sand, well bonded, (Glacial Till).	G#779713 A-4, ML
								171.6	47.5 — Note: Likely strata change based on drilling action.	
50										

Remarks:

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3. Hammer serial number 377

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-103
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	219.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey/H. Hollauer	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/31/2024-6/6/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	113+89.7, 29.4 LT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in.	**Water Level*:**	2.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	12D	24/24	50.0 - 52.0	11/17/29/27	46	73	RC		Grey, wet, hard, lean CLAY, trace fine and medium and coarse sand, moderately plastic, well bonded, (Glacial Till).	G#779714 WC=19% LL=31 PL=15 A-6, CL
								165.1	54.0	
55	13D	24/24	55.0 - 57.0	17/22/36/37	58	92			Grey, wet, hard, SILT, little fine sand, trace medium and coarse sand and fine gravel, slightly plastic, well bonded, (Glacial Till).	
								160.1	59.0	
60									Top of Bedrock at Elev. 160.1 Note: NW casing telescoped 45.0-59.0 ft prior to coring. Note: Roller cone advanced through probable bedrock from 59.0-63.0 ft.	
	R1	32/28	63.0 - 65.7	RQD = 31%			NQ		R1: Bedrock: Light grey, aphanitic, QUARTZITE, fresh to slightly weathered, hard. Primary joins dipping at horizontal to low angles, spaced very close to close, smooth and planar, mostly fresh, open. Secondary joints dipping at high to vertical angles, spaced moderately, rough and undulating, mostly fresh, open. Possible iron and maganese staining on few surfaces. VASSALBORO FORMATION Rock Quality = Poor Recovery = 88% R1: Core Times (min:sec): 63-64 ft (1:30) 64-65 ft (2:00) 65-66 ft (1:30)	
65	R2	36/36	65.7 - 68.7	RQD = 47%						
	R3	28/28	68.7 - 71.0	RQD = 0%						
70	R4	24/22	71.0 - 73.0	RQD = 75%					R2: Bedrock: Light grey, aphanitic, QUARTZITE, fresh to slightly weathered, hard. Primary joints dipping at horizontal to low angles, spaced close, smooth and planar to rough and undulating, mostly fresh, open. Secondary joints dipping at high angles, spaced moderately, smooth and planar, mostly fresh, tight to open. Weathered joint at 66.2 ft. Few calcite stringers. Maganese staining on few surfaces. Calcite inclusion at 68.1 ft. VASSALBORO FORMATION Rock Quality = Poor Recovery = 100% R2: Core Times (min:sec): 66-67 ft (1:30)	
	R5	18/20	73.0 - 74.5	RQD = 94%						
75								144.6		

Remarks:

1. Photoionization Detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm)
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3. Hammer serial number 377

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-103
WIN: 025315.00

Driller:	M. Nadeau	**Elevation (ft.)**	219.1	**Auger ID/OD:**	HSA 2.5" ID
Operator:	Northern Test Boring, Inc.	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	T. Ramey/H. Hollauer	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/31/2024-6/6/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2 in. ID
Boring Location:	113+89.7, 29.4 LT	**Casing ID/OD:**	SW/HW/NW-6.0/4.0/3.0 in.	**Water Level*:**	2.2
Hammer Efficiency Factor:	0.949	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										67-68 ft (2:00) R3: Bedrock: Light grey, aphanitic, QUARTZITE, fresh to slightly weathered, hard. Primary joints dipping low to moderate angles, spaced very close to close, smooth and planar, fresh, open. Secondary joints dipping at high to vertical angles, spaced close to moderately close, smooth and planar, fresh, tight to open. Manganese staining on most surfaces. Calcite accretion on one high angle joint surface. VASSALBORO FORMATION Rock Quality = Very Poor Recovery = 100% R3: Core Times (min:sec): 68-69 ft (1:45) 69-70 ft (1:45) 70-71 ft (2:30) 65-66 ft (1:30) R4: Bedrock: Light grey, aphanitic, QUARTZITE, fresh to slightly weathered, hard. One compound joint with both low and high angle joints, few calcite stringers throughout. VASSALBORO FORMATION Rock Quality = Fair Recovery = 92% R4: Core Times (min:sec): 71-72 ft (2:00) 72-73 ft (2:10) R5: Bedrock: Light grey, aphanitic, QUARTZITE, fresh to slightly weathered, hard. Two high angle joints spaced moderately, smooth and planar, tight to open, fresh. Manganese staining on surfaces. VASSALBORO FORMATION Rock Quality = Excellent Recovery = 111% R5: Core Times (min:sec): 73-74 ft (1:45) 74-74.5 ft (0:30) 74.5 **Bottom of Exploration at 74.5 feet below ground surface.**	
80											
85											
90											
95											
100											

Remarks:

1. Photoionization Detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm)
2. HW casing broken at 30.0 ft. HW casing from 30.0-45.0 ft unable to be removed upon completion of drilling.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-201
WIN: 025315.00

Driller:	B. Enos	**Elevation (ft.)**	211.0	**Auger ID/OD:**	N/A
Operator:	New England Boring Contractors	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Estes	**Rig Type:**	Mobile Drill B-53 #23	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/29/2024-10/29/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	114+45.2, 138.7 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	3.6
Hammer Efficiency Factor:	0.834	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
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PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/9	0.0 - 2.0	2/5/9/10	14	19	PUSH		Olive-brown, moist, medium dense, Silty fine to coarse SAND, little fine gravel, loosely bonded, (Glacial Till).	
								209.0	2.0	
	2D	24/18	2.0 - 4.0	7/7/8/9	15	21			Olive-brown, moist, stiff, fine Sandy SILT, little fine gravel, slightly mottled, loosely bonded, slightly plastic, (Glacial Till).	
	3D	24/8	4.0 - 6.0	3/8/10/12	18	25	10		Similar to 2D description	
5							40			
							82			
							100			
							120			
	4D	24/11	9.0 - 11.0	14/19/15/13	24	33	88		Olive-brown to grey-brown, wet, hard, fine Sandy SILT, little fine to coarse gravel, loosely to moderately bonded, slightly plastic, (Glacial Till). Note: Washed ahead of casing from 9.0 to 14.0 ft.	
10							40			
							45			
							49			
							53		Note: No recovery with 2.5 in. standard split spoon. Drove 3 in. split spoon for 11 in. of recovery.	
	5D	24/0	14.0 - 16.0	26/26/25/31	51	71	69		Grey-brown, wet, hard, fine Sandy SILT, little fine to coarse gravel, moderately bonded, slightly plastic, (Glacial Till). Note: Encountered possible boulder at 16.7 ft. Note: Washed ahead of casing from 16.7 to 19.0 ft.	
15							93			
							165 /0.7			
							75			
							82			
								192.0	19.0	
	6D	16/14	19.0 - 20.3	19/23/50(4")			20		Grey-brown to olive-brown, wet, very dense, Silty-Clayey fine to coarse SAND, some fine to coarse gravel, few cobbles and boulders, moderately bonded, (Glacial Till). Note: Split spoon sample refusal at 20.3 ft on possible boulder. Note: Washed ahead of casing from 19.0 to 24.0 ft.	
20							39			
							52			
							62			
							85			
25	7D	24/12	24.0 - 26.0	11/11/15/18	26	36	100		Grey-brown, wet, medium dense, fine to coarse SAND, some fine to coarse gravel, some clay-silt, loosely bonded, few cobbles,	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-201
WIN: 025315.00

Driller:	B. Enos	**Elevation (ft.)**	211.0	**Auger ID/OD:**	N/A
Operator:	New England Boring Contractors	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Estes	**Rig Type:**	Mobile Drill B-53 #23	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/29/2024-10/29/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	114+45.2, 138.7 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	3.6
Hammer Efficiency Factor:	0.834	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							78			(Glacial Till). Note: Washed ahead of casing from 24.0 to 29.0 ft.	
							80				
							90				
							115				
	8D	24/20	29.0 - 31.0	26/40/63/49	103	143	RC			Grey-brown to olive-brown, wet, very dense, fine to coarse SAND, some fine to coarse gravel, some clay-silt, loosely bonded, few cobbles, (Glacial Till).	
30											
								177.0		34.0	
	9D	24/24	34.0 - 36.0	9/19/26/32	45	63				Grey-brown, wet, hard, SILT, some fine to coarse sand, trace fine to coarse gravel, moderately bonded, slightly plastic, (Glacial Till).	
35								175.0		36.0 **Bottom of Exploration at 36.0 feet below ground surface.**	
40											
45											
50											

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-202
WIN: 025315.00

Driller:	B. Enos	**Elevation (ft.)**	210.9	**Auger ID/OD:**	N/A
Operator:	New England Boring Contractors	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Estes	**Rig Type:**	Mobile Drill B-53 #23	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/29/2024-10/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	115+48.7, 114.2 RT	**Casing ID/OD:**	HW/NW-4.0/3.0 in. ID	**Water Level*:**	3.2
Hammer Efficiency Factor:	0.834	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.0 - 2.0	2/4/9/10	13	18	PUSH			Olive-brown, dry, stiff, SILT, some fine to coarse sand, little fine to coarse gravel, trace organics and roots, (Glacial Till).	
	2D	24/18	2.0 - 4.0	8/9/16/31	15	21				Olive-brown, moist, very stiff, SILT, some fine to coarse sand, little fine to coarse gravel, slightly plastic, loosely bonded, (Glacial Till).	
	3D	24/20	4.0 - 6.0	8/12/13/13	25	35	46			Similar to 2D description	
5							74				
							80				
							86			Note: Drilling indicated increased gravel content at 7.5 ft.	
							120				
	4D	24/8	9.0 - 11.0	17/14/11/14	25	35	29			Olive-brown to grey-brown, wet, very stiff, SILT, some fine to coarse sand, some fine to coarse gravel, slightly plastic, moderately bonded, (Glacial Till). Note: Washed ahead of casing from 9.0 to 14.0 ft.	
10							31				
							80				
							57				
							58				
	5D	24/14	14.0 - 16.0	5/7/6/14			49			Grey-brown to grey, wet, stiff, SILT, some fine to coarse sand, little fine to coarse gravel, pockets of medium to coarse sand and clay-silt, slightly plastic, moderately bonded, (Glacial Till). Note: Washed ahead of casing from 14.0 to 19.0 ft.	
15							54				
							59				
							67				
							84				
	6D	6/3	19.0 - 19.5	75			PUSH			Similar to 5D description Note: Split spoon sample refusal at 19.5 ft on possible boulder. Note: Washed ahead through possible boulder, switched to NW casing and pulled HW casing out. Note: Washed ahead of casing from 19.0 to 24.0 ft.	
20											
							47				
							62				
								186.9		24.0	
25	7D	24/11	24.0 - 26.0	9/10/10/14	20	28	6			Olive-brown to grey-brown, wet, very stiff, Sandy SILT, little	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-202
WIN: 025315.00

Driller:	B. Enos	**Elevation (ft.)**	210.9	**Auger ID/OD:**	N/A
Operator:	New England Boring Contractors	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Estes	**Rig Type:**	Mobile Drill B-53 #23	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/29/2024-10/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	115+48.7, 114.2 RT	**Casing ID/OD:**	HW/NW-4.0/3.0 in. ID	**Water Level*:**	3.2
Hammer Efficiency Factor:	0.834	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							6			fine to coarse gravel, slightly pastic, moderately bonded, (Glacial Till). Note: Washed ahead of casing from 24.0 to 29.0 ft.	
							15				
							25				
							28				
	8D	24/12	29.0 - 31.0	12/12/14/18	26	36	4			Grey-brown, wet, very stiff, fine to coarse Sandy SILT, little fine to coarse gravel, slightly plastic, moderately bonded with frequent seams and layers of fine to coarse sand, well-graded, (Glacial Till). Note: Washed ahead of casing from 29.0 to 34.0 ft.	
30							31				
							32				
							23				
							47				
								176.9		34.0	
	9D	24/24	34.0 - 36.0	19/24/28/33	52	72				Olive-brown to grey-brown, wet, hard, SILT, little fine to coarse sand, trace fine gravel, slightly plastic, moderately to well bonded, (Glacial Till).	
35								174.9		36.0	
										Bottom of Exploration at 36.0 feet below ground surface.	
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-203
WIN: 025315.00

Driller:	B. Enos	**Elevation (ft.)**	208.9	**Auger ID/OD:**	N/A
Operator:	New England Boring Contractors	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Estes	**Rig Type:**	Mobile Drill B-53 #23	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/30/2024-10/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	116+52.3, 97.0 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	0.0
Hammer Efficiency Factor:	0.834	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/1	0.0 - 2.0	WOR/WOH/WOH/ WOH			PUSH	208.1	Dark brown, wet, PEAT, (Swamp Deposit). 0.8	
									Brown, wet, very soft, SILT, trace fine sand, trace organic material and roots, (Swamp Deposit).	
	2D	24/15	2.0 - 4.0	1/4/10/16	14	19		206.6	2.3 Olive-brown to grey-brown, mottled, wet, stiff to very stiff, fine to coarse Sandy SILT, trace fine to coarse gravel, slightly plastic, moderately bonded, (Glacial Till).	
	3D	24/9	4.0 - 6.0	8/9/6/6	15	21	6		Olive-brown, wet, stiff, fine to coarse Sandy SILT, little fine to coarse gravel, slightly plastic, moderately bonded, (Glacial Till). Note: Washed ahead of casing from 4.0 to 9.0 ft.	
5							7			
							17			
							44			
							34	199.9	9.0 Olive-brown to grey-brown, wet, very stiff, SILT, some fine to coarse sand, little fine to coarse gravel, with a fine sandy silt layer, slightly plastic, moderately bonded, (Glacial Till). Note: Washed ahead of casing from 9.0 to 14.0 ft.	
	4D	24/11	9.0 - 11.0	9/12/7/11	19	26	18			
10							16			
							20			
							18			
							23	194.9	14.0 Olive-brown to grey-brown, wet, very stiff, fine to coarse Sandy SILT, little fine to coarse gravel, slightly plastic, moderately bonded, (Glacial Till). Note: Washed ahead of casing from 14.0 to 19.0 ft.	
	5D	24/15	14.0 - 16.0	10/9/10/11	19	26	8			
15							52			
							81			
							85			
							98	189.9	19.0 Grey-brown, wet, hard, fine to coarse Gravelly SILT, some fine to coarse sand, slightly plastic, loosely to moderately bonded, (Glacial Till). Note: Washed ahead of casing from 19.0 to 24.0 ft.	
	6D	24/8	19.0 - 21.0	15/19/17/21	36	50	30			
20							46			
							64			
							89			
							90	184.9	24.0 Grey-brown, wet, very dense, Clayey-Silty fine to coarse	
25	7D	24/8	24.0 - 26.0	28/23/26/16	49	68	35			

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 100 Bridge No. 5957 Over I-95
Location: Palmyra, Maine

Boring No.: BB-PI95-203
WIN: 025315.00

Driller:	B. Enos	**Elevation (ft.)**	208.9	**Auger ID/OD:**	N/A
Operator:	New England Boring Contractors	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Estes	**Rig Type:**	Mobile Drill B-53 #23	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/30/2024-10/30/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	116+52.3, 97.0 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	0.0
Hammer Efficiency Factor:	0.834	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

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							39			GRAVEL, some fine to coarse sand, slightly plastic, loosely to moderately bonded, (Glacial Till). Note: Washed ahead of casing from 24.0 to 29.0 ft.	
							36				
							62			Note: Drill wash indicated change at 28 ft	
							86	180.9		28.0	
	8D	6/6	29.0 - 29.5	19/50(0")			RC			Brown, wet, hard, fine to coarse Sandy SILT, little fine to coarse gravel, slightly plastic, moderately to well bonded, (Glacial Till). Note: Standard split spoon sample refusal at 29.5 ft on cobble or possible bedrock.	
30											
								177.1		31.8	
										Note: Drilling indicates possible moderately weathered bedrock at 31.8 ft.	
								174.8		Light grey to grey, possible moderately weathered bedrock, trace clay	
	9D	1/1	34.0 - 34.1	50(1")						34.1	
35										**Bottom of Exploration at 34.1 feet below ground surface.**	
40											
45											
50											

Remarks:

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BEDROCK CORE PHOTOGRAPHS
ROUTE 100 BRIDGE NO. 5957 OVER INTERSTATE 95
MAINEDOT WIN 025315.00
ROUTE 100, PALMYRA, MAINE

Top Row: BB-PI95-101, Run No. R1: 89.0 ft (left) to 91.6 ft (middle-left); BB-PI95-101, Run No. R2: 91.6 ft (middle) to 92.7 ft (middle); BB-PI95-101, Run No. R3: 92.7 ft (middle-left) to 93.5 ft (middle-right); BB-PI95-101, Run No. R4: 93.5 ft (middle-right) to approximately 94.5 ft (right)

Middle Upper Row: BB-PI95-101, Run No. R4 (continued): approximately 94.5 ft (left) to 98.5 ft (middle-right)

Middle Lower Row: BB-PI95-103, Run No. R1: 63.0 ft (left) to 65.7 ft (middle); BB-PI95-103, Run No. R2: 65.7 (middle) to approximately 68.0 ft (right)

Bottom Row: BB-PI95-103, Run No. R2 (continued): approximately 68.0 ft (left) to 68.7 ft (right); BB-PI95-103, Run No. R3: 68.7 ft (left) to 71.0 ft (middle-right); BB-PI95-103, Run No. R4: 71.0 ft (middle-right) to 73.0 ft (right)

BEDROCK CORE PHOTOGRAPHS
ROUTE 100 BRIDGE NO. 5957 OVER INTERSTATE 95
MAINEDOT WIN 025315.00
ROUTE 100, PALMYRA, MAINE

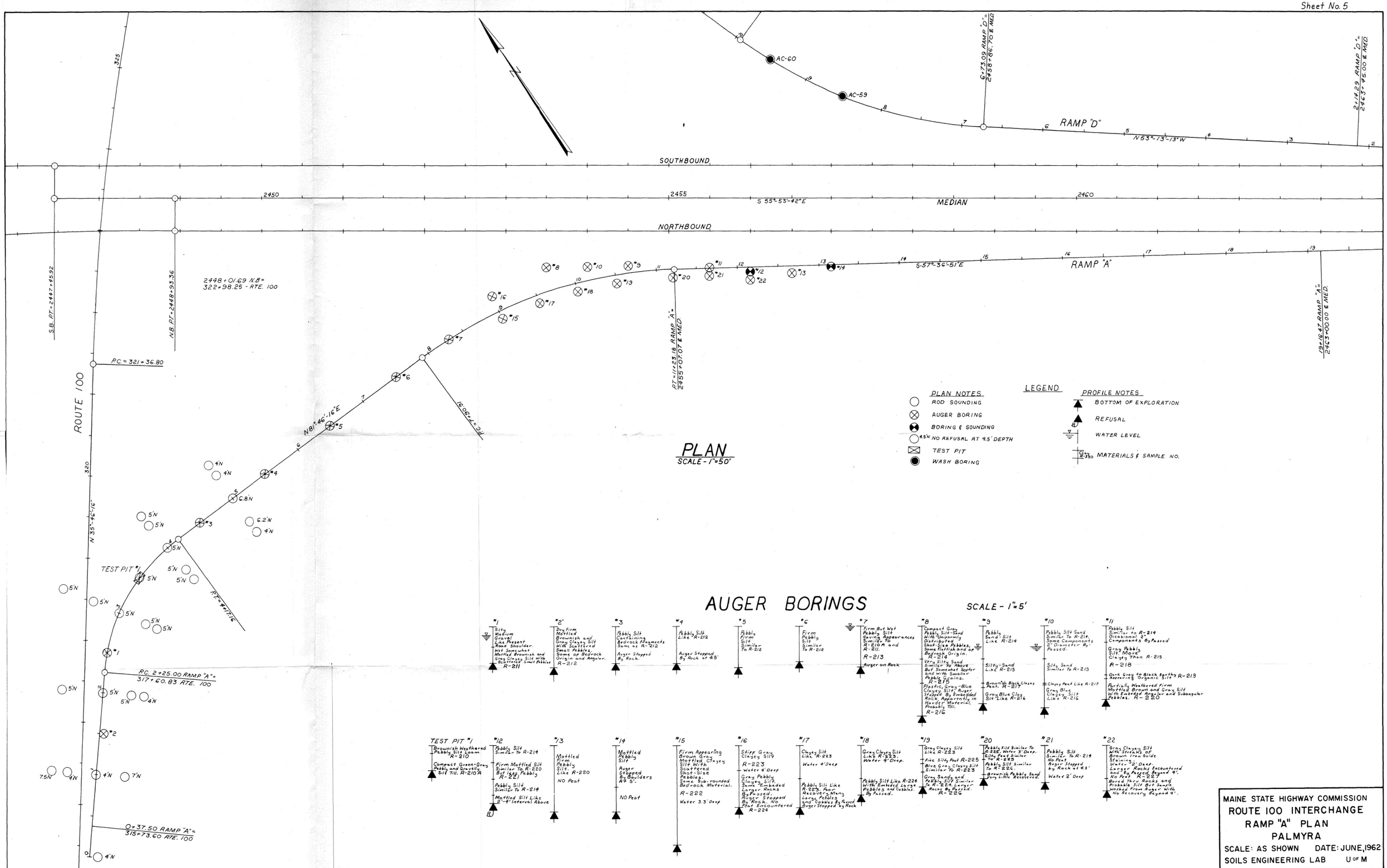
Top Row: BB-PI95-102, Run No. R1: 75.2 ft (left) to 77.4 ft (middle-left); BB-PI95-102, Run No. R2: 77.4 ft (middle-left) to 78.8 ft (middle-right); BB-PI95-102, Run No. R3: 78.8 ft (middle-right) to approximately 80.2 ft (right)

Middle Upper Row: BB-PI95-102, Run No. R3: approximately 80.2 ft (left) to 81.0 ft (right); BB-PI95-102, Run No. R4: 81.0 ft (left) to 84.2 ft (middle-right)

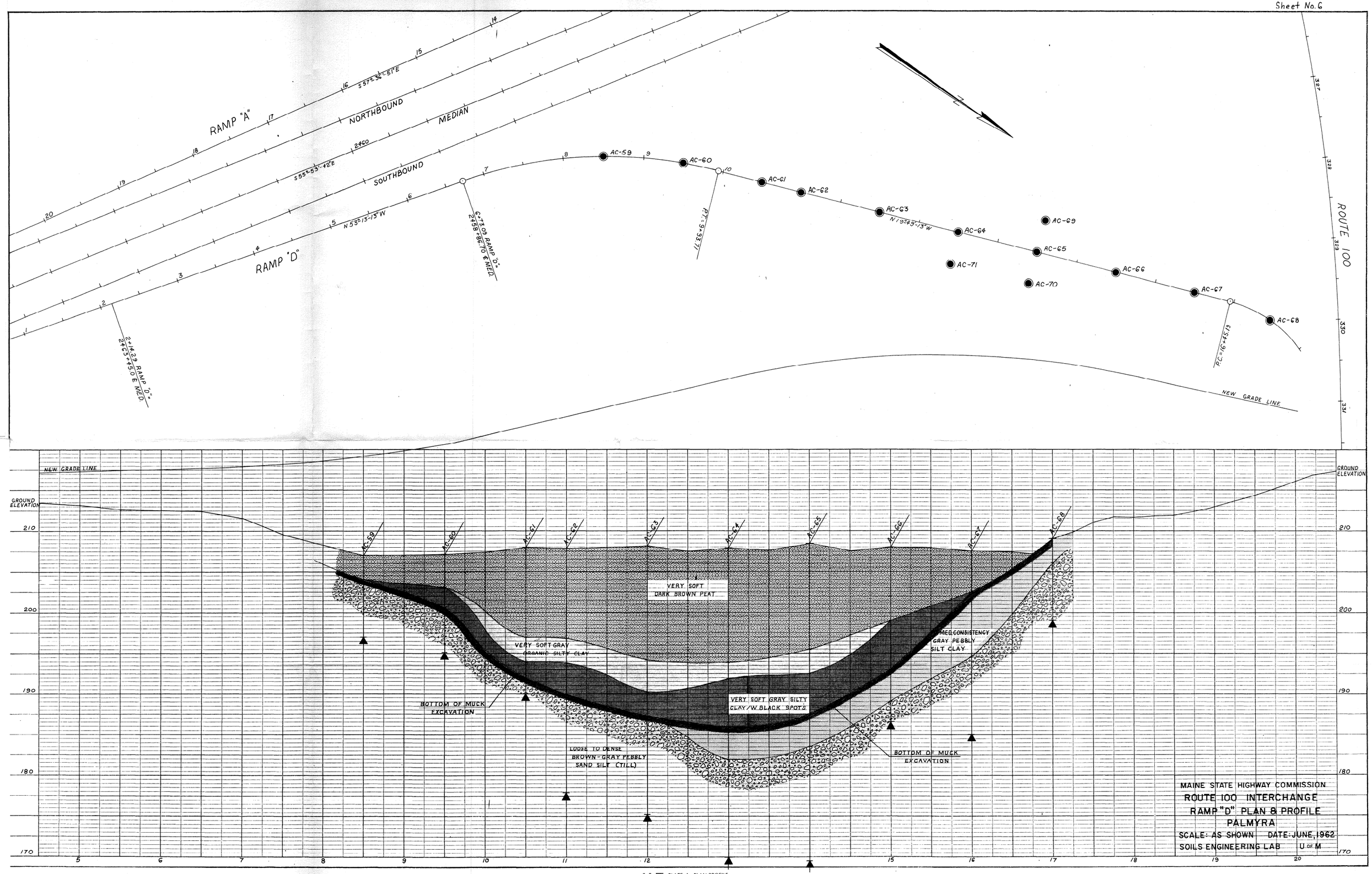
Middle Lower Row: Empty

Bottom Row: BB-PI95-103, Run No. R5: 73.0 ft (left) to 74.5 ft (middle-left)

Historic Boring Logs



MAINE STATE HIGHWAY COMMISSION
ROUTE 100 INTERCHANGE
RAMP "A" PLAN
PALMYRA
SCALE: AS SHOWN DATE: JUNE, 1962
SOILS ENGINEERING LAB U OF M


RAMP "A"
NORTHBOUND
MEDIAN
SOUTHBOUND
RAMP "D"
S 57°-36'-51" E
S 55°-53'-42" E
N 53°-13'-13" W
N 19°-45'-13" W
2460
2+14.29 RAMP "D"
2463+45.0 ℄ MED.
C+73.09 RAMP "D"
2465+86.70 ℄ MED.
P.T.=9+93.71
P.C.=16+45.19
AC-59
AC-60
AC-61
AC-62
AC-63
AC-64
AC-65
AC-66
AC-67
AC-68
AC-69
AC-70
AC-71
ROUTE 100
NEW GRADE LINE
GROUND ELEVATION
VERY SOFT DARK BROWN PEAT
VERY SOFT GRAY ORGANIC SILTY CLAY
MED CONSISTENCY GRAY PEBBLY SILT CLAY
VERY SOFT GRAY SILTY CLAY / W. BLACK SPOTS
BOTTOM OF MUCK EXCAVATION
LOOSE TO DENSE BROWN-GRAY PEBBLY SAND SILT (TILL)
MAINE STATE HIGHWAY COMMISSION
ROUTE 100 INTERCHANGE
RAMP "D" PLAN & PROFILE
PALMYRA
SCALE: AS SHOWN DATE: JUNE, 1962
SOILS ENGINEERING LAB U OF M

BORING NOTES

All samples and vanes are made ahead of casing

Ground water table

Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blow

Location of sample or sample attempt

Number and type of dry sample

ID S&H Sampler #1290's
IC 2" O.D. 16 ga. seamless tubing
IU 3½" O.D. 16 ga. seamless tubing

IW Wash sample and number

M_D Unsuccessful sample attempt and type of sampler

Number of blows required to drive spoon or tubing one foot with 350 ft. lbs. of energy per blow

H Sampling spoon or seamless tubing driven by static weight of drill rods and hammer

P Piston sampler

Field vane test

Bottom of boring (may not be bottom of soil strata)

Refusal of drill rods or casing (may not be ledge)

R 71 % Locations cored by diamond bit and per cent recovery of rock

SHEAR NOTES

● Field vane shear strengths

X Laboratory vane shear strengths

→ Shear strengths in excess of capacity of equipment

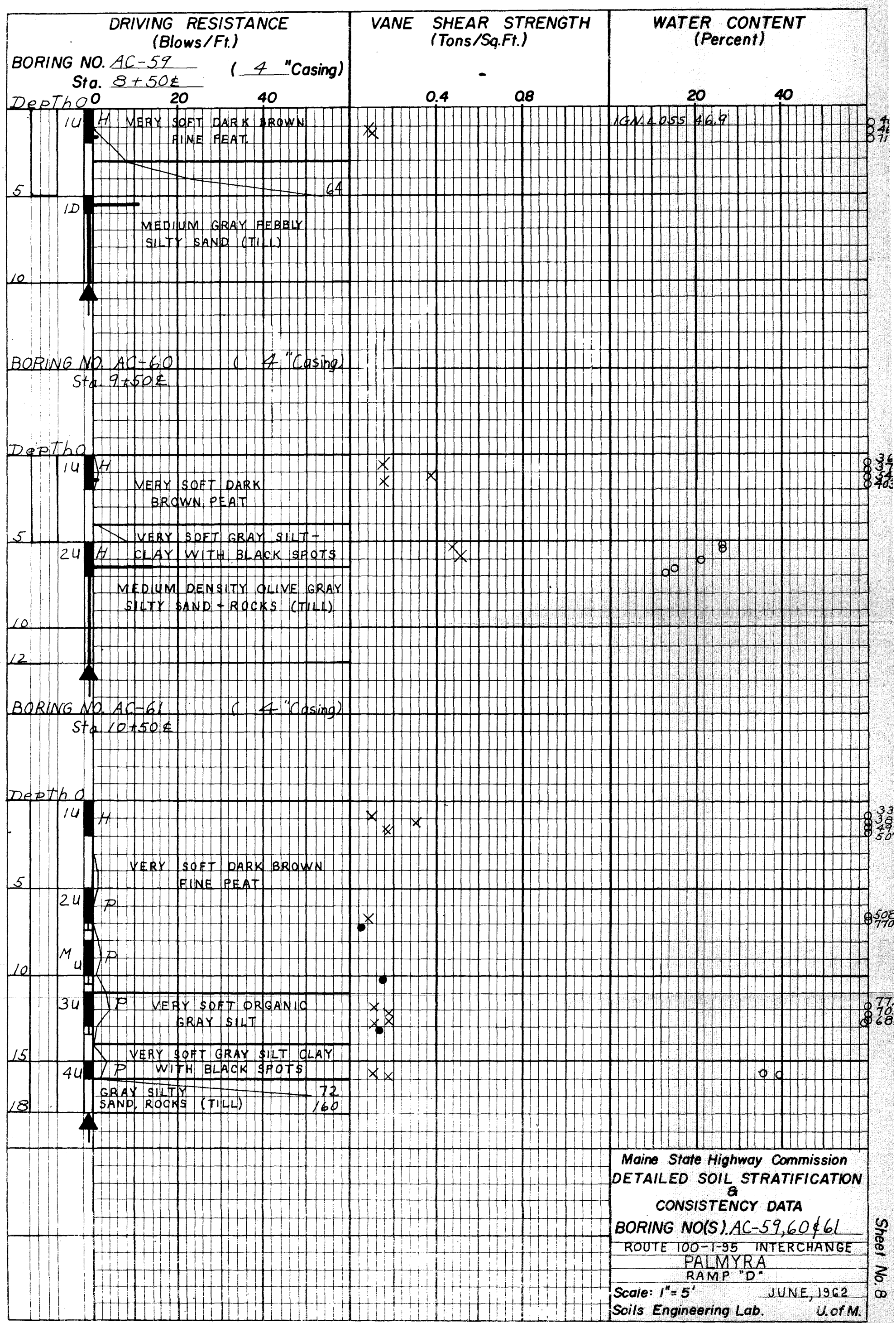
O One half unconfined compressive strengths

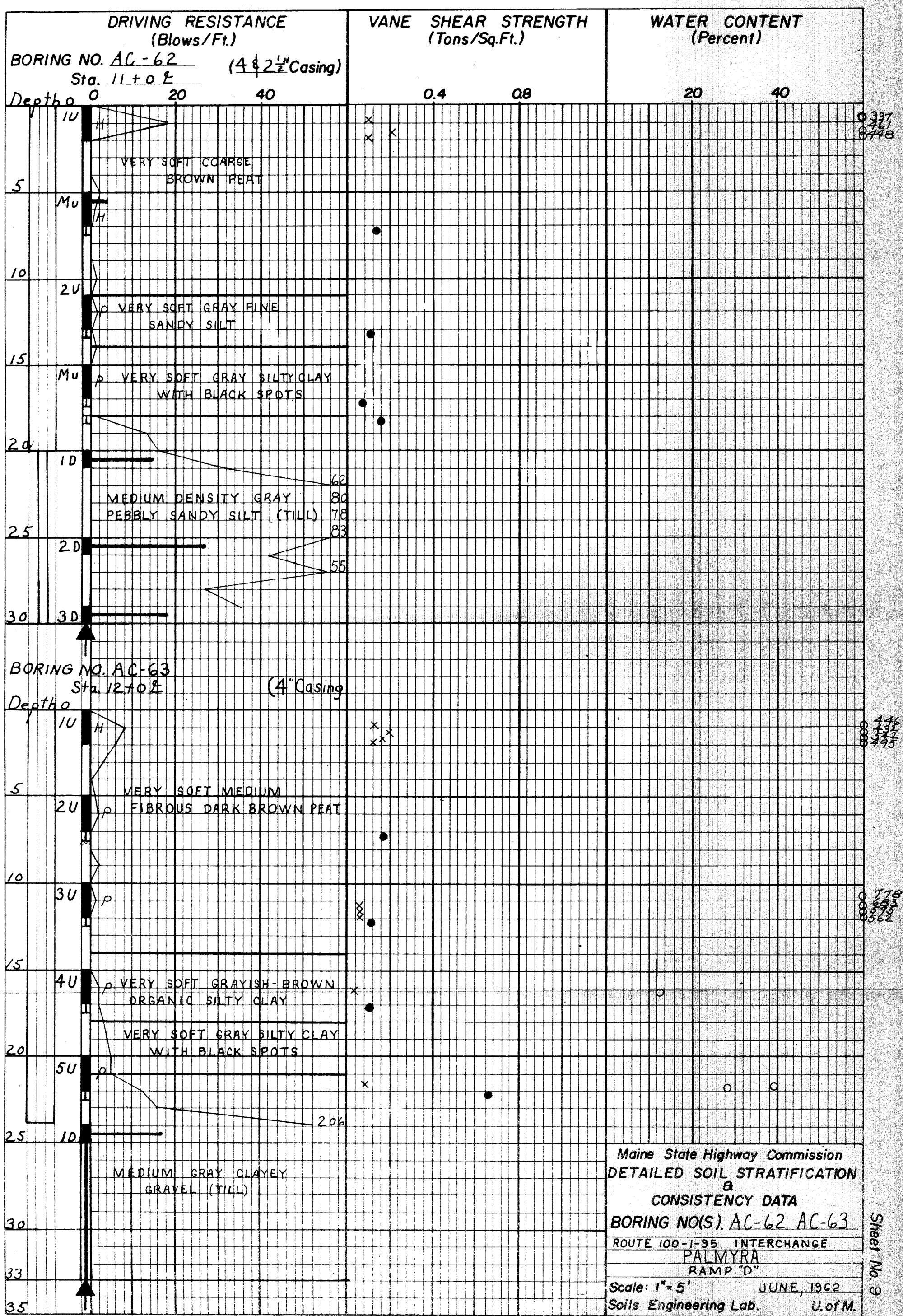
WATER CONTENT NOTES

O Natural water contents, given as per cent of dry weight

⊗---X Plastic and liquid limits

Ignition losses are given as per cent of dry weight


DRIVING RESISTANCE (Blows/Ft.)
VANE SHEAR STRENGTH (Tons/Sq.Ft.)
WATER CONTENT (Percent)
BORING NO. AC-59 (4 "Casing)
Sta. 8+50℄
Depth 0
0 20 40
0.4 0.8
20 40
1U H VERY SOFT DARK BROWN FINE PEAT
64
1D MEDIUM GRAY PEBBLY SILTY SAND (TILL)
1GM 4.055 46.9
BORING NO. AC-60 (4 "Casing)
Sta. 9+50℄
1U H VERY SOFT DARK BROWN PEAT
VERY SOFT GRAY SILT-CLAY WITH BLACK SPOTS
2U H
MEDIUM DENSITY OLIVE GRAY SILTY SAND + ROCKS (TILL)
BORING NO. AC-61 (4 "Casing)
Sta. 10+50℄
1U H
VERY SOFT DARK BROWN FINE PEAT
2U P
M U P
3U P VERY SOFT ORGANIC GRAY SILT
VERY SOFT GRAY SILT CLAY WITH BLACK SPOTS
4U P
GRAY SILTY SAND, ROCKS (TILL)
72
160
Maine State Highway Commission
DETAILED SOIL STRATIFICATION & CONSISTENCY DATA
BORING NO(S). AC-59, 60 & 61
ROUTE 100-I-95 INTERCHANGE
PALMYRA
RAMP "D"
Scale: 1" = 5'
JUNE, 1962
Soils Engineering Lab.
U. of M.
Sheet No. 8



Maine State Highway Commission

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING NO(S). AC-62 AC-63

ROUTE 100-I-95 INTERCHANGE

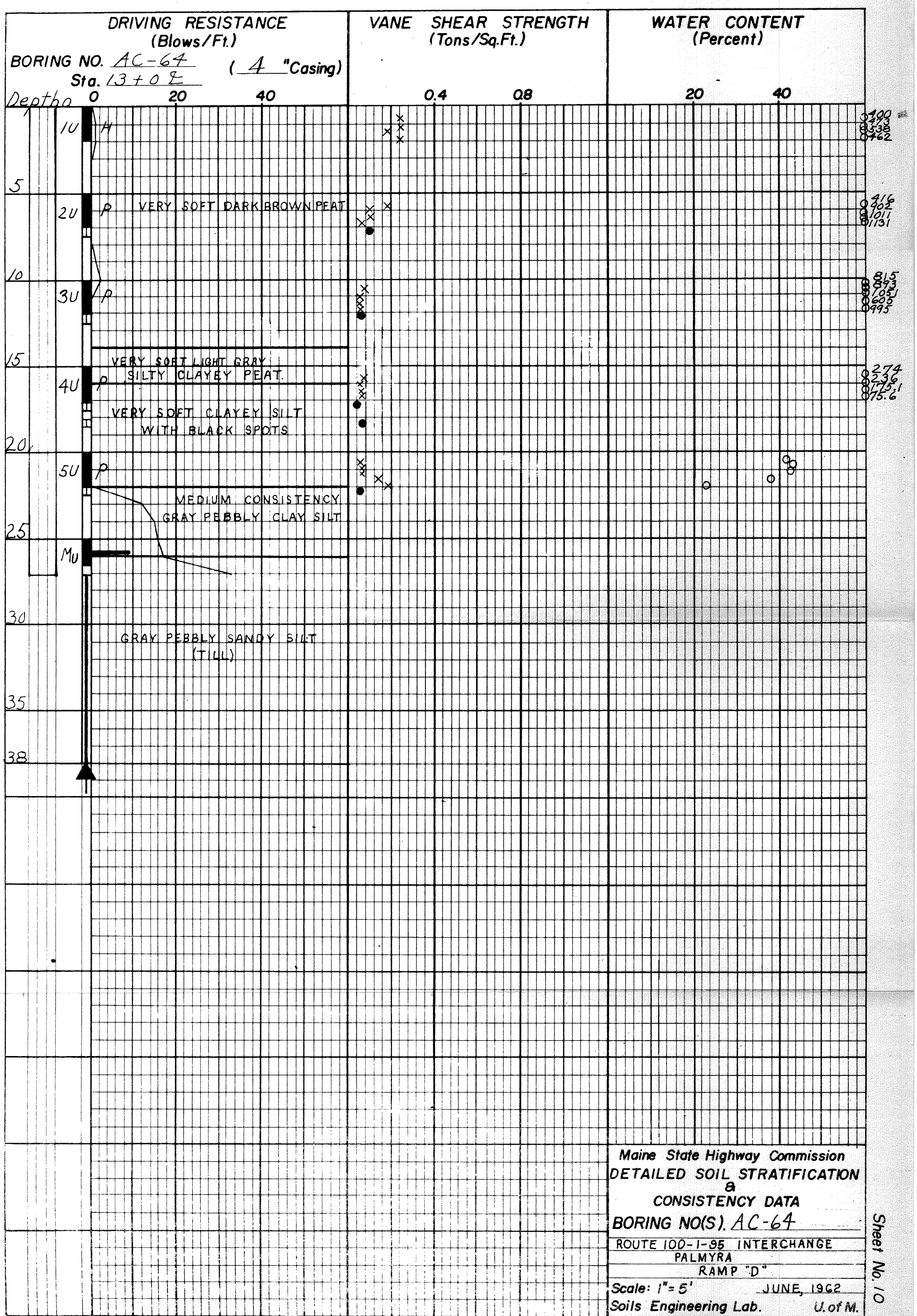
PALMYRA

RAMP "D"

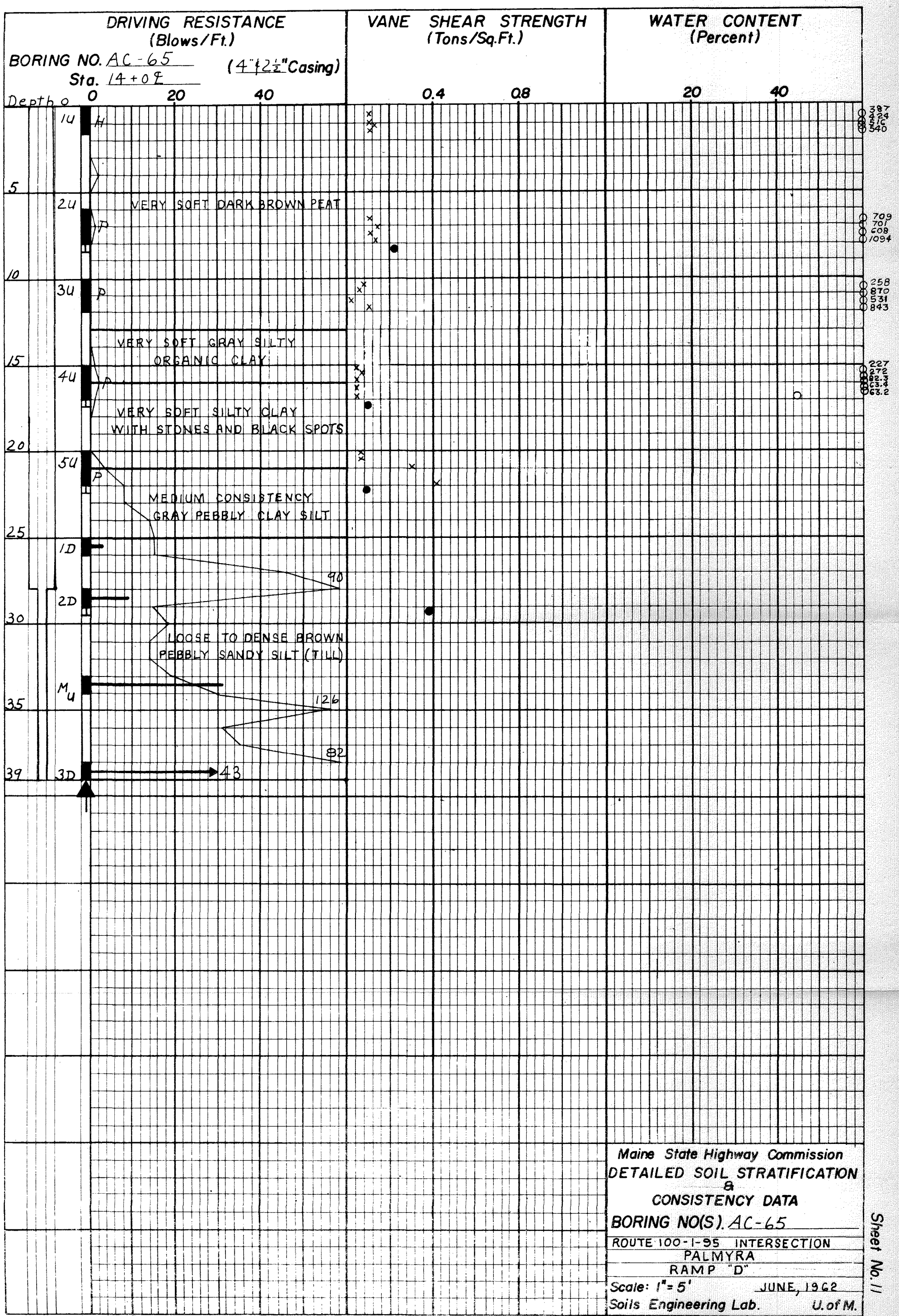
Scale: 1" = 5' JUNE, 1962

Soils Engineering Lab. U. of M.

Sheet No. 9


DRIVING RESISTANCE
(Blows/Ft.)
BORING NO. AC-64
Sta. 13+0 ℄
(4 "Casing)
Depth
0
20
40
VANE SHEAR STRENGTH
(Tons/Sq.Ft.)
0.4
0.8
WATER CONTENT
(Percent)
20
40
1U
H
2U
P
VERY SOFT DARK BROWN PEAT
3U
P
VERY SOFT LIGHT GRAY
SILTY CLAYEY PEAT.
4U
P
VERY SOFT CLAYEY SILT
WITH BLACK SPOTS
5U
P
MEDIUM CONSISTENCY
GRAY PEBBLY CLAY SILT
MU
GRAY PEBBLY SANDY SILT
(TILL)
5
10
15
20
25
30
35
38
490
473
538
462
416
902
1011
1131
815
893
1051
605
995
274
236
475.1
875.6
Maine State Highway Commission
DETAILED SOIL STRATIFICATION
&
CONSISTENCY DATA
BORING NO(S). AC-64
ROUTE 100-I-95 INTERCHANGE
PALMYRA
RAMP "D"
Scale: 1" = 5'
JUNE, 1962
Soils Engineering Lab.
U. of M.

