GEOTECHNICAL DESIGN REPORT
ROUTE 103 NEW BRIDGE
MAINE DOT PIN 15110.00
YORK, MAINE

PREPARED FOR:
Maine Department of Transportation
Augusta, Maine

PREPARED BY:
GZA GeoEnvironmental, Inc.
Portland, Maine

March 2009
File No. 09.0025577.00

GZA
GeoEnvironmental, Inc.

Engineers and Scientists

March 30, 2009
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Maine DOT PIN 15110.00

Ms. Laura Krusinski, P.E.
Maine Department of Transportation
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Augusta, Maine 04333-0016

Re: Geotechnical Design Report
Route 103 New Bridge
York, Maine

Dear Laura:

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GZA GeoEnvironmental, Inc. (GZA) is pleased to provide you with this Geotechnical Design Report for the Route 103 New Bridge in York, Maine. Our work was completed in accordance with contract GCA No. U1210060627, GZA's Work Plan Dated June 10, 2008 (revised July 31, 2008 and Contract Modifications 1 and 2, dated August 26, 2008 and September 2, 2008, respectively), and the attached Limitations contained in Appendix A of the report. This report was prepared by Jennifer R. Tooley, P.E. under the supervision of Christopher L. Snow, P.E.

It has been a pleasure serving you on this project. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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1.0 INTRODUCTION

This report presents the results of GZA's subsurface exploration and geotechnical evaluation for replacement of the Route 103 New Bridge in York, Maine. Our services were provided in accordance with contract GCA No. U1210060627, GZA Work Plan Dated June 10, 2008 (revised July 31, 2008 and Contract Modifications 1 and 2, dated August 26, 2008 and September 2, 2008, respectively), and the attached Limitations contained in **Appendix A** of the report.

1.1 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface soil conditions and to provide geotechnical engineering recommendations for the proposed Route 103 New Bridge replacement. To meet these objectives, GZA completed the following Scope of Services:

- Conducted a site visit to observe surficial conditions; and reviewed existing bridge plans, test boring data, and mapped surficial and bedrock geology of the site;
- Coordinated and observed a subsurface exploration program consisting of seven test borings;
- Conducted a laboratory testing program to evaluate engineering properties of the site soils;
- Conducted geotechnical engineering analyses to evaluate foundations for the replacement bridge;
- Developed geotechnical engineering recommendations including foundation alternatives and foundation design recommendations for the preferred foundation type(s); and
- Prepared this report summarizing our findings and design recommendations.

1.2 BACKGROUND

New Bridge carries Route 103 over the York River in York, Maine, as shown in **Figure 1, Locus Plan**. The existing bridge consists of a 490-foot long, 26-foot wide, steel girder, concrete deck structure. The substructure consists of timber pile-supported abutments and timber pile bent piers at 26-foot spacing. Every fourth pier is a double bent supporting a joint in the superstructure.

The department intends to replace the bridge superstructure and foundations with a new 33-foot wide bridge consisting of seven 55- to 80-foot spans supported on two new integral abutments and six new piers. It is anticipated that the replacement bridge alignment will generally match the existing bridge alignment, with a widened deck and five-foot wide sidewalk. It is anticipated that the replacement bridge will be supported on integral abutments with driven HP-pile foundations and pile bent piers bearing on concrete filled steel pipe piles or square prestressed concrete piles. Route 103 will be shut down completely during demolition and bridge replacement.

2.0 SUBSURFACE EXPLORATIONS

2.1 PREVIOUS SUBSURFACE INVESTIGATION

Design drawings of the existing bridge prepared by the State Highway Commission, Bridge Division, dated January 1955, were provided for GZA's use. The drawings include logs of twelve test borings, completed in November 1955, drilled through the then-existing railroad embankments at each end, and at regular intervals along the riverbed in between. The borings were drilled through about 40 to 80 feet of soil, which from the ground surface down consisted of embankment fill, peat, alluvial (primarily sand) deposits, marine silt and clay, sand and gravel, and dense glacial till. Bedrock was confirmed at two locations beneath the riverbed at a depth of approximately 70 to 80 feet below ground surface.

2.2 RECENT SUBSURFACE INVESTIGATION

GZA completed a subsurface investigation program consisting of seven test borings. One boring was completed at each abutment and five were completed at or near the proposed pier locations. The locations of the borings were determined approximately in the field by taping from existing features shown on the bridge plans. The boring locations are shown on **Figure 2, Boring Location Plan.**

Approximate ground surface elevations at the abutment borings (BB-YYR-401 and BB-YYR-406) were interpolated by GZA from the existing bridge survey provided by VHB. To estimate the approximate ground surface elevations at the remainder of the borings, GZA interpolated the bridge deck elevation from the existing conditions survey provided by VHB, then subtracted the measured distance from the bridge deck to the mud line. Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988 (NAVD 1988). Boring locations and ground surface elevations at the borings are approximate and are considered accurate only to the degree implied by the methods used to determine them.

The borings were drilled to depths of 72 to 119 feet below ground surface and were terminated in bedrock. New Hampshire Boring, Inc. of Londonderry, New Hampshire coordinated utility clearance and provided drilling services. Their work was completed between August 11 and September 15, 2008. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**.

The borings were drilled using 4-inch casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the borings using a 24-inch sampler, a spooling-winch, and a safety hammer. The New Hampshire Boring standard penetration testing system used on this project was calibrated in October of 2008 and found to have an average energy transfer efficiency of 45 percent. A report on that calibration was provided under separate cover. All raw field N-values have been corrected to N_{60}, the standard energy of a rope and cathead system. Thin wall tube samples were collected in fine-grained soils. Field vane shear tests were performed with a tapered vane (1.8-in. x 4.6-in.) in four of the seven boring locations at varying intervals in fine-grained soils. Calculations of the vane constant are included in **Appendix E**. Two-inch diameter bedrock cores were obtained at each boring location. Core lengths of 7 to 15 feet were drilled to assess the nature of the bedrock.

3.0 LABORATORY TESTING

GZA completed a laboratory soil and bedrock testing program to confirm visual soil classification, evaluate frost classification and scour parameters, and estimate engineering properties of the soils and rock. The program included ten gradation analysis / AASHTO Classification / Frost Classification / Unified Soil Classification System (USCS) assessments, seven sets of Atterberg Limits, and three one-dimensional consolidation tests on soil samples taken from the borings. Three unconfined compression and modulus determinations were completed on selected bedrock samples. Results of the testing are included in **Appendix C**, and included on the boring logs in **Appendix B**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available literature, surficial geologic units mapped in the Route 103 New Bridge area include artificial fill, nearshore marine deposits, and Presumpscot Formation deposits. The following are brief descriptions of the geologic units:

- The fill material is described as a mixture of till, sand, gravel, and rock along with other miscellaneous man-made fill material.
- The nearshore marine deposits are described as thin, discontinuous till, water-deposited sediments, and/or wetland deposits overlying bedrock. These deposits are noted in coastal areas where glacial sediments were largely eroded and redeposited during glacial regression. Areas of bedrock outcrops are locally abundant.
- The Presumpscot Formation deposits are described as massive to laminated, gray to bluish-gray silt and clay, which weathers to brownish or greenish-gray. This deposit locally may include minor sand and gravel and occurs as a blanket deposit over bedrock and older glacial sediments. These sediments were deposited on the sea floor during late-glacial marine submergence.
- The southern abutment area is mapped as wetland and salt marsh deposits consisting of peat, muck, silt, and clay and is subject to tidal flooding.
- According to the Bedrock Geologic Map of Maine (1985) and the Bedrock Geology of the Kittery 1:100,000 Quadrangle, Maine, and New Hampshire (2008), bedrock in the vicinity of the site consists of phyllite and quartzite, with basalt dikes and sills, and is mapped as the Kittery Formation.

4.2 SUBSURFACE SOILS

Six primary soil units: Embankment Fill, Marine Nearshore Deposits (including Channel Sediments, Gravel Sediments, and Sand Sediments), Clay, and Glacial Till were encountered at the recently completed test borings. The encountered thicknesses and generalized description, including USCS classification, in descending order from ground surface, are presented in the following table. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs included in **Appendix B**. The soil units are also shown in relation to the bridge alignment on **Figure 3, Interpretative Subsurface Profile**.

GENERALIZED SUBSURFACE CONDITIONS		
Soil Unit	**Approx. Encountered Thickness (ft)**	**Description**
Embankment Fill	24	Loose to dense, brown, fine to coarse SAND, some to trace Gravel, some to trace Silt (USCS: GM, SM). Encountered in BB-YYR-401 and BB-YYR-406.
Marine Nearshore Deposits (Channel Sediments)	17 to 20	Medium dense to dense, brown fine to medium SAND, some to trace Silt, trace Gravel (USCS: SM). Encountered in BB-YYR-402 and BB-YYR-403.
Clay	9 to 29	Soft to very stiff, brown to dark gray, lean CLAY with little to some fine to medium SAND (variable amounts; typically bedding or layers), occasional Silt laminations, trace Gravel. Trace shell fragments, roots, and wood pieces. H_2S odor noted in upper samples (USCS: CL, AASHTO: A-4, A-6, A-7, A-7-6) Encountered in all of the borings. Water Content Range (8 samples): 23% to 41% Plastic Limit Range (7 samples): 9 to 25 Liquid Limit Range (7 samples): 25 to 46 Plasticity Index (7 samples): 16 to 22 Liquidity Index (7 samples): 0.29 to 0.94 (plastic behavior range) Field Vane Shear Strength Range: 2090 psf to greater than 6300 psf (initial), 315 psf to 2215 psf (remolded) Sensitivity Range (5 samples): 1.4 to 8 (low to medium sensitivity) Torvane Shear Strength Range (3 samples): 1160 psf to 2600 psf
Marine Nearshore Deposits (Gravel)	30	Loose to Dense, brown GRAVEL, some to trace fine to coarse Sand, trace Silt. Probable old stream channel (USCS: GM-GP). Encountered in BB-YYR-404 only.
Marine Nearshore Deposits (Sand)	3 to 51	Medium dense to dense, gray, fine to medium SAND, some to trace Silt, little to trace Gravel (USCS: SM-SC). Encountered in all of the borings.
Glacial Till	38 to 56	Medium dense to very dense, gray and brown, fine to coarse SAND and GRAVEL, some to little Silt with Cobbles and Boulders (USCS: SM, GP-GM). Encountered in all of the borings.

4.3 GROUNDWATER

All of the test borings except BB-YYR-401 and BB-YYR-406 were drilled in the York River where groundwater fluctuates with the tidal level. Water was introduced into the abutment borings during the drilling operations. As a result, stabilized groundwater levels were not determined. Wet to saturated soil samples were encountered at a depth of approximately 10 feet in the abutment borings. Based on these data, groundwater levels at the abutments are anticipated to be on the order of 5 to 10 feet below existing grade. Groundwater levels fluctuate due to season, tides, precipitation, infiltration and construction activity in the area. Therefore, groundwater levels during and after construction are likely to vary from those encountered at the time of the test borings.

4.4 BEDROCK

Bedrock was cored in all of the test borings. Two rock types were encountered, phyllite and basalt.

The phyllite was described as hard, fresh to moderately weathered, fine-grained to aphanitic, and dark gray. Joints were very close to closely spaced, low angle to near-vertical, planar and smooth to rough and undulating, fresh and tight to partly open, with occasional quartz stringers. The Rock Quality Designation (RQD) ranged from 0 to 85 percent, with an average of 24 percent. Laboratory testing yielded an unconfined compressive strength of 37 ksi and a secant modulus of 7,600 ksi.

The basalt was described as hard, fresh, fine- to medium-grained, and gray. Joints were close to widely spaced, dipping at low to moderate angles, planar, rough, fresh and tight to partly open, with occasional quartz stringers. The RQD ranged from 0 to 61 percent, with an average of 37 percent. Laboratory testing yielded an unconfined compressive strength of 35 ksi and a secant modulus of 10,000 ksi.

5.0 ENGINEERING EVALUATIONS

5.1 SCOUR CONSIDERATIONS

Two soil units were encountered on the riverbed surface, Marine Nearshore Deposits (granular) and Clay. Laboratory testing was performed on selected samples to assist in scour evaluations. Gradation analyses were performed on the granular deposits and Atterberg Limits were performed on the cohesive soil. The testing results are summarized below.

MARINE NEARSHORE DEPOSITS (GRANULAR)		
	BB-YYR-402 (1D)	BB-YYR-403 (1D)
D_{50} (mm)	0.52	0.53
D_{95} (mm)	1.8	2.2
Clay		
	BB-YYR-405A (1D)	BB-YYR-405A (3D)
Liquid Limit (LL)	45	25
Plasticity Index (PI)	22	16

5.2 SEISMIC CONSIDERATIONS

Evaluation of the seismic site class was based on the N-Bar approach in accordance with AASHTO LRFD Bridge Design Specification, 4th Edition, 2007 with 2008 Interims (herein referred to as LRFD) Table C3.10.3.1-1. N-bar is defined as the average, corrected SPT value for the upper 100 feet of the soil profile. For the New Route 103 Bridge site in York, N-bar exceeds 15 blows per foot (bpf); therefore, the site should be assigned to Site Class D.

The United States Geological Survey software Seismic Design Parameters Version 2.10 was provided with LRFD and was used to develop parameters for bridge design. Based on the site address, the software provided the recommended AASHTO Response Spectrum for a 7 percent probability of exceedence in 75 years. These results are summarized as follows:

Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.155	As - Site Class D
0.2	0.298	SDs - Site Class D
1.0	0.106	SD1 - Site Class D

Per LRFD Article 3.10.6 the site is assigned to Seismic Zone 1 based on a calculated SD1 of 0.106. Seismic design requirements for multispan bridges in LRFD 4.7.4.4 indicate no seismic analysis is required for multispan bridges in Seismic Zone 1, however, minimum support length requirements per LRFD 4.7.4.4 and connection restraint requirements in LRFD 3.10.9.2 apply. Additional seismic analysis guidance is provided in the Maine DOT Bridge Design Guide (BDG)

5.3 RESISTANCE FACTORS

Resistance factors herein are based on LRFD Articles 6.5.4.2 and 10.5.5.2.3. The following table presents the resistance factors used for the Route 103 New Bridge.

RESISTANCE FACTORS				
Condition	Steel H-Pile	Steel Pipe Pile	Prestressed Concrete	AASHTO LRFD Reference
Structural Limit State [Axial] (Combined Axial & Bending)	[$\Phi_c = 0.50$][1] ($\Phi_c = 0.70$) ($\Phi_f = 1.0$)			6.5.4.2
		[$\Phi_c = 0.60$][1] ($\Phi_c = 0.80$) ($\Phi_f = 1.0$)		6.5.4.2
			TBD [2]	5.5.4.2.1
Horizontal Resistance of a Pile or Pile Group	1.0	1.0	1.0	Table 10.5.5.2.3-1
Nominal Resistance of Single Pile in Axial Compression – Static Analysis Method	0.45	0.45	0.45	Table 10.5.5.2.3-1 Norlund/Thurman Method
Nominal Resistance of Single Pile in Axial Compression – Dynamic Analysis and Static Load Test Methods	0.65	0.65	0.65	Table 10.5.5.2.3-1 Dynamic Test with Signal Matching
Pile Drivability Analysis	1.0	1.0	1.0	Table 10.5.5.2.3-1

1. Pile tips are recommended due to glacial till soils, therefore, severe driving conditions and associated resistance factors are used.
2. Structural designer, VHB is responsible for evaluating combined stresses in precast concrete piles.

The factor for Nominal Resistance of Single Pile in Axial Compression, Φ_{dyn}, will be used to establish the required nominal geotechnical resistance to which the foundation piles will be driven. This factor is specified in AASHTO Table 10.5.5.2.3 as 0.65 for the case where installation controls will include wave equation (WEAP) analysis and a dynamic method including dynamic measurements combined with signal-matching analysis.

5.4 SUMMARY OF CALCULATED PILE RESISTANCES

We understand that the proposed bridge foundations will consist of integral abutments supported on ASTM A572, Grade 50, HP12x74 steel H-piles; and bent-type piers consisting of either closed ended, concrete-filled, 24-inch diameter, ¾-inch wall thickness, ASTM A252 Grade 3 steel pipe piles; or 24-inch, square, prestressed, concrete piles. Based on the resistance factors noted above and the analyses described in subsequent sections, GZA evaluated the resistances for the proposed piles. The results of those analyses are summarized in the table that follows.

SUMMARY OF CALCULATED PILE RESISTANCES (KIPS)			
Condition	ASTM A572 Grade 50 (fy = 50ksi) HP 12x74 Steel H-Pile	ASTM A252, Grade 3 Steel (fy = 45ksi) 24 x ¾-Inch, Closed-End, Concrete-Filled, Pipe Pile	24-Inch Square Prestressed Concrete f'c = 6ksi
Factored Axial Structural Resistance (No Bending) Hard-Driving	545	1469 (ignoring concrete)	1397
Factored Axial Resistance Based on Drivability	242	507	494
Nominal Geotechnical Axial Resistance	372	780	760

5.5 EVALUATION OF ABUTMENT FOUNDATIONS

5.5.1 Abutment Type

We understand that the abutments will consist of integral abutments, supported on steel HP 12x74 piles. The new abutments will be located approximately 15 feet behind the existing abutments. The existing pile-supported abutments will be demolished down to the level required by the replacement bridge design. The piles will be cut off, if necessary, and abandoned in place.

5.5.2 Frost Protection

Fill soils are anticipated to be present at the abutments, either as existing fill, or imported backfill. Based on the Maine DOT Bridge Design Guide (BDG), Section 5.2.1 the Freezing Index for the site is 1200, and with low-moisture content (<10%) soils, the estimated depth of frost penetration is 6 feet. However, since integral abutments are proposed, an embedment of at least 4.0 feet should be provided in accordance with Figure 5.2 of the Maine DOT BOG.

5.5.3 Abutment Soil Profile

GZA developed the following representative subsurface profile for use in evaluating both abutment foundations.

STRATA DESIGNATION	APPROX. BASE EL. (FT-NAVD 88)	APPROX THICKNESS (FT)	REPRESENTATIVE Φ (°) OR S_U (PSF) FOR LAYER	DESCRIPTION
Embankment Fill	-2	20	34°	Medium dense to dense, fine to coarse SAND, some Gravel, some to trace Silt.
Clay	-23	21	375 psf	Layered hard to soft, brown to gray, Lean CLAY and SILT, <u>and</u> loose to medium dense fine to medium SAND, trace Gravel. Trace shell fragments.
Marine Nearshore Deposits (Sand)	-37	14	34°	Medium dense to dense, gray, fine to medium SAND, some to trace Silt, little to trace Gravel.
Glacial Till	-80	43	42°	Dense to Very dense, gray and brown, fine to coarse SAND and GRAVEL, some to little Silt, with Cobbles and Boulders.
Bedrock	--	--	--	Hard, fresh to slightly weathered, fine-grained, gray, PHYLLITE.

5.5.4 Abutment Settlement

The bridge designer, VHB, proposes a 10-inch grade raise at Abutment 1. GZA evaluated the potential settlement anticipated due to the raise in grade. The settlement evaluation first considered the site-specific stress history of the lean clay deposits.

Maximum past pressures were evaluated based on the three one-dimensional consolidation tests. Those results are interpreted to show maximum past pressures greater than 8 ksf in both tests from the very stiff upper crust (BB-YYR-401, U1 and U2); and on the order of 3.6 ksf in the deeper stiff material (BB-YYR-406, U2). Evaluation of the maximum past pressures relative to the calculated in-situ effective stresses indicates that the crust layer is heavily over consolidated (by more than 7 ksf) and the deeper material is over consolidated by about 0.6 ksf.

The estimated stress increase due to an assumed 10-inch grade raise is about 0.1 ksf. Since the in-situ stress plus the stress increase is less than the maximum past pressure throughout the soil profile, consolidation settlement will occur as recompression.

Settlement of less than about ¼- inch is estimated to result from the 10-inch grade change. Due to the heavily over consolidated nature of the material and considering that settlement will occur as recompression, settlements are expected to occur rapidly as loads are applied. Due to the small magnitude of estimated settlement, settlement mitigation measures are not recommended. It is GZA's opinion that settlement of this magnitude is not sufficient to initiate downdrag, therefore downdrag loads should not be considered in design of the pile foundations.

5.5.5 Pile Loading Data

VHB provided factored structural design loadings for use in pile foundation design via emails on October 22 and 26, 2008. The loading data are provided in **Appendix D**, and summarized in the table below.

FACTORED ABUTMENT LOADS			
Load Combination	P_V* (kips)	P_H (kips)	Longitudinal Thermal Deflection (in.)
Service I	865	6	0.9
Strength I	1210	0	0.9
Strength III	800	13	0.9

*Load does not include pile self weight or downdrag. Includes entire superstructure load plus weight of abutment.

Preliminary design was completed assuming a 5-pile abutment. The required nominal axial geotechnical resistance was determined by distributing the Strength I factored load evenly over the 5 piles. The self-weight of the steel H-pile (on the order of 3 kips per pile) was ignored for preliminary analyses.

5.5.6 Axial Pile Resistance

Static pile resistances were calculated using the Norlund Method, based on standard penetration test results from the borings. The static geotechnical pile resistance evaluation provided an estimate of nominal geotechnical resistance, and friction distribution for use as inputs for wave equation analyses. The static analyses also indicated that the pile resistance would be derived primarily from side friction and end bearing in glacial till. Consequently, no reduction in axial resistance was applied for pile group interaction. As previously noted, potential settlement is not considered sufficient to initiate downdrag loads.

5.5.7 Lateral Pile Resistance

GZA completed a series of lateral pile capacity analyses to estimate probable HP-pile top deflections and bending stresses under strength limit state design loads. Analyses were completed in the transverse and longitudinal directions, assuming that the strong (X-X) axes are oriented to resist transverse displacement. Deflection estimates assumed that the piles were driven into existing subgrade soils, and assumed that the pile locations would be predrilled to allow for greater displacement of the bridge abutment. For preliminary analysis purposes, it was assumed that the predrilling would go from the base of the pile cap to the top of the clay and that after the driving was completed, the holes would be backfilled with a compressible material that would limit load transfer to the subgrade.

Fixity for a steel HP 12x74 pile was estimated for both fully embedded and predrilled pile lengths. For fully embedded pile the fixity was estimated to be 7.7 feet below bottom of pile cap when loaded in the strong (X-X) axis and 6.2 feet when loaded in the weak (Y-Y) axis. If the abutment pile locations were predrilled to the top of the clay stratum, fixity in the strong (X-X) axis was estimated to be 11.5 feet and in the weak (Y-Y) axis was estimated to be 8.7 feet below the clay surface. Fixity was determined according to LRFD methods outlined in Article 10.7.3.13.4.

GZA evaluated potential deflection and bending stress, using the Davisson and Robinson method referenced in LRFD Article C10.7.3.13.4, under the strength limit state loads, assuming the pile was fully fixed at both ends, that no soil reactions occurred in between, and that all piles were plumb. Results of the analyses are summarized in the table below.

STEEL H-PILE ESTIMATED DEFLECTIONS - ABUTMENTS						
Load Combination	Abut No.	Condition	Fixity Depth (ft)	Unsupported Length (ft)	Top Deflection (in.)	Bending Stress (ksi)
Strength III	2	Fully Embedded	7.7 (T)	7.7 (T)	0.01 (T)	-
			6.2 (L)	6.2 (L)	0.9 (L)	174
Strength III	2	Predrilled	11.5 (T)	26.5 (T)	0.4 (T)	-
			8.7 (L)	23.7 (L)	0.9 (L)	12

*T=Transverse Deflection - estimated by GZA using strength limit state lateral load. Does not include deflection due to thermal loads.
L=Longitudinal Deflection - estimated by VHB due to thermal loads.

The estimated transverse deflections are within the limits provided by Steve Hodgeton, P.E. of VHB, that is: less than ½-inch transverse.

VHB provided a maximum imposed longitudinal displacement due to thermal loads equal to 0.9 inches at the pile cap. GZA evaluated the bending stresses that would be induced by this magnitude of deflection at the pile cap. If the piles are to be driven directly into the embankment fill without predrilling, the estimated bending stress would be approximately 174 ksi. The estimated bending stress exceeds the yield stress of 50 ksi for Grade 50 steel, therefore other measures must be considered to reduce bending stress. GZA evaluated predrilling as a means to increase the depth to fixity and reduce bending stress. We estimate that predrilling through the embankment fill down to the top of the clay would result in an estimated bending stress of about 12 ksi. Based on these considerations, predrilling is recommended. Please refer to **Section 6.3** for additional details.

5.5.8 Preliminary Wave Equation Analysis

A preliminary wave equation analysis was performed to assess drivability. The analysis used the previously described design soil profile. The analyses were performed for a 5-pile configuration assuming a MKT DE 50B open-end diesel pile driving hammer with a manufacturer's rated energy of 42,500 foot pounds, driving a 65-foot long HP12x74 steel H-pile.

The factored side friction and end bearing resistances of the pile were estimated by GZA using the Norlund method and a resistance factor of $\Phi = 0.65$. For a 372 kip nominal capacity (resistance factor of 0.65 applied to maximum factored Strength I load of 242 kips), the side friction represented approximately 30 percent of the total capacity. Consequently, we used the 30 percent side friction value for wave equation analysis.

Results of the preliminary wave equation analysis indicate that the HP12x74 steel H-piles can be driven to a final penetration resistance of 10 blows per inch with a corresponding driving stress of approximately 27 kips per square inch (ksi), and an estimated stroke of about 8.0 feet. The anticipated driving stress is less than the driving stress limit of 45 ksi for ASTM A572, Grade 50 steel.

Based on the static analyses and wave equation analyses, we anticipate pile lengths in the range of about of 40 to 50 feet below the bottom of proposed pile cap elevation at each abutment. The pile length estimates assume approximately 5-feet of penetration into the glacial till. We anticipate that the piles will develop resistance through a combination of side friction and end bearing, with the tips bearing in dense glacial till. Cast steel pile points should be used to prevent damage to the pile tips during driving.

5.6 EVALUATION OF PIER FOUNDATIONS

5.6.1 Pier Foundation Type

We understand that the proposed pier substructures will consist of pile bents with cast in place concrete caps. Two alternative pile types, closed ended, concrete-filled, 24-inch diameter, ¾-inch wall thickness, steel pipe piles and 24-inch square prestressed concrete piles, were considered for this evaluation.

5.6.2 Design Profiles

GZA evaluated subsurface conditions and developed two representative design soil profiles for use in pier foundation evaluations. Pier profile A represents proposed southern piers 1 through 3, where clay overlies more competent granular soils. Pier profile B represents proposed northern piers 4 through 6, where a thin very stiff clay layer is sandwiched between granular materials. The profiles are summarized in the tables that follow.

PIER PROFILE A (PIERS 1 THROUGH 3)				
Strata Designation	Approx. Base El. (ft-NAVD 88)	Approx. Thickness (ft)	Representative Φ (°) or s_u (psf) for layer	Description
Clay	-30	20	250 psf	Soft, gray, Lean CLAY, some fine Sand (typically bedding or layers), trace Gravel. Trace shell fragments, roots, and wood pieces.
Marine Nearshore Deposits (Sand)	-80	51	33°	Medium dense to dense, gray, fine to medium SAND, little to trace Silt, little to trace Gravel.
Glacial Till	-130	48	40°	Medium dense to very dense, gray and brown, fine to medium SAND and GRAVEL, little Silt, with Cobbles and Boulders.
Bedrock	--	--	--	Hard, fresh to slightly weathered, fine-grained, gray, BASALT or PHYLLITE.

PIER PROFILE B (PIERS 4 THROUGH 6)				
Strata Designation	Approx. Base El. (ft-NAVD 88)	Approx Thickness (ft)	Representative Φ (°) or s_u (psf) for layer	Description
Marine Nearshore Deposits (Channel Sediments)	-40	20	33	Medium dense to dense, brown fine to medium SAND, some to trace Silt, trace Gravel.
Clay	-50	10	2000 psf	Medium stiff to very stiff, gray, Lean CLAY, some fine Sand (typically bedding or layers), trace Gravel.
Marine Nearshore Deposits (Sand)	-55	5	36°	Medium dense to dense, gray, fine to medium SAND, little to trace Gravel, little to trace Silt.
Glacial Till	-95	40	38°	Medium dense to very dense, gray and brown, fine to medium SAND and GRAVEL, little Silt with Cobbles and Boulders.
Bedrock	--	--	--	Hard, fresh to slightly weathered, fine-grained, gray, BASALT or PHYLLITE.

5.6.3 Pile Loading Data

VHB provided factored structural design loadings for use in pile foundation design via email on October 22 and 26, 2008. The loading data are provided in **Appendix D**, and summarized in the table below.

FACTORED PIER LOADS				
Load Combination	P_V* (kips)	P_{H1} (kips)	P_{H2} (kips)	Longitudinal Deflection due to thermal (in.)
Service I	1620	5	15	0.7
Strength I	2255	5	0	0.7
Strength III	1540	5	35	0.7
Extreme I	1745	5	0	0

*Load does not include pile or pile casing self weight. Load includes entire superstructure load plus weight of pile cap.
** Local pier and contraction scour not included per LRFD C3.4.1.

The loads provided for the pier locations did not include pile self weight. The factored self weight of each pile type can be estimated using the unit weights listed in the table below. The tabulated values include the Strength load factor, DC, equal to 1.25.

FACTORED PILE SELF WEIGHT		
Pile Type	Factored Unit Weight (lb/ft)	
	Above water	Below water
24-inch Diameter Concrete-Filled Steel Pipe Pile	750	506
24-inch square Prestressed Concrete Pile	750	438

The longest piles are anticipated at Pier 2. Assuming 5 feet penetration into glacial till and including the factored self weight, the total factored loads are 779 kips per pile for the concrete filled steel pipe pile and 760 kips per pile for the prestressed concrete pile for the controlling Strength I load case.

5.6.4 Axial Pile Geotechnical Resistance

Static pile bearing resistance was calculated using the Nordlund Method, based on standard penetration test results from the borings. The static pile resistance evaluation provided an estimate of nominal geotechnical capacity, and friction distribution for use as inputs for wave equation analyses. The static analyses also indicated that the pile resistance would be derived primarily from side friction and end bearing in glacial till. Consequently, no reduction in axial resistance was applied for pile group interaction. No filling is anticipated in the river, consequently downdrag loads are not considered for the pier foundations.

5.6.5 Lateral Pile Resistance – Concrete Filled Steel Pipe Pile

GZA completed a series of lateral pile resistance analyses to evaluate pile top deflections under design loadings. Analyses were completed in the transverse and longitudinal directions, assuming the outer piles of each bent are battered at a 2H:12V (toe out) batter, and the pile embedment begins at the present river bed level.

The controlling case for lateral loading was Strength III, equal to 60 kips per bent (transverse). The compressed batter piles are anticipated to provide most of the lateral reaction applied to each bent. GZA estimated the lateral component of the batter pile reaction to be approximately 55 kips for the estimated 308 kips axial load plus 27 kips self-weight of the pile (least anticipated). Assuming this distribution of loads, approximately 1 kip per pile will be required to react transverse lateral loads in bending.

Fixity for the concrete filled steel pipe pile was estimated to be 22 feet below mud line for design Profile A, where the surficial soils are clay. For Profile B, where the upper soils are granular, fixity was estimated to be 12 feet below mud line. Fixity was calculated according to LRFD methods outlined in Article 10.7.3.13.4. Fixity was also calculated for the tallest pier under scour conditions at Profile B, assuming 15 feet of granular material loss due to scour. Under scour conditions the depth to fixity was estimated to be 13 feet below the scoured mud line.

GZA's evaluation of fixity depth and lateral deflection used the assumptions that all the piles were plumb and that all lateral loads would be reacted in bending. The deflections calculated using this approach are upper bound values, greater than would be calculated by including the beneficial effect of the batter pile. Results of the analyses are summarized in the table that follows.

ESTIMATED DEFLECTIONS 24" DIAMETER, ¾" WALL, CONCRETE FILLED, STEEL PIPE PILE					
Load Combination	Pier Bent	Subsurface Soil	Fixity Depth (ft)	Unsupported Length (ft)	Top Deflection (in.)
Strength III	1	Clay	22	35[1]	0.5 (T)
					0.2 (L)
Strength III	3	Clay	22	52[2]	1.4 (T)
					0.4 (L)
Strength III	4	Sand	12	47	1.1 (T)
					0.4 (L)
Extreme I	4	Sand	13	62	1.1 (T)
					1.1 (L)

*T=Transverse Deflection; L=Longitudinal Deflection
** Does not include deflection or load due to thermal effects.
1. shortest pier – pier 6.
2. longest pier – pier 3.

VHB provided an estimated pile top deflection of 0.7 inches due to thermal loads. The stresses due to induced thermal deformations were not included in this preliminary evaluation, but should be considered by the bridge designer during final design of the piles.

The estimated transverse deflections are within the limits provided by Steve Hodgeton, P.E. of VHB, that is: less than 2-inches transverse at the longest pier, and less than 1-inch at the shortest pier. Deflections for the Extreme I case (scoured) were estimated to be on the order of 1-1/8 inch.

5.6.6 Preliminary Wave Equation Analysis – Concrete Filled 24-inch Diameter Steel Pipe Pile

GZA performed a preliminary wave equation analysis using the previously described design soil Profile A, and an assumed MKT 70 DE70/50B, open-end diesel pile driving hammer with a manufacturer's rated energy of 70,000 foot pounds. The analysis assumed a 101-foot long 24-inch diameter, ¾-inch wall thickness, steel pipe pile driven to bearing in glacial till. The assumed pile length was based on the distance between the bottom of the proposed pile cap and an assumed pile penetration of approximately 5 feet into glacial till.

The side friction and end bearing resistances of the pile were estimated by GZA using the Nordlund method. The ultimate required geotechnical resistance was calculated as approximately 779 kips for Profile A (Pier 2) (resistance factor of 0.65 applied to maximum factored Strength I load of 506 kips including factored self-weight of the pile). This load will control the required nominal capacity since the longest piles are anticipated there. The side friction at Pier 2 was estimated to be about 60 percent of the total capacity.

For the assumed driving system, the preliminary wave equation analyses indicate that the 24-inch diameter, ¾-inch wall thickness, steel pipe piles can be driven to a nominal resistance of 779 kips with a final penetration resistance of 10 blows per inch and a corresponding driving stress of approximately 27 ksi, and an estimated stroke of about 8.5 feet. The anticipated driving stress is less than the limiting driving stress of 40.5 ksi for ASTM 252, Grade 3 steel (45 ksi yield stress). Since the calculated final penetration resistance is in the range of about 5 to 14 and the calculated driving stress is well below the limit established by LRFD 10.7.8, the hammer system is judged to be appropriately sized for this project.

Geotechnical pile resistance estimates and wave equation analyses indicate that the piles can be installed to the required nominal capacity by penetrating at least 5 feet into the glacial till stratum. Due to variability in the density of the glacial till, penetrations may vary by 10 feet or more locally. Based on an assumed 5 to 10 feet of penetration into the glacial till bearing stratum, plumb pile lengths are anticipated to range from about 55 to 105 feet below the top of pile cap elevation. Batter piles will be slightly longer.

5.6.7 Lateral Pile Resistance – Prestressed Concrete Pile

GZA completed a series of lateral pile resistance analyses to evaluate pile top deflections under the design loadings. Analyses were completed in the transverse and longitudinal directions, assuming the outer piles of each bent are battered at a 2H:12V (toe out) batter, and the pile embedment begins at the present river bed level.

The controlling lateral load was the same as for the concrete filled pipe piles, 60 kips per bent (transverse). The estimated the lateral component of the batter pile reaction was assumed to be the same as previously noted, 55 kips. It is estimated that about 1 kip per pile will be required to react lateral loads in bending.

Fixity for the concrete pile was estimated to occur 21 feet below mud line for design Profile A, where the surficial soils are clay. For Profile B, where the upper soils are granular, fixity was estimated to be 12 feet below mud line. Fixity was estimated according to LRFD methods outlined in Article 10.7.3.13.4. Fixity was also calculated for the tallest pier under Extreme I scour conditions at Profile B, assuming 15 feet of granular material loss due to scour. Under scour conditions the depth to fixity was estimated to be 12 feet below scoured mud line. Since the depth to fixity was only one foot less than for the pipe piles, the preliminary evaluations used the same fixity depths as the pipe pile evaluation.

GZA's evaluation of fixity depth and lateral deflection assumed that all the piles were plumb and that all lateral loads would be reacted in bending. The bending stresses and deflections calculated using this approach are upper bound values, greater than would be calculated by including the beneficial effect of the batter pile. Results of the analyses are summarized in the table that follows.

ESTIMATED DEFLECTIONS 24-INCH SQUARE PRESTRESSED CONCRETE PILE					
Load Combination	Pier Bent	Subsurface Soil	Fixity Depth (ft)	Unsupported Length (ft)	Top Deflection (in.)
Strength III	1	Clay	22	35[1]	0.5 (T)
					0.2 (L)
Strength III	3	Clay	22	52[2]	1.6 (T)
					0.5 (L)
Strength III	4	Sand	12	47	1.2 (T)
					0.4 (L)
Extreme I	4	Sand	13	63	1.1 (T)
					1.1 (L)

*T=Transverse Deflection; L=Longitudinal Deflection
** Does not include deflection or load due to thermal effects.
1. shortest pier – pier 6.
2. longest pier – pier 3.

As with the concrete filled pipe piles, the stresses due to induced thermal deformations were not included in this preliminary evaluation but should be considered by the bridge designer during final design of the piles.

The estimated transverse deflections are within the limits provided by Steve Hodgeton, P.E. of VHB, that is: less than 2-inches transverse at the longest pier, and less than 1-inch at the shortest pier. Deflections for the Extreme I case (scoured) were estimated to be on the order of 1-1/8 inch.

5.6.8 Preliminary Wave Equation Analysis – Prestressed Concrete Pile

GZA performed a similar preliminary wave equation analysis to that previously described for the concrete filled steel pipe pile. The analysis assumed an MKT 70 DE70/50B open-end diesel pile driving hammer with a manufacturer's rated energy of 70,000 foot pounds and an 85-foot long 24-inch square prestressed concrete pile, driven to bearing in marine sand. The assumed pile length was based on the distance between the bottom of the proposed pile cap and an assumed pile penetration of approximately 55 feet into marine sand at Pier 2.

The side friction and end bearing resistances of the pile were estimated by GZA using the Nordlund method. The nominal required geotechnical resistance was calculated as approximately 760 kips for Profile A (Pier 2) (resistance factor of 0.65 applied to maximum factored Strength I load of 494 kips including the factored self weight of the pile). This load will control the required nominal resistance since the longest piles are anticipated there. The side friction at pier 2 was estimated to be about 75 percent of the total capacity.

For the assumed driving system, the preliminary wave equation analyses indicate that the 24-inch square prestressed concrete piles can be driven to a nominal resistance of 760 kips with a final penetration resistance of 12 blows per inch and corresponding driving stresses of approximately 1.7 ksi compression, and approximately 0.4 ksi tension and an estimated stroke of about 7.5 feet. The anticipated compression and tension driving stresses at end-of-driving are less than the limiting driving stresses of 4.4 ksi in compression and 1.2 ksi in tension for an assumed 6,000 psi 28-day strength concrete pile with 700 psi prestress.

Since the calculated final penetration resistance is in the range of about 5 to 14 and the calculated driving stresses are less than the maximum limits, the hammer system is judged to be appropriately sized for this project. However, tension stresses will be much higher while the piles are penetrating the clay stratum because there is no significant tip resistance. The contract documents should require that the hammer be fueled down or prevented from firing until the pile tip penetrates underlying granular soils, to avoid overstressing the pile in tension.

To limit driving damage, the pile tip detail should include a cast-in 1½-inch minimum thickness, steel end plate.

Static pile analyses and wave equation analyses indicate that the piles can be installed to the required nominal resistance by penetrating at least 5 feet into the glacial till stratum. Due to variability in the glacial till, penetrations may vary by 10 feet or more locally. Based on an assumed 5 to 10 feet of penetration into the glacial till bearing stratum, plumb pile lengths are anticipated to range from about 55 to 85 feet below the top of pile cap elevation. Batter piles will be slightly longer.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 GENERAL

GZA completed geotechnical engineering evaluations based on currently available subsurface exploration data, bridge construction plans, mapped surficial geology, and observation of visible conditions during a June 2008 site visit.

6.2 RECOMMENDATIONS FOR INTEGRAL ABUTMENTS

- The proposed replacement bridge abutments may be supported on HP12x74, ASTM A572 Grade 50 (50 ksi yield stress) steel H-piles driven to the required nominal resistance in glacial till.
- The piles should be driven to a nominal geotechnical resistance of 372 kips in order to achieve the maximum factored axial pile load of 242 kips per pile, with a resistance factor of 0.65 applied.
- Each abutment pile location should be predrilled from the bottom of pile cap level to the top of the lean clay stratum to redistribute and reduce bending stresses in the piles. The estimated depth of predrilling is on the order of 10 feet at abutment 1 and about 15 feet at abutment 2.
- The abutment piles should be driven in the predrilled holes, then the annular space backfilled with tire derived fuel (TDF) or other compressible material that will allow lateral movement of the piles. TDF consists of recycled tires shredded to a typical size of 1 ½” or less. Temporary casings should be used to keep the holes open during driving. Alternatively, permanent casings could be installed either with or without compressible backfill.
- We recommend that GZA perform further evaluation of TDF material focusing on laboratory gradation analysis and filter compatibility; and also perform more detailed analysis of the soil / structure interaction of the pre-drilled abutment piles under thermally-induced lateral displacements using a model such based on the COM624 approach.
- The proposed configuration with 5 piles per abutment and the piles oriented with the strong (X-X) axes resisting transverse bending is suitable for supporting the design lateral loadings, provided that predrilling is undertaken as noted.

- Abutment Pile lengths are anticipated to be in the range of 45 to 55 feet below the bottom of proposed pile cap.
- Splices should be made in accordance with Maine DOT Standard Specification Section 501.09 – Splicing Piles. No splices should be allowed within 15 feet of the pile top.
- A preliminary wave equation analysis was performed to evaluate pile drivability. The analysis used a MKT 50B open-end diesel pile-driving hammer with a manufacturer's rated energy of 42,500 foot pounds. The results show that the assumed driving system can install a 45- to 55-foot long, HP12x74, ASTM A572 Grade 50 steel H-pile to the required bearing stratum and required nominal resistance with a final penetration resistance of about 10 blows per inch, at a stroke of approximately 8.0 feet, and a maximum compressive driving stresses of about 27 ksi.
- The preliminary wave equation results indicate that the piles can be installed to the required nominal geotechnical resistance, without exceeding allowable driving stresses. The final penetration resistance is within the preferred range of 5 to 14 blows per inch, indicating that the assumed hammer has adequate rated energy to drive the piles.
- To limit driving damage, the steel H-piles should be fitted with protective driving tips in accordance with Maine DOT Standard Specification Section 501.10 – Pile Tips.
- Backfill for the integral abutments should consist of granular borrow for underwater backfill in accordance with Maine DOT Standard Specification Section 703.19.
- Foundation drainage should be provided in accordance with Sections 5.4.2.11 and 5.4.1.4 of the Maine DOT BDG. The use of French drains or prefabricated drainage board on the uphill side of abutments and wing walls, combined with a series of 4-inch diameter weep holes placed through the wall, and spaced approximately 10-feet center-to-center should provide adequate drainage for the proposed abutments.

6.3 RECOMMENDATIONS FOR BENT PIERS

- The proposed replacement bridge piers may consist of either concrete filled 24-inch diameter, ¾-inch wall thickness, ASTM A252 Grade 3 (45 ksi yield stress) steel pipe piles or 24-inch square prestressed concrete piles (assumed 6,000 psi 28-day strength concrete and 700 psi effective prestress) driven to the required bearing resistance in dense glacial till.
- The 24-inch diameter, ¾-inch wall thickness, steel pipe piles should be driven to a nominal resistance of 780 kips in order to achieve the maximum factored axial pile load of 694 kips plus an estimated factored self weight of 85 kips per pile, with a resistance factor of 0.65 applied.
- The 24-inch square prestressed concrete piles should be driven to a nominal resistance of 760 kips in order to achieve the maximum factored axial pile load of 694 kips plus an estimated factored self weight of 66 kips per pile, with a resistance factor of 0.65 applied.
- Construction documents should require that the piles be driven to at least the minimum required tip penetration elevations shown in the table below, and to the required penetration resistance, as determined by wave equation analysis, dynamic load testing, and signal matching analysis. For estimating purposes, it is anticipated that the piles will penetrate about 5 feet into the glacial till. However we note that the till material is variable and therefore the actual penetration will likely exceed 10 feet at some locations.

PIER	STA.	MINIMUM REQUIRED TIP PENETRATION ELEVATION (FT NAVD 88)	ESTIMATED TYPICAL TIP PENETRATION ELEVATION (FT NAVD 88)
1	12+17.50	-51	-56
2	12+97.50	-83	-88
3	13+77.50	-73	-78
4	14+57.50	-56	-61
5	15+37.50	-50	-55
6	16+17.50	-37	-42

- The proposed configuration with 5 piles per bent, and the outside piles battered (toe out) 2H:12V, is suitable for supporting the design lateral loadings. The lateral component from the compressed batter pile is anticipated to react most of the transverse lateral loading with the remaining capacity coming from bending of the individual piles. Based on the assumptions noted herein, anticipated lateral deflections are within allowable limits provided by the structural engineer, VHB.

- Preliminary wave equation analyses were performed to evaluate pile-drivability. An MKT 70 DE70/50B open-end diesel pile-driving hammer with a manufacturer's rated energy of 70,000 foot pounds was evaluated for driving the unfilled steel pipe pile and the prestressed concrete pile options. Analyses indicated that the assumed driving system could install a 55- to 101-foot long, 24-inch, ASTM A272 Grade III unfilled steel pipe pile or 24-inch prestressed concrete pile (6,000 psi concrete with 700 psi effective prestress) to the required tip penetration elevation with final penetration resistances in the range of 9 to 13 blows per inch. Calculated maximum compressive driving stresses were about 27 ksi for the concrete filled steel pipe pile. For the prestressed concrete pile, the calculated maximum driving stresses were 1.7 ksi in compression, and about 0.4 ksi in tension.

- The preliminary wave equation results indicate that for the assumed pile driving system, the piles can be installed to the required nominal pile resistance, without exceeding the driving stress limits for both the unfilled steel pipe pile and the prestressed concrete pile options. Since the calculated final penetration resistance was in the range of about 5 to 14 and the calculated driving stresses were within allowable limits, the hammer system is judged to be appropriately sized for this project.

- For the prestressed concrete pile alternative, the contract documents should require that the hammer be fueled-down or prevented from firing until the pile tip penetrates underlying granular soils, to avoid overstressing the piles in tension.

- To limit driving damage, the prestressed concrete pile tip detail should include a cast-in 1½-inch minimum thickness, steel end plate.

- The unfilled steel pipe piles should be fitted with 60-degree conical, cast steel points to protect them during driving.

- Based on the Maine DOT BDG, steel pipe piles require a fusion-bonded epoxy protective coating applied to a minimum of 10 feet below the streambed or 2 feet below the total scour depth. Cathodic protection should be used in addition to fusion-bonded epoxy coating due to the corrosive, salt water environment. Cathodic protection is not recommended at locations where the anodes will not remain submerged through the full tide cycle (Pier 1 and Pier 6). At these locations, internal reinforcing steel should be provided to at least ten feet below the point of fixity to provide supplemental bending capacity should the steel shell be reduced by corrosion during the design life.

7.0 CONSTRUCTION CONSIDERATIONS

Construction considerations are intended to provide a basis for design development and to identify significant issues that will impact project construction. These items are provided in the paragraphs that follow.

7.1 PILE INSTALLATION CONTROL

The contract documents should require that the pile installation be controlled by preconstruction wave equation analysis of the contractor's proposed pile driving system, dynamic testing with signal matching of one pile at each abutment and one pile at each of the river piers. The dynamic tests should be performed at the end of initial drive and during restrike at least 24 hours after end of initial driving to assess potential relaxation of the bearing soils.

7.2 TEMPORARY LATERAL SUPPORT

It is proposed to close down Route 103 and detour the traffic during bridge replacement. As a result, we anticipate that excavations can be achieved using sloped open cut techniques. It will be necessary to remove any portions of those existing foundations that would interfere with the proposed replacement abutment foundations. We note that existing timber piles, concrete abutments and boulders are present at the site that could present obstructions to pile driving.

7.3 DEWATERING

Groundwater levels are expected to be near or below the bottom of footing levels during construction of the abutments. We anticipate that seepage inflow and precipitation entering excavations can be handled by open pumping from sumps installed in the bottoms of excavations.

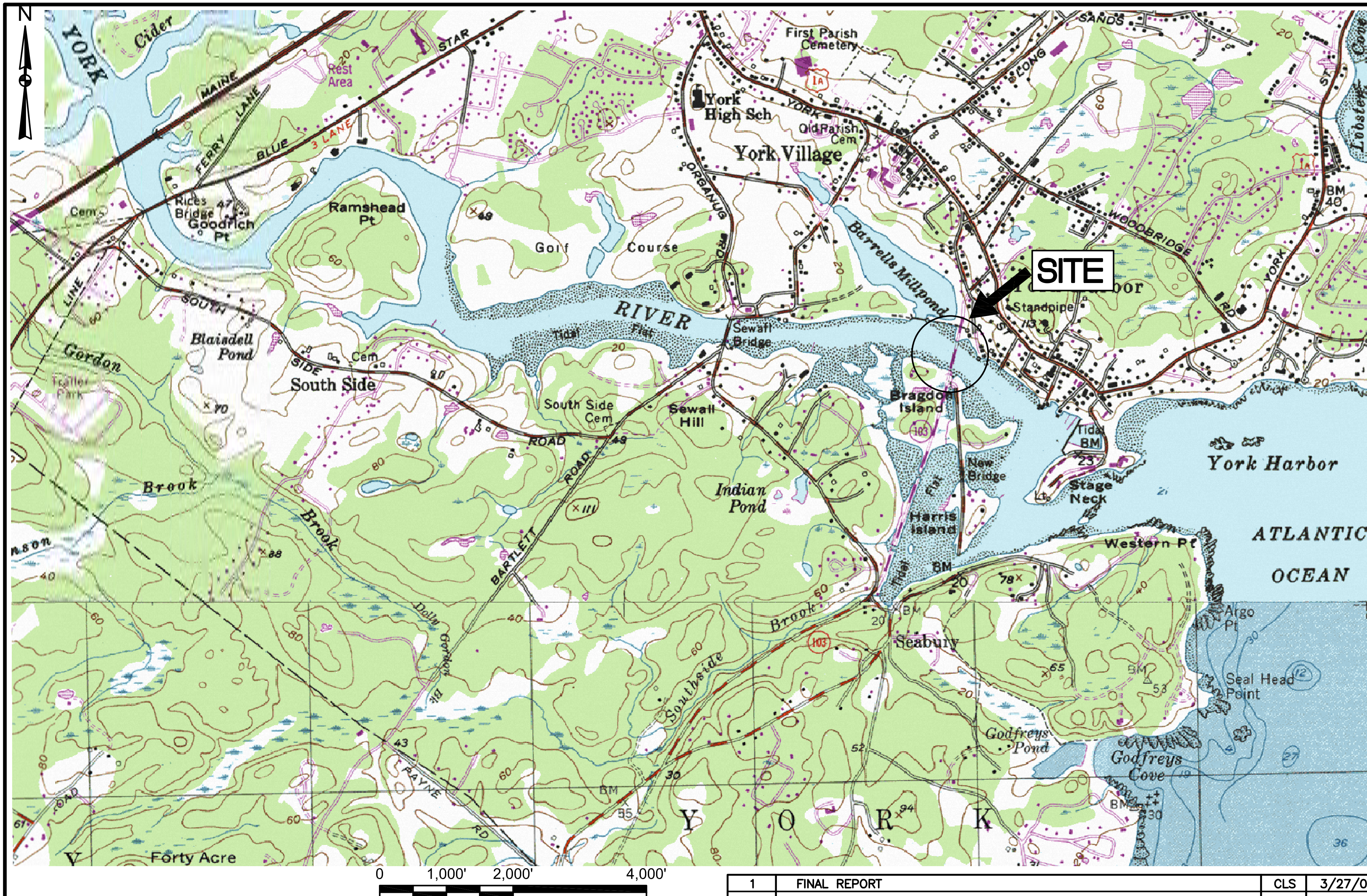
The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods that preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater should comply with all local, state, and federal regulations.

7.4 REUSE OF EXISTING EMBANKMENT FILL

Based on the results of gradation analyses performed on selected samples, the existing embankment fill does not meet the gradation requirements of Maine DOT Standard Specification Section 703.19 Granular Borrow for Underwater Backfill. Therefore, the existing embankment fill is not suitable for use as backfill of the abutments. It typically meets the gradation requirements of Maine DOT Standard Specification Section 703.19 Granular Borrow for Embankment Fill, and may be used as fill in embankment areas. Portions of the material with fines content exceeding 20 percent may be used in side slopes or other areas that will not be beneath proposed pavements.

FIGURES

©2008– GZA GeoEnvironmental, Inc. GZA–P:\25577.00 Rte 103 New Bridge\DWG\locus.dwg [LOCUS] March 27, 2009 – 10:59am Jennifer.Tooley



NO.	ISSUE/DESCRIPTION	BY	DATE
1	FINAL REPORT	CLS	3/27/09

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PREPARED FOR:
MAINE DEPARTMENT OF TRANSPORTATION

ROUTE 103 NEW BRIDGE

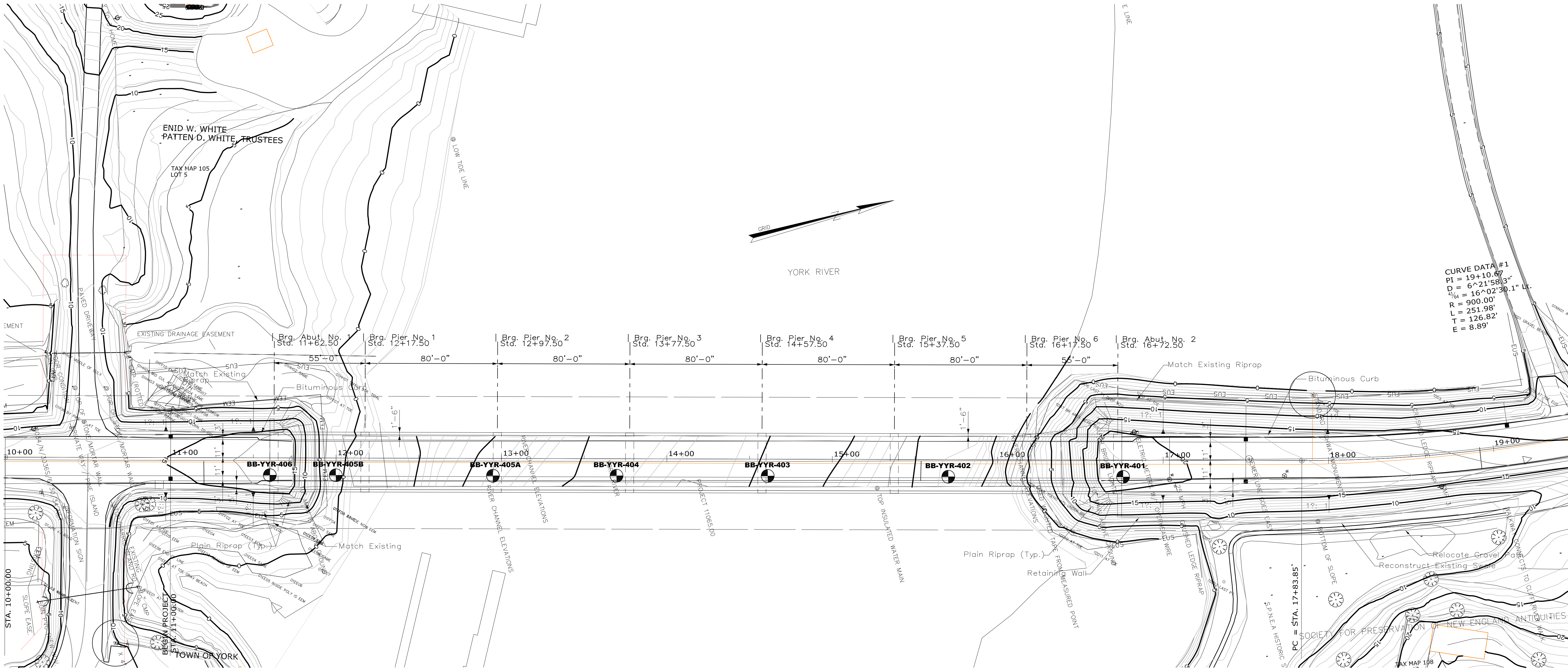
YORK, MAINE

LOCUS PLAN

PROJ MGR:	CLS
DESIGNED BY:	JRT
REVIEWED BY:	JVE
DRAWN BY:	JRT
CHECKED BY:	CLS
SCALE:	1" = 2000'

DATE: NOVEMBER 2008
PROJECT NO.: 09.0025577.00
REVISION NO.: 1

FIGURE 1
SHEET NO.



LEGEND

BB-YYR-401 — GZA BORING LOCATION

0 15 30 60 90
SCALE IN FEET

NOTES:

1) BASE MAP DEVELOPED FROM PLAN PROVIDED BY VHB, ENTITLED "BDPLAN," TRANSMITTED ELECTRONICALLY TO GZA AND RECEIVED ON NOVEMBER 24, 2008.

2) THE LOCATION OF THE TEST BORINGS WERE APPROXIMATELY DETERMINED BY TAPE MEASUREMENTS FROM EXISTING TOPOGRAPHIC FEATURES. THESE DATA SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

3) TEST BORINGS WERE PERFORMED BY NEW HAMPSHIRE BORING, INC OF LONDONDERRY, NEW HAMPSHIRE BETWEEN AUGUST 11, 2008 AND SEPTEMBER 15, 2008 AND OBSERVED BY GZA PERSONNEL.

NO.	ISSUE/DESCRIPTION	BY	DATE
1	FINAL REPORT	CLS	3/27/09

ROUTE 103 NEW BRIDGE

YORK, MAINE

BORING LOCATION PLAN

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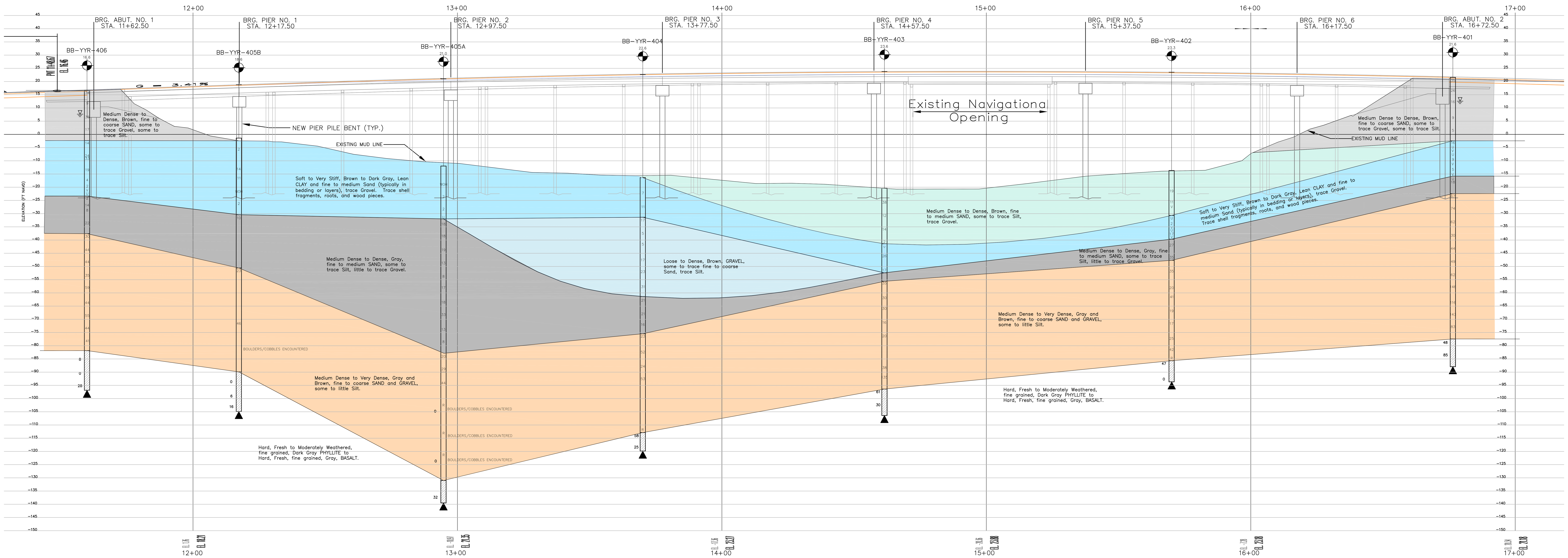
PREPARED FOR: MAINE DOT

PROJ MGR: CLS	REVIEWED BY: JVE	CHECKED BY: CLS	FIGURE
DESIGNED BY: JRT	DRAWN BY: JRT	SCALE: AS DEFINED	2
DATE: NOV 2008	PROJECT NO. 09.0025577.00	REVISION NO. 1	SHEET NO.



0 10 20 40 60
SCALE IN FEET

LEGEND:

- GZ-406 — BOREHOLE DESIGNATION
- 16.5 — GROUND SURFACE ELEVATION (FT NAVD 88)
- 30 — SPT N60 VALUE
- WOR — WEIGHT OF ROD
- WATER LEVEL BASED ON SAMPLE OBSERVATION
- T — 3-INCH DIAMETER TUBE SAMPLE
- V — VANE SHEAR TEST
- R — SPLIT SPOON REFUSAL (>50 BLOWS FOR 1" PENETRATION
- 30 — RQD OF CORE RUN
- ROCK CORE
- BOTTOM OF BOREHOLE

NOTES:

1) BASE MAP DEVELOPED FROM PLAN PROVIDED BY VHB, ENTITLED "15110.00 NEW BRIDGE PROFILE.DWG" RECIEVED SEPTEMBER 12, 2008 , ORIGINAL SCALE.

2) THE LOCATION OF THE TEST BORINGS WERE APPROXIMATELY DETERMINED BY TAPE MEASUREMENTS FROM EXISTING BRIDGE FEATURES. THESE DATA SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

3) THE GROUND SURFACE ELEVATIONS WERE APPROXIMATELY DETERMINED USING EXISTING BRIDGE DECK CENTERLINE ELEVATION AND OFFSET MEASUREMENTS FROM CENTERLINE. APPROXIMATE MUD LINE ELEVATIONS WERE DETERMINED AT BRIDGE DECK BORING LOCATIONS BY MEASUREMENTS OF DRILLING CASING FROM TOP OF BRIDGE DECK TO MUD LINE.

4) THE STATIFICATION LINES ARE BASED UPON INTERPOLATIONS BETWEEN WIDELY SPACED TEST BORINGS AND THUS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES. ACTUAL TRANSITIONS MAY VARY FROM THOSE SHOWN.

5) TEST BORINGS WERE PERFORMED BY NEW HAMPSHIRE TEST BORING, INC. OF LONDONDERRY, NEW HAMPSHIRE BETWEEN AUGUST 11, 2008 AND SEPTEMBER 15, 2008 AND OBSERVED BY GZA PERSONNEL.

NO.	ISSUE/DESCRIPTION	BY	DATE
1	FINAL REPORT	CLS	3/27/09

ROUTE 103 NEW BRIDGE

YORK, MAINE

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY:
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PREPARED FOR:
MAINE STATE
DEPARTMENT OF TRANSPORTATION

PROJ MGR: CLS	REVIEWED BY: RJM	CHECKED BY: JRT	FIGURE
DESIGNED BY: JRT	DRAWN BY: MJD	SCALE: 1" = 20'	3
DATE: OCT 2008	PROJECT NO. 09.0025577.00	REVISION NO. 1	SHEET NO.

APPENDIX A

LIMITATIONS

LIMITATIONS

Explorations

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

2. The generalized soil profile described in the text 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 are probably more erratic. For specific information, refer to the boring logs.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, and other factors occurring since the time measurements were made.

Review

4. In the event that any changes in the nature, design, or location of the proposed structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by GZA GeoEnvironmental, Inc. It is recommended that this firm be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

Construction

5. It is recommended that this firm be retained to provide soil engineering services during construction of the excavation and foundation phases of the work. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

Use of Report

6. This soil and foundation engineering report has been prepared for this project by GZA GeoEnvironmental, Inc. This report is for design purposes only and is not sufficient to prepare an accurate bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to design considerations only.

7. This report has been prepared for this project by GZA GeoEnvironmental, Inc. for the exclusive use of the Maine Department of Transportation and their project team for specific application to the Route 103 New Bridge Replacement in York, Maine in accordance with generally accepted soil and foundation engineering practices. No Warranty, express or implied, is made.

APPENDIX B – BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-401
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	21.6	**Auger ID/OD:**	na
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/20/08-08/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta 16+76, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							HP	21.1		Asphalt (approximately 6") 0.5	
	1D	24/10	1.0 - 3.0	20-23-23-37	46	35				Dense, Brown, fine to coarse SAND and Gravel, trace Silt, Dry.	A-1-b, GM WC=4%
	2D	24/10	4.0 - 6.0	11-21-13-13	34	26	36			Medium Dense, Brown, fine to coarse SAND and Gravel, trace Silt, Moist.	
5							27				
							27				
							27				
							30				
	3D	24/7	9.0 - 11.0	11-13-9-7	21	16	NR			Medium Dense, Brown, fine to coarse SAND, Some Gravel, Little Silt (Shell fragments)	A-1-b, SM WC=13%
10											
							13				
							14				
							13				
	4D	24/5	14.0 - 16.0	8-6-6-8	12	9	8			Loose, Brown, fine to medium SAND and Gravel, little Silt.	
15							16				
							21				
							21				
							15				
	5D	24/5	19.0 - 21.0	8-3-4-2	7	5	5			Loose, Brown, fine to coarse SAND and Silt, some gravel.	
20							10				
							16				
							26				
							11	-2.4		24.0	
25	6D	24/6	24.0 - 26.0	8-8-8-6	16	12	OH			Stiff, Dark Gray, CLAY and fine Sand, Shell Fragments.	

Remarks:

HP=Hydraulic Push
NR=Not Recorded
OH=Open Hole

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: New Bridge, Route 103 Location: York, Maine	Boring No.: BB-YYR-401 PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	21.6	**Auger ID/OD:**	na
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/20/08-08/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta 16+76, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							OH				
	U1	24/24	26.0 - 28.0							CLAY	A-6, CL WC=25% LL=38, PL=19 PI=19 Tv=1.3 tsf UW=124 pcf
	V1		28.5 - 29.0	Su=>6300 psf						1.8 x 4.6 in tapered vane raw torque readings: V1=>100 ft.- lbs.	
	U2	24/24	29.0 - 31.0							Fine SAND and CLAY.	A-6, CL WC=23% LL=32, PL=16 PI=16 Tv=1.15 tsf UW=127 pcf
30											
	7D	24/10	31.0 - 33.0	3-7-5-7	12	9				Loose, Brown, fine SAND, some Clay. 1.8 x 4.6 in tapered vane raw torque readings: V2= >100/65 ft.-lbs. (Initial/remolded)	
	V2		31.5 - 32.0	Su=>6300/4115psf							
	8D	24/19	34.0 - 36.0	3-2-4-5	6	5				Medium Stiff, Gray, CLAY, little fine Sand.	
35											
	V3		36.5 - 37.0	Su=5000/2215 psf						1.8 x 4.6 in tapered vane raw torque readings: V3= 79/35 ft.-lbs. (Initial/Remolded)	
	U3	24/4	37.0 - 39.0				17	-15.4		37.0 Brown, fine to medium SAND, some Clay and Silt.	
							14				
	9D	24/12	39.0 - 41.0	7-9-15-52	24	18	25			Medium Dense, Top 8": fine to medium SAND, Iron Oxide staining. Middle 2" : Gray, Silt and Clay layer Bottom 2": Silt and Clay laminations, Iron Oxide staining.	
40							76				
							80				
							83				
							OH				
	10D	24/14	44.0 - 46.0	29-44-45-52	89	67		-22.4		44.0 Very Dense, Brown, fine SAND, some silt.	
45											
	11D	24/16	49.0 - 51.0	36-50-48-49	98	74				Very Dense, Brown, fine SAND, some silt. Iron Oxide staining.	A-3, SM WC=19%
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	New Bridge, Route 103			**Boring No.:**	BB-YYR-401
Location:	York, Maine			**PIN:**	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	21.6	**Auger ID/OD:**	na
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/20/08-08/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta 16+76, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							OH				
	12D	24/14	54.0 - 56.0	19-38-45-54	83	62				Very Dense, Brown, fine SAND, some silt.	
55											
	13D	24/15	59.0 - 61.0	17-22-18-18	40	30	43			Dense, Top: Brown, fine SAND, some silt. Middle Top: Gray, fine SAND, some silt. Middle Bottom: Red/Brown, fine to coarse SAND, some silt (Iron Oxide staining). Bottom: Brown, fine SAND, some silt.	
60							42				
							58				
							56				
							64				
	14D	24/11	64.0 - 66.0	23-28-30-30	58	44	OH			Dense, Brown, fine to medium SAND, little Silt, little coarse Sand, trace Gravel. Iron Oxide staining.	A-1-b, SM WC=15%
65											
	15D	24/9	69.0 - 71.0	30-34-31-29	65	49				Dense, Brown fine to coarse SAND, little to some Silt, little Gravel.	
70											
	16D	24/5	74.0 - 76.0	23-49-33-43	82	62	40			Very Dense, Brown, fine to coarse SAND, little to some Silt, little Gravel.	
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-401
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	21.6	**Auger ID/OD:**	na
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/20/08-08/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta 16+76, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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Hammer Efficiency Factor = Annual Calibration Value
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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							71				
							72				
							46				
							68				
	17D	24/9	79.0 - 81.0	26-99-95-76	194	146	42			Very Dense, Top: Brown, fine to medium SAND. Bottom: Gray, Glacial Till.	
80							36				
							65				
							80				
							84				
	18D	24/11	84.0 - 86.0	52-80-75-68	155	116	OH			Very Dense, Brown, GRAVEL and fine to medium SAND, little Silt.	A-1-a, GP-GM WC=8%
85											
	19D	15/7	89.0 - 90.3	70-90-100/3"	190	143				Very Dense, Gray, fine to coarse SAND, some Gravel, little Silt.	
90											
	20D	24/15	94.0 - 96.0	76-58-52-80	110	83				Very Dense, Gray, fine to coarse SAND and SILT, little Gravel, little Silt, fine Sand lenses, Possible re-worked Glacial Till.	A-4, SM WC=9%
95											
	R1	60/54	99.5 - 104.5	RQD = 48%			NQ	-77.9		99.5	
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine
Boring No.: BB-YYR-401
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	21.6	**Auger ID/OD:**	na
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/20/08-08/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta 16+76, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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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							NQ			Bedrock: Hard, fresh to slightly weathered, fine grained, Dark Gray PHYLLITE. Joints are very close to close, high angle, planar, smooth, fresh and tight to partially open. Occasional Quartz veins. R1: Core Times (min) 99.5-100.5 (6) 100.5-101.5 (6) 101.5-102.5 (5) 102.5-103.5 (6) 103.5-104.5 (5)	
	R2	60/59	104.5 - 109.5	RQD = 85%						Hard, fresh to slightly weathered, fine grained, Dark Gray PHYLLITE. Joints are very close to close, low angle, planar, smooth, fresh and tight to partially open. Occasional Quartz veins. R2: Core Times (min) 104.5-105.5 (6) 105.5-106.5 (6) 106.5-107.5 (6) 107.5-108.5 (6) 108.5-109.5 (6)	
105											
								-87.9		109.5	
110										**Bottom of Exploration at 109.50 feet below ground surface.**	
115											
120											
125											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	New Bridge, Route 103	**Location:**	York, Maine	**Boring No.:**	BB-YYR-402
				PIN:	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	-13.7	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/28/08-09/02/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 15+70, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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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							HP			Mud line approximately 37 feet below top of bridge deck. See Note 1.	
5											
	1D	24/10	7.0 - 9.0	3-11-14-18	25	19	11			Medium Dense, Brown, fine to medium SAND, trace Silt.	A-1-b, SM WC=20%
							29				
10							38				
							30				
							21				
	2D	24/12	12.0 - 14.0	WOR-6-8-14	14	11	8			Medium Dense, Brown, fine to medium SAND, some Silt (laminations, Gray Silt lenses)	
	V1		13.0 - 13.5	Su=5570/1265 psf			20			1.8 x 4.6 in tapered vane raw torque reading: V1= 88/20 ft.-lbs. (Initial/Remolded)	
							17				
15							16				
							9	-30.7		17.0	
	U1	24/24	17.0 - 19.0				HP				
	3D	24/12	19.0 - 21.0	1-1-1-1	2	2				Stiff, Top 3": Brown, SILT and CLAY, Bottom 9": Gray, CLAY and SILT	
20	V2		20.0 - 20.5	Su=2090/760 psf						1.8 x 4.6 in tapered vane raw torque reading: V2=33/12 ft.-lbs. (Initial/Remolded)	
	U2	24/19	22.0 - 24.0								
25	4D	24/24	24.0 - 26.0	4-9-14-18	23	17				Medium dense, Top 8": Dark Gray, fine to medium SAND, trace Clay. Middle 6": Light Brown, CLAY, some fine to medium Sand. Bottom	

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 7 ft below mudline.
HP=Hydraulic Push
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine
Boring No.: BB-YYR-402
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-13.7	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/28/08-09/02/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 15+70, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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T_v = Pocket Torvane Shear Strength (psf)
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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PL = Plastic 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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	V3		25.0 - 25.5	Su=2785/1960 psf			HP			10": Gray, fine SAND and CLAY 1.8 x 4.6 in tapered vane raw torque reading: V3= 44/31 ft.-lbs. (Initial/Remolded)	
								-40.7		27.0	
	5D	24/12	27.0 - 29.0	8-13-23-31	36	27	22			Medium Dense, Gray, fine SAND and SILT, trace Clay.	
							30				
							40				
30							37				
							45				
	6D	24/12	32.0 - 34.0	26-21-25-20	46	35	43			Dense, Gray, fine SAND, little Silt.	
							40				
								-47.7		34.0	
							38				
35							38				
							40				
	7D	24/12	37.0 - 39.0	21-22-25-21	47	35	67			Dense, Brown, fine to coarse SAND and GRAVEL, little Silt.	
							71				
							67				
40							72				
							70				
							71				
	8D	24/3	43.5 - 45.5	14-13-14-16	27	20	10			Medium Dense, Brown, fine to coarse SAND, little Gravel, trace Silt.	
							33				
45							48				
							51				
	9D	24/9	47.0 - 49.0	31-32-22-17	54	41	OH			Dense, Brown, fine to coarse SAND and GRAVEL, little Silt.	
50											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 7 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

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: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-402
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-13.7	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/28/08-09/02/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 15+70, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐	Hydraulic ☒	Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							60				
							62				
	10D	24/5	52.0 - 54.0	11-13-12-10	25	19	36			Medium Dense, Brown, GRAVEL and fine to coarse Sand, little Silt.	
							46				
							52				
55							50				
							49				
	11D	24/6	57.0 - 59.0	10-11-12-10	23	17	37			Medium Dense, Brown, GRAVEL and fine to coarse SAND, little Silt.	
							52				
							47				
60							49				
							50				
	12D	24/3	62.0 - 64.0	15-15-18-18	33	25	55			Medium Dense, Brown, GRAVEL and fine to coarse SAND, some Silt.	
							54				
							60				
65							65				
							70				
	13D	24/10	67.0 - 69.0	15-35-21-25	56	42	85			Dense, Gray, fine to coarse SAND, some Gravel, little Silt	
							84				
							95				
70							100				
	14D		72.0 - 72.0	100/0"			113	-85.7		72.0	
	R1	60/57	72.0 - 77.0	RQD = 47%			NQ			Bedrock: Hard, Fresh, fine grained, Gray, BASALT. Joints are close, low angle, planar, rough, fresh, partially open. R1: Core Times (min) 72-73 (7) 73-74 (7) 74-75 (6)	
75											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 7 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	New Bridge, Route 103			**Boring No.:**	BB-YYR-402
Location:	York, Maine			**PIN:**	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	-13.7	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/28/08-09/02/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 15+70, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							NQ			75-76 (6) 77-78 (8)	
	R2	36/24	77.0 - 80.0	RQD = 0%						Hard, Fresh, Fine grained, Gray BASALT. Joints are close, low angle to moderately dipping, planar, rough, fresh, tight to partially open. R2: Core Times (min) 77-78 (8) 78-79 (14) 79-80 (19)	
80								-93.7		80.0 **Bottom of Exploration at 80.00 feet below ground surface.**	
85											
90											
95											
100											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 7 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-403
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-20.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/25/08-08/28/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 14+61, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							HP			Mud line approximately 44 feet below top of bridge deck. See Note 1.	
5	1D	24/3	5.0 - 7.0	145-40-11-5	51	38	33			Dense, Gray, fine SAND, little Silt, trace Gravel. (Pushed on piece of gravel or cobble)	
							17				
							18				
							18				
							23				
10	2D	24/12	10.0 - 12.0	5-7-9-9	16	12	30			Medium Dense, Brown, fine to medium SAND, some Silt, trace Gravel.	A-1-b, SM WC=17%
							30				
							29				
							26				
							27				
15	3D	24/12	15.0 - 17.0	8-7-12-11	19	14	20			Medium Dense, Brown, fine to medium SAND, some Silt, trace Gravel. Occasional Gray Silt varves.	
							15				
							10				
							10				
							11				
20	4D	24/15	20.0 - 22.0	WOR-1-1-1	2	2	20	-40.4		20.0 Very Stiff, Top 7": Gray, fine to medium SAND, some Silt. Bottom 8": Gray, SILT.	
	V1		21.0 - 21.5	Su=2975/1900 psf			17			1.8 x 4.6 in tapered vane raw torque reading: V1= 47/30 ft.-lbs. (Initial/Remolded)	
							12				
							13				
25							11				

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 5 ft below mudline.
2. At 60 feet below mudline, heaving sand and gravel was observed 1 ft. into casing. Advanced casing to 66.5 ft.