Sheet No. 10



Maine State Highway Commission

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

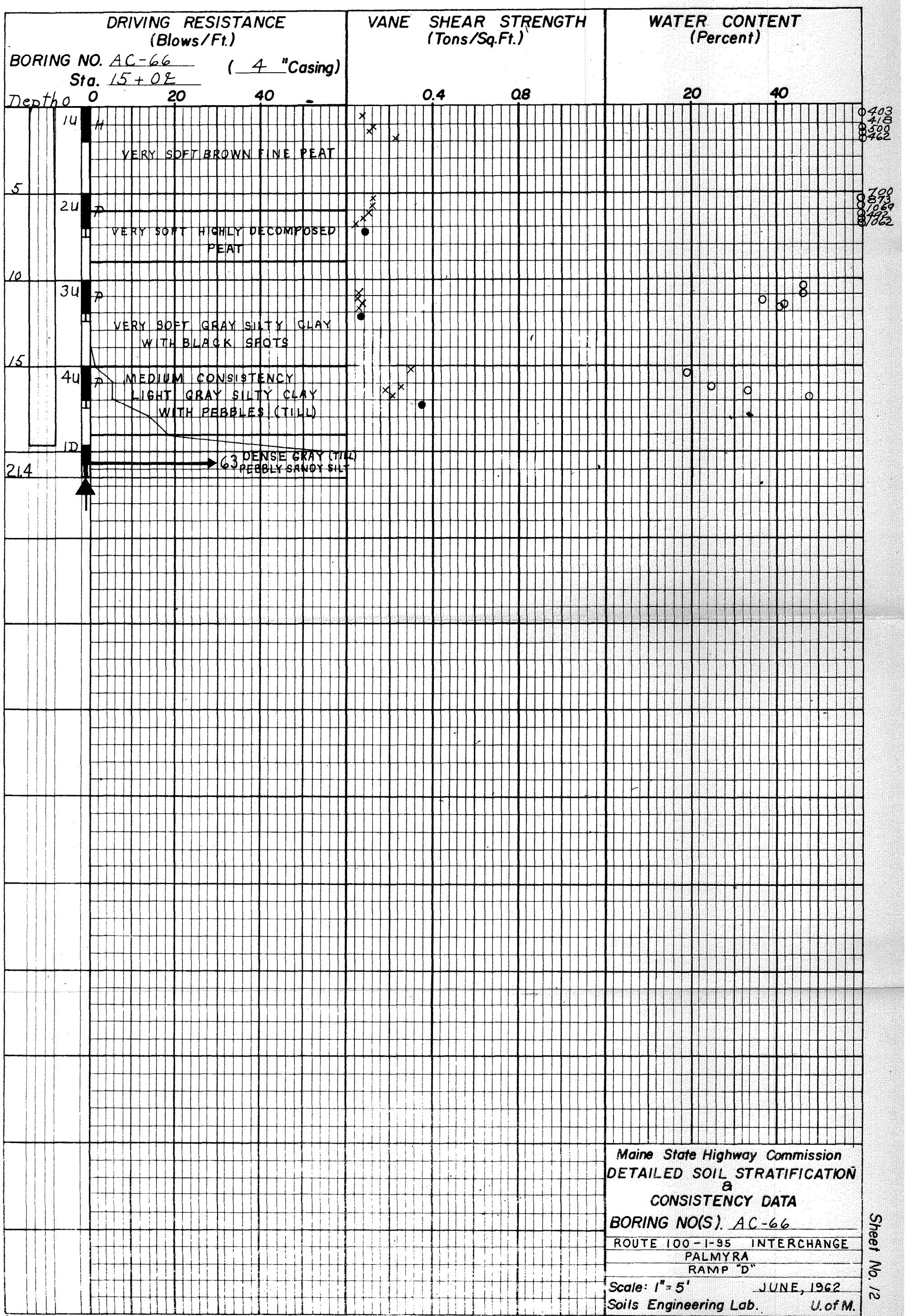
BORING NO(S). AC-65

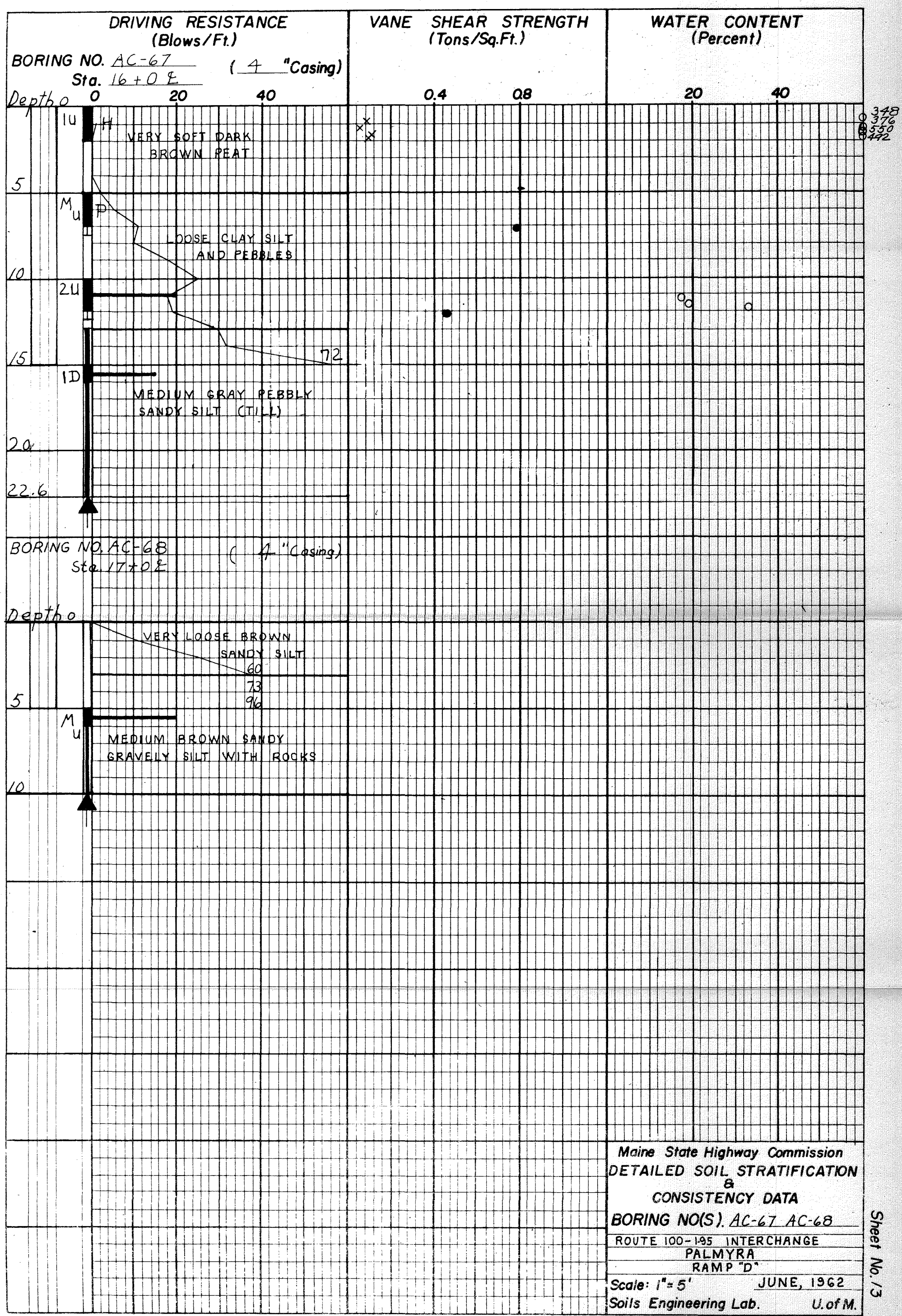
ROUTE 100-I-95 INTERSECTION
PALMYRA
RAMP "D"

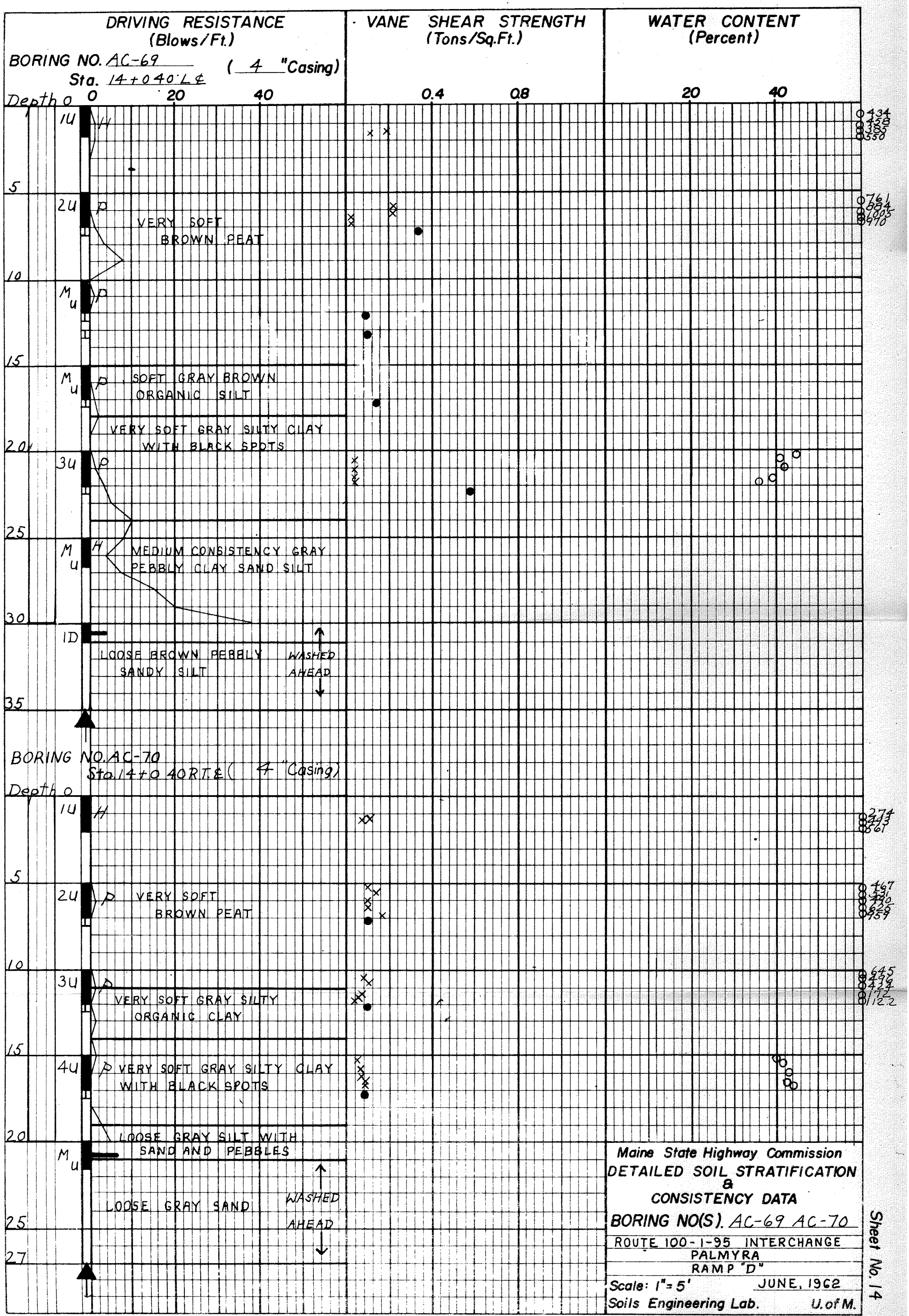
Scale: 1" = 5' JUNE, 1962

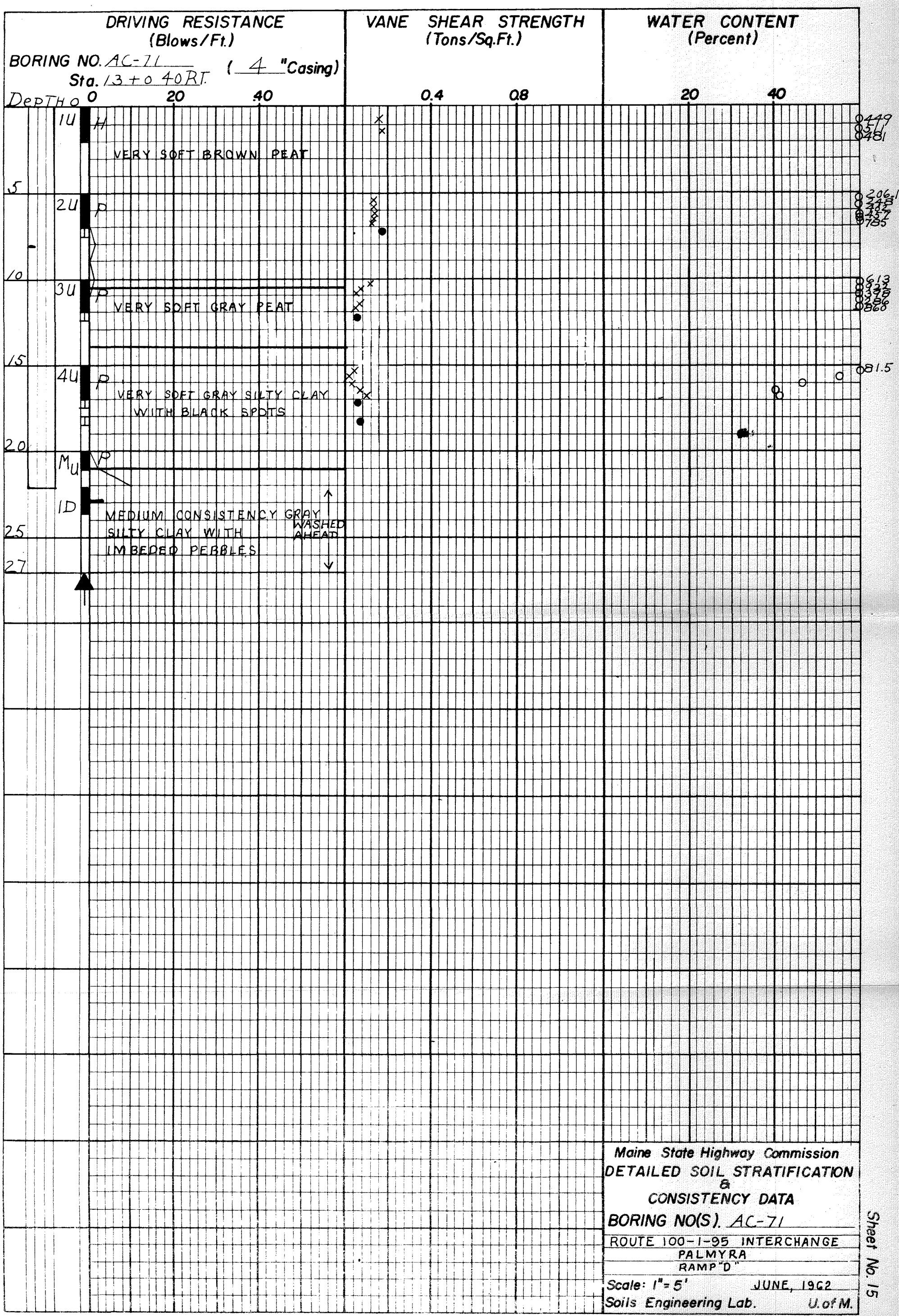
Soils Engineering Lab. U. of M.

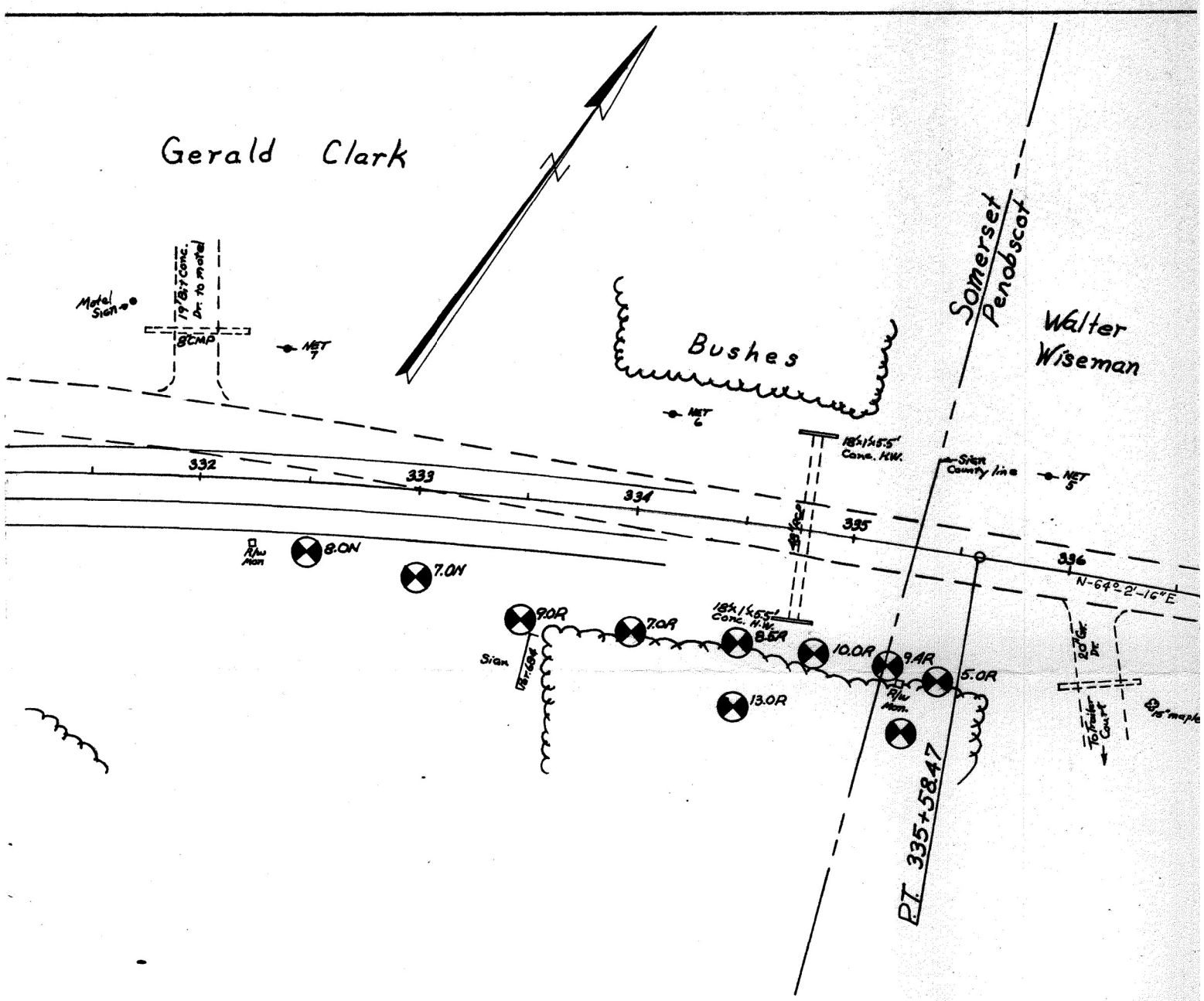
Sheet No. 11

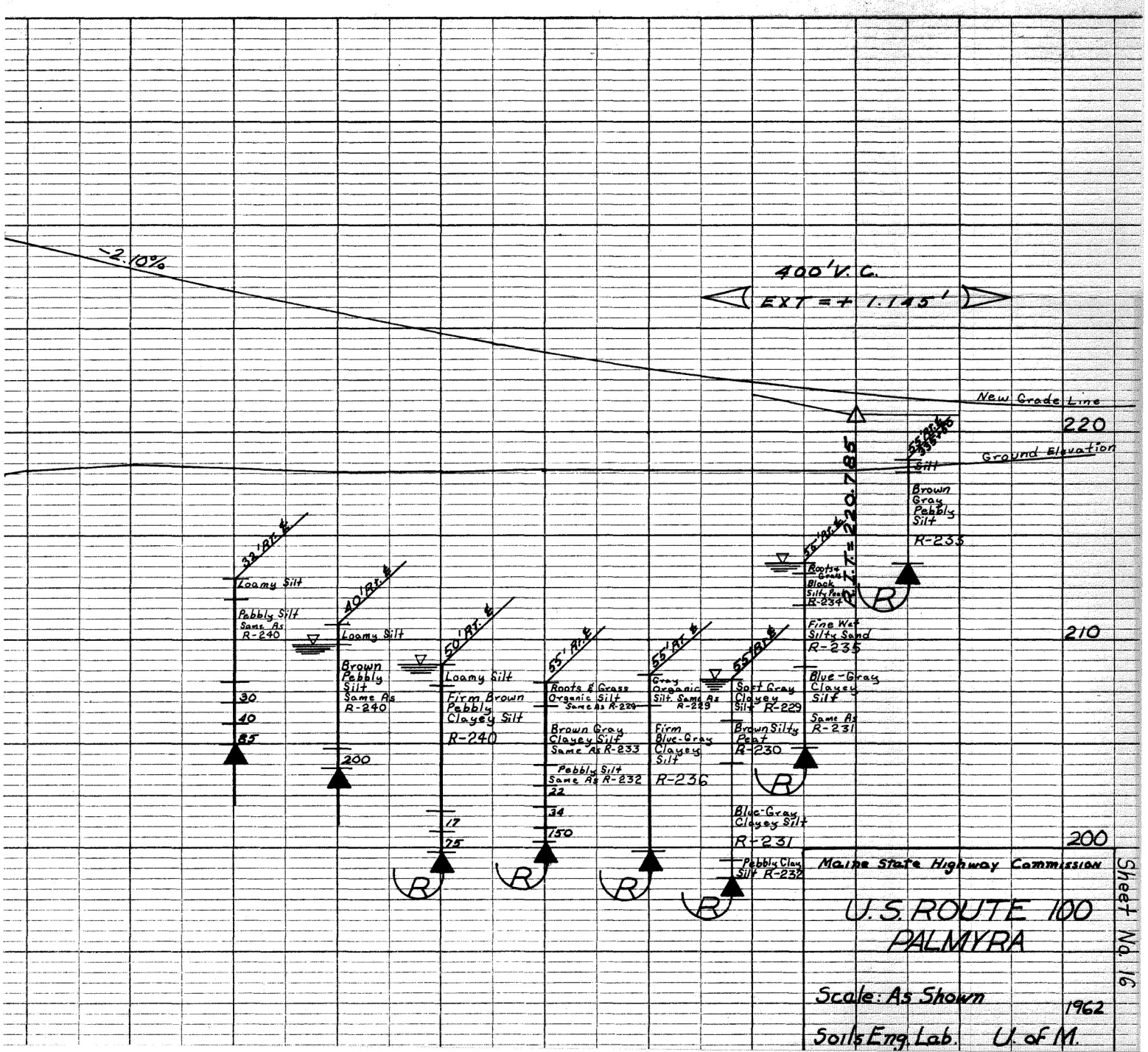

DRIVING RESISTANCE (Blows/Ft.)
BORING NO. AC-66
Sta. 15+02
(4 "Casing)
Depth 0
0
20
40
VANE SHEAR STRENGTH (Tons/Sq.Ft.)
0.4
0.8
WATER CONTENT (Percent)
20
40
1U
H
VERY SOFT BROWN FINE PEAT
5
2U
P
VERY SOFT HIGHLY DECOMPOSED PEAT
10
3U
P
VERY SOFT GRAY SILTY CLAY WITH BLACK SPOTS
15
4U
P
MEDIUM CONSISTENCY LIGHT GRAY SILTY CLAY WITH PEBBLES (TILL)
1D
21.4
63 DENSE GRAY (TILL) PEBBLY SANDY SILT
403
418
500
462
700
873
1069
492
1062
Maine State Highway Commission
DETAILED SOIL STRATIFICATION & CONSISTENCY DATA
BORING NO(S). AC-66
ROUTE 100-I-95 INTERCHANGE
PALMYRA
RAMP "D"
Scale: 1" = 5'
JUNE, 1962
Soils Engineering Lab.
U. of M.
Sheet No. 12


DRIVING RESISTANCE (Blows/Ft.)
VANE SHEAR STRENGTH (Tons/Sq.Ft.)
WATER CONTENT (Percent)
BORING NO. AC-67
Sta. 16+0 ℄
(4 "Casing)
Depth 0
0
20
40
0.4
0.8
20
40
1U
H
VERY SOFT DARK BROWN PEAT
5
Mu
P
LOOSE CLAY SILT AND PEBBLES
10
2U
72
15
1D
MEDIUM GRAY PEBBLY SANDY SILT (TILL)
20
22.6
348
376
550
492
BORING NO. AC-68
Sta. 17+0 ℄
(4 "Casing)
Depth 0
VERY LOOSE BROWN SANDY SILT
60
73
96
5
Mu
MEDIUM BROWN SANDY GRAVELY SILT WITH ROCKS
10
Maine State Highway Commission
DETAILED SOIL STRATIFICATION & CONSISTENCY DATA
BORING NO(S). AC-67 AC-68
ROUTE 100-I-95 INTERCHANGE
PALMYRA
RAMP "D"
Scale: 1" = 5'
JUNE, 1962
Soils Engineering Lab.
U. of M.
Sheet No. 13


DRIVING RESISTANCE (Blows/Ft.)
VANE SHEAR STRENGTH (Tons/Sq.Ft.)
WATER CONTENT (Percent)
BORING NO. AC-69 (4 "Casing)
Sta. 14+040' L ℄
Depth 0
0 20 40
0.4 0.8
20 40
1U
2U
VERY SOFT BROWN PEAT
Mu
SOFT GRAY BROWN ORGANIC SILT
VERY SOFT GRAY SILTY CLAY WITH BLACK SPOTS
3U
MEDIUM CONSISTENCY GRAY PEBBLY CLAY SAND SILT
1D
LOOSE BROWN PEBBLY SANDY SILT
WASHED AHEAD
BORING NO. AC-70
Sta. 14+040 R.T. ℄ (4 "Casing)
VERY SOFT BROWN PEAT
VERY SOFT GRAY SILTY ORGANIC CLAY
4U
VERY SOFT GRAY SILTY CLAY WITH BLACK SPOTS
LOOSE GRAY SILT WITH SAND AND PEBBLES
LOOSE GRAY SAND
WASHED AHEAD
Maine State Highway Commission
DETAILED SOIL STRATIFICATION & CONSISTENCY DATA
BORING NO(S). AC-69 AC-70
ROUTE 100-I-95 INTERCHANGE
PALMYRA
RAMP "D"
Scale: 1" = 5'
JUNE, 1962
Soils Engineering Lab.
U. of M.
Sheet No. 14


DRIVING RESISTANCE
(Blows/Ft.)
BORING NO. AC-71
Sta. 13+0 40RT.
(4 "Casing)
DEPTH 0
0
20
40
VANE SHEAR STRENGTH
(Tons/Sq.Ft.)
0.4
0.8
WATER CONTENT
(Percent)
20
40
1U
H
VERY SOFT BROWN PEAT
5
2U
P
10
3U
P
VERY SOFT GRAY PEAT
15
4U
P
VERY SOFT GRAY SILTY CLAY
WITH BLACK SPOTS
20
MU
P
1D
MEDIUM CONSISTENCY GRAY
SILTY CLAY WITH
IMBEDED PEBBLES
WASHED AHEAD
25
27
449
571
481
206.1
248
202
257
785
613
378
286
860
81.5
Maine State Highway Commission
DETAILED SOIL STRATIFICATION
&
CONSISTENCY DATA
BORING NO(S). AC-71
ROUTE 100-I-95 INTERCHANGE
PALMYRA
RAMP "D"
Scale: 1" = 5'
JUNE, 1962
Soils Engineering Lab.
U. of M.
Sheet No. 15

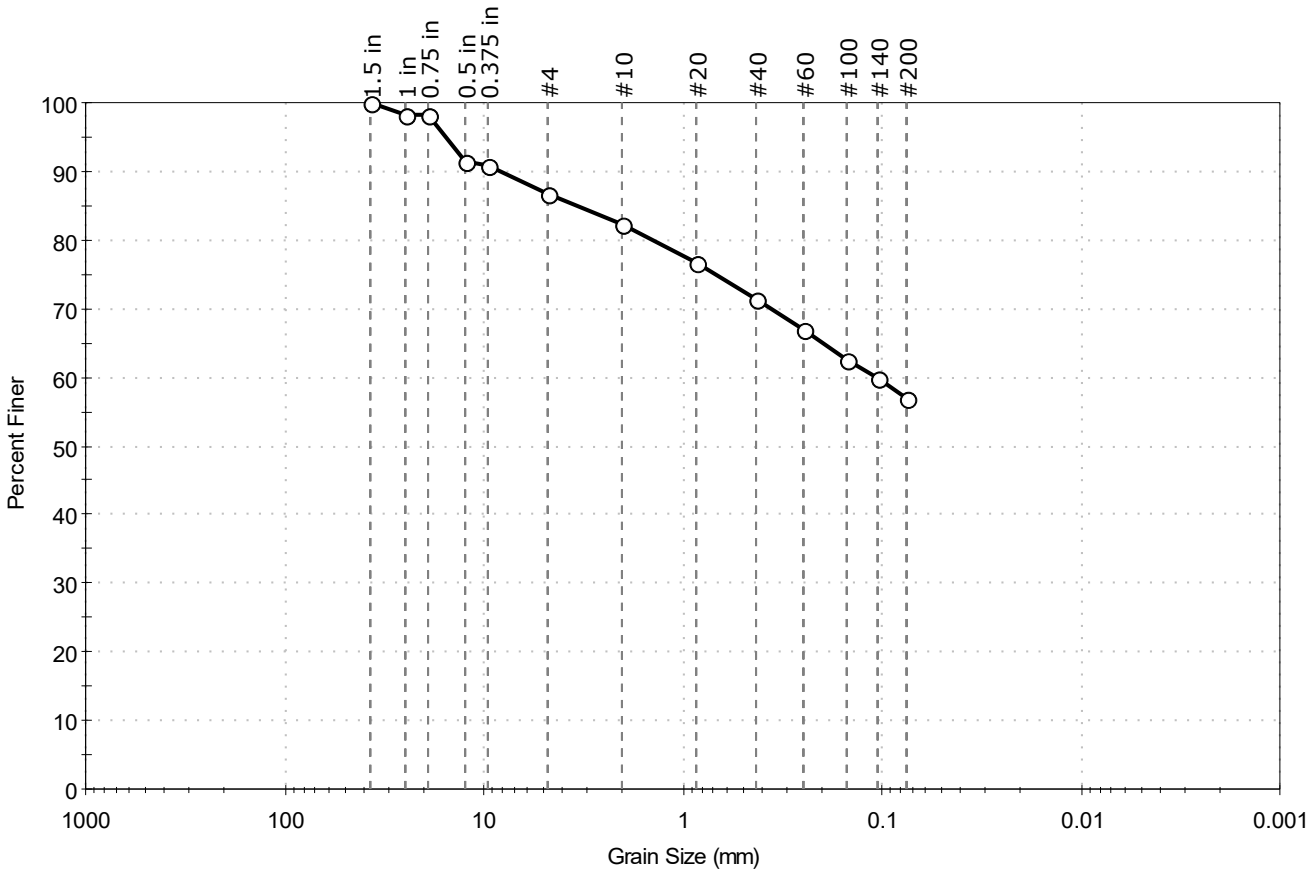

Gerald Clark
Bushes
Somerset
Penobscot
Walter Wiseman
Motel Sign
19'8"x7' Conc. Dr. to motel
8"CMP
NET 7
NET 6
NET 5
18"x145.5' Conc. H.W.
Sign County line
332
333
334
335
336
N-64°-2'-16"E
R/W Mon
8.0N
7.0N
9.0R
7.0R
8.5R
10.0R
9.4R
5.0R
13.0R
Sign
To Trailer Court
P.T. 335+58.47


-2.10%
400' V.C.
EXT = + 1.145'
New Grade Line
Ground Elevation
220
210
200
V.C. = 210.785
33' Rt. ℄
Loamy Silt
Pebbly Silt Same As R-240
30
40
85
40' Rt. ℄
Loamy Silt
Brown Pebbly Silt Same As R-240
200
50' Rt. ℄
Loamy Silt
Firm Brown Pebbly Clayey Silt
R-240
17
75
55' Rt. ℄
Roots & Grass
Organic Silt Same As R-229
Brown Gray Clayey Silt Same As R-233
Pebbly Silt Same As R-232
22
34
150
55' Rt. ℄
Gray Organic Silt Same As R-229
Firm Blue-Gray Clayey Silt
R-236
55' Rt. ℄
Soft Gray Clayey Silt R-229
Brown Silty Peat R-230
Blue-Gray Clayey Silt R-231
Pebbly Clay Silt R-232
55' Rt. ℄
Roots & Black Silty Peat R-234
Fine Wet Silty Sand R-235
Blue-Gray Clayey Silt Same As R-231
Silt
Brown Gray Pebbly Silt
R-233
Maine State Highway Commission
U.S. ROUTE 100
PALMYRA
Scale: As Shown
1962
Soils Eng. Lab. U. of M.
Sheet No. 16

APPENDIX B
Geotechnical Laboratory Test Results

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (2D, 3D, 4D)	Test Date:	08/12/24	Checked By:	ank
Depth :	2-11.2'	Test Id:	779699		
Test Comment:	---				
Visual Description:	Moist, brown sandy clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	13.3	29.9	56.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	98		
0.75 in	19.00	98		
0.5 in	12.50	92		
0.375 in	9.50	91		
#4	4.75	87		
#10	2.00	82		
#20	0.85	77		
#40	0.42	71		
#60	0.25	67		
#100	0.15	63		
#140	0.11	60		
#200	0.075	57		

Coefficients

D_{85} = 3.4078 mm	D_{30} = N/A
D_{60} = 0.1083 mm	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Sandy Lean CLAY (CL)

AASHTO Silty Soils (A-4 (2))

Sample/Test Description

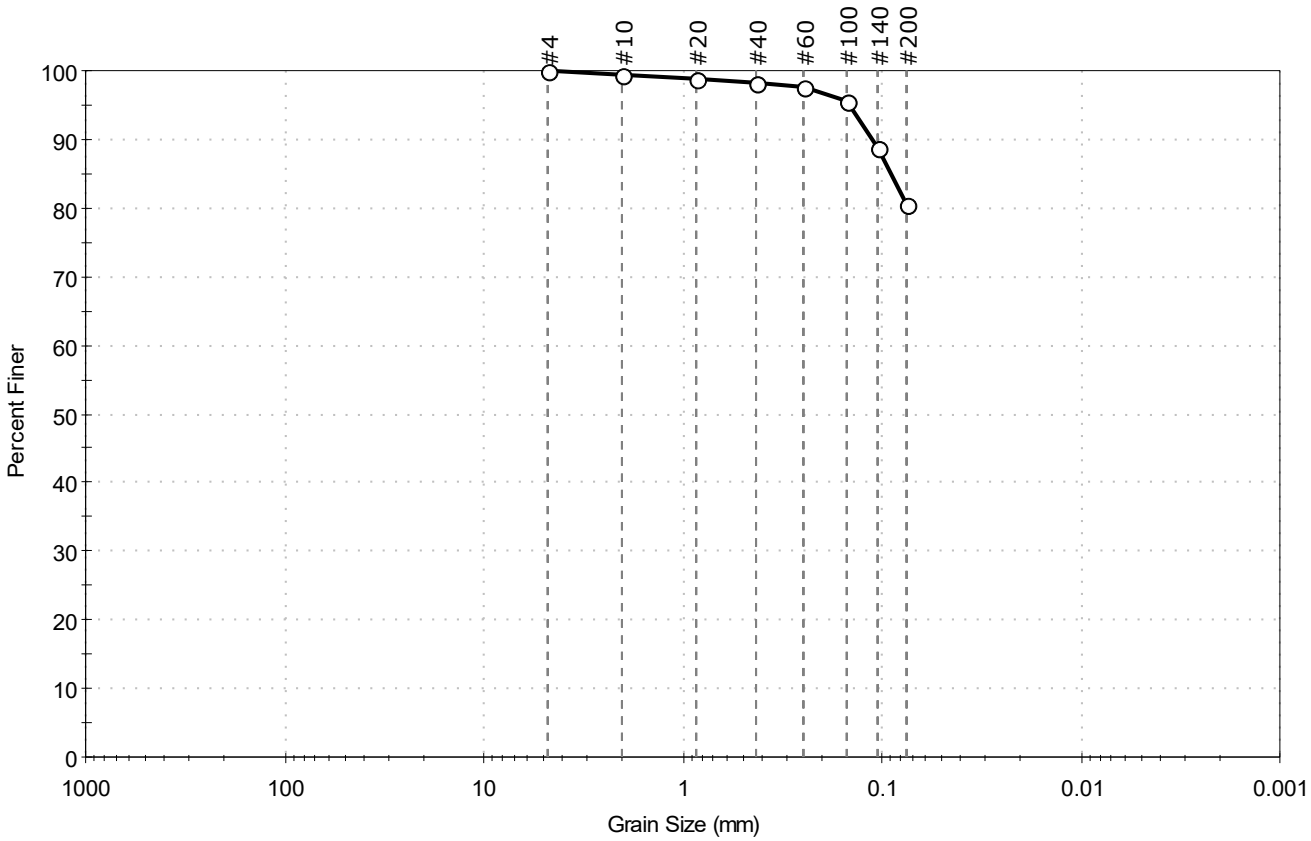
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 8/13/2024 10:29:37 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	ajl
Sample ID:	5D	Test Date:	08/09/24	Checked By:	ank
Depth :	15-17'	Test Id:	779700		
Test Comment:	---				
Visual Description:	Moist, light olive brown silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	19.5	80.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	99		
#40	0.42	98		
#60	0.25	98		
#100	0.15	96		
#140	0.11	89		
#200	0.075	80		

Coefficients

D_{85} = 0.0904 mm D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

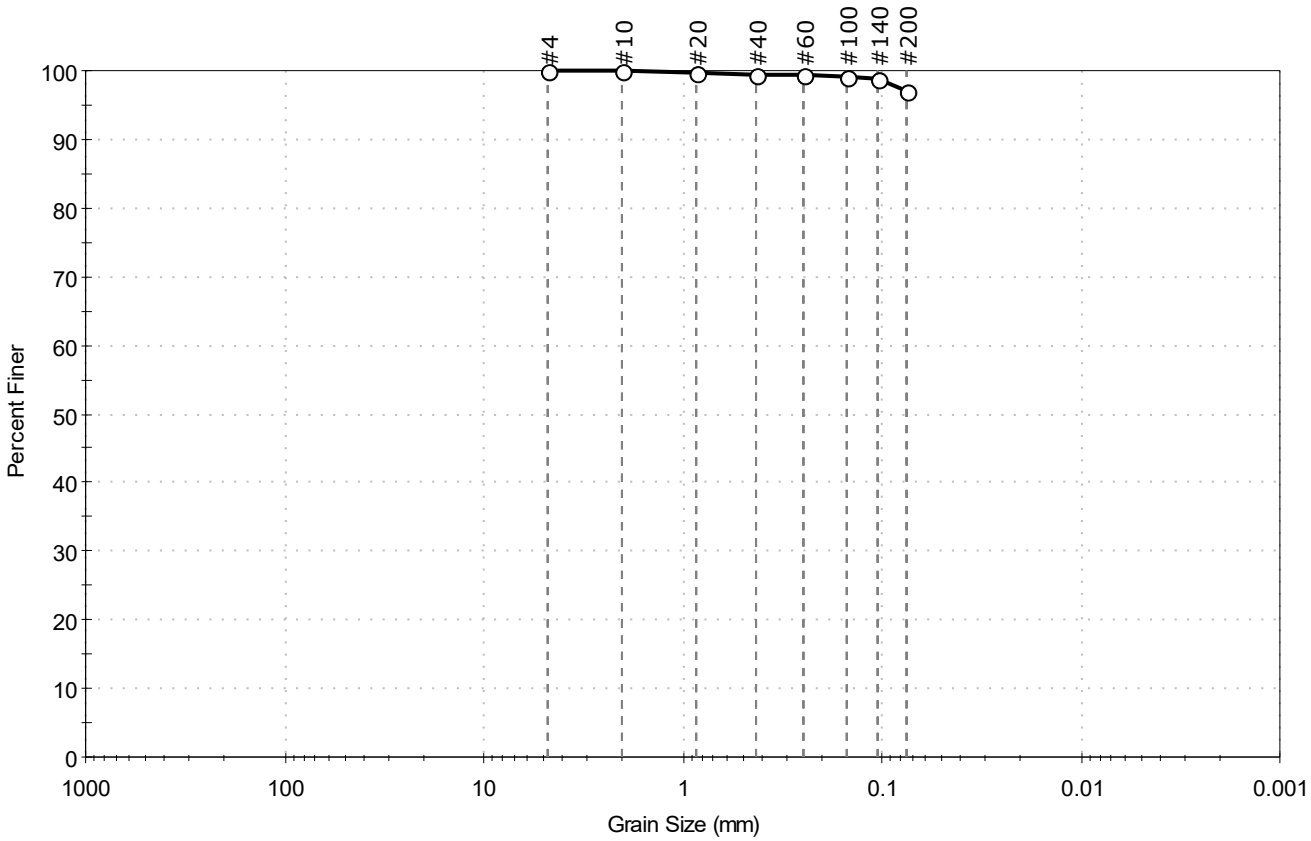
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

printed 8/13/2024 10:29:37 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	ajl
Sample ID:	6D	Test Date:	08/09/24	Checked By:	ank
Depth :	20-22'	Test Id:	779701		
Test Comment:	---				
Visual Description:	Moist, dark gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	3.0	97.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	99		
#60	0.25	99		
#100	0.15	99		
#140	0.11	99		
#200	0.075	97		

Coefficients

D_{85} = N/A D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Silty Soils (A-4 (6))

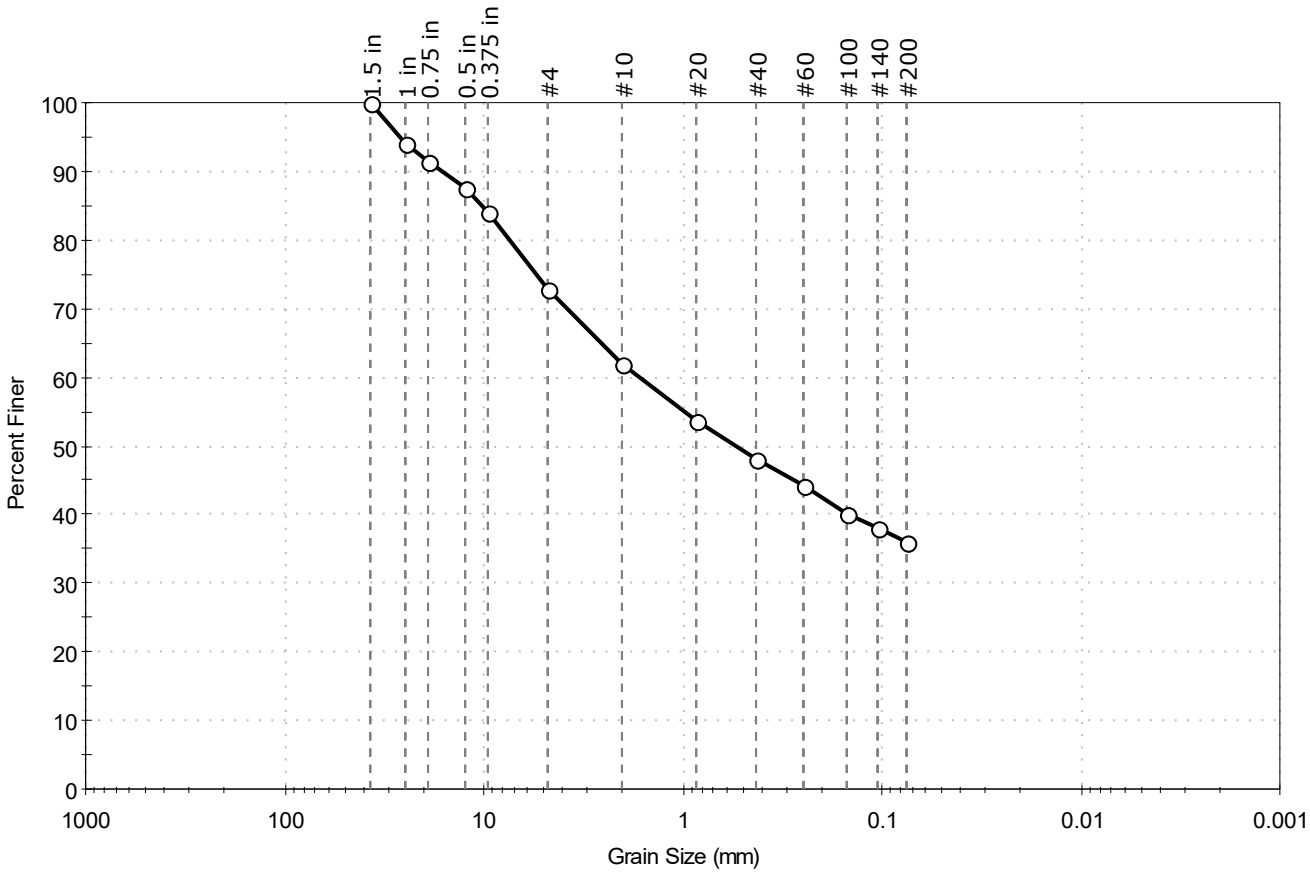
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (8D, 9D)	Test Date:	08/13/24	Checked By:	ank
Depth :	30-36.2'	Test Id:	779702		
Test Comment:	---				
Visual Description:	Moist, grayish brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	27.1	36.9	36.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	94		
0.75 in	19.00	91		
0.5 in	12.50	88		
0.375 in	9.50	84		
#4	4.75	73		
#10	2.00	62		
#20	0.85	54		
#40	0.42	48		
#60	0.25	44		
#100	0.15	40		
#140	0.11	38		
#200	0.075	36		

Coefficients

D_{85} = 10.2440 mm D_{30} = N/A
D_{60} = 1.6313 mm D_{15} = N/A
D_{50} = 0.5355 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

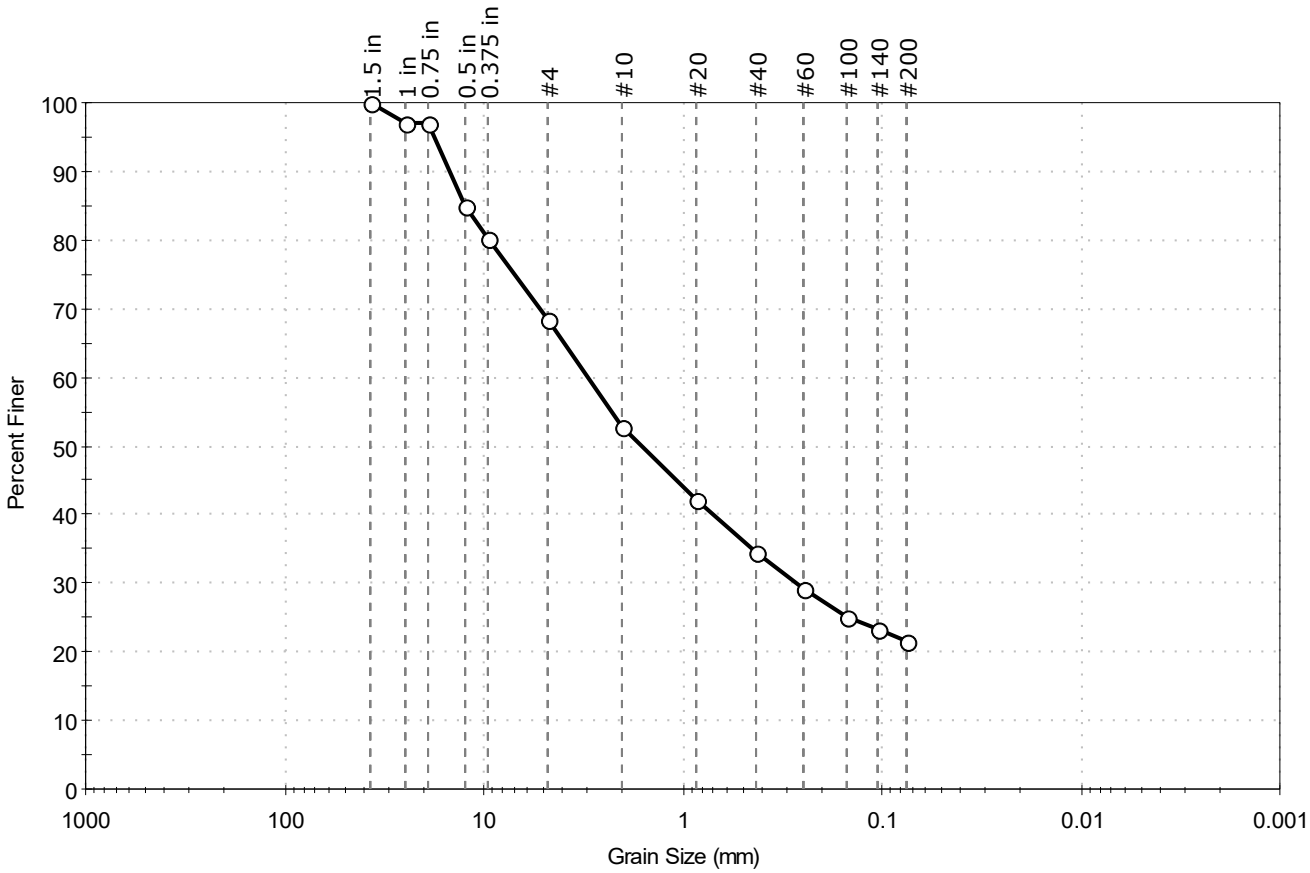
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 8/13/2024 10:29:39 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (11D, 12D)	Test Date:	08/13/24	Checked By:	ank
Depth :	45-52'	Test Id:	779703		
Test Comment:	---				
Visual Description:	Moist, brownish gray silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	31.5	46.9	21.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	97		
0.75 in	19.00	97		
0.5 in	12.50	85		
0.375 in	9.50	80		
#4	4.75	68		
#10	2.00	53		
#20	0.85	42		
#40	0.42	35		
#60	0.25	29		
#100	0.15	25		
#140	0.11	23		
#200	0.075	22		

Coefficients

D_{85} = 12.5124 mm D_{30} = 0.2717 mm

D_{60} = 2.9640 mm D_{15} = N/A

D_{50} = 1.5821 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

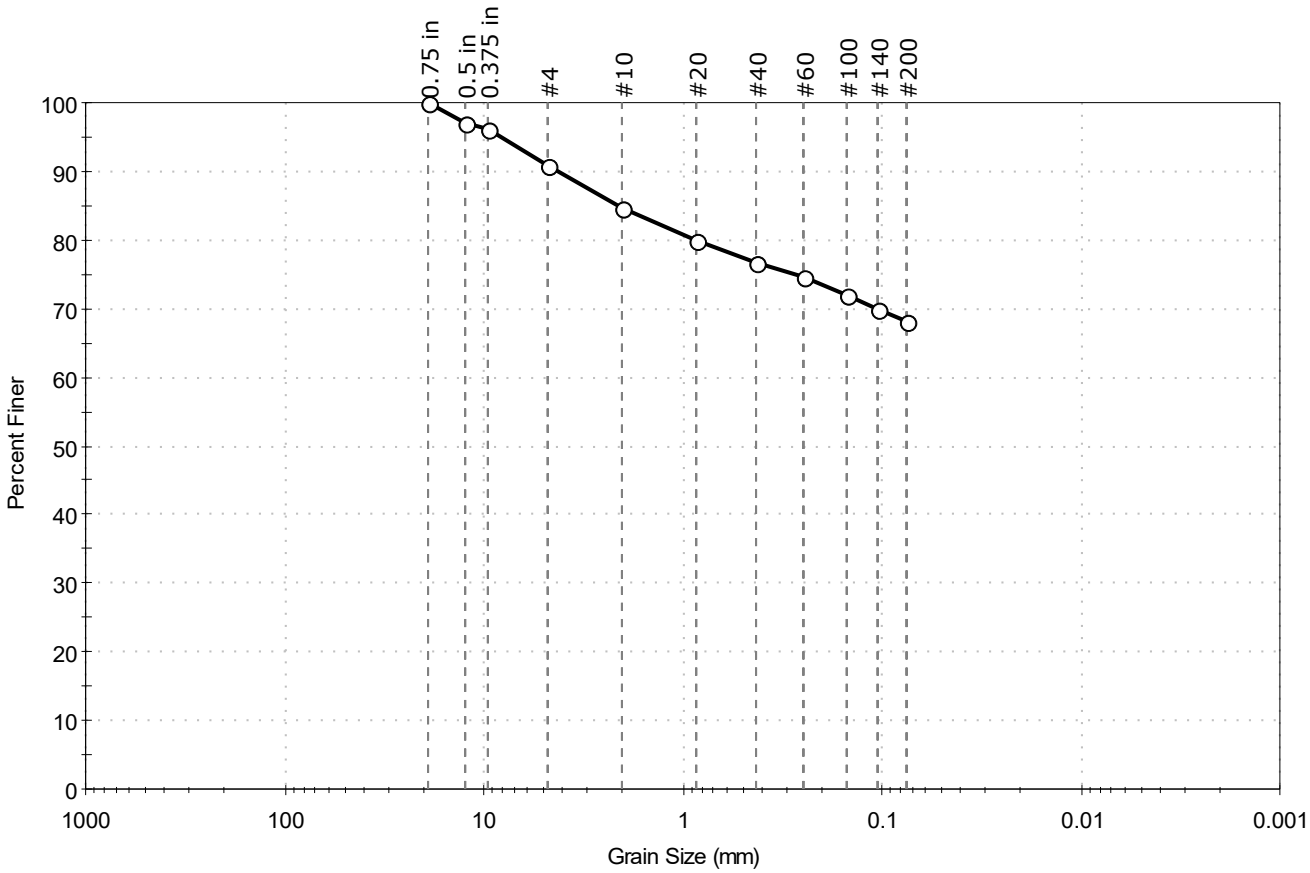
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 8/13/2024 10:29:39 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (14D, 15D, 16D	Test Date:	08/13/24	Checked By:	ank
Depth :	70-81.4'	Test Id:	779704		
Test Comment:	---				
Visual Description:	Moist, brown silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.0	22.8	68.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	96		
#4	4.75	91		
#10	2.00	85		
#20	0.85	80		
#40	0.42	77		
#60	0.25	75		
#100	0.15	72		
#140	0.11	70		
#200	0.075	68		