HP=Hydraulic Push; OH=Open Hole; NR=Not Recorded

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: New Bridge, Route 103 Location: York, Maine	Boring No.: BB-YYR-403 PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-20.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/25/08-08/28/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 14+61, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**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 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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	5D	24/7	25.0 - 27.0	4-17-18-18	35	26	17			Very Stiff, Gray, Silty CLAY, little fine to medium Sand, trace Gravel.	
							27				
							25				
							38				
							42				
30	6D	24/0	30.0 - 32.0	10-10-12-12	22	17	9	-50.4		30.0 No Recovery.	
							22				
	6AD	24/6	32.0 - 34.0				37			Medium Dense, Gray, fine to medium SAND, little Silt.	
							41				
							45				
35	7D	24/10	35.2 - 37.2	15-21-25-33	46	35	5	-55.6		35.2 Dense, Top 2": Gray, fine to medium SAND, trace Silt. Bottom 8": Brown, fine to medium SAND, little Silt.	
							OH				
40	8D	24/11	40.0 - 42.0	17-19-21-20	40	30	25			Medium Dense, Brown, fine to coarse SAND, some Silt, little Gravel (Iron Oxide Staining)	
							51				
							56				
							58				
							58				
45	9D	24/5	45.0 - 47.0	17-18-22-15	40	30	OH			Medium Dense, Brown, fine to coarse SAND and SILT, some Gravel.	
							44				
							53				
							52				
							54				
50											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 5 ft below mudline.
2. At 60 feet below mudline, heaving sand and gravel was observed 1 ft. into casing. Advanced casing to 66.5 ft.

HP=Hydraulic Push; OH=Open Hole; NR=Not Recorded

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-403
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-20.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/25/08-08/28/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 14+61, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**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 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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	10D	24/6	50.0 - 52.0	14-10-11-11	21	16	18			Medium Dense, Brown, fine to coarse SAND, some Gravel, little Silt.	
							38				
							42				
							42				
							43				
55	11D	24/8	55.0 - 57.0	11-12-14-14	26	20	NR			Medium Dense, Brown, fine to coarse SAND, some Silt, trace Gravel (Iron Oxide Staining). NR=Not Recorded	
							35				
							38				
							67				
							89				
60							40			No Sample.	
							90				
							98				
							90				
							85				
65							OH				
	12D	24/9	66.5 - 68.5	13-11-40-58	51	38				Top 6": Dense, Brown, fine to coarse SAND, some Silt, little Gravel. Middle 2": Brown, GRAVEL, some fine Sand, little Silt. Bottom 1": fine to coarse SAND and SILT, trace Gravel. Heaving spoon pushed 6" before sample blow count.	
							160				
							128				
70	13D	18/3	70.5 - 72.0	20-47-133/6"	180	135	OH			Very Dense, Gray, GRAVEL and coarse SAND, trace Silt.	
							219				
							190				
							206				
75											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 5 ft below mudline.
2. At 60 feet below mudline, heaving sand and gravel was observed 1 ft. into casing. Advanced casing to 66.5 ft.

HP=Hydraulic Push; OH=Open Hole; NR=Not Recorded

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	New Bridge, Route 103	**Boring No.:**	BB-YYR-403		
Location:	York, Maine	**PIN:**	15110.00		
Driller:	New Hampshire Boring	**Elevation (ft.)**	-20.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/25/08-08/28/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 14+61, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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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							NR				
	R1	60/45	76.0 - 81.0	RQD = 61%			NQ	-96.4		76.0 Bedrock: Hard, Fresh, fine to medium grained, gray, BASALT. Joints are close to wide, low angle, planar, rough, fresh, partially open. Quartz deposits throughout. R1: Core Times (min) 76-77 (6) 77-78 (6) 78-79 (6) 79-80 (4) 80-81 (7)	
80											
	R2	60/28	81.0 - 86.0	RQD = 30%						Hard, Fresh, fine grained, gray, BASALT. Joints are close, low angle, planar, rough, fresh, partially open to open. R2: Core Times (min) 81-82 (4) 82-83 (2) 83-84 (5) 84-85 (6) 85-86 (10)	
85											
								-106.4		86.0 **Bottom of Exploration at 86.00 feet below ground surface.**	
90											
95											
100											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 5 ft below mudline.
2. At 60 feet below mudline, heaving sand and gravel was observed 1 ft. into casing. Advanced casing to 66.5 ft.

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: New Bridge, Route 103

Location: York, Maine

Boring No.: BB-YYR-404

PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-16.4	**Auger ID/OD:**	NA
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/11/08-08/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 13+70, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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WOH = weight of 140lb. hammer
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S_u = Insitu Field Vane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HP			Mud line approximately 39 feet below top of bridge deck. See Note 1.	
							21				
							26				
							26				
5	1D	24/12	5.0 - 7.0	9-6-3-4	9	7	17			Medium stiff. Top 6": Dark Gray/Black fine to medium SAND and Shell Fragments. Bottom 6": Gray organic SILT and CLAY.	
							19				
							31				
							33				
							33				
10	2D	24/0	10.0 - 12.0	8-7-7-7	14	11	16			No recovery. See Note 2.	
							30				
							47				
							40				
							36				
15	3D	24/7	15.0 - 17.0	10-11-11-10	22	17	34	-31.4		15.0 Medium Dense, Brown GRAVEL, little medium to coarse Sand, trace Silt.	
							50				
							51				
							39				
							30				
20	4D	24/5	20.0 - 22.0	9-3-3-4	6	5	30			Loose, Brown, GRAVEL, trace fine to medium Sand.	
							22				
							22				
							28				
							24				
25											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 2 ft below mudline.
2. Sample 2D: Driller noted gravel, twigs and wood while spinning the tri-cone.
3. Probable Boulders encountered between 65 to 69 feet; Casing bent during drilling; Removed casing and readvanced boring with a new piece of lead casing.
4. At 80 ft., Driller unable to collect sample due to sand heaving into 4" casing.

HP=Hydrualic Push, OH=Open Hole, NR=Not Recorded

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: New Bridge, Route 103

Location: York, Maine

Boring No.: BB-YYR-404

PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-16.4	**Auger ID/OD:**	NA
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/11/08-08/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 13+70, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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	5D	24/3	25.0 - 27.0	5-3-3-4	6	5	32			Loose, Brown, GRAVEL, little fine to medium Sand, trace Silt.	
							31				
							30				
							38				
							44				
30	6D	24/12	30.0 - 32.0	12-11-11-18	22	17	63			Medium Dense, Brown, GRAVEL, some fine to medium Sand, trace Silt.	
							48				
							48				
							40				
							44				
35	7D	24/12	35.0 - 37.0	22-15-15-26	30	23	OH			Medium Dense, Brown, GRAVEL, some fine to medium Sand, little Silt.	
							67				
							100				
							84				
40	8D	24/8	40.0 - 42.0	43-24-17-15	41	31	OH			Dense, Brown, GRAVEL and fine to coarse SAND, little Silt.	
							50				
							47				
							42				
45	9D	24/10	45.0 - 47.0	10-15-13-13	28	21	42	-61.4		45.0 Medium Dense, Brown, fine to coarse SAND, little Silt, trace Gravel.	
							51				
							68				
							97				
							100				
50											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 2 ft below mudline.
2. Sample 2D: Driller noted gravel, twigs and wood while spinning the tri-cone.
3. Probable Boulders encountered between 65 to 69 feet; Casing bent during drilling; Removed casing and readvanced boring with a new piece of lead casing.
4. At 80 ft., Driller unable to collect sample due to sand heaving into 4" casing.

HP=Hydrualic Push, OH=Open Hole, NR=Not Recorded

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:	New Bridge, Route 103			**Boring No.:**	BB-YYR-404
Location:	York, Maine			**PIN:**	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	-16.4	**Auger ID/OD:**	NA
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/11/08-08/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 13+70, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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	10D	24/6	50.0 - 52.0	12-13-15-30	28	21	92			Medium Dense, Brown, fine to coarse SAND, little Gravel, trace Silt.	
							94				
							105				
							100				
							97				
55	11D	24/6	55.0 - 57.0	11-11-10-10	21	16	84			Medium Dense, Gray, fine to medium SAND, some Silt, trace Gravel.	
							97				
							110				
							115				
							100				
60	12D	24/15	60.0 - 62.0	14-15-15-17	30	23	109	-76.4		60.0 Medium Dense, Brown, fine to medium SAND, some Silt, trace Gravel.	
							105				
							114				
							114				
							101				
65	13D	24/10	65.0 - 67.0	24-49-20-13	69	52	OH			Very Dense, Gray, GRAVEL and fine to coarse SAND, trace Silt. See Note 3.	
70	14D	24/1	70.0 - 72.0	19-15-17-22	32	24				Medium Dense, Piece of Gravel/Probable Broken Cobble.	
75											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 2 ft below mudline.
2. Sample 2D: Driller noted gravel, twigs and wood while spinning the tri-cone.
3. Probable Boulders encountered between 65 to 69 feet; Casing bent during drilling; Removed casing and readvanced boring with a new piece of lead casing.
4. At 80 ft., Driller unable to collect sample due to sand heaving into 4" casing.

HP=Hydrualic Push, OH=Open Hole, NR=Not Recorded

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: New Bridge, Route 103 Location: York, Maine	Boring No.: BB-YYR-404 PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-16.4	**Auger ID/OD:**	NA
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/11/08-08/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 13+70, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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	24/8	75.0 - 77.0	24-36-35-21	71	53	OH			Very Dense, Gray, GRAVEL, some fine to medium Sand, little Silt.	
80							100			See Note 4.	
							>100				
85											
90											
							150				
							155				
							152				
95	16D	7/4	95.0 - 95.6	134-60/1"			NR			Very Dense, light Gray.	
	R1	60/35	96.5 - 101.5	RQD = 58%			NQ	-112.9		96.5 Bedrock: Hard, Fresh, fine-grained, gray, BASALT. Joints are close to moderate, low angle to moderately dipping, planar, rough, fresh and tight. R1: Core Times (min) 96.5-97.5 (8) 97.5-98.5 (6) 98.5-99.5 (6)	
100											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 2 ft below mudline.
2. Sample 2D: Driller noted gravel, twigs and wood while spinning the tri-cone.
3. Probable Boulders encountered between 65 to 69 feet; Casing bent during drilling; Removed casing and readvanced boring with a new piece of lead casing.
4. At 80 ft., Driller unable to collect sample due to sand heaving into 4" casing.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-404
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-16.4	**Auger ID/OD:**	NA
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/11/08-08/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 13+70, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							NQ			99.5-100.5 (6) 100.5-101.5 (6)	
	R2	24/6	101.5 - 103.5	RQD = 25%						Hard, fresh, fine-grained, gray. BASALT. Joints are close to moderate, low angle to moderately dipping, planar, rough, fresh and tight. R2: Core Times (min) 101.5-102.5 (15) 102.5-103.5 (8)	
								-119.9		103.5	
										Bottom of Exploration at 103.50 feet below ground surface.	
105											
110											
115											
120											
125											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 2 ft below mudline.
2. Sample 2D: Driller noted gravel, twigs and wood while spinning the tri-cone.
3. Probable Boulders encountered between 65 to 69 feet; Casing bent during drilling; Removed casing and readvanced boring with a new piece of lead casing.
4. At 80 ft., Driller unable to collect sample due to sand heaving into 4" casing.

HP=Hydrualic Push, OH=Open Hole, NR=Not Recorded

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-405A
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-12.0	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/02/08-09/10/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+95, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							HP			Mud line approximately 33 feet below top of bridge deck. See Note 1.	
5											
	1D	24/6	6.0 - 8.0	WOH						Very Soft, Gray, CLAY, Shell fragments. Hydrogen Sulfide odor, Organic.	A-7,CL WC=41% LL=45, PL=23 PI=22
10											
	2D	24/11	11.0 - 13.0	1-1-2-2	3	2				Soft, Dark Gray, CLAY, Roots and Wood Pieces. Piece of Gravel, Organic.	
15											
	3D	24/16	16.0 - 18.0	3-2-1-1	3	2				Soft, Gray, CLAY with approximately 2" thick fine Sand beds/layers.	A-4, CL WC=24% LL=25, PL=9 PI=16
	U1	24/16	19.0 - 21.0							Gray, fine SAND, some Silt and Clay.	
20								-32.0		20.0	
	4D	24/5	21.0 - 23.0	7-11-13-12	24	18	31			Medium Dense, Gray, fine to medium SAND, some Silt, trace Gravel.	
							37				
							37				
							33				
25											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 21 ft below mudline.
2. Probable boulders from 86 feet to 88 feet below ground surface, based on drilling behavior.

HP=Hydraulic Push
NR=Not Recorded

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine
Boring No.: BB-YYR-405A
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-12.0	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/02/08-09/10/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+95, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							38				
	5D	24/5	26.0 - 28.0	11-12-12-12	24	18	29			Medium Dense, Gray, fine to medium SAND, little Gravel, trace Silt.	
							39				
							37				
							43				
30							51				
	6D	24/8	31.0 - 33.0	7-12-13-15	25	19	23			Medium Dense, Gray, fine to medium SAND, some Silt, trace Gravel	
							27				
							31				
							33				
35							40				
	7D	24/5	36.0 - 38.0	8-9-8-6	17	13	28			Medium Dense, Gray, fine to coarse SAND, little Silt, trace Gravel.	
							40				
							44				
							44				
40							45				
	8D	24/0	41.0 - 43.0	5-5-6-6	11	8	50			Loose, Gray, fine to coarse SAND, some Gravel, Little Silt.	
							53				
	8AD	24/5	43.0 - 45.0				63				
							75				
45							80				
	9D	24/4	46.0 - 48.0	10-11-11-11	22	17	66			Medium dense, Gray, fine to medium SAND, little Gravel, trace Silt.	
							67				
							68				
50							79				

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 21 ft below mudline.
2. Probable boulders from 86 feet to 88 feet below ground surface, based on drilling behavior.

HP=Hydraulic Push
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-405A
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-12.0	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/02/08-09/10/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+95, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							78				
	10D	24/5	51.0 - 53.0	10-13-11-11	24	18	74			Medium Dense, Gray, fine to medium SAND, some Gravel, trace Silt.	
							80				
							84				
							97				
55							100				
	11D	24/12	56.0 - 58.0	10-18-26-16	44	33	94			Dense, Gray, fine to medium SAND, little Gravel, trace Silt.	
							107				
							130				
							115				
60							110				
	12D	24/6	61.0 - 63.0	8-9-8-9	17	13	46			Medium Dense, Gray, fine to coarse SAND, trace Silt, trace Gravel.	
							60				
							65				
							90				
65							98				
	13D	24/12	66.0 - 68.0	5-6-5-7	11	8	42			Loose, Gray, fine to medium SAND, little Gravel, trace Silt.	
							46				
							53				
							70				
70							99	-83.0		71.0	
	14D	24/5	71.0 - 73.0	6-14-13-11	27	20	110			Medium Dense, Gray, fine to medium SAND and Gravel.	
							115				
							128				
							135				
75											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 21 ft below mudline.
2. Probable boulders from 86 feet to 88 feet below ground surface, based on drilling behavior.

HP=Hydraulic Push
NR=Not Recorded

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: New Bridge, Route 103
Location: York, Maine
Boring No.: BB-YYR-405A
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-12.0	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/02/08-09/10/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+95, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal

Hammer Efficiency Factor: 0.45 **Hammer Type:** Automatic ☐ Hydraulic ☒ Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							164				
	15D	24/8	76.0 - 78.0	35-20-18-15	38	29	132			Medium Dense, Gray, fine to medium SAND and Gravel.	
							134				
							108				
							124				
80							151				
	16D	24/7	81.0 - 83.0	34-34-24-17	58	44	111			Dense, Gray, GRAVEL, some fine to medium Sand, little Silt	
							110				
							167				
							181				
85							215				
							>200			See Note 2.	
	17D	6/6	88.0 - 88.5	90-50/0"						Very Dense, Gray, GRAVEL, some fine to medium Sand, little Silt	
90	R1	60/6	90.0 - 95.0	RQD = 0%						Cobbles/Boulders R1: Core Times (min) 90-91 (3) 91-92 (2) 92-93 (2) 93-94 (2) 94-95 (3)	
							160				
							>100				
95											
	18D	5/5	96.0 - 96.4	175/5"						Very Dense, Gray, GRAVEL, some fine to medium Sand, little Silt	
							131				
							215				
100											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 21 ft below mudline.
2. Probable boulders from 86 feet to 88 feet below ground surface, based on drilling behavior.

HP=Hydraulic Push
NR=Not Recorded

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103		**Boring No.:**	BB-YYR-405A
Location: York, Maine		**PIN:**	15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-12.0	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/02/08-09/10/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+95, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
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q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							190				
							195				
							205				
							260				
							550				
105							225				
	19D	3/3	106.0 - 106.3	125/3"			520			Very Dense, Gray, GRAVEL and coarse Sand.	
							505				
							509				
							515				
110							407				
	R2	60/1	111.0 - 116.0	RQD = 0%			250			Boulders and Cobbles. R2: Core Times (min) 111-112 (2) 112-113 (1) 113-114 (2) 114-115 (2) 115-116 (2)	
							300				
							450				
							NR				
115										Wash Water: Boulders and cobbles, Gravel and Sand, Black, Gray, White Rock chips.	
							NQ	-131.0		119.0	
120											
	R3	56/42	123.0 - 127.7	RQD = 32%						Bedrock: Hard, Fresh, Fine grained, Gray, BASALT. Joints are close, low angle to moderately dipping, planar, rough, fresh, tight to partially open. R3: Core Times (min)	
125											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 21 ft below mudline.
2. Probable boulders from 86 feet to 88 feet below ground surface, based on drilling behavior.

HP=Hydraulic Push
NR=Not Recorded

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Boring No.: BB-YYR-405A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-405A
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-12.0	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/02/08-09/10/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+95, 10.5 Rt	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**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 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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.
125							NQ			123-124 (5) 124-125 (4) 125-126 (5) 126-127 (10)	
								-139.7		127.7	
										Bottom of Exploration at 127.70 feet below ground surface.	
130											
135											
140											
145											
150											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 21 ft below mudline.
2. Probable boulders from 86 feet to 88 feet below ground surface, based on drilling behavior.

HP=Hydraulic Push
NR=Not Recorded

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: New Bridge, Route 103 Location: York, Maine	Boring No.: BB-YYR-405B PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-1.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/11/08-09/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+17 10.5 RT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
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S_u = Insitu Field Vane Shear Strength (psf)
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							HP			Mud line approximately 20 feet below the top of bridge deck. See Note 1.	
	1D	24/4	4.0 - 6.0	1-1-1-1	2	2				Soft, Dark Gray, ORGANIC SILT, shell fragments	
5											
10	2D	24/24	10.5 - 12.5	1-7-11-15	18	14				Stiff, Top 10": Gray, CLAY and SILT, laminated with 1/8-1/16" Silt lenses. Bottom 14": Light brown, Clayey SILT.	
	3D	24/24	14.0 - 16.0	4-6-8-9	14	11				Stiff, Light Brown, CLAY, Varves, Root Structure.	A-7-6, CL WC=31% LL=46, PL=25 PI=21
15											
	4D	24/24	19.0 - 21.0	WOH						Very Soft, Dark Gray, CLAY and SILT	
20											
25	5D	24/24	24.0 - 26.0	WOH-1-1-1	2	2				Soft, Gray, CLAY.	A-7-6, CL WC=40%

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 35 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-405B
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-1.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/11/08-09/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+17 10.5 RT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal

Hammer Efficiency Factor: 0.45 **Hammer Type:** Automatic ☐ Hydraulic ☒ Rope & Cathead ☐

Definitions:
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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
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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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							HP				LL=43, PL=21 PI=22
	6D	24/13	29.0 - 31.0	5-4-9-14	13	10		-30.4		Loose, Gray, fine SAND and SILT 29.0	
30											
35							70				
							68				
							66				
							65				
							67				
40							92				
							83				
							OH				
45											
	7D	24/8	49.0 - 51.0	8-13-18-22	31	23		-50.4		Medium Dense, Brown, fine SAND, trace Silt. 49.0	
50											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 35 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	New Bridge, Route 103	**Location:**	York, Maine	**Boring No.:**	BB-YYR-405B
				PIN:	15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	-1.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/11/08-09/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+17 10.5 RT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
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S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							OH				
							44				
55							45				
							40				
							39				
							37				
							98				
60							65				
							68				
							64				
							69				
							79				
65							83				
							76				
							83				
							84				
	8D	24/9	69.0 - 71.0	27-26-35-28	61	46	76			Dense, Gray, fine to medium SAND, little Gravel.	
70							84				
							100				
							77				
							85				
							85				
75											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 35 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	New Bridge, Route 103			**Boring No.:**	BB-YYR-405B
Location:	York, Maine			**PIN:**	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	-1.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/11/08-09/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+17 10.5 RT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐	Hydraulic ☒	Rope & Cathead ☐

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Insitu Field Vane Shear Strength (psf)	$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt	SSA = Solid Stem Auger	T_v = Pocket Torvane Shear Strength (psf)	WC = water content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt	RC = Roller Cone	N-uncorrected = Raw field SPT N-value	PL = Plastic Limit
V = Insitu Vane Shear Test	WOH = weight of 140lb. hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR = weight of rods	N_{60} = SPT N-uncorrected corrected for hammer efficiency	G = Grain Size Analysis
	WO1P = Weight of one person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	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							117				
							130				
							120				
							118				
							87				
80							74				
							78			Boulders encountered during drilling.	
							103				
							92				
							90				
85							152				
							180				
							680				
							NQ	-89.9		88.5	
90											
	R1	24/17	92.5 - 94.5	RQD = 0%						Bedrock: Hard , Moderately weathered, Aphantic, Dark Gray, PHYLLITE. Joints are very close, high angle, undulating, rough. R1: Core Times (min) 92.5-93.5 (5) 93.5-94.5 (6)	
	R2	60/26	94.5 - 99.5	RQD = 6%						Hard, Moderately weathered, Aphantic, Dark Gray, PHYLLITE. Joints are very close, high angle, undulating, rough. R2: Core Times (min) 94.5-95.5 (4) 95.5-96.5 (10) 96.5-97.5 (10) 97.5-98.5 (9) 98.5-99.5 (5)	
95											
	R3	48/37	99.5 - 103.5	RQD = 16%						Hard, Moderately weathered, Aphantic, Dark Gray, PHYLLITE. Joints	
100											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 35 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

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:	New Bridge, Route 103			**Boring No.:**	BB-YYR-405B
Location:	York, Maine			**PIN:**	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	-1.4	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/11/08-09/15/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	ST 12+17 10.5 RT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person

S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							NQ			are very close, high angle, undulating, rough. R3: Core Times (min) 99.5-100.5 (5) 100.5-101.5 (5) 101.5-102.5 (5) 102.5-103.5 (5)	
								-104.9		103.5 **Bottom of Exploration at 103.50 feet below ground surface.**	
105											
110											
115											
120											
125											

Remarks:

1. Driller advanced 4" casing through bridge deck (5 1/8" asphalt; 5 7/8" concrete). Casing was advanced by own weight or hydraulic push to 35 ft below mudline.
HP=Hydraulic Push
OH=Open Hole

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: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-406
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	16.6	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/15/08-08/20/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 11+60, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed

Hammer Efficiency Factor: 0.45 **Hammer Type:** Automatic ☐ Hydraulic ☒ Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							10	16.1		Asphalt (approximately 6") 0.5	
	1D	24/10	1.0 - 3.0	22-24-21-18	45	34	20			Dense, Light Brown, fine to coarse SAND and GRAVEL, little Silt, Asphalt at top of sample. Dry.	A-1-b, SM WC=5%
							17				
							15				
	2D	24/12	4.0 - 6.0	6-6-6-4	12	9	OH			Loose, Light Brown, fine to medium SAND and SILT, little Gravel. Moist.	A-4, SM WC=12%
5											
	3D	24/5	9.0 - 11.0	29-32-14-12	46	35	63			Dense, Brown, fine to medium SAND and SILT, some gravel.	
10							42			See Note 1.	
							23				
							24				
							28				
	4D	24/3	14.0 - 16.0	11-8-14-12	22	17	37			Medium Dense, Brown, fine to medium SAND and SILT, some Gravel.	
15							55				
							22				
							24				
							30	-2.4		19.0	
	5D	24/10	19.0 - 21.0	10-9-9-6	18	14	HP			Medium Dense, Gray, fine SAND and SILT, trace Clay.	
20											
	6D	24/18	24.0 - 26.0	27-20-24-29			OH				
25	6AD	24/20	24.0 - 26.0	11-16-17-19	44 32	33 24				Hard, Gray, SILT and CLAY.	

Remarks:

1. Driller noted an obstruction approximately 10 feet below ground surface. Retrieved casing. Readvanced boring.
2. Offset boring BB-YYR-406 approximately 3 feet north; advanced boring to 24 feet below ground surface with no sampling, continued typical sampling interval below 24 feet.
3. Spoon refusal at 99.2 feet b.g.s.; Driller advanced roller cone to 100.5 feet to confirm possible bedrock and not boulder.

HP=Hydraulic Push; OH=Open Hole

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:	New Bridge, Route 103	**Location:**	York, Maine	**Boring No.:**	BB-YYR-406
				PIN:	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	16.6	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/15/08-08/20/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 11+60, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							OH			See Note 2.	
	7D	24/24	29.0 - 31.0	8-11-14-16	25	19				Very Stiff, Gray, SILT and CLAY.	
30											
	MU1			-							
	8D	24/24	33.0 - 35.0	3-2-3-4	5	4				Soft, Gray, CLAY and Silt/Clay laminations.	CL WC=31%
35	9D V1	24/24	35.0 - 37.0 35.5 - 36.0	WOR-1-2-4 Su=2530/315 psf	3	2				Very Stiff, Gray, SILT and CLAY laminated fine Sand lenses. 1.8 x 4.6 in tapered vane raw torque readings: V1=40/5 ft.-lbs. (Initial/Remolded)	
	U2	24/24	37.0 - 39.0								A-6, CL WC=35% LL=37, PL=18 PI=19 Tv=0.58 tsf UW=120 pcf
	10D	24/24	39.0 - 41.0	4-1-1-1	2	2				Soft. Top 8": Brown/Gray, fine SAND (6" layers), little Silt. Bottom 10": Gray, Silt and Clay.	
40								-23.4		40.0	
	11D	24/20	41.0 - 43.0	WOR-4-14-21	18	14				Medium Dense, Gray, SILT and GRAVEL, some Clay, little fine to medium Sand.	
	12D	24/14	44.0 - 46.0	2-4-6-6	10	8	64			Loose, Gray, fine SAND and SILT.	
45							53				
							47				
							49				
							47				
	13D	24/11	49.0 - 51.0	11-13-16-18	29	22	60			Medium Dense, Brown, fine SAND, trace Silt, Iron Oxide staining.	
50											

Remarks:

1. Driller noted an obstruction approximately 10 feet below ground surface. Retrieved casing. Readvanced boring.
2. Offset boring BB-YYR-406 approximately 3 feet north; advanced boring to 24 feet below ground surface with no sampling, continued typical sampling interval below 24 feet.
3. Spoon refusal at 99.2 feet b.g.s.; Driller advanced roller cone to 100.5 feet to confirm possible bedrock and not boulder.

HP=Hydraulic Push; OH=Open Hole

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:	New Bridge, Route 103	**Location:**	York, Maine	**Boring No.:**	BB-YYR-406
				PIN:	15110.00
Driller:	New Hampshire Boring	**Elevation (ft.)**	16.6	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/15/08-08/20/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 11+60, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							78				
							77				
							75				
							72				
	14D	24/12	54.0 - 56.0	9-18-22-26	40	30	80	-37.4		54.0 Dense, Brown, fine to medium SAND, trace Gravel, trace Silt.	
55							84				
							90				
							95				
							92				
	15D	24/9	59.0 - 61.0	27-31-27-29	58	44	91			Dense, Brown, GRAVEL, some fine to coarse Sand, little Silt.	
60							83				
							75				
							85				
							81				
	16D	24/11	64.0 - 66.0	31-30-28-21	58	44	OH			Dense, Brown, fine to coarse SAND, some Silt, little Gravel.	
65											
	17D	24/12	69.0 - 71.0	19-20-26-27	46	35				Dense, Brown, fine to medium SAND, trace Gravel (Iron Oxide staining), trace Silt.	
70											
	18D	24/12	74.0 - 76.0	28-38-41-43	79	59				Very Dense, Brown, fine to medium SAND, trace Gravel, trace Silt.	
75											

Remarks:

1. Driller noted an obstruction approximately 10 feet below ground surface. Retrieved casing. Readvanced boring.
2. Offset boring BB-YYR-406 approximately 3 feet north; advanced boring to 24 feet below ground surface with no sampling, continued typical sampling interval below 24 feet.
3. Spoon refusal at 99.2 feet b.g.s.; Driller advanced roller cone to 100.5 feet to confirm possible bedrock and not boulder.

HP=Hydraulic Push; OH=Open Hole

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Bridge, Route 103		**Boring No.:**	BB-YYR-406
Location: York, Maine		**PIN:**	15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	16.6	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/15/08-08/20/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 11+60, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							OH				
	19D	24/12	79.0 - 81.0	25-31-27-31	58	44	>100			Dense, Brown, GRAVEL, some fine to medium Sand, little Silt.	
80											
	20D	24/10	84.0 - 86.0	46-44-29-27	73	55	115			Very Dense, Top 8": Brown, GRAVEL, some fine to medium Sand, little Silt. Bottom 2": Gray, SILT.	
85							133				
							130				
							146				
							156				
	21D	24/10	89.0 - 91.0	23-24-34-48	58	44	>100			Dense, Brown, GRAVEL, some fine to medium Sand, little Silt.	
90											
	22D	24/11	94.0 - 96.0	36-34-21-21	55	41				Dense, Brown, GRAVEL and fine to coarse SAND, little Silt.	
95											
								-81.9		98.5	
100	23D	2/2	99.0 - 99.2	100/2"						Refusal. Rock fragment. SS Tip. See Remark 3.	

Remarks:

1. Driller noted an obstruction approximately 10 feet below ground surface. Retrieved casing. Readvanced boring.
2. Offset boring BB-YYR-406 approximately 3 feet north; advanced boring to 24 feet below ground surface with no sampling, continued typical sampling interval below 24 feet.
3. Spoon refusal at 99.2 feet b.g.s.; Driller advanced roller cone to 100.5 feet to confirm possible bedrock and not boulder.

HP=Hydraulic Push; OH=Open Hole

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: New Bridge, Route 103
Location: York, Maine

Boring No.: BB-YYR-406
PIN: 15110.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	16.6	**Auger ID/OD:**	na
Operator:	Greg	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	08/15/08-08/20/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 11+60, 10.5 Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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	R1	60/41	100.5 - 105.5	RQD = 8%			NQ	-83.9		100.5 Bedrock: Hard, fresh to slightly weathered, fine- grained, gray PHYLLITE. Joints are very close, moderately dipping to vertical, planar, smooth, tight. R1: Core Times (min) 100.5-101.5 (6) 101.5-102.5 (6) 102.5-103.5 (5) 103.5-104.5 (9) 104.5-105.5 (5)	
105	R2	36/30	105.5 - 108.5	RQD = 0%						Hard, slightly weathered, highly fractured, fine-grained, gray, PHYLLITE. R2: Core Times (min) 105.5-106.5 (3) 106.5-107.5 (7) 107.5-108.5 (8)	
	R3	60/58	108.5 - 113.5	RQD = 28%						Hard, fresh to slightly weathered, fine- grained gray, PHYLLITE. Joints are very close to moderately spaced, moderately dipping to high angle, planar, rough to smooth, fresh to discolored, partially open to tight, occasional Iron Oxide staining. Extensive weathering, Quartz vane noted at apporximately 110 feet. R3: Core times (min) 108.5-109.5 (3) 109.5-110.5 (3) 110.5-111.5 (3) 111.5-112.5 (4) 112.5-113.5 (6)	
110											
								-96.9		113.5 **Bottom of Exploration at 113.50 feet below ground surface.**	
115											
120											
125											

Remarks:

1. Driller noted an obstruction approximately 10 feet below ground surface. Retrieved casing. Readvanced boring.
2. Offset boring BB-YYR-406 approximately 3 feet north; advanced boring to 24 feet below ground surface with no sampling, continued typical sampling interval below 24 feet.
3. Spoon refusal at 99.2 feet b.g.s.; Driller advanced roller cone to 100.5 feet to confirm possible bedrock and not boulder.

HP=Hydraulic Push; OH=Open Hole

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

APPENDIX C – LABORATORY TESTING RESULTS

LABORATORY TESTING DATA SHEET

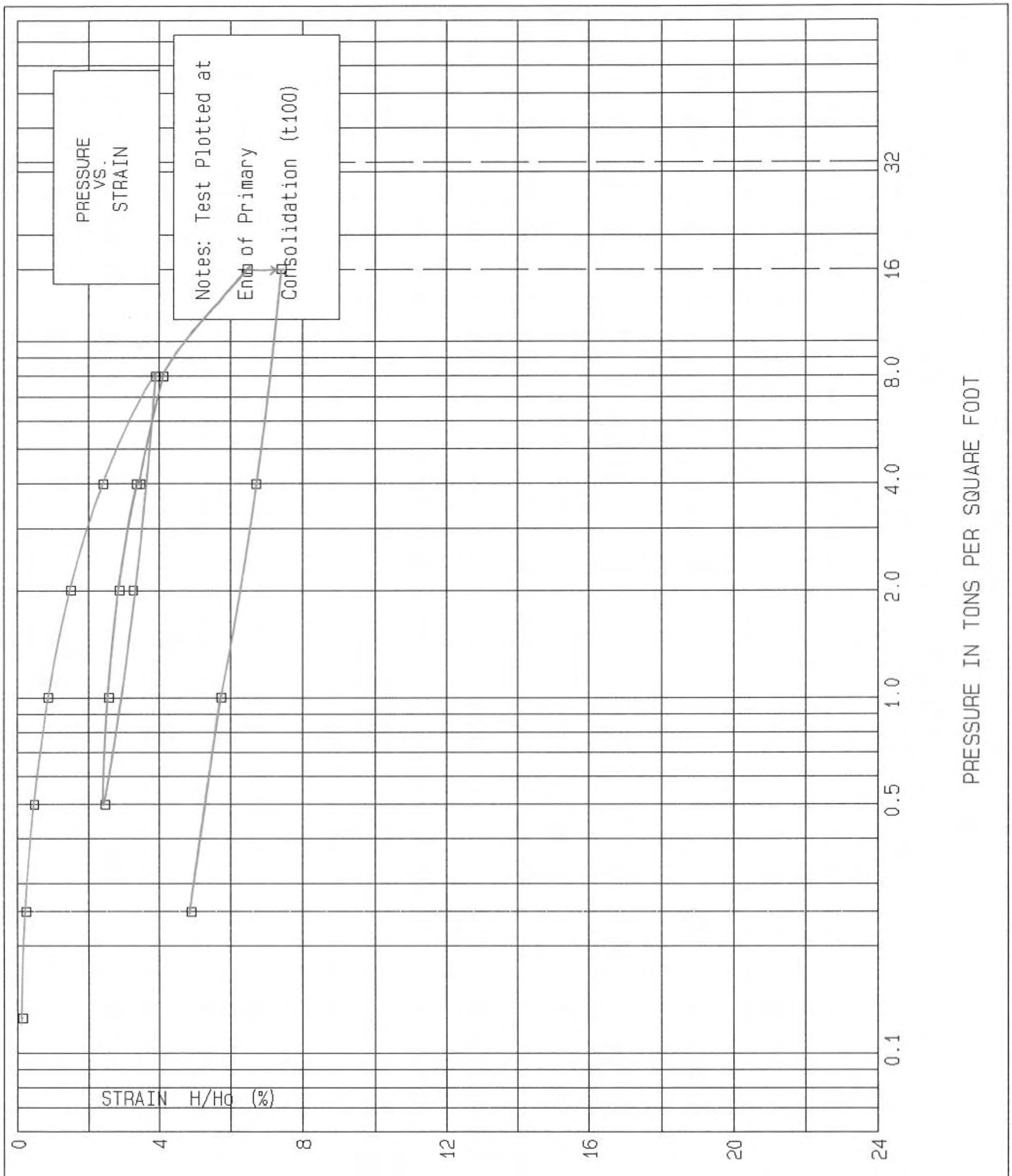
Project Name: Rte 103 Bridge
Project No.: 09.0025577.00
Project Engineer: J. Tooley

Location: York, ME
Assigned By: J. Tooley
Report Date: 10/9/2008

Reviewed By: [signature]
Date Reviewed: 10/9/08

Boring No.	Sample No.	Depth ft.	Lab No.	Identification Tests							Strength Tests						Consol.	
				Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	ORG %	Dry unit wt. pcf	Perme-ability cm/sec	Torvane or Type Test	σ_c psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	$\frac{C_c}{1 + e_0}$	Laboratory Log and Soil Description
BB-YYR-401	U-1	29-31	9		Average Total Unit Weight (29.0-31.0') = 124.3 Pcf													
		29.0-29.5		Sample Saved														Gray Lean Clay trace fine Sand trace Silt Lenses (CL) Very Stiff Consistency
		29.5		24.9								Tv= 1.25 tsf						
		29.7-29.8		25.4						100.5							0.09	
		29.8-29.9		25.0	38	19												
		30.1		27.7								Tv= 1.30 tsf						
		30.1-30.6		Sample Saved														
		30.6		25.8								Tv= 1.30 tsf						

GZA GeoEnvironmental, Inc.



SOIL DESCRIPTION: Gray Lean Clay (CL)

SAMPLE DIAM. 2.5 IN.	LIQUID LIMIT 38 %	PLASTIC LIMIT 19 %	SPECIFIC GRAVITY 2.67

	WATER CONTENT, %	DRY UNIT WEIGHT, pcf	VOID RATIO	SATURATION, %	SAMPLE HEIGHT, INCH
INITIAL	25.40	100.47			0.80
FINAL	24.30	105.62			0.76

Rte 103 Bridge

CONSOLIDATION TEST

BORING NO. BBYYR401
SAMPLE U-1
DEPTH 29.7-29.8'
TECH. MBP
REVIEWER DAS

TEST SERIES NO. C9.1
DATE 10/9/08
FILE 25577.00

FIGURE

Job Number 25577.00
Project Name .... Rte 103 Bridge
Test Number C9.1
Sample Description .. Gray Lean Clay (CL)
Boring Number BBYYR401
Sample Number U-1
Depth .... 29.7-29.8'

Technician .... MBP
Reviewer .... DAS

NOTES: Test Plotted at
End of Primary
Consolidation (t100)

Specific Gravity .... 2.6700E+00
Sample Dry Weight .... 1.0357E+02
Sample Diameter .... 2.5000E+00
Initial Void Ratio .... 0.0000E+00
Initial Sample Height 8.0000E-01
Final Sample Height 7.6100E-01
Initial Water Content .... 2.5400E+01
Final Water Content ...... 2.4300E+01
Liquid Limit .... 3.8000E+01
Plastic Limit .... 1.9000E+01
Initial Saturation .... 0.0000E+00
Final Saturation .... 0.0000E+00
T50 data excluded
T90 data included

Increment umber	Pressure (TSF)	Final Dial	Percent Strain	Void Ratio	T90 (min)	Cv (T90) (/10000) cm²/sec	Cα
1	0.000	0.0	0.00	0.658			
2	0.125	11.0	0.14	0.656			
3	0.250	19.0	0.24	0.654			
4	0.500	38.0	0.47	0.650	1.7	85.21	
5	1.000	70.0	0.87	0.644	1.7	84.66	
6	2.000	121.0	1.51	0.633	2.3	61.93	
7	4.000	195.0	2.44	0.618	2.0	70.09	
8	8.000	313.0	3.91	0.593	2.6	52.61	
9	2.000	262.0	3.27	0.604			
10	0.500	199.0	2.49	0.617			
11	1.000	208.0	2.60	0.615	1.4	98.97	
12	2.000	230.0	2.87	0.611	1.0	138.01	
13	4.000	269.0	3.36	0.602	1.2	114.11	
14	4.000	278.0	3.47	0.601			0.00035
15	8.000	331.0	4.14	0.590	1.7	79.41	
16	16.000	517.0	6.46	0.551	2.6	50.32	
17	16.000	592.0	7.40	0.536			0.0034
18	4.000	536.0	6.70	0.547			
19	1.000	460.0	5.75	0.563			
20	0.250	392.0	4.90	0.577			

LABORATORY TESTING DATA SHEET

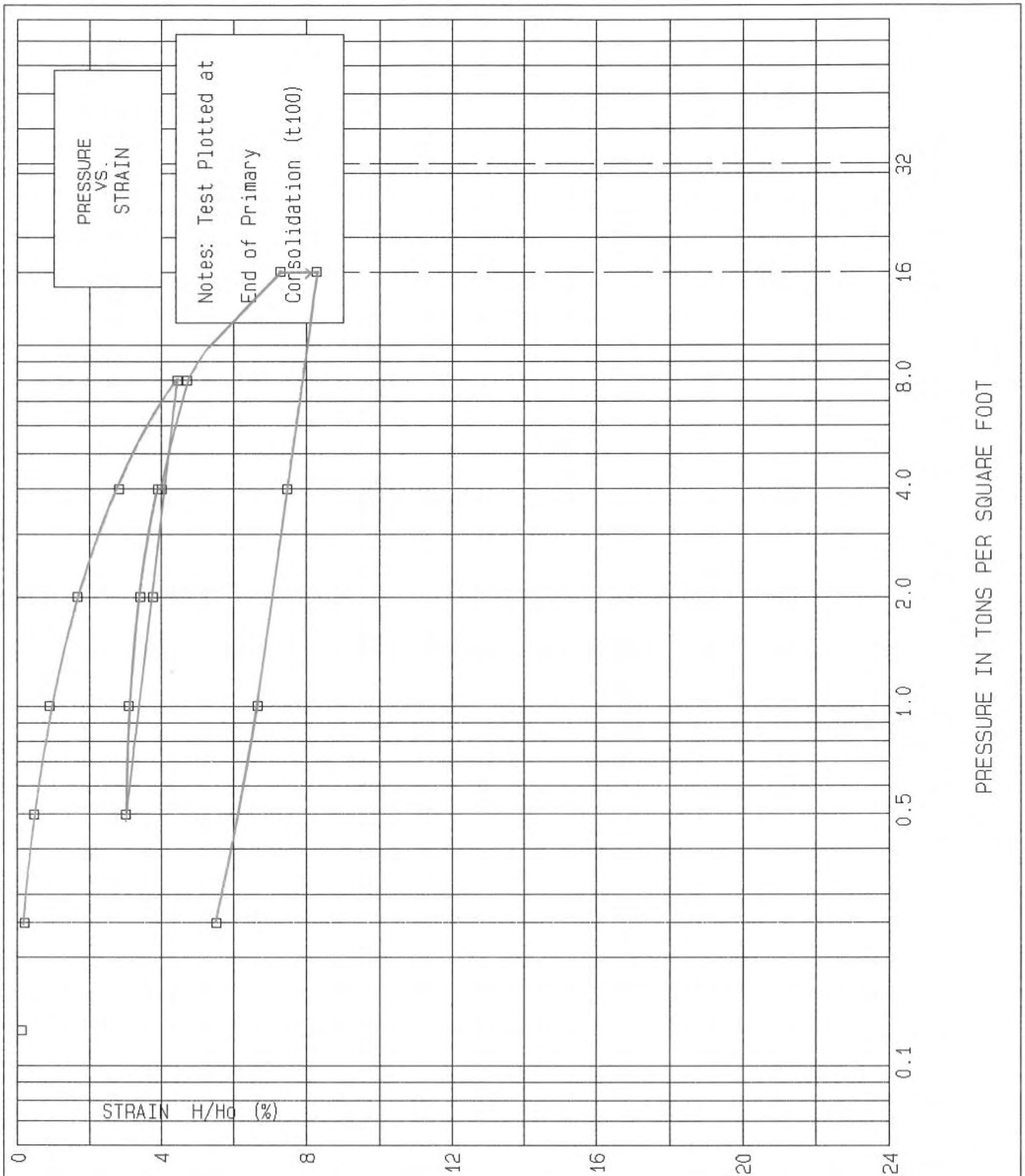
Project Name Rte 103 Bridge
Project No. 09.0025577.00
Project Engineer J. Tooley

Location York, ME
Assigned By J. Tooley
Report Date 10/10/2008

Reviewed By [signature]
Date Reviewed 10/10/08

Boring No.	Sample No.	Depth ft.	Lab No.	Identification Tests: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	ORG %	Dry unit wt. pcf	Strength Tests: Perme-ability cm/sec	Torvane or Type Test	σ_c psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	Consol. $\frac{C_c}{1+e_0}$	Laboratory Log and Soil Description
BB-YYR-401	U-2	37-39	10		Average Total Unit Weight (37.0-39.0') = 127.3 Pcf													
		37.0-37.5		Sample Saved														Gray Lean Clay little fine Sand trace Silt Lenses (CL) Very Stiff Consistency
		37.5		23.2								Tv= 1.10 tsf						
		37.6-37.7		22.6			86											
		37.7-37.8		23.1						104.2							0.10	
		37.8-38.0		23.1	32	16												
		38.1		21.4								Tv= 1.15 tsf						
		38.1-38.6		Sample Saved														
		38.6		23.3								Tv= 1.15 tsf						

GZA GeoEnvironmental, Inc.



SOIL DESCRIPTION: Gray CLAY & SILT, little f Sand

SAMPLE DIAM. 2.5 IN.	LIQUID LIMIT 32 %	PLASTIC LIMIT 16 %	SPECIFIC GRAVITY 2.67

	WATER CONTENT, %	DRY UNIT WEIGHT, pcf	VOID RATIO	SATURATION, %	SAMPLE HEIGHT, INCH
INITIAL	23.10	104.24			0.80
FINAL	21.10	110.30			0.76

Rte 103 Bridge

CONSOLIDATION TEST

BORING NO.	BBYYR401	TEST SERIES NO.	C10.1
SAMPLE	U-2		
DEPTH	37.7-37.8'	DATE	10/10/08
TECH.	MBP		
REVIEWER	DAS	FILE	25577.00

FIGURE

GZA GEOENVIRONMENTAL, INC.
ENGINEERS AND SCIENTISTS

APPENDIX E-16

Job Number 25577.00
Project Name .... Rte 103 Bridge
Test Number C10.1
Sample Description .. Gray CLAY & SILT, little f Sand
Boring Number BBYYR401
Sample Number U-2
Depth .... 37.7-37.8'

Technician .... MBP
Reviewer .... DAS

NOTES: Test Plotted at
End of Primary
Consolidation (t100)

Specific Gravity .... 2.6700E+00
Sample Dry Weight .... 1.0745E+02
Sample Diameter .... 2.5000E+00
Initial Void Ratio .... 0.0000E+00
Initial Sample Height 8.0000E-01
Final Sample Height 7.5600E-01
Initial Water Content .... 2.3100E+01
Final Water Content ...... 2.1100E+01
Liquid Limit .... 3.2000E+01
Plastic Limit .... 1.6000E+01
Initial Saturation .... 0.0000E+00
Final Saturation .... 0.0000E+00
T50 data excluded
T90 data included

Increment umber	Pressure (TSF)	Final Dial	Percent Strain	Void Ratio	T90 (min)	Cv (T90) (/10000) cm²/sec	C α
1	0.000	0.0	0.00	0.598			
2	0.125	8.0	0.10	0.597			
3	0.250	16.0	0.20	0.595	1.2	121.21	
4	0.500	36.0	0.45	0.591	2.3	63.02	
5	1.000	72.0	0.90	0.584	2.6	55.36	
6	2.000	133.0	1.66	0.572	2.6	54.68	
7	4.000	226.0	2.82	0.553	2.3	60.62	
8	8.000	356.0	4.45	0.527	3.2	42.33	
9	2.000	302.0	3.77	0.538			
10	0.500	241.0	3.01	0.550			
11	1.000	247.0	3.09	0.549	1.2	114.27	
12	2.000	273.0	3.41	0.544	2.0	68.28	
13	4.000	312.0	3.90	0.536	1.7	79.66	
14	4.000	321.0	4.01	0.534			0.00037
15	8.000	378.0	4.72	0.523	2.0	66.71	
16	16.000	582.0	7.27	0.482	4.0	32.23	
17	16.000	661.0	8.26	0.466			0.0035
18	4.000	598.0	7.47	0.479			
19	1.000	532.0	6.65	0.492			
20	0.250	441.0	5.51	0.510			

LABORATORY TESTING DATA SHEET

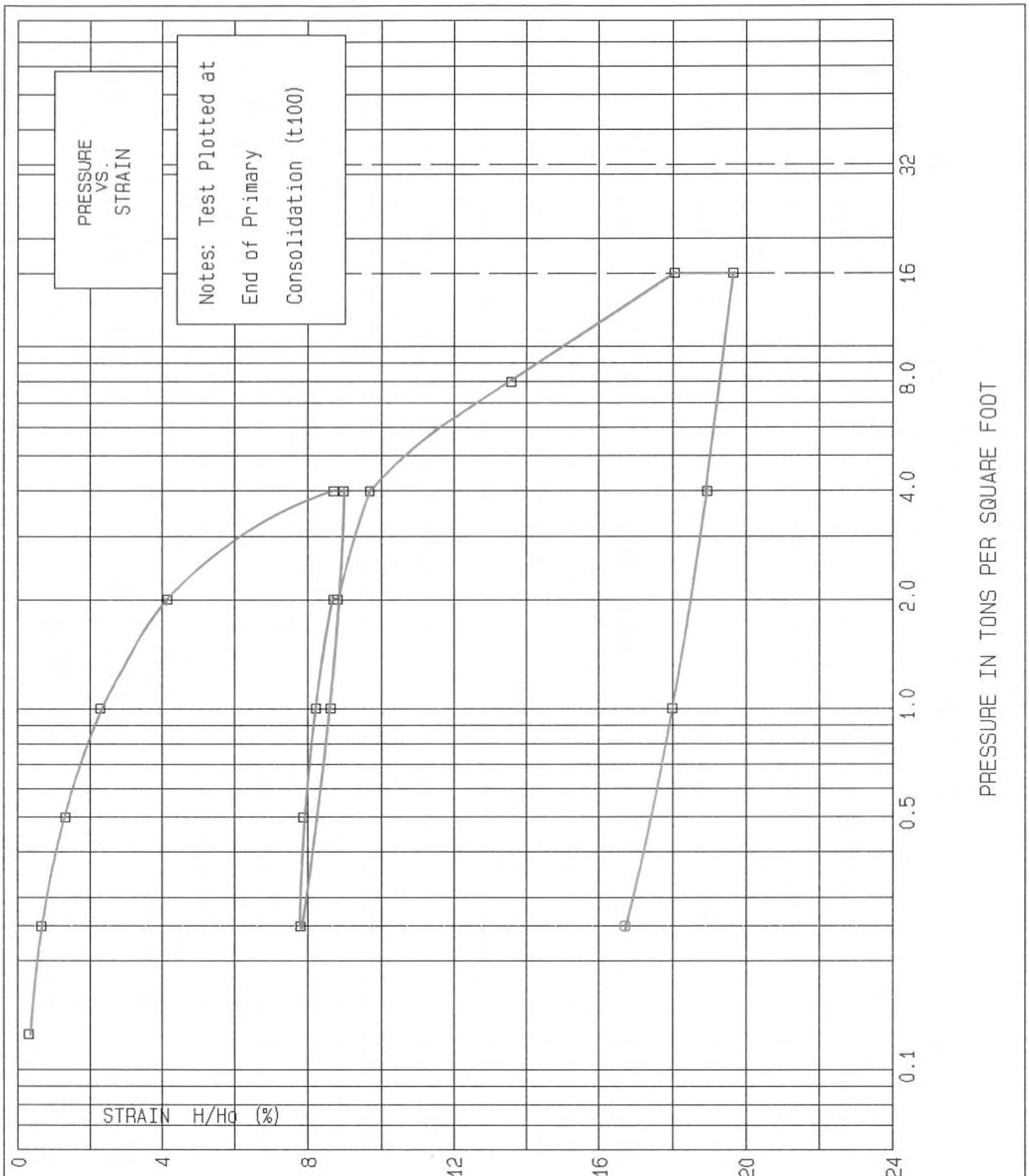
Project Name Rte 103 Bridge
Project No. 09.0025577.00
Project Engineer J. Tooley

Location York, ME
Assigned By J. Tooley
Report Date 10/10/2008

Reviewed By [signature]
Date Reviewed 10/10/08

				Identification Tests							Strength Tests						Consol.	
Boring No.	Sample No.	Depth ft.	Lab No.	Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	ORG %	Dry unit wt. pcf	Perme- ability cm/sec	Torvane or Type Test	σ_c psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	$\frac{C_c}{1+e_0}$	Laboratory Log and Soil Description
BB-YYR-406	U-2	37-39	11		Average Total Unit Weight (37.0-38.9') = 119.7 Pcf													
		37.0-37.5		Sample Saved														
		37.5		33.4								Tv= 0.55 tsf						
		37.7-37.8		35.3						87.8							0.16	Gray Lean Clay trace fine Sand trace Silt Lenses (CL) Stiff Consistency
		37.8-38.0		34.9	37	18												
		38.1		31.3								Tv= 0.58 tsf						
		38.1-38.6		Sample Saved														
		38.6		30.9								Tv= 0.60 tsf						

GZA GeoEnvironmental, Inc.



SOIL DESCRIPTION: Gray CLAY & SILT, trace Sand

SAMPLE DIAM. 2.5 IN. LIQUID LIMIT 37 % PLASTIC LIMIT 18 % SPECIFIC GRAVITY 2.67

	WATER CONTENT, %	DRY UNIT WEIGHT, pcf	VOID RATIO	SATURATION, %	SAMPLE HEIGHT, INCH
INITIAL	35.32	87.75			0.80
FINAL	24.55	105.41			0.67

Rte 103 Bridge

CONSOLIDATION TEST

BORING NO. BBYYR406
SAMPLE U-2
DEPTH 37.7-37.8'
TECH. MST
REVIEWER DAS

TEST SERIES NO. C11.1
DATE 10/10/08

FILE 25577.00

FIGURE

GZA GEOENVIRONMENTAL, INC.
ENGINEERS AND SCIENTISTS

Job Number 25577.00
Project Name .... Rte 103 Bridge
Test Number C11.1
Sample Description .. Gray CLAY & SILT, trace Sand
Boring Number BBYYR406
Sample Number U-2
Depth .... 37.7-37.8'

Technician .... MST
Reviewer .... DAS

NOTES: Test Plotted at
End of Primary
Consolidation (t100)

Specific Gravity .... 2.6700E+00
Sample Dry Weight .... 9.0460E+01
Sample Diameter .... 2.5000E+00
Initial Void Ratio .... 0.0000E+00
Initial Sample Height 8.0000E-01
Final Sample Height 6.6600E-01
Initial Water Content .... 3.5320E+01
Final Water Content ...... 2.4550E+01
Liquid Limit .... 3.7000E+01
Plastic Limit .... 1.8000E+01
Initial Saturation .... 0.0000E+00
Final Saturation .... 0.0000E+00
T50 data excluded
T90 data included

Increment umber	Pressure (TSF)	Final Dial	Percent Strain	Void Ratio	T90 (min)	Cv (T90) (/10000) cm²/sec	C α
1	0.000	0.0	0.00	0.899			
2	0.125	24.0	0.30	0.893			
3	0.250	51.0	0.64	0.886	2.9	49.84	
4	0.500	106.0	1.32	0.873	4.0	35.76	
5	1.000	183.0	2.29	0.855	4.0	35.17	
6	2.000	332.0	4.15	0.820	4.8	28.47	
7	4.000	697.0	8.71	0.733	10.2	12.52	
8	4.000	717.0	8.96	0.728			
9	1.000	689.0	8.61	0.735			
10	0.250	622.0	7.77	0.751			
11	0.500	630.0	7.87	0.749	3.2	38.74	
12	1.000	657.0	8.21	0.743	3.2	38.55	
13	2.000	697.0	8.71	0.733	2.6	47.02	
14	2.000	705.0	8.81	0.731			0.0003
15	4.000	775.0	9.69	0.715	4.0	30.04	
16	8.000	1087.0	13.59	0.641	6.8	16.75	
17	16.000	1445.0	18.06	0.556	4.8	21.54	
18	16.000	1571.0	19.64	0.526			0.0056
19	4.000	1516.0	18.95	0.539			
20	1.000	1440.0	18.00	0.557			

GZA GeoEnvironmental, Inc.

State of Maine - Department of Transportation
Laboratory Testing Summary Sheet

Route 103 Bridge
Town(s): York

MDOT Project Number:
GZA Project Number: 09.0025577.00

Boring Identification Number	Sample Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification Unified	AASHTO	Frost
BB-YYR-406	S-8			33-35			31.1					
BB-YYR-405B	S-3			14-16			30.8	46	21	CL	A-7	IV
BB-YYR-405B	S-5			24-26			40.2	43	22	CL	A-7	IV

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

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

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

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

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

GZA GeoEnvironmental, Inc.

State of Maine - Department of Transportation
Laboratory Testing Summary Sheet

Route 103 Bridge
Town(s): York

MDOT Project Number:

GZA Project Number: 09.0025577.00

Boring Identification Number	Sample Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification Unified	Classification AASHTO	Classification Frost
BB-YYR-401	S-1			1-3			4.2			GM	A-1-b	I
BB-YYR-406	S-1			1-3			4.6			SM	A-1-b	I
BB-YYR-406	S-2			4-6			11.6			SM	A-4	II
BB-YYR-401	S-3			9-11			12.9			SM	A-1-b	I
BB-YYR-401	S-11			49-51			18.6			SM	A-1-b	II
BB-YYR-401	S-14			64-66			15.3			SM	A-1-b	II
BB-YYR-401	S-18			84-86			8.4			GP-GM	A-1-a	0
BB-YYR-401	S-20			94-96			9.3			SM	A-4	II
BB-YYR-403	S-1			7-9			17.4			SM	A-1-b	II
BB-YYR-402	S-1			6-8			20.0			SM	A-1-b	II
BB-YYR-405A	S-1			6-8			41.3	45	22	CL	A-7	IV
BB-YYR-405A	S-3			16-18			24.2	25	16	CL	A-4	IV

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

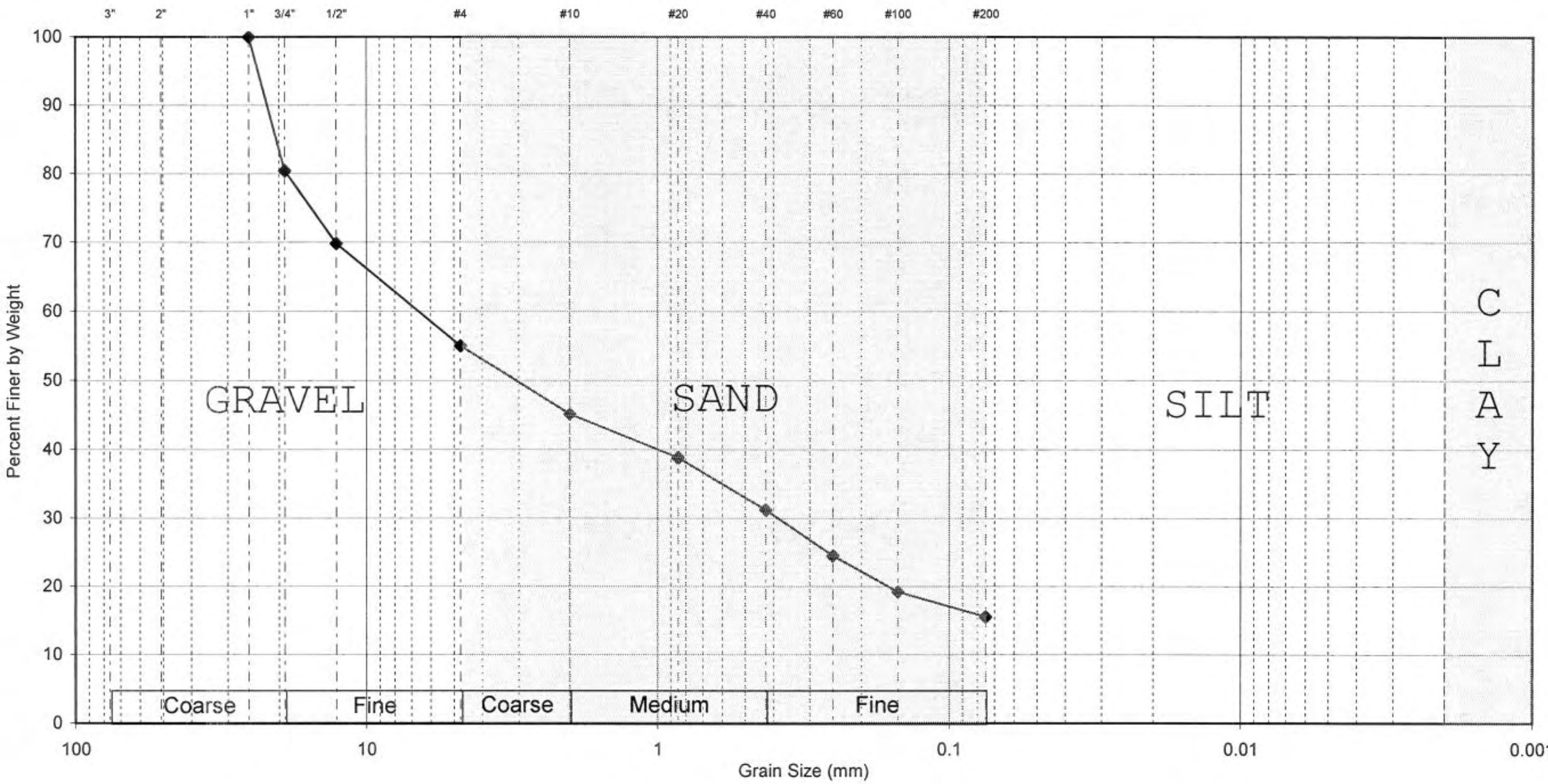
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

U.S. STANDARD SIEVE AND HYDROMETER

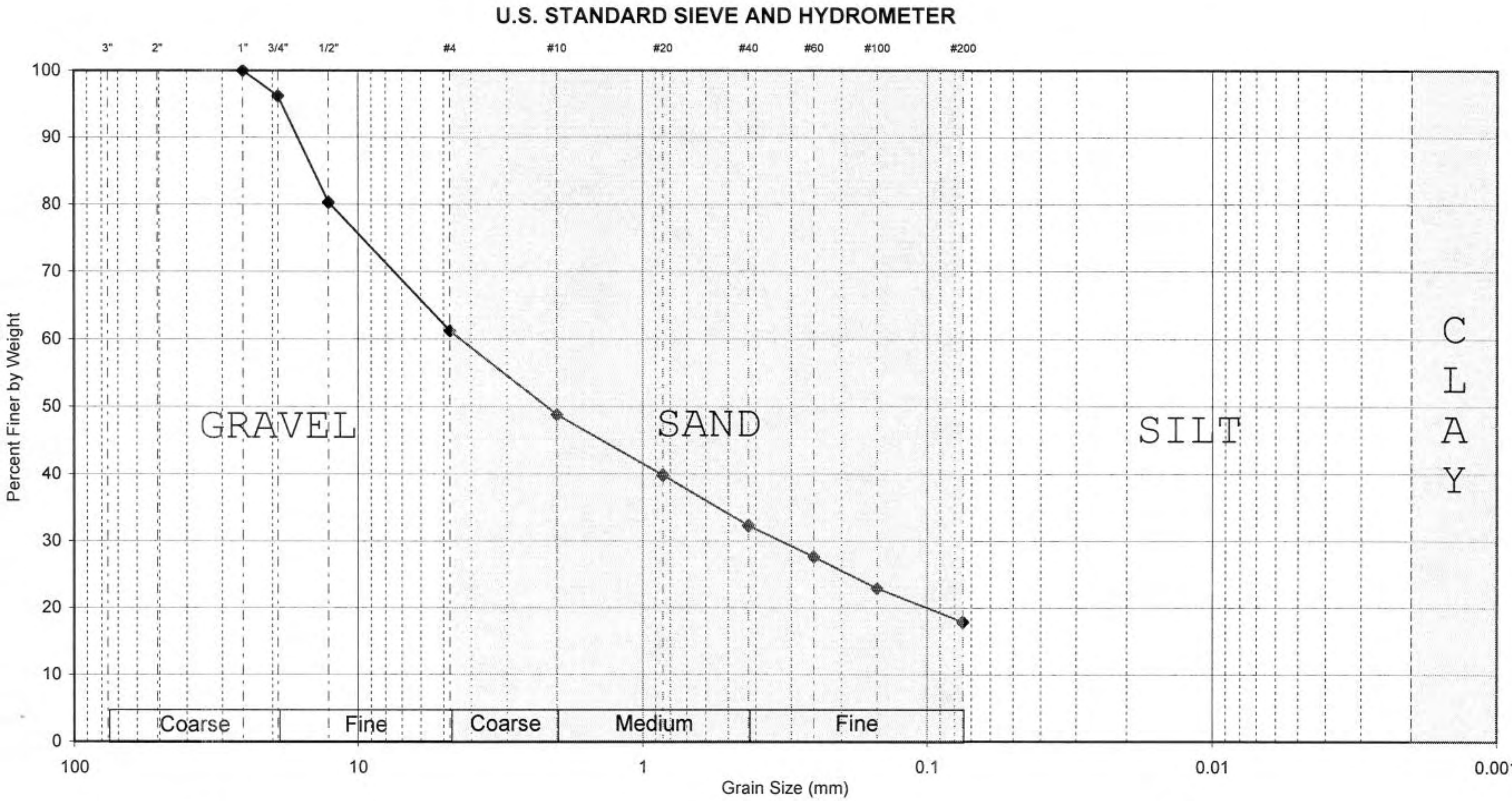


Gravel	Sand	Fines
45.0%	39.5%	15.6%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
16	BB-YYR-401	S-1	1-3'	Brown Silty Gravel with Sand (GM)	4.2			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by:	TJB	Date:	10/3/08
Reviewed by:	MBP	Date:	10/6/08



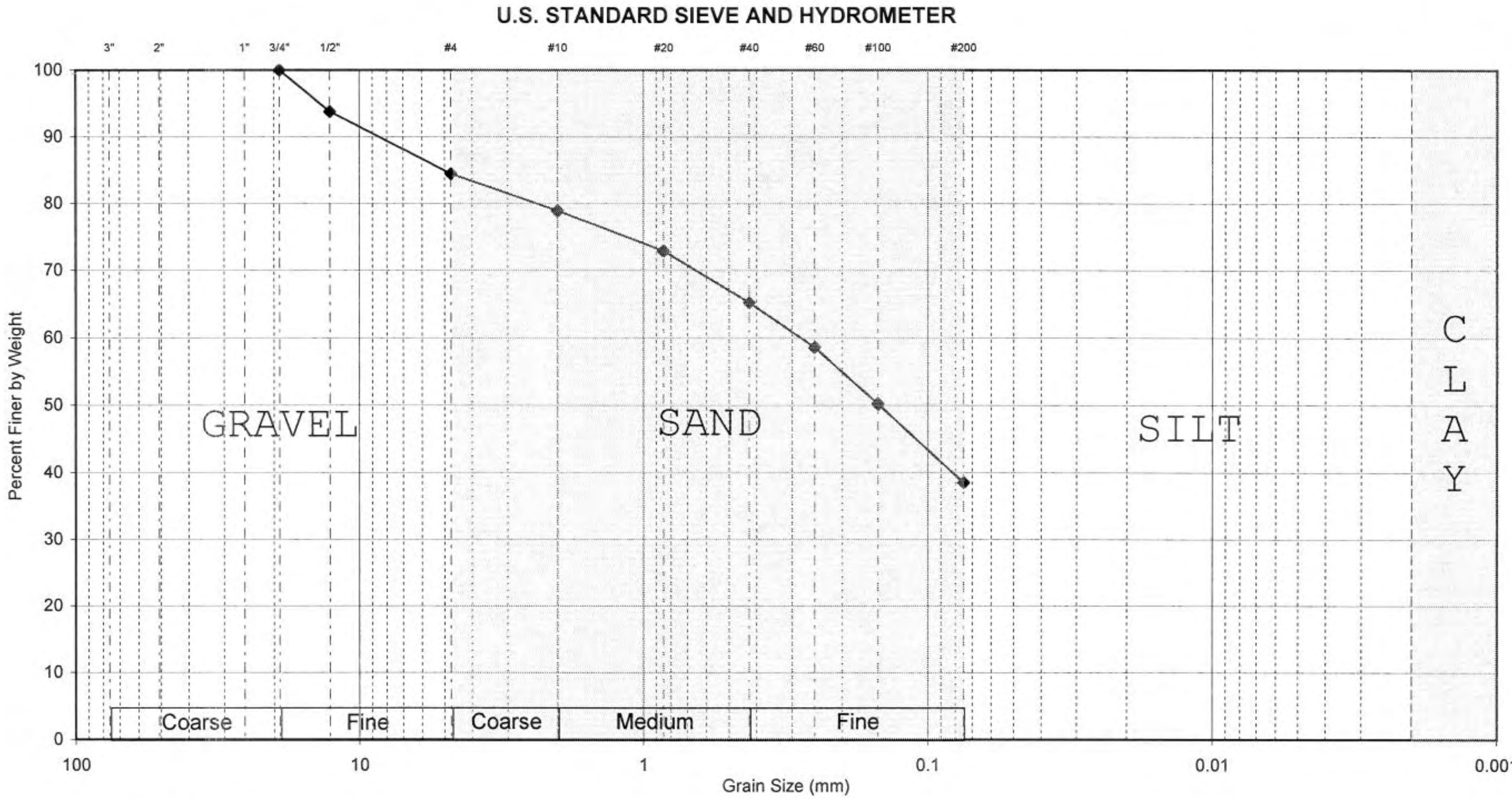
Gravel	Sand	Fines
38.8%	43.3%	17.9%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
17	BB-YYR-406	S-1	1-3'	Brown Silty Sand with Gravel (SM)	4.6			

GZA

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08

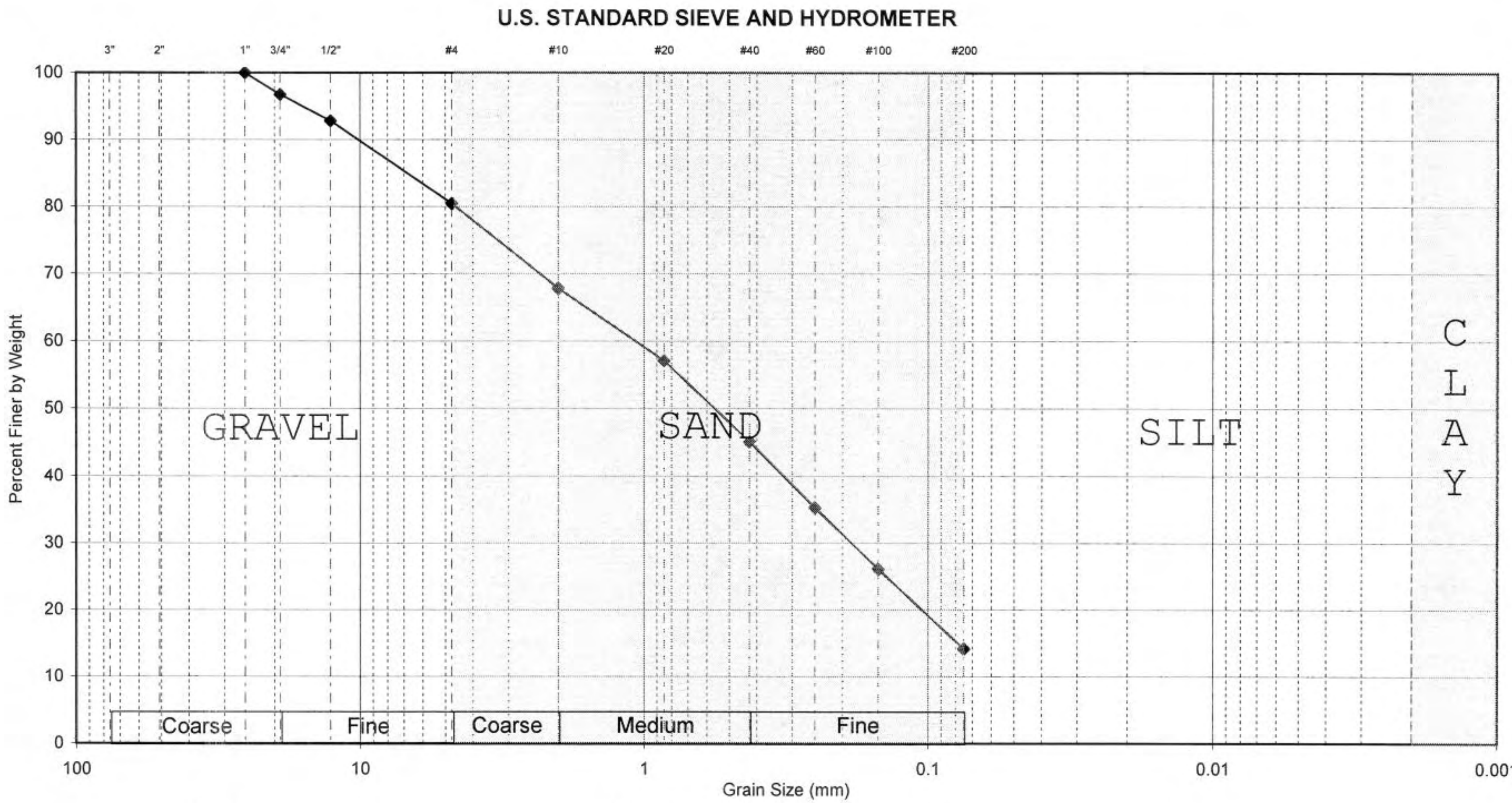


Gravel	Sand	Fines
15.6%	45.9%	38.5%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
18	BB-YYR-406	S-2	4-6'	Brown Silty Sand with Gravel (SM)	11.6			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08

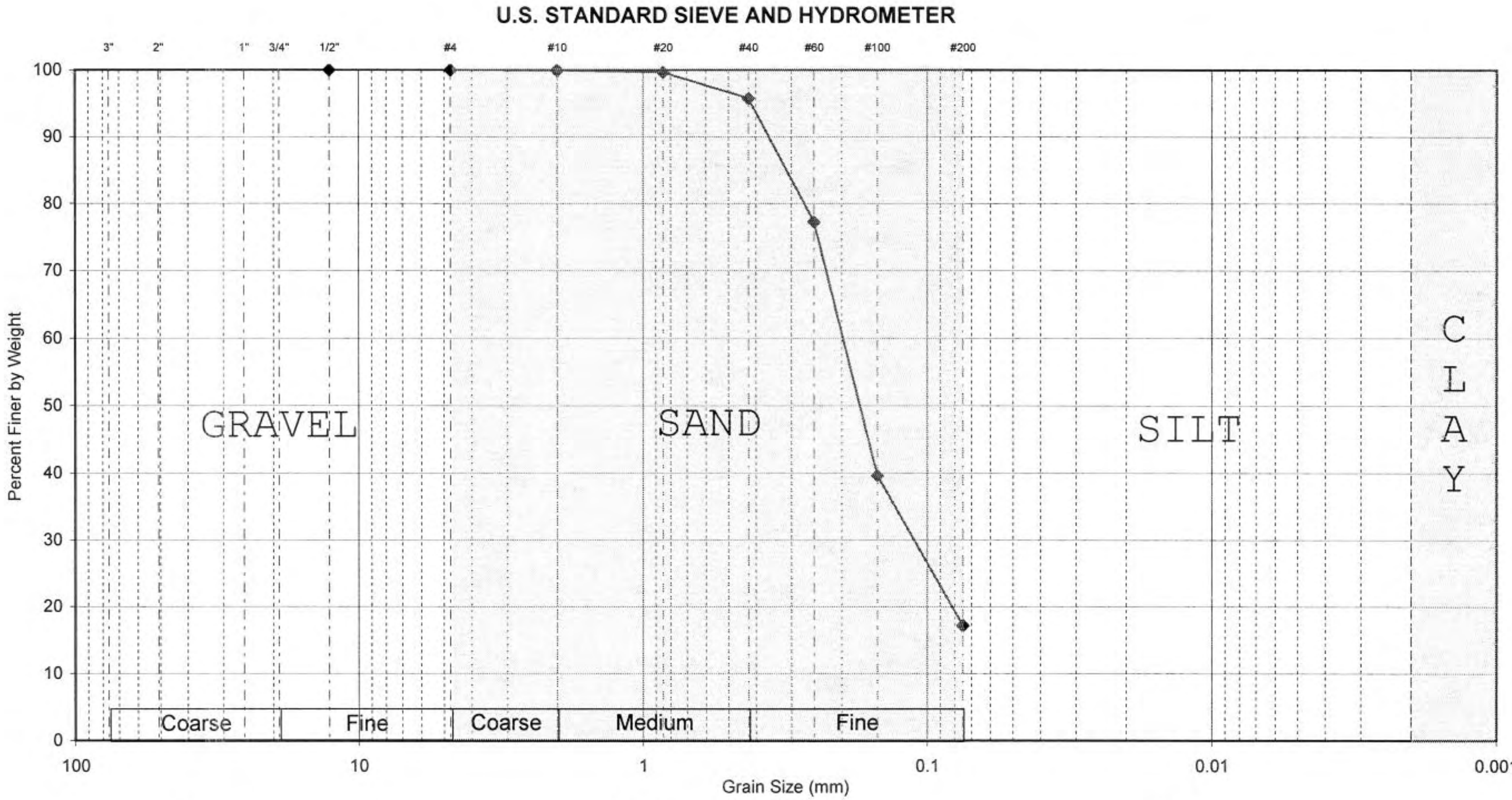


Gravel	Sand	Fines
19.6%	66.3%	14.1%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
19	BB-YYR-401	S-3	9-11'	Dark Brown Silty Sand with Gravel (SM)	12.9			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08