Coefficients

D_{85} = 2.0931 mm D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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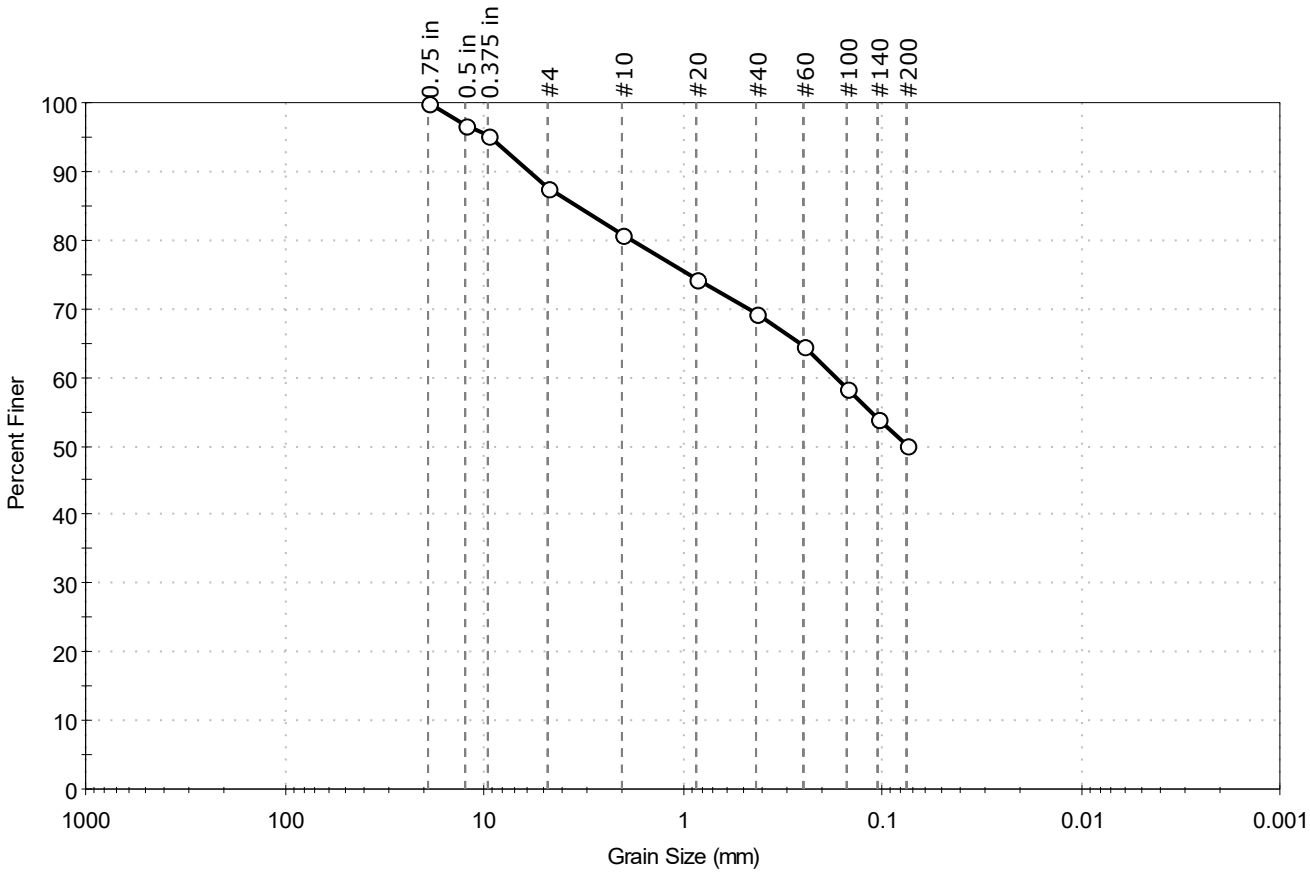
Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	ajl
Sample ID:	3D	Test Date:	08/09/24	Checked By:	ank
Depth :	5-7'	Test Id:	779705		
Test Comment:	---				
Visual Description:	Moist, light olive brown sandy clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.4	37.4	50.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	95		
#4	4.75	88		
#10	2.00	81		
#20	0.85	74		
#40	0.42	69		
#60	0.25	65		
#100	0.15	58		
#140	0.11	54		
#200	0.075	50		

Coefficients

D_{85} = 3.4223 mm D_{30} = N/A

D_{60} = 0.1709 mm D_{15} = N/A

D_{50} = N/A D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM Sandy Lean CLAY (CL)

AASHTO Silty Soils (A-4 (1))

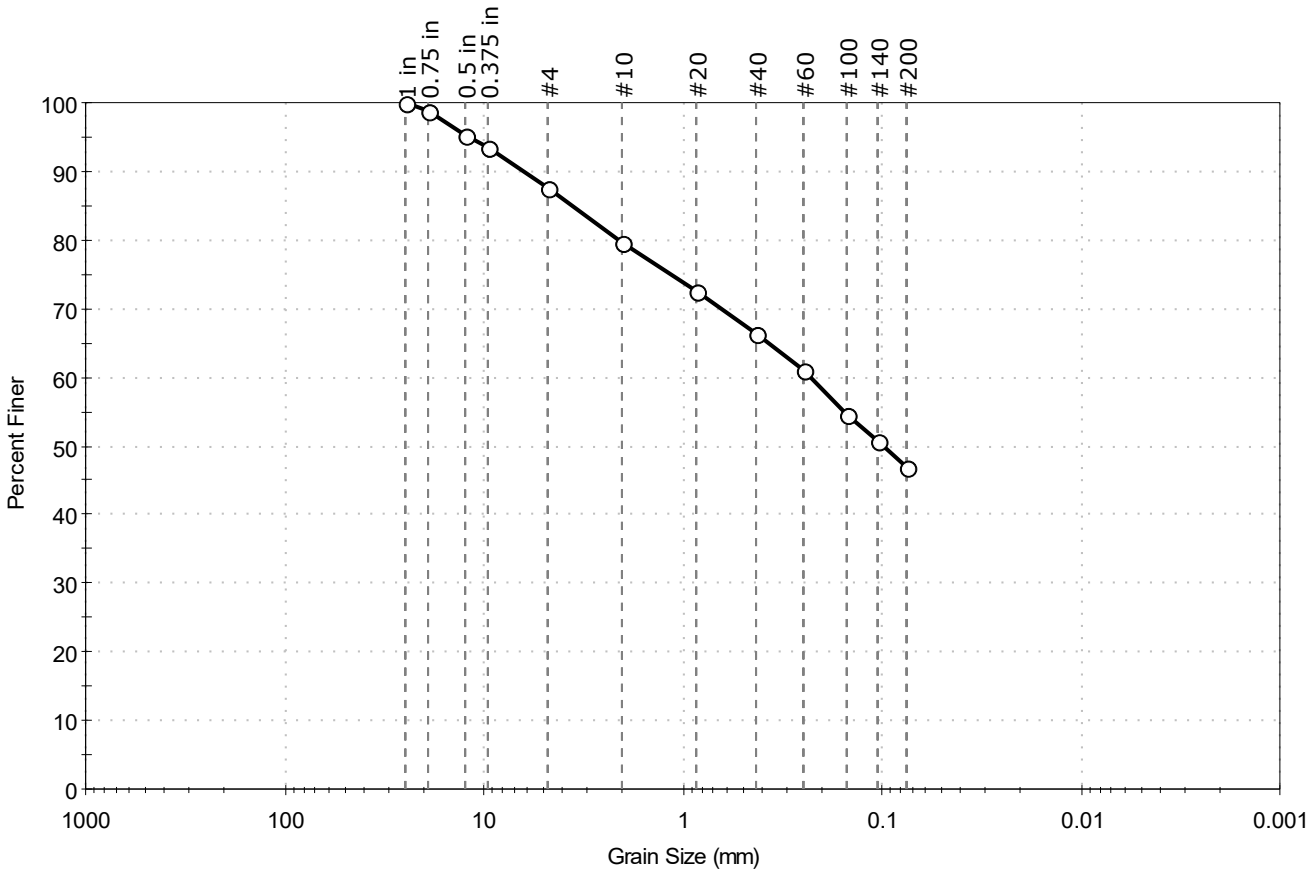
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (4D, 5D)	Test Date:	08/13/24	Checked By:	ank
Depth :	10-17'	Test Id:	779706		
Test Comment:	---				
Visual Description:	Moist, olive brown silty, clayey sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.3	40.9	46.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	99		
0.5 in	12.50	95		
0.375 in	9.50	94		
#4	4.75	88		
#10	2.00	80		
#20	0.85	73		
#40	0.42	66		
#60	0.25	61		
#100	0.15	55		
#140	0.11	51		
#200	0.075	47		

Coefficients

D_{85} = 3.5413 mm D_{30} = N/A

D_{60} = 0.2319 mm D_{15} = N/A

D_{50} = 0.0990 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM Silty, Clayey SAND (SC-SM)

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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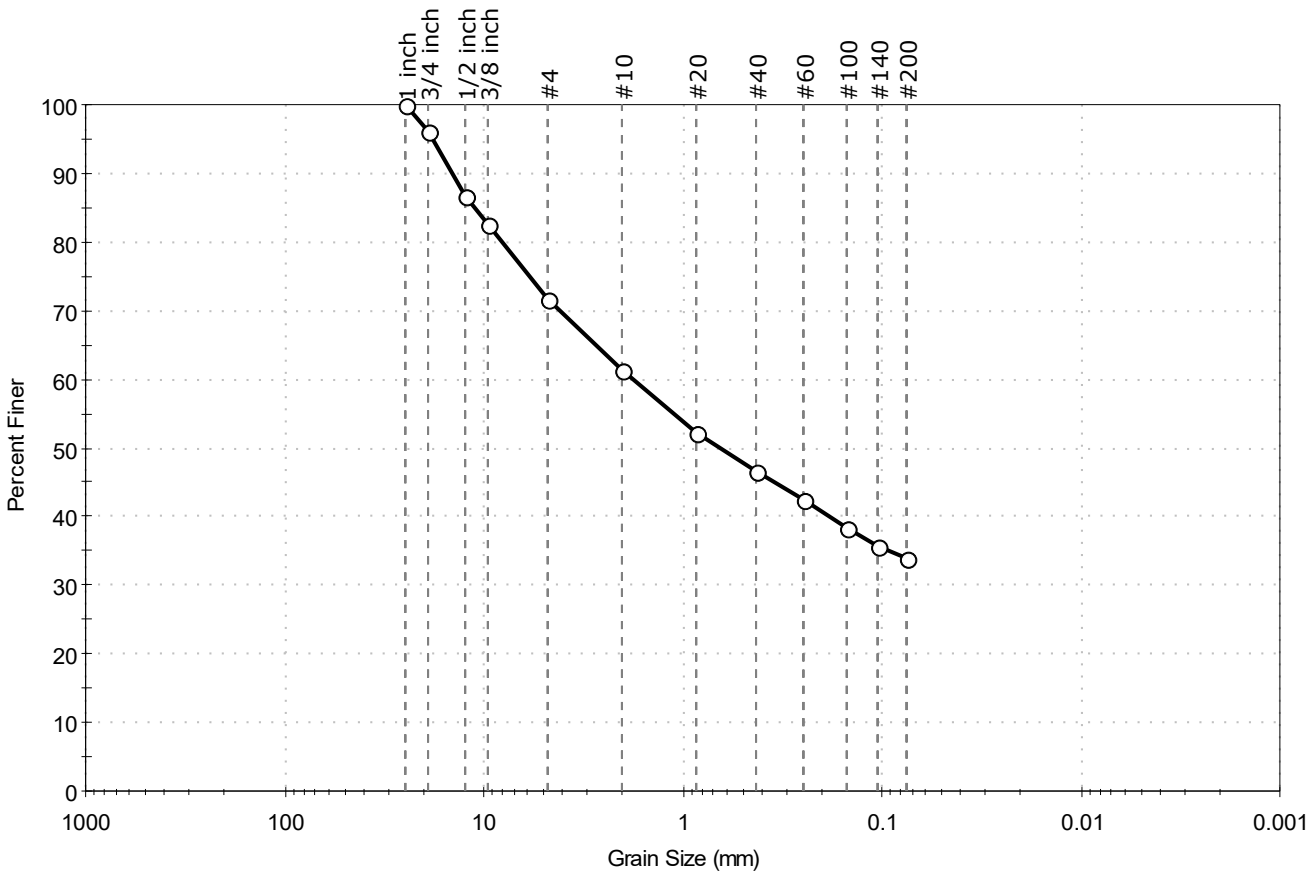
Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	ajl
Sample ID:	7D	Test Date:	08/12/24	Checked By:	ank
Depth :	25-27'	Test Id:	779707		
Test Comment:	---				
Visual Description:	Moist, grayish brown silty, clayey sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	28.3	37.8	33.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	96		
1/2 inch	12.50	87		
3/8 inch	9.50	83		
#4	4.75	72		
#10	2.00	61		
#20	0.85	52		
#40	0.42	46		
#60	0.25	42		
#100	0.15	38		
#140	0.11	36		
#200	0.075	34		

Coefficients

D_{85} = 11.1828 mm D_{30} = N/A

D_{60} = 1.7628 mm D_{15} = N/A

D_{50} = 0.6483 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM Silty, Clayey SAND with Gravel (SC-SM)

AASHTO Silty Gravel and Sand (A-2-4 (0))

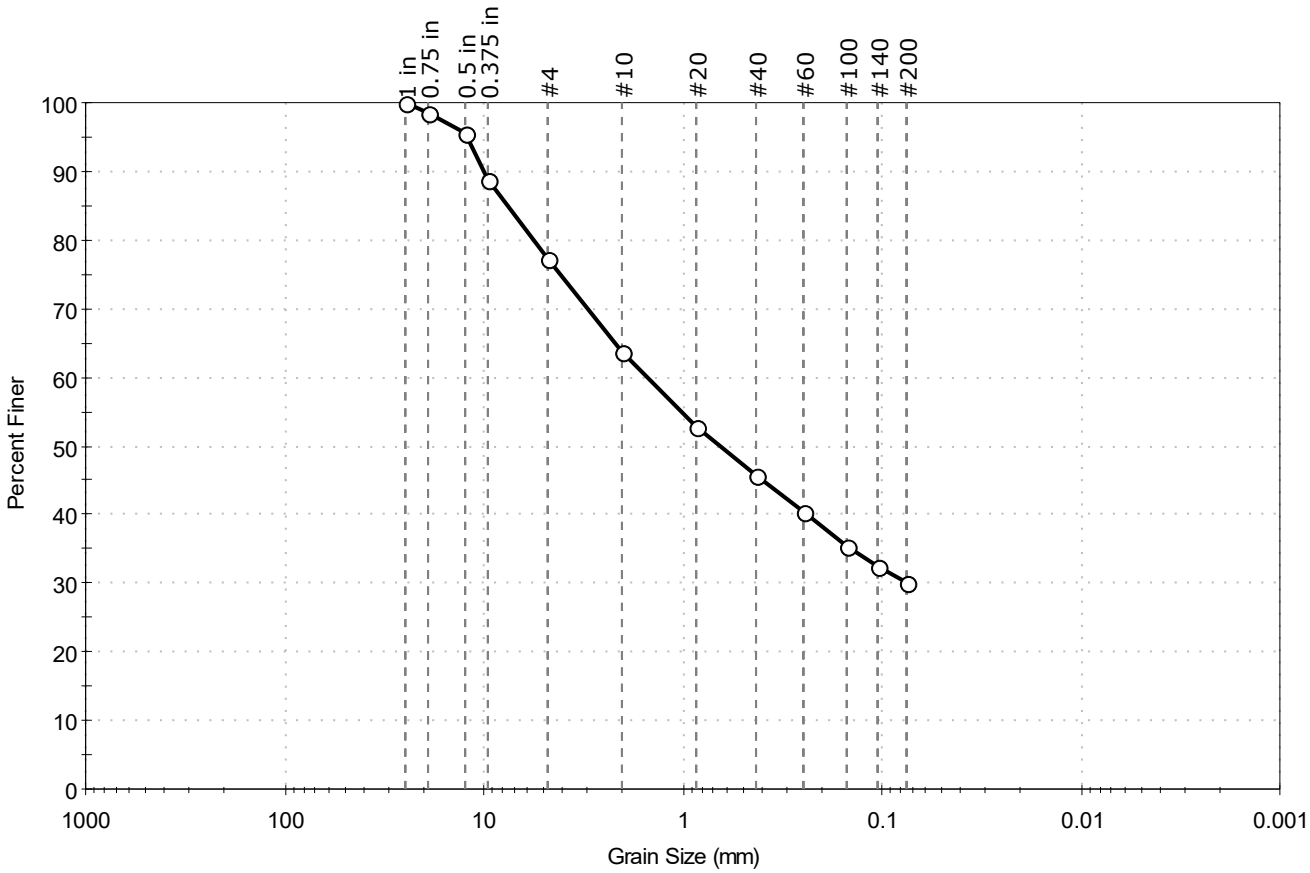
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (8D, 9D)	Test Date:	08/13/24	Checked By:	ank
Depth :	30-35.81.4'	Test Id:	779708		
Test Comment:	---				
Visual Description:	Moist, gray silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	22.6	47.3	30.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	99		
0.5 in	12.50	96		
0.375 in	9.50	89		
#4	4.75	77		
#10	2.00	64		
#20	0.85	53		
#40	0.42	46		
#60	0.25	41		
#100	0.15	35		
#140	0.11	32		
#200	0.075	30		

Coefficients

D_{85} = 7.5493 mm D_{30} = N/A

D_{60} = 1.5021 mm D_{15} = N/A

D_{50} = 0.6460 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

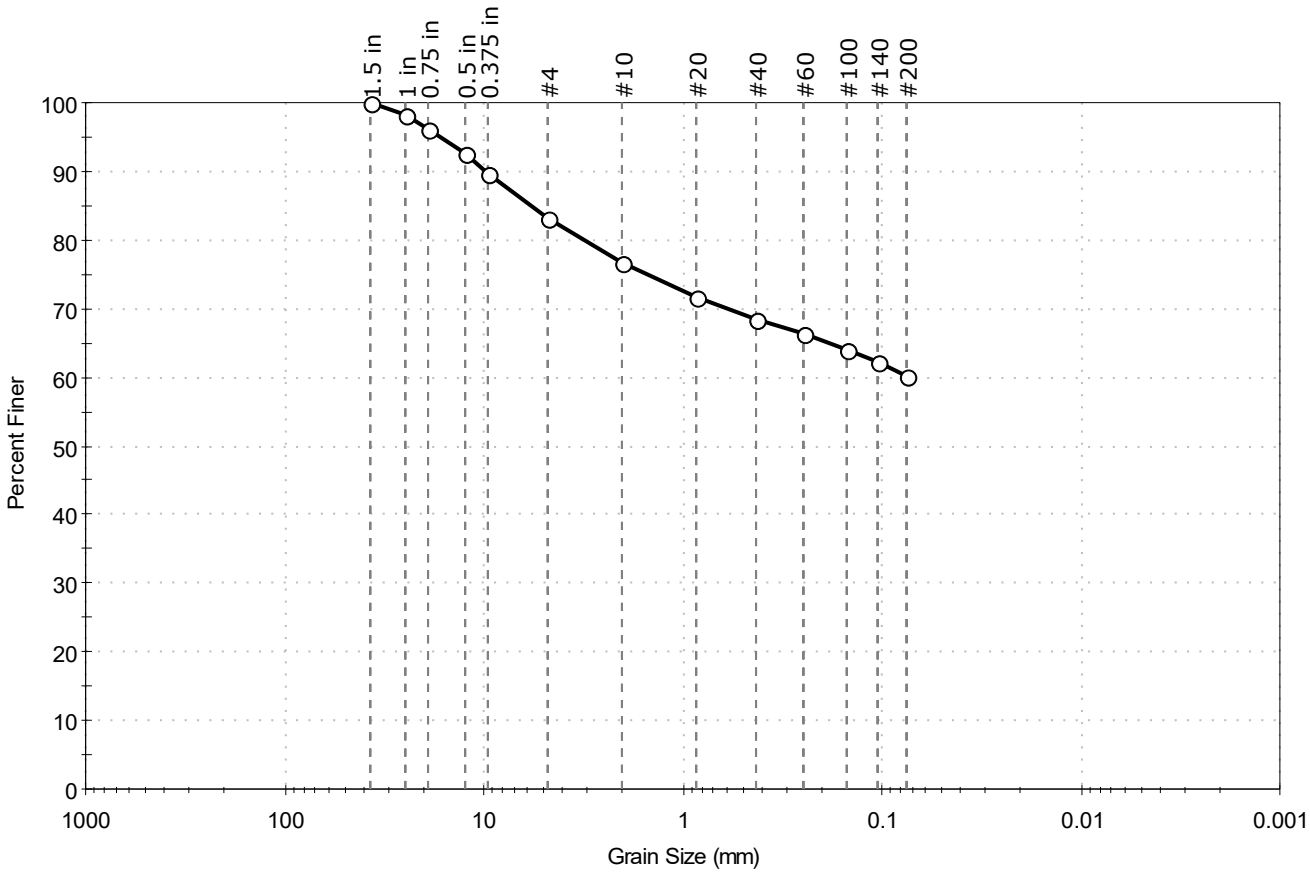
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (10D, 11D, 12D	Test Date:	08/12/24	Checked By:	ank
Depth :	40-52'	Test Id:	779709		
Test Comment:	---				
Visual Description:	Moist, brownish gray silt with sand and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	16.8	23.2	60.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	98		
0.75 in	19.00	96		
0.5 in	12.50	93		
0.375 in	9.50	90		
#4	4.75	83		
#10	2.00	77		
#20	0.85	72		
#40	0.42	69		
#60	0.25	66		
#100	0.15	64		
#140	0.11	62		
#200	0.075	60		

Coefficients

D_{85} = 5.7651 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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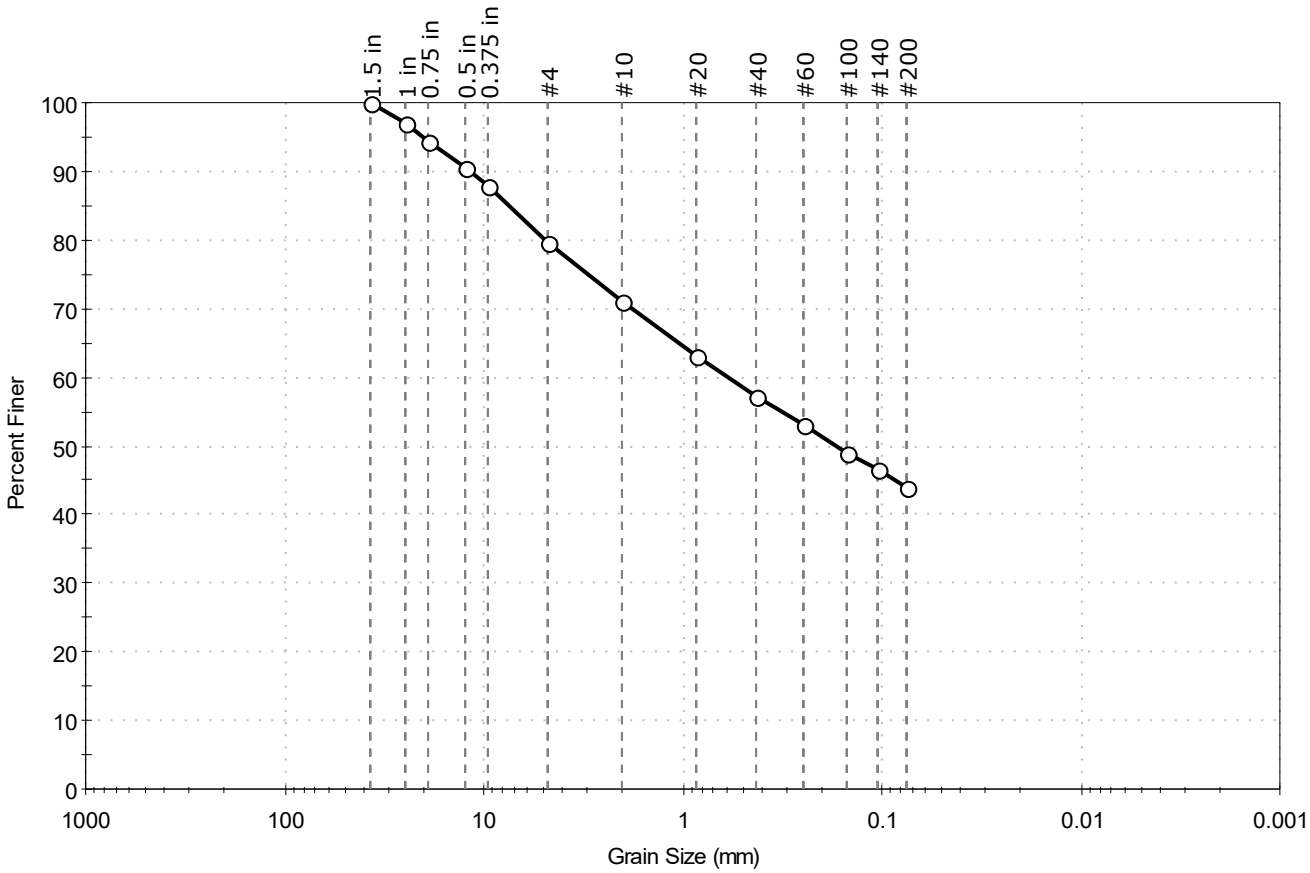
Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-103	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (2D, 3D)	Test Date:	08/12/24	Checked By:	ank
Depth :	2-6'	Test Id:	779710		
Test Comment:	---				
Visual Description:	Moist, olive brown silty, clayey sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.4	35.7	43.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	97		
0.75 in	19.00	95		
0.5 in	12.50	91		
0.375 in	9.50	88		
#4	4.75	80		
#10	2.00	71		
#20	0.85	63		
#40	0.42	57		
#60	0.25	53		
#100	0.15	49		
#140	0.11	46		
#200	0.075	44		

Coefficients

D_{85} = 7.4633 mm	D_{30} = N/A
D_{60} = 0.5851 mm	D_{15} = N/A
D_{50} = 0.1729 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Silty, Clayey SAND with Gravel (SC-SM)

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

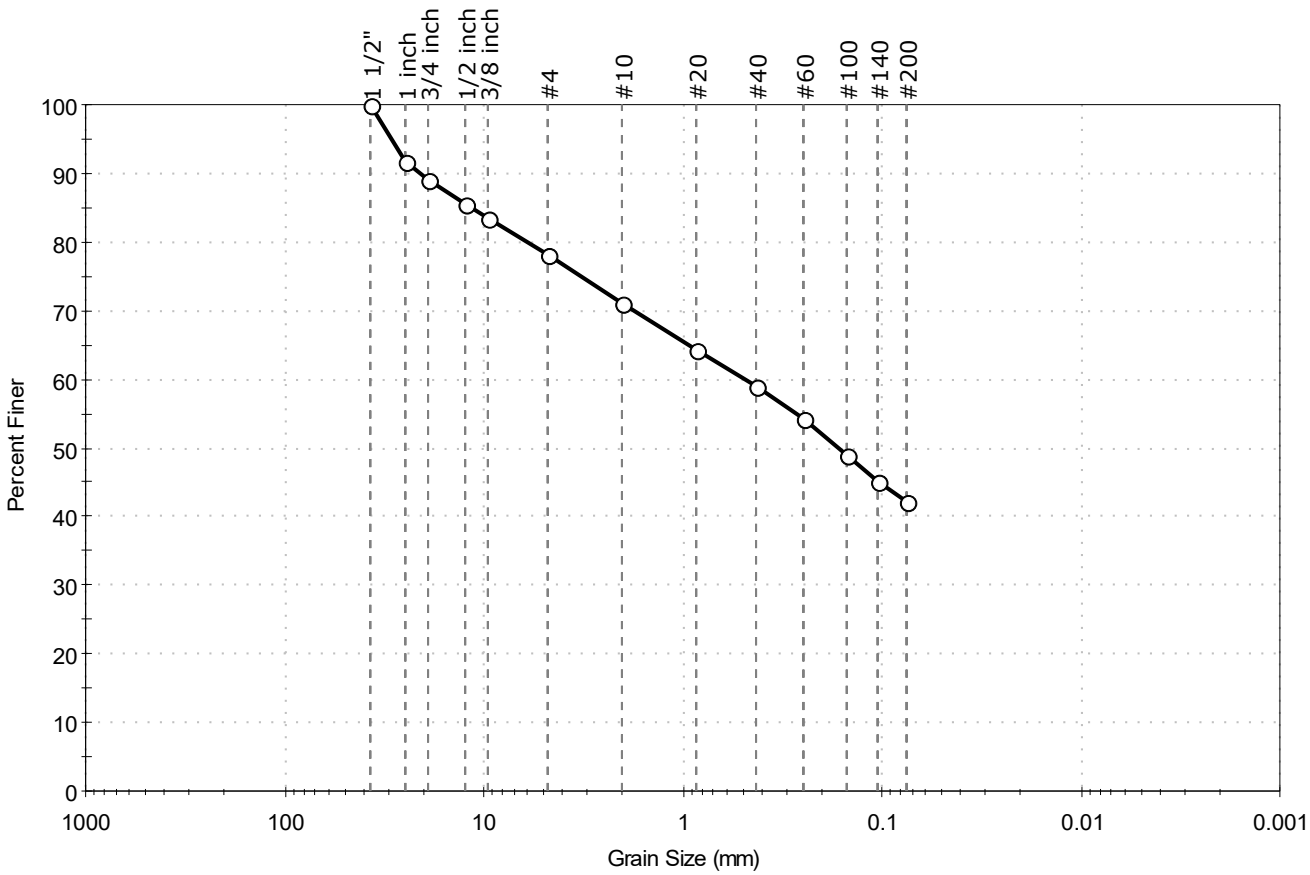
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 8/13/2024 10:29:43 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-103	Sample Type:	Jar	Tested By:	ajl
Sample ID:	4D	Test Date:	08/12/24	Checked By:	ank
Depth :	10-12'	Test Id:	779711		
Test Comment:	---				
Visual Description:	Moist, olive brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	21.8	36.1	42.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2"	37.50	100		
1 inch	25.00	92		
3/4 inch	19.00	89		
1/2 inch	12.50	86		
3/8 inch	9.50	84		
#4	4.75	78		
#10	2.00	71		
#20	0.85	64		
#40	0.42	59		
#60	0.25	54		
#100	0.15	49		
#140	0.11	45		
#200	0.075	42		

Coefficients

D_{85} = 11.5126 mm D_{30} = N/A

D_{60} = 0.4839 mm D_{15} = N/A

D_{50} = 0.1657 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

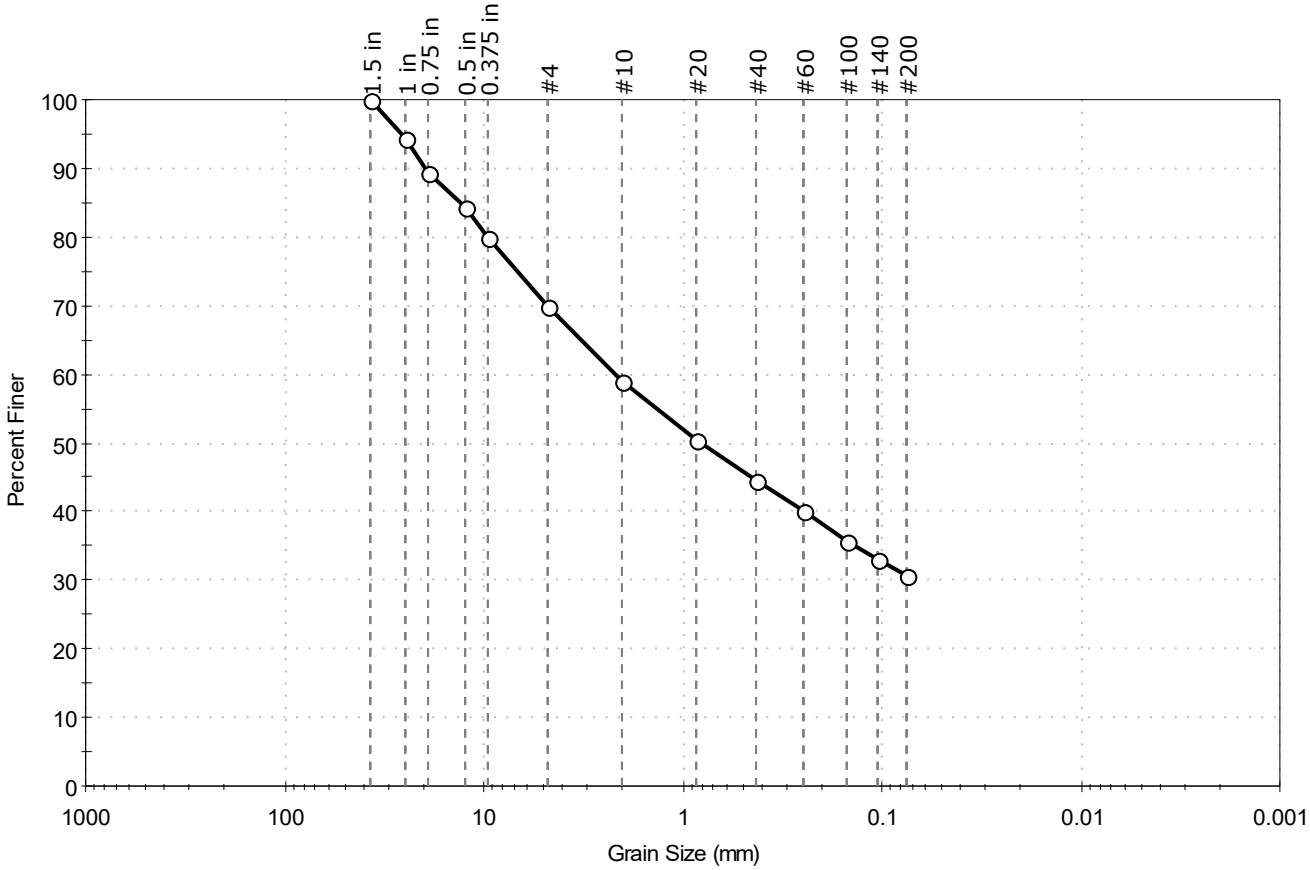
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 8/13/2024 10:29:43 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-103	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (6D, 7D)	Test Date:	08/12/24	Checked By:	ank
Depth :	20-27'	Test Id:	779712		
Test Comment:	---				
Visual Description:	Moist, grayish brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	30.0	39.2	30.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	94		
0.75 in	19.00	89		
0.5 in	12.50	84		
0.375 in	9.50	80		
#4	4.75	70		
#10	2.00	59		
#20	0.85	50		
#40	0.42	45		
#60	0.25	40		
#100	0.15	36		
#140	0.11	33		
#200	0.075	31		

Coefficients

D_{85} = 13.2730 mm D_{30} = N/A
D_{60} = 2.1543 mm D_{15} = N/A
D_{50} = 0.8109 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

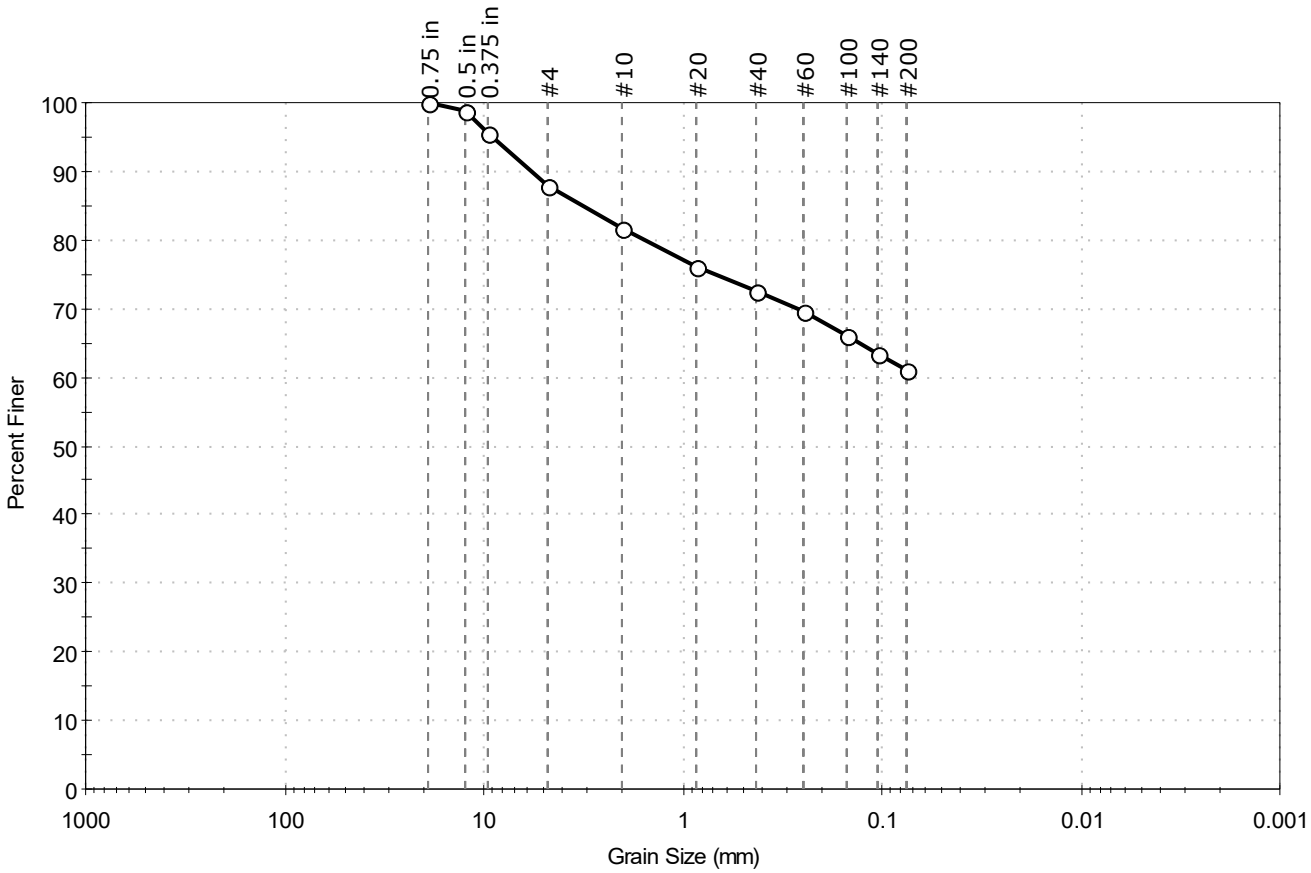
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 8/13/2024 10:29:44 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-103	Sample Type:	Jar	Tested By:	ajl
Sample ID:	Composite (10D, 11D)	Test Date:	08/09/24	Checked By:	ank
Depth :	40-47'	Test Id:	779713		
Test Comment:	---				
Visual Description:	Moist, brownish gray silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.1	26.9	61.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	99		
0.375 in	9.50	96		
#4	4.75	88		
#10	2.00	82		
#20	0.85	76		
#40	0.42	72		
#60	0.25	69		
#100	0.15	66		
#140	0.11	63		
#200	0.075	61		

Coefficients

D_{85} = 3.1830 mm D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

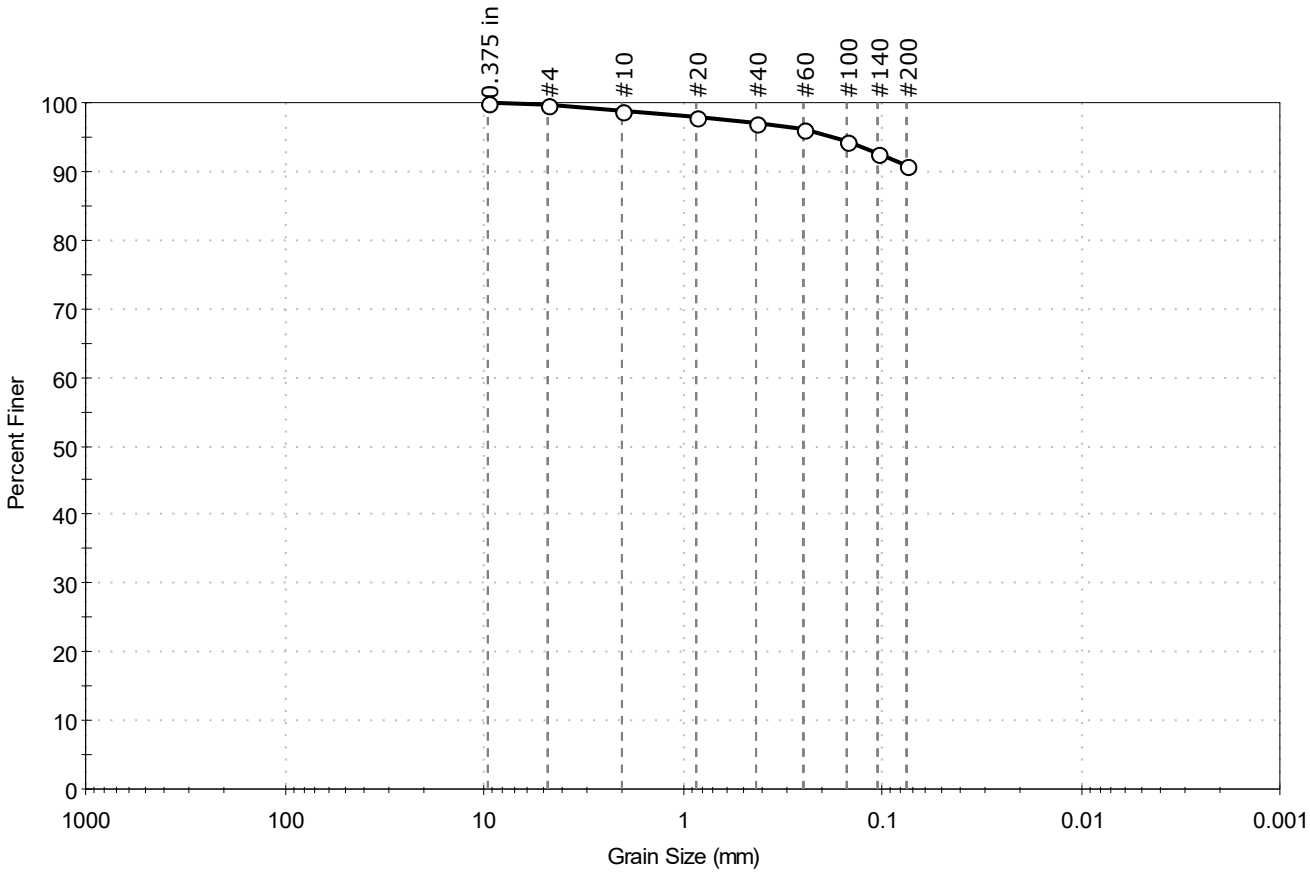
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 8/13/2024 10:29:44 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-103	Sample Type:	Jar	Tested By:	ajl
Sample ID:	12D	Test Date:	08/09/24	Checked By:	ank
Depth :	50-52'	Test Id:	779714		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.4	8.8	90.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	98		
#40	0.42	97		
#60	0.25	96		
#100	0.15	94		
#140	0.11	93		
#200	0.075	91		

Coefficients

D_{85} = N/A D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (13))

Sample/Test Description

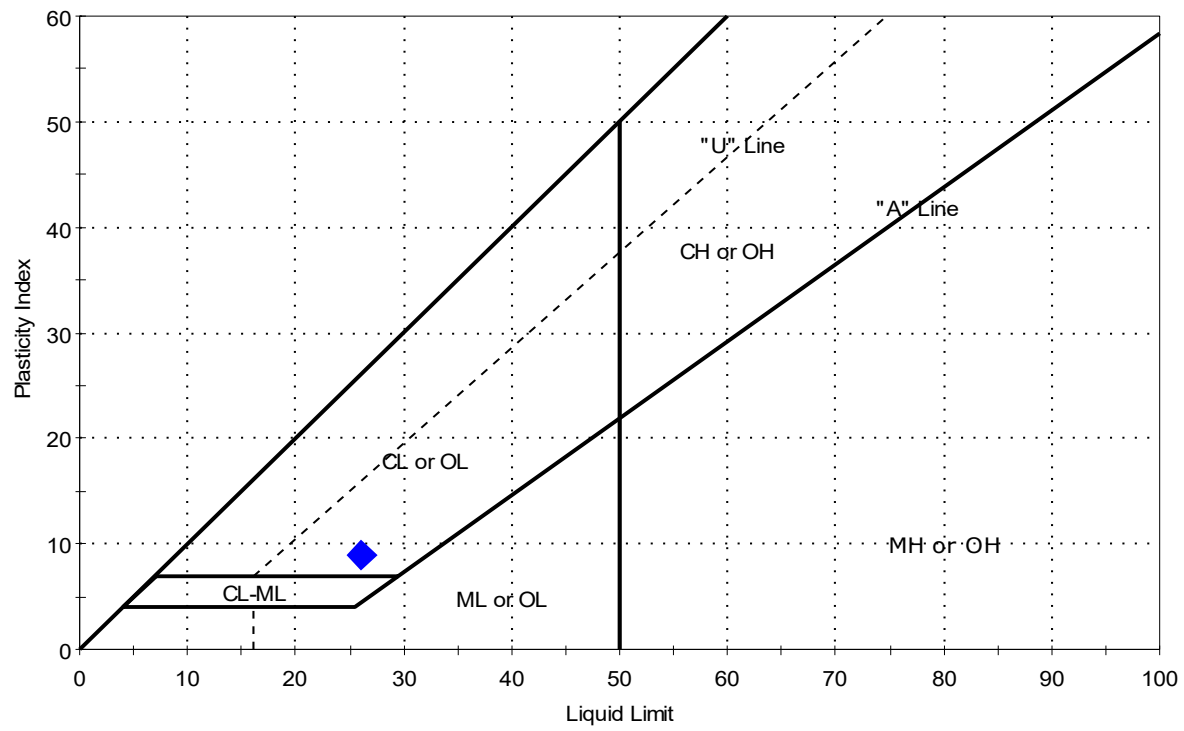
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

printed 8/13/2024 10:29:45 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	cam
Sample ID:	Composite (2D, 3D, 4D)	Test Date:	08/12/24	Checked By:	ank
Depth :	2-11.2'	Test Id:	779692		
Test Comment:	---				
Visual Description:	Moist, brown sandy clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	Composite (2D, 3D, 4D)	B-PI95-10	2-11.2'	14	26	17	9	-0.4	Sandy Lean CLAY (CL)

Sample Prepared using the WET method

29% Retained on #40 Sieve

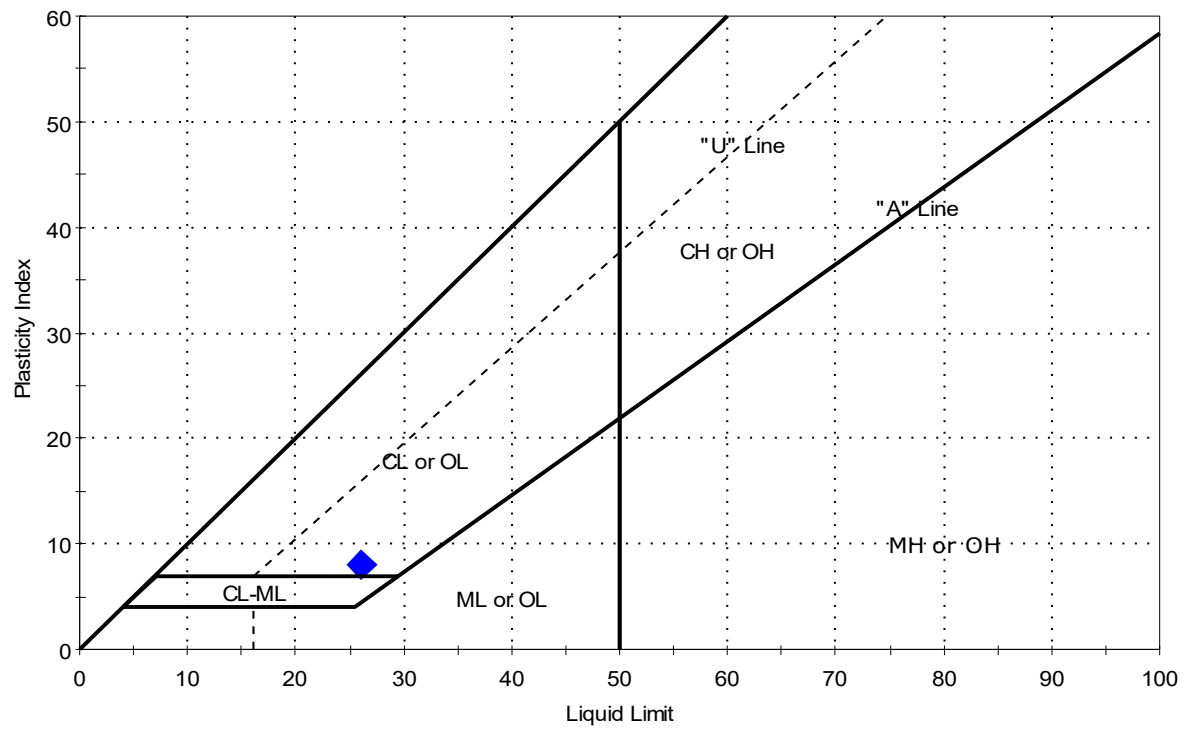
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-101	Sample Type:	Jar	Tested By:	cam
Sample ID:	6D	Test Date:	08/09/24	Checked By:	ank
Depth :	20-22'	Test Id:	779693		
Test Comment:	---				
Visual Description:	Moist, dark gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6D	B-PI95-10	20-22'	22	26	18	8	0.5	Lean CLAY (CL)

Sample Prepared using the WET method

1% Retained on #40 Sieve

Dry Strength: VERY HIGH

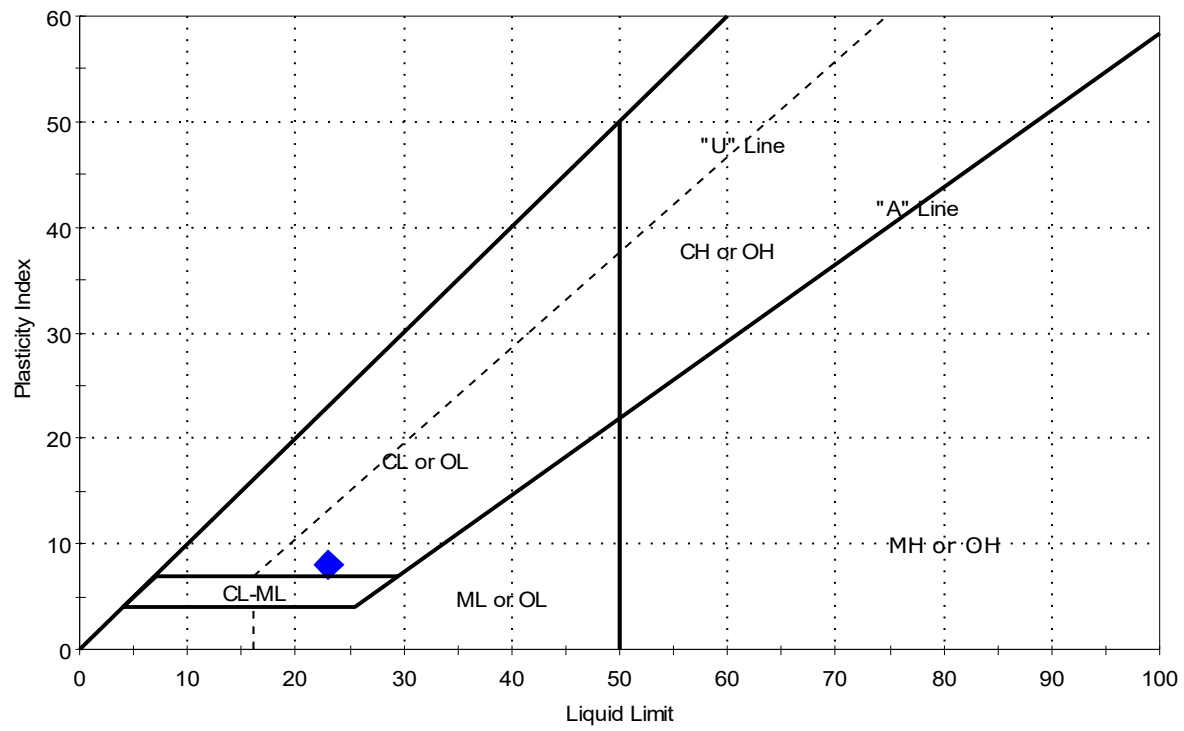
Dilatancy: SLOW

Toughness: LOW

printed 8/13/2024 10:30:23 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	cam
Sample ID:	3D	Test Date:	08/09/24	Checked By:	ank
Depth :	5-7'	Test Id:	779694		
Test Comment:	---				
Visual Description:	Moist, light olive brown sandy clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-PI95-10	5-7'	10	23	15	8	-0.7	Sandy Lean CLAY (CL)