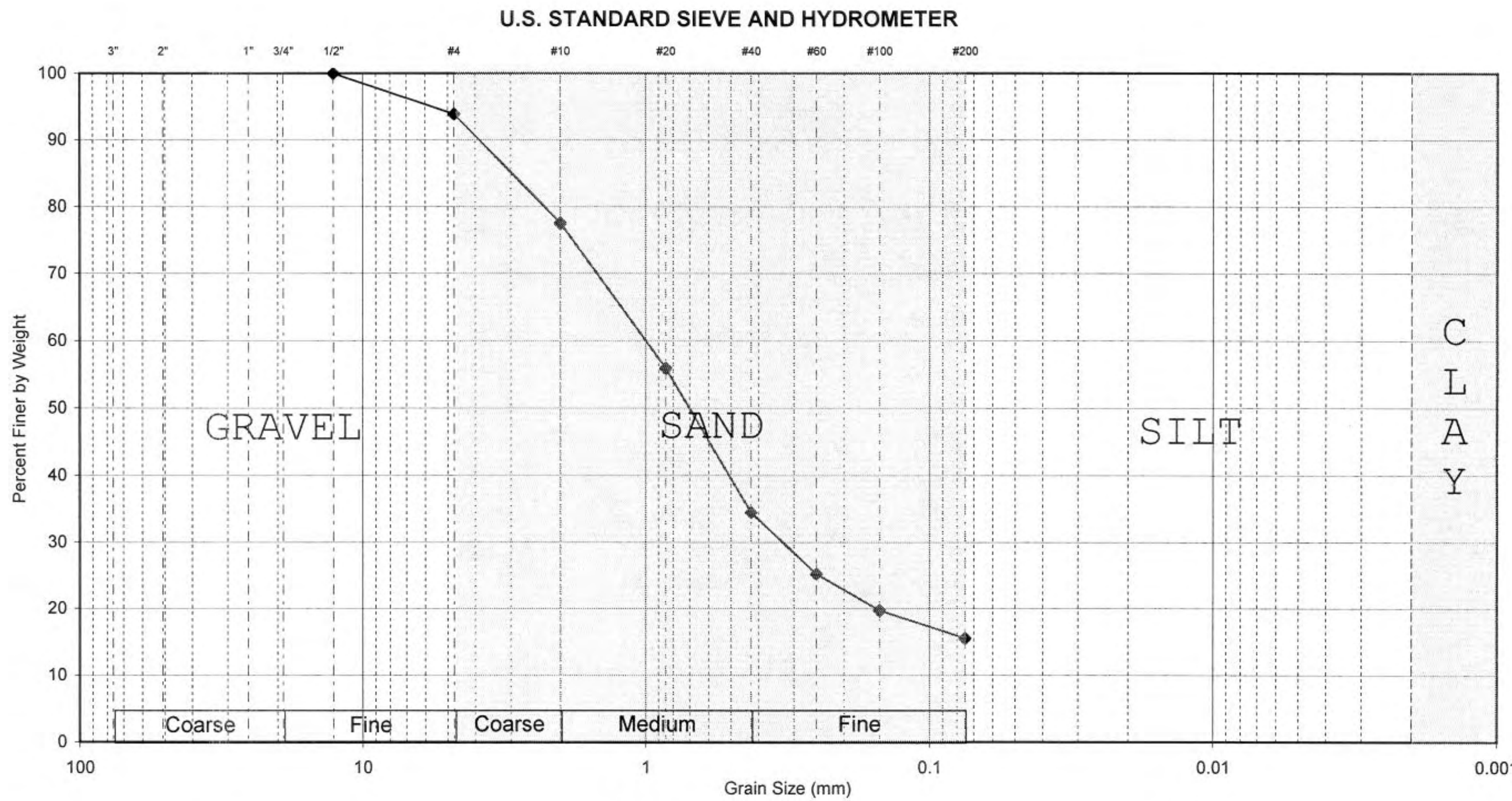


Gravel	Sand	Fines
0.0%	82.8%	17.2%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
20	BB-YYR-401	S-11	49-51'	Brown Silty Sand (SM)	18.6			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08



Gravel	Sand	Fines
6.1%	78.4%	15.5%

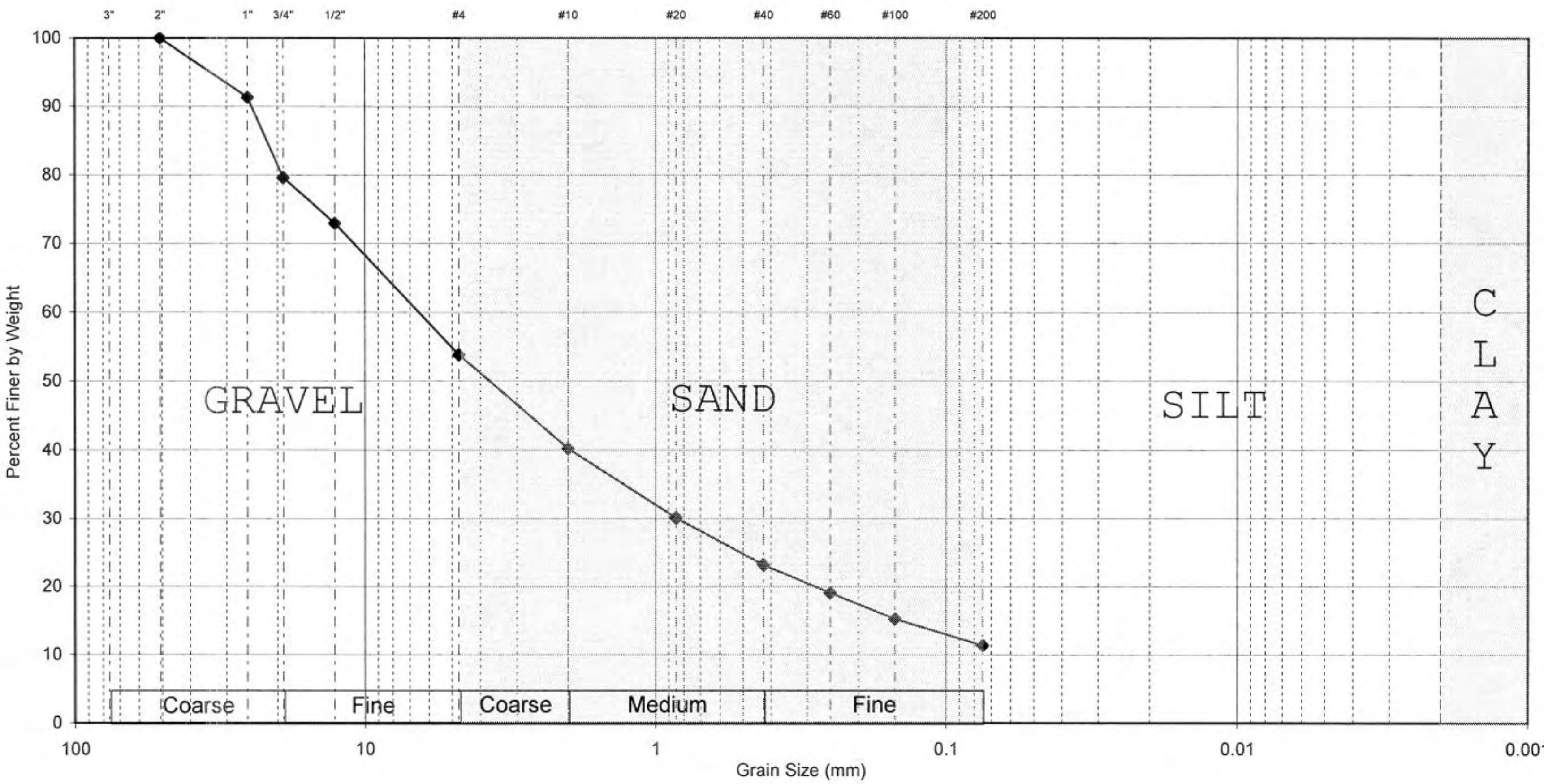
Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
21	BB-YYR-401	S-14	64-66'	Orange-Brown Silty Sand (SM)	15.3			

GZA

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by:	TJB	Date:	10/3/08
Reviewed by:	MBP	Date:	10/6/08

U.S. STANDARD SIEVE AND HYDROMETER

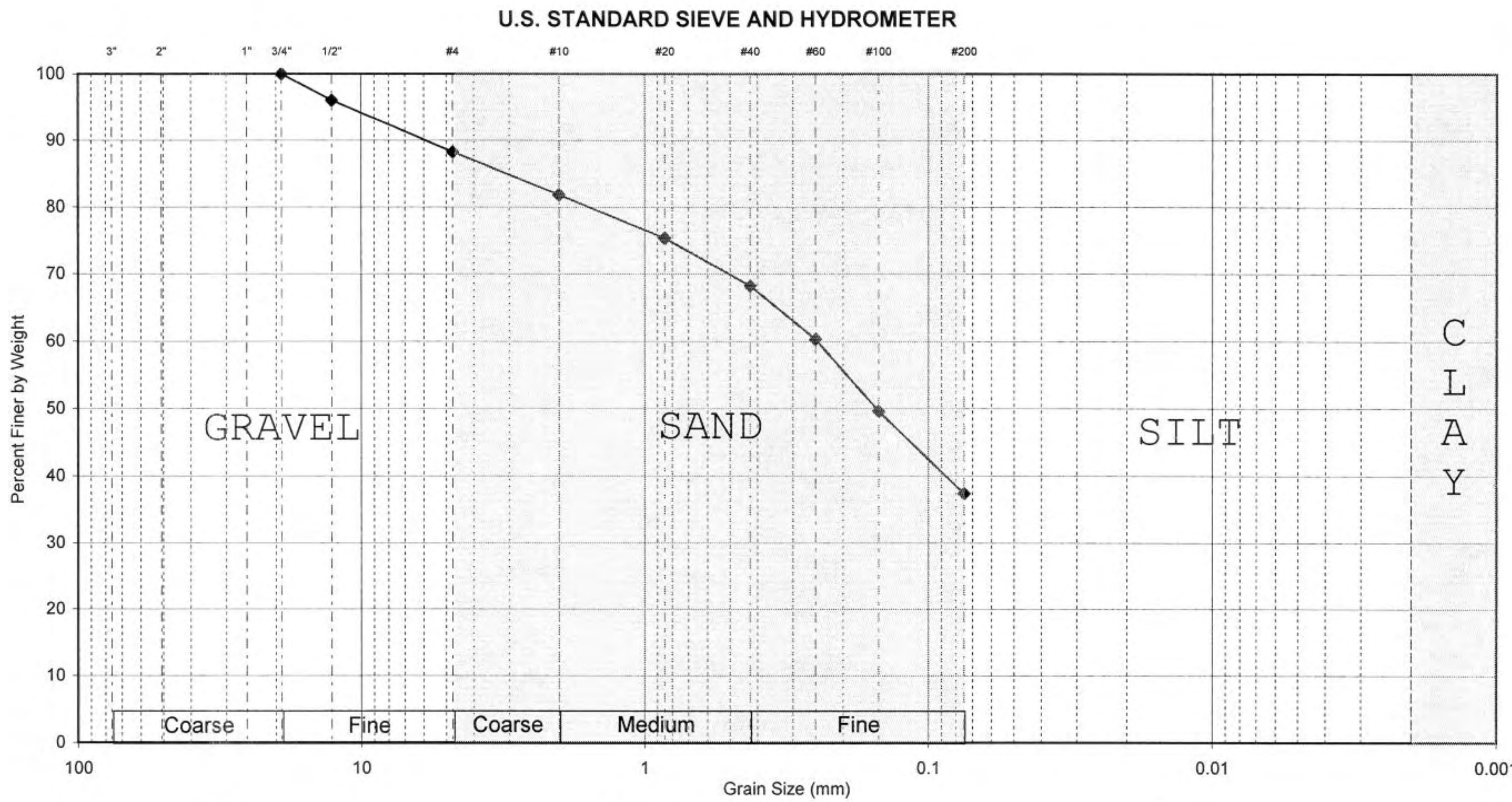


Gravel	Sand	Fines
46.1%	42.5%	11.3%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
22	BB-YYR-401	S-18	84-86'	Brown Poorly-graded Gravel with Silt & Sand (GP-GM)	8.4			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08



Gravel	Sand	Fines
11.8%	50.8%	37.4%

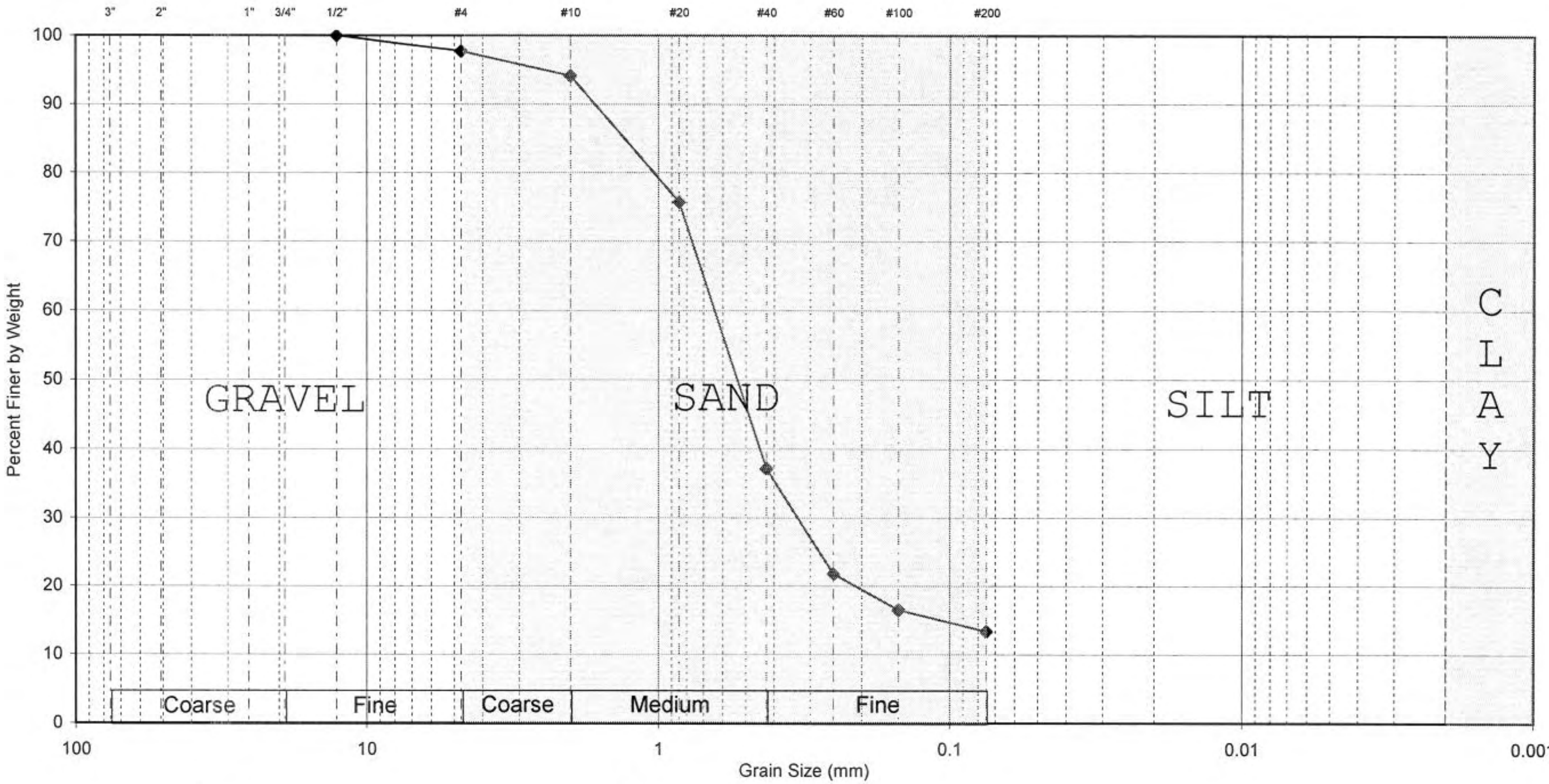
Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
23	BB-YYR-401	S-20	94-96'	Gray Silty Sand (SM)	9.3			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08

GZA

U.S. STANDARD SIEVE AND HYDROMETER

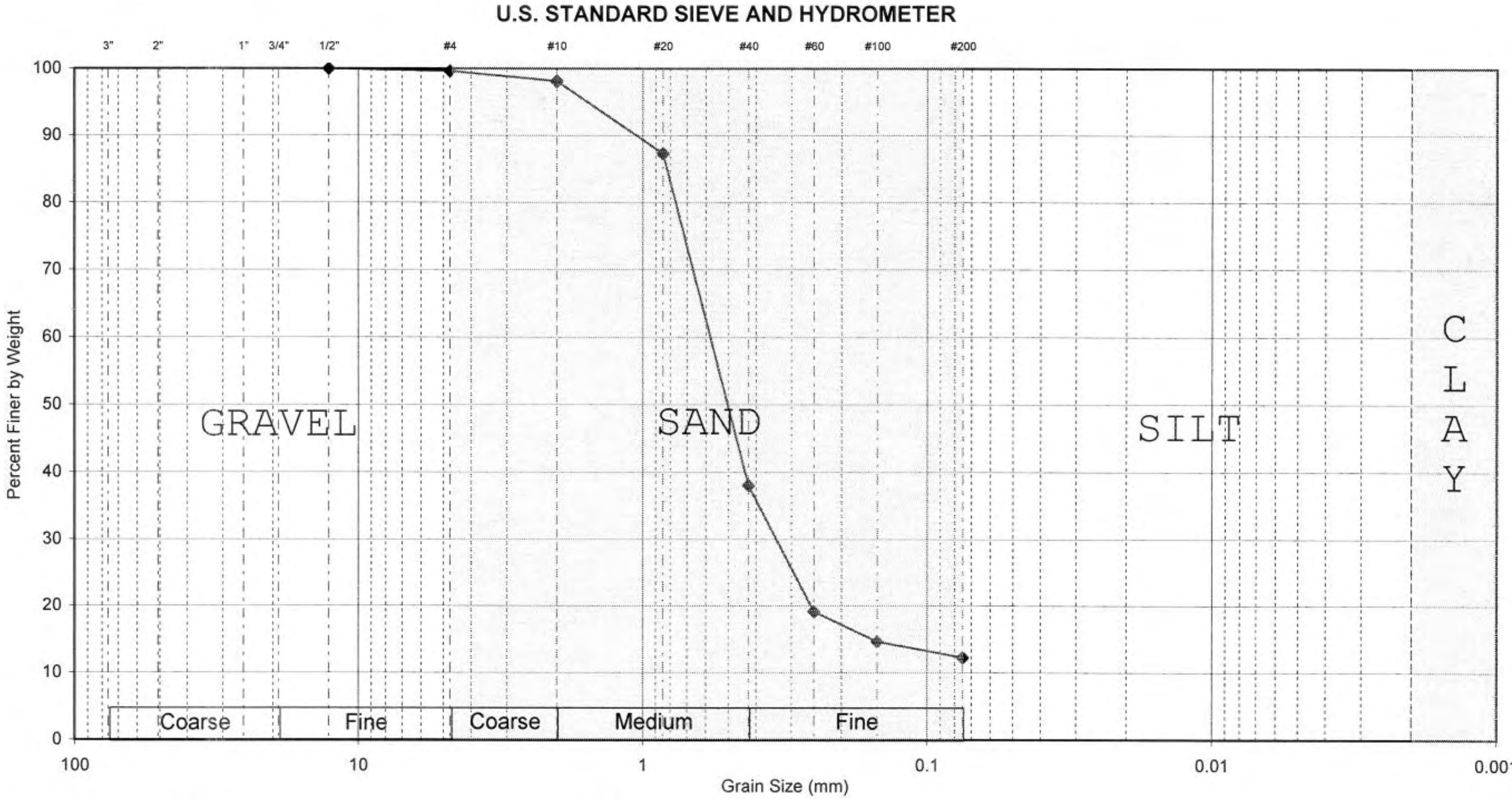


Gravel	Sand	Fines
2.3%	84.4%	13.4%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
24	BB-YYR-403	S-1	7-9'	Brown Silty Sand (SM)	17.4			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00

Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08



Gravel	Sand	Fines
0.4%	87.4%	12.3%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
25	BB-YYR-402	S-1	6-8'	Brown Silty Sand (SM)	20.0			

Route 103 Bridge
York, ME
GZA File # 09.0025577.00
Tested by: TJB Date: 10/3/08
Reviewed by: MBP Date: 10/6/08

LABORATORY TESTING DATA SHEET

Project Name Rte 103 Bridge
Project No. 09.0025577.00
Project Manager J. Tooley

Location York, ME
Assigned By J. Tooley
Report Date 10/13/2008

Reviewed By [signature]
Date Reviewed 10/14/08

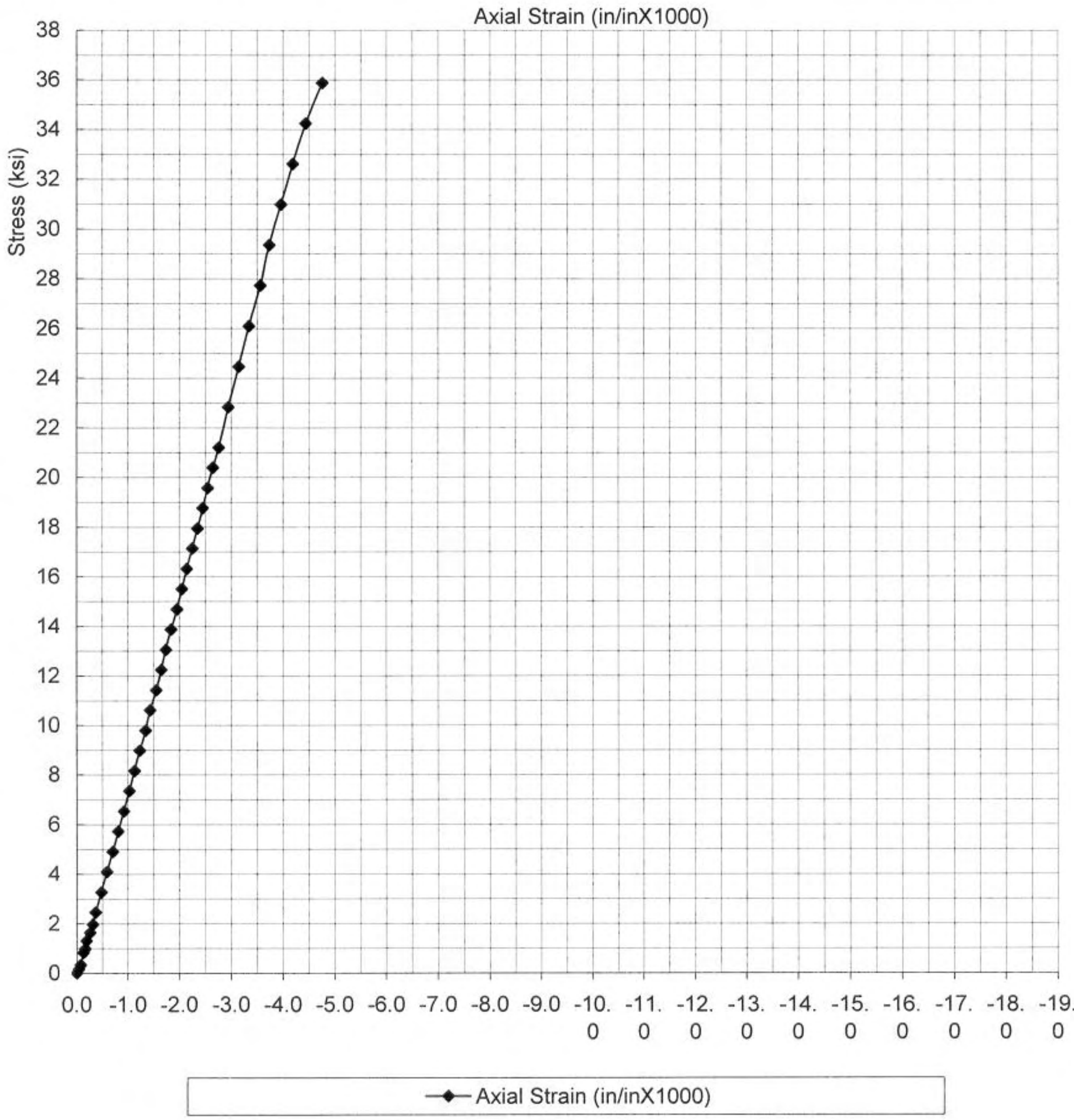
Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data: Water Content %	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests: (3) Other Tests	(4) Strength KSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	$\sigma\tau$ KSI	Is_{50} KSI	Rock Formation or Description or Remarks
BB-YYR-403	C-1	122.5-122.9	28		1.976	4.554	180.3			U	36.69	0.48		7.61				
BB-YYR-401	C-2	107.5-107.9	29		1.978	4.580	181.2			U	34.82	0.36		10.0				

(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) P=Petrographic PLD=Point Load (diametrical), PLA= Point Load (Axial) RST= Splitting Tensile U= Unconfined Compressive Strength
(4) Taken at Peak Deviator Stress

(5) Strain at Deviator Stress
(6) Represents Confining Stress on Triaxial Tests
(7) Represents Secant Modulus at 50% of Total Failure Stress
(8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

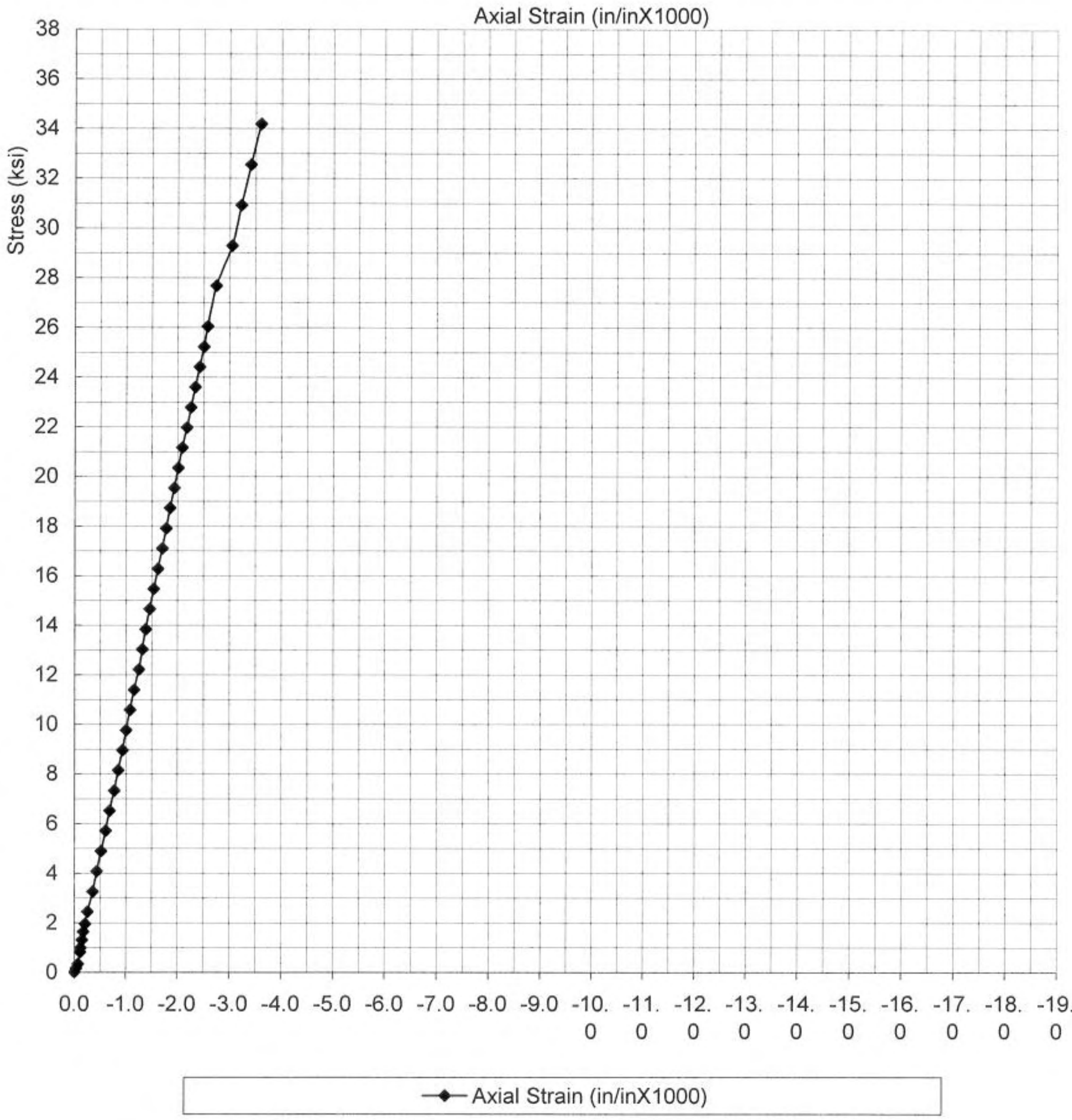
Rte 103 Bridge
York, ME



Rock Testing

Boring No.	BB-YYR-403	File No.	09.0025577.00
Sample No.	C-1	Date:	10/2/2008
Depth:	122.5-122.9'	Test No.	U 28

Rte 103 Bridge
York, ME



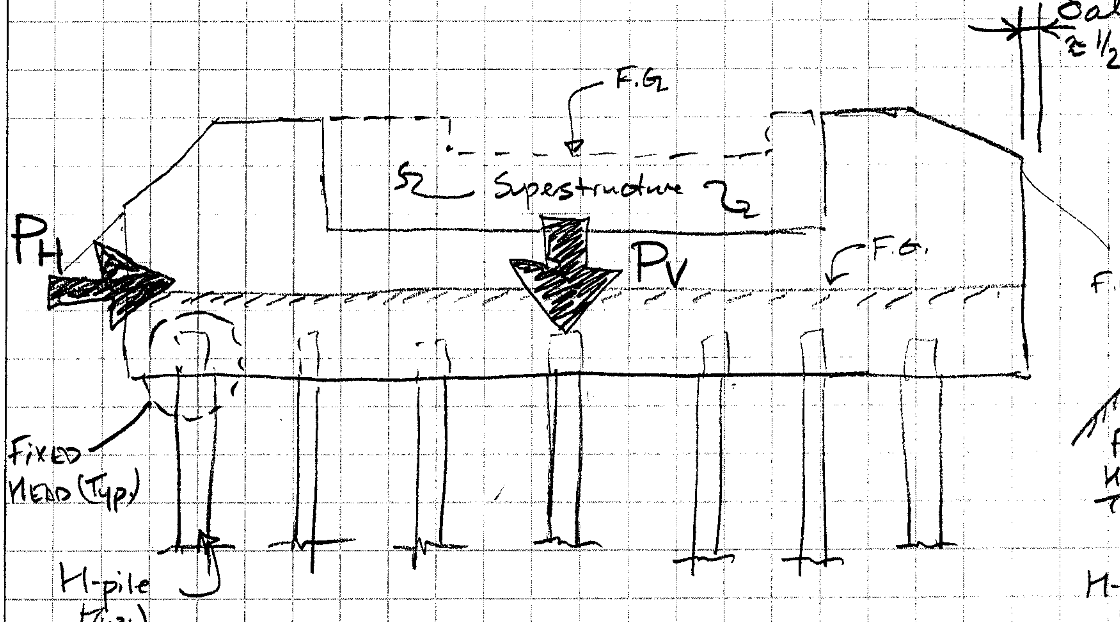
Rock Testing

Boring No.	BB-YYR-401	File No.	09.0025577.00
Sample No.	C-2	Date:	10/2/2008
Depth:	107.5-107.9'	Test No.	U 29

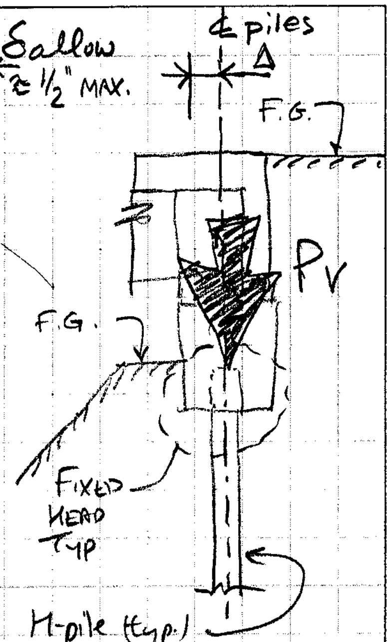
APPENDIX D – FACTORED LOADS PROVIDED BY VHB

VHB Computations

Project: YORK, ME
Project # 51979.10
Location: NEW BRIDGE
Sheet 2 of 2
Calculated by: SMH
Date: 10/21/08
Checked by: LSG
Date: 10/23/08
Title PRELIM SUBSTR. LOADS @ ABUTMENTS



ABUTMENT ELEVATION



INTEG. ABUTMENT SECTION

	P_V*	P_H	Δ
SERVICE I	865^k	50^k	0.9"
STRENGTH I	1210^k	ϕ	0.9"
STRENGTH III	800^k	110^k	0.9"

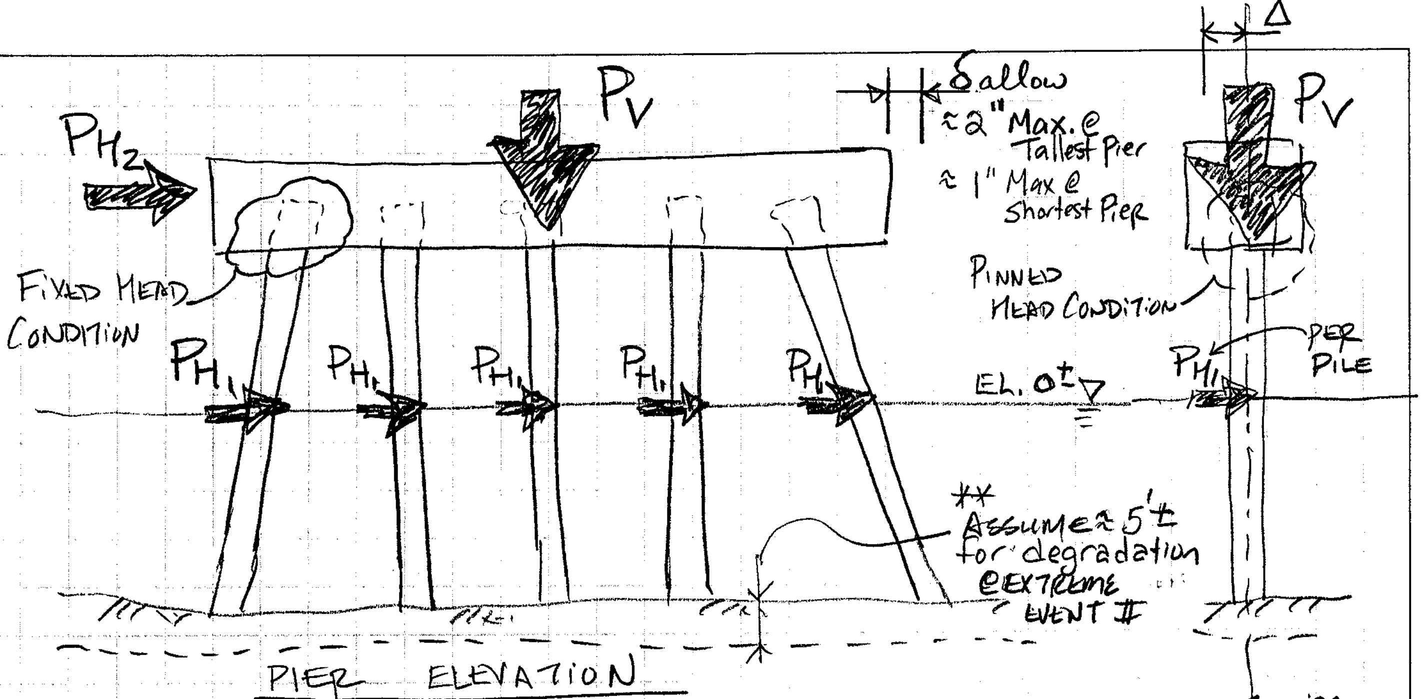
* LOAD DOES NOT INCLUDE PILE SELF WT or DOWNDRAG. INCLUDES ENTIRE SUPERSTRUCTURE LOAD PLUS SELF WT OF ABUTMENT.

$1.0\,DC + 1.0\,DW + 1.0(LL) = (385 + 210) + 35 + 235 = 865^k$

$1.25\,DC + 1.5\,DW + 1.75(LL) = 1.25(595) + 1.5(35) + 1.75(235) = 1210^k$

$1.25\,DC + 1.5\,DW \cong 800^k$

Project: YORK, ME
Project # 51979.10
Location: NEW BRIDGE
Sheet 2 of 2
Calculated by: SMH
Date: 10/21/08
Checked by: LSG
Date: 10/23/08
Title PRELIM SUBSTR. LOADS @ PIER



	P_V*	P_{H_1}	P_{H_2}	Δ
SERVICE I	1620k	5k	15k	0.7"
STRENGTH I	2255k	5k	ϕ	0.7"
STRENGTH III	1540k	5k	35k	0.7"
EXTREME I	1745	5k	ϕ	ϕ

Δ = FACTORED DISP. FROM THERMAL MOVEMENT @ WORSE CASE PIER

** LOCAL PIER + CONTRACTION SCOUR NOT INCLUDED PER LRFD C3.4.1

* DOES NOT INCLUDE PILE OR PILE CASING SELF WT. LOAD INCLUDES ENTIRE SUPERSTRUCTURE LOAD PLUS WEIGHT OF PILE CAP

$1.0\,DC + 1.0\,DW + 1.0(LL+I) = (1000+110)+100+410 = 1620^k$

$1.0\,WA = 1.0 \times 5^k = 5^k$

$1.25\,DC + 1.5\,DW + 1.75\,(LL+I) = 1.25(1110)+1.5(100)+1.75(410) = 2255^k$ (1540^k w/o LL+I) (1745^k with 0.5LL)

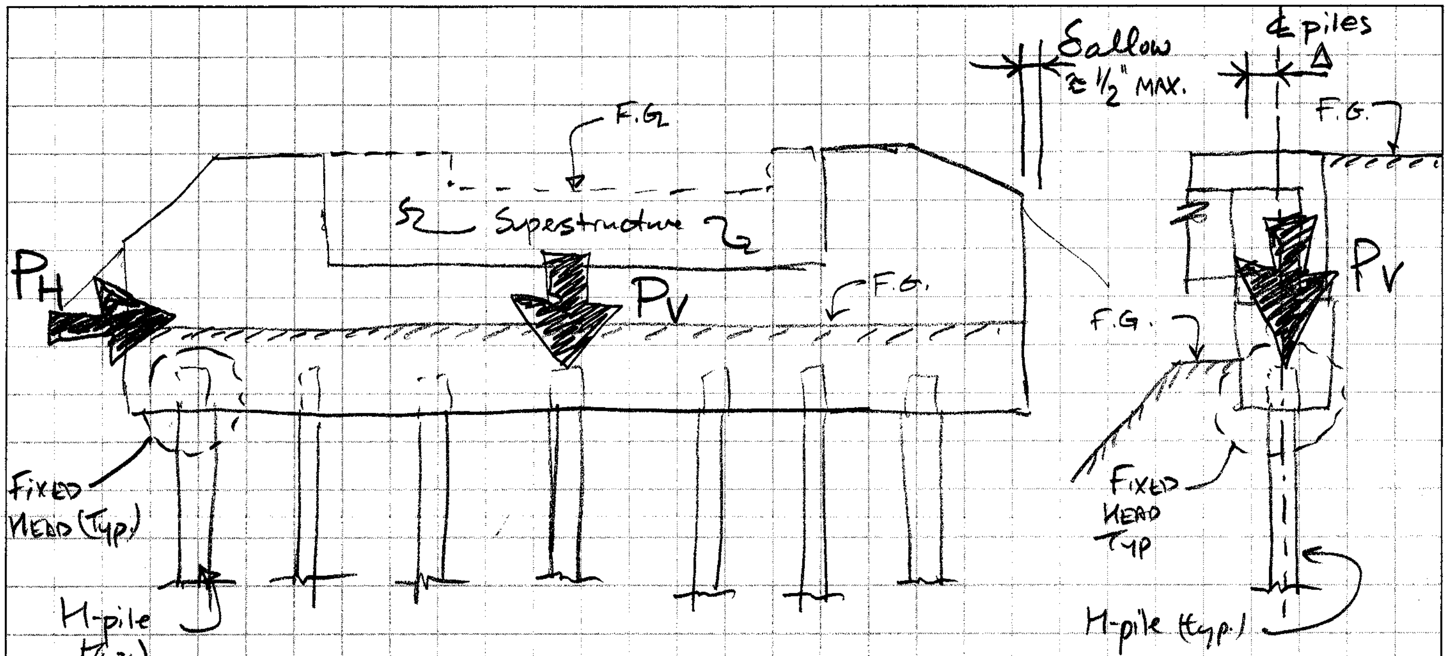
$0.3\,WS + 1.0\,WL = 0.3(24) + (8) = 15^k$

$1.4\,WS = 1.4(24) = 35^k$

VHB Computations

Project: YORK, ME
Location: NEW BRIDGE
Calculated by: SMH
Checked by: LSG
Project # 51979.10
Sheet 2 of 2
Date: 10/21/08
Date: 10/23/08
Title PRELIM SUBSTR. LOADS @ ABUTMENTS

REVISIONS: △1 LATERAL LOAD 11/24/08



ABUTMENT ELEVATION

INTEG. ABUTMENT SECTION

	P_V*	P_H △	△
SERVICE I	865^k	~~50^k~~ 6^k	0.9"
STRENGTH I	1210^k	ϕ	0.9"
STRENGTH III	800^k	~~110^k~~ 13^k	0.9"

△1 PER SMH'S CONVERSATION W/ GEOTECH 11/24/08, UPDATE TRANSVERSE WIND LOADING FOR THE TRIBUTARY WIDTH OF THE END SPANS AS OPPOSED TO ONE HALF OF BRIDGE
– LSG 11/24/08 SMH 11/24/08

~~PIERS W/ BATTERED END PILES PROVIDE SIGNIFICANT LATERAL LOAD RESISTANCE @ PIERS~~

* LOAD DOES NOT INCLUDE PILE SELF WT or DOWNDRAG. INCLUDES ENTIRE SUPERSTRUCTURE LOAD PLUS SELF WT OF ABUTMENT.