Sample Prepared using the WET method

31% Retained on #40 Sieve

Dry Strength: HIGH

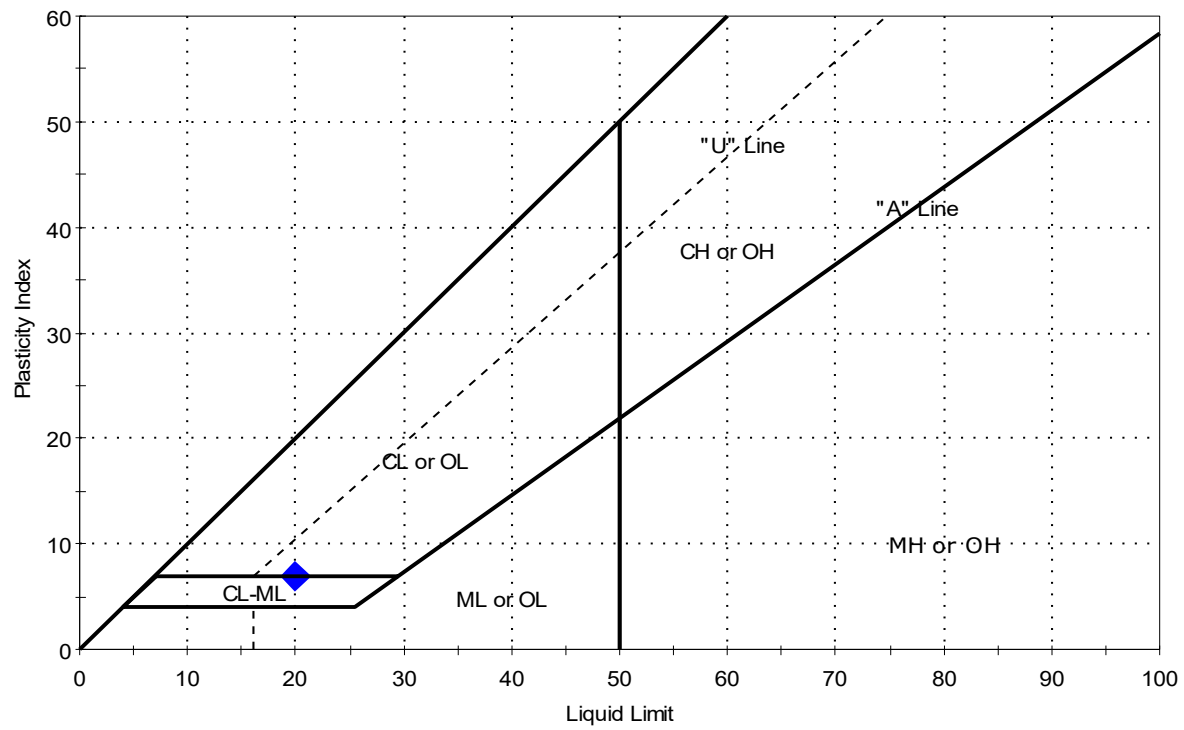
Dilatancy: SLOW

Toughness: LOW

printed 8/13/2024 10:30:23 AM

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	cam
Sample ID:	Composite (4D, 5D)	Test Date:	08/12/24	Checked By:	ank
Depth :	10-17'	Test Id:	779695		
Test Comment:	---				
Visual Description:	Moist, olive brown silty, clayey sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	Composite (4D, 5D)	B-PI95-10	10-17'	9	20	13	7	-0.5	Silty, Clayey SAND (SC-SM)

Sample Prepared using the WET method

34% Retained on #40 Sieve

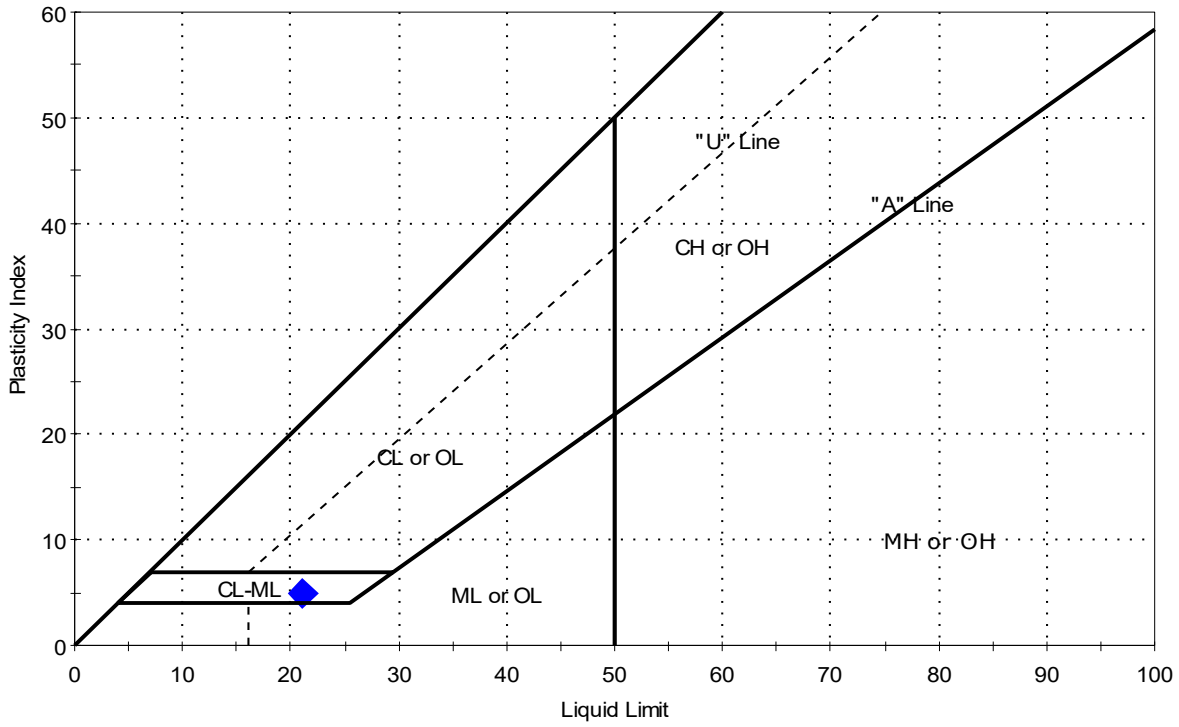
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-102	Sample Type:	Jar	Tested By:	cam
Sample ID:	7D	Test Date:	08/12/24	Checked By:	ank
Depth :	25-27'	Test Id:	779696		
Test Comment:	---				
Visual Description:	Moist, grayish brown silty, clayey sand with gravel				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	7D	B-PI95-10	25-27'	9	21	16	5	-1.3	Silty, Clayey SAND with Gravel (SC-SM)

Sample Prepared using the WET method

54% Retained on #40 Sieve

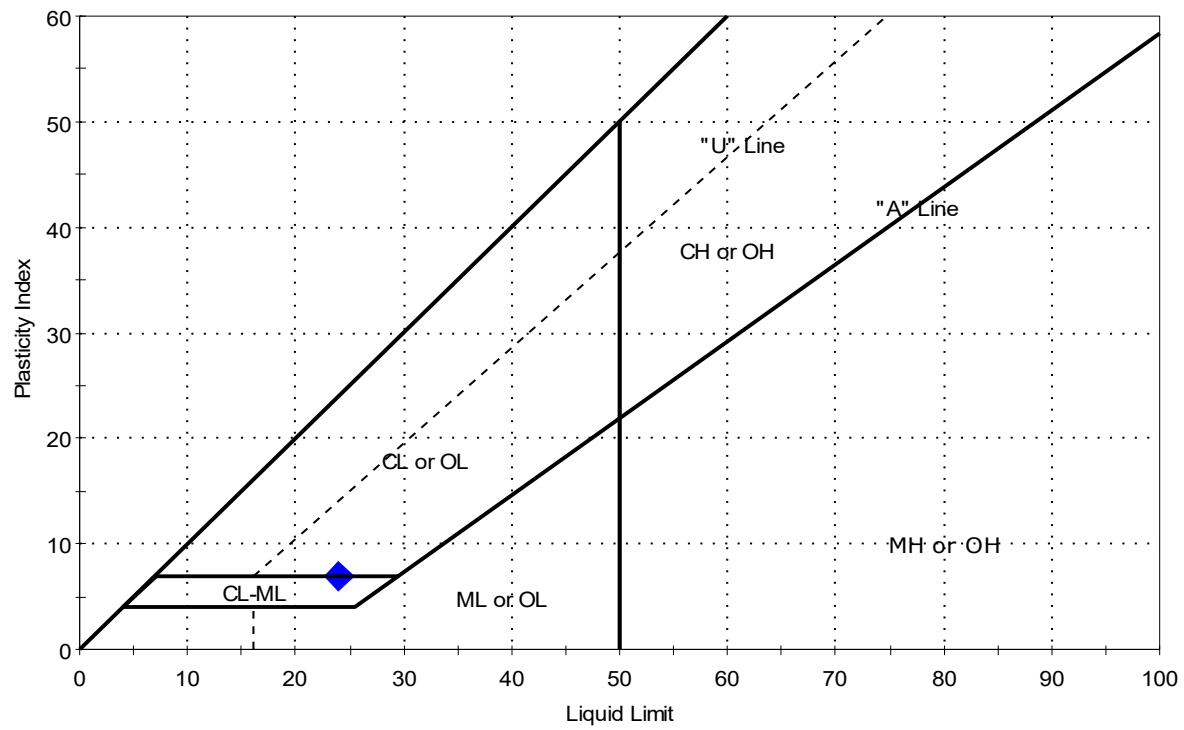
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-103	Sample Type:	Jar	Tested By:	cam
Sample ID:	Composite (2D, 3D)	Test Date:	08/12/24	Checked By:	ank
Depth :	2-6'	Test Id:	779697		
Test Comment:	---				
Visual Description:	Moist, olive brown silty, clayey sand with gravel				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	Composite (2D, 3D)	B-PI95-10	2-6'	13	24	17	7	-0.5	Silty, Clayey SAND with Gravel (SC-SM)

Sample Prepared using the WET method

43% Retained on #40 Sieve

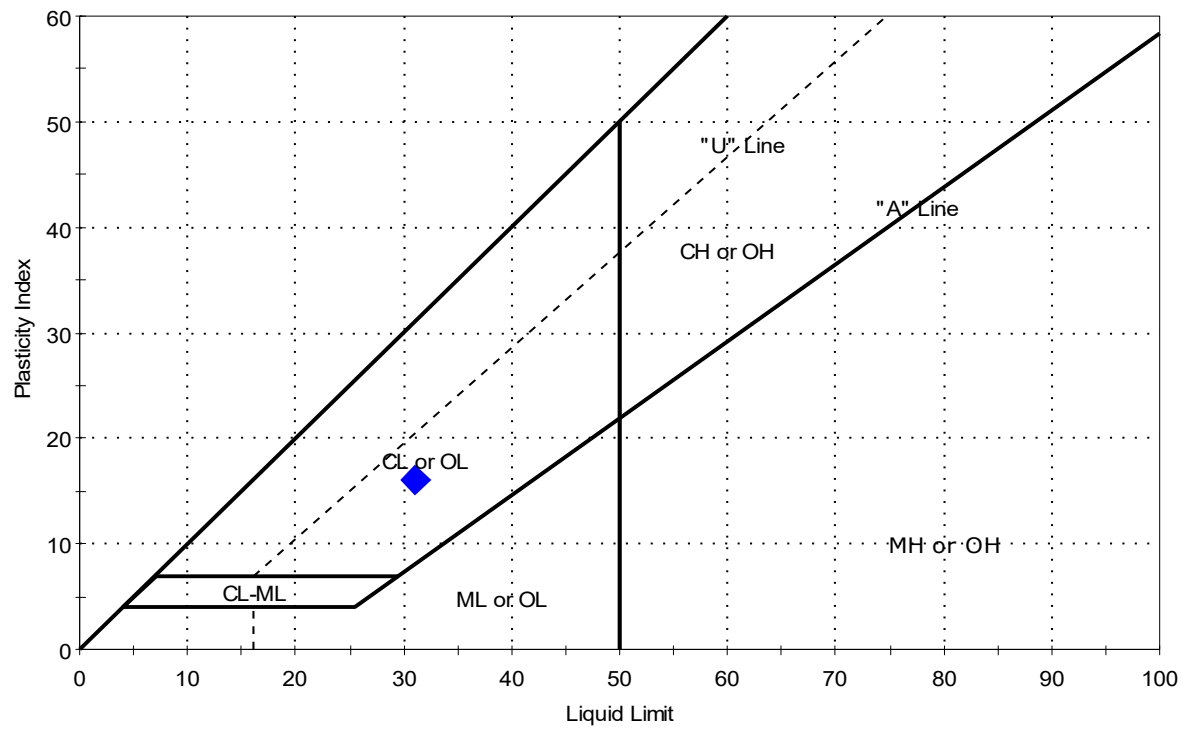
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	Route 100 Bridge No. 5957				
Location:	Palmyra, ME			Project No:	GTX-319517
Boring ID:	BB-PI95-103	Sample Type:	Jar	Tested By:	cam
Sample ID:	12D	Test Date:	08/09/24	Checked By:	ank
Depth :	50-52'	Test Id:	779698		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	12D	B-PI95-10	50-52'	19	31	15	16	0.2	Lean CLAY (CL)

Sample Prepared using the WET method

3% Retained on #40 Sieve

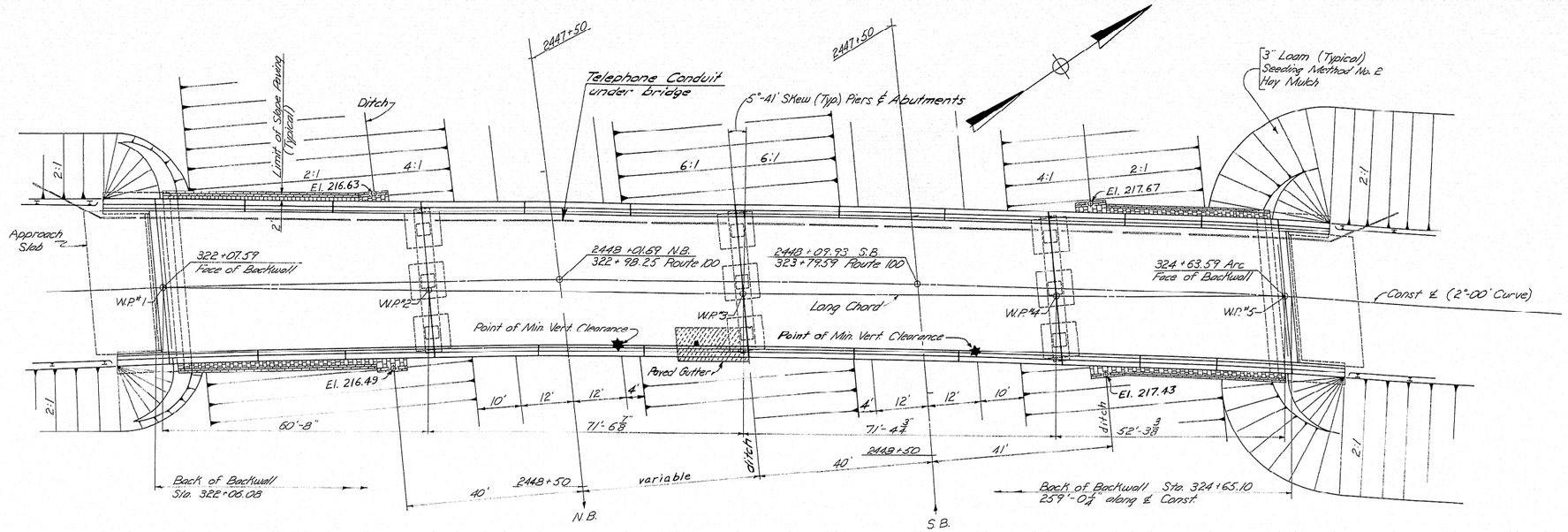
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

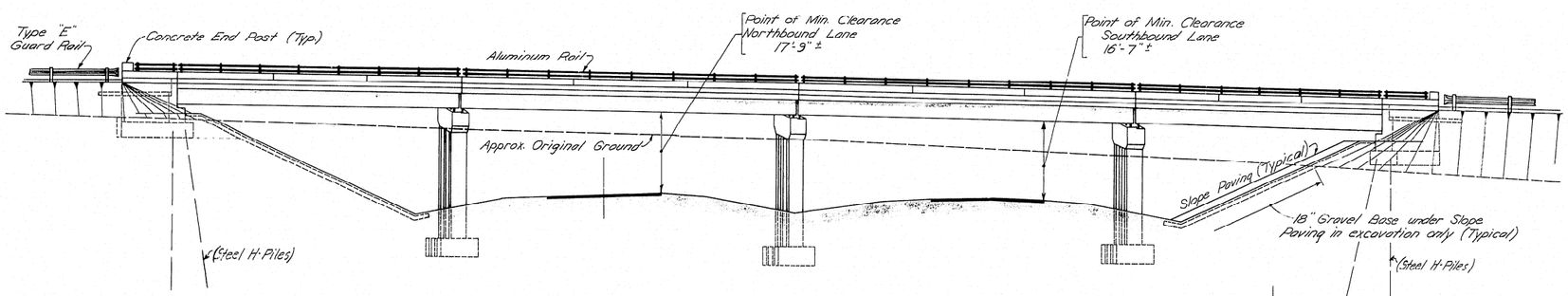
APPENDIX C
Historic Drawings

B. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-95-9(29	11	68



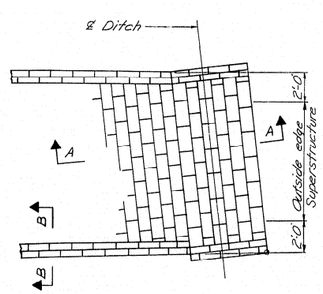
PLAN

Scale 1" = 15'

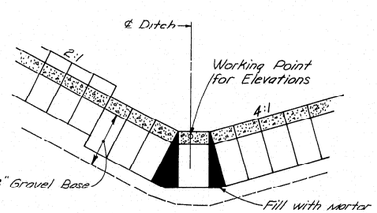


ELEVATION

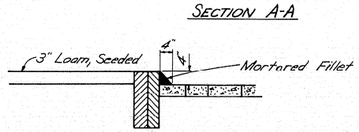
Scales - Vert. & Horiz. 1" = 15'



PLAN



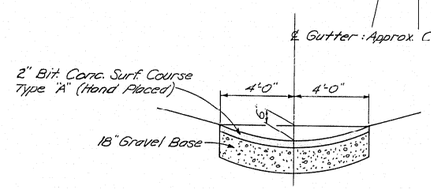
SECTION A-A



SECTION B-B

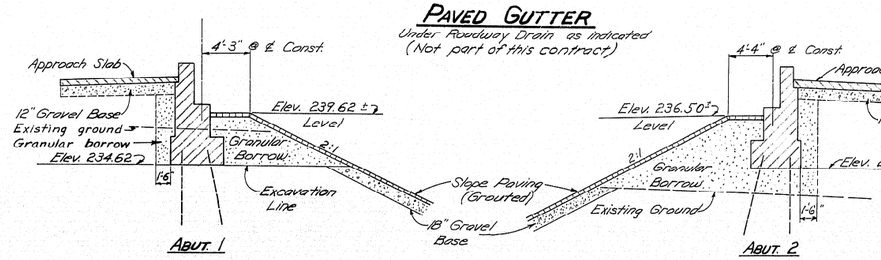
SLOPE PAVING

The 18" Gravel Base under Slope Paving may be reduced or omitted if in the opinion of the Engineer the existing material is suitable. Payment for Excavation for Gravel Base under Slope Paving is to be made under Item 203-4, Earth Excavation.



PAVED GUTTER

Under Roadway Drain as indicated (Not part of this contract)



NORMAL SECTIONS

(Perpendicular to Abutments)

At Abut. 2 place Granular Borrow to El. 230.58 before driving pile. No stones to be larger than 6" in area where piles are to be driven.

ESTIMATE OF BRIDGE QUANTITIES

DESCRIPTION	QUANTITIES
Earth Excavation	155 c.y.
Structural Earth Excavation, Abutments & Retaining Walls	125 c.y.
Structural Earth Excavation, Piers	240 c.y.
Granular Borrow	450 c.y.
Gravel Base Course - In Place Measurement	190 c.y.
Bituminous Concrete Surface Course, Type "A"	95 tons
Portland Cement Concrete, Abutments & Retaining Walls	196 c.y.
Portland Cement Concrete, Piers	185 c.y.
Portland Cement Concrete, Roadway & Sidewalk Slabs on Str. Bridges	293 c.y.
Portland Cement	1010 bbl.
Structural Steel, Fabricated & Delivered	238,700 lbs.
Structural Steel, Erection	238,700 lbs.
Bronze or Copper-Alloy Bearing & Expansion Plates, Delivered	178 lbs.
Bronze or Copper-Alloy Bearing & Expansion Plates, Placing	178 lbs.
Reinforcing Steel, Delivered	91,460 lbs.
Reinforcing Steel, Placing	91,460 lbs.
Shear Connectors	L.S.
Steel H-Beam Piles, 42 lbs per foot	1015 l.f.
Structural Steel, Field Painting	238,700 lbs.
Aluminum Rail	545 l.f.
Membrane Waterproofing	875 s.y.
Slope Paving	455 s.y.
Granite Bridge Curb	560 l.f.

SPECIFICATIONS

DESIGN: A.A.S.H.O. Standard Specifications for Highway Bridges 1961 and interim revisions.

CONTRACT: State of Maine, State Highway Commission, Standard Specifications for Highway and Bridges, Revision of Jan. 1956. and Supplemental Specifications.

LIVE LOADING: H20-S16-44

ALLOWABLE STRESSES:
Structural Steel A.S.T.M. Designation A36 - f_s = 20,000 p.s.i.
Structural Steel A.S.T.M. Designation A7 - f_s = 18,000 p.s.i.
Intermediate Grade Reinforcing Steel - f_s = 20,000 p.s.i.
Concrete (n = 10) - f_c = 1200 p.s.i.

CONCRETE CLASSIFICATION:
All Concrete _____ Class "A"

TRAFFIC

A.D.T. 1962	2565
A.D.T. 1982	34.95
D.H.V.	419
T.	11%
D.	60%
V.	60 m.p.h.

DESIGN - COFFIN DET - R.B.P.
TRACE - R.G.G.
CHECK - DRESELLY
BRIDGE NO.
SURVEY -
PLOT -

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

ROUTE NO. 100 BRIDGE
OVER
INTERSTATE NO. 95
IN THE TOWN OF
PALMYRA, SOMERSET COUNTY

GENERAL PLAN

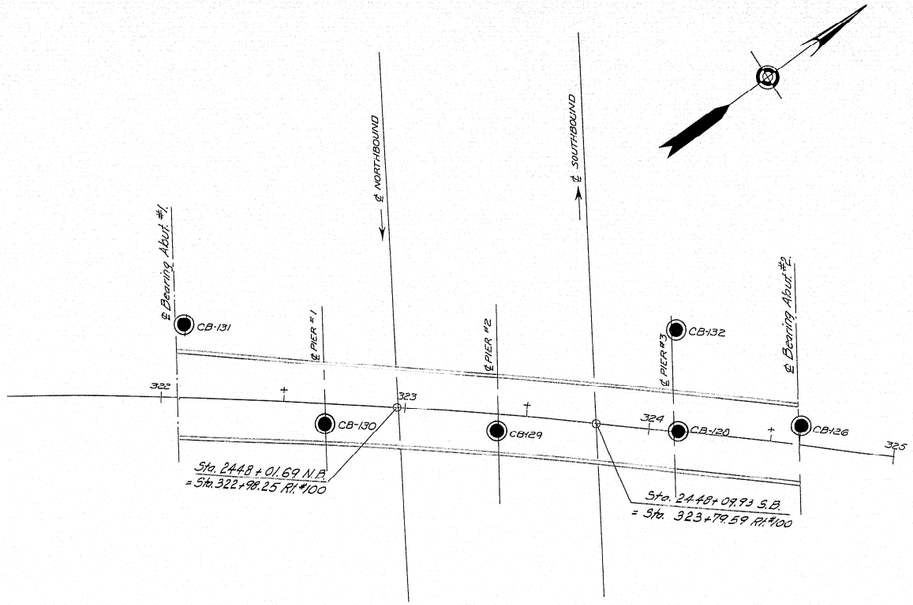
SHEET 1 OF 15 AUGUSTA, MAINE June 1962

M-1895

11

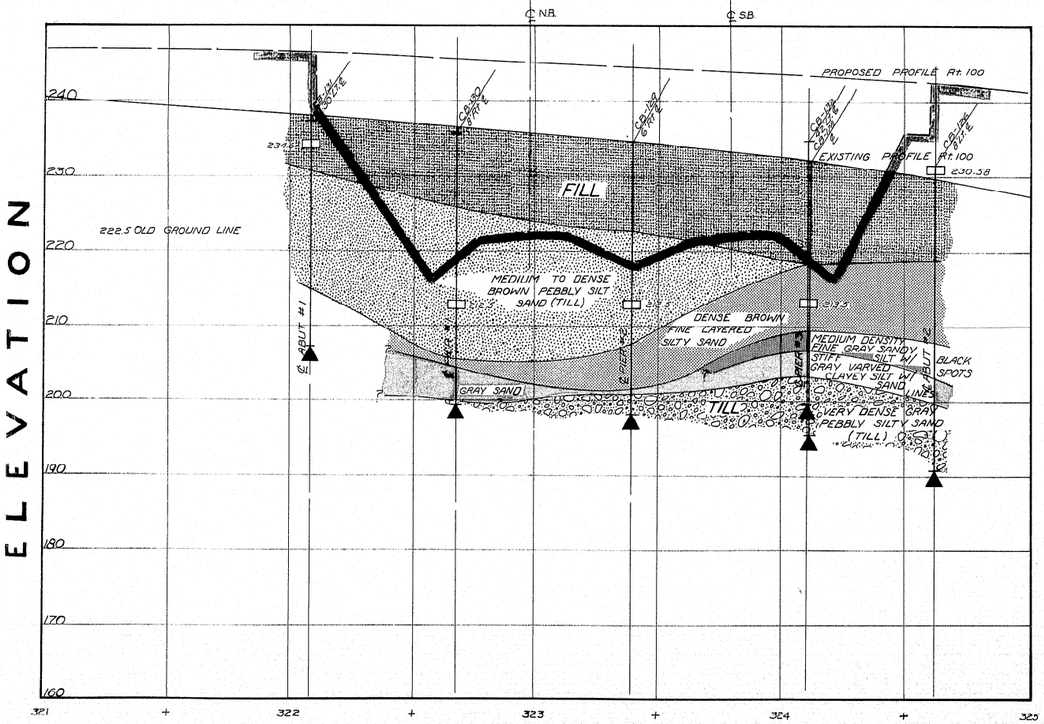
K+E 135 IMPERIAL U. L. M. INC.

B. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-95-7(29	12	68



PLAN

SCALE 1" = 30'



PROFILE

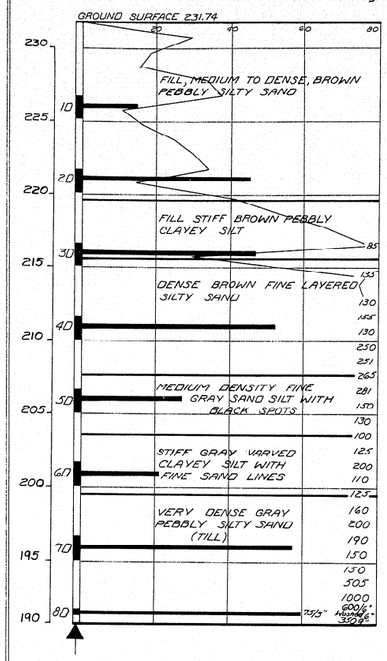
SCALE HOR. 1" = 30'
VERT. 1" = 10'

DRIVING RESISTANCE

BLOWS / FOOT

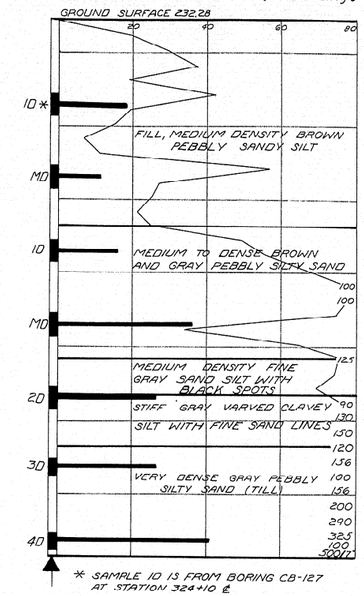
BORING CB-126

STATION 324+63 8' Lt. (2½ Casing)



BORING CB-128

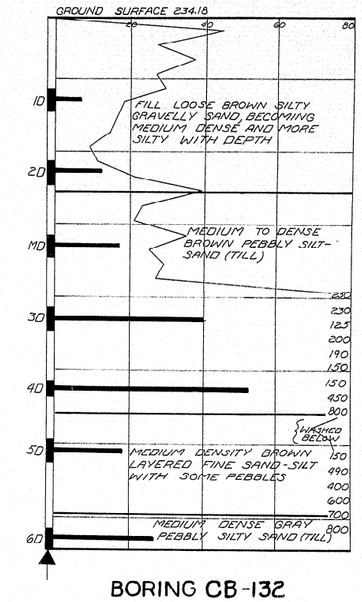
STATION 324+12 ℄ (2½ Casing)



* SAMPLE 10 IS FROM BORING CB-127 AT STATION 324+10 ℄

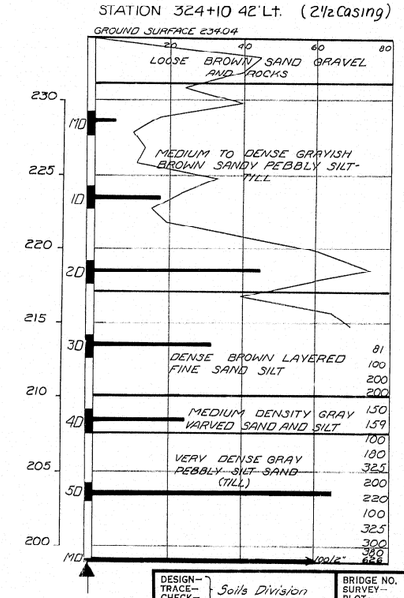
BORING CB-129

STATION 323+39 6' Rt. (2½ Casing)



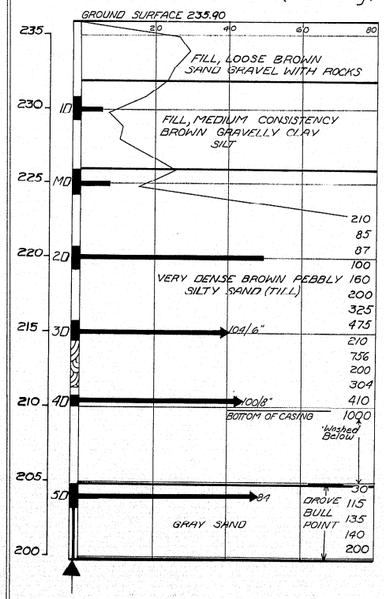
BORING CB-132

STATION 324+10 42' Lt. (2½ Casing)



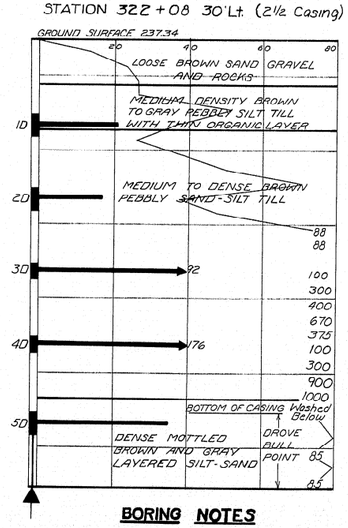
BORING CB-130

STATION 322+68 8' Rt. (2½ Casing)



BORING CB-131

STATION 322+08 30' Lt. (2½ Casing)



BORING NOTES

ALL SAMPLES AND VANES ARE MADE AHEAD OF CASING
NUMBER OF BLOWS REQUIRED TO DRIVE EXTRA HEAVY CASING ONE FOOT WITH 400 ft. lbs. OF ENERGY PER BLOW
LOCATION OF SAMPLE OR SAMPLE ATTEMPT
NUMBER AND TYPE OF DRY SAMPLE
S & H SAMPLER #1290'S
UNSUCCESSFUL SAMPLE ATTEMPT AND TYPE OF SAMPLER
NUMBER OF BLOWS REQUIRED TO DRIVE SPOON OR TUBING ONE FOOT WITH 350 ft. lbs. OF ENERGY PER BLOW
BOTTOM OF BORING (MAY NOT BE BOTTOM OF SOIL STRATA)

DESIGN— TRACE— CHECK— } Soils Division

BRIDGE NO.
SURVEY—
PLOT—

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

ROUTE NO. 100
OVER
INTERSTATE NO. 95
IN THE TOWN OF
PALMYRA
SOMERSET COUNTY
FOUNDATION SURVEY

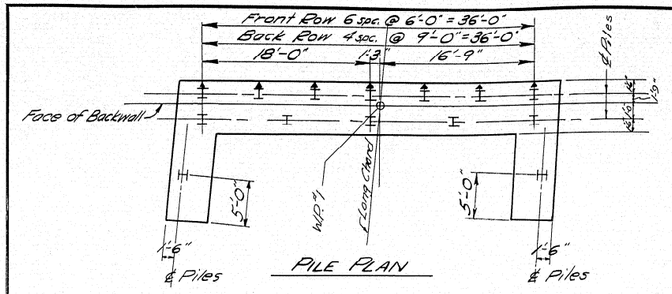
SHEET 2 OF 15 AUGUSTA, MAINE June 1962

M-1897

12

G. D. CO., NO. 3799 88767

R. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-95-4(29	13	68



PILE PLAN

PILE NOTES

All piles to be 10BP42.

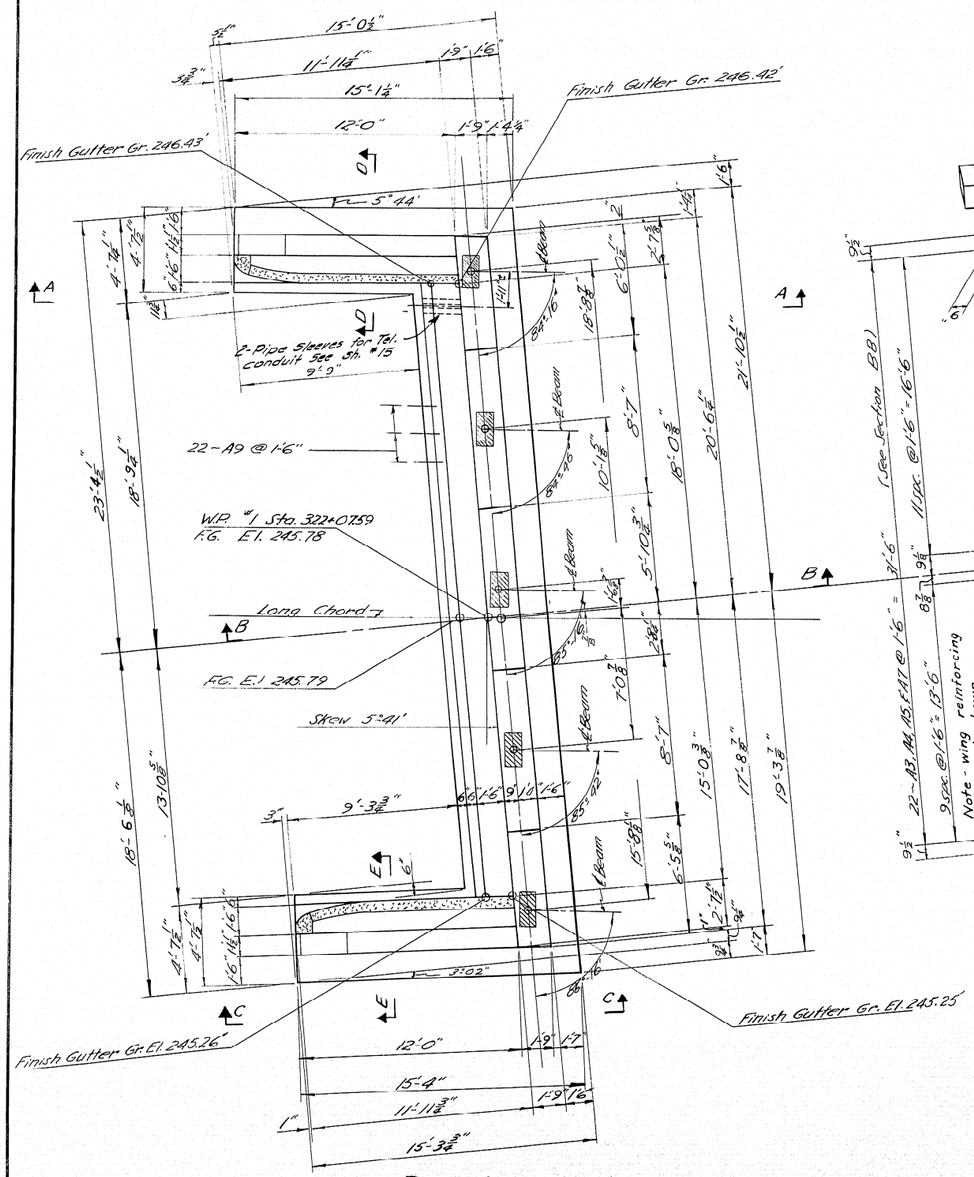
Piles indicated thus batter 2½"/ft. in the direction indicated by the arrow.

Max. pile load 37 tons.

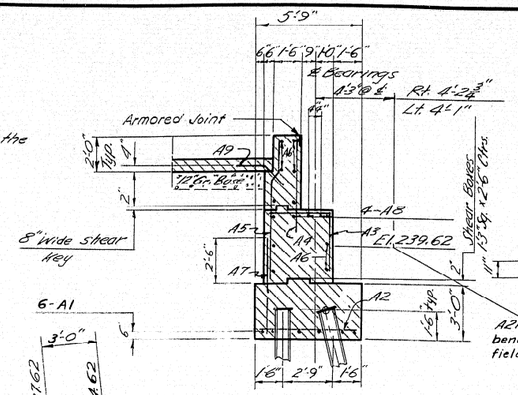
Estimated pile length 25 feet.

14 Pile required with caps.

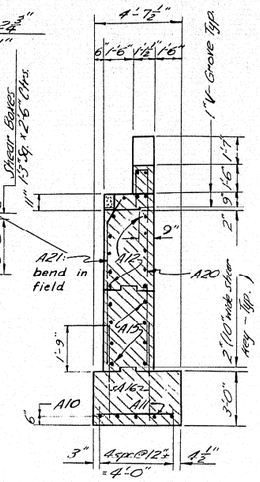
See Sheet No. 4. for pile cap detail.



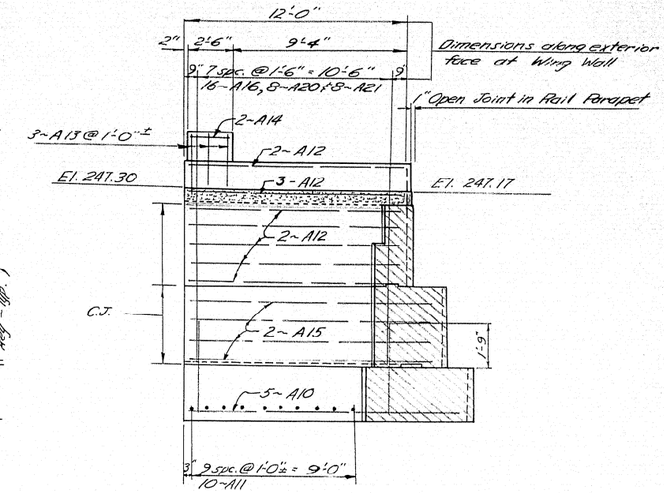
PLAN ABUT. NO. 1



SECTION B-B

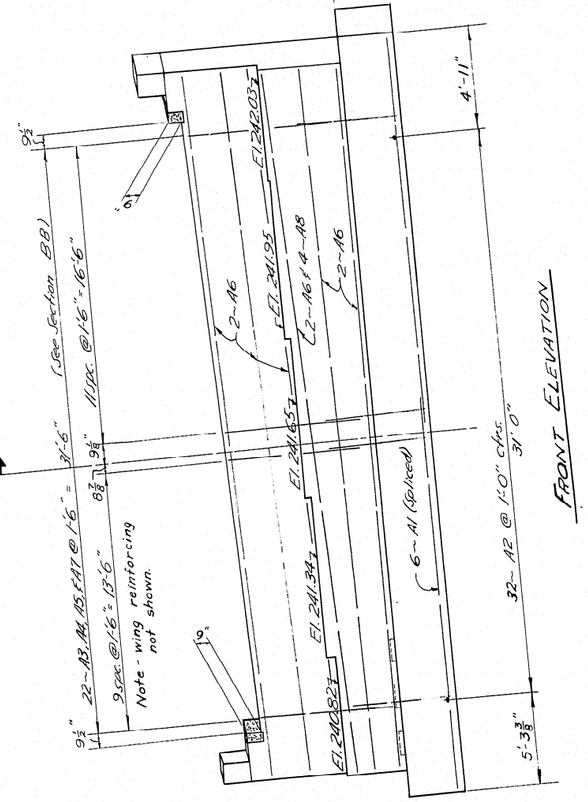


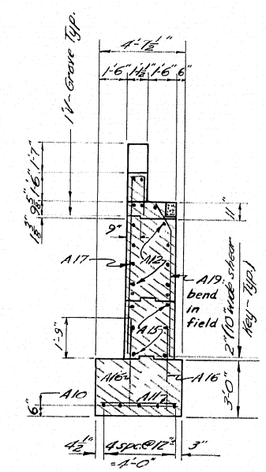
SECTION D-D



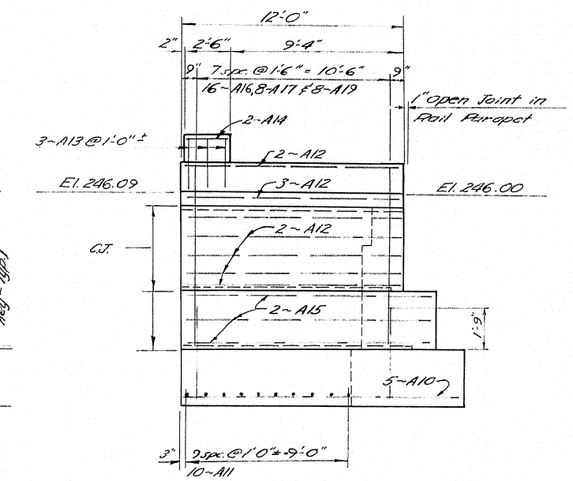
SECTION A-A

Note - Wing piles and abutment reinforcing not shown in Sections A-A, D-D, E-E; and View C-C.

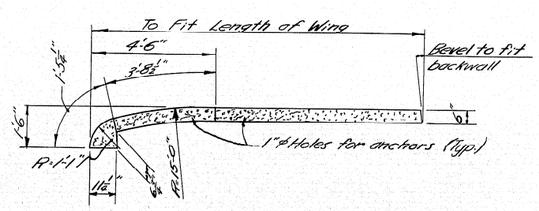




SECTION E-E



VIEW C-C



GRANITE CURB LAYOUT

Required at each abutment - 1 as shown; 1 opp. hand.
NOTE: Payment for Circular Granite Bridge Curb to be made under Item 901-21, Granite Bridge Curb.

GENERAL NOTES

1. 2" clear to reinforcing except as noted.
2. Dress shaded bearing areas on bridge seats 1" larger all around than masonry plates and to exact elevations shown.
3. Aluminum bridge railing not shown.
4. References:
   a. Aluminum Rail — Sh. # 14
   b. Reinforcing Steel — Sh. # 15
   c. Granite Curb Detail — Sh. # 14
   d. Armored Joint Detail — Sh. # 8 & #10
   e. Fill @ Abuts. — Sh. # 1
5. Anchor bolts for aluminum railing to be in place before concrete for rail parapet is placed.
6. Reinforcing steel to be placed to clear piles.
7. Caulk around edges of masonry plates with an approved caulking material. Payment for this work shall be incidental to the contract items. Typical for abutments and piers.

DESIGN- Darity TRACE- Evans CHECK- Drosella	BRIDGE NO. SURVEY- PLOT-

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

ROUTE NO. 100 BRIDGE

OVER

INTERSTATE NO. 95

IN THE TOWN OF

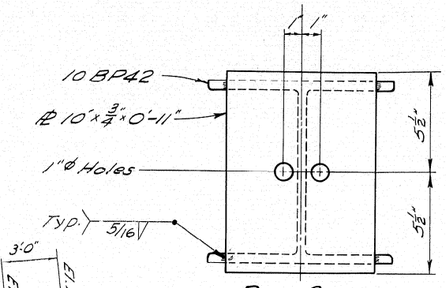
PALMYRA, SOMERSET COUNTY

ABUTMENT NO. 1 DETAIL

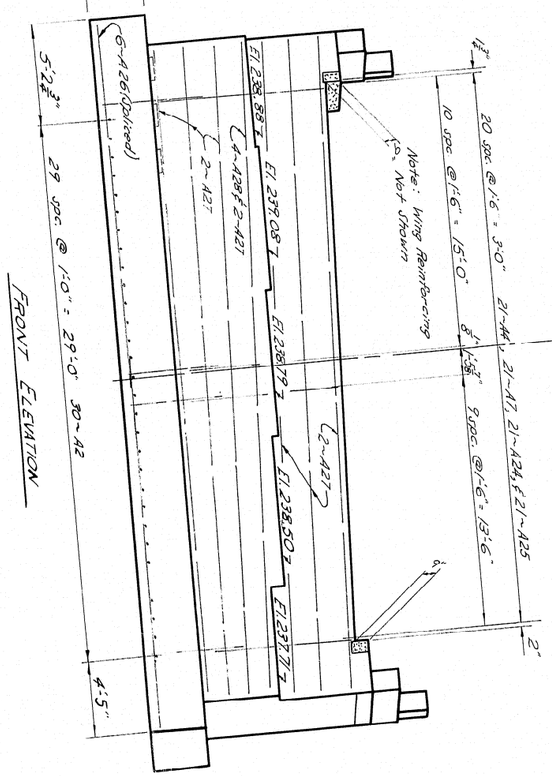
SHEET 3 OF 15 AUGUSTA, MAINE June 1962

M-1898

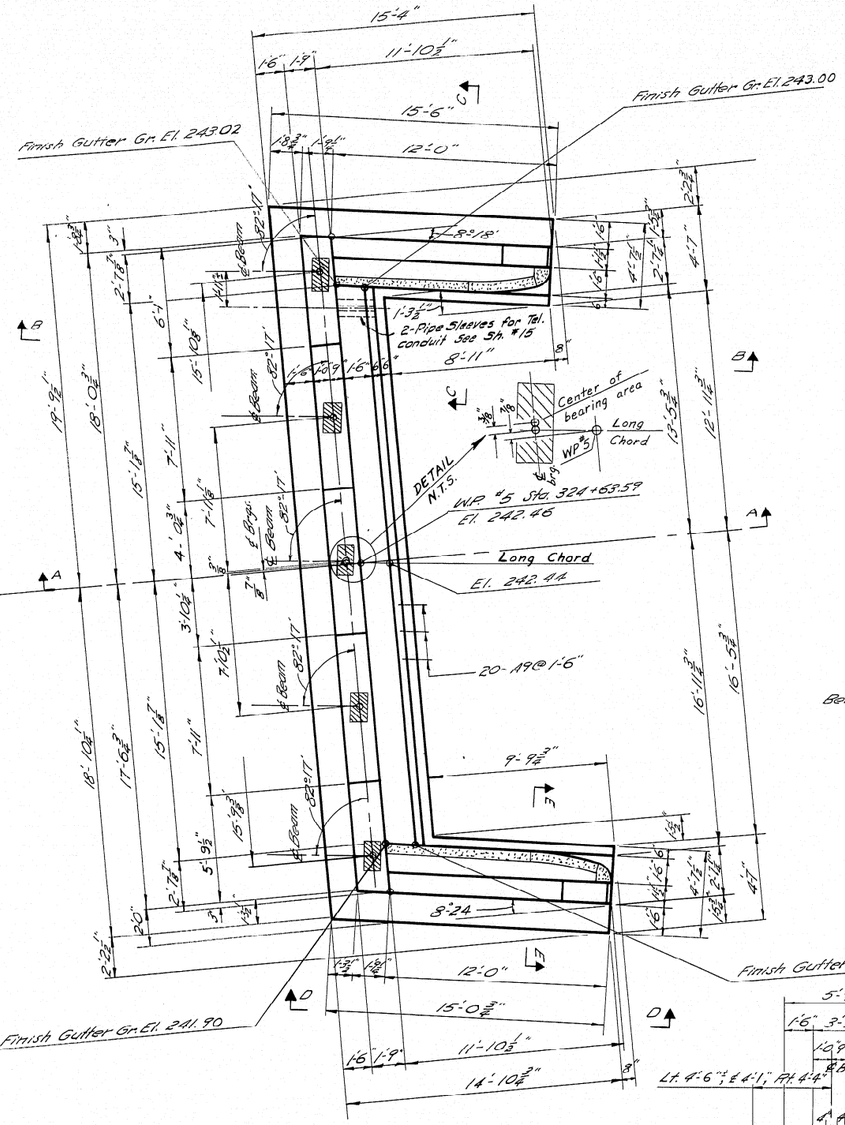
B. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-95-7(29)	14	68



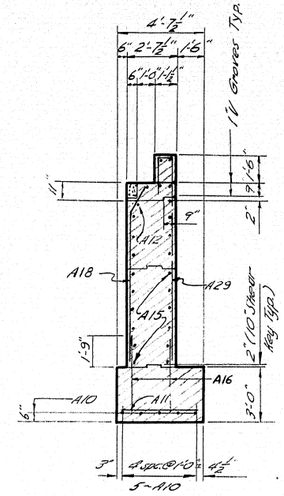
PILE CAP
27 Required



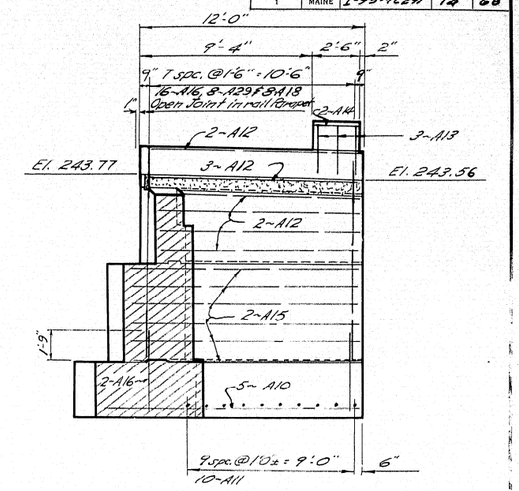
FRONT ELEVATION



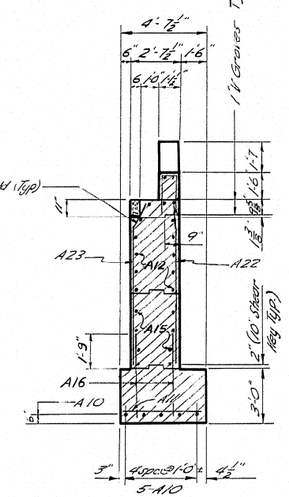
PLAN ABUTMENT No. 2



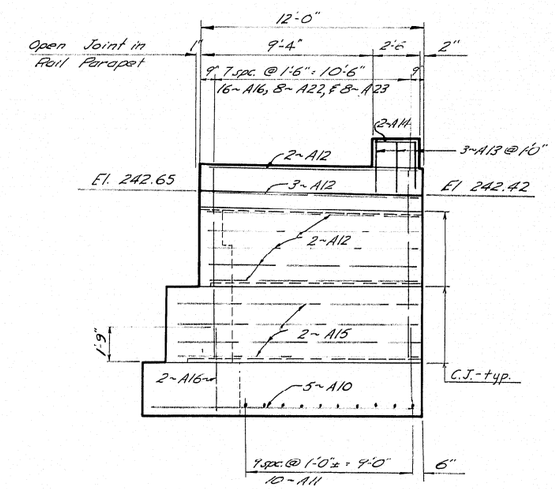
SECTION C-C



SECTION B-B
Note: Wing reinforcing only shown.

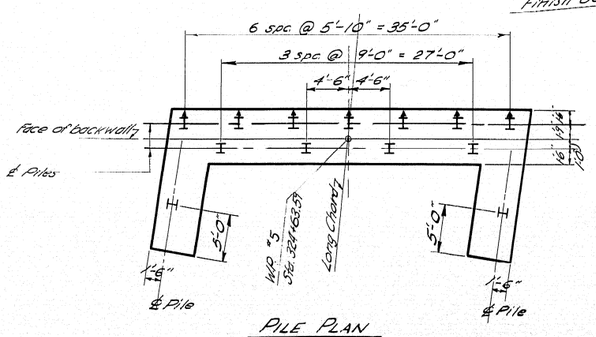


SECTION E-E



VIEW D-D
Note: Wing reinforcing only shown.

Note: See sheet No. 3 for General Notes.



PILE PLAN

PILE NOTES

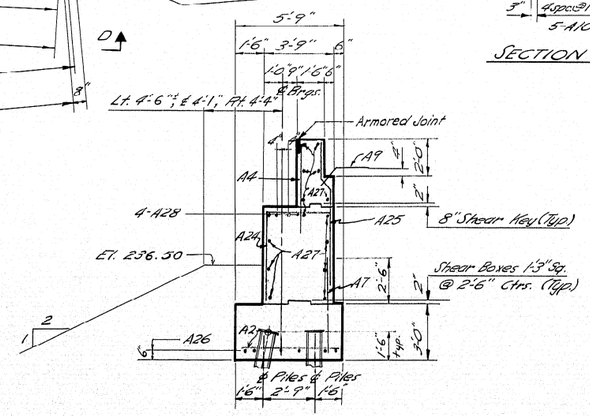
All piles to be 10BP42.

Pile indicated thus ⊥ batter 2 1/2"/ft. in the direction indicated by the arrow.

Max. pile load 37 tons.

Estimated pile length 45 feet.

13 piles required with caps.



SECTION A-A

DESIGN— Darrily Lyon TRACE— CHECK— Dresnally	BRIDGE NO. SURVEY— PLOT—

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

ROUTE NO. 100 BRIDGE
OVER
INTERSTATE NO. 95
IN THE TOWN OF
PALMYRA, SOMERSET COUNTY

ABUTMENT NO. 2 DETAIL

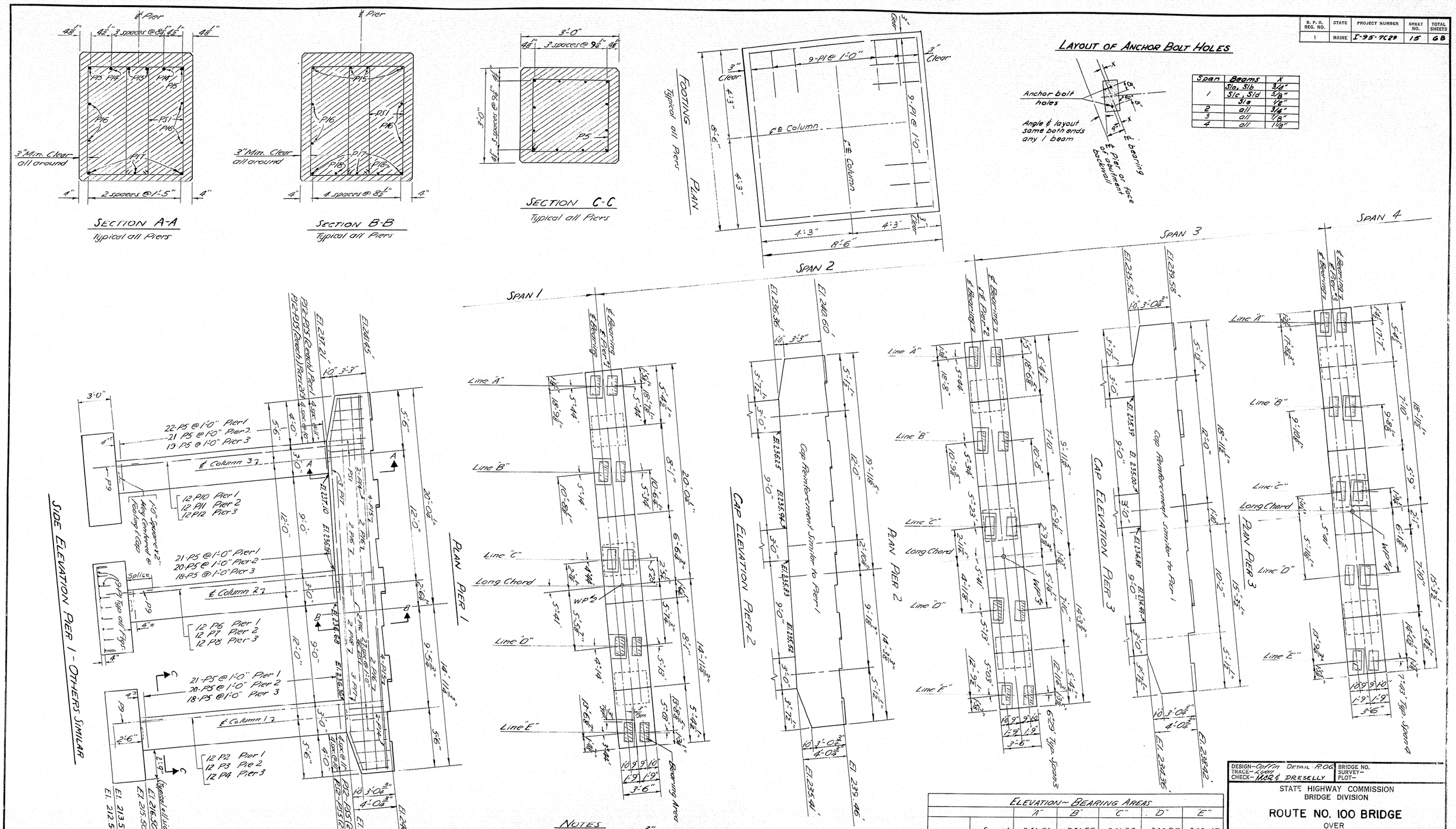
SHEET 4 OF 15 AUGUSTA, MAINE June 1962

M-1899

14

K+E 168 IMPERIAL TRACING CLOTH 5 X. D. L. M., INC. 67027

D. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-95-7(28)	15	68



Span	Beams	X
1	S1a, S1b	3/4"
	S1c, S1d	5/8"
	S1e	1/2"
2	all	3/4"
3	all	1/8"
4	all	1 1/8"

NOTES

Chamfer all exposed edges of concrete 3/4".

Dress all shaded bearing areas 1" larger all around, than masonry bearing plates and to exact elevations shown in chart.

Maximum foundation pressure = 2.3 T/□'

ELEVATION-- BEARING AREAS

		"A"	"B"	"C"	"D"	"E"
PIER 1	Span 1	241.61	241.55	241.26	240.98	240.47
	Span 2	241.45	241.16	240.88	240.59	240.30
PIER 2	Span 2	240.78	240.50	240.22	239.93	239.65
	Span 3	240.60	240.31	240.03	239.75	239.46
PIER 3	Span 3	239.74	239.46	239.17	238.88	238.60
	Span 4	239.58	239.18	239.49	239.20	238.42

DESIGN-Coffin TRACE-Lyon CHECK-MAR & DRESELLY DETAIL R.O.G. BRIDGE NO. SURVEY- PLOT-

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

ROUTE NO. 100 BRIDGE
OVER
INTERSTATE NO. 95
IN THE TOWN OF
PALMYRA, SOMERSET COUNTY
PIERS

SHEET 5 OF 15 AUGUSTA, MAINE June 1962

M-[redacted] 1900 (15)

K+E 100 IMPERIAL D. L. H. INC.

APPENDIX D
Geotechnical Calculations

Soil Properties

PROBLEM STATEMENT & OBJECTIVE

Determine strength parameters (friction angle and undrained shear strength) based on typical engineering correlations.

EXECUTIVE SUMMARY

Refer to the attached strength parameter plots for recommended values.

REFERENCES

1. Friction Angle calculated per Peck, Hansen, and Thornburn (1974): $-0.0016*(N_{60})^2 + 0.3726*N_{60}+26.286$
2. $S_{u,1}$ calculated per Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Terzaghi and Peck 1967
3. $S_{u,2}$ calculated per Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Tschebotarioff 1973

AVAILABLE INFORMATION

1. Boring logs BB-PI95-101, -102, -103, -201, -202, and -203.
2. Geotechnical laboratory testing results.

CALCULATIONS

1. Calculated strength values are indicated on the following summary tables.

CONCLUSION

1. See strength plots (Sheet No 8).

Boring BB-PI95-101

Sample No.	Sample El. (ft, NAVD 88)	Geologic Deposit	USCS Symbol	Friction Angle (ϕ', deg.)	Undrained Shear Strength (S_u, psf)		
					$S_{u,1}$	$S_{u,2}$	$S_{u,avg}$
1D	229.1	FILL	ML	#N/A	4,753	5,709	5,231
2D	227.1	FILL	CL	#N/A	4,114	4,942	4,528
3D	224.1	FILL	CL	#N/A	4,913	5,901	5,407
4D	219.1	FILL	CL	#N/A	7,948	9,547	8,747
5D	214.1	GLACIOLAC. DEP.	ML	#N/A	5,392	6,477	5,934
6D	209.1	GLACIOLAC. DEP.	CL	#N/A	2,495	2,997	2,746
7D	204.1	GLACIAL TILL	ML	#N/A	5,221	6,272	5,747
8D	199.1	GLACIAL TILL	SM	47.1	#N/A	#N/A	#N/A
9D	194.1	GLACIAL TILL	SM	45.7	#N/A	#N/A	#N/A
10D	189.1	GLACIAL TILL	SM	45.7	#N/A	#N/A	#N/A
11D	184.1	GLACIAL TILL	SM	47.7	#N/A	#N/A	#N/A
12D	179.1	GLACIAL TILL	SM	47.3	#N/A	#N/A	#N/A
13D	164.1	GLACIAL TILL	ML	#N/A	11,675	14,025	12,850
14D	159.1	GLACIAL TILL	ML	#N/A	15,082	18,119	16,600
15D	154.1	GLACIAL TILL	ML	#N/A	14,230	17,095	15,663
16D	149.1	GLACIAL TILL	ML	#N/A	10,610	12,745	11,678
17D	144.1	GLACIAL TILL	ML	#N/A	10,610	12,745	11,678

Note:

1. #N/A indicates strength parameter is not applicable based on the soil type.

Boring BB-PI95-102

Sample No.	Sample El. (ft, NAVD 88)	Geologic Deposit	USCS Symbol	Friction Angle (ϕ', deg.)	Undrained Shear Strength (S_u, psf)		
					$S_{u,1}$	$S_{u,2}$	$S_{u,avg}$
1D	217.6	GLACIAL TILL	ML	#N/A	1,878	2,255	2,066
2D	215.6	GLACIAL TILL	ML	#N/A	3,475	4,174	3,825
3D	212.6	GLACIAL TILL	CL	#N/A	4,593	5,517	5,055
4D	207.6	GLACIAL TILL	SC-SM	40.1	#N/A	#N/A	#N/A
5D	202.6	GLACIAL TILL	SC-SM	39.0	#N/A	#N/A	#N/A
6D	197.6	GLACIAL TILL	SW	44.1	#N/A	#N/A	#N/A
7D	192.6	GLACIAL TILL	SC-SM	41.8	#N/A	#N/A	#N/A
8D	187.6	GLACIAL TILL	SM	45.2	#N/A	#N/A	#N/A
9D	182.6	GLACIAL TILL	SM	45.7	#N/A	#N/A	#N/A
10D	177.6	GLACIAL TILL	ML	#N/A	11,249	13,513	12,381
11D	172.6	GLACIAL TILL	ML	#N/A	9,758	11,722	10,740
12D	167.6	GLACIAL TILL	ML	#N/A	10,397	12,490	11,443
13D	162.6	GLACIAL TILL	ML	#N/A	9,971	11,978	10,974
14D	157.6	GLACIAL TILL	ML	#N/A	9,758	11,722	10,740
15D	152.6	GLACIAL TILL	ML	#N/A	9,758	11,722	10,740
16D	147.6	GLACIAL TILL	ML	#N/A	10,610	12,745	11,678

Note:
1. #N/A indicates strength parameter is not applicable based on the soil type.

Boring BB-PI95-103

Sample No.	Sample El. (ft, NAVD 88)	Geologic Deposit	USCS Symbol	Friction Angle (ϕ', deg.)	Undrained Shear Strength (S_u, psf)		
					$S_{u,1}$	$S_{u,2}$	$S_{u,avg}$
1D	219.1	FILL	ML	#N/A	919	1,104	1,012
2D	217.1	GLACIAL TILL	SC-SM	32.4	#N/A	#N/A	#N/A
3D	215.1	GLACIAL TILL	SC-SM	30.5	#N/A	#N/A	#N/A
4D	209.1	GLACIAL TILL	SM	40.1	#N/A	#N/A	#N/A
5D	204.1	GLACIAL TILL	ML	#N/A	5,211	6,259	5,735
6D	199.1	GLACIAL TILL	SM	40.9	#N/A	#N/A	#N/A
7D	194.1	GLACIAL TILL	SM	47.7	#N/A	#N/A	#N/A
8D	189.1	GLACIAL TILL	GP	45.2	#N/A	#N/A	#N/A
9D	184.1	GLACIAL TILL	GP	45.7	#N/A	#N/A	#N/A
10D	179.1	GLACIAL TILL	ML	#N/A	13,378	16,072	14,725
11D	174.1	GLACIAL TILL	ML	#N/A	18,064	21,701	19,882
12D	169.1	GLACIAL TILL	CL	#N/A	9,758	11,722	10,740
13D	164.1	GLACIAL TILL	ML	#N/A	12,313	14,792	13,553

Note:
1. #N/A indicates strength parameter is not applicable based on the soil type.

Boring BB-PI95-201

Sample No.	Sample El. (ft, NAVD 88)	Geologic Deposit	USCS Symbol	Friction Angle (ϕ', deg.)	Undrained Shear Strength (S_u, psf)		
					$S_{u,1}$	$S_{u,2}$	$S_{u,avg}$
1D	211.0	GLACIAL TILL	SM	31.4	#N/A	#N/A	#N/A
2D	209.0	GLACIAL TILL	ML	#N/A	2,067	2,482	2,274
3D	207.0	GLACIAL TILL	ML	#N/A	2,488	2,988	2,738
4D	202.0	GLACIAL TILL	ML	#N/A	3,330	4,000	3,665
5D	197.0	GLACIAL TILL	ML	#N/A	8,075	9,700	8,887
6D	192.0	GLACIAL TILL	SC-SM	42.7	#N/A	#N/A	#N/A
7D	187.0	GLACIAL TILL	SM	37.2	#N/A	#N/A	#N/A
8D	182.0	GLACIAL TILL	SM	47.4	#N/A	#N/A	#N/A
9D	177.0	GLACIAL TILL	ML	#N/A	8,383	10,071	9,227

Note:

1. #N/A indicates strength parameter is not applicable based on the soil type.

Boring BB-PI95-202

Sample No.	Sample El. (ft, NAVD 88)	Geologic Deposit	USCS Symbol	Friction Angle (ϕ', deg.)	Undrained Shear Strength (S_u, psf)		
					$S_{u,1}$	$S_{u,2}$	$S_{u,avg}$
1D	210.9	GLACIAL TILL	ML	#N/A	1,786	2,145	1,965
2D	208.9	GLACIAL TILL	ML	#N/A	2,067	2,482	2,274
3D	206.9	GLACIAL TILL	ML	#N/A	3,470	4,168	3,819
4D	201.9	GLACIAL TILL	ML	#N/A	3,470	4,168	3,819
5D	196.9	GLACIAL TILL	ML	#N/A	2,029	2,437	2,233
6D	191.9	GLACIAL TILL	ML	#N/A	7,916	9,509	8,712
7D	186.9	GLACIAL TILL	ML	#N/A	3,517	4,224	3,871
8D	181.9	GLACIAL TILL	ML	#N/A	4,584	5,506	5,045
9D	176.9	GLACIAL TILL	ML	#N/A	9,694	11,645	10,669

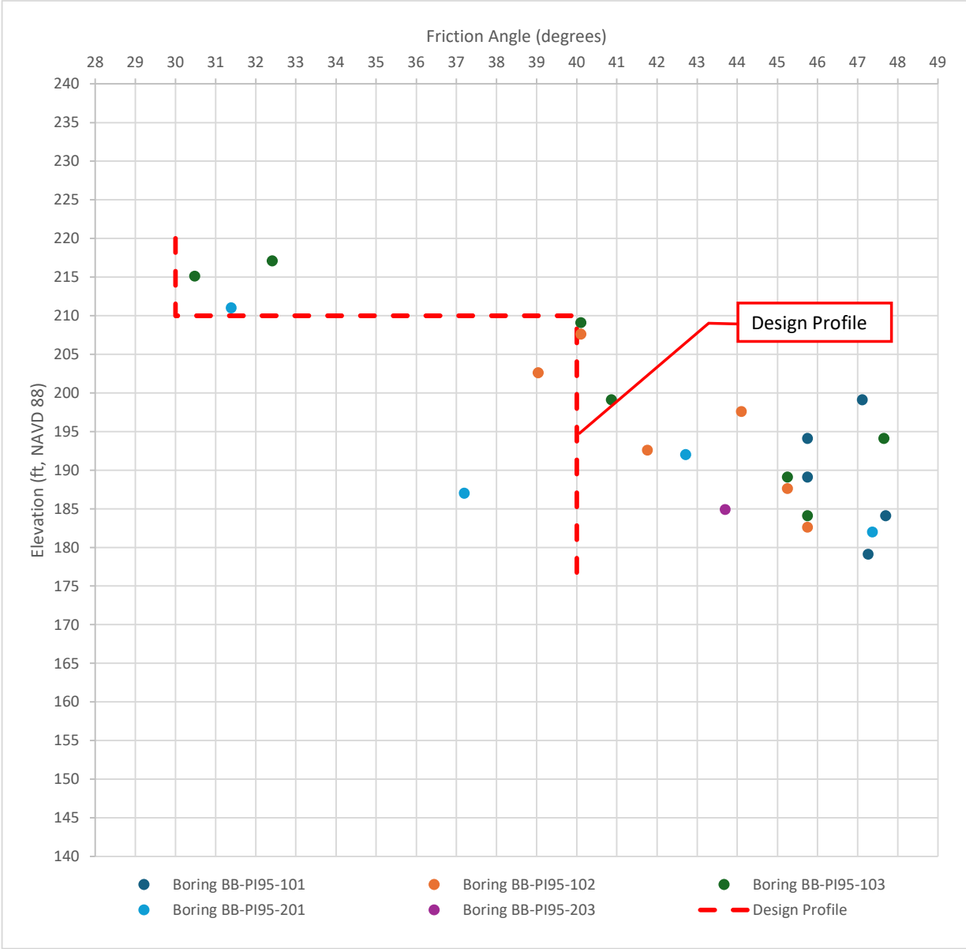
Note:

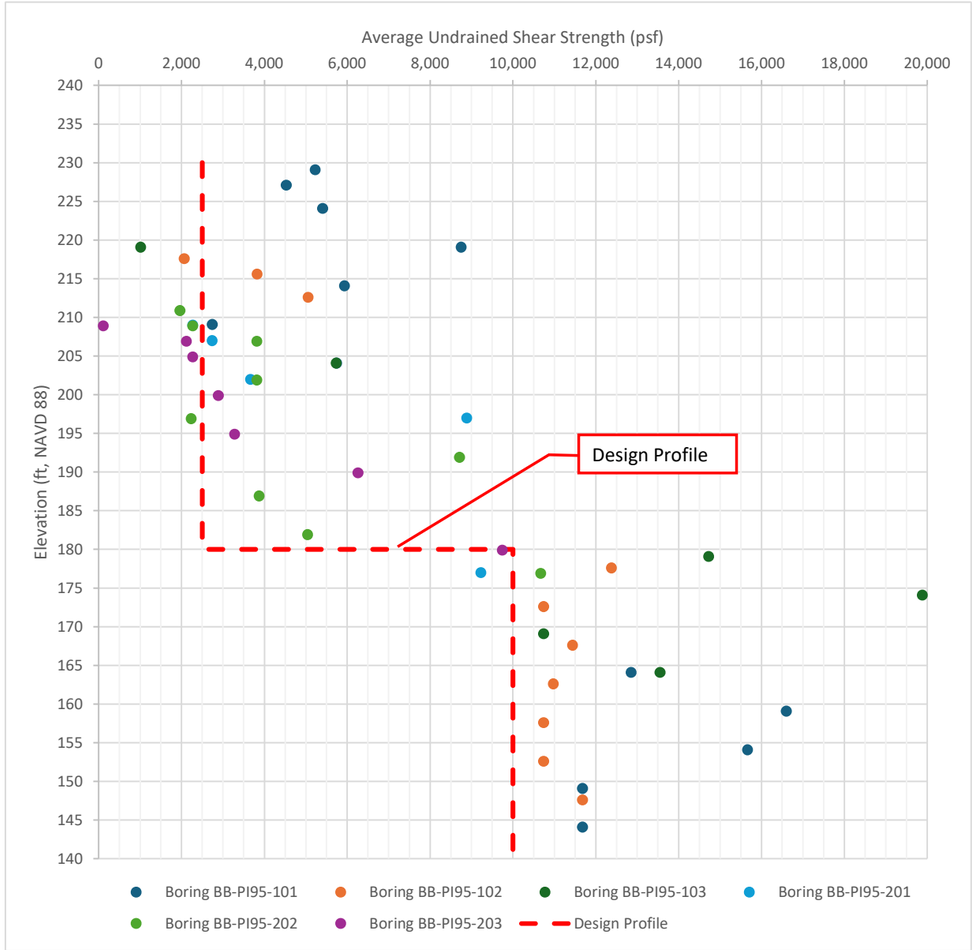
1. #N/A indicates strength parameter is not applicable based on the soil type.

Boring BB-PI95-203

Sample No.	Sample El. (ft, NAVD 88)	Geologic Deposit	USCS Symbol	Friction Angle (ϕ', deg.)	Undrained Shear Strength (S_u, psf)		
					$S_{u,1}$	$S_{u,2}$	$S_{u,avg}$
1D	208.9	SWAMP DEPOSIT	ML	#N/A	101	121	111
2D	206.9	GLACIAL TILL	ML	#N/A	1,926	2,313	2,120
3D	204.9	GLACIAL TILL	ML	#N/A	2,067	2,482	2,274
4D	199.9	GLACIAL TILL	ML	#N/A	2,628	3,156	2,892
5D	194.9	GLACIAL TILL	ML	#N/A	2,984	3,584	3,284
6D	189.9	GLACIAL TILL	ML	#N/A	5,688	6,833	6,261
7D	184.9	GLACIAL TILL	GC	43.7	#N/A	#N/A	#N/A
8D	179.9	GLACIAL TILL	ML	#N/A	8,851	10,633	9,742

Note:
1. #N/A indicates strength parameter is not applicable based on the soil type.


Friction Angle (degrees)
28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49
Elevation (ft, NAVD 88)
240 235 230 225 220 215 210 205 200 195 190 185 180 175 170 165 160 155 150 145 140
Design Profile
Boring BB-PI95-101
Boring BB-PI95-102
Boring BB-PI95-103
Boring BB-PI95-201
Boring BB-PI95-203
Design Profile


Average Undrained Shear Strength (psf)
0 2,000 4,000 6,000 8,000 10,000 12,000 14,000 16,000 18,000 20,000
Elevation (ft, NAVD 88)
240 235 230 225 220 215 210 205 200 195 190 185 180 175 170 165 160 155 150 145 140
Design Profile
Boring BB-PI95-101
Boring BB-PI95-102
Boring BB-PI95-103
Boring BB-PI95-201
Boring BB-PI95-202
Boring BB-PI95-203
Design Profile

Seismic Site Class and Design Parameters

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class and risk-targeted design response spectra for the proposed bridge in accordance with the requirements of AASHTO LRFD Section 3.10.3.1..

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the borings, we recommend the risk-targeted design response spectrum shown on Sheet 7, considering the maximum acceleration at a given period for Seismic Site Classes C/D, D, and D/E.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.
2. Seed. H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K. (1984), "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," Report No. UCB/EERC-84/14, Earthquake Engineering Research Center, University of California, Berkeley, California.
3. Dickenson, SE (1994). Dynamic Response of Soft and Deep Cohesive Soils during the Loma Prieta Earthquake of October 17, 1989, PhD thesis, Dept. of Civil and Enviro. Eng., University of California, Berkeley, CA.
4. Sykora, DE, and KH Stokoe (1983). Correlations of in-situ measurements in sands of shear wave velocity, Soil Dyn. Earthq. Eng., 20:125–36.

AVAILABLE INFORMATION

1. Boring logs BB-PI95-101, -102, and -103 drilled by Northern Test Boring, Inc. in May and June 2024 and monitored by Haley & Aldrich. "-200" series borings were not drilled at proposed bridge structures.
2. Elevations reference the NAVD 88 datum.

ASSUMPTIONS

1. Because shear wave velocity of soil and bedrock was not measured directly, correlations were used to estimate shear wave velocity (Vs) from standard penetration test data (i.e., SPT N-values).
2. Bedrock is assumed to have a shear wave velocity of 2,100 ft/sec, corres. to the low end of "soft rock" in LRFD Table 3.10.3.1-1.
3. Because the borings were term. in bedrock less than 100 ft below existing ground surface, which is the total depth range used to determined Site Class in accordance with LRFD, the shear wave velocity in the reference above was assumed to represent bedrock between the top of bedrock encountered in the borings and 100 ft.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.10.3.1-1), that require completion of a site-specific ground motion response evaluation. If the subsurface conditions at the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.

2. If shear wave velocity of soil and bedrock was not measured directly, use correlations with avail. soil and bedrock properties to estimate shear wave velocity.

For cohesionless soils use Seed, et al. 1984

$$v_s = \sqrt{\frac{G_{max}}{\rho}}$$

$$G_{max} = 1000(K_2)_{max} * \sigma'_m{}^{0.5}$$

$$(K_2)_{max} = 20[(N_1)_{60}]^{0.33}$$

$$\sigma'_m = \left(\frac{1 + 2 * K_0}{3}\right) * \sigma'_v$$

Where:
K_0 = 0.4 (assumed)
$N_{1,60}$ = overburden & energy-corrected SPT N value
σ'v = vertical effective stress
ρ = soil density

PROCEDURE

For cohesive soils with vane shear data use Dickerson 1994

$$v_s(fps) = 18{S_u}^{0.475}$$

For cohesive soils without vane shear data or cohesionless soils use Sykora 1983

195 = Factor for cohesive soils
250 = Factor for cohesionless soils
N = uncorrected SPT N-value
d = sample depth

$$v_s(fps) = Factor(N^{0.17})(d^{0.2})$$

For cohesionless soils, use the minimum Vs from Seed et al. 1984 and Sykora 1983

3. Categorize the site using the average shear wave velocity in the upper 100 ft.

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

$$\overline{V_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of *i* th soil (ft/s).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

4. Where measured shear wave velocity data is not available, categorize the site using each of the following factors applied to the average shear wave velocity.

$\overline{V_s} * 1.0$

$\overline{V_s} * 1.3$

$\overline{V_s}/1.3$

SITE CLASS DEFINITIONS

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile	$\overline{v}_s$ Calculated or Estimated
A	Hard rock	$\overline{v}_s > 5{,}000$ ft/s
B	Medium hard rock	3,000 ft/s $< \overline{v}_s \leq 5{,}000$ ft/s
B/C	Soft rock	2,100 ft/s $< \overline{v}_s \leq 3{,}000$ ft/s
C	Very dense sand or hard clay	1,450 ft/s $< \overline{v}_s \leq 2{,}100$ ft/s
C/D	Dense sand or very stiff clay	1,000 ft/s $< \overline{v}_s \leq 1{,}450$ ft/s
D	Medium dense sand or stiff clay	700 ft/s $< \overline{v}_s \leq 1{,}000$ ft/s
D/E	Loose sand or medium stiff clay	500 ft/s $< \overline{v}_s \leq 700$ ft/s
E	Very loose sand or soft clay	$\overline{v}_s \leq 500$ ft/s
F	Soils requiring site-specific ground response evaluations, such as: • Peats or highly organic clays ($H > 10.0$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25.0$ ft with $PI > 75$) • Very thick, soft/medium stiff clays ($H > 120$ ft)	

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

where:

$\overline{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile

Exploration ID:	BB-PI95-101
Ground Surface El.:	229.1
Ground Water Depth (ft)	11.2

Hammer Efficiency Factor: 0.949

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesive or Cohesionless	Unit Weight (pcf)	σ_v (psf)	σ'_v (psf)	SPT N (blows/ft)	N_{60} (blows/ft)	C_N	$(N_1)_{60}$ (blows/ft)	Su psf	$(K_2)_{max}$	Vs (Seed et al.) (ft/s)	Vs (Dickenson) (ft/s)	Sykora Factor	Vs (Sykora) (ft/s)	Vs (ft/s)	d/Vs
1D	0.1	229.0	Fill	2.0	Cohesive	120	12	12	30	47	1.7	81	#N/A	#N/A	#N/A	#N/A	195	219	219	0.0091
2D	2.0	227.1	Fill	3.0	Cohesive	120	240	240	26	41	1.7	70	#N/A	#N/A	#N/A	#N/A	195	390	390	0.0077
3D	5.0	224.1	Fill	5.0	Cohesive	120	600	600	31	49	1.7	83	#N/A	#N/A	#N/A	#N/A	195	482	482	0.0104
4D	10.0	219.1	Fill	5.0	Cohesive	120	1200	1200	50	79	1.3	104	#N/A	#N/A	#N/A	#N/A	195	601	601	0.0083
5D	15.0	214.1	Glacio. Dep.	5.0	Cohesive	120	1800	1563	30	47	1.2	55	#N/A	#N/A	#N/A	#N/A	195	598	598	0.0084
6D	20.0	209.1	Glacio. Dep.	5.0	Cohesive	120	2400	1851	14	22	1.1	24	#N/A	#N/A	#N/A	#N/A	195	556	556	0.0090
7D	25.0	204.1	Glacial Till	5.0	Cohesive	120	3000	2139	26	41	1.0	41	#N/A	#N/A	#N/A	#N/A	195	646	646	0.0077
8D	30.0	199.1	Glacial Till	5.0	Cohesionless	120	3600	2427	62	98	0.9	91	#N/A	90	959	#N/A	250	996	959	0.0052
9D	35.0	194.1	Glacial Till	5.0	Cohesionless	120	4200	2715	50	79	0.9	69	#N/A	82	943	#N/A	250	990	943	0.0053
10D	40.0	189.1	Glacial Till	5.0	Cohesionless	120	4800	3003	50	79	0.8	66	#N/A	81	959	#N/A	250	1017	959	0.0052
11D	45.0	184.1	Glacial Till	5.0	Cohesionless	120	5400	3291	82	130	0.8	103	#N/A	94	1057	#N/A	250	1132	1057	0.0047
12D	50.0	179.1	Glacial Till	15.0	Cohesionless	120	6000	3579	87	138	0.8	105	#N/A	94	1082	#N/A	250	1168	1082	0.0139
13D	65.0	164.1	Glacial Till	5.0	Cohesive	120	7800	4443	55	87	0.7	60	#N/A	#N/A	#N/A	#N/A	195	888	888	0.0056
14D	70.0	159.1	Glacial Till	5.0	Cohesive	120	8400	4731	71	112	0.7	75	#N/A	#N/A	#N/A	#N/A	195	941	941	0.0053
15D	75.0	154.1	Glacial Till	5.0	Cohesive	120	9000	5019	67	106	0.6	68	#N/A	#N/A	#N/A	#N/A	195	945	945	0.0053
16D	80.0	149.1	Glacial Till	5.0	Cohesive	120	9600	5307	50	79	0.6	50	#N/A	#N/A	#N/A	#N/A	195	911	911	0.0055
17D	85.0	144.1	Glacial Till	1.8	Cohesive	120	10200	5595	50	79	0.6	48	#N/A	#N/A	#N/A	#N/A	195	922	922	0.0020
R1	86.8	142.3	Bedrock	13.2	Bedrock														2100	0.0063
			Totals =	100.0																

Totals =	0.1249
vs-bar (ft/s) =	801

Factor	vs-bar (ft/s)	Site Class
vs-bar*1.3	1041	C/D
vs-bar*1.0	801	D
vs-bar/1.3	616	D/E

Exploration ID:	BB-PI95-102
Ground Surface El.:	217.6
Ground Water Depth (ft)	5.8

Hammer Efficiency Factor: 0.949

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesive or Cohesionless	Unit Weight (pcf)	σ_v (psf)	σ'_v (psf)	SPT N (blows/ft)	N_{60} (blows/ft)	C_N	$(N_1)_{60}$ (blows/ft)	Su psf	$(K_2)_{max}$	Vs (Seed et al.) (ft/s)	Vs (Dickenson) (ft/s)	Sykora Factor	Vs (Sykora) (ft/s)	Vs (ft/s)	d/Vs
1D	0.1	217.5	Glacial Till	2.0	Cohesive	120	12	12	12	19	1.7	32	#N/A	#N/A	#N/A	#N/A	195	188	188	0.0107
2D	2.0	215.6	Glacial Till	3.0	Cohesive	120	240	240	22	35	1.7	59	#N/A	#N/A	#N/A	#N/A	195	379	379	0.0079
3D	5.0	212.6	Glacial Till	5.0	Cohesive	120	600	600	29	46	1.7	78	#N/A	#N/A	#N/A	#N/A	195	477	477	0.0105
4D	10.0	207.6	Glacial Till	5.0	Cohesionless	120	1200	938	39	62	1.5	92	#N/A	90	758	#N/A	250	739	739	0.0068
5D	15.0	202.6	Glacial Till	5.0	Cohesionless	120	1800	1226	31	49	1.3	64	#N/A	80	762	#N/A	250	770	762	0.0066
6D	20.0	197.6	Glacial Till	5.0	Cohesionless	120	2400	1514	50	79	1.2	93	#N/A	90	855	#N/A	250	885	855	0.0058
7D	25.0	192.6	Glacial Till	5.0	Cohesionless	120	3000	1802	36	57	1.1	61	#N/A	79	834	#N/A	250	875	834	0.0060
8D	30.0	187.6	Glacial Till	5.0	Cohesionless	120	3600	2090	50	79	1.0	79	#N/A	86	902	#N/A	250	960	902	0.0055
9D	35.0	182.6	Glacial Till	5.0	Cohesionless	120	4200	2378	50	79	0.9	74	#N/A	84	922	#N/A	250	990	922	0.0054
10D	40.0	177.6	Glacial Till	5.0	Cohesive	120	4800	2666	53	84	0.9	74	#N/A	#N/A	#N/A	#N/A	195	801	801	0.0062
11D	45.0	172.6	Glacial Till	5.0	Cohesive	120	5400	2954	46	73	0.8	61	#N/A	#N/A	#N/A	#N/A	195	800	800	0.0062
12D	50.0	167.6	Glacial Till	5.0	Cohesive	120	6000	3242	49	78	0.8	62	#N/A	#N/A	#N/A	#N/A	195	826	826	0.0061
13D	55.0	162.6	Glacial Till	5.0	Cohesive	120	6600	3530	47	74	0.8	57	#N/A	#N/A	#N/A	#N/A	195	836	836	0.0060
14D	60.0	157.6	Glacial Till	5.0	Cohesive	120	7200	3818	46	73	0.7	54	#N/A	#N/A	#N/A	#N/A	195	848	848	0.0059
15D	65.0	152.6	Glacial Till	5.0	Cohesive	120	7800	4106	46	73	0.7	52	#N/A	#N/A	#N/A	#N/A	195	862	862	0.0058
16D	70.0	147.6	Glacial Till	0.3	Cohesive	120	8400	4394	50	79	0.7	55	#N/A	#N/A	#N/A	#N/A	195	887	887	0.0003
R1	70.3	147.3	Bedrock	29.7	Bedrock														2100	0.0141
			Totals =	100.0																

Totals =	0.1159
vs-bar (ft/s) =	863

Factor	vs-bar (ft/s)	Site Class
vs-bar*1.3	1122	C/D
vs-bar*1.0	863	D
vs-bar/1.3	664	D/E

Exploration ID:	BB-PI95-103
Ground Surface El.:	219.1
Ground Water Depth (ft)	2.2

Hammer Efficiency Factor: 0.949

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesive or Cohesionless	Unit Weight (pcf)	σ_v (psf)	σ'_v (psf)	SPT N (blows/ft)	N_{60} (blows/ft)	C_N	$(N_1)_{60}$ (blows/ft)	Su psf	$(K_2)_{max}$	Vs (Seed et al.) (ft/s)	Vs (Dickenson) (ft/s)	Sykora Factor	Vs (Sykora) (ft/s)	Vs (ft/s)	d/Vs
1D	0.1	219.0	Fill	2.0	Cohesive	120	12	12	6	9	1.7	16	#N/A	#N/A	#N/A	#N/A	195	167	167	0.0120
2D	2.0	217.1	Glacial Till	2.0	Cohesionless	120	240	240	15	24	1.7	40	#N/A	69	470	#N/A	250	455	455	0.0044
3D	4.0	215.1	Glacial Till	6.0	Cohesionless	120	480	368	10	16	1.7	27	#N/A	60	488	#N/A	250	488	488	0.0123
4D	10.0	209.1	Glacial Till	5.0	Cohesionless	120	1200	713	39	62	1.7	105	#N/A	94	723	#N/A	250	739	723	0.0069
5D	15.0	204.1	Glacial Till	5.0	Cohesive	120	1800	1001	29	46	1.4	66	#N/A	#N/A	#N/A	#N/A	195	594	594	0.0084
6D	20.0	199.1	Glacial Till	5.0	Cohesionless	120	2400	1289	37	59	1.3	74	#N/A	84	792	#N/A	250	841	792	0.0063
7D	25.0	194.1	Glacial Till	5.0	Cohesionless	120	3000	1577	68	108	1.2	124	#N/A	100	906	#N/A	250	975	906	0.0055
8D	30.0	189.1	Glacial Till	5.0	Cohesionless	120	3600	1865	50	79	1.1	84	#N/A	87	885	#N/A	250	960	885	0.0056
9D	35.0	184.1	Glacial Till	5.0	Cohesionless	120	4200	2153	50	79	1.0	78	#N/A	85	907	#N/A	250	990	907	0.0055
10D	40.0	179.1	Glacial Till	5.0	Cohesive	120	4800	2441	63	100	0.9	92	#N/A	#N/A	#N/A	#N/A	195	825	825	0.0061
11D	45.0	174.1	Glacial Till	5.0	Cohesive	120	5400	2729	85	134	0.9	118	#N/A	#N/A	#N/A	#N/A	195	889	889	0.0056
12D	50.0	169.1	Glacial Till	5.0	Cohesive	120	6000	3017	46	73	0.8	61	#N/A	#N/A	#N/A	#N/A	195	818	818	0.0061
13D	55.0	164.1	Glacial Till	4.0	Cohesive	120	6600	3305	58	92	0.8	73	#N/A	#N/A	#N/A	#N/A	195	867	867	0.0046
R1	59.0	160.1	Bedrock	41.0	Bedrock														2100	0.0195
			Totals =	100.0																

Totals =	0.1089
vs-bar (ft/s) =	918

Factor	vs-bar (ft/s)	Site Class
vs-bar*1.3	1193	C/D
vs-bar*1.0	918	D
vs-bar/1.3	706	D

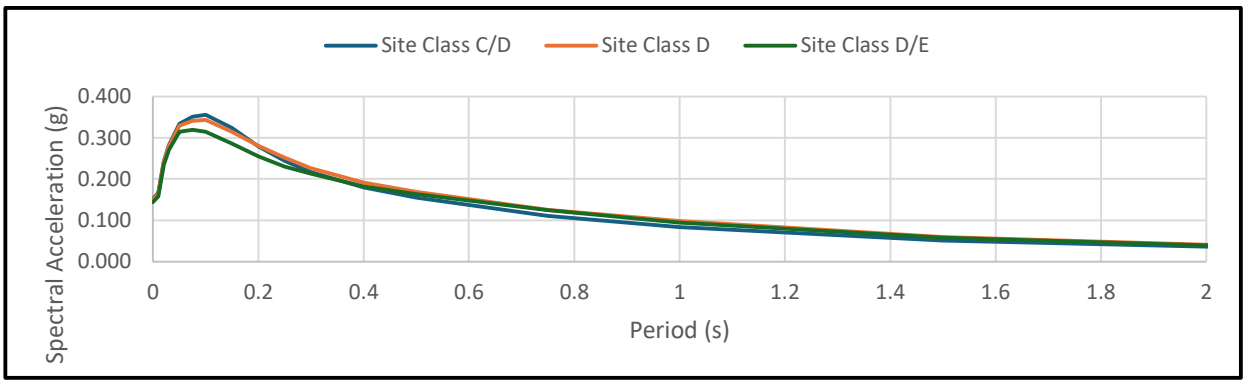
RESULTS SUMMARY

Boring	Site Classes
BB-PI95-101	**C/D, D, D/E**
BB-PI95-102	**C/D, D, D/E**
BB-PI95-103	**C/D, D**

From USGS Hazard Tool for the site location:

Site Class	C/D	D	D/E	**Max Accel.**
Period (s)	Spectral Acceleration (g)			
0	0.152	0.150	0.144	**0.152**
0.01	0.168	0.165	0.158	**0.168**
0.02	0.243	0.242	0.234	**0.243**
0.03	0.282	0.280	0.270	**0.282**
0.05	0.334	0.329	0.314	**0.334**
0.075	0.351	0.341	0.319	**0.351**
0.1	0.356	0.343	0.314	**0.356**
0.15	0.324	0.315	0.286	**0.324**
0.2	0.279	0.281	0.255	**0.281**
0.25	0.244	0.252	0.230	**0.252**
0.3	0.217	0.226	0.213	**0.226**
0.4	0.180	0.191	0.182	**0.191**
0.5	0.155	0.169	0.164	**0.169**
0.75	0.111	0.126	0.124	**0.126**
1	0.0831	0.0986	0.0950	**0.0986**
1.5	0.0515	0.0600	0.0585	**0.0600**
2	0.0364	0.0405	0.0396	**0.0405**
3	0.0202	0.0217	0.0212	**0.0217**
4	0.0133	0.0140	0.0136	**0.0140**
5	0.0095	0.0100	0.0096	**0.0100**
7.5	0.0053	0.0056	0.0055	**0.0056**
10	0.0033	0.0035	0.0034	**0.0035**

Equivalent 1.0 s acceleration coefficient, S_{D1} (g) = **0.0986**



CONCLUSIONS & RECOMMENDATIONS

Based on the subsurface conditions encountered at the borings, we recommend the risk-targeted design spectrum shown above, considering the maximum accleration at a given period for Site Classes C/D, D, and D/E.

Approach Embankment Settlement

PROBLEM STATEMENT & OBJECTIVE

Determine the magnitude of elastic settlement due to proposed approach embankments

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.
2. Kulhawy, F. H. and Mayne, P. W., Manual on Estimating Soil Properties for Foundation Design, 1990.

AVAILABLE INFORMATION

1. Boring logs BB-PI95-101, BB-PI95-102, and BB-PI95-103.
2. Embankment cross sections provided by HNTB on 11 March 2026.

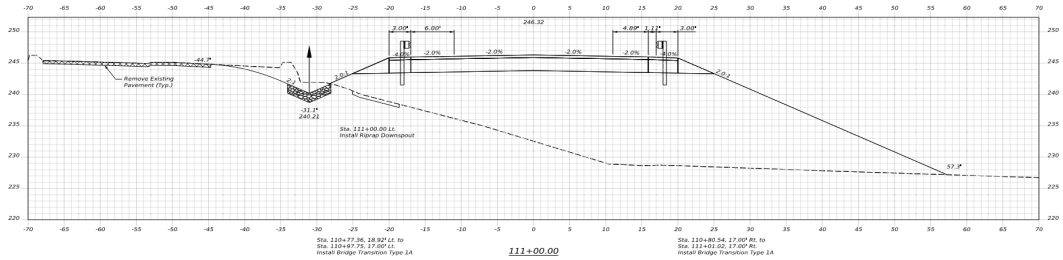
ASSUMPTIONS

1. Immediate settlement, primary consolidation, and secondary compression from construction of the ex. embank. has been completed.
2. Settlement analyses performed with Settle3 software program (v. 5.022, 9/7/23 by Rocscience Inc.)
3. Cross section at Sta. 111+00 has been determined to be critical, based on proposed fill heights and subs. conditions encountered in borings.
4. Loading in Settle3 will consider 100-ft long loads at a given station to minimize the impact of boundary effects.
5. New fill will have a unit weight of 125 pcf.
6. The following dates will be considered:
   - 5 Years
   - 20 Years
   - 75 Years
7. Settlement in Fill, Glaciolacustrine Deposits, and Glacial Till, is considered to be elastic.
8. Modulus of Elasticity (E, E_{sur}) for sands determined based on typical values provided by Settle3.
9. Modulus of Elasticity (E, E_{sur}) for silts and clays determined based on typ. values per by Kulhawy and Mayne (1990).