$1.0DC + 1.0DW + 1.0(LL) = (385 + 210) + 35 + 235 = 865^k$

$1.25DC + 1.5DW + 1.75(LL) = 1.25(595) + 1.5(35) + 1.75(235) = 1210^k$

$1.25DC + 1.5DW \cong 800^k$

APPENDIX E – CALCULATIONS

Geotechnical Design Calculations
Route 103 New Bridge
York, Maine
GZA File No. 09.0025577.00
Maine DOT PIN 15110.00

Prepared for:
Ms. Laura Krusinski, P.E.
Maine Department of Transportation
State House Station 16
Augusta, Maine 04333-0016

STATE OF MAINE
CHRISTOPHER L. SNOW
No. 7275
LICENSED
PROFESSIONAL ENGINEER
3-30-09

Prepared by;
Christopher l. Snow, P.E.
Senior Project Manager
GZA GeoEnvironmental, Inc.
Portland, Maine

CALCULATIONS INDEX

Project: NewBridge, Route 103
GZA Project No. 09.0025577.00

Abutment Axial Analysis

Steel H-Pile:

Factored Load Calculation

Factored capacity = required ultimate
Resistance factor of = 0.65

$N_{piles} := 5$

Load $P_v := 1210\text{kip}$

Load Per Pile $P_{pile} := \dfrac{P_v}{N_{piles}}$ $P_{pile} = 242\text{ kip}$

Factored Load $P_{f.pile} := \dfrac{P_{pile}}{0.65}$ $P_{f.pile} = 372\text{ kip}$

Profile A - controlling profile (longest pile length)

WEAP Analysis

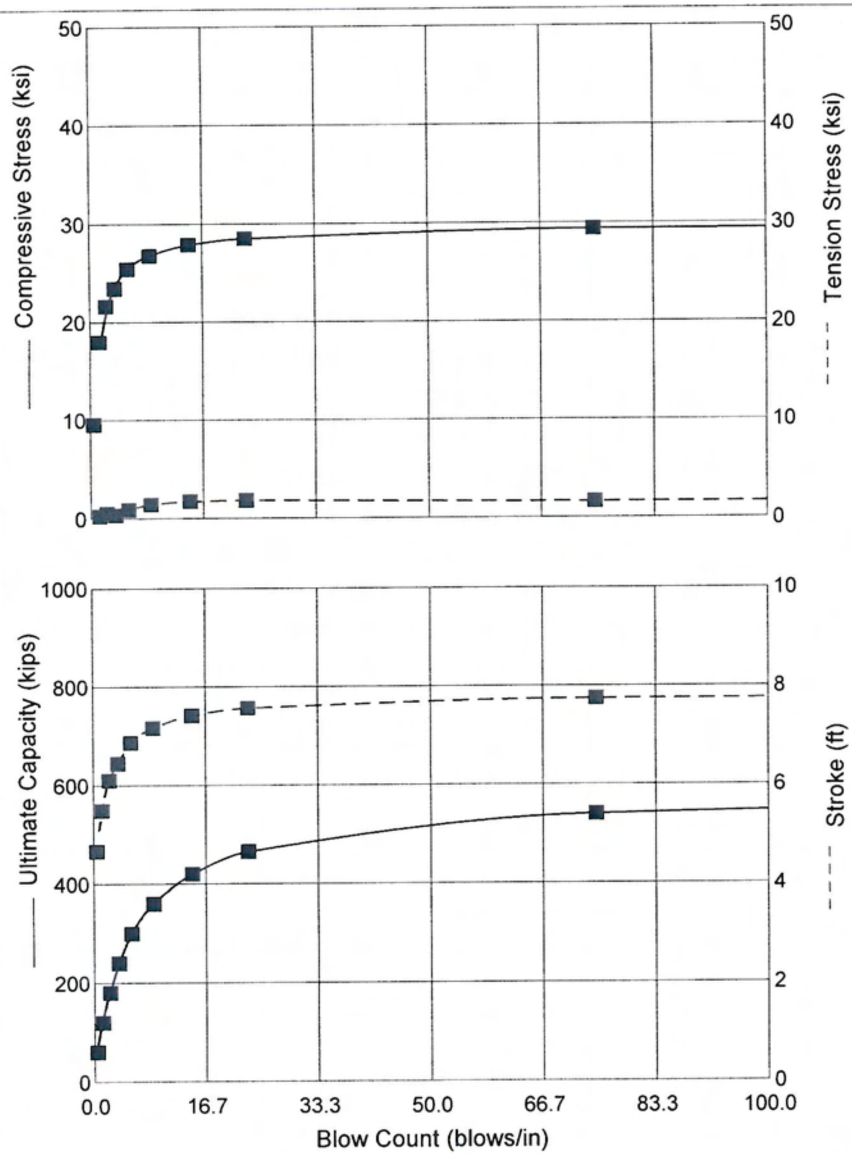
Run WEAP analysis based on half-plugged condition with 5ft of stick up and skin friction distribution from S-Pile (pile length=60ft)
Skin friction approximately 30% of total capacity near top of till layer.

Section Area = 0.51 ft^2 (73.8 in^2)-Half Plugged

File	Hammer	Erated (ft-k)	Side Fric %	Quake side	Quake toe	Blow/in	Req ULT (K)	Driving Stress (ksi)	
Abut-1A	MKT DE 50B	42.5	30	0.1	0.1	10	372	27	OK
Abut-2A	MKT DE 70B	59.5	30	0.1	0.1	5	372	33	possible too big

GZA Geo Environmental, Inc.
Route 103 Bridge, Abutment

23-Jan-2009
GRLWEAP (TM) Version 2005

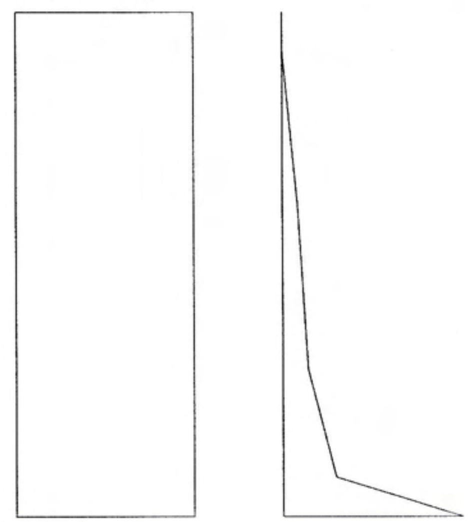


MKT DE 50B

Efficiency	0.800
Helmet	1.34 kips
Hammer Cushion	34790 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.120 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	65.00 ft
Pile Penetration	60.00 ft
Pile Top Area	21.80 in2

Pile Model

Skin Friction
Distribution



Res. Shaft = 30 %
(Proportional)

GZA Geo Environmental, Inc.
Route 103 Bridge, Abutment

23-Jan-2009
GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
60.0	9.55	0.00	0.5	4.66	14.88
120.0	17.98	0.27	1.3	5.49	12.52
180.0	21.61	0.55	2.4	6.10	12.33
240.0	23.44	0.35	3.7	6.44	12.39
300.0	25.41	0.90	5.6	6.86	13.30
360.0	26.77	1.44	8.9	7.16	14.07
420.0	27.87	1.75	14.7	7.41	14.76
465.0	28.51	1.85	23.0	7.56	15.08
540.0	29.41	1.67	74.6	7.74	15.48
600.0	29.80	1.74	365.5	7.81	15.59

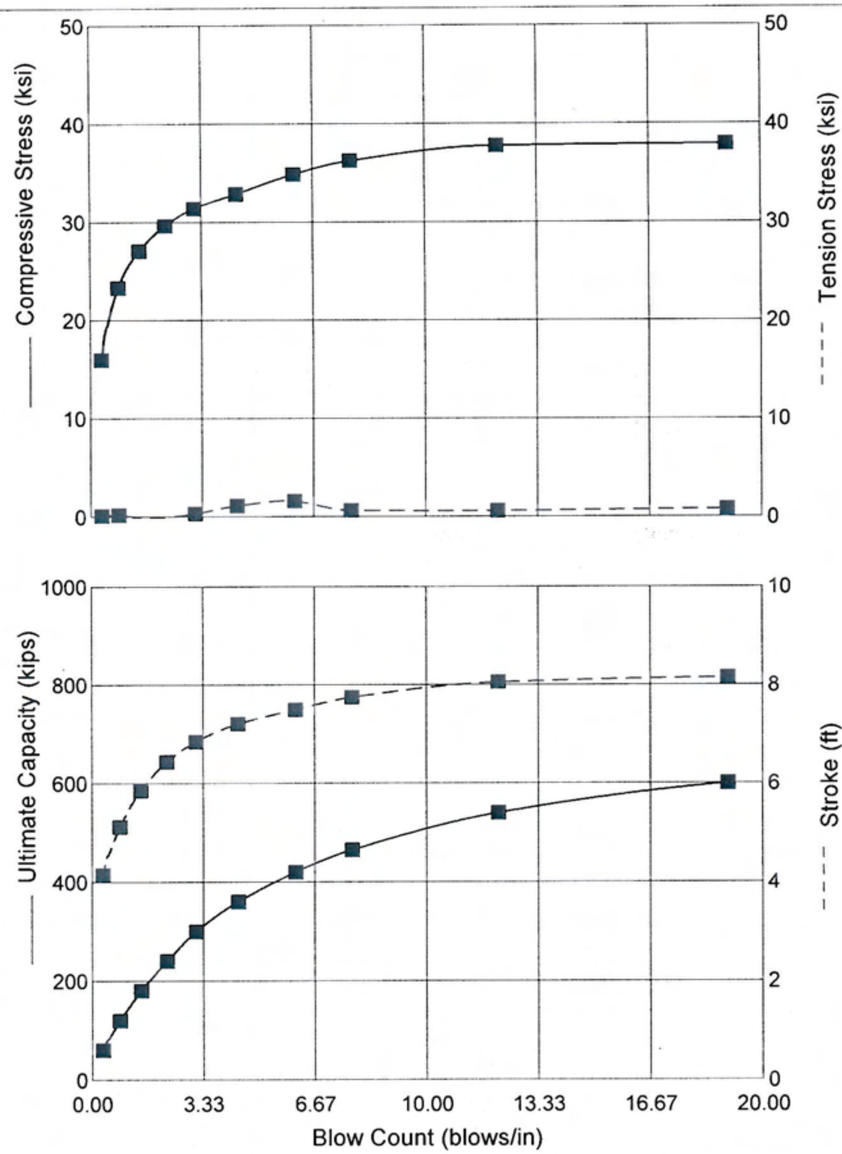
Interpolate for 372 K

$$\frac{420-372}{420-360} = \frac{27.87-x}{27.87-26.77} \qquad x = 26.99$$

$$\frac{420-372}{420-360} = \frac{14.7-x}{14.7-8.9} \qquad x = 10.06$$

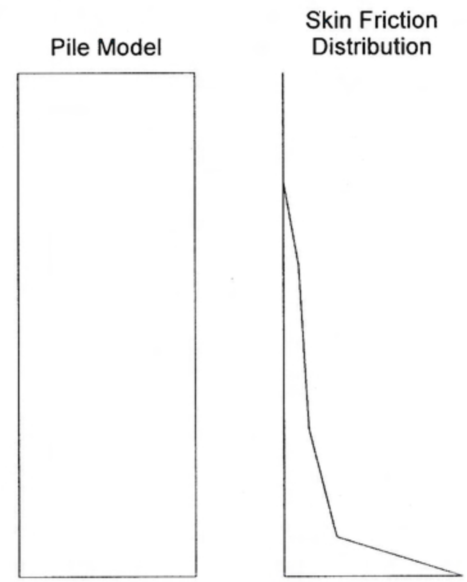
GZA Geo Environmental, Inc.
Route 103 Bridge, Abutment

14-Nov-2008
GRLWEAP (TM) Version 2005



MKT DE 70B

Efficiency	0.800
Helmet	1.34 kips
Hammer Cushion	34790 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.120 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	65.00 ft
Pile Penetration	60.00 ft
Pile Top Area	21.80 in2



Res. Shaft = 30 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
60.0	15.96	0.11	0.3	4.15	26.06
120.0	23.31	0.20	0.8	5.12	22.03
180.0	27.05	0.00	1.5	5.85	20.70
240.0	29.64	0.00	2.3	6.43	20.61
300.0	31.39	0.32	3.1	6.84	21.32
360.0	32.86	1.12	4.4	7.20	22.31
420.0	34.86	1.61	6.1	7.49	23.31
465.0	36.24	0.66	7.8	7.74	24.21
540.0	37.78	0.62	12.2	8.05	25.29
600.0	37.92	0.82	19.0	8.15	25.58

Interpolate for 372 K

$$\frac{420-372}{420-360} = \frac{34.86 - x}{34.86 - 32.86} \qquad x = 33.26$$

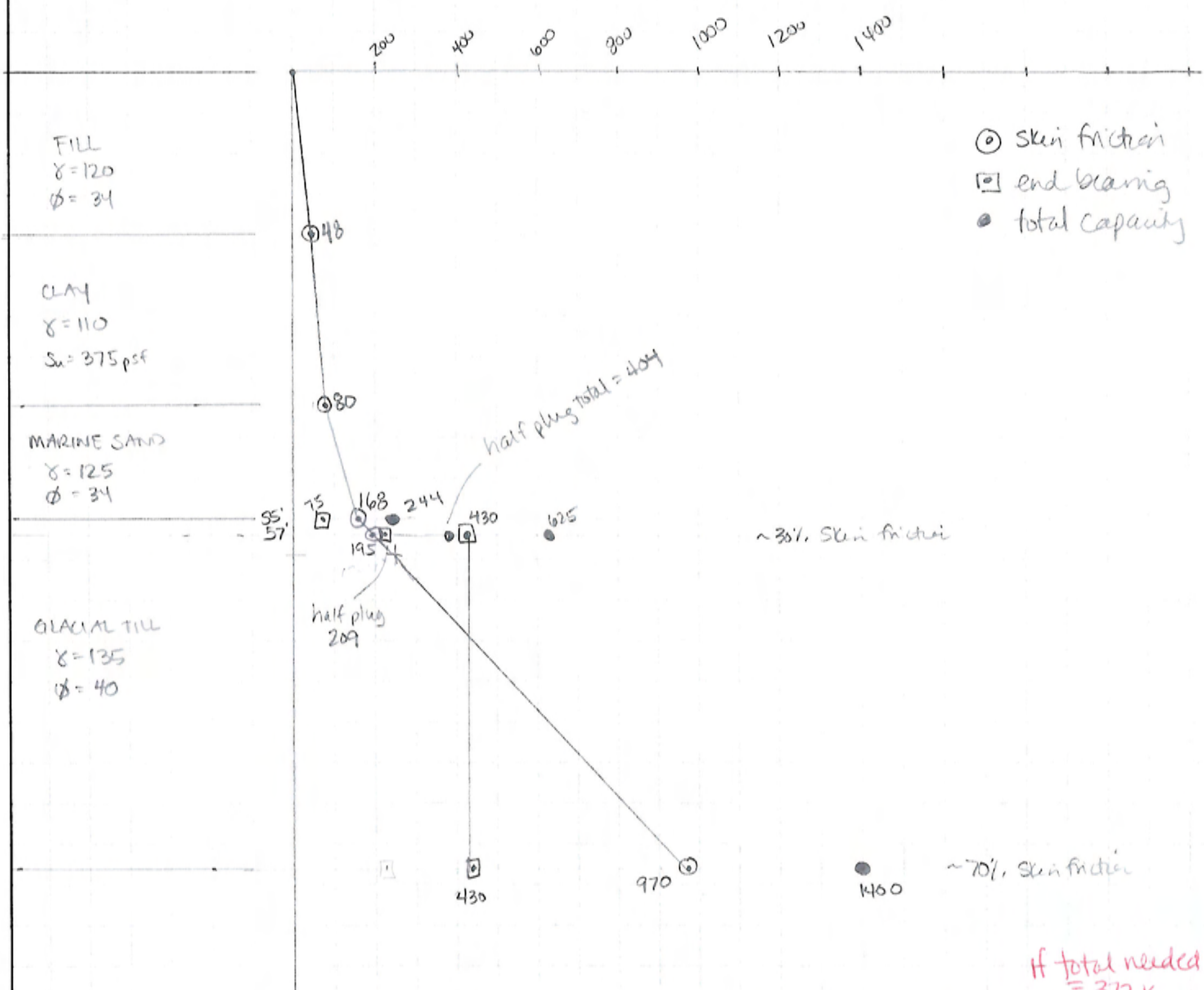
$$\frac{420-372}{420-360} = \frac{6.1 - x}{6.1 - 4.4} \qquad x = 4.7$$

GZA GeoEnvironmental, Inc.
Four Free Street
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
http://www.gza.com

Engineers and Scientists

JOB ______
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______

ABUTMENT PROFILE



If total needed = 372k

If ASSUME ½ plug is also linear EB = 209 ∴ skin = 256
This occurs around 60' b.g.s.

EB = 209 ∴ skin = 163
~ top of till @ ~55'

∴ Pile should reach capacity between 55 and 60'

Still valid!

#1

ÚÄÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 New Bridg	Client	: ME DOT
File Name	: rte103abut	Project Manager	: CLS
Date	: 11/13/10	Computed by	: jrt

Depth of Top of Pile = 0.00 ft. Pile length = 98.00 ft.
Depth to Water Table = 10.00 ft.
Type of Pile = H Pile
HP 12x74

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	1200.00	34.00	--	4.06
2	Cohesive	21.00	2275.80	---	--	4.06
3	Cohesionless	14.00	3213.80	34.00	--	4.06
4	Cohesionless	43.00	5212.90	40.00	--	4.06

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	26.79	47.44
2	Cohesive	375.00	375.00	----	-----	31.96
3	Cohesionless	--	--------	----	26.79	88.93
4	Cohesionless	--	--------	----	31.51	802.26

Total Side Friction : 970.59

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
6773.80	40.00	-----	1.03	160.00	836.39

Limiting End Bearing Resistance : 429.69

Ultimate Static Pile Capacity : 1400.29

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

#2

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 New Bridg	Client	: ME DOT
File Name	: rte103abut	Project Manager	: CLS
Date	: 11/13/10	Computed by	: jrt

Depth of Top of Pile = 0.00 ft. Pile length = 57.00 ft.
Depth to Water Table = 10.00 ft.
Type of Pile = H Pile
HP 12x74

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	1200.00	34.00	--	4.06
2	Cohesive	21.00	2275.80	---	--	4.06
3	Cohesionless	14.00	3213.80	34.00	--	4.06
4	Cohesionless	2.00	3724.60	40.00	--	4.06

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	26.80	47.49
2	Cohesive	375.00	375.00	----	-----	31.95
3	Cohesionless	--	--------	----	26.80	89.02
4	Cohesionless	--	--------	----	31.52	26.70

Total Side Friction : 195.16

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
3797.20	40.00	-----	1.03	160.00	468.86

Limiting End Bearing Resistance : 429.69

Ultimate Static Pile Capacity : 624.85

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

#3

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 New Bridg	Client	: ME DOT
File Name	: rte103abut	Project Manager	: CLS
Date	: 11/13/10	Computed by	: jrt

Depth of Top of Pile = 0.00 ft. Pile length = 55.00 ft.
Depth to Water Table = 10.00 ft.
Type of Pile = H Pile
HP 12x74

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	1200.00	34.00	--	4.06
2	Cohesive	21.00	2275.80	---	--	4.06
3	Cohesionless	14.00	3213.80	34.00	--	4.06

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	26.79	47.44
2	Cohesive	375.00	375.00	----	-----	31.96
3	Cohesionless	--	--------	----	26.79	88.93

Total Side Friction : 168.33

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
3652.00	34.00	-----	1.03	55.60	131.04

Limiting End Bearing Resistance : 75.65

Ultimate Static Pile Capacity : 243.98

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

#4

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 New Bridg	Client	: ME DOT
File Name	: rte103abut	Project Manager	: CLS
Date	: 11/13/10	Computed by	: jrt

Depth of Top of Pile = 0.00 ft. Pile length = 57.00 ft.
Depth to Water Table = 10.00 ft.
Type of Pile = H Pile
HP 12x74

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	1200.00	34.00	--	4.06
2	Cohesive	21.00	2275.80	---	--	4.06
3	Cohesionless	14.00	3213.80	34.00	--	4.06
4	Cohesionless	2.00	3724.60	40.00	--	4.06

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	26.80	47.49
2	Cohesive	375.00	375.00	----	-----	31.95
3	Cohesionless	--	--------	----	26.80	89.02
4	Cohesionless	--	--------	----	31.52	26.70

Total Side Friction : 195.16

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
3797.20	40.00	-----	0.50	160.00	227.83

half plug

Limiting End Bearing Resistance : 208.80

Ultimate Static Pile Capacity : 403.96

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

Design Profile- Sand

Young's Modulus of Steel $E_s := 29000 \cdot ksi$

Moment of Inertia HP 12 x 74 $I_x := 569 \cdot in^4$ $I_x = 0.027\ ft^4$

$I_y := 186 \cdot in^4$ $I_y = 0.009\ ft^4$

Abutment Lateral Analysis

Load Applied at Top of Pile Provided by VHB	P_H	Defined Below
Distance From Fixity to Top of Pile	L	Defined Below
Distance From Fixity to Top of Clay	L_1	Defined Below
Depth of Predrill	L_2	Defined Below
Young's Modulus of Steel	$E_s = 29000\ ksi$	
Allowable Deflections Given By VHB	Δ_{Max} = 0.5 in for Transverse	
	Δ_{Max} = 0.9 in for Longitudinal	

Case 1: Fully Supported

Calculate Fixity

n_h values (LRFD Table C10.4.6.3-2)

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

Soil medium to dense- conservative estimation use medium $n_h := 0.556 \cdot \frac{ksi}{ft}$

$E_p := E_s$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_y := 1.8\left[\left(\frac{E_p \cdot I_y}{n_h}\right)^{0.2}\right]$

$Fixity_y = 6.16\,ft$

$Fixity_x := 1.8\left[\left(\frac{E_p \cdot I_x}{n_h}\right)^{0.2}\right]$

$Fixity_x = 7.7\,ft$

$P_H := \frac{110}{5} kip$ 110 kip is total load divided by 5 piles

$P_H = 22\,kip$

Calculate Displacement

Transverse

$$\Delta_{max} := \frac{P_H \cdot Fixity_x^{\,3}}{12 \cdot E_s \cdot I_x}$$

$\Delta_{max} = 0.09\,in$ < 0.5 in OK

Longitudinal- Solve for P when Δ_{max} = 0.9 in

If Δ_{max} = 0.9
in

$$P_{all} := \frac{0.9 \cdot in \cdot 12 \cdot E_s \cdot \left(I_y\right)}{Fixity_y^{\,3}}$$

$$P_{all} = 145\,kip$$

Solve for Associated Pile Stress

$$M := \frac{P_{all} \cdot \left(Fixity_y\right)}{2}$$

$$M = 5337.68\,in \cdot kip$$

$$S_{yy} := 30.4in^3$$

$$\sigma := \frac{M}{S_{yy}}$$

$\sigma = 176\,ksi$ **YIELD**

Case 2: Predrill to Clay Layer (assume 15' predrill length)

Calculate New Fixity Based on Clay Layer

Undrained Shear Strength of Clay (ksf) $S_u := 0.375$

Soil Modulus for Clays $E_{clay} := S_u \cdot 0.465\,\text{ksi}$

$E_{clay} = 0.17\,\text{ksi}$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_x := 1.4\left[\left(\frac{E_p \cdot I_x}{E_{clay}}\right)^{0.25}\right]$

$Fixity_x = 11.51\,\text{ft}$

$Fixity_y := 1.4\left[\left(\frac{E_p \cdot I_y}{E_{clay}}\right)^{0.25}\right]$

$Fixity_y = 8.7\,\text{ft}$

$P_H := \frac{110}{5}\text{kip}$ 110 kip is total load divided by 5 piles

$P_H = 22\,\text{kip}$

Calculate Displacement

Transverse

$L_x := Fixity_x + 15ft \quad L_x = 26.51\,ft$

$$\Delta_{max} := \frac{P_H \cdot L_x^3}{12 \cdot E_s \cdot I_x}$$

$\Delta_{max} = 3.6\,in$

Longitudinal- Solve for P when Δ_{max} = 0.9 in

If $\Delta_{max} = 0.9$ in

$L_y := Fixity_y + 15ft \quad L_y = 23.7\,ft$

$$P_{all} := \frac{0.9 \cdot in \cdot 12 \cdot E_s \cdot (I_y)}{L_y^3}$$

$P_{all} = 2.5\,kip$

Solve for Associated Pile Stress

$$M := \frac{P_{all} \cdot (L_y)}{2}$$

$M = 360\,in \cdot kip$

$S_{yy} := 30.4in^3$

$$\sigma := \frac{M}{S_{yy}}$$

$\sigma = 11.8\,ksi$

Case 3: Assume Backfill With Loose Sand After Predrill

Calculate New Fixity Based on Loose Sand

n_h values (LRFD Table C10.4.6.3-2)

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

Soil Loose, Submerged- $n_h := 0.208 \cdot \frac{ksi}{ft}$

$E_p := E_s$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_y := 1.8\left[\left(\frac{E_p \cdot I_y}{n_h}\right)^{0.2}\right]$

$Fixity_y = 7.5\,ft$

$Fixity_x := 1.8\left[\left(\frac{E_p \cdot I_x}{n_h}\right)^{0.2}\right]$

$Fixity_x = 9.4\,ft$

$P_H := \frac{110}{5} kip$ 110 kip is total load divided by 5 piles

$P_H = 22\,kip$

Calculate Displacement

Transverse

$$\Delta_{max} := \frac{P_H \cdot Fixity_x^{\ 3}}{12 \cdot E_s \cdot I_x}$$

$$\Delta_{max} = 0.2\,\text{in}$$

Longitudinal- Solve for P when Δ_{max} = 0.9 in

If Δ_{max} = 0.9 in

$$P_{all} := \frac{0.9 \cdot in \cdot 12 \cdot E_s \cdot \left(I_y\right)}{Fixity_y^{\ 3}}$$

$$P_{all} = 80.1\,\text{kip}$$

Solve for Associated Pile Stress

$$M := \frac{P_{all} \cdot \left(Fixity_y\right)}{2}$$

$$M = 3602\,\text{in}\cdot\text{kip}$$

$$S_{yy} := 30.4\text{in}^3$$

$$\sigma := \frac{M}{S_{yy}}$$

$$\sigma = 118\,\text{ksi}$$ **YIELD**

Case 4: Reduced Loads From VHB

Calculate Fixity

n_h values (LRFD Table C10.4.6.3-2)

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

Soil medium to dense- conservative estimation use medium $n_h := 0.556 \cdot \frac{ksi}{ft}$

$E_p := E_s$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_y := 1.8\left[\left(\frac{E_p \cdot I_y}{n_h}\right)^{0.2}\right]$

$Fixity_y = 6.16\,ft$

$Fixity_x := 1.8\left[\left(\frac{E_p \cdot I_x}{n_h}\right)^{0.2}\right]$

$Fixity_x = 7.7\,ft$

$P_H := \frac{13}{5}kip$ 13 kip is total load divided by 5 piles

$P_H = 2.6\,kip$

Calculate Displacement

Transverse

$$\Delta_{max} := \frac{P_H \cdot Fixity_x^{\ 3}}{12 \cdot E_s \cdot I_x}$$

$$\Delta_{max} = 0.01\,\text{in}$$

Longitudinal- Solve for P when Δ_{max} = 0.9 in

If Δ_{max} = 0.9 in

$$P_{all} := \frac{0.9 \cdot \text{in} \cdot 12 \cdot E_s \cdot \left(I_y\right)}{Fixity_y^{\ 3}}$$

$$P_{all} = 145\,\text{kip}$$

Solve for Associated Pile Stress

$$M := \frac{P_{all} \cdot \left(Fixity_y\right)}{2}$$

$$M = 5338\,\text{in} \cdot \text{kip}$$

$$S_{yy} := 30.4\text{in}^3$$

$$\sigma := \frac{M}{S_{yy}}$$

$\sigma = 175.6\,\text{ksi}$ **YIELD**

Case 5: Reduced Loads Form VHB and Predrill to Clay Layer (assume 15' predrill length)

Calculate New Fixity Based on Clay Layer

Undrained Shear Strength of Clay (ksf) $S_u := 0.375$

Soil Modulus for Clays $E_{clay} := S_u \cdot 0.465\,\text{ksi}$

$E_{clay} = 0.17\,\text{ksi}$

Fixity determined by LRFD 10.7.3.13.4 $\text{Fixity}_x := 1.4\left[\left(\frac{E_p \cdot I_x}{E_{clay}}\right)^{0.25}\right]$

$\text{Fixity}_x = 11.51\,\text{ft}$

$\text{Fixity}_y := 1.4\left[\left(\frac{E_p \cdot I_y}{E_{clay}}\right)^{0.25}\right]$

$\text{Fixity}_y = 8.7\,\text{ft}$

$P_H := \frac{13}{5}\,\text{kip}$ 110 kip is total load divided by 5 piles

$P_H = 2.6\,\text{kip}$

Calculate Displacement

Transverse

$L_x := Fixity_x + 15ft \quad L_x = 26.51\,ft$

$$\Delta_{max} := \frac{P_H \cdot L_x^{3}}{12 \cdot E_s \cdot I_x}$$

$\Delta_{max} = 0.4\,in$

Longitudinal- Solve for P when Δ_{max} = 0.9 in

If Δ_{max} = 0.9 in

$L_y := Fixity_y + 15ft \quad L_y = 23.7\,ft$

$$P_{all} := \frac{0.9 \cdot in \cdot 12 \cdot E_s \cdot (I_y)}{L_y^{3}}$$

$P_{all} = 2.5\,kip$

Solve for Associated Pile Stress

$$M := \frac{P_{all} \cdot (L_y)}{2}$$

$M = 360\,in \cdot kip$

$S_{yy} := 30.4in^{3}$

$$\sigma := \frac{M}{S_{yy}}$$

$\sigma = 11.8\,ksi$

Steel Bent Piers (24" steel pipe pile, 3/4" wall thickness, concrete filled):

Density of Concrete	$\gamma_c := 150 \cdot pcf$
Density of Steel	$\gamma_s := 492 \cdot pcf$
Section Area of Steel	$A_s := 0.38 \cdot ft^2$
Section Area of Concrete	$A_c := 2.76 \cdot ft^2$
Composite weight of Pile	γ_{Pile} In lb/ft
Buoyant composite weight of pile	$\gamma_{b.pile}$
Compressive strength of concrete	$f_c := 6 \cdot ksi$
Young's modulus of concrete	E_c in ksi

$$E_c := \left[33000 \cdot \left(\frac{\gamma_c}{1000pcf}\right)^{1.5} \cdot \sqrt{\frac{f_c}{ksi}}\right] \cdot ksi$$

$$E_c = 4696\,ksi$$

Young's modulus of steel $\quad E_s := 29000 \cdot ksi$

Compute Composite Weight of Pile

$$\gamma_{pile} := \gamma_c \cdot A_c + \gamma_s \cdot A_s \qquad \gamma_{pile} = 601\,\frac{lbf}{ft}$$

Compute Buoyant Composite Weight of Pile

$$\gamma_{water} := 62.4pcf$$

$$\gamma_{b.pile} := (\gamma_c - \gamma_{water}) \cdot A_c + (\gamma_s - \gamma_{water}) \cdot A_s \qquad \gamma_{b.pile} = 405\,\frac{lbf}{ft}$$

Factored Load Calculation

Factored capacity = required ultimate
Resistance factor = 0.65

$N_{piles} := 5$

Load $P_v := 2255\text{kip}$

Load Per Pile $P_{pile} := \dfrac{P_v}{N_{piles}}$ $P_{pile} = 451\,\text{kip}$

Factored Load $P_{f.pile} := \dfrac{P_{pile}}{0.65}$ $P_{f.pile} = 694\,\text{kip}$

Profile A - controlling profile (longest pile length)

WEAP Analysis

Run WEAP analysis based on closed end condition with 30ft of pile in air (from deck to mudline) and skin friction distribution from S-Pile (pile length=71ft)
Skin friction approximately 60% of total capacity near top of till layer.