HALEY ALDRICH	CALCULATIONS	File No.:	0208880-000
		Sheet:	2 of 2
Client:	HNTB Corporation	Date:	3/23/2026
Project:	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	Computed by:	TPJ
Subject:	Approach Embankment Settlement	Checked by:	EMH

STA 111+00

Geometry



Soil Profile and Properties - Borings BB-PI95-101

Groundwater Elevation = 217.9 ft

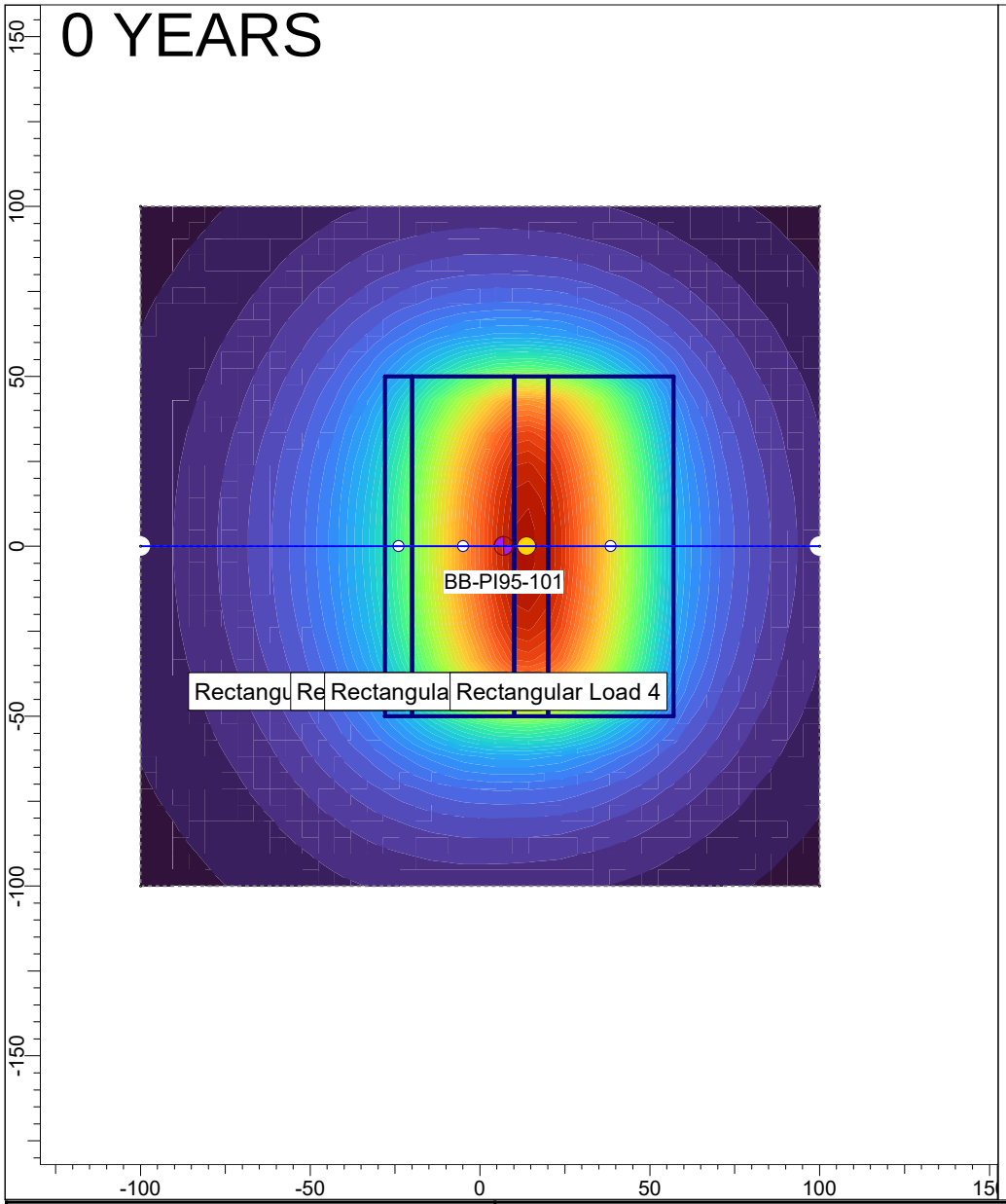
Elevation

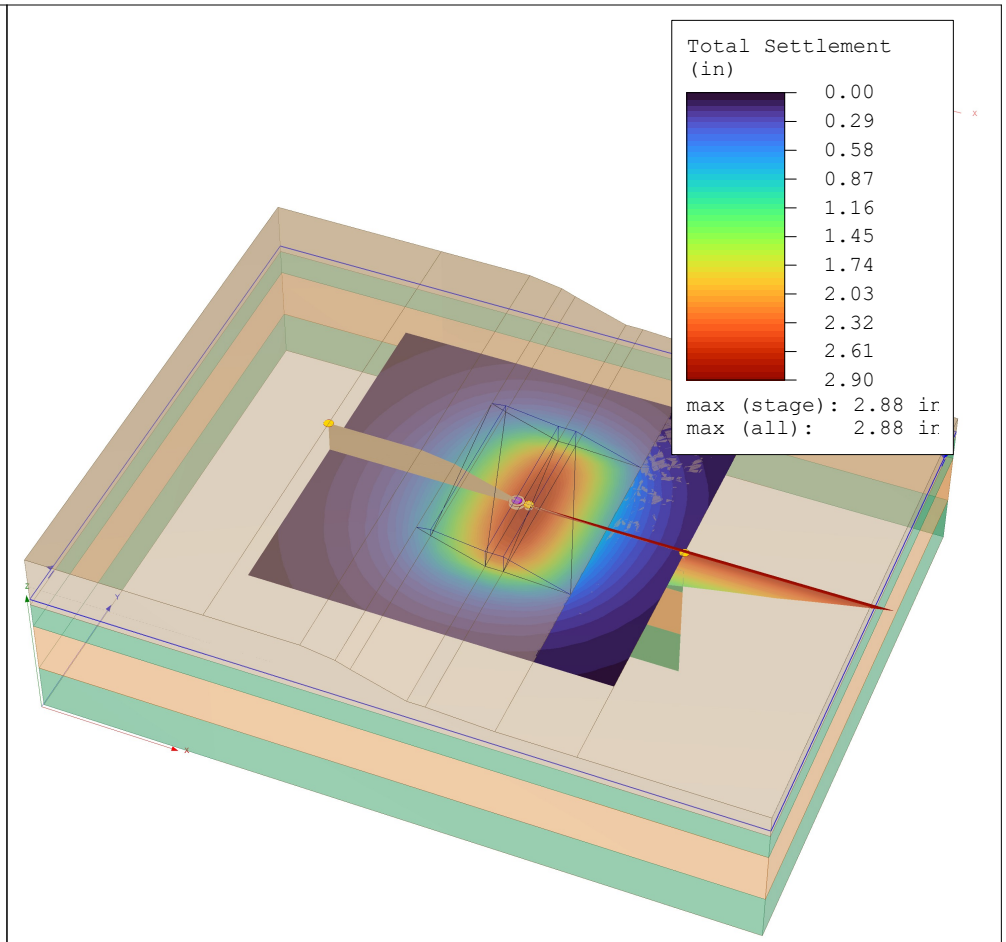
Elevation			
Varies	**Fill (Hard Clay)**	γ = 115 pcf	E/Esur = 400 ksf
214.1	**Glacial Deposits (Very Stiff to Hard Clay)**	γ = 125 pcf	E/Esur = 300 ksf
199.1	**Glacial Deposits (Very Dense Sand)**	γ = 125 pcf	E/Esur = 1,300 ksf
169.1	**Glacial Deposits (Very Stiff to Hard Clay)**	γ = 125 pcf	E/Esur = 300 ksf
142.3	**Bedrock**		

Results

	Cumulative Max. Settlement within Travel Lanes (in.)		
Time after New Fill Placed	5 Year	20 Year	75 Years
Elastic Settlement	2.9	2.9	2.9

0 YEARS

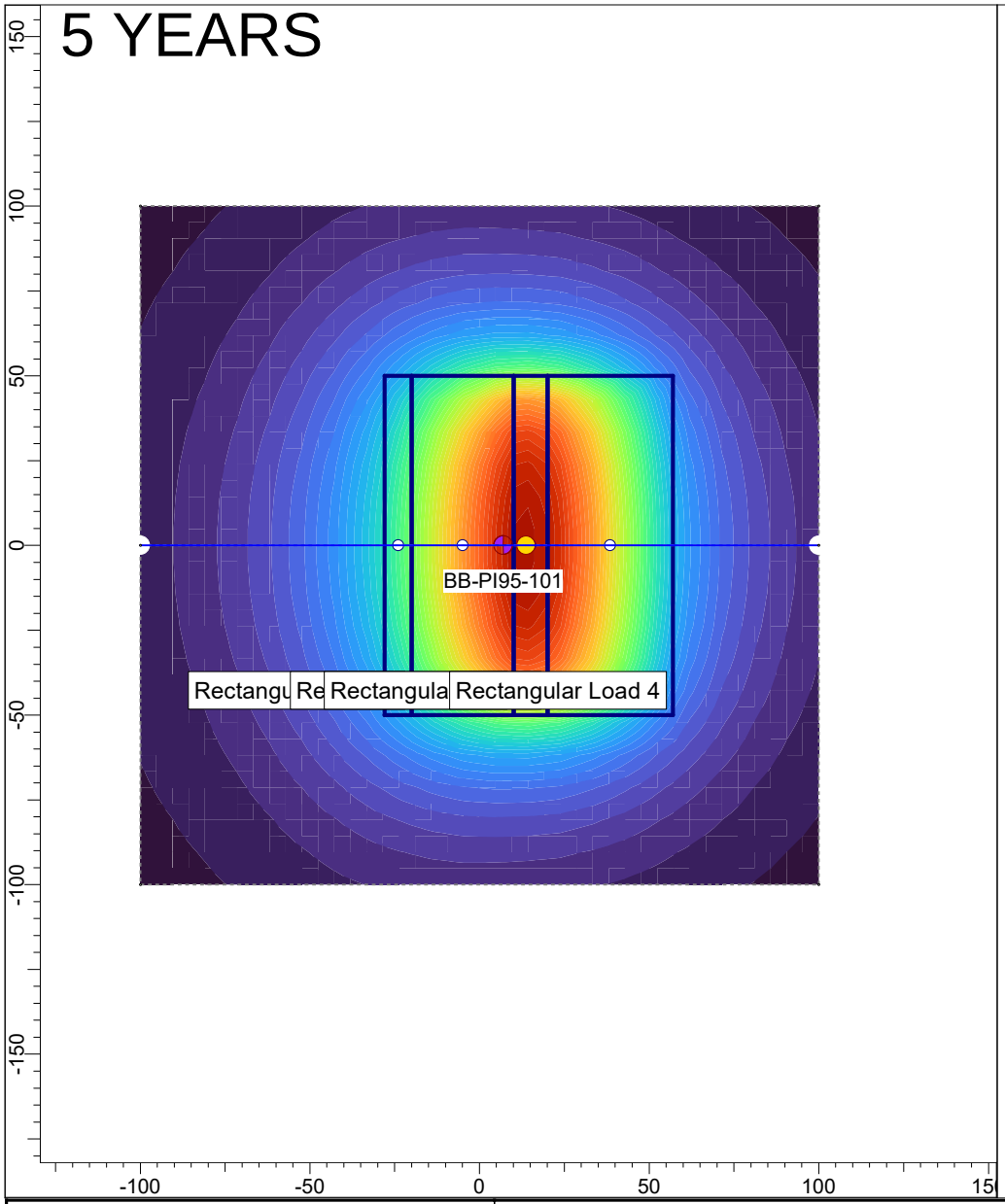


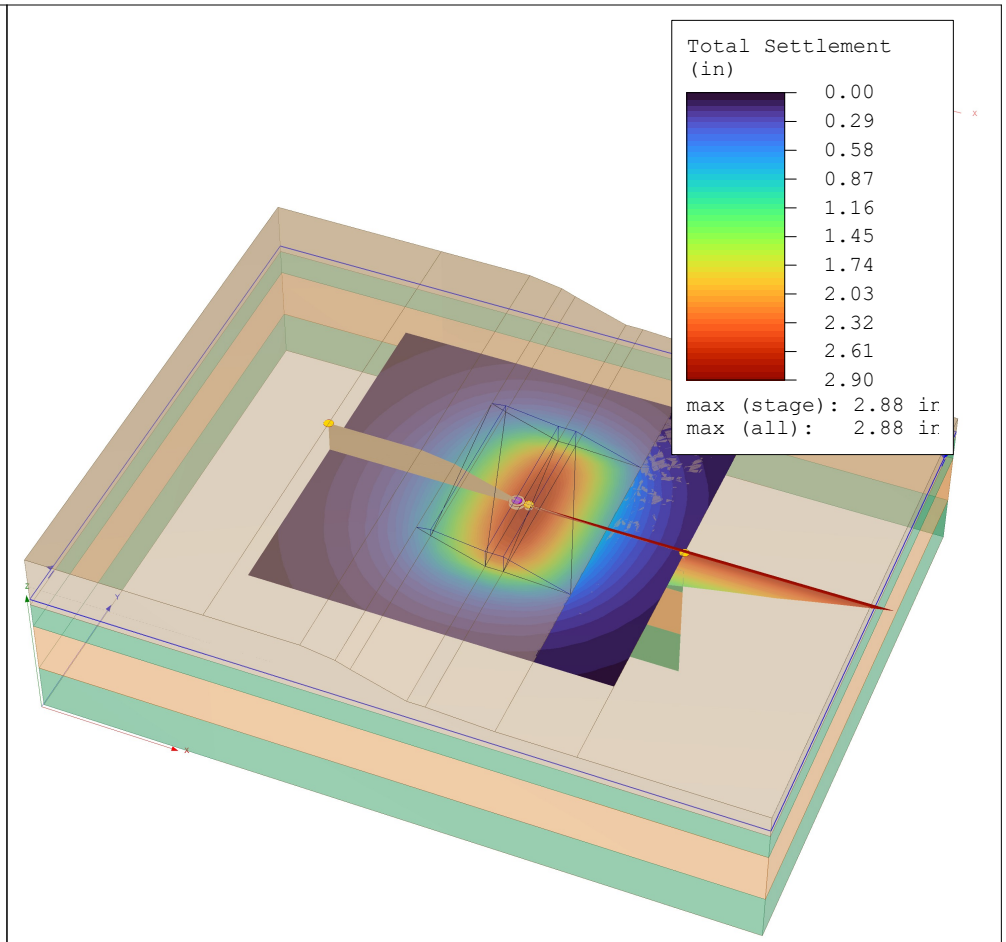


rocscience	Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	
	Analysis Description	Sta. 111+00 Settlement	
	Drawn By	TPJ	Company: Haley & Aldrich, Inc.
	Date	3/16/2026, 12:50:38 PM	File Name: 2026-0316-Palmyra-Sta 111+00-Settlement-d1.s3z

SETTLE3 5.022

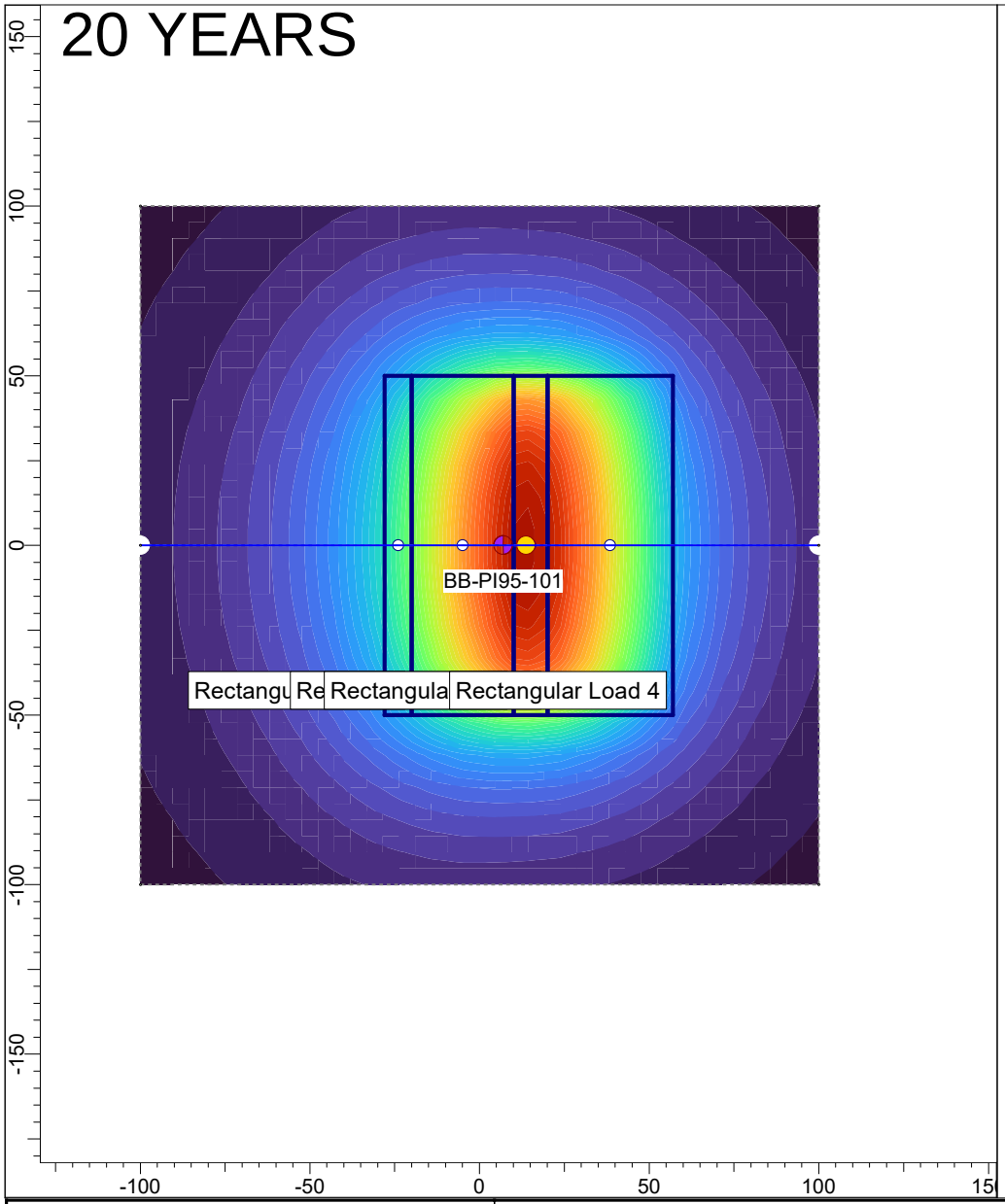
5 YEARS

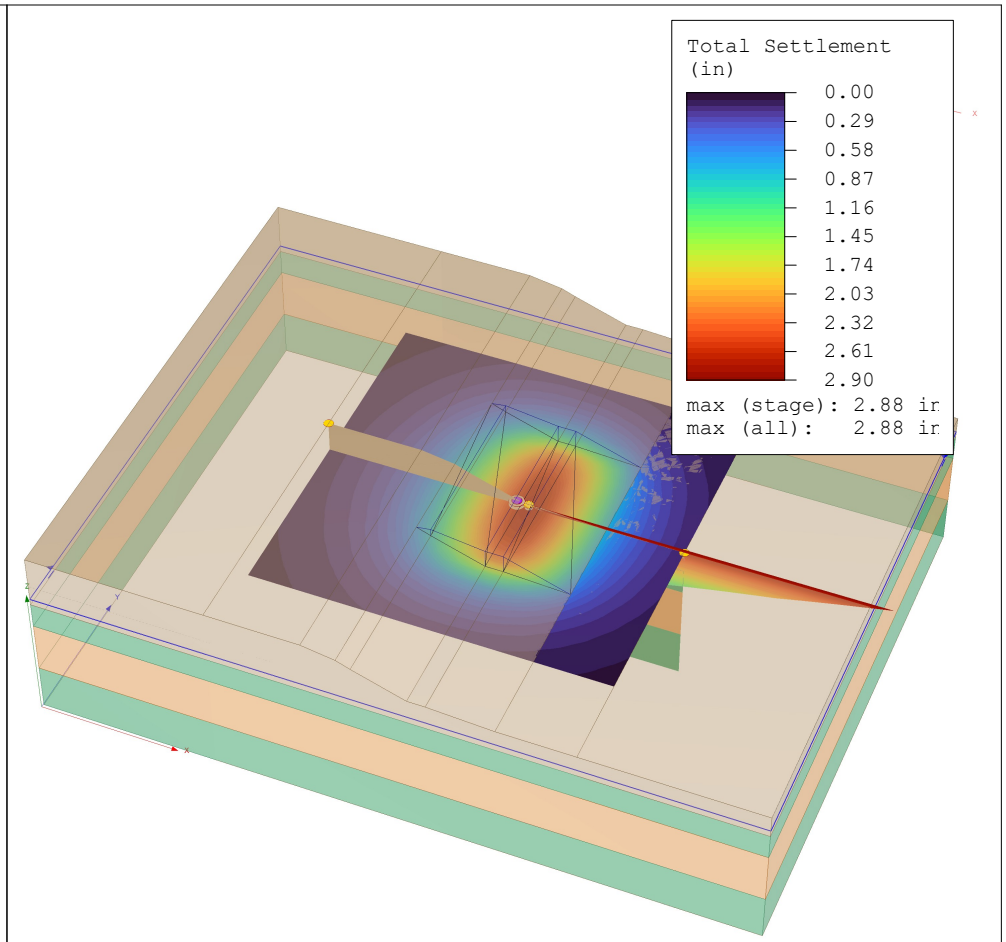




rocscience	*Project*	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	
	Analysis Description	Sta. 111+00 Settlement	
	Drawn By TPJ	*Company*	Haley & Aldrich, Inc.
SETTLE3 5.022	*Date* 3/16/2026, 12:50:38 PM	*File Name*	2026-0316-Palmyra-Sta 111+00-Settlement-d1.s3z

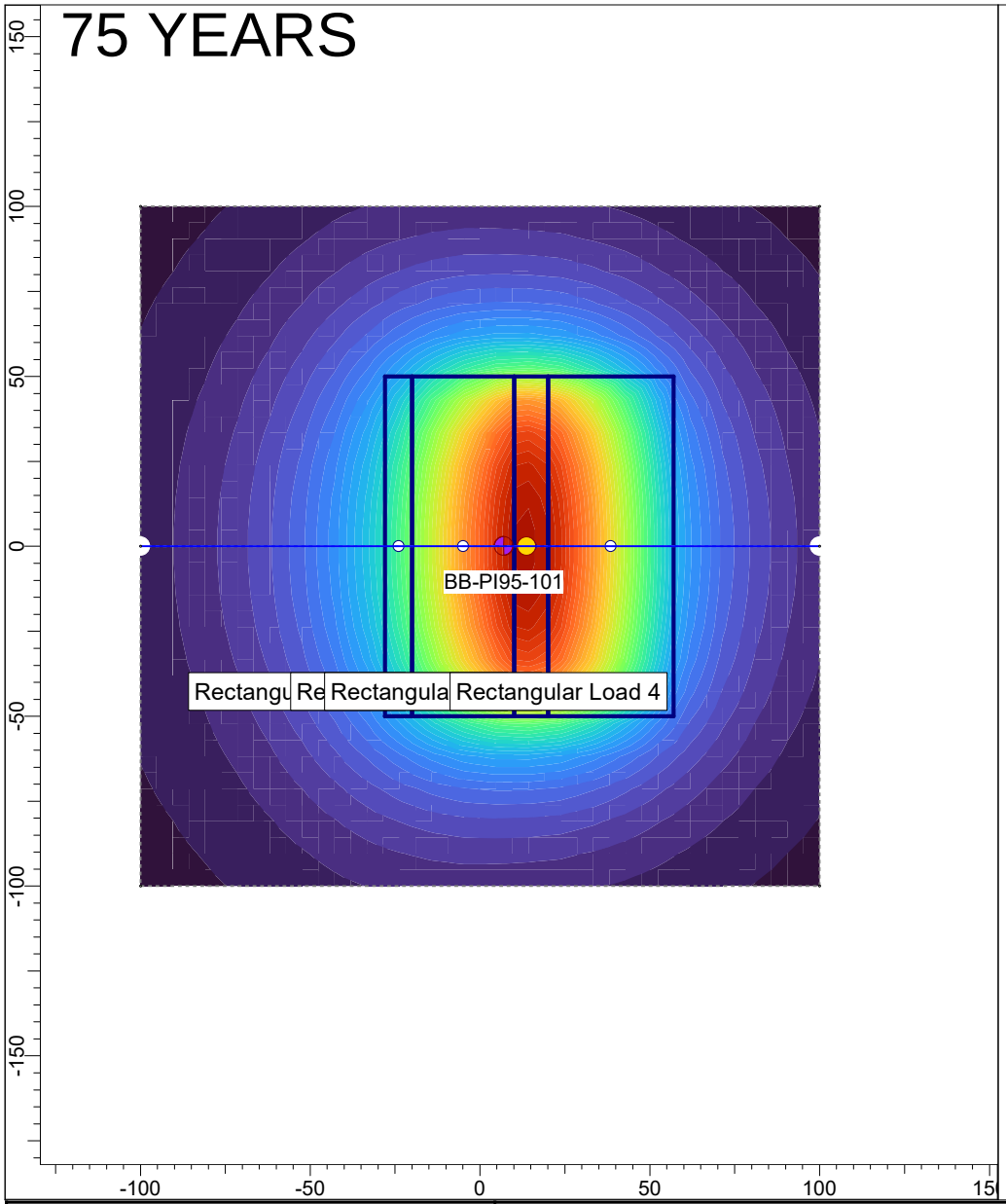
20 YEARS

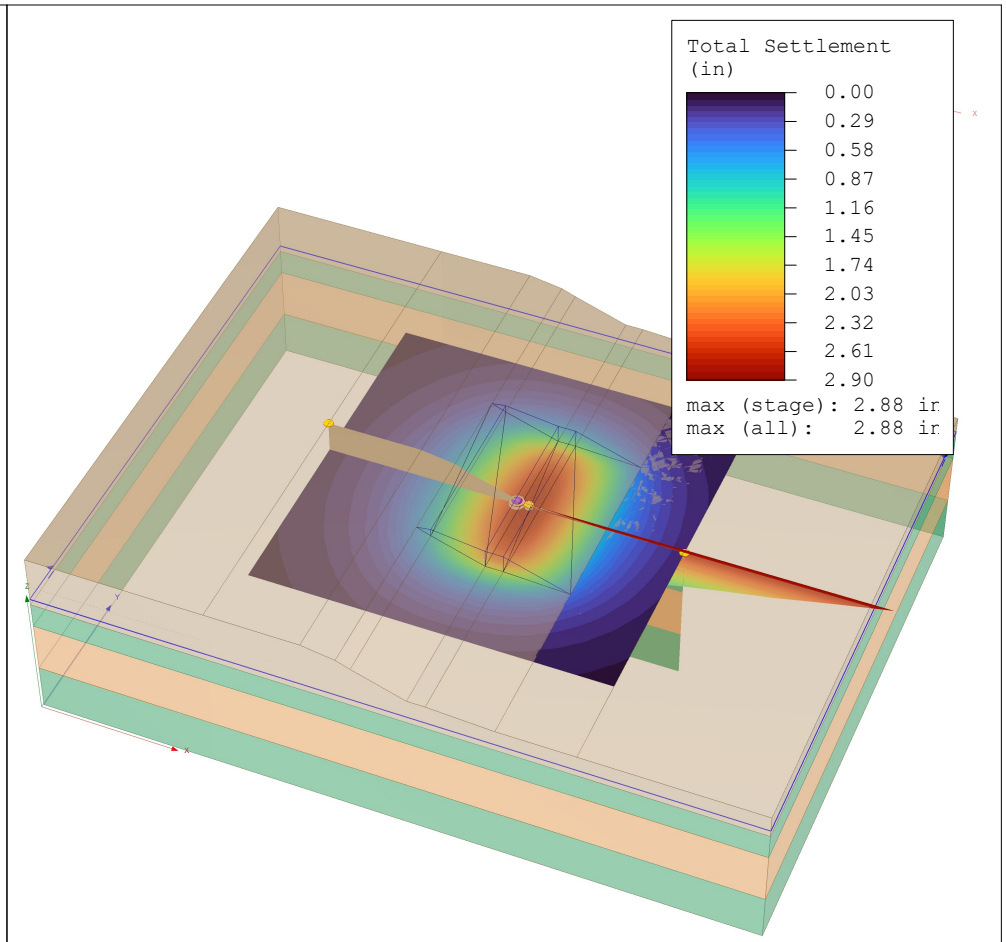




rocscience	*Project*	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	
	Analysis Description	Sta. 111+00 Settlement	
	Drawn By TPJ	*Company*	Haley & Aldrich, Inc.
SETTLE3 5.022	*Date* 3/16/2026, 12:50:38 PM	*File Name*	2026-0316-Palmyra-Sta 111+00-Settlement-d1.s3z

75 YEARS





rocscience	*Project*	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	
	Analysis Description	Sta. 111+00 Settlement	
	Drawn By TPJ	*Company*	Haley & Aldrich, Inc.
SETTLE3 5.022	*Date* 3/16/2026, 12:50:38 PM	*File Name*	2026-0316-Palmyra-Sta 111+00-Settlement-d1.s3z

Estimate of Single Pile Nominal Lateral Capacity for Global Stability Analyses

HALEY ALDRICH	CALCULATIONS	File No.:	0208880-000
		Sheet:	1 of 2
Client:	HNTB Corporation	Date:	3/23/2026
Project:	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	Computed by:	NAS
Subject:	Estimate of Single Pile Nominal Lateral Capacity for Global Stability Analyses	Checked by:	CFE

PROBLEM STATEMENT & OBJECTIVE

Estimate nominal lateral capacity of a single pile for global stability analyses.

EXECUTIVE SUMMARY

Assume each pile can resist 14 kips of shear for global stability analyses.

REFERENCES

1. Pile Foundation Analysis and Design, Poulos and Davis, 1980.
2. FHWA NHI-05-042, Design and Construction of Driven Pile Foundations, Reference Manual, Volume 1, April 2006.
3. NAVFAC DM7.2 Foundations and Earth Structures, 2025.

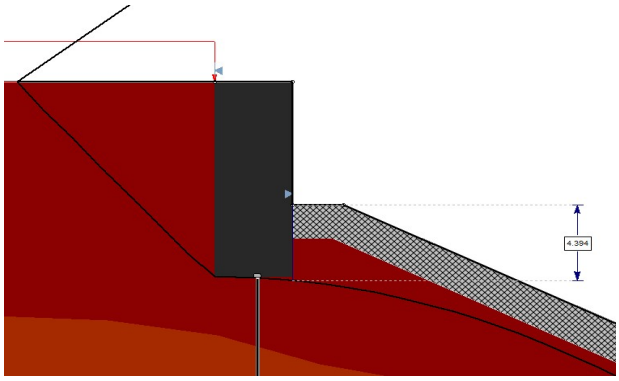
AVAILABLE INFORMATION

Haley & Aldrich, Inc. Preliminary Geotechnical Design Report for subject project, dated 30 October 2025.

ASSUMPTIONS

1. Lateral capacity of pile derived from prop. embank. soils w/in crit. fail. surf. below foreslope bench (see snip below).
2. Future lateral pile analyses conducted by HNTB may inform allowable lateral capacity of piles.
3. Fixed-head and short pile conditions.

<u>Snip of Critical Stability Failure Surface Approx. 4.4 ft Below Foreslope Bench (through Proposed Embankment Fill):</u>



PROCEDURE

Follow procedure in Section 7.2.2 (Brom's Theory) in Poulos and Davis, 1980.

CALCULATIONS

See Sheet 2.

= Input Cell

STEP 1. DETERMINE SOIL PROFILE WITHIN CRITICAL STABILITY FAILURE SURFACE (SEE SNIP ON PREVIOUS SHEET)

Material	Depth to Bottom of Material (ft)	Effective Unit Weight, γ' (pcf)	Friction Angle, ϕ (degrees)
Foreslope Bench	0	--	--
Proposed Embank. Fill	4.4	125	32

STEP 2. CHECK IF SHORT PILE ASSUMPTION IS VALID

Bottom of New Fill (Cohesionless); Refer to Equation 6-78 in NAVFAC DM 7.2

Elastic modulus of the pile, E_p =	29,000	ksi
Moment of inertia of the pile, I_p =	904	$in.^4$
Pile width, d =	14	in. (HP14x102 pile)
Increase rate in subgrade modulus with depth for sands, n_h =	1.11	ksi/ft (Table 6-44, medium dense and dry/moist)
Depth to fixity, Z_f =	7.4	ft
Z/Z_f =	0.6	< 1.1; short pile assumption valid

STEP 3. DETERMINE NOMINAL LATERAL RESISTANCE AT CRITICAL STABILITY FAILURE SURFACE

Bottom of New Fill (Cohesionless); Refer to Equation 7.19

Pile width, d =	14	in. (HP14x102 pile)
Passive earth pressure coefficient, K_p =	3.3	
Nominal lateral resistance of soil, H_u =	**14**	**kips**

Global Stability

HALEY ALDRICH	CALCULATIONS	File No.:	0208880-000
		Sheet:	1 of 1
Client:	HNTB Corporation	Date:	3/24/2026
Project:	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	Computed by:	TPJ
Subject:	Global Stability	Checked by:	EMH

PROBLEM STATEMENT AND OBJECTIVE

Calculate the global stability factors of safety at Abutment Nos. 1 and 2 in the longitudinal and transverse directions.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th edition, 2024
2. Slide2 version 9.029 by RocScience.
3. MaineDOT Bridge Design Guide, 2003 with interim revisions through June 2018.

AVAILABLE INFORMATION

1. 60% progress set plans provided by HNTB on 11 March 2026.
2. Boring logs BB-PI95-101, -102, -103, -201, -202, and -203.

ASSUMPTIONS

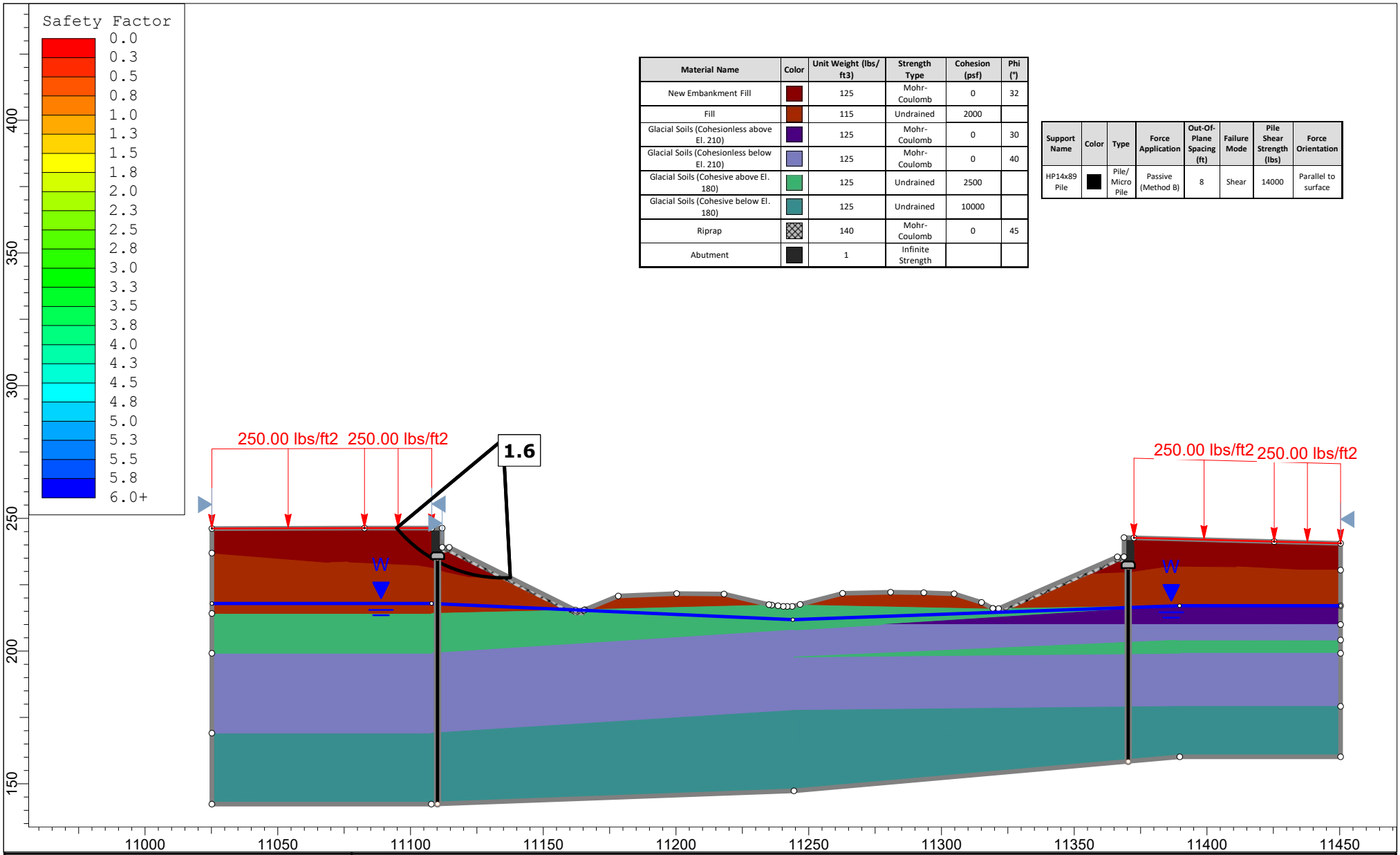
1. Water levels based on measurements during drilling.
2. A 250 psf traffic surcharge will be modeled along the roadway.
3. Seismic cases will have a seismic force of $A_s/2$ (0.152g/2) = 0.076g based on the seismic site class calculations.
4. Failure surfaces were assessed using Spencer's method.
5. Stability model stratigraphy and soil parameters based on boring logs and design soil properties calculations.
6. Considered circular and non-circular failure surfaces.
7. Refer to Estimate of Single Pile Nominal Lateral Capacity for Global Stability Analyses Calc Package for lateral resistance calculation of HP14x102 piles supporting the abutments (14k and 8 ft pile spacing).
8. New Embankment Fill is assumed to be granular borrow with a friction angle of 32° and unit weight of 125 pcf.
9. Based on soil descriptions from the boring logs Fill is assumed to be cohesive with an undrained shear strength of 2,000 psf and unit weight of 115 pcf. The upper 2 ft of Fill for the Abut. No. 2 Transverse section is assumed to be cohesive with an undrained shear strength of 1000 psf and unit weight of 115 pcf.
10. The self-weight of the abutment and abutment loads will be transferred directly to the piles.

RESULTS

Proposed Substructure	Direction of Failure Surface to Roadway	Calculated Factor of Safety			
		Static		Pseudo-Static	
		Circular	Non-Circular	Circular	Non-Circular
Abut. No. 1	Longitudinal	1.6	1.4	1.4	1.2
	Transverse	1.4	1.4	1.1	1.1
Abut. No. 2	Longitudinal	1.8	1.5	1.6	1.3
	Transverse	2.0	1.8	1.7	1.5

Based on AASHTO LRFD Section 11.6.2.3, an acceptable resistance factor for where the geotechnical parameters and subsurface stratigraphy are well defined is 0.75 (F.S. = 1/0.75 = **1.3**).

Based on recent MaineDOT guidance (L. Krusinski), a minimum pseudo-static factor of safety of 1.1 is acceptable.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Embankment Fill		125	Mohr-Coulomb	0	32
Fill		115	Undrained	2000	
Glacial Soils (Cohesionless above El. 210)		125	Mohr-Coulomb	0	30
Glacial Soils (Cohesionless below El. 210)		125	Mohr-Coulomb	0	40
Glacial Soils (Cohesive above El. 180)		125	Undrained	2500	
Glacial Soils (Cohesive below El. 180)		125	Undrained	10000	
Riprap		140	Mohr-Coulomb	0	45
Abutment		1	Infinite Strength		

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
HP14x89 Pile		Pile/ Micro Pile	Passive (Method B)	8	Shear	14000	Parallel to surface

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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Longitudinal	*Scenario*	Static - Circular
Drawn By	Sherwood	*Company*	H&A
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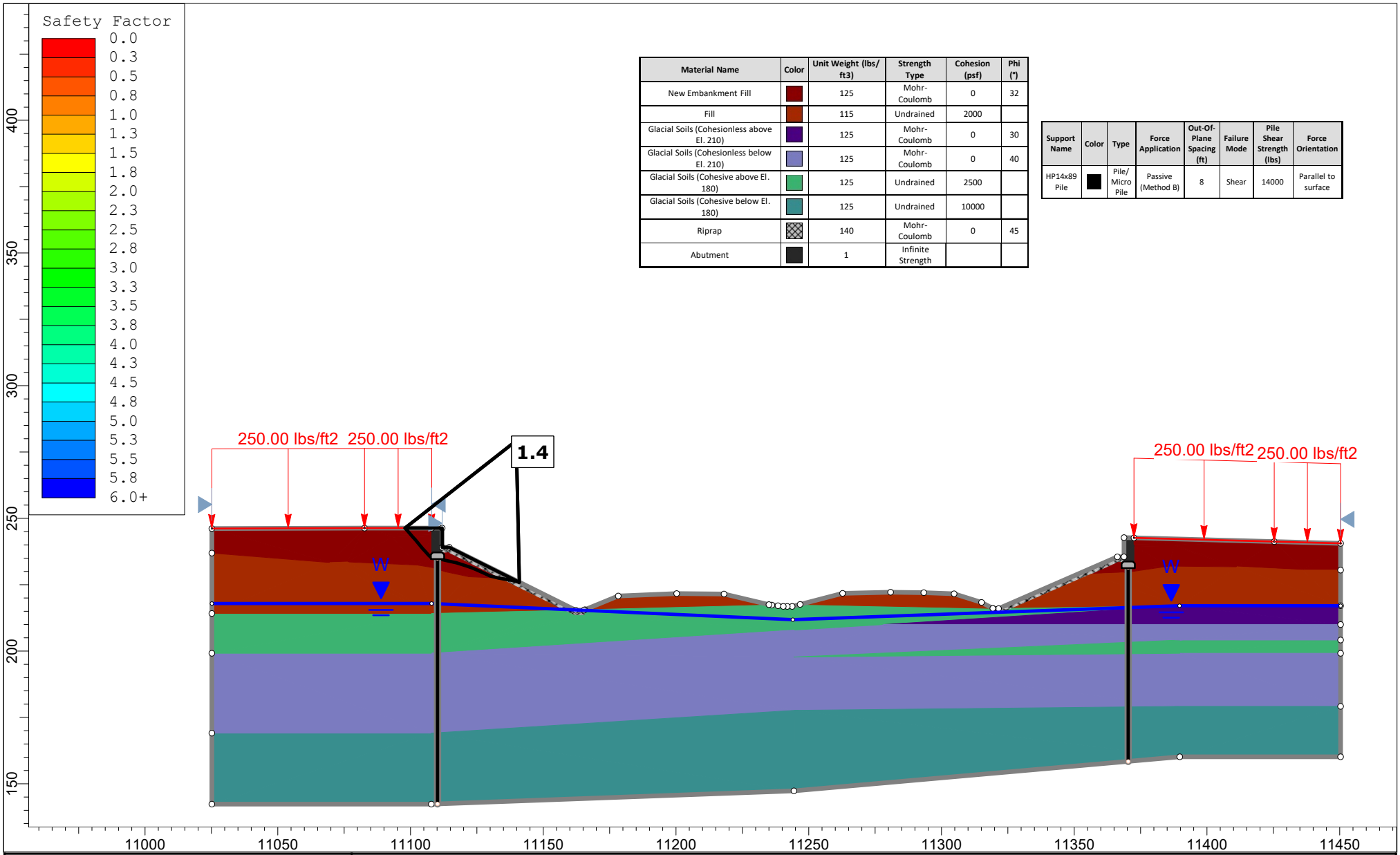

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Glacial Soils (Cohesionless below El. 210)		125	Mohr-Coulomb	0	40
Glacial Soils (Cohesive above El. 180)		125	Undrained	2500	
Glacial Soils (Cohesive below El. 180)		125	Undrained	10000	
Riprap		140	Mohr-Coulomb	0	45
Abutment		1	Infinite Strength		

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
HP14x89 Pile		Pile/ Micro Pile	Passive (Method B)	8	Shear	14000	Parallel to surface

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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Longitudinal	*Scenario*	Static - Non-Circular
Drawn By	Sherwood	*Company*	H&A
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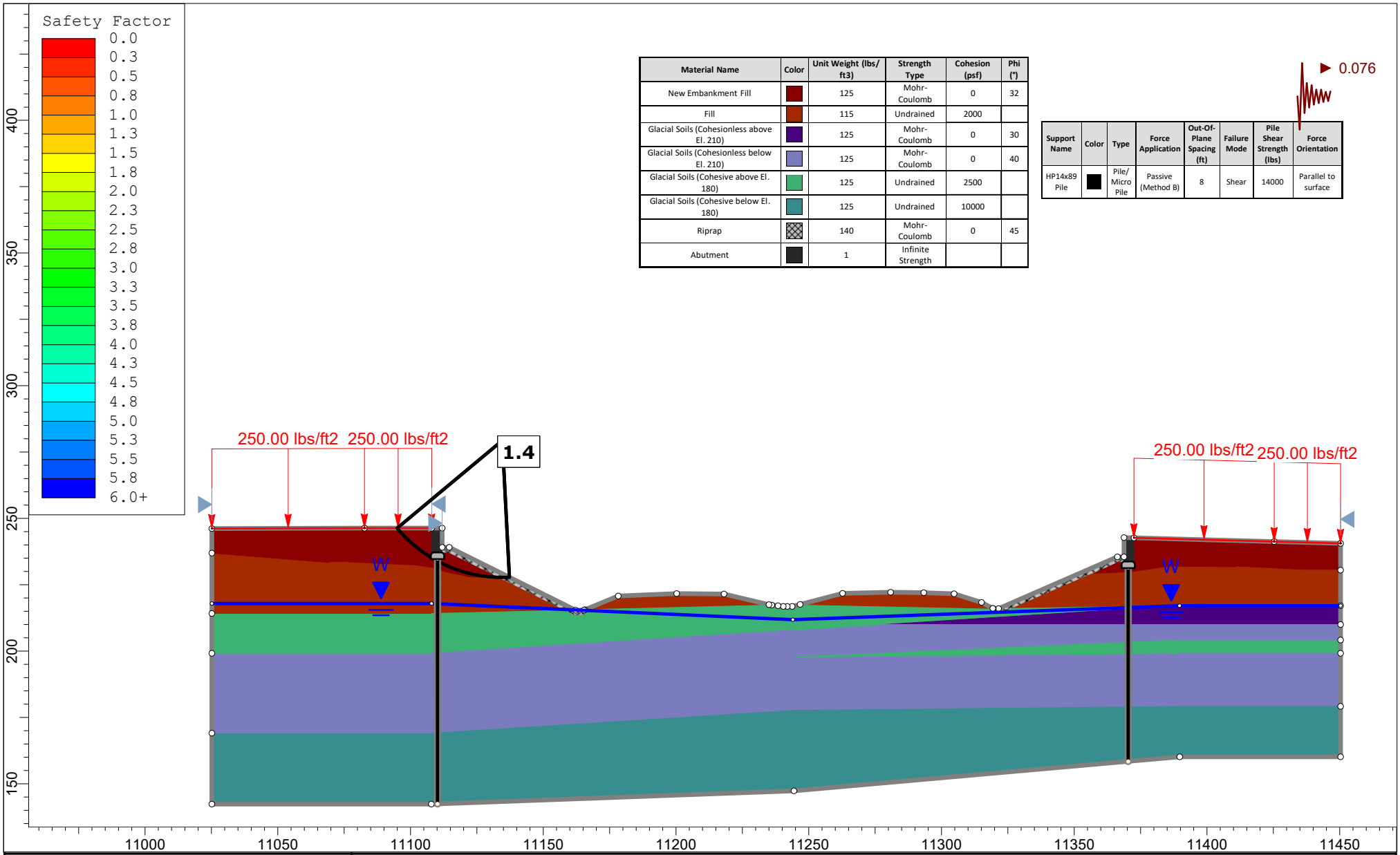

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Glacial Soils (Cohesive above El. 180)		125	Undrained	2500	
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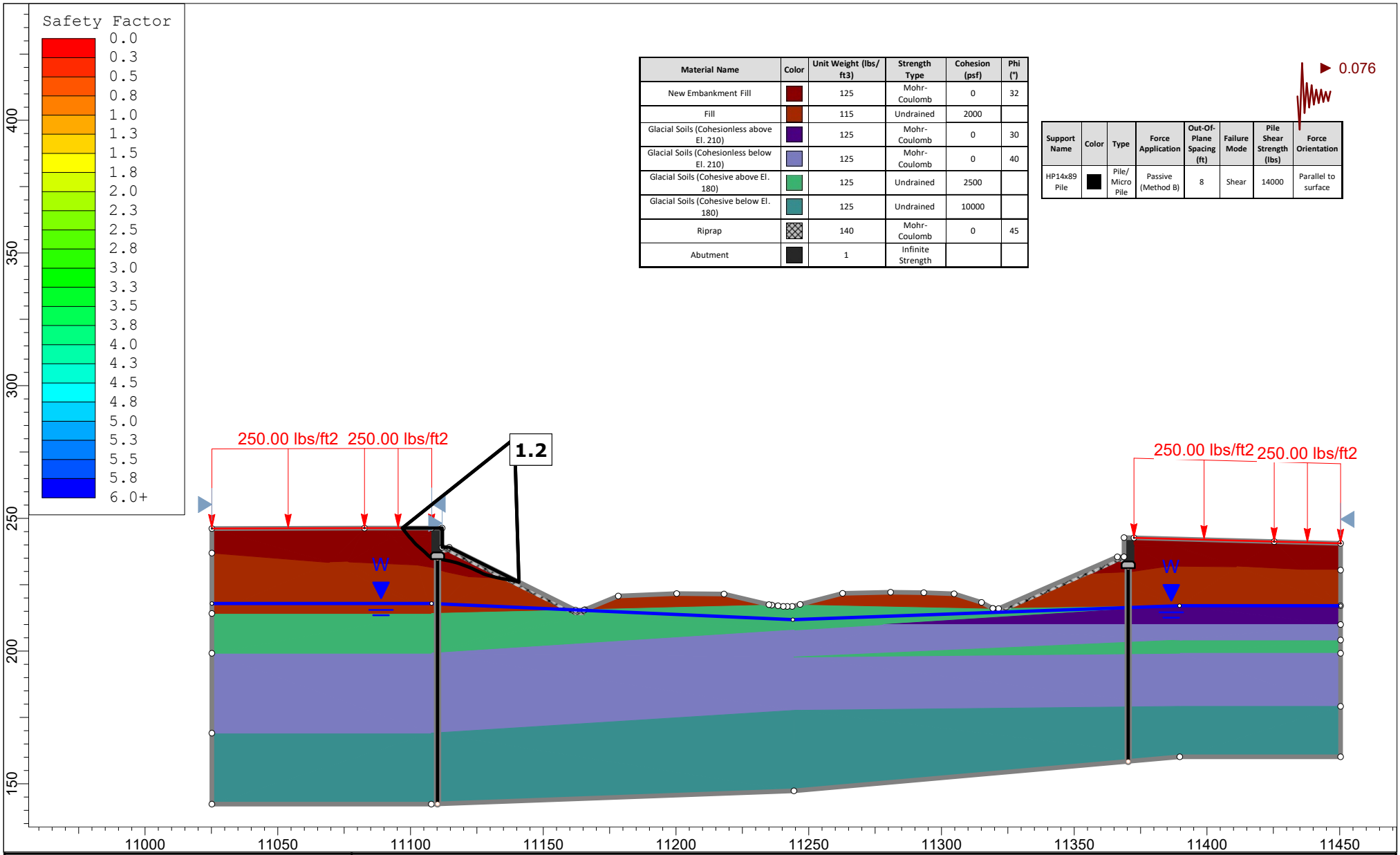
SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Longitudinal	*Scenario*	Pseudo-Static - Circular
Drawn By	Sherwood	*Company*	H&A
Date	2/24/2025, 1:16:20 PM	*File Name*	2026-0307-HAI-Palmyra-Abut 1-Long-D4.slmd



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Abutment		1	Infinite Strength		

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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Longitudinal	*Scenario*	Pseudo-Static - Non-Circular
Drawn By	Sherwood	*Company*	H&A
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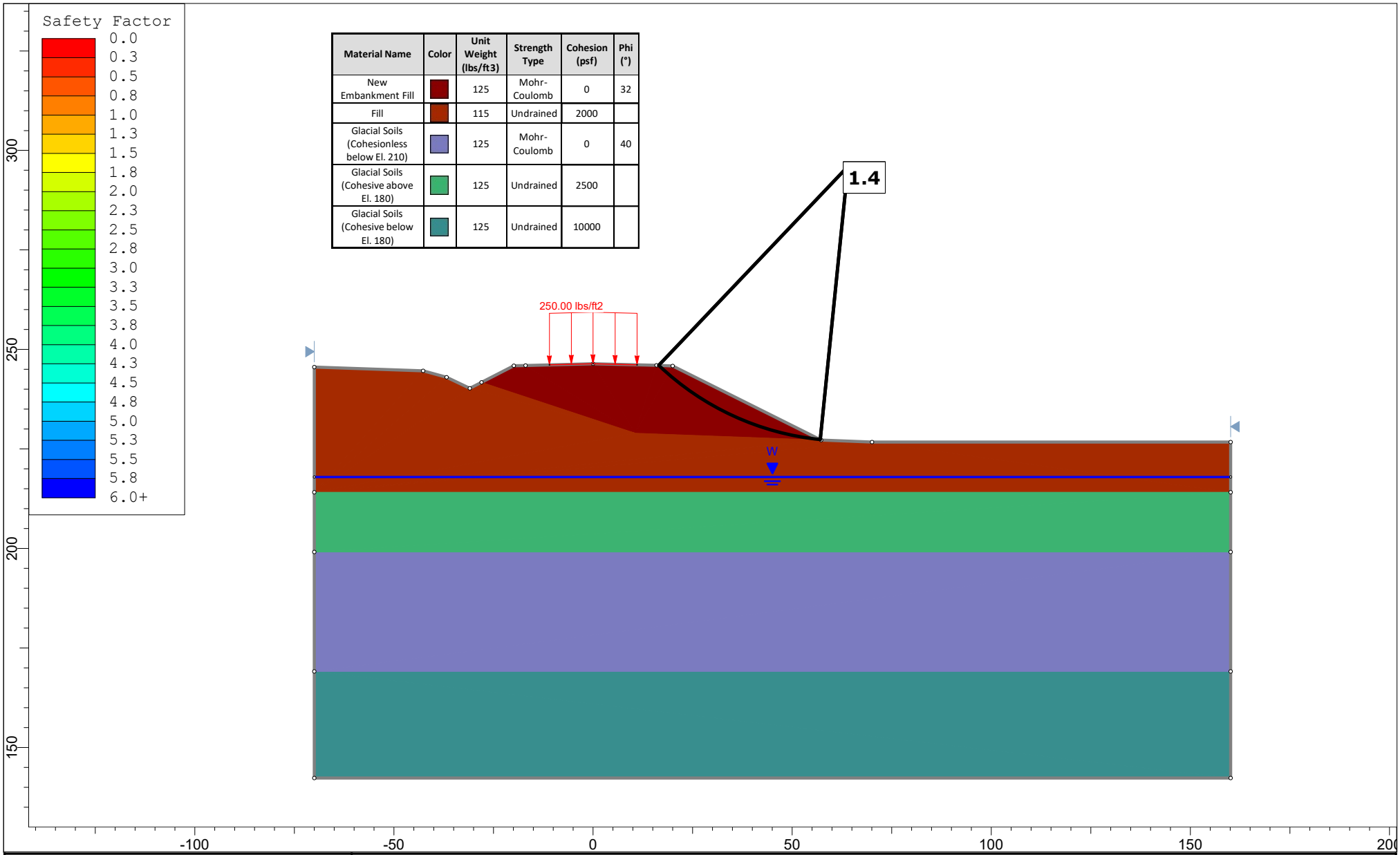