Assume water at mudline

Self weight (DC=1.25 per LRFD load factors)

$W_{pile} := 83\text{ft}\cdot\gamma_{b.pile} + 18\text{ft}\cdot\gamma_{pile}$ $W_{pile} = 44\,\text{kip}$

$Total_{load} := P_{f.pile} + \dfrac{W_{pile}\cdot 1.25}{0.65}$ $Total_{load} = 779\,\text{kip}$

EB @ top of glacial till layer = 1312 k Depth OK

File	Hammer	Erated (ft-k)	Side Fric %	Quake side	Quake toe	Blow/in	Req ULT (K)	Driving Stress (ksi)	
Pier1A	MKT DE 70B	59.5	60	0.1	0.1	17	779	24	too small
Pier2A	MKT70DE70/50B	70	60	0.1	0.1	10	779	27	OK

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Profile 1A - Steel

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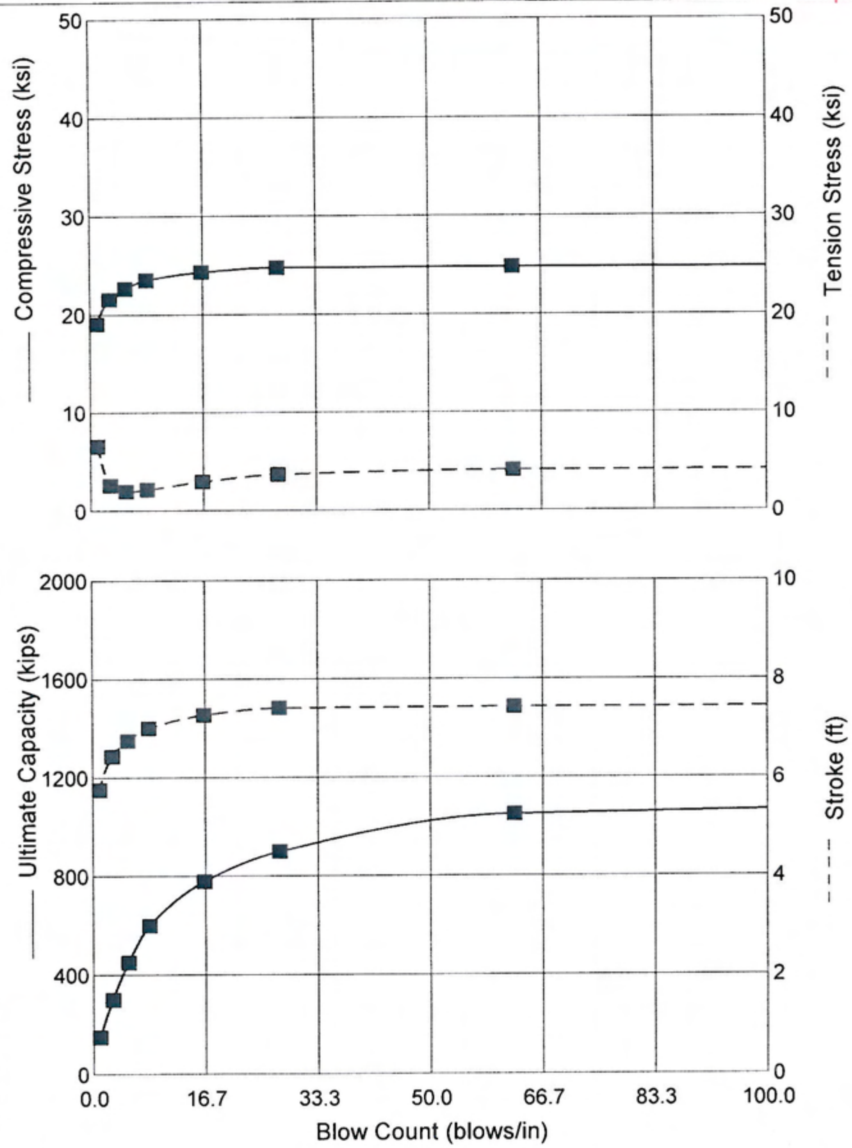
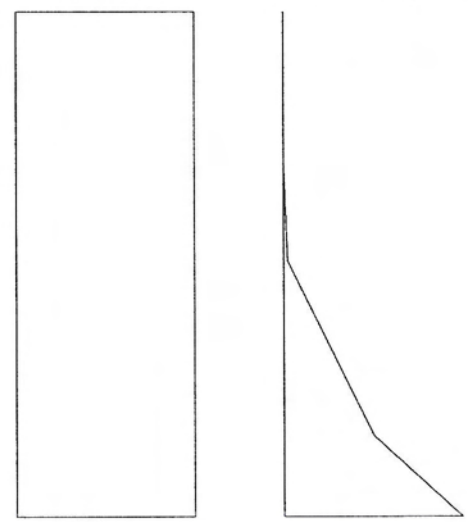


MKT DE 70B

Efficiency	0.800	
Helmet	1.34	kips
Hammer Cushion	34790	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.120	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	101.00	ft
Pile Penetration	71.00	ft
Pile Top Area	54.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 60 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	19.04	6.57	1.0	5.75	20.45
300.0	21.56	2.60	2.9	6.43	19.74
450.0	22.65	1.99	5.3	6.75	19.50
600.0	23.52	2.17	8.4	7.00	20.45
779.0	24.32	2.96	16.5	7.27	21.50
900.0	24.79	3.69	27.7	7.41	22.06
1050.0	24.82	4.13	62.5	7.43	22.04
1200.0	25.01	4.49	532.0	7.49	22.28
1350.0	25.12	4.75	9999.0	7.53	22.42

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Pier 2A – Steel

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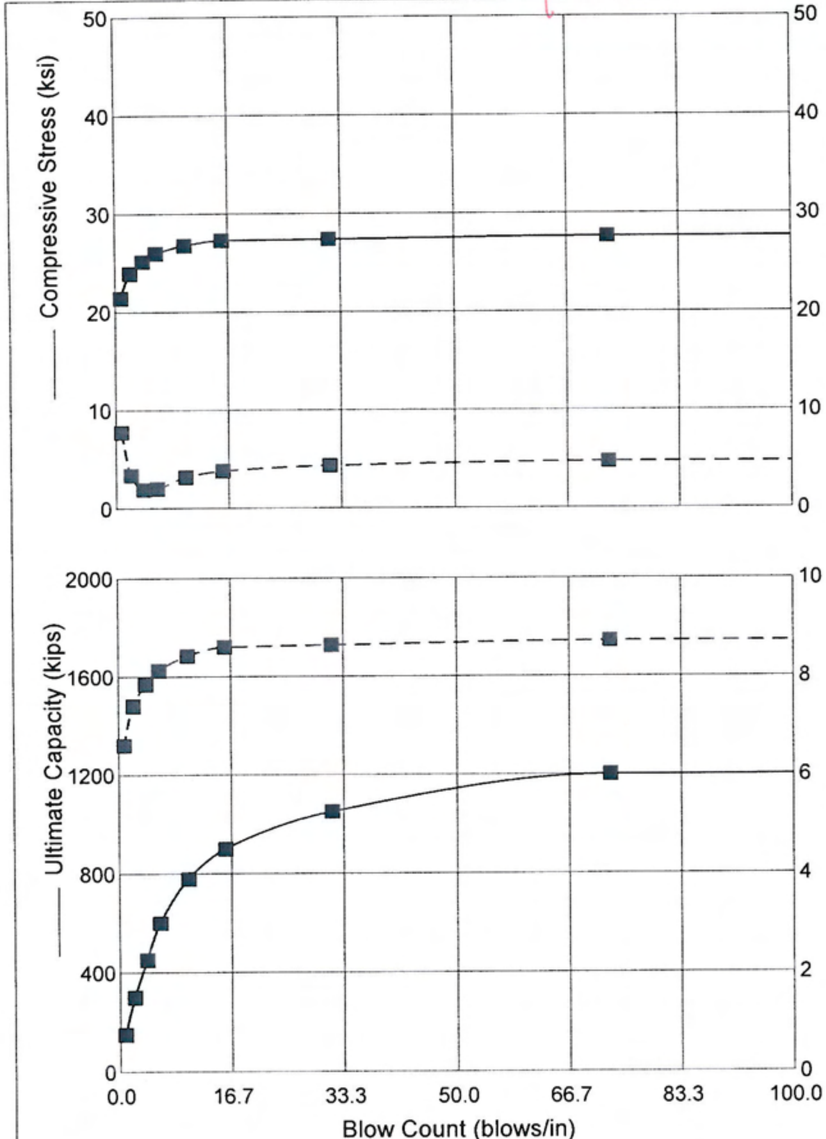
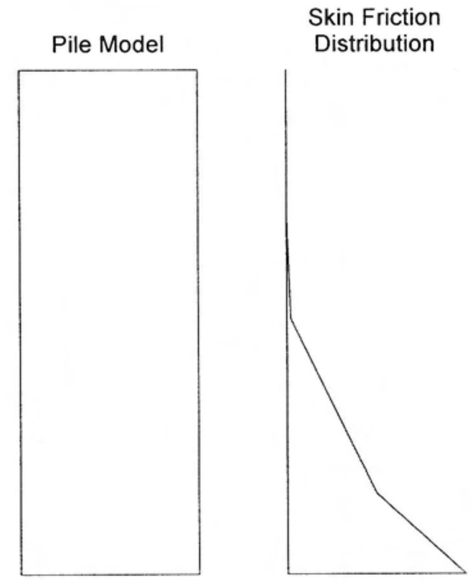


MKT 70 DE70/50B

Efficiency	0.800
Helmet	1.34 kips
Hammer Cushion	34790 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.120 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	101.00 ft
Pile Penetration	71.00 ft
Pile Top Area	54.40 in2



Res. Shaft = 60 %
(Proportional)

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23-Jan-2009
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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	21.39	7.75	0.8	6.60	26.20
300.0	23.92	3.39	2.2	7.40	25.20
450.0	25.13	1.92	4.1	7.83	24.72
600.0	25.96	2.01	6.1	8.12	25.75
779.0	26.76	3.17	10.3	8.42	26.95
900.0	27.29	3.82	15.8	8.60	27.70
1050.0	27.40	4.35	31.6	8.64	27.80
1200.0	27.67	4.75	72.8	8.72	28.09
1350.0	27.79	5.07	9999.0	8.79	28.37

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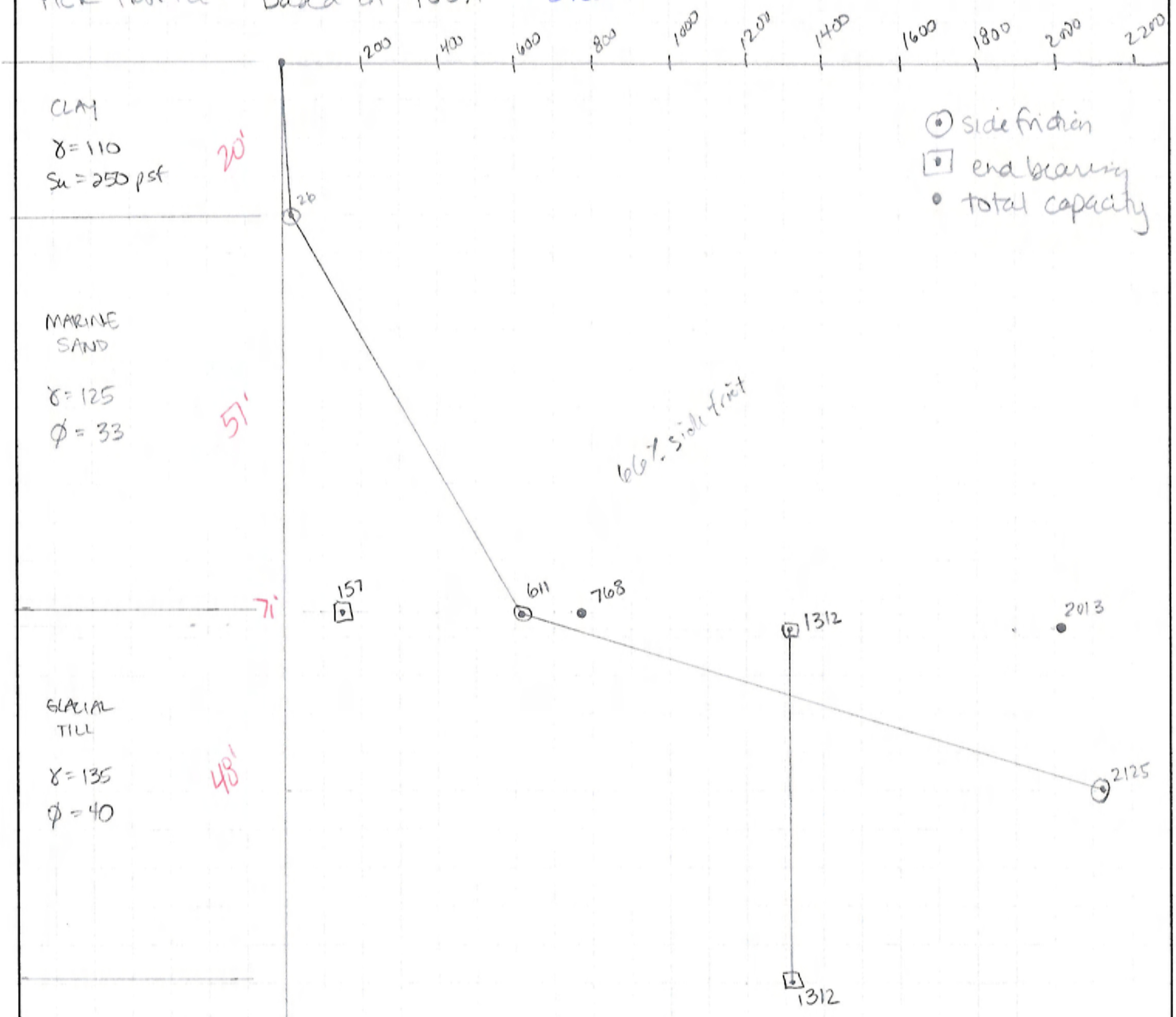
Engineers and Scientists

JOB ______
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______

PIER PROFILE - Based on 405A - Steel



If pile is driven to bottom of sand w/ no consideration of fill below
EB = 157, skin = 611 ⇒ total = 768

(ϕ = 0.65)
total load = 644 kip
∴ drive to top of fill
length OK!

EB @ top of fill layer = 1312

∴ Pile must be driven to bear in fill (~71')

#1

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 - pier	Client	: MDOT
File Name	: pier1	Project Manager	: CLS
Date	: 11/14/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft.
Pile length = 119.00 ft.
Depth to Water Table = 0.00 ft.
Diameter of pile = 24.00 in.
Type of Pile = Pipe Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesive	20.00	476.00	---	--	6.28
2	Cohesionless	51.00	2548.30	33.00	--	6.28
3	Cohesionless	48.00	5887.00	40.00	--	6.28

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesive	250.00	209.50	----	-----	26.33
2	Cohesionless	--	--------	----	26.33	585.01
3	Cohesionless	--	--------	----	31.92	3022.66

Total Side Friction : 3634.00

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
7629.40	40.00	-----	3.14	160.00	2876.22

Limiting End Bearing Resistance : 1311.93

Ultimate Static Pile Capacity : 4945.93

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

#12

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 - pier	Client	: MDOT
File Name	: pier1	Project Manager	: CLS
Date	: 11/14/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft.   Pile length = 73.00 ft.
Depth to Water Table = 0.00 ft.
Diameter of pile = 24.00 in.
Type of Pile = Pipe Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesive	20.00	476.00	---	--	6.28
2	Cohesionless	51.00	2548.30	33.00	--	6.28
3	Cohesionless	2.00	4217.20	40.00	--	6.28

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesive	250.00	209.50	----	-----	26.33
2	Cohesionless	--	--------	----	26.33	585.01
3	Cohesionless	--	--------	----	31.92	90.22

Total Side Friction : 701.56

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
4289.80	40.00	-----	3.14	160.00	1617.22

Limiting End Bearing Resistance : 1311.93

Ultimate Static Pile Capacity : 2013.49

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#3

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name : Route 103 - pier
File Name : pier1
Date : 11/14/10

Client : MDOT
Project Manager : CLS
Computed by : JRT

Depth of Top of Pile = 0.00 ft.
Depth to Water Table = 0.00 ft.
Diameter of pile = 24.00 in.
Type of Pile = Pipe Pile

Pile length = 71.00 ft.

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesive	20.00	476.00	---	--	6.28
2	Cohesionless	51.00	2548.30	33.00	--	6.28

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesive	250.00	209.50	----	-----	26.33
2	Cohesionless	--	--------	----	26.33	585.01

Total Side Friction : 611.34

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
4144.60	33.00	-----	3.14	47.20	374.49

Limiting End Bearing Resistance : 157.08

Ultimate Static Pile Capacity : 768.42

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

Profile B

WEAP Analysis

Run WEAP analysis based on closed end condition with 40ft of pile in air (from deck to mudline) and skin friction distribution from S-Pile (pile length=35ft)
Skin friction 30% of total capacity near top of till layer

Assume water at mudline

Self weight (DC=1.25 per LRFD load factors)

$$W_{pile} := 55ft \cdot \gamma_{b.pile} + 20ft \cdot \gamma_{pile} \qquad W_{pile} = 34\,kip$$

$$Total_{load} := P_{f.pile} + \frac{W_{pile} \cdot 1.25}{0.65} \qquad Total_{load} = 760\,kip$$

File	Hammer	Erated (ft-k)	Side Fric %	Quake side	Quake toe	Blow/in	Reg ULT (K)	Driving Stress ksi
Pier1B	MKT DE 70B	59.5	30	0.1	0.1	11	760	22 OK
Pier2B	MKT 70DE70/50B	70	30	0.1	0.1	9	760	24 OK

EB @ top of glacial till layer is 757 k   Depth OK

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Route 103 New Bridge, Pier 2nd Profile

Profile B - Steel

23-Jan-2009
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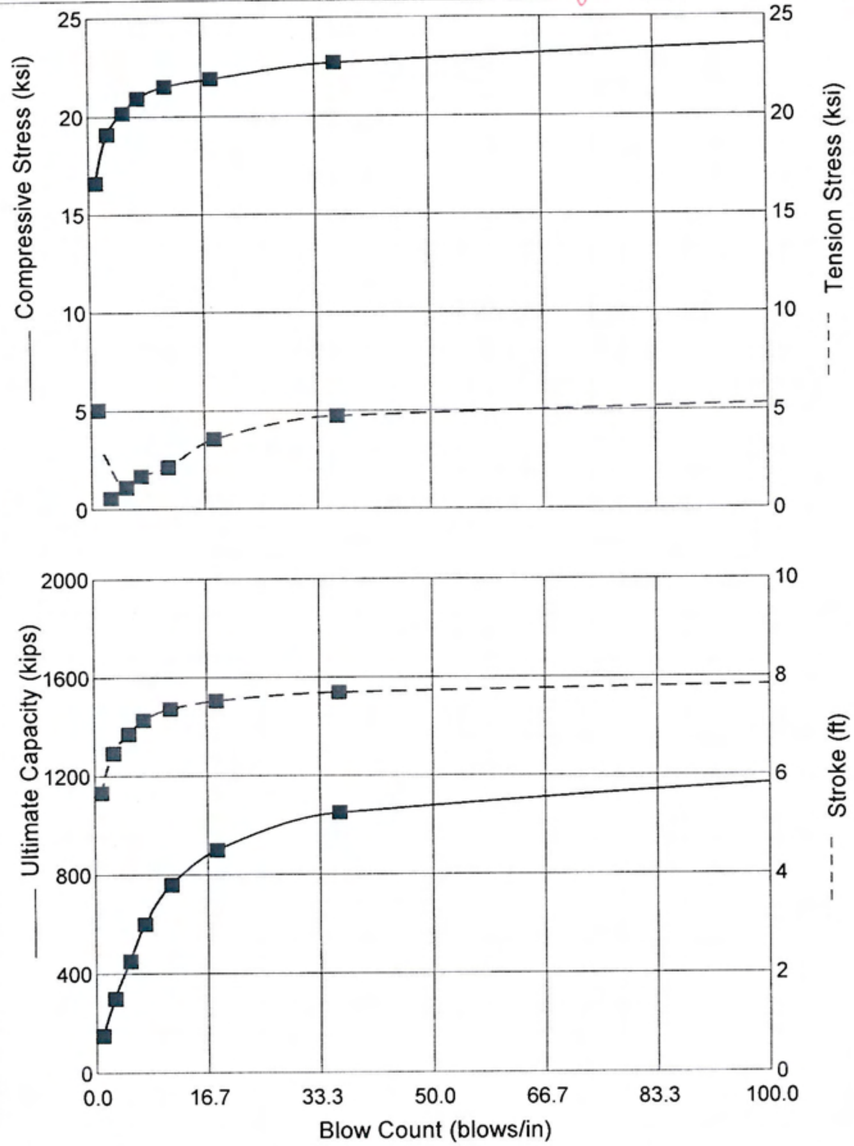
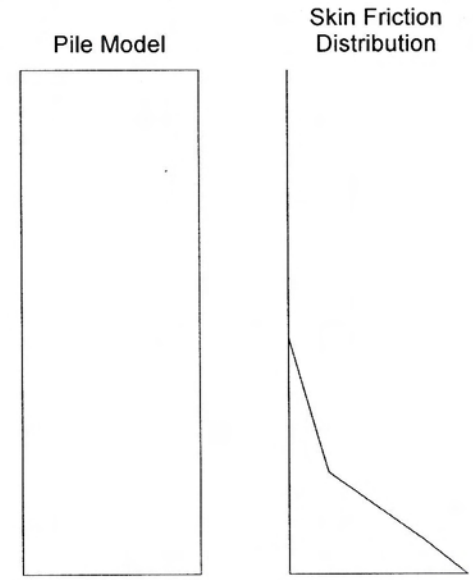


MKT DE 70B

Efficiency	0.800
Helmet	0.00 kips
Hammer Cushion	17640 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.120 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	75.00 ft
Pile Penetration	35.00 ft
Pile Top Area	54.40 in2



Res. Shaft = 30 %
(Proportional)

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Route 103 New Bridge, Pier 2nd Profile

23-Jan-2009
GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	16.61	5.05	1.0	5.66	20.93
300.0	19.09	0.58	2.8	6.46	19.80
450.0	20.17	1.11	5.1	6.85	19.79
600.0	20.92	1.69	7.3	7.13	20.77
760.0	21.51	2.13	11.3	7.36	21.64
900.0	21.90	3.57	18.1	7.52	22.32
1050.0	22.71	4.74	36.2	7.69	23.10
1200.0	23.86	5.50	145.8	7.87	23.91
1350.0	24.81	5.85	9999.0	8.05	24.80

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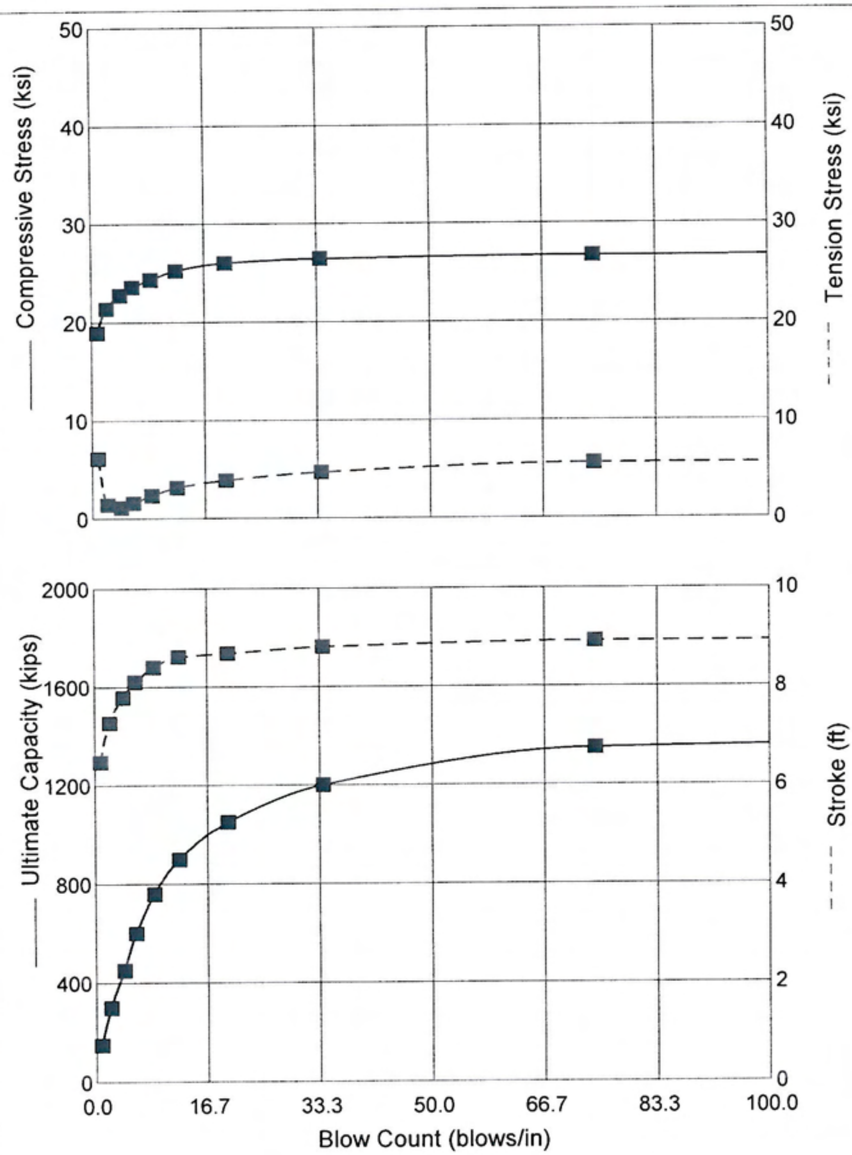
Route 103 New Bridge, Pier 2nd Profile

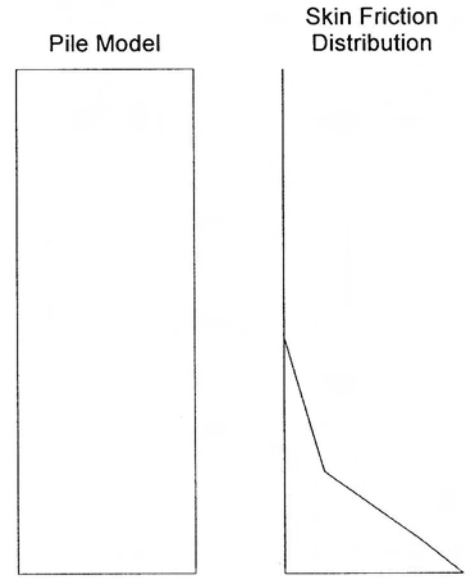
Profile B- Steel

23-Jan-2009
GRLWEAP (TM) Version 2005



MKT 70 DE70/50B

Efficiency	0.800
Helmet	3.09 kips
Hammer Cushion	34825 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.120 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	75.00 ft
Pile Penetration	35.00 ft
Pile Top Area	54.40 in2



Res. Shaft = 30 %
(Proportional)

GZA Geo Environmental, Inc.
Route 103 New Bridge, Pier 2nd Profile

23-Jan-2009
GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	18.92	6.13	0.9	6.47	26.66
300.0	21.36	1.44	2.3	7.27	24.94
450.0	22.75	1.12	4.3	7.78	25.05
600.0	23.58	1.61	6.1	8.10	25.98
760.0	24.33	2.36	8.8	8.40	27.06
900.0	25.24	3.17	12.5	8.61	27.82
1050.0	26.02	3.91	19.8	8.68	28.08
1200.0	26.44	4.73	33.9	8.81	28.58
1350.0	26.71	5.64	74.2	8.92	29.05
1500.0	26.83	6.03	579.5	9.03	29.46

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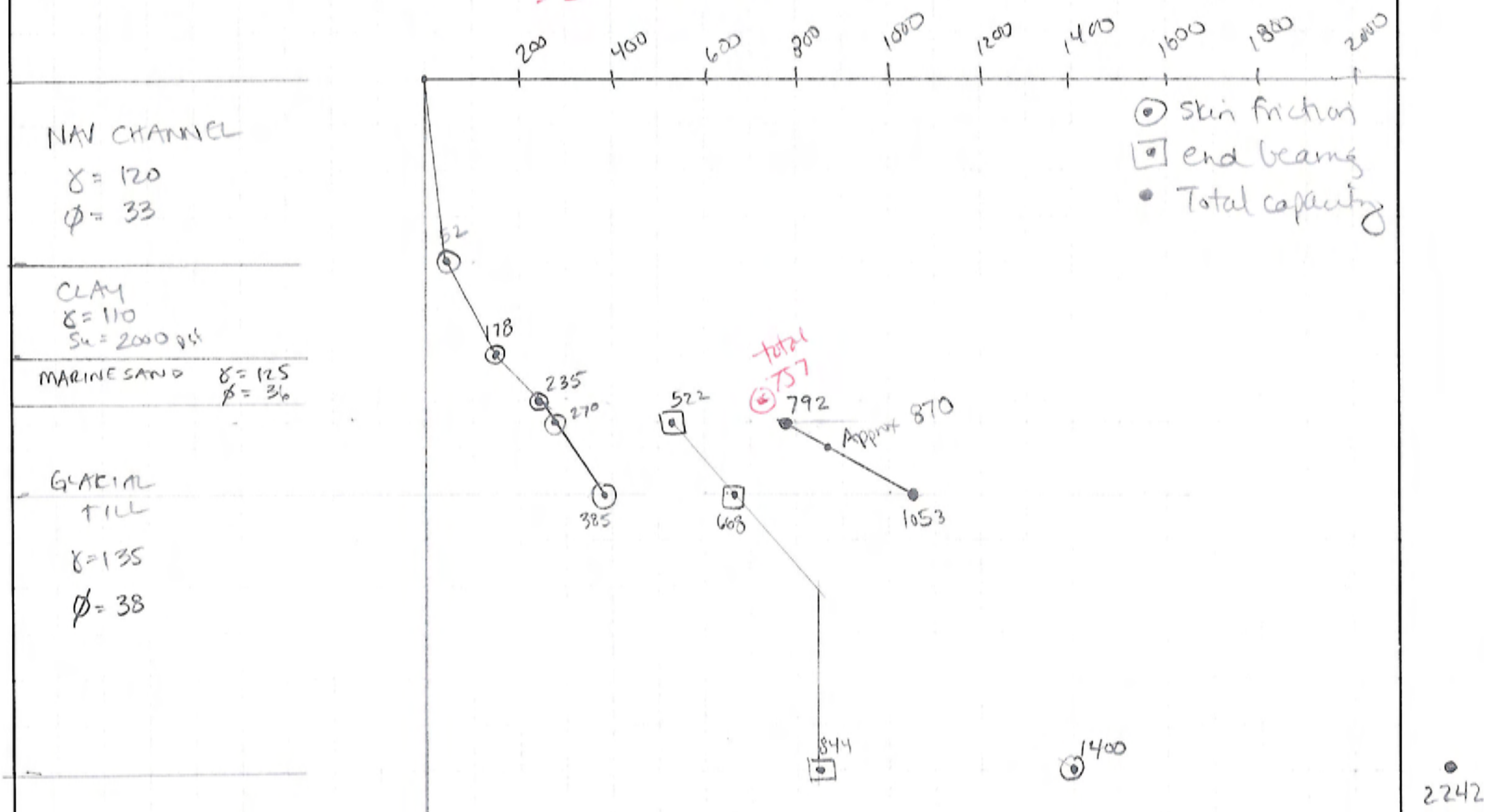
PIER PROFILE – Based on 403 – Steel



Based on S-Pile Pile length needs to be approx. 40'

pile needs to bear on till ~1' in to reach 760 k

#4

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: pier4	Project Manager	: CLS
Date	: 11/14/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft. Pile length = 75.00 ft.
Depth to Water Table = 0.00 ft.
Diameter of pile = 24.00 in.
Type of Pile = Pipe Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	576.00	33.00	--	6.28
2	Cohesive	10.00	1390.00	---	--	6.28
3	Cohesionless	5.00	1784.50	36.00	--	6.28
4	Cohesionless	40.00	3393.00	38.00	--	6.28

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	26.33	51.86
2	Cohesive	2000.00	2000.00	----	-----	125.66
3	Cohesionless	--	--------	----	28.73	57.96
4	Cohesionless	--	--------	----	30.32	1162.89

Total Side Friction : 1398.37

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
4845.00	38.00	-----	3.14	110.40	1213.25

Limiting End Bearing Resistance : 843.83

Ultimate Static Pile Capacity : 2242.20

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#5

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: pier4.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft.   Pile length = 37.00 ft.
Depth to Water Table = 0.00 ft.
Diameter of pile = 24.00 in.
Type of Pile = Pipe Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	576.00	33.00	--	6.28
2	Cohesive	10.00	1390.00	---	--	6.28
3	Cohesionless	5.00	1784.50	36.00	--	6.28
4	Cohesionless	2.00	2013.60	38.00	--	6.28

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	26.33	51.86
2	Cohesive	2000.00	2000.00	----	-----	125.66
3	Cohesionless	--	--------	----	28.73	57.96
4	Cohesionless	--	--------	----	30.32	34.51

Total Side Friction : 269.98

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
2086.20	38.00	-----	3.14	110.40	522.41

Limiting End Bearing Resistance : 843.83

Ultimate Static Pile Capacity : 792.39

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#6

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: pier4.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft.
Depth to Water Table = 0.00 ft.
Diameter of pile = 24.00 in.
Type of Pile = Pipe Pile

Pile length = 45.00 ft.

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	576.00	33.00	--	6.28
2	Cohesive	10.00	1390.00	---	--	6.28
3	Cohesionless	5.00	1784.50	36.00	--	6.28
4	Cohesionless	10.00	2304.00	36.00	--	6.28

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	26.33	51.86
2	Cohesive	2000.00	2000.00	----	-----	125.66
3	Cohesionless	--	--------	----	28.73	57.96
4	Cohesionless	--	--------	----	28.73	149.66

Total Side Friction : 385.14

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
2667.00	38.00	-----	3.14	110.40	667.85

Limiting End Bearing Resistance : 843.83

Ultimate Static Pile Capacity : 1052.99

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Prestressed Concrete Bent Piers:

Density of Concrete $\gamma_c := 150 \cdot pcf$

Section Area of Concrete $A_c := 4 \cdot ft^2$

Weight of Pile γ_{Pile} in lb/ft

Buoyant weight of pile $\gamma_{b.pile}$

Compressive strength of concrete $f_c := 6 \cdot ksi$

Young's modulus of concrete E_c in ksi

$$E_c := \left[33000 \cdot \left(\frac{\gamma_c}{1000pcf} \right)^{1.5} \cdot \sqrt{\frac{f_c}{ksi}} \right] \cdot ksi$$

$$E_c = 4696\,ksi$$

Compute Composite Weight of Pile

$\gamma_{pile} := \gamma_c \cdot A_c$ $\quad \gamma_{pile} = 600 \frac{lbf}{ft}$

Compute Buoyant Composite Weight of Pile

$\gamma_{water} := 62.4pcf$

$\gamma_{b.pile} := (\gamma_c - \gamma_{water}) \cdot A_c$ $\quad \gamma_{b.pile} = 350 \frac{lbf}{ft}$

Factored Load Calculation

Factored capacity = required ultimate
Resistance factor = 0.65

$N_{piles} := 5$

Load $P_v := 2255kip$

Load Per Pile $P_{pile} := \frac{P_v}{N_{piles}}$ $\quad P_{pile} = 451\,kip$

Factored Load $P_{f.pile} := \frac{P_{pile}}{0.65}$ $\quad P_{f.pile} = 694\,kip$

Profile A - controlling profile (longest pile length)

WEAP Analysis

Run WEAP analysis based on 24" concrete pile with 30ft of pile in air (from deck to mudline) and skin friction distribution from S-Pile (pile length 55ft)
Skin friction 75% of total capacity near top of till layer

Effective Prestress $\sigma_{pre.eff} := 700\text{psi}$

Allowable Compression $\sigma_{all} := \left(0.85 \cdot f_c - \sigma_{pre.eff}\right)$

$\sigma_{all} = 4.4\,\text{ksi}$

Allowable Tension $\sigma_{all.ten} := 6 \cdot \sqrt{\frac{f_c}{\text{psi}}}\,\text{psi} + \sigma_{pre.eff}$

$\sigma_{all.ten} = 1.16\,\text{ksi}$

Assume water at mudline

Self weight (DC=1.25 per LRFD load factors)

$W_{pile} := 67\text{ft} \cdot \gamma_{b.pile} + 18\text{ft} \cdot \gamma_{pile}$ $W_{pile} = 34\,\text{kip}$

$Total_{load} := P_{f.pile} + \frac{W_{pile} \cdot 1.25}{0.65}$ $Total_{load} = 760\,\text{kip}$

Total at 55 ft b.g.s. approximately 750 k (still in sand)

Need to drive further to obtain capacity

File	Hammer	Erated (ft-k)	Side Fric %	Quake side	Quake toe	Blow/in	Reg ULT (K)	Driving Stress ksi
CPier1A	MKT 70 DE70/50B	70	75	0.1	0.1	12	760	1.7 (compression) OK 0.4 (tension)

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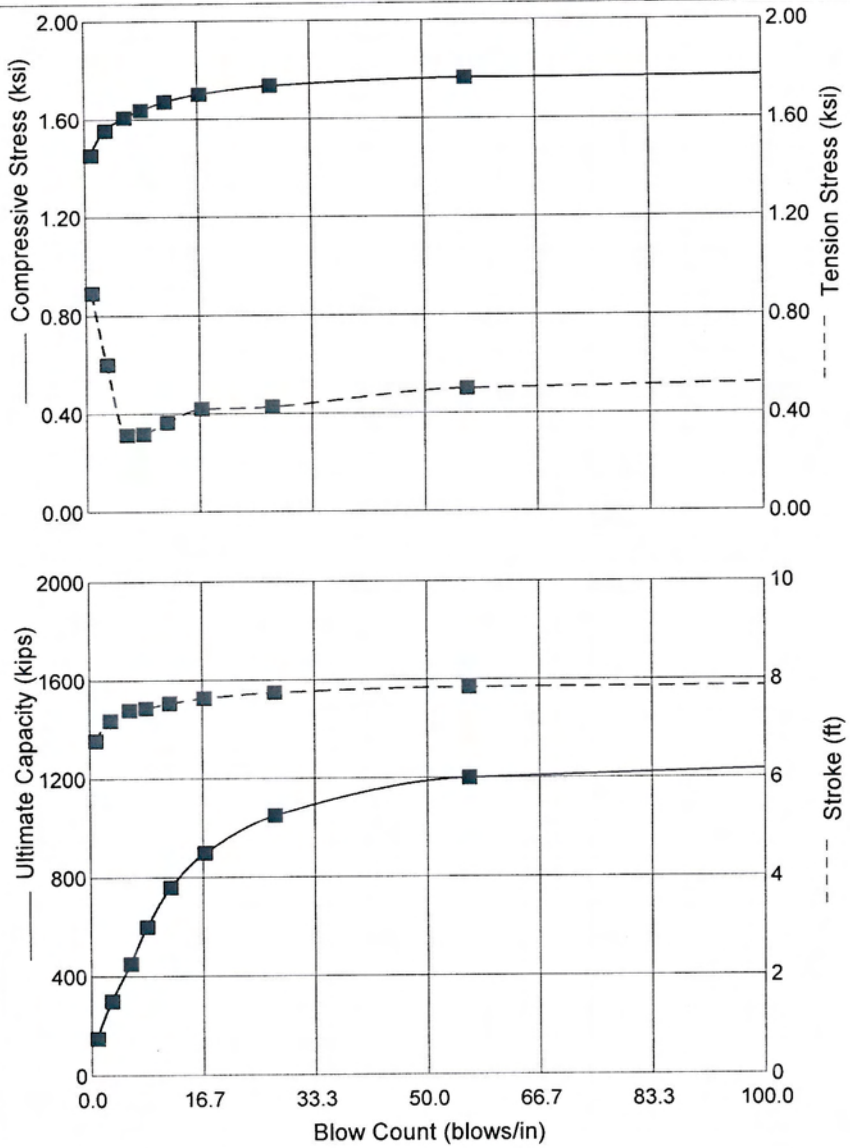
Route 103 Bridge, Conc. Pier, Profile 1

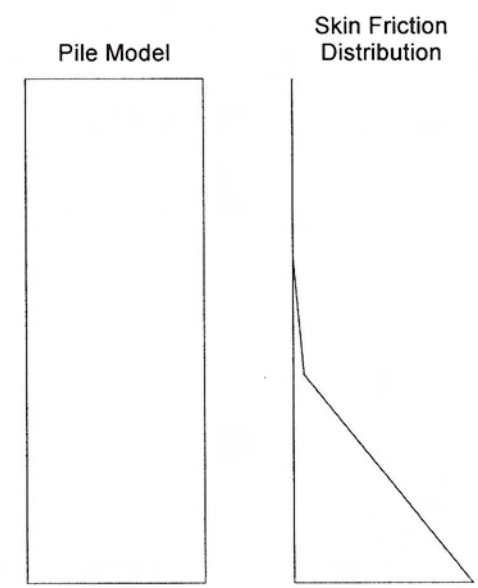
Profile 1A - Concrete

23-Jan-2009
GRLWEAP (TM) Version 2005



MKT 70 DE70/50B

Efficiency	0.800	
Helmet	3.66	kips
Hammer Cushion	42963	kips/in
Pile Cushion	2880	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.120	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	85.00	ft
Pile Penetration	55.00	ft
Pile Top Area	576.00	in2



Res. Shaft = 75 %
(Proportional)

GZA Geo Environmental, Inc. 23-Jan-2009
Route 103 Bridge, Conc. Pier, Profile 1 GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	1.45	0.89	0.9	6.77	14.11
300.0	1.55	0.60	3.1	7.18	12.10
450.0	1.61	0.31	5.9	7.39	12.26
600.0	1.64	0.32	8.4	7.43	11.87
760.0	1.67	0.36	11.9	7.53	11.37
900.0	1.70	0.42	17.0	7.63	11.24
1050.0	1.74	0.43	27.4	7.74	11.43
1200.0	1.76	0.50	56.3	7.84	11.60
1350.0	1.79	0.59	341.9	7.94	11.76
1500.0	1.82	0.66	9999.0	8.02	11.91

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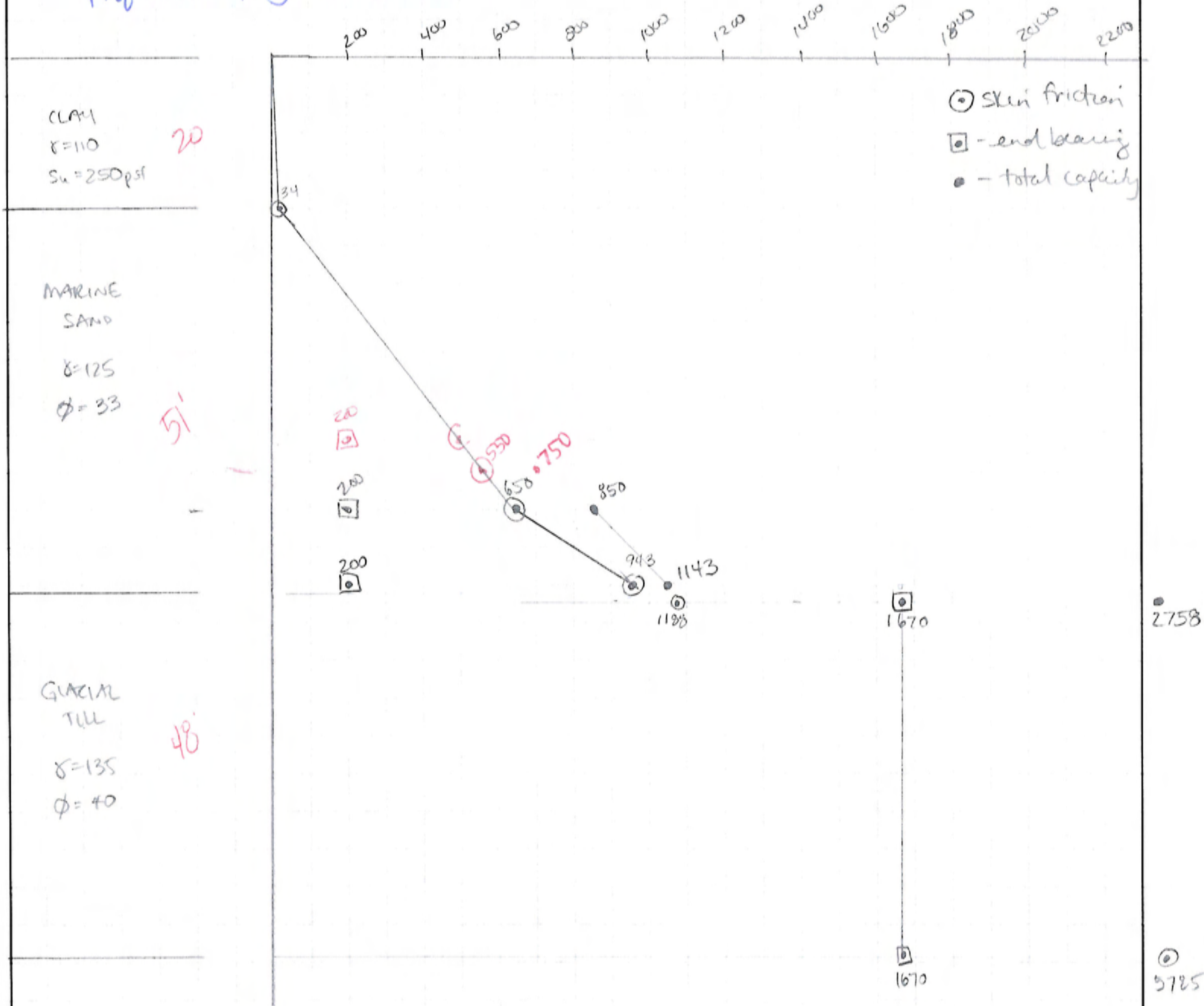
Profile A - 405A concrete



Pile needs to be driven to approx 63' to reach capacity ~ 867

approx 55' to reach ~ 750 K capacity

∴ 56' or so for 760 K.

#1

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: cpier1.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft. Pile length = 119.00 ft.
Depth to Water Table = 0.00 ft.
Width of pile = 24.00 in.
Type of Pile = Precast Concrete Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesive	20.00	476.00	---	--	8.00
2	Cohesionless	51.00	2548.30	33.00	--	8.00
3	Cohesionless	48.00	5887.00	40.00	--	8.00

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesive	250.00	209.50	----	-----	33.52
2	Cohesionless	--	--------	----	30.16	909.48
3	Cohesionless	--	--------	----	36.56	4841.74

Total Side Friction : 5784.73

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
7629.40	40.00	-----	4.00	160.00	3662.11

Limiting End Bearing Resistance : 1670.40

Ultimate Static Pile Capacity : 7455.13

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#2

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name : Route 103 Pier
File Name : cpier1.spl
Date : 11/17/10

Client : MDOT
Project Manager : CLS
Computed by : JRT

Depth of Top of Pile = 0.00 ft.
Depth to Water Table = 0.00 ft.
Width of pile = 24.00 in.
Type of Pile = Precast Concrete Pile

Pile length = 73.00 ft.

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesive	20.00	476.00	---	--	8.00
2	Cohesionless	51.00	2548.30	33.00	--	8.00
3	Cohesionless	2.00	4217.20	40.00	--	8.00

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesive	250.00	209.50	----	-----	33.52
2	Cohesionless	--	--------	----	30.16	909.48
3	Cohesionless	--	--------	----	36.56	144.52

Total Side Friction : 1087.52

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
4289.80	40.00	-----	4.00	160.00	2059.10

Limiting End Bearing Resistance : 1670.40

Ultimate Static Pile Capacity : 2757.92

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#3

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: cpier1.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft. Pile length = 71.00 ft.
Depth to Water Table = 0.00 ft.
Width of pile = 24.00 in.
Type of Pile = Precast Concrete Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesive	20.00	476.00	---	--	8.00
2	Cohesionless	51.00	2548.30	33.00	--	8.00

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesive	250.00	209.50	----	-----	33.52
2	Cohesionless	--	--------	----	30.16	909.48

Total Side Friction : 943.00

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
4144.60	33.00	-----	4.00	47.20	476.81

Limiting End Bearing Resistance : 200.00

Ultimate Static Pile Capacity : 1143.00

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#4

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: cpier1.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft. Pile length = 60.00 ft.
Depth to Water Table = 0.00 ft.
Width of pile = 24.00 in.
Type of Pile = Precast Concrete Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesive	20.00	476.00	---	--	8.00
2	Cohesionless	40.00	2204.00	33.00	--	8.00

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesive	250.00	209.50	----	-----	33.52
2	Cohesionless	--	--------	----	30.16	616.94

Total Side Friction : 650.46

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
3456.00	33.00	-----	4.00	47.20	403.24

Limiting End Bearing Resistance : 200.00

Ultimate Static Pile Capacity : 850.46

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Profile B

WEAP Analysis

Run WEAP analysis based on 24" concrete pile with 40ft of pile in air (from deck to mudline) and skin friction distribution from S-Pile (pile length 36ft)
Skin friction 40% of total capacity near top of till layer

Effective Prestress $\sigma_{pre.eff} := 700psi$

Allowable Compression $\sigma_{all} := \left(0.85 \cdot f_c - \sigma_{pre.eff}\right)$

$\sigma_{all} = 4.4\,ksi$

Allowable Tension $\sigma_{all.ten} := 6 \cdot \sqrt{\frac{f_c}{psi}}\,psi + \sigma_{pre.eff}$

$\sigma_{all.ten} = 1.16\,ksi$

Assume water at mudline

Self weight (DC=1.25 per LRFD load factors)

$W_{pile} := 56ft \cdot \gamma_{b.pile} + 20ft \cdot \gamma_{pile}$ $W_{pile} = 32\,kip$

$Total_{load} := P_{f.pile} + \frac{W_{pile} \cdot 1.25}{0.52}$ $Total_{load} = 770\,kip$

Total capacity @ top of till =750 kips drive to criteria for capacity (additional approx 1' more)

File	Hammer	Erated (ft-k)	Side Fric %	Quake side	Quake toe	Blow/in	Reg ULT (K)	Driving Stress ksi
CPier1B	MKT70DE70/50B	70	40	0.1	0.1	13	770	1.6 (compression) OK 0.34 (tension)

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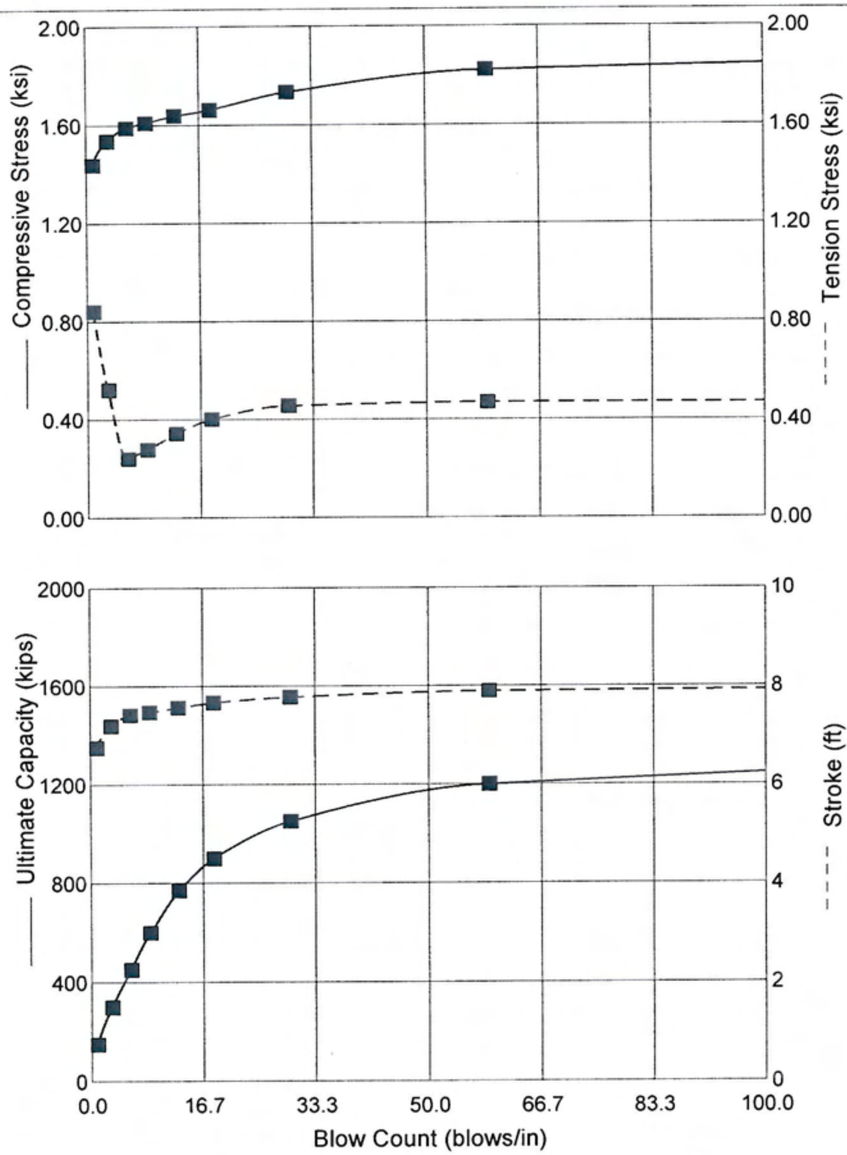
Route 103 Bridge, Conc. Pier, Profile 1

Profile B – Concrete

23-Jan-2009
GRLWEAP (TM) Version 2005

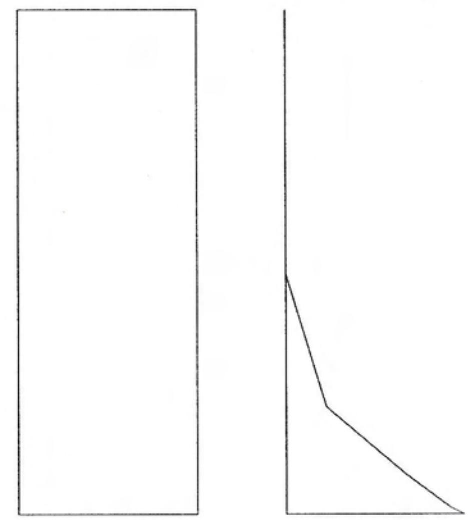


MKT 70 DE70/50B

Efficiency	0.800	
Helmet	3.66	kips
Hammer Cushion	42963	kips/in
Pile Cushion	2880	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.120	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	76.00	ft
Pile Penetration	36.00	ft
Pile Top Area	576.00	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 40 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	1.44	0.84	1.0	6.75	14.41
300.0	1.54	0.52	3.1	7.19	12.39
450.0	1.59	0.24	6.0	7.41	12.55
600.0	1.61	0.28	8.8	7.47	12.00
770.0	1.64	0.34	13.1	7.56	11.50
900.0	1.66	0.40	18.3	7.66	11.51
1050.0	1.73	0.46	29.7	7.77	11.56
1200.0	1.82	0.47	59.0	7.89	11.69
1350.0	1.90	0.47	250.9	7.99	11.88
1500.0	1.95	0.49	9999.0	8.07	12.03

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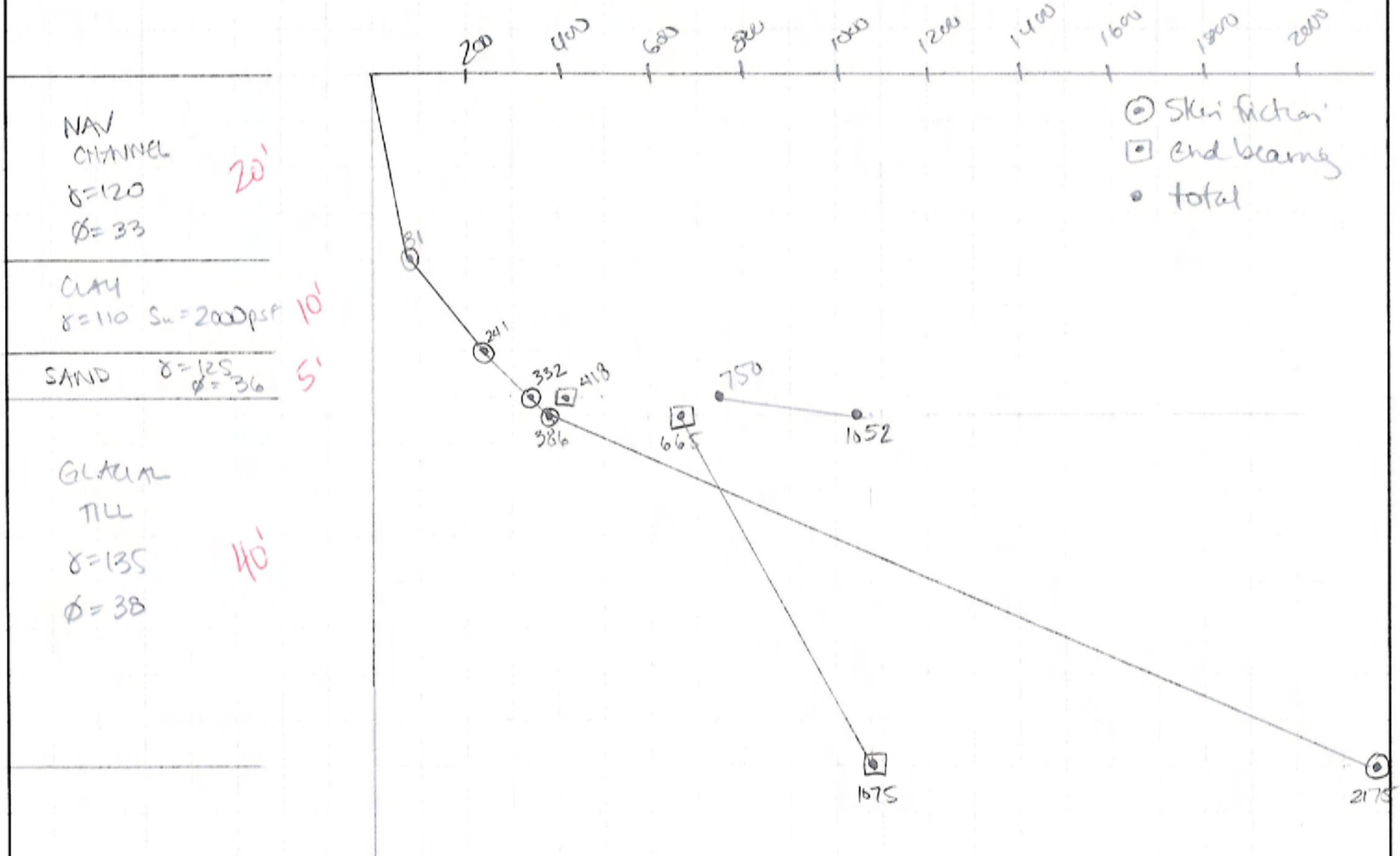
Pier Profile – 403 – Concrete



Pile doesn't get enough capacity until it bears in the till (~1 or 2 ft)

Pile length for capacity is 36 ft.

#5

ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: cpier4.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft. Pile length = 75.00 ft.

Depth to Water Table = 0.00 ft.

Width of pile = 0.00 in.

Type of Pile = Precast Concrete Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	576.00	33.00	--	8.00
2	Cohesive	10.00	1390.00	---	--	8.00
3	Cohesionless	5.00	1784.50	36.00	--	8.00
4	Cohesionless	40.00	3393.00	38.00	--	8.00

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	30.16	80.62
2	Cohesive	2000.00	2000.00	----	-----	160.00
3	Cohesionless	--	--------	----	32.90	91.16
4	Cohesionless	--	--------	----	34.73	1843.55

Total Side Friction : 2175.32

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
4845.00	38.00	-----	4.00	110.40	1544.76

Limiting End Bearing Resistance : 1074.40

Ultimate Static Pile Capacity : 3249.72

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#6

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: cpier4.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft. Pile length = 37.00 ft.
Depth to Water Table = 0.00 ft.
Width of pile = 0.00 in.
Type of Pile = Precast Concrete Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	576.00	33.00	--	8.00
2	Cohesive	10.00	1390.00	---	--	8.00
3	Cohesionless	5.00	1784.50	36.00	--	8.00
4	Cohesionless	2.00	2013.60	38.00	--	8.00

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	30.16	80.62
2	Cohesive	2000.00	2000.00	----	-----	160.00
3	Cohesionless	--	--------	----	32.90	91.16
4	Cohesionless	--	--------	----	34.73	54.70

Total Side Friction : 386.48

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
2086.20	38.00	-----	4.00	110.40	665.15

Limiting End Bearing Resistance : 1074.40

Ultimate Static Pile Capacity : 1051.63

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#7

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: Route 103 Pier	Client	: MDOT
File Name	: cpier4.spl	Project Manager	: CLS
Date	: 11/17/10	Computed by	: JRT

Depth of Top of Pile = 0.00 ft. Pile length = 35.00 ft.
Depth to Water Table = 0.00 ft.
Width of pile = 0.00 in.
Type of Pile = Precast Concrete Pile

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	20.00	576.00	33.00	--	8.00
2	Cohesive	10.00	1390.00	---	--	8.00
3	Cohesionless	5.00	1784.50	36.00	--	8.00

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	30.16	80.62
2	Cohesive	2000.00	2000.00	----	-----	160.00
3	Cohesionless	--	--------	----	32.90	91.16

Total Side Friction : 331.77

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
1941.00	36.00	-----	4.00	77.60	417.72

Limiting End Bearing Resistance : 606.40

Ultimate Static Pile Capacity : 749.50

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Fixity Determination:

Unit Weight of Concrete $\gamma_c := 0.15$

Compressive Strength of Concrete $f_c := 6.0$

Young's Modulus of Concrete $E_c := \left(33000 \cdot \gamma_c^{1.5} \cdot \sqrt{f_c}\right) \cdot 1\text{ksi}$ $E_c = 4695.98\ \text{ksi}$

Young's Modulus of Steel $E_s := 29000 \cdot \text{ksi}$

Steel Bent Piers

E_p must be in ksi and I in ft^4
E_{clay}=0.465*Su (Su must be in ksf, results is in ksi)

Steel to Concrete Modulus Ratio $n := \frac{E_s}{E_c}$ $n = 6.18$

Diameter of Pipe Pile $\phi_p := 24 \cdot \text{in}$

Corrosion Loss per MDOT Bridge Design Guide $c := \frac{1}{8}\text{in}$

Pile Wall Thickness $wall_p := \frac{3}{4}\text{in}$

Diameter of Concrete core $\phi_c := \phi_p - 2 \cdot wall_p$ $\phi_c = 22.5\ \text{in}$

Corrosion Calculations

Diameter of Corroded Pile $\phi_{p.c} := \phi_p - 2 \cdot c$ $\phi_{p.c} = 23.75\ \text{in}$

Corroded Pile Wall Thickness $wall_{p.c} := wall_p - c$ $wall_{p.c} = 0.625\ \text{in}$

Moment of Inertia Calculations

Moment of inertia of concrete core $I_c := \frac{\pi \cdot \left(\phi_c^4\right)}{64}$ $I_c = 12580.56\ \text{in}^4$

Moment of inertia of steel pipe $I_s := \frac{\pi \cdot \left(\phi_{p.c}^4 - \phi_c^4\right)}{64}$ $I_s = 3037.4\ \text{in}^4$

Composite Moment of inertia $I_t := \left(\frac{I_c}{n} + I_s\right)$ $I_t = 0.24\ \text{ft}^4$

Profile A - Clay

Undrained Shear Strength of Clay (ksf) $S_u := 0.25$

Soil Modulus for Clays $E_{clay} := S_u \cdot 0.465 \text{ksi}$

$E_{clay} = 0.12 \text{ ksi}$

Modulus of Elasticity of Pile $E_p := E_s$

Weak axis Moment of Inertia for Pile $I_w := I_t$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_{s.clay} := 1.4\left[\left(\frac{E_p \cdot I_w}{E_{clay}}\right)^{0.25}\right]$

$Fixity_{s.clay} = 22.01 \text{ ft}$

Profile B - Sand

n_h values (LRFD Table C10.4.6.3-2)

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

Soil medium to dense- conservative estimation use medium $n_h := 0.556 \cdot \frac{\text{ksi}}{\text{ft}}$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_{s.sand} := 1.8\left[\left(\frac{E_p \cdot I_t}{n_h}\right)^{0.2}\right]$

$Fixity_{s.sand} = 11.93 \text{ ft}$

Square Concrete Bent Piers:

Dimensions of Concrete Pile $b := 24\text{in}$

$h := 24\text{in}$

Moment of Inertia Calculations

Moment of inertia of concrete $I_c := \frac{b \cdot (h^3)}{12}$

$I_c = 1.33\,\text{ft}^4$

E_p must be in ksi and I in ft^4
E_s=0.465*Su (Su must be in ksf, results is in ksi)

Profile A - Clay

Undrained Shear Strength of Clay (ksf) $S_u := 0.25$

Soil Modulus for Clays $E_{clay} := S_u \cdot 0.465\,\text{ksi}$

$E_{clay} = 0.12\,\text{ksi}$

Modulus of Elasticity of Pile $E_p := E_c$ $E_p = 4695.98\,\text{ksi}$

Weak axis Moment of Inertia for Pile $I_w := I_c$ $I_w = 1.33\,\text{ft}^4$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_{c.clay} := 1.4\left[\left(\frac{E_p \cdot I_w}{E_{clay}}\right)^{0.25}\right]$

$Fixity_{c.clay} = 21.33\,\text{ft}$

Profile B - Sand

n_h values (LRFD Table C10.4.6.3-2)

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

Soil medium to dense- conservative estimation use medium $n_h := 0.556 \cdot \frac{ksi}{ft}$

Fixity determined by LRFD 10.7.3.13.4 $Fixity_{c.sand} := 1.8\left[\left(\frac{E_p \cdot I_w}{n_h}\right)^{0.2}\right]$

$Fixity_{c.sand} = 11.63\,ft$

Lateral Analysis

Steel Bent Piers

Fixity of Steel Piles Profile A	$Fixity_{s.clay} = 22.01\ ft$	
Fixity of Steel Piles Profile B	$Fixity_{s.sand} = 11.93\ ft$	
Load Applied at Top of Bent Pier Provided by VHB	P_{H2}	Defined Below
Load Applied at Elevation 0 of Bent Pier Provided by VHB	P_{H1}	Defined Below
Equivalent Load Applied Top of Bent Pier of P_{H1}	$P_{H1.EQ}$	Calculated Below
Distance From Mudline to El. 0	L_1	Defined Below
Distance From El. 0 to Top Of Bent Pier	L_2	Defined Below
Moment Length for P_{H1}	$L_{.3}$=Fixity+L_1	
Unsupported Length	L_u =Fixity+L_1+L_2	
Young's Modulus of Steel	$E_s = 29000\ ksi$	
Allowable Deflections Given By VHB	Δ_{Max} = 1 in at Shortest Pier 2 in at Longest Pier	For Transverse
	Δ_{Max} = 0.7 in for Longitudinal	

Case 1

Profile A at Bent 3

$P_{H2} := \frac{35}{5} kip$ 7 is total load divided by 5 piles (Strength III)

$P_{H1} := 5kip$ (Strength III)

Calculate $P_{H1.EQ}$

$L_1 := 10ft$ $L_2 := 20ft$

$L_3 := Fixity_{s.clay} + L_1$

$L_u := Fixity_{s.clay} + L_1 + L_2$

$M_1 := P_{H1} \cdot L_3$ $M_1 = 160\ ft \cdot kip$

$P_{H1.EQ} := \frac{M_1}{L_u}$ $P_{H1.EQ} = 3.08\ kip$

Calculate Displacement

Transverse

$P := P_{H2} + P_{H1.EQ}$ $P = 10.08\ kip$

$$\Delta_{max} := \frac{P \cdot L_u^3}{12 \cdot E_s \cdot I_t}$$

$\Delta_{max} = 1.39\ in$ > 1 in Check P_{all}

If Δ_{max} = 1 in

$$P_{all} := \frac{1 \cdot in \cdot 12 \cdot E_s \cdot (I_t)}{L_u^3}$$

$P_{all} = 7.27\ kip$

Longitudinal

$$\Delta_{max} := \frac{P_{H1.EQ} \cdot (L_u^3)}{12 \cdot E_s \cdot I_t}$$

$\Delta_{max} = 0.42\ in$ <0.7 in OK

Case 2

Profile A at Bent -Shortest Pier

$P_{H2} := \frac{35}{5}\text{kip}$ 7 is total load divided by 5 piles (Strength III)

$P_{H1} := 5\text{kip}$ (Strength III)

Calculate $P_{H1.EQ}$

$L_1 := 3\text{ft}$ $L_2 := 10\text{ft}$

$L_3 := Fixity_{s.clay} + L_1$

$L_u := Fixity_{s.clay} + L_1 + L_2$

$M_1 := P_{H1} \cdot L_3$ $M_1 = 125\,\text{ft} \cdot \text{kip}$

$P_{H1.EQ} := \frac{M_1}{L_u}$ $P_{H1.EQ} = 3.57\,\text{kip}$

Calculate Displacement

Transverse

$P := P_{H2} + P_{H1.EQ}$ $P = 10.57\,\text{kip}$

$$\Delta_{max} := \frac{P \cdot L_u^{\,3}}{12 \cdot E_s \cdot I_t}$$

$\Delta_{max} = 0.44\,\text{in}$ < 1 in OK

Longitudinal

$$\Delta_{max} := \frac{P_{H1.EQ} \cdot \left(L_u^{\,3}\right)}{12 \cdot E_s \cdot I_t}$$

$\Delta_{max} = 0.15\,\text{in}$ <0.7 in OK

If Assumed the imposed Δ=0.7in is in the same direction as the 5 kip load, then Δ_{max}=0.15+0.7= 0.85in

Case 3

Profile B at Bent 4-Longest Pier

$P_{H2} := \frac{35}{5} kip$ 7 is total load divided by 5 piles (Strength III)

$P_{H1} := 5kip$ (Strength III)

Calculate $P_{H1.EQ}$

$L_1 := 20ft$ $L_2 := 15ft$

$L_3 := Fixity_{s.sand} + L_1$

$L_u := Fixity_{s.sand} + L_1 + L_2$

$M_1 := P_{H1} \cdot L_3$ $M_1 = 160\, ft \cdot kip$

$P_{H1.EQ} := \frac{M_1}{L_u}$ $P_{H1.EQ} = 3.4\, kip$

Calculate Displacement

Transverse

$P := P_{H2} + P_{H1.EQ}$ $P = 10.4\, kip$

$$\Delta_{max} := \frac{P \cdot L_u^{\,3}}{12 \cdot E_s \cdot I_t}$$

$\Delta_{max} = 1.05\, in$ > 2 in OK

Longitudinal

$$\Delta_{max} := \frac{P_{H1.EQ} \cdot \left(L_u^{\,3}\right)}{12 \cdot E_s \cdot I_t}$$

$\Delta_{max} = 0.34\, in$ <0.7 in OK

If Assumed the imposed Δ=0.7in is in the same direction as the 5 kip load, then Δ_{max}=0.34+0.7= 1.04in

Lateral Analysis

DID NOT REIVSE
OPTION WAS ELIMINATED

Concrete Bent Piers

Fixity of Concrete Piles Profile A	$\text{Fixity}_{c.clay} = 21.33\,\text{ft}$	
Fixity of Concrete Piles Profile B	$\text{Fixity}_{c.sand} = 11.63\,\text{ft}$	
Load Applied at Top of Bent Pier Provided by VHB	P_{H2}	Defined Below
Load Applied at Elevation 0 of Bent Pier Provided by VHB	P_{H1}	Defined Below
Equivalent Load Applied Top of Bent Pier of P_{H1}	$P_{H1.EQ}$	Calculated Below
Distance From Mudline to El. 0	L_1	Defined Below
Distance From El. 0 to Top Of Bent Pier	L_2	Defined Below
Moment Length for P_{H1}	$L_{.3}$=Fixity+L_1	
Unsupported Length	L_u =Fixity+L_1+L_2	
Young's Modulus of Concrete	$E_c = 4696\,\text{ksi}$	
Allowable Deflections Given By VHB	Δ_{Max} = 1 in at Shortest Pier 2 in at Longest Pier	For Transverse
	Δ_{Max} = 0.7 in for Longitudinal	

Case 1

Profile A at Bent 3

$P_{H2} := \frac{35}{5}\text{kip}$ 7 is total load divided by 5 piles

$P_{H1} := 5\text{kip}$

Calculate $P_{H1.EQ}$

$L_1 := 10\text{ft}$ $L_2 := 20\text{ft}$

$L_3 := \text{Fixity}_{c.clay} + L_1$

$L_u := \text{Fixity}_{c.clay} + L_1 + L_2$

$M_1 := P_{H1} \cdot L_3$ $M_1 = 157\,\text{ft}\cdot\text{kip}$

$P_{H1.EQ} := \frac{M_1}{L_u}$ $P_{H1.EQ} = 3.05\,\text{kip}$

Calculate Displacement

Transverse

$P := P_{H2} + P_{H1.EQ}$ $P = 10.05\,\text{kip}$

$$\Delta_{max} := \frac{P \cdot L_u^{\,3}}{12 \cdot E_c \cdot I_c}$$

$\Delta_{max} = 1.51\,\text{in}$ > 1 in Check P_{all}

If Δ_{max} = 1 in

$$P_{all} := \frac{1 \cdot \text{in} \cdot 12 \cdot E_c \cdot (I_c)}{L_u^{\,3}}$$

$P_{all} = 6.67\,\text{kip}$

Longitudinal

$$\Delta_{max} := \frac{P_{H1.EQ} \cdot \left(L_u^{\,3}\right)}{12 \cdot E_c \cdot I_c}$$

$\Delta_{max} = 0.46\,\text{in}$ <0.7 in OK

Case 2

Profile A at Bent -Shortest Pier

$P_{H2} := \frac{35}{5} kip$ 7 is total load divided by 5 piles

$P_{H1} := 5kip$

Calculate $P_{H1.EQ}$

$L_1 := 3ft$ $L_2 := 10ft$

$L_3 := Fixity_{s.clay} + L_1$

$L_u := Fixity_{s.clay} + L_1 + L_2$

$M_1 := P_{H1} \cdot L_3$ $M_1 = 125\, ft \cdot kip$

$P_{H1.EQ} := \frac{M_1}{L_u}$ $P_{H1.EQ} = 3.57\, kip$

Calculate Displacement

Transverse

$P := P_{H2} + P_{H1.EQ}$ $P = 10.57\, kip$

$$\Delta_{max} := \frac{P \cdot L_u^{\,3}}{12 \cdot E_c \cdot I_c}$$

$\Delta_{max} = 0.5\, in$ < 1 in OK

Longitudinal

$$\Delta_{max} := \frac{P_{H1.EQ} \cdot \left(L_u^{\,3}\right)}{12 \cdot E_c \cdot I_c}$$

$\Delta_{max} = 0.17\, in$ <0.7 in OK

If Assumed the imposed Δ=0.7in is in the same direction as the 5 kip load, then Δ_{max}=0.17+0.7= 0.87in

Case 3

Profile B at Bent 4-Longest Pier

$P_{H2} := \frac{35}{5} kip$ 7 is total load divided by 5 piles

$P_{H1} := 5kip$

Calculate $P_{H1.EQ}$

$L_1 := 20ft$ $L_2 := 15ft$

$L_3 := Fixity_{s.sand} + L_1$

$L_u := Fixity_{s.sand} + L_1 + L_2$

$M_1 := P_{H1} \cdot L_3$ $M_1 = 160\, ft \cdot kip$

$P_{H1.EQ} := \frac{M_1}{L_u}$ $P_{H1.EQ} = 3.4\, kip$

Calculate Displacement

Transverse

$P := P_{H2} + P_{H1.EQ}$ $P = 10.4\, kip$

$$\Delta_{max} := \frac{P \cdot L_u^{\,3}}{12 \cdot E_c \cdot I_c}$$

$\Delta_{max} = 1.19\, in$ > 2 in OK

Longitudinal

$$\Delta_{max} := \frac{P_{H1.EQ} \cdot \left(L_u^{\,3}\right)}{12 \cdot E_c \cdot I_c}$$

$\Delta_{max} = 0.39\, in$<0.7 in OK

If Assumed the imposed Δ=0.7in is in the same direction as the 5 kip load, then Δ_{max}=0.39+0.7=1.09in

Scour Analysis

Profile B at Longest Pier-Bent 4-with 15 feet of assumed Scour (Extreme I)

Scour Fixity of Steel Piles Profile A	$Fixity_{s.scour.clay}$	Defined Below
Scour Fixity of Steel Piles Profile B	$Fixity_{s.scour.sand}$	Defined Below
Scour Fixity of Concrete Piles Profile A	$Fixity_{c.scour.clay}$	Defined Below
Scour Fixity of Concrete Piles Profile B	$Fixity_{c.scour.sand}$	Defined Below
Load Applied at Top of Bent Pier Provided by VHB	P_{H2}	Defined Below
Load Applied at Elevation 0 of Bent Pier Provided by VHB	P_{H1}	Defined Below
Equivalent Load Applied Top of Bent Pier of P_{H1}	$P_{H1.EQ}$	Calculated Below
Distance From Mudline to El. 0	L_1	Defined Below
Distance From El. 0 to Top Of Bent Pier	L_2	Defined Below
Moment Length for P_{H1}	$L_{.3}$=Fixity+L_1	
Unsupported Length	L_u=Fixity+L_1+L_2	
Young's Modulus of Steel	$E_s = 29000\,ksi$	
Young's Modulus of Concrete	$E_c = 4696\,ksi$	

Allowable Deflections Given By VHB

Δ_{Max} = 1 in at Shortest Pier   For Transverse
2 in at Longest Pier

Δ_{Max} = 0.7 in for Longitudinal

ScourUndrained Shear Strength of Clay (ksf) $S_u := 2$

Scour Soil Modulus for Clays $E_{clay} := S_u \cdot 0.465 ksi$

$E_{clay} = 0.93\,ksi$

Fixity Calculations

$$\text{Fixity}_{s.scour.clay} := 1.4\left[\left(\frac{E_s \cdot I_t}{E_{clay}}\right)^{0.25}\right] \qquad \text{Fixity}_{c.scour.clay} := 1.4\left[\left(\frac{E_c \cdot I_c}{E_{clay}}\right)^{0.25}\right]$$

$$\text{Fixity}_{s.scour.clay} = 13.09\,\text{ft} \qquad \text{Fixity}_{c.scour.clay} = 12.68\,\text{ft}$$

$$\text{Fixity}_{s.scour.sand} := 1.8\left[\left(\frac{E_s \cdot I_t}{n_h}\right)^{0.2}\right] \qquad \text{Fixity}_{c.scour.sand} := 1.8\left[\left(\frac{E_c \cdot I_c}{n_h}\right)^{0.2}\right]$$

$$\text{Fixity}_{s.scour.sand} = 11.93\,\text{ft} \qquad \text{Fixity}_{c.scour.sand} = 11.63\,\text{ft}$$

Assume Fixity at 13 feet for Scour Calculations

Bent Piers

$P_{H1} = 5\,\text{kip}$

$P_{H2} := 0\text{kip}$ Therefore Longitudinal and Transverse Deflections Are Equal

Calculate $P_{H1.EQ}$

$L_1 := 35\text{ft} \qquad L_2 := 15\text{ft}$

$L_3 := 13\text{ft} + L_1$

$L_u := L_1 + L_2 + 13\text{ft}$

$M_1 := P_{H1} \cdot L_3 \qquad M_1 = 240\,\text{ft}\cdot\text{kip}$

$P_{H1.EQ} := \frac{M_1}{L_u} \qquad P_{H1.EQ} = 3.81\,\text{kip}$

Steel

$$\Delta_{max.s} := \frac{P_{H1.EQ} \cdot L_u^{\,3}}{12 \cdot E_s \cdot I_t}$$

$\Delta_{max.s} = 0.93\,\text{in}$

Concrete

$$\Delta_{max.c} := \frac{P_{H1.EQ} \cdot L_u^{\,3}}{12 \cdot E_c \cdot I_c}$$

$\Delta_{max.c} = 1.06\,\text{in}$

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

STATION 34 BRIDGE
OVER TIDAL ESTUARY
YORK, MAINE

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York County

Soils Report No. 2004-21

PIN 11066.00
Fed No. AC-BH-1106(600)X
June 2004

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Sheets

Appendices

GEOTECHNICAL DESIGN SUMMARY

The purpose of this Design Report is to make geotechnical recommendations for the Station 34 Bridge over the Tidal Estuary in York, Maine in order to facilitate the replacement of the existing bridge. The proposed bridge will consist of pre-cast, prestressed, concrete butted box beams with a bituminous wearing surface founded on driven, integral H-piles. During the site subsurface investigation, a significant, compressible, clayey silt layer was encountered.

The following design recommendations are discussed in detail in the attached Report:

Integral Abutment H-Piles - Piles for use at abutments may be HP 12 x 53, HP 14 x 73 or HP 14 x 89. Grade 50 steel ksi is assumed. The first pile driven at the site should be dynamically tested for capacity. The piles should be driven to refusal on or within the bedrock. The piles should be fitted with pile driving points to protect the tips and to improve the penetration of the piles. Design axial loads should be shown on the plans.

Pile Capacities - Using 50 ksi steel, the geotechnical capacity of the piles is less than the structural capacity and therefore governs.

Pile Type	Allowable end bearing capacity, $Q_{t,\ allow}$ (Kulhaway & Goodman, Driven) FS = 2.25	Total Allowable $Q_{total,\ allow}$ Per Structural Capacity: 50 ksi FS = 4
HP 12 x 53	**152 kips**	194 kips
HP 14 x 73	**210 kips**	268 kips
HP 14 x 89	**257 kips**	326 kips

Frost Protection - All foundations placed on native subgrade soils or fill should be founded a minimum of 4.0 ft below finished exterior grade for frost protection.

Settlement - Any settlement of the bridge abutments will be due to the elastic compression of the piling. The roadway will be widened and settlements resulting from the placement of fills for the widened roadway may be on the order of 1 to 2 inches. This settlement will occur over a long period of time (years) and may require minor attention by a maintenance crew.

1.0 INTRODUCTION

A subsurface investigation and geotechnical design for the replacement of the Station 34 Bridge over the Tidal Estuary in York, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This Report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing bridge was constructed in 1957 and consists of a timber pile supported, three-span, steel girder superstructure with a concrete deck. The existing structure has a span of length 66 ft. Physical inspection indicates that the bridge is approaching