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Embankment Fill		125	Mohr-Coulomb	0	32
Fill		115	Undrained	2000	
Glacial Soils (Cohesionless below El. 210)		125	Mohr-Coulomb	0	40
Glacial Soils (Cohesive above El. 180)		125	Undrained	2500	
Glacial Soils (Cohesive below El. 180)		125	Undrained	10000	

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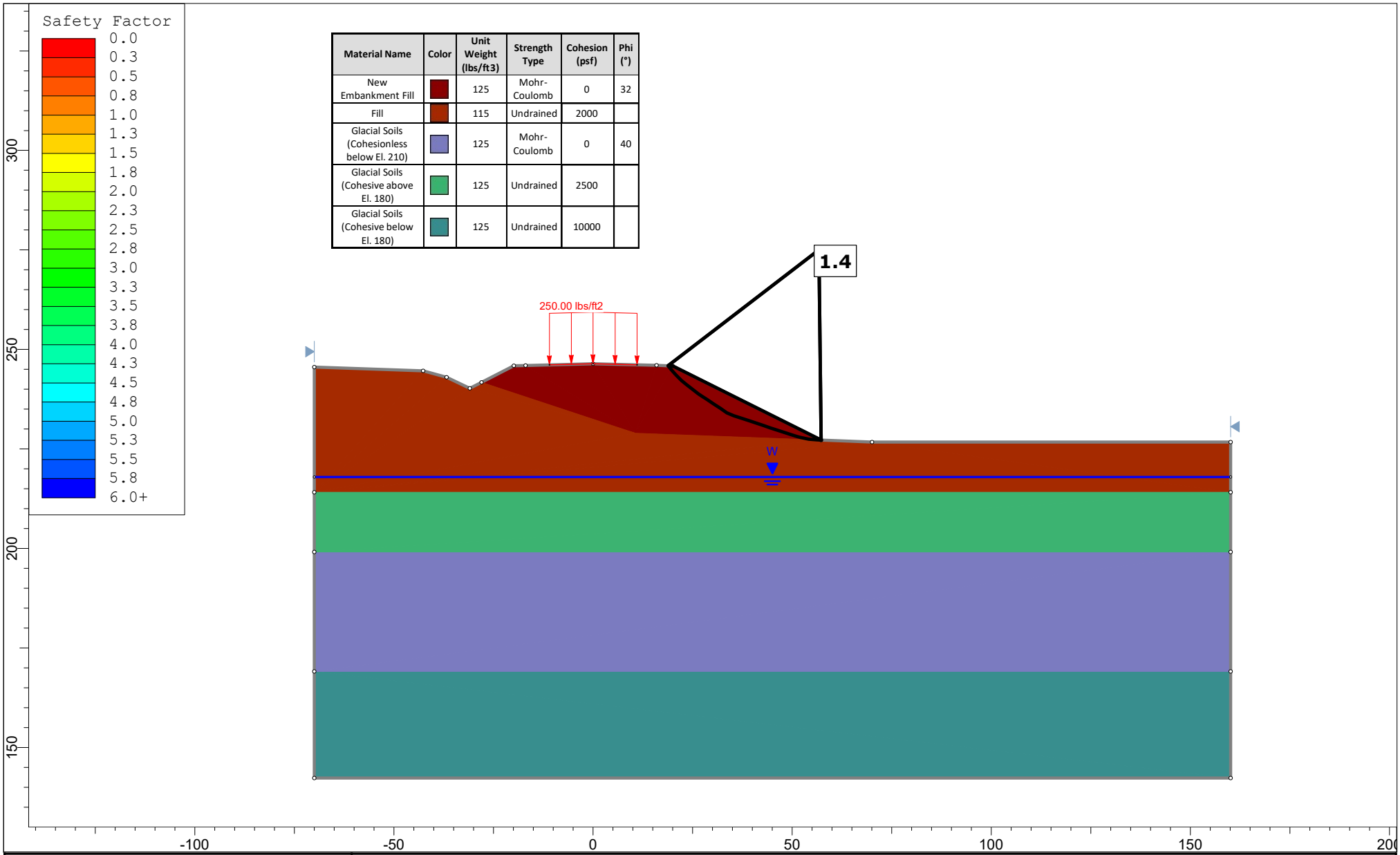
SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Transverse	*Scenario*	Static - Circular
Drawn By	Sherwood	*Company*	H&A
Date	2/24/2025, 1:16:20 PM	*File Name*	2026-0324-HAI-Palmyra-Abut 1-Transverse-D4.slmd



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Fill		115	Undrained	2000	
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Glacial Soils (Cohesive above El. 180)		125	Undrained	2500	
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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Transverse	*Scenario*	Static - Non-Circular
Drawn By	Sherwood	*Company*	H&A
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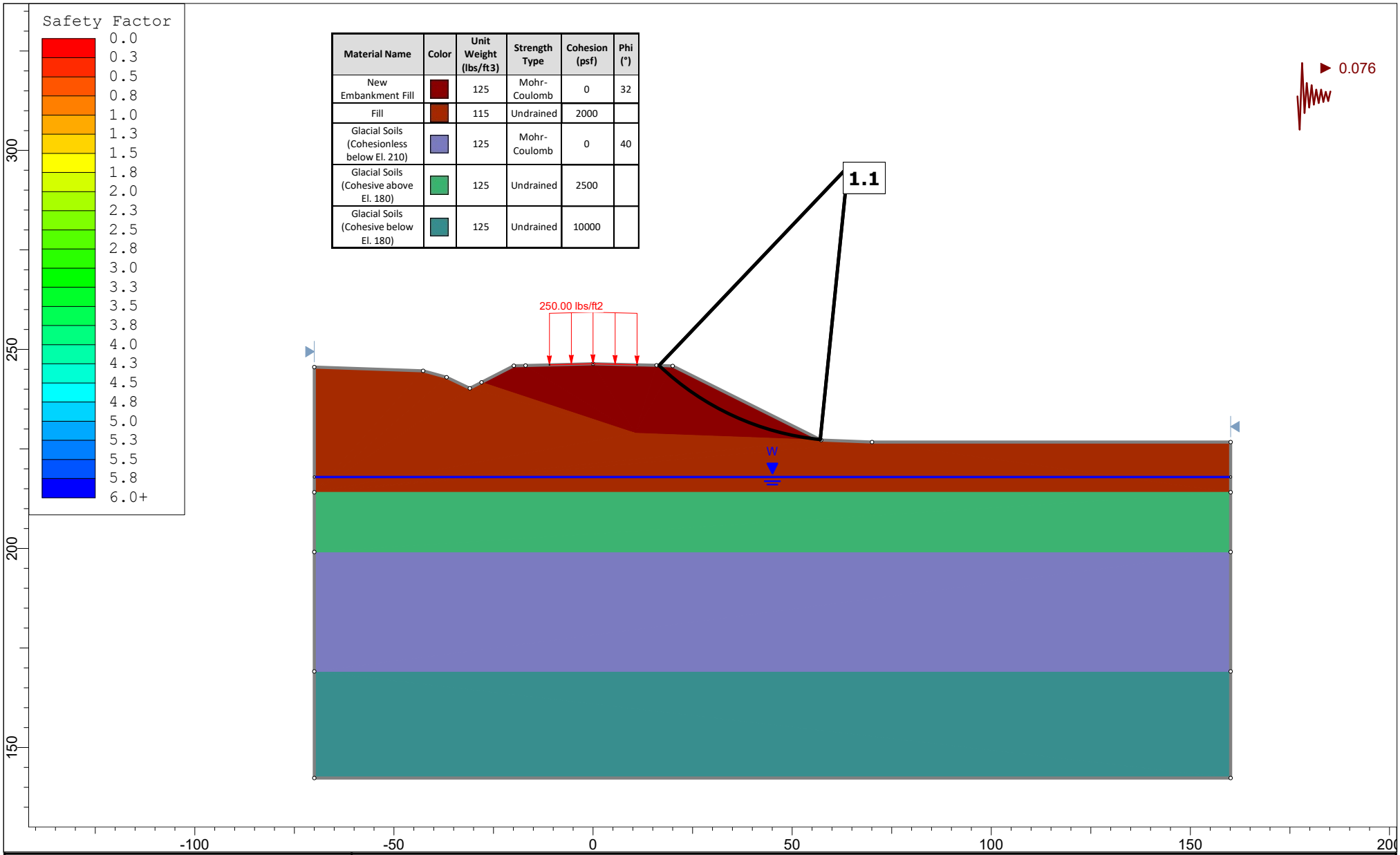

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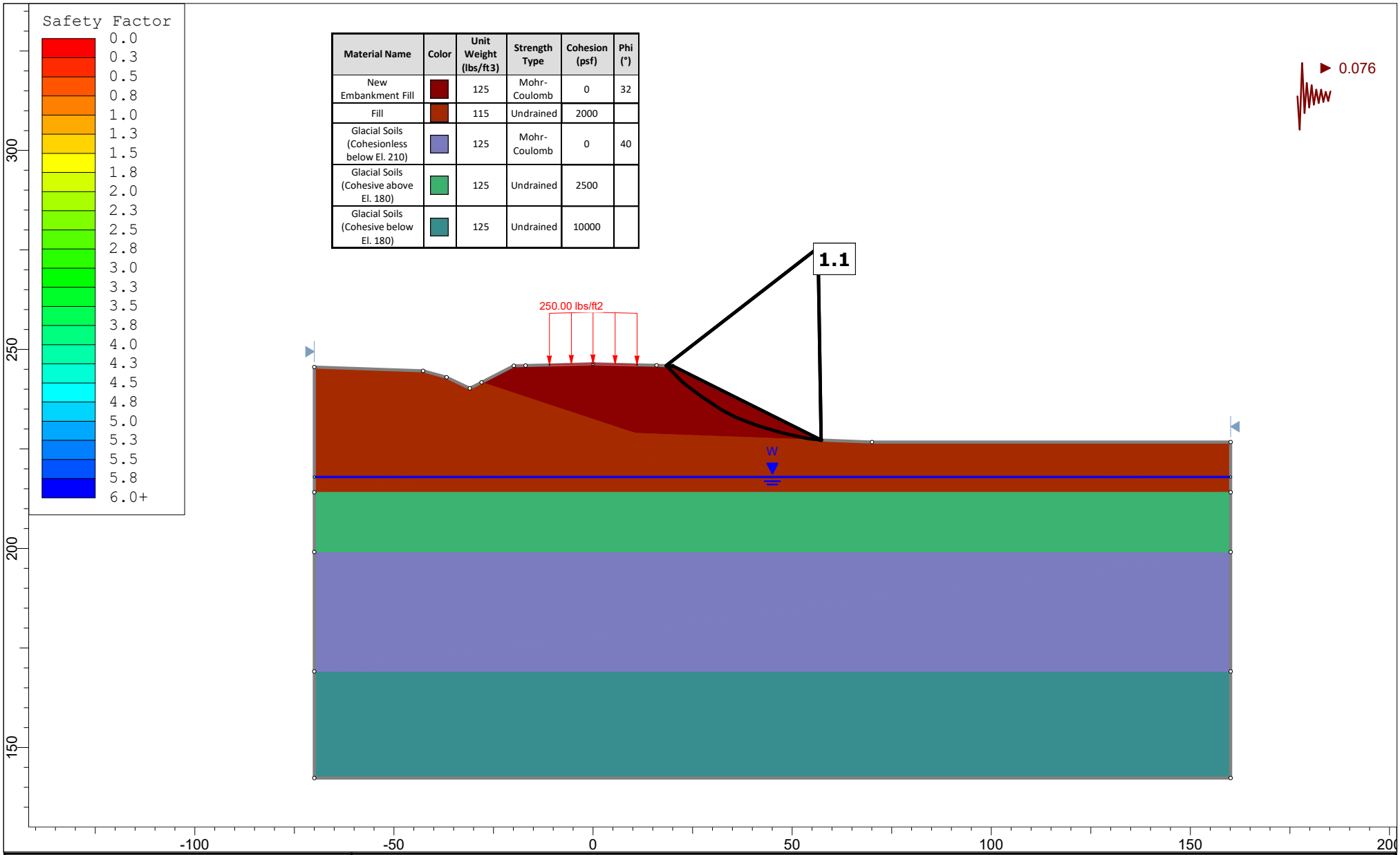
SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Transverse	*Scenario*	Pseudo-Static - Circular
Drawn By	Sherwood	*Company*	H&A
Date	2/24/2025, 1:16:20 PM	*File Name*	2026-0324-HAI-Palmyra-Abut 1-Transverse-D4.slmd



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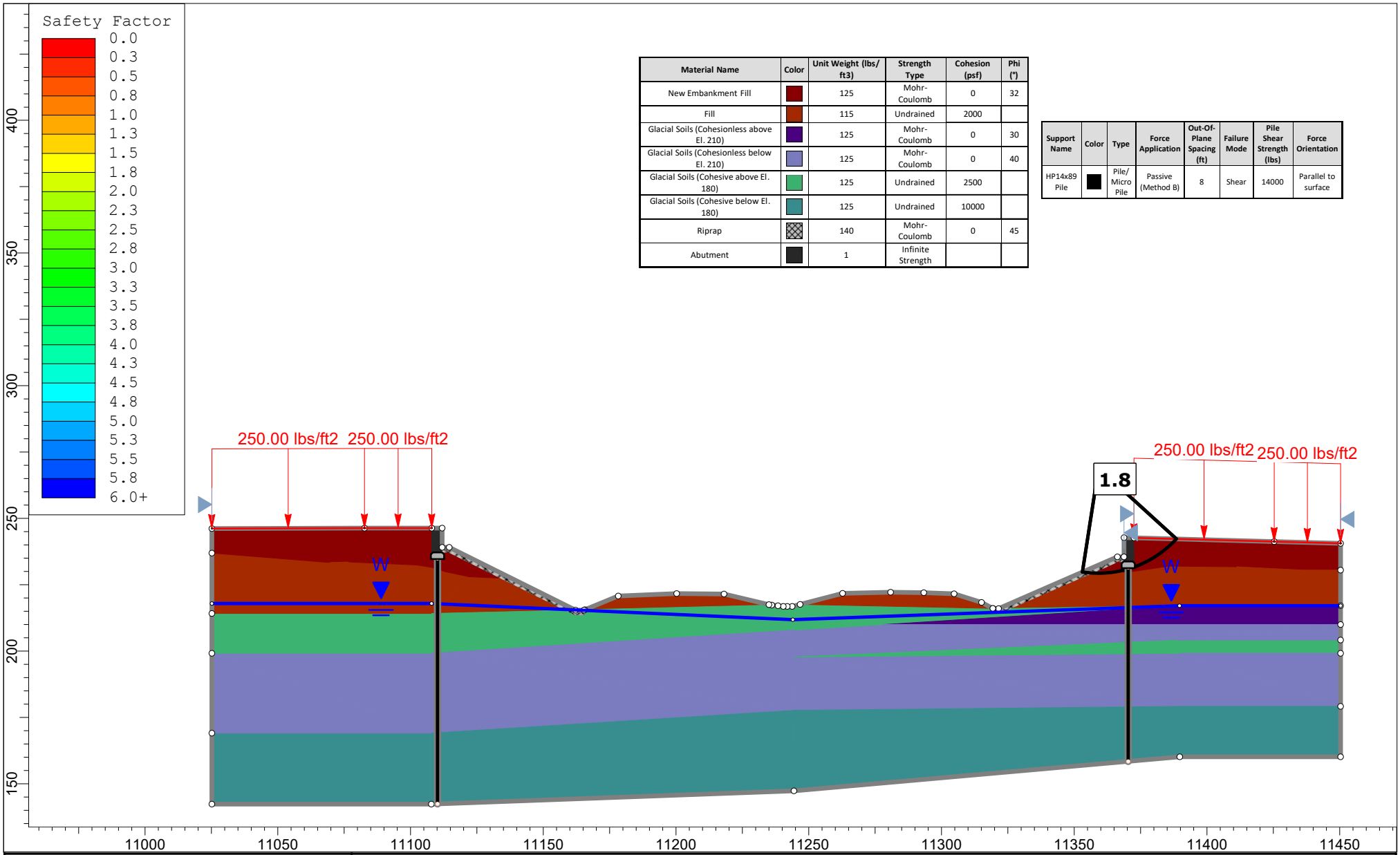
SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 1 - Transverse	*Scenario*	Pseudo-Static - Non-Circular
Drawn By	Sherwood	*Company*	H&A
Date	2/24/2025, 1:16:20 PM	*File Name*	2026-0324-HAI-Palmyra-Abut 1-Transverse-D4.slmd



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Riprap		140	Mohr-Coulomb	0	45
Abutment		1	Infinite Strength		

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
HP14x89 Pile		Pile/ Micro Pile	Passive (Method B)	8	Shear	14000	Parallel to surface

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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 2 - Longitudinal	*Scenario*	Static - Circular
Drawn By	Sherwood	*Company*	H&A
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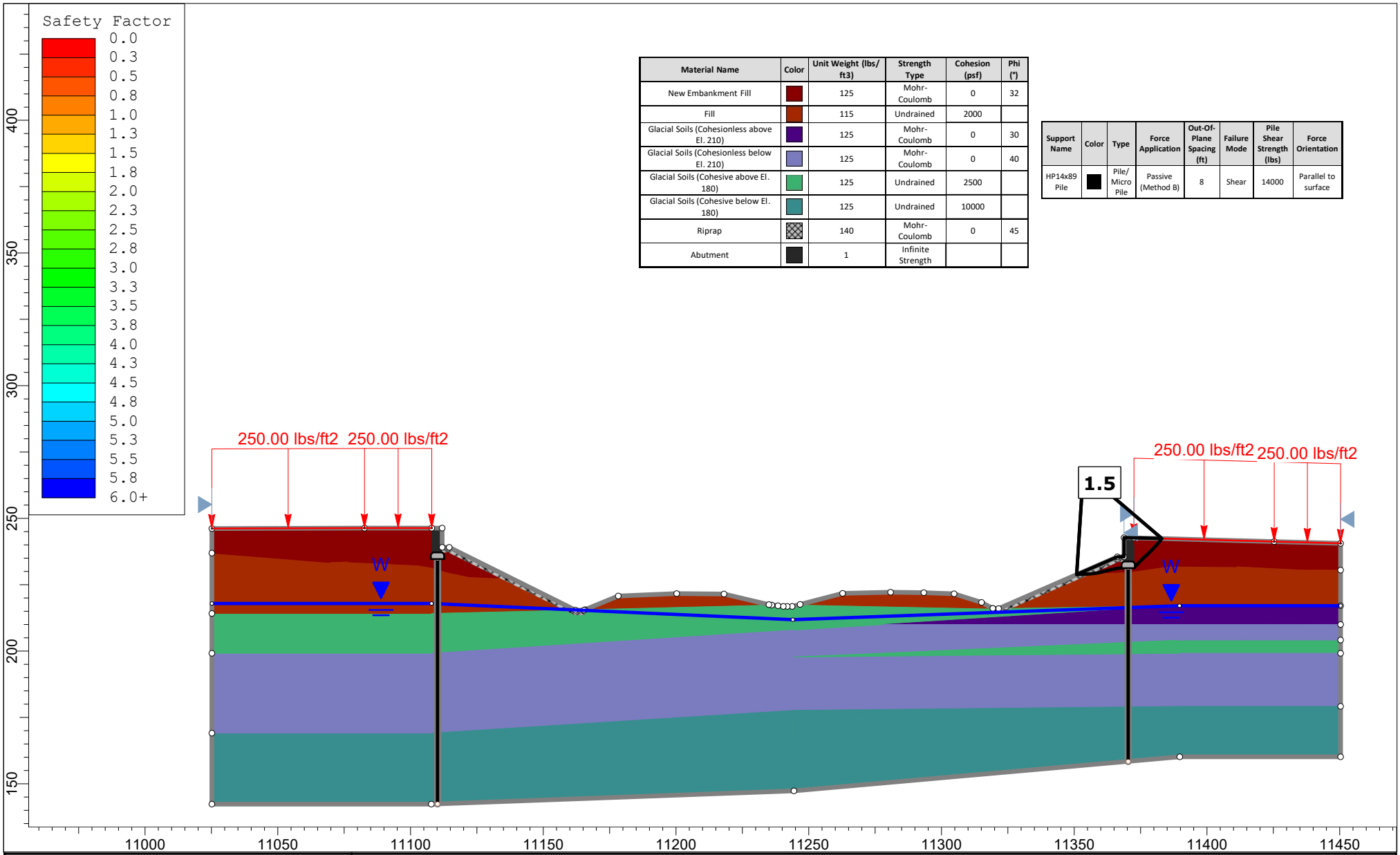

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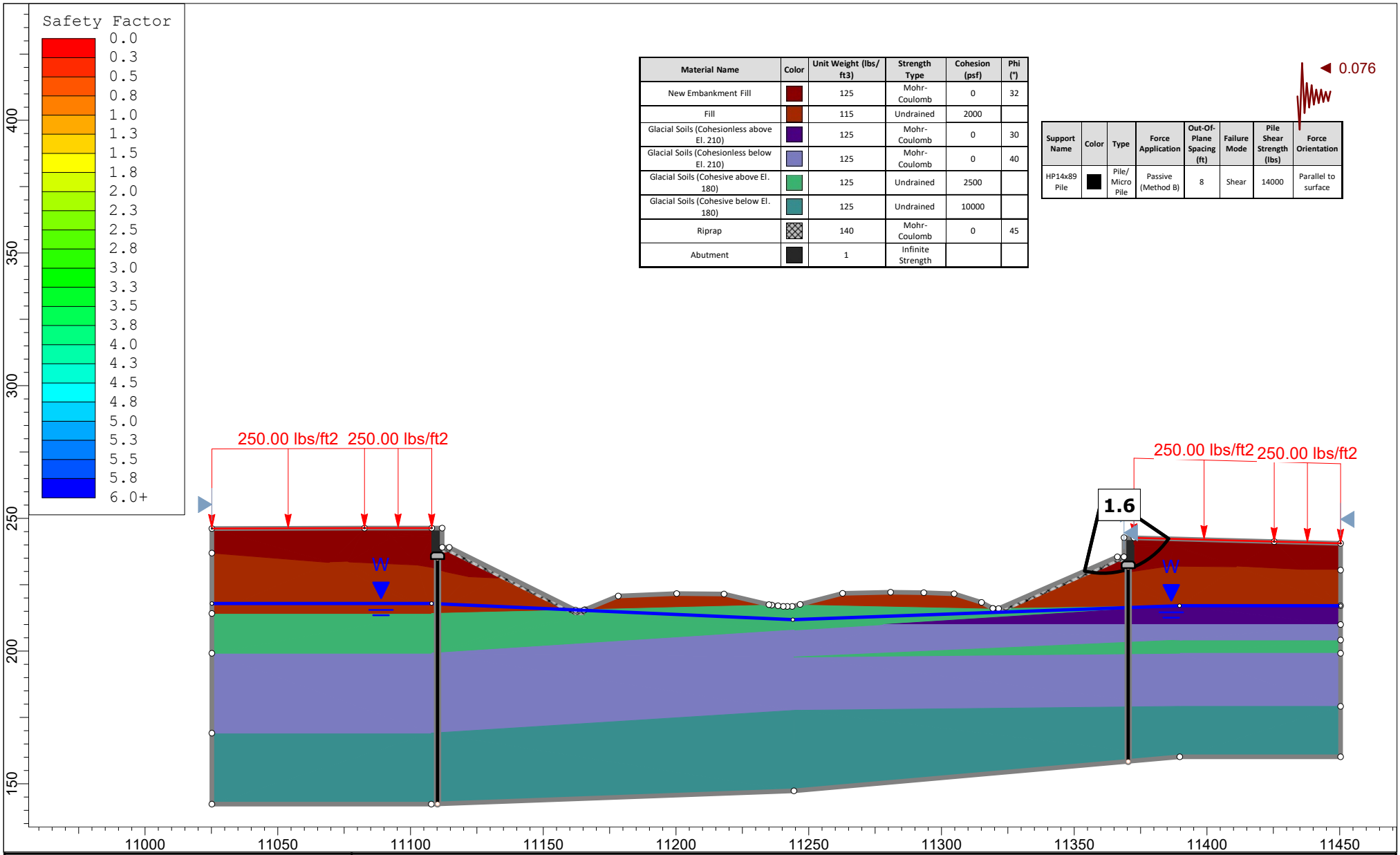
SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 2 - Longitudinal	*Scenario*	Static - Non-Circular
Drawn By	Sherwood	*Company*	H&A
Date	2/24/2025, 1:16:20 PM	*File Name*	2026-0307-HAI-Palmyra-Abut 2-Long-D4.slmd



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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 2 - Longitudinal	*Scenario*	Pseudo-Static - Circular
Drawn By	Sherwood	*Company*	H&A
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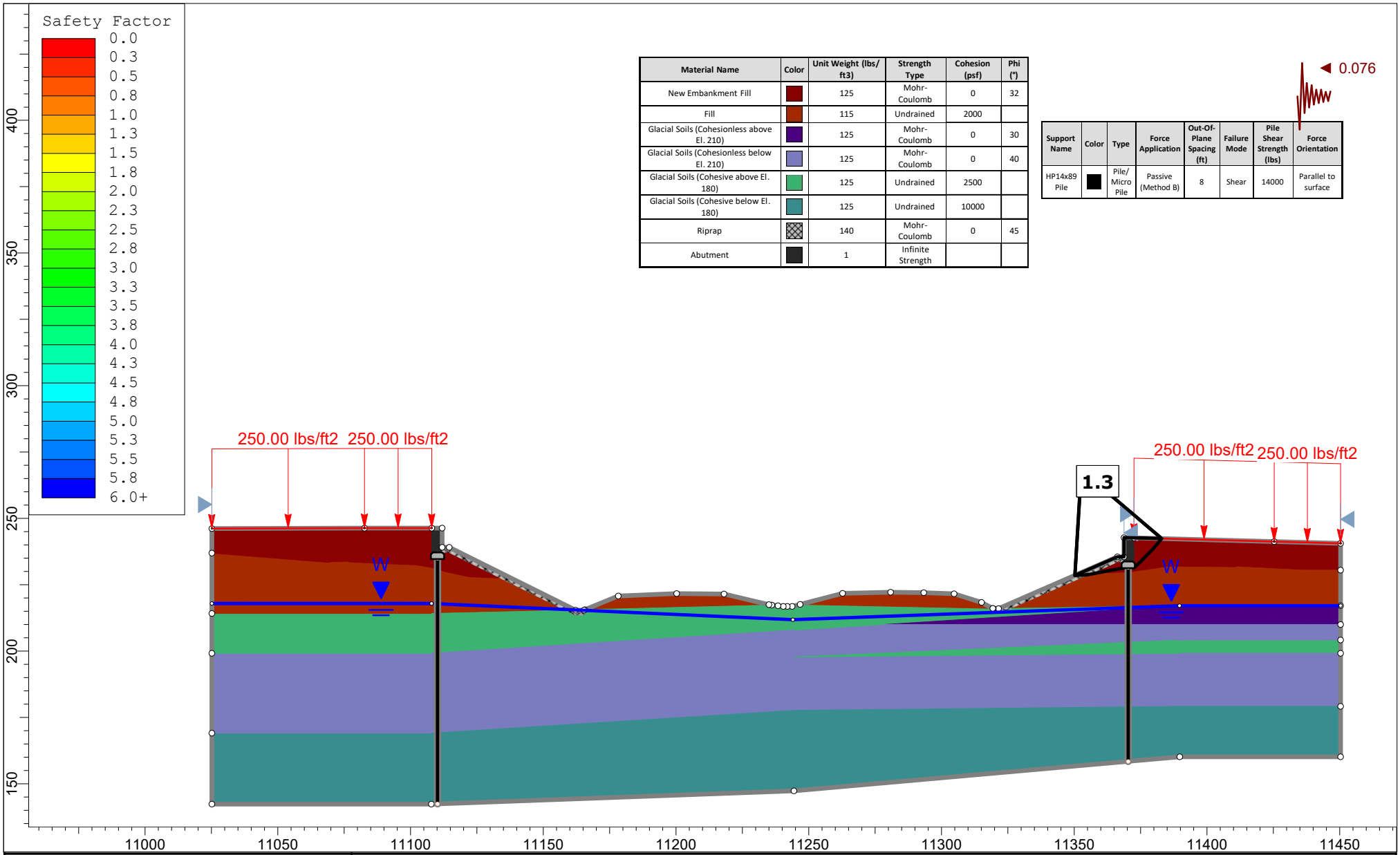

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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
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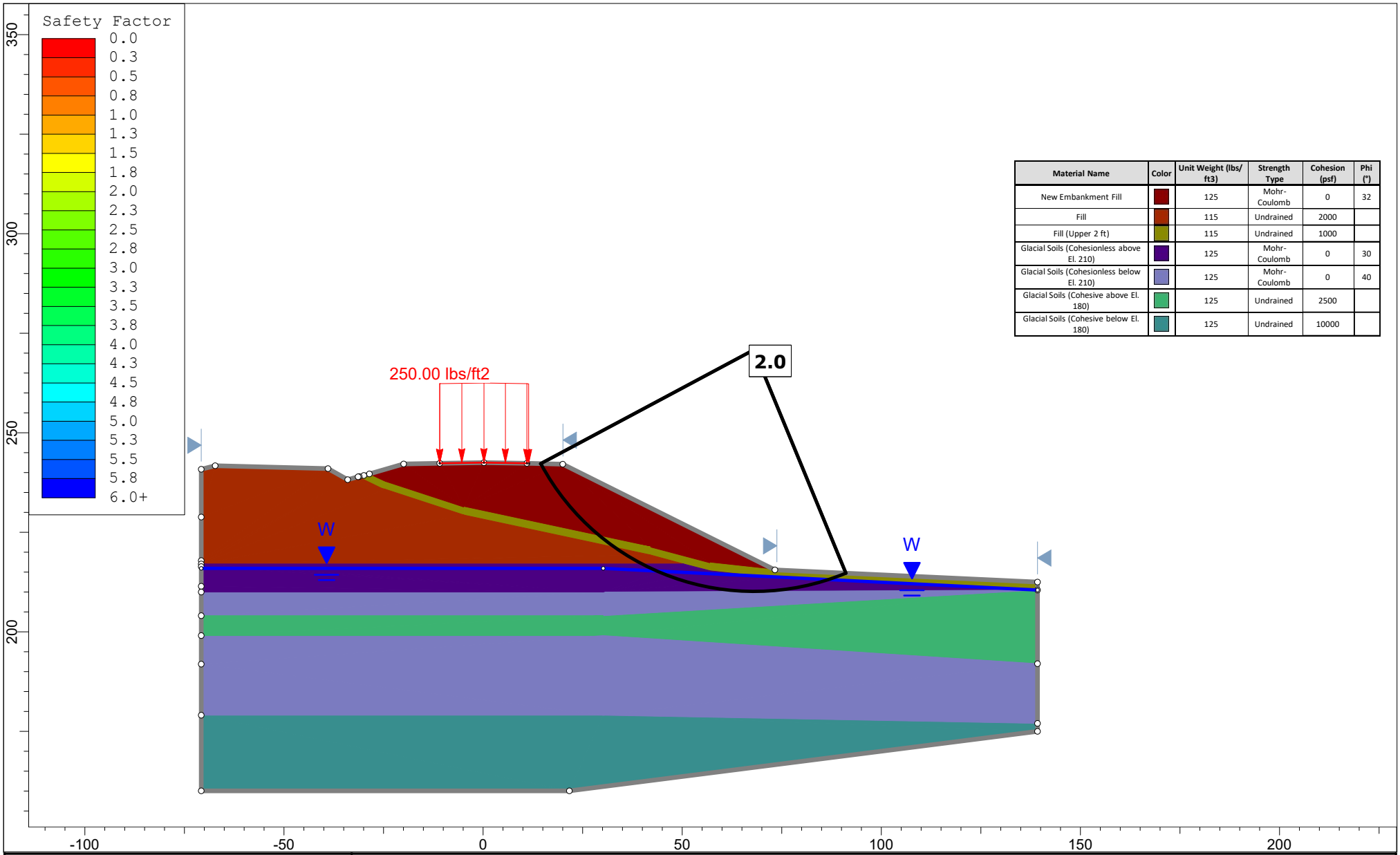
Drawn By	Sherwood	*Company*	H&A
Date	2/24/2025, 1:16:20 PM	*File Name*	2026-0307-HAI-Palmyra-Abut 2-Long-D4.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Embankment Fill		125	Mohr-Coulomb	0	32
Fill		115	Undrained	2000	
Fill (Upper 2 ft)		115	Undrained	1000	
Glacial Soils (Cohesionless above El. 210)		125	Mohr-Coulomb	0	30
Glacial Soils (Cohesionless below El. 210)		125	Mohr-Coulomb	0	40
Glacial Soils (Cohesive above El. 180)		125	Undrained	2500	
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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 2 - Transverse	*Scenario*	Static - Circular
Drawn By	Sherwood	*Company*	H&A
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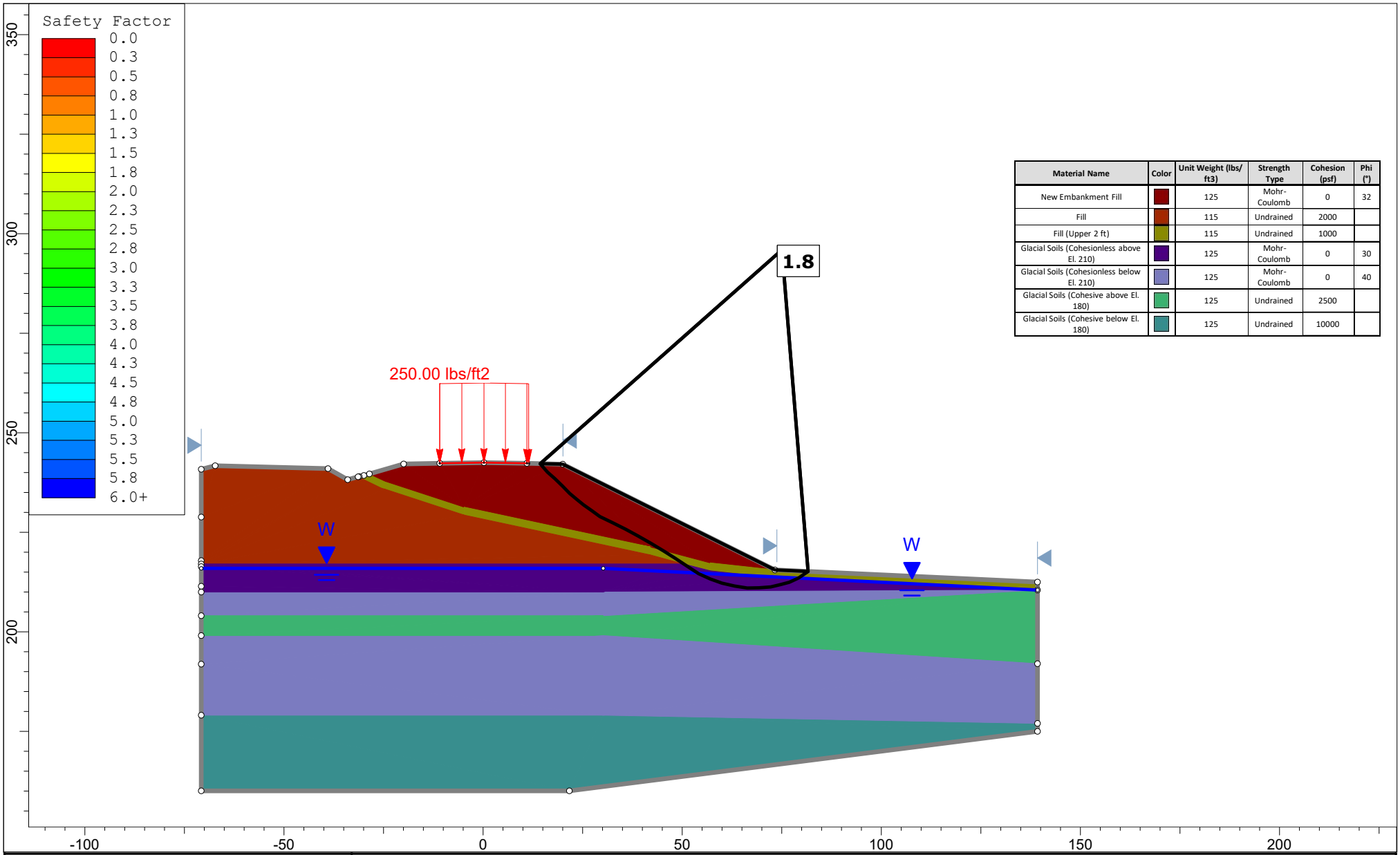

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
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Glacial Soils (Cohesionless below El. 210)		125	Mohr-Coulomb	0	40
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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 2 - Transverse	*Scenario*	Static - Non-Circular
Drawn By	Sherwood	*Company*	H&A
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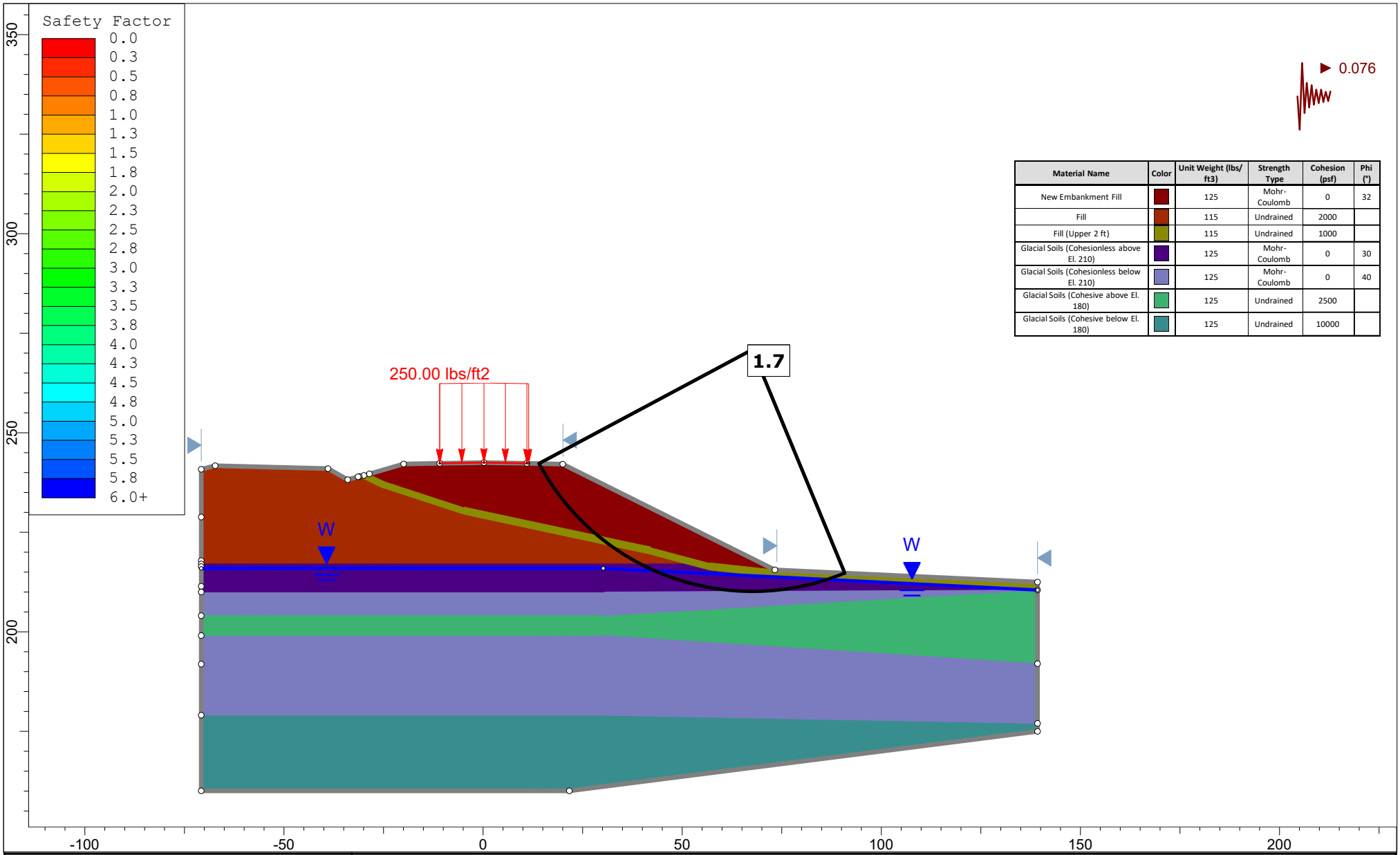

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SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 2 - Transverse	*Scenario*	Pseudo-Static - Circular
Drawn By	Sherwood	*Company*	H&A
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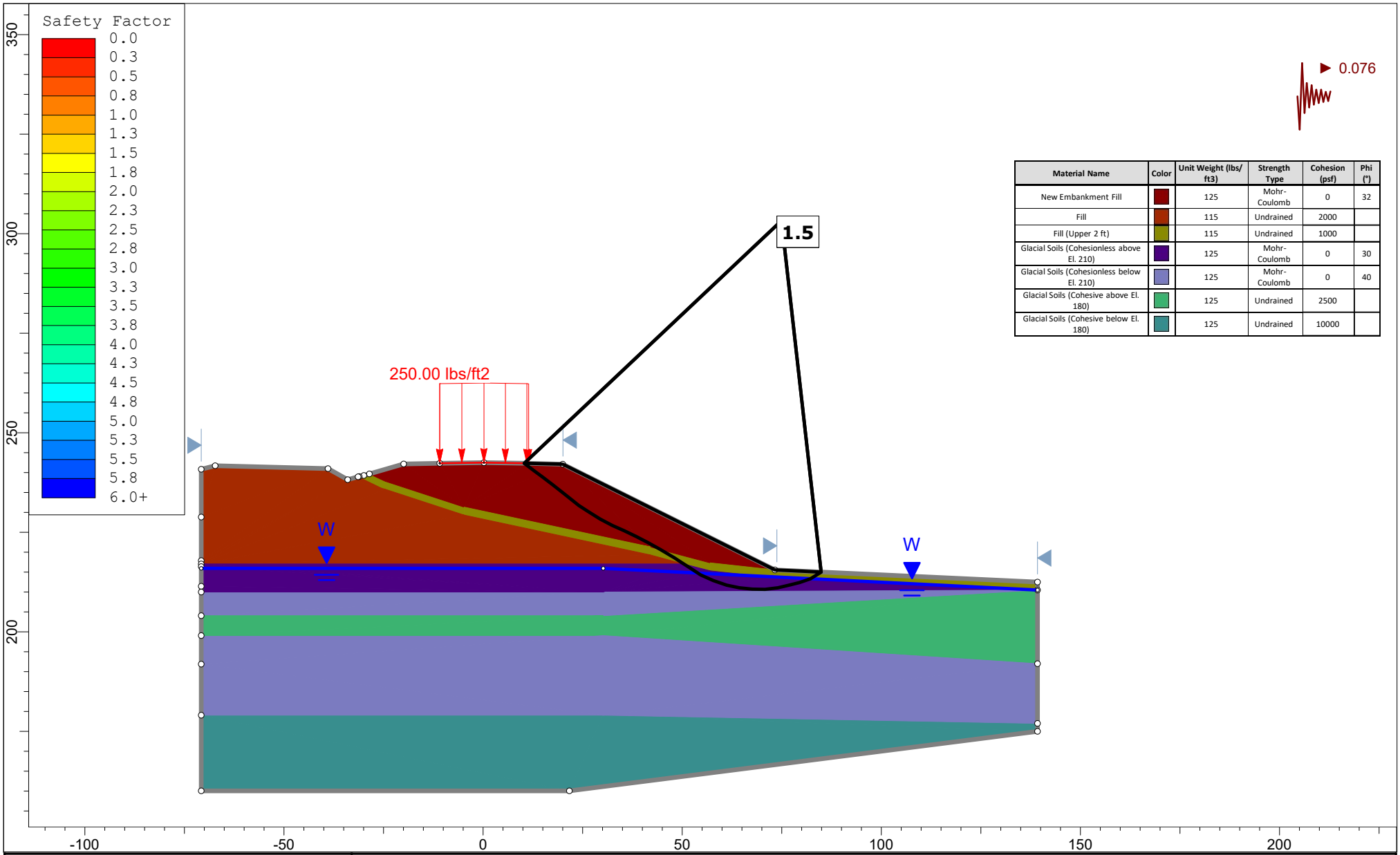

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Glacial Soils (Cohesionless above El. 210)		125	Mohr-Coulomb	0	30
Glacial Soils (Cohesionless below El. 210)		125	Mohr-Coulomb	0	40
Glacial Soils (Cohesive above El. 180)		125	Undrained	2500	
Glacial Soils (Cohesive below El. 180)		125	Undrained	10000	

rocscience

SLIDEINTERPRET 9.029

Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00		
Group	Abut. 2 - Transverse	*Scenario*	Pseudo-Static - Non-Circular
Drawn By	Sherwood	*Company*	H&A
Date	2/24/2025, 3:42:52 PM	*File Name*	2025-0224-HAI-Palmyra-Abut 2-Trans-D2.slmd

Pile Structural Resistance

PROBLEM STATEMENT AND OBJECTIVE

Calculate the structural axial pile resistances.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.

ASSUMPTIONS

1. Per HNTB request, assume HP14x102 steel H-pile for the structural evaluations.
2. A pile yield stress of 50 ksi was considered for the structural resistance evaluations.
3. The pile axial structural resistance was evaluated assuming no corrosion.
4. The pile unbraced length is 0 ft (fully embedded pile) for the structural axial resistance calculations.
5. A resistance factor of 0.5 for the pile axial compression at the Strength Limit State was selected based on AASHTO LRFD Section 6.5.4.2 (H-Piles subjected to severe driving conditions where use of a pile tip is necessary).
6. A resistance factor of 1.0 for the pile compression axial resistance at the Extreme Event Limit State was selected based on AASHTO LRFD Section 10.5.5.3.

SUMMARY

Steel H-pile Section	Pile Nominal Structural Resistance in Axial Compression (kips)	Factored Structural Resistance (kips)		
		Service Limit State (ϕ=1.0)	Strength Limit State (ϕ=0.5)	Extreme Limit State (ϕ=1.0)
HP14x102	1,505	1,505	753	1,505

Notes:

1. The structural resistance was calculated using Section 6.9.4.1 as referenced from Section 10.7.3.13.
2. The calculated resistance presented above only addresses axial resistance.
3. If the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated by others.

STRUCTURAL RESISTANCE - AASHTO LRFD

Note: This calculation only addresses axial resistance, if the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated by others. Refer to Section 6.15.

The nominal resistance shall not exceed the values obtained from Section 6.9.4.1 with the resistance factors specified in Section 6.5.4.2 and 6.15 for severe driving conditions.

10.7.3.13 - Pile Structural Resistance
10.7.3.12.1 - Steel Piles
The nomial axial compression resistance in the structural limit state for piles loaded in compression shall be as specified in Article 6.9.4.1 for non composite piles. The resistance factors for the compression limit state are specified in Article 6.5.4.2.

6.9.4 - Noncomposite Members
6.9.4.1 - Nominal Compressive Resistance

If $\frac{P_0}{P_e} \leq 2.25$, then $P_n = P_0 \ * (0.658^{\frac{P_0}{P_e}})$ 6.9.4.1.1-1

Otherise: $P_n = 0.877P_e$ 6.9.4.1.1-2

where
A_g = gross cross-sectional area of the member
F_y = specified minimum yield strength
P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional buckling or flexural-torisonal buckling as applicable
P_0 = nominal yield resistance = F_yA_g

6.9.4.1.2 - Elastic Flexural Buckling Resistance

The elastic critical buckling resistance, P_e, based on flexural buckling shall be taken as

$$P_e = \frac{\pi^2 E}{(\frac{Kl}{r_s})^2} A_g \qquad 6.9.4.1.2\text{-}1$$

where
A_g = gross cross-sectional area of the member
K = effective length factor in the plane of buckling determined as specified in Article 4.6.2.5
l = unbraced length in the plane of buckling
r_s = radius of gyration about the axis normal to the plane of buckling

STRUCTURAL AXIAL RESISTANCE CALCULATIONS

Pile	Gross Section Area A_g (in^2)	Corrosion (in)	Effective Gross Area A_{eff} (in^2)	Effective Length Factor K	Unbraced l (ft)	Yield Stress fy (ksi)	Elastic Modulus E (ksi)	Radius of Gyration r_{weak} (in)	P_0 (kips)	P_e (kips)	P_0/P_e	P_n (kips)	ϕ (Strength Limit State)	**ϕP_n (kips)**
HP14x102	30.1	0	30.1	2	0	50	29,000	3.56	1,505	1.9E+15	7.9E-13	1,505	0.5	**753**

Note: The resistance factor of 0.5 for the Strength Limit State was selected based on Section 6.5.4.2 for H-piles subjected to severe driving conditions where the use of a pile tip is necessary.

Pile Drivability Resistance

PROBLEM STATEMENT & OBJECTIVE

Determine nominal and factored driveability resistances for HP14x102 steel H-piles being considered to support the proposed abutments.

EXECUTIVE SUMMARY

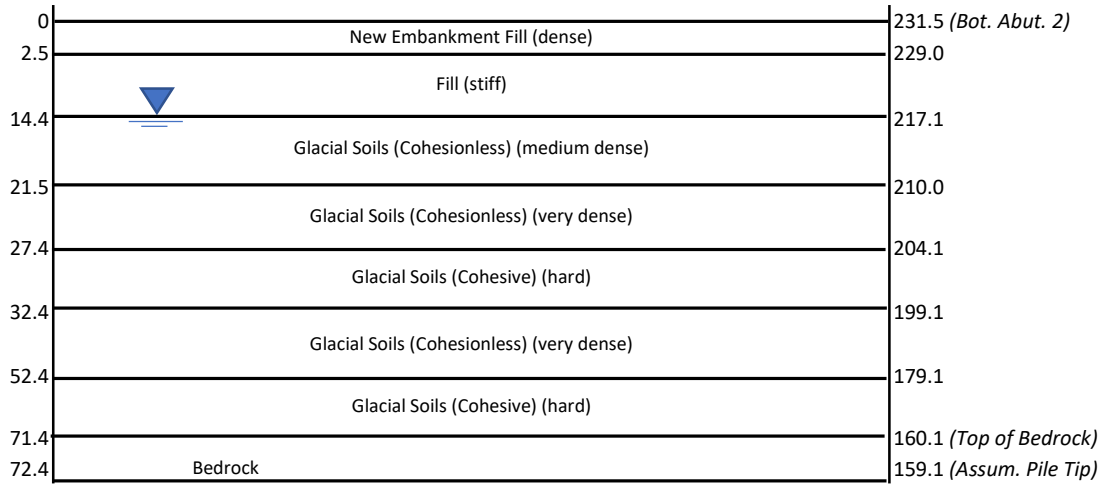
HP14x102 steel H-piles can be installed to a nominal resistance equal to 700 kips (factored resist. = 455 kips) with a Delmag D 30-32 OED hammer operating on the max fuel setting at penetration resistances equal to 7 bpi and with maximum compressive stresses during driving equal to 43 ksi respectively.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.
2. Maine Department of Transportation Standard Specifications, March 2020, with interim revisions through 9/7/22.
3. GRLWEAP 2014 v14.1.20.1 technical manual.

AVAILABLE INFORMATION

1. Boring log BB-PI95-103. We are assuming stratigraphy at both abutments are similar, from a pile driving standpoint.
2. Soil profile below is based on boring BB-PI95-103 stratigraphy below the bottom of Abutment No. 2 (El. 231.5).
3. Design Soil Properties calculation package.



ASSUMPTIONS

1. Elevations are in feet (ft) and reference the North American Vertical Datum of 1988 (NAVD88).
2. Soil and bedrock elevations based on referenced boring log.
3. Piles will be driven approximately 1 ft into bedrock based on conditions encountered in the referenced boring.
4. Steel H-pile section HP14x102 will be analyzed. Piles are assumed to be plumb (vertical).
5. Piles are approx. 77.4-ft long, which includes 1 ft of pen. into bedrock and 5 ft of stickup above bot. of Abut. No. 2.
6. Factored resist. is taken as the nominal resist. multiplied by 0.65 (resistance factor for CAPWAP dynamic testing).
7. Shaft and toe quake and damp. factors selected based on experience, soil cond., and default values per WEAP.
8. Penetration resistance is limited to a maximum of 15 bpi in accordance with the MaineDOT Std. Specifications.
9. Limit pile compressive stress during driving to 0.9Fy or 45 ksi for Fy = 50 ksi steel. It is noted that in our experience, WEAP tends to overpredict compressive driving stresses (i.e., actual compressive driving stresses are less than those predicted/calculated by WEAP.
10. Select pile hammer(s) based on those readily available to local pile driving contractors.

CALCULATIONS AND RESULTS

HP14x102 Steel H-pile (see Sheet No. 3 and 4):

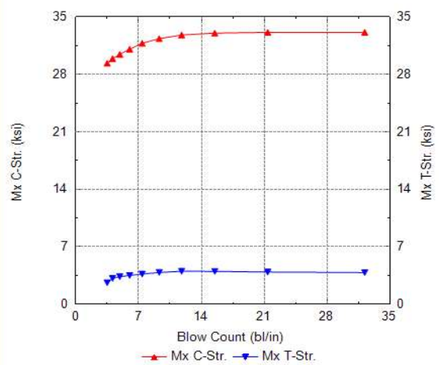
Pile Hammer Make and Model =	Delmag D 30-32	
Maximum Rated Energy =	75,438	ft-lbs.
Fuel Setting =	Maximum	
Ram Stroke =	9 to 10	ft
Nominal Resistance =	700	kips
Transferred Energy at Nominal Resistance =	38,980	ft-lbs.
Maximum Compressive Stress =	32	ksi
Penetration Resistance =	7	bpi
Factored Compressive Resistance =	455	kips (f_{dyn} = 0.65)

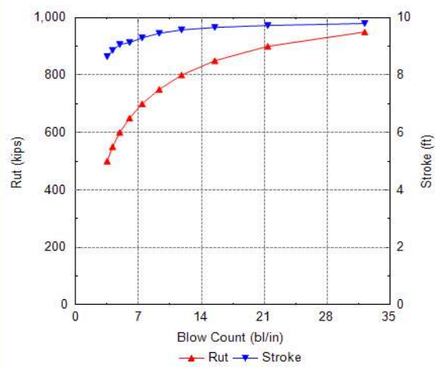
Nominal resistance is limited by the penetration resistance (see Sheet No. 3).

The HP14x102 steel H-pile can be driven to the top of bedrock w/out overstressing during driving (see Sheet No. 4).

HART CROWSER
Rte. 100 Bridge - Palmyra - Abutment No. 2 + HP14x102

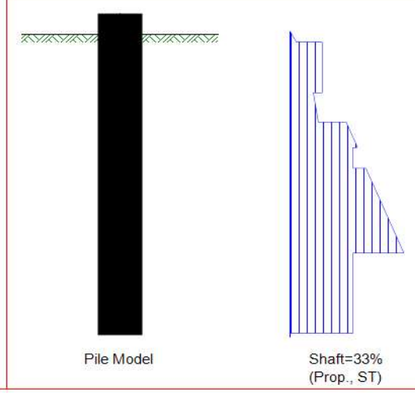
1/16/2026
GRLWEAP 14.1.20.1





DELMAG D 30-32

Ram Weight	6.60	kips
Efficiency	0.800	
Pressure	1460.0 (100%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.115	s/ft
Toe Damping	0.150	s/ft
Pile Length	77.40	ft
Pile Penetration	72.40	ft
Pile Top Area	30.00	in²
RSA	No	



HART CROWSER
Rte. 100 Bridge - Palmyra - Abutment No. 2 + HP14x102

1/16/2026
GRLWEAP 14.1.20.1

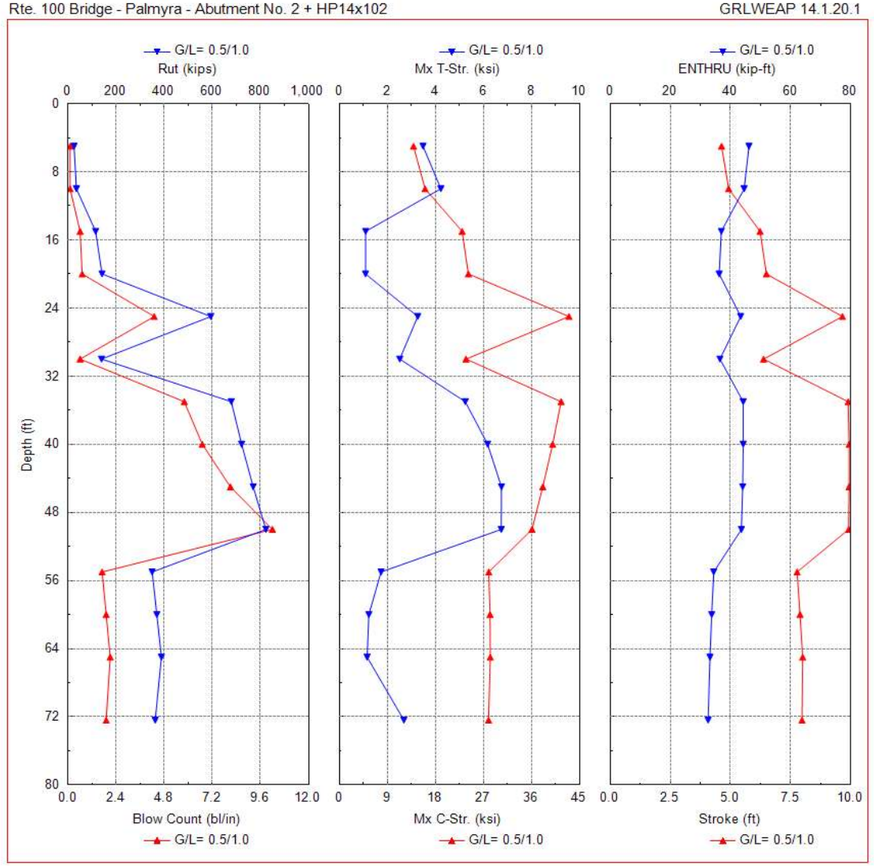
Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Hammer	Trans. Ratio
kips	ksi	ksi	ksi	bl/in	ft	kip-ft	DELMAG	%
500.0	29.35	28.55	2.60	3.4	8.64	36.75	D 30-32	48.7
550.0	29.91	29.06	3.13	4.1	8.86	37.56	D 30-32	49.8
600.0	30.44	29.52	3.34	4.9	9.06	38.23	D 30-32	50.7
650.0	31.07	29.69	3.49	6.0	9.13	38.46	D 30-32	51.0
700.0	31.79	30.06	3.66	7.4	9.30	38.98	D 30-32	51.7
750.0	32.35	30.41	3.85	9.3	9.46	39.53	D 30-32	52.4
800.0	32.80	30.68	4.00	11.8	9.57	40.02	D 30-32	53.1
850.0	33.03	30.88	3.99	15.5	9.66	40.24	D 30-32	53.3
900.0	33.12	31.04	3.91	21.4	9.73	40.43	D 30-32	53.6
950.0	33.11	31.20	3.84	32.2	9.80	40.63	D 30-32	53.9

Bearing Graph | Variable History | Extrema Result

Press F1 for General Help Topics or F3 for Specific Help on Current Parameters

HART CROWSER 1/16/2026

Rte. 100 Bridge - Palmyra - Abutment No. 2 + HP14x102 GRLWEAP 14.1.20.1



HART CROWSER 1/16/2026

Rte. 100 Bridge - Palmyra - Abutment No. 2 + HP14x102 GRLWEAP 14.1.20.1

Gain/Loss 1 at Shaft and Toe 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/in	Mx C-Str. ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	24.8	5.6	19.2	0.1	13.92	3.49	4.63	46.2	D 30-32
10.0	34.4	15.3	19.2	0.1	16.08	4.23	4.93	44.6	D 30-32
15.0	114.5	25.2	89.3	0.6	23.02	1.10	6.23	37.0	D 30-32
20.0	141.0	37.9	103.0	0.7	24.18	1.10	6.50	36.3	D 30-32
25.0	593.9	63.2	530.7	4.3	43.03	3.27	9.66	43.4	D 30-32
30.0	139.0	88.9	50.1	0.6	23.74	2.52	6.38	36.5	D 30-32
35.0	678.7	118.7	560.0	5.8	41.55	5.25	9.90	44.3	D 30-32
40.0	721.9	161.9	560.0	6.7	39.98	6.17	9.94	44.3	D 30-32
45.0	770.0	209.9	560.0	8.1	38.11	6.75	9.94	44.1	D 30-32
50.0	822.9	262.8	560.0	10.2	36.10	6.74	9.92	43.7	D 30-32
55.0	349.9	299.8	50.1	1.7	28.00	1.74	7.78	34.5	D 30-32
60.0	369.1	318.9	50.1	1.9	28.24	1.24	7.90	33.8	D 30-32
65.0	388.2	338.1	50.1	2.1	28.29	1.16	8.01	33.2	D 30-32
72.4	362.5	362.5	0.0	1.9	27.96	2.69	7.98	32.6	D 30-32

Total driving time: 68 minutes; Total Number of Blows: 2677 (starting at penetration 16.4 ft)

Driveability | Variable History | Extrema Result | Summary

Press F1 for General Help Topics or F3 for Specific Help on Current Parameters

Soil Profile and Properties for Lateral Pile Analysis

HALEY ALDRICH	CALCULATIONS	File No.:	0208880-000
		Sheet:	1 of 3
Client:	HNTB Corporation	Date:	1/15/2026
Project:	Route 100 Bridge No. 5957 over Interstate 95 - MaineDOT WIN 025315.00	Computed by:	TPJ
Subject:	Soil Profile and Properties for Lateral Pile Analysis	Checked by:	NAS/BCS

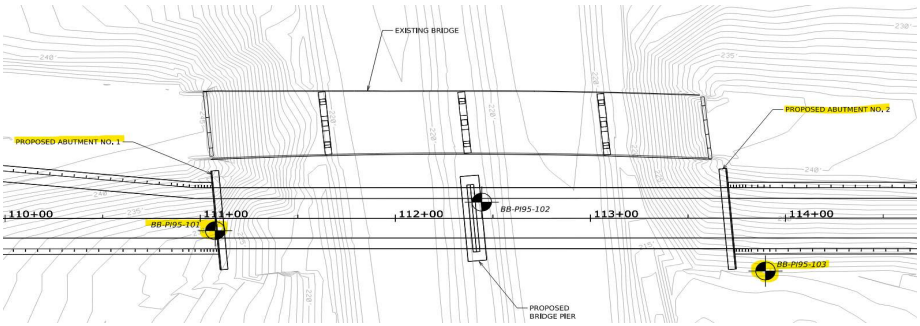
PROBLEM STATEMENT & OBJECTIVE

Determine design soil profiles and soil properties for HNTBs use in completeing lateral pile analyses at Abutments 1 and 2.

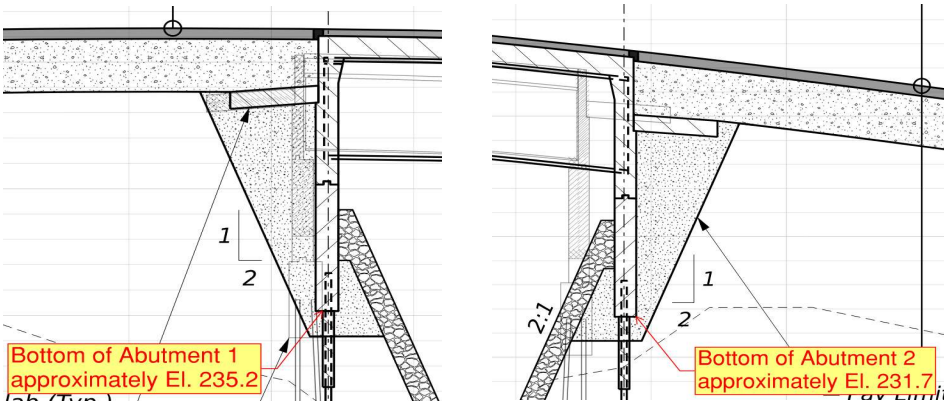
AVAILABLE INFORMATION, REFERENCES, AND ASSUMPTIONS

1. Boring logs BB-PI95-101 (Abut. 1) and BB-PI95-103 (Abut. 2) used to determine soil props. See loc. in plan snip below.
2. Recommended soil modulus and strain factors are based on Ensoft, Inc. LPILE v2022 Technical Manual, March 2024.
3. Bottom of abutment elevations taken from final PIC plans dated 27 June 2025 prepared by HNTB.
4. Elevations reference NAVD 88 datum.

SNIP OF BORING LOCATION PLAN

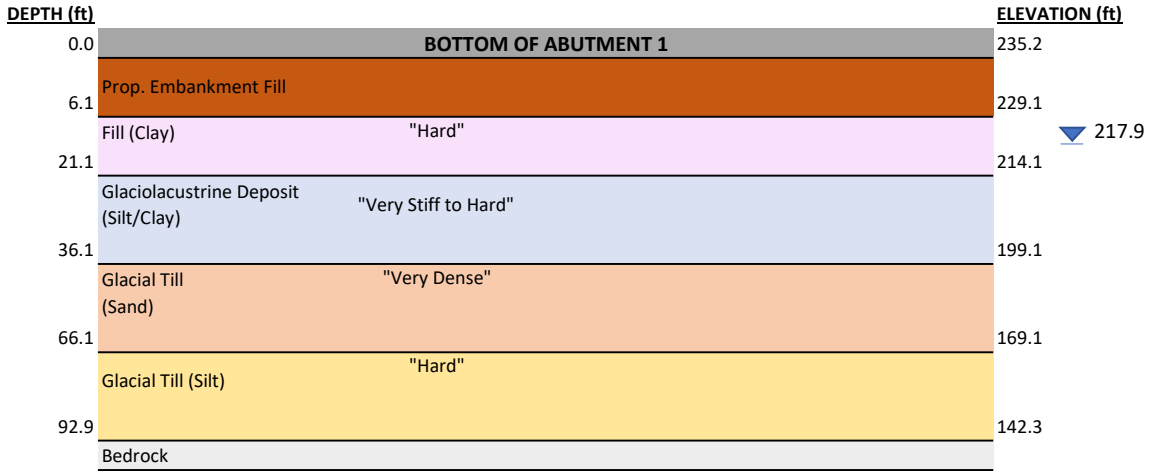


ABUTMENT PROFILES



SOIL PROFILE FOR ABUTMENT 1

Soil profile below shows approximate average thicknesses encountered below the bottom of Abutment 1 (El. 235.2).



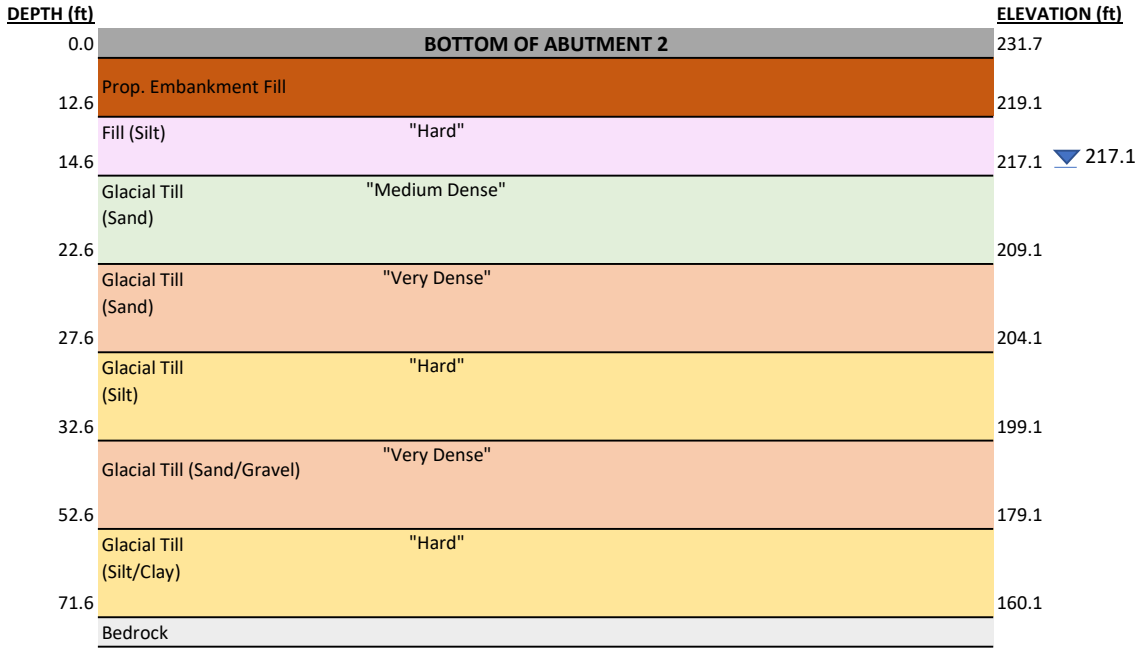
Summary of Soil Property Input Parameters (Abutment 1)

Stratum	Top of Stratum Elevation	Stratum Thickness (ft)	Moisture Condition	Unit Weight (pcf)		Friction Angle (ϕ, deg.)	Undrained Shear Strength (S_u, psf)	Soil Modulus (k, pci)	Strain Factor (e_{50}, -)
				Total	Buoyant				
Proposed Embankment Fill (Granular Borrow)	235.2	6.1	Dry	125	NA	32	NA	200	NA
Fill (clay; hard)	229.1	11.2	Dry	115	NA	NA	2,000	NA	0.007
Fill (clay; hard)	217.9	3.8	Wet	115	52.6	NA	2,000	NA	0.007
Glaciolacustrine Deposit (silt/clay; very stiff to hard)	214.1	15.0	Wet	125	62.6	NA	2,500	NA	0.005
Glacial Till (sand; very dense)	199.1	30.0	Wet	125	62.6	40	NA	125	NA
Glacial Till (silt; hard)	169.1	26.8	Wet	125	62.6	NA	10,000	NA	0.004

HALEY ALDRICH	CALCULATIONS	File No.:	0208880-000
		Sheet:	3 of 3
Client:	HNTB Corporation	Date:	1/15/2026
Project:	Route 100 Bridge No. 5957 over Interstate 95 - MaineDOT WIN 025315.00	Computed by:	TPJ
Subject:	Abutment 2 Soil Profile and Properties for Lateral Pile Analysis	Checked by:	NAS/BCS

SOIL PROFILE FOR ABUTMENT 2

Soil profile below shows approximate average thicknesses encountered below the bottom of Abutment 2 (El. 231.7).



Summary of Soil Property Input Parameters (Abutment 2)

Stratum	Top of Stratum Elevation	Stratum Thickness (ft)	Moisture Condition	Unit Weight (pcf)		Friction Angle (ϕ, deg.)	Undrained Shear Strength (S_u, psf)	Soil Modulus (k, pci)	Strain Factor (e_{50}, -)
				Total	Buoyant				
Proposed Embankment Fill (Granular Borrow)	231.7	12.6	Dry	125	NA	32	NA	200	NA
Fill (silt; stiff)	219.1	2.0	Dry	115	NA	NA	2,000	NA	0.007
Glacial Till (sand; medium dense)	217.1	8.0	Wet	125	62.6	30	NA	60	NA
Glacial Till (sand; very dense)	209.1	5.0	Wet	125	62.6	40	NA	125	NA
Glacial Till (silt; hard)	204.1	5.0	Wet	125	62.6	NA	2,500	NA	0.005
Glacial Till (sand/gravel; very dense)	199.1	20.0	Wet	125	62.6	40	NA	125	NA
Glacial Till (silt/clay; hard)	179.1	19.0	Wet	125	62.6	NA	10,000	NA	0.004

Pier Footing Bearing Resistance

PROBLEM STATEMENT & OBJECTIVE

Calculate the Strength and Extreme Event Limit State bearing resistances for a footing supporting the Pier that is bearing on undisturbed, naturally-deposited Glacial Till.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.

AVAILABLE INFORMATION

1. PIC plans provided by HNTB on 11 December 2025.
2. Boring log BB-PI95-102, which was drilled near Pier No. 1.
3. Laboratory test results (grain size, water content, Atterberg limits).
4. Design soil properties calculation (attached).
5. Proposed pier footing size received from HNTB on 4/2/2026.

ASSUMPTIONS

1. Bottom of footing is at El. 211.5 based on an assumed finish grade of El. 217 and a 5.5 ft maximum frost depth. Note that the adjacent existing pier footing is bearing at approximately El. 212.5
2. Subsurface conditions based on boring BB-PI95-102 (logs attached).
3. Footing will bear on undisturbed naturally-deposited Glacial Till.
4. Fully saturated soils exist beneath the footing based on borehole water level measurements made during drilling.
5. Footing size considered: 14.5 ft wide x 32.83 ft long as provided by HNTB.
6. The vertical load eccentricity is 0 since the footing is fully embedded in soil.
7. Both cohesive and cohesionless soil strength conditions in the Glacial Till were assessed for the Strength Limit State. A weighted average of the two values is recommended for the bearing resistance at the Strength Limit State.
   - Case 1 - Cohesive soil properties: 130 pcf (unit weight) and 2,500 psf (cohesion)
   - Case 2 - Cohesionless soil properties: 130 pcf (unit weight) and 36 degrees (friction angle)

   Soil properties (cohesion, friction angle) determination spreadsheet is attached.

Please note that based on the historic bridge drawings the "maximum foundation pressure" at the base of the existing Pier No. 2 footing is 2.3 tons/ft^2 (4,600 lb/ft^2), and the footing dimensions are 8.6 ft x 8.6 ft (square).

CONCLUSIONS AND RECOMMENDATIONS

The following bearing resistances were calculated for a 14.5 ft X 32.83 ft pier footing bearing at El. 211.5.

Strength Limit State

The recommended factored bearing resistance for the Strength Limit State is = 11 ksf

Extreme Event Limit State

The recommended factored bearing resistance for the Extreme Event Limit State is = 20 ksf

BACKGROUND INFORMATION FROM AASHTO LRFD

$$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a\text{-}1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \quad (10.6.3.1.2a\text{-}4)$$

where:

c = cohesion, taken as undrained shear strength (ksf)
N_c = cohesion term (undrained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)
N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)
N_γ = unit weight (footing width) term (drained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)
γ_q = total (moist) unit weight of soil above the bearing depth of the footing (kcf)
γ_f = total (moist) unit weight of soil below the bearing depth of the footing (kcf)
D_f = footing embedment depth (ft)
B = footing width (ft)
$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table, as specified in Table 10.6.3.1.2a-2 (dim)
s_c, s_γ, s_q = footing shape correction factors, as specified in Table 10.6.3.1.2a-3 (dim)
d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation determined from Eq. 10.6.3.1.2a-10 (dim)
i_c, i_γ, i_q = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

For $\phi_f = 0$:

$$i_c = 1 - (nH/cBLN_c) \quad (10.6.3.1.2a\text{-}5)$$

For $\phi_f > 0$:

$$i_c = i_q - [(1 - i_q)/(N_q - 1)] \quad (10.6.3.1.2a\text{-}6)$$

CASE 1 (cohesive)

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL\cot\phi_f)}\right]^n \quad (10.6.3.1.2a\text{-}7)$$

CASE 2 (cohesionless)

$$i_\gamma = \left[1 - \frac{H}{V + cBL\cot\phi_f)}\right]^{(n+1)} \quad (10.6.3.1.2a\text{-}8)$$

$$n = [(2 + L/B)/(1 + L/B)]\cos^2\theta + [(2 + B/L)/(1 + B/L)]\sin^2\theta \quad (10.6.3.1.2a\text{-}9)$$

where:

B = footing width (ft)
L = footing length (ft)
H = unfactored horizontal load (kips)
V = unfactored vertical load (kips)
θ = projected direction of load in the plane of the footing, measured from the side of length, L (degrees)

10.6.3.1.2b—Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2b\text{-}1)$$

$$\phi^* = \tan^{-1}(0.67\tan\phi_f) \quad (10.6.3.1.2b\text{-}2)$$

where:

c^* = reduced effective stress soil cohesion for punching shear (ksf)
ϕ^* = reduced effective stress soil friction angle for punching shear (degrees)

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic', 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

SEE BORING BB-PI95-102 AND FROST DEPTH CALC.

Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1 + \left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1 - 0.4\left(\frac{B}{L}\right)$	$1 + \left(\frac{B}{L}\tan\phi_f\right)$

$$d_q = 1 + 2\tan\phi_f(1 - \sin\phi_f)^2 \arctan\left(\frac{D_f}{B}\right) \quad (10.6.3.1.2a\text{-}10)$$

Eq. 10.6.3.1.2a-10 has been verified to cover a range of friction angle, ϕ_f, of 32 degrees to 42 degrees, and a range of D_f/B of 1 to 8. Depth correction factor values beyond this range have not been verified at this time.

$$H_{crit} = \frac{(3B)\ \ln\left(\frac{q_1}{q_2}\right)}{2\left(1 + \frac{B}{L}\right)} \quad (10.6.3.1.2d\text{-}1)$$

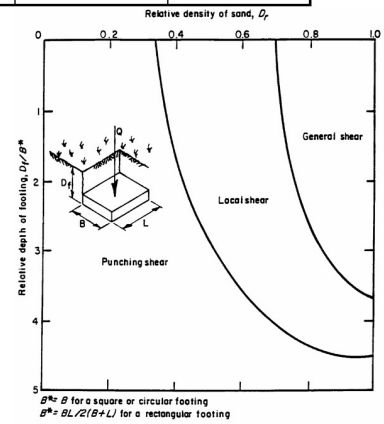


Figure C10.6.3.1.2b-1—Modes of Bearing Capacity Failure for Footings in Sand

HALEY ALDRICH	CALCULATIONS	File No.:	0208880-000
		Sheet:	3 of 3
Client:	Maine Department of Transportation	Date:	4/7/2026
Project:	Route 100 Bridge No. 5957 Over I-95 - MaineDOT WIN 25315.00	Computed by:	TPJ
Subject:	Pier No. 1 Bearing Resistance	Checked by:	EMH

CALCULATIONS

Input Parameters	Layer 1 Cohesive (B = 30 ft)	Layer 2 Cohesionless (B = 30 ft)
f (deg.) =	0	36
g_t (pcf) =	130	130
c (psf) =	2,500	0
D_f (ft) =	5.5	5.5
D_w (ft) =	5.8	5.8
B (ft) =	14.5	14.5
e_B (ft) =	0.00	0.00
L (ft) =	32.83	32.83
e_L (ft) =	0.0	0.0
RF =	0.45	0.45
Depth Corr., (Y/N)?	Y	Y
Upper Layer Thickness =	4.0	
Calculations & Output		
B_{eff} = B' (ft) =	14.5	14.5
L_{eff} = L' (ft) =	32.8	32.8
N_f = f(f) =	1.0	3.9
N_c = f_1(f) =	5.1	50.7
N_q = f_2(f) =	1.0	37.8
Ng = f_3(f) =	0.0	56.4
s_c =	1.09	1.33
s_q =	1.00	1.32
s_g =	1.00	0.82
d_q =	1.00	1.00
C_{wq} =	1.00	1.00
C_{wg} =	0.51	0.51
N_{cm} =	5.59	67.40
N_{qm} =	1.00	49.93
N_{gm} =	0.00	46.44
q_n (psf) =	14,700	57,885
q_n (ksf) =	14.7	57.9
H_{crit} (ft) =	20.7	
2B (ft) =	29.0	
% of Weighted Average Attributed to Upper Layer	75%	
Weighted Average q_n (ksf) =	25.5	
Weighted Average Strength Bearing Resistance (ksf) =	**11.5**	
Weighted Average Ext. Event Bear. Res. (ksf) =	**20.4**	

Notes:

1. Refer to Sheet 2 for definition of input parameters.
2. Analysis does not consider inclined load and inclined load adjustment factors, nor does it adjust for footings near slopes.
3. RF = resistance factor.
4. e_B and e_L are the vertical load eccentricities in the B and L directions, respectively. Check code guidance for maximum vertical load eccentricities allowed.
5. B_{eff} and L_{eff} are the effective footing dimensions considering vertical load eccentricity and are equal to B-2e_B and L-2e_L, respectively.
6. Footing settlement at the Service Limit State calculated separately.

BEARING RESISTANCE AT THE EXTREME EVENT LIMIT STATE

1. 11.5.8 - Resistance Factors for Extreme Event Limit State.
2. Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state. For overall stability of the retaining wall when earthquake loading is included, a resistance factor, ϕ, of 0.9 shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSE Walls.
3. Use nominal resistance calculated for the Strength Limit State and apply a resistance factor of 0.8 from AASHTO LRFD 2024 Section 11.5.8 to obtain the factored resistance at Extreme Event Limit State.

Pier Footing Settlement

PROBLEM STATEMENT & OBJECTIVE

Estimate elastic settlement of spread footing supporting the Pier.

REFERENCES

1. AASHTO LRFD Specifications, 10th Edition, 2024.

AVAILABLE INFORMATION

1. Draft PDR plans.
2. Boring log BB-PI95-102.
3. Laboratory test results.
4. Soil properties determination spreadsheet (attached).
5. Proposed pier footing size and service limit state bearing pressure received from HNTB on 4/2/2026.

ASSUMPTIONS

1. Glacial Till consists of "inorganic SILT" or "well graded fine to medium silty sand".
2. Settlement will be elastic only (i.e., no time-dependent consolidation settlement).
3. Pier footing will have a contact (service limit state) pressure of 6,180 psf based on email correspondence with HNTB.
4. The bottom of footing at Pier will be at El. 210 (per PDR plans).
5. Pier footing will be 14.5-ft wide (B) and 32.8-ft long (L) based on email correspondence with HNTB.
6. Subsurface conditions are based on those encountered in boring BB-PI95-102.
7. Groundwater is 5.8 ft BGS for boring BB-PI95-102 based on water level measurements made during drilling.

CALCULATIONS

1. See following pages.

CONCLUSIONS

Boring No.	Material Type	Elastic Settlement (in.)		Consolidation Settlement (in.)	Total Settlement (in.)
		Hough Method	Schmertmann Method		
BB-PI95-102	inorganic silt and f-m silty sand	1.4	1.3	0.0	1.4

Anticipate 1 to 1.5 in. of elastic settlement based on 6,180 psf (maximum) contact pressure along the base of the footing at the Service Limit State.

HALEY ALDRICH	CALCULATIONS	File No.	0208880-000
		Sheet	2 of 4
Client	HNTB Corporation	Date	4/7/2026
Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	Computed by	EMH
Subject	Settlement of Pier Footing	Checked by	TPJ

CALCULATIONS

AASHTO Empirical Method (per 10.6.2.4.2b - Hough Method)
for COHESIONLESS SOIL

$$S = \sum_{i=1}^{n} \Delta Hi \qquad \text{where} \qquad \Delta Hi = Hc \frac{1}{C'} \log \frac{\sigma_o' + \Delta\sigma v}{\sigma_o'}$$

n = number of soil layers within zone of stress influence of the footing. Divide subsurface profile into layers to a depth of about 3B below footing. The maximum layer spreadsheet should be about 10 ft.

ΔHi = elastic settlement of layer i (ft)

Hc = initial height of layer i (ft)

C' = bearing capacity index from Figure 10.6.2.4.2-1 (below);
note - use CORRECTED $N_{1,60}$

σo' = initial vertical effective stress at the midpoint of layer i (ksf)

Δσv = increase in vertical stress at the midpoint of layer I (ksf)

Boring No. =	BB-PI95-102	
Total Unit Weight of Soil, γ_t =	130	pcf
Unit Weight of Water, γ_w =	62.4	pcf
Footing Depth BGS, D_f =	5.5	ft
Groundwater Depth BGS, D_w =	5.8	ft
Footing Length, L =	32.8	ft
Footing Width, B =	14.5	ft
Contact Pressure, q =	6,180	psf

Calc. tot. stress at base of ftg. =	715	psf
Calc. water pressure at base of ftg. =	0	psf
Calc. eff. stress at base of ftg., Po =	715	psf
Calc. net fnd. pressure, Dp =	5,465	psf

Sublayer					$(N_1)_{60}$ (bpf)	C'	σ_o' (lb/ft^2)	$\Delta\sigma_v$* (lb/ft^2)	Δh_i (ft)
Top Depth (ft)	Bottom Depth (ft)	Midpoint Depth (ft)	Midpoint Depth BGS (ft)	Thickness (ft)					
0.0	4.5	2.3	7.8	4.5	58	95	886	5,007	0.039
4.5	9.5	7.0	12.5	5.0	67	155	1,207	3,435	0.019
9.5	14.5	12.0	17.5	5.0	53	125	1,545	2,476	0.017
14.5	19.5	17.0	22.5	5.0	76	180	1,883	1,874	0.008
19.5	24.5	22.0	27.5	5.0	56	130	2,221	1,470	0.008
24.5	29.5	27.0	32.5	5.0	73	170	2,559	1,185	0.005
29.5	34.5	32.0	37.5	5.0	72	170	2,897	976	0.004
34.5	39.5	37.0	42.5	5.0	72	120	3,235	818	0.004
39.5	44.5	42.0	47.5	5.0	59	100	3,573	696	0.004
44.5	49.5	47.0	52.5	5.0	60	100	3,911	599	0.003
49.5	54.5	52.0	57.5	5.0	55	90	4,249	522	0.003
54.5	59.5	57.0	62.5	5.0	52	85	4,587	458	0.002
59.5	63.3	61.4	66.9	3.8	50	85	4,884	411	0.002

*$\Delta\sigma_v$ is based on 2V:1H load distribution

Elastic Settlement (ft) =	0.1
Elastic Settlement (in) =	1.4
Consolidation Settlement (ft) =	0.0
Consolidation Settlement (in) =	0.0
Total Settlement (in) =	1.4

CALCULATIONS

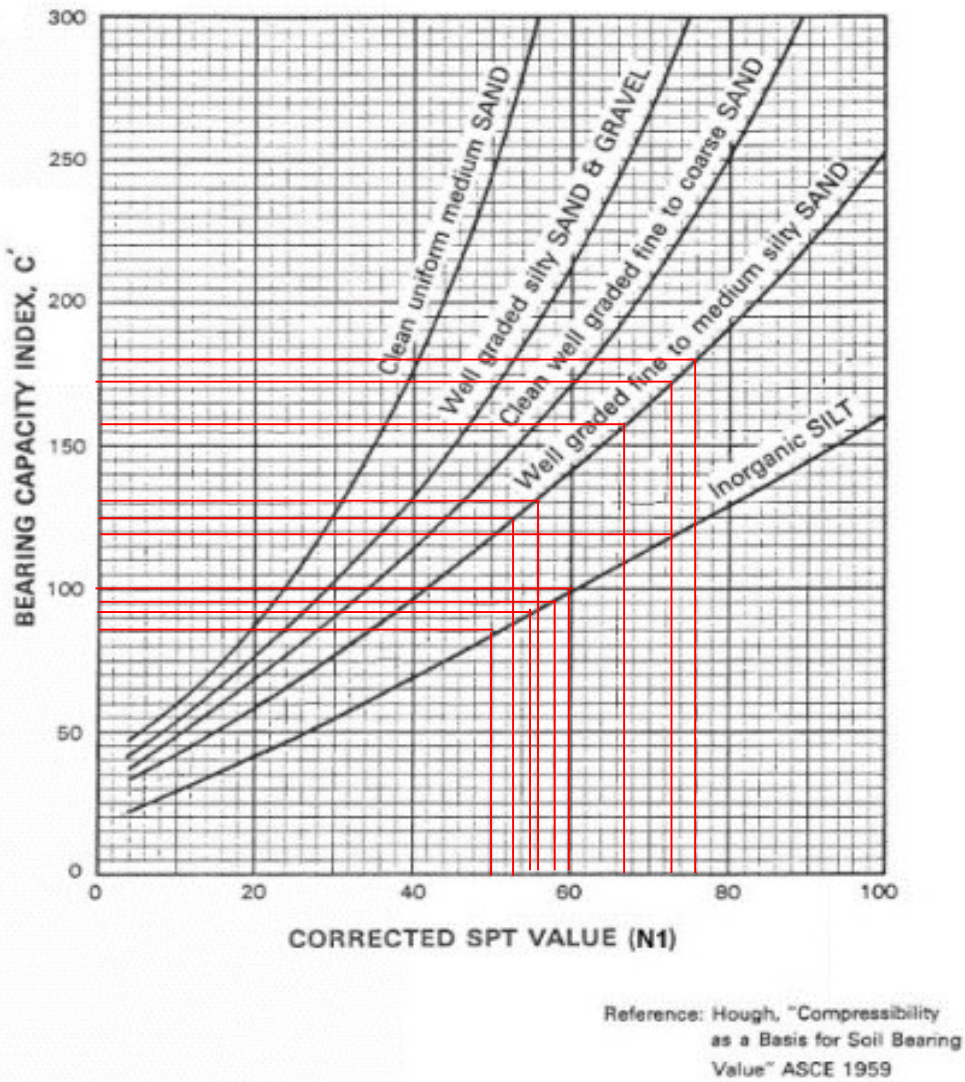


Figure 10.6.2.4.2b-1—Bearing Capacity Index versus Corrected *SPT* (Hough, 1959, as modified in Samtani and Nowatzki, 2006)

HALEY ALDRICH	CALCULATIONS	File No.	0208880-000
		Sheet	4 of 4
Client	HNTB Corporation	Date	4/7/2026
Project	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 25315.00	Computed by	EMH
Subject	Settlement of Pier Footing	Checked by	TPJ

CALCULATIONS

AASHTO Empirical Method (per 10.6.2.4.2c - Schmertmann Method)
for COHESIONLESS SOIL

$$S_i = C_1 C_2 \Delta p \sum_{i=1}^{n} \Delta J_i \quad \text{where} \quad \Delta Ji = Hc\left(\frac{I_z}{144XE}\right)$$

n = number of soil layers within zone of stain influence of the footing.

ΔJi = elastic spring stiffness of later i (ft/ksf)
Hc = initial height of layer i (ft)

Boring No. =	BB-PI95-102	
Total Unit Weight of Soil, γ_t =	130	pcf
Unit Weight of Water, γ_w =	62.4	pcf
Footing Depth BGS, D_f =	5.5	ft
Groundwater Depth BGS, D_w =	5.8	ft
Footing Length, L =	32.8	ft
Footing Width, B =	14.5	ft
Contact Pressure, q =	6,180	psf
L/B	2.3	

Calc. tot. stress at base of ftg. =	715	psf
Calc. water pressure at base of ftg. =	0	psf
Calc. eff. stress at base of ftg., Po =	715	psf
Calc. net fnd. pressure, Dp =	5,465	psf
Calc. eff. Stress at depth to peak strain influence factor, Pop (axisymmetric)	1,224	psf
Calc. eff. Stress at depth to peak strain influence factor, Pop (plane strain)	1,714	psf
C1	0.93	
C2 (75 years)	1.58	
X	1.32	

Sublayer					$(N_1)_{60}$ (bpf)	Soil Type assumed for correlation in Table 10.6.2.4.2c-1	Elastic Modulus (ksi)	z/B	Iz (axisymmetric)	Iz (plane strain)	Iz (weighted average)	ΔJ (ft/ksf)
Top Depth (ft)	Bottom Depth (ft)	Midpoint Depth (ft)	Midpoint Depth BGS (ft)	Thickness (ft)								
0.0	4.5	2.3	7.8	4.5	58	1	3.2	0.2	0.29	0.27	0.29	2.09E-03
4.5	9.5	7.0	12.5	5.0	67	2	6.5	0.5	0.69	0.43	0.63	2.56E-03
9.5	14.5	12.0	17.5	5.0	53	2	5.1	0.8	0.56	0.60	0.57	2.89E-03
14.5	19.5	17.0	22.5	5.0	76	2	7.4	1.2	0.39	0.64	0.45	1.60E-03
19.5	24.5	22.0	27.5	5.0	56	2	5.4	1.5	0.23	0.56	0.30	1.47E-03
24.5	29.5	27.0	32.5	5.0	73	2	7.1	1.9	0.07	0.48	0.16	5.95E-04
29.5	34.5	32.0	37.5	5.0	72	2	7.0	2.2	0.00	0.41	0.09	3.46E-04
34.5	39.5	37.0	42.5	5.0	72	1	4.0	2.6	0.00	0.33	0.07	4.84E-04
39.5	44.5	42.0	47.5	5.0	59	1	3.3	2.9	0.00	0.25	0.06	4.50E-04
44.5	49.5	47.0	52.5	5.0	60	1	3.4	3.2	0.00	0.17	0.04	3.04E-04
49.5	54.5	52.0	57.5	5.0	55	1	3.1	3.6	0.00	0.09	0.02	1.81E-04
54.5	59.5	57.0	62.5	5.0	52	1	2.9	3.9	0.00	0.02	0.00	3.19E-05
59.5	63.3	61.4	66.9	3.8	50	1	2.8	4.2	0.00	0.00	0.00	0.00E+00

Elastic Settlement (ft) =	0.1
Elastic Settlement (in) =	1.3
Consolidation Settlement (ft) =	0.0
Consolidation Settlement (in) =	0.0
Total Settlement (in) =	1.3

Pier Footing Sliding Resistance

HALEY ALDRICH	CALCULATIONS	File No.	0208880-000
		Sheet	1 of 1
Client	HNTB Corporation	Date	3/24/2026
Project	Route 100 Bridge No. 5957 Over I-95 - MaineDOT WIN 25315.00	Computed by	NAS
Subject	Pier Footing Sliding Resistance	Checked by	TPJ

PROBLEM STATEMENT AND OBJECTIVE

Determine the coefficient of friction between the footing and Glacial Till, resistance factor for sliding for the Strength Limit State, and resistance factor for sliding for the Extreme Event Limit State for the central pier footing on Glacial Till.

EXECUTIVE SUMMARY

	Glacial Till
The coefficient of friction between the footing and subgrade is =	**0.40**
The resistance factor for sliding at the Strength Limit State is =	**0.85**
The resistance factor for sliding at the Extreme Event Limit State is =	**1.0**

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th edition, 2024
2. MaineDOT Bridge Design Guide, 2003 with interim revisions through June 2018.

AVAILABLE INFORMATION

1. Boring log BB-PI95-102, drilled near the proposed pier.

ASSUMPTIONS

1. Abutment footing will bear on undisturbed Glacial Till at approximately El. 211. Hard sandy clay encountered at El. 211.

CALCULATIONS

Coefficient of Friction Between Concrete and Glacial Till

Nominal sliding resistance between the cast-in-place concrete footing and Glacial Till is dependent on the coefficient of friction (tan δ) at the interface between the footing and Glacial Till.

Estimated footing/Glacial Till interface friction angle (δ):

22 to 26 deg., friction angle for mass concrete on "very stiff and hard residual or preconsolidated clay" per AASHTO LRFD Table C3.11.5.3-1

Recommended δ =	**22**	deg., friction angle between footing/Glacial Till
Recommended tan δ =	**0.40**	dim., coefficient of friction

Resistance Factors

Strength Limit State

AASHTO LRFD does not prescribe a sliding resistance factor for shallow foundations on Glacial Till.
For cast-in-place concrete on clay, the sliding resistance factor is = **0.85** (Table 10.5.5.2.2-1)

Extreme Event Limit State

Section 10.5.5.3.3 of AASHTO LRFD prescribes a resistance factor of **1.0** for the design of foundations against sliding at the Extreme Event Limit State.

Frost Penetration

PROBLEM STATEMENT AND OBJECTIVE:

Determine maximum depth of frost penetration for Pier based on soil and groundwater conditions, as well as geographic site location. Maximum depth of frost penetration not evaluated for Abutment No. 1 and Abutment No. 2 because subsurface conditions within the existing approach embankments are unknown (i.e., borings were drilled at toe of approach embank. slopes). Maximum depth of frost penetration for Abutment No. 1 and Abutment No. 2 will be determined in final design.

REFERENCES:

1. MaineDOT Bridge Design Guide, August 2003, with Interim updates through June 2018.
2. Boring logs BB-PI95-101, BB-PI95-102, and BB-PI95-103.
3. Laboratory test results.
4. Bottom of abutment and pier footings/pile caps elevations from draft PDR.

EVALUATION:

1. Gather relevant information from soil samples collected from borings drilled near proposed substructure locations:

Substructure (GS Elev.)	Boring No.	Sample No.	Depth BGS (ft)	Sample Elev. (ft, NAVD88)	USCS Group Symbol	Moisture Content (%)
Abutment No. 1 (El. 229.1)	BB-PI95-101	1D	0 to 2	228.1	ML	--
		2D	2 to 4	226.1	CL	14.0
		3D	5 to 7	223.1	CL	14.0
		4D	10 to 11.2	218.5	CL	14.0
Pier (El. 217.6)	BB-PI95-102	1D	0 to 2	216.6	ML	--
		2D	2 to 4	214.6	ML	--
		3D	5 to 7	211.6	CL	10.0
		4D	10 to 11.2	206.6	SC-SM	9.0
		5D	15 to 17	201.6	SC-SM	9.0
Abutment No. 2 (El. 219.1)	BB-PI95-103	1D	0 to 2	218.1	ML	--
		2D	2 to 4	216.1	SC-SM	13.0
		3D	4 to 6	214.1	SC-SM	13.0

2. Per MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1800°F - days. Conservative; actual design freezing index varies from 1700 to 1800 °F-days.

3. Estimate maximum depth of frost penetration using MaineDOT Bridge Design Guide Table 5-1, the design freezing index above, and a "design" moisture content of the soil (i.e., assume average of available moisture content results).

Substructure	Soil Type	Design Moisture Content (%)	Frost Penetration Depth
Pier	Fine-grained	10	64 in (5.3 ft)

Recommend Pier footing be founded a minimum of 5.5 ft below the lowest adjacent ground surface exposed to freezing (i.e., at or below El. 212; approximate). Please note that based on the historic bridge drawings the existing Pier No. 2 bottom of footing is at approximately El. 212.5.

HALEY ALDRICH	CALCULATIONS	File No.:	0208880-000
		Sheet:	2 of 2
Client:	HNTB Corporation	Date:	11/5/2024
Project:	Route 100 Bridge No. 5957 over I-95 - MaineDOT WIN 025315.00	Computed by:	TPJ
Subject:	Frost Penetration Depth Evaluation Pier No. 1	Checked by:	BCS

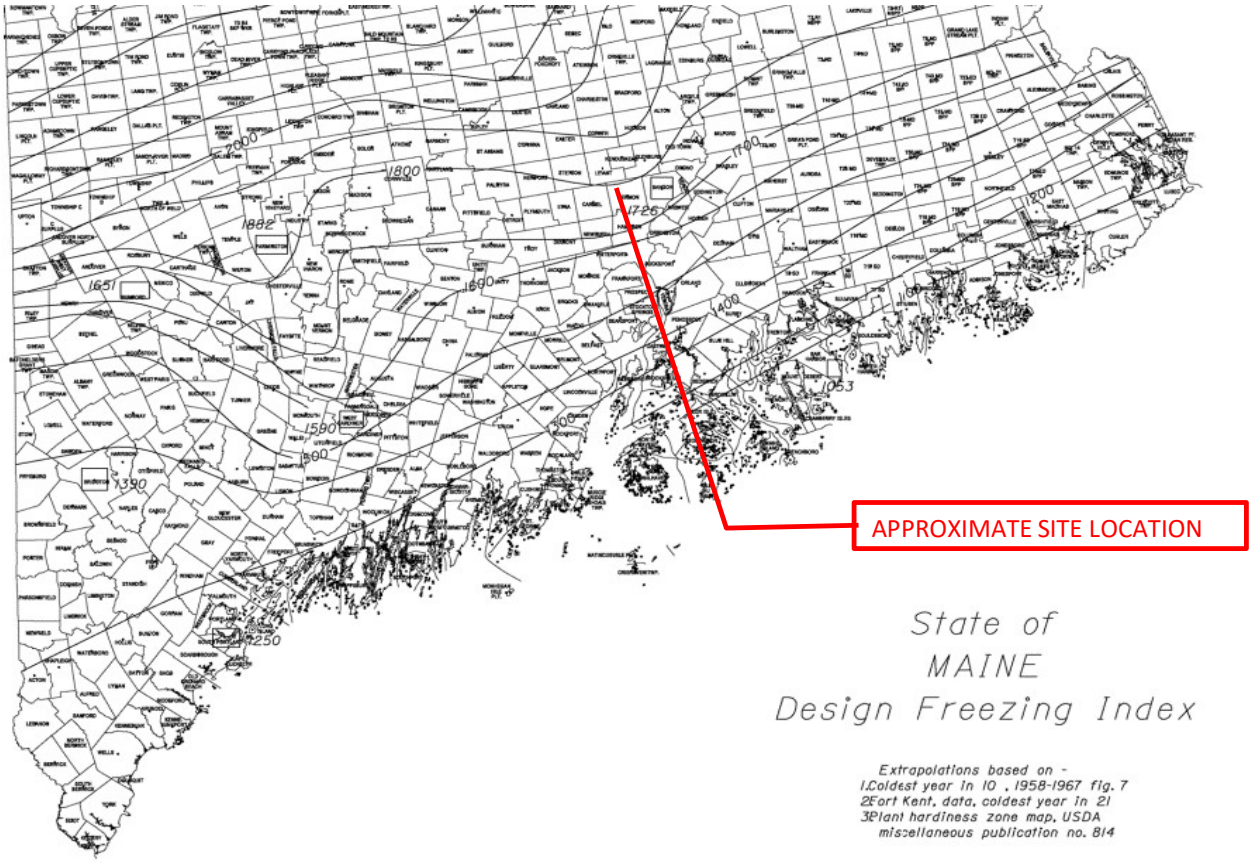


Table 5-1 Depth of Frost Penetration

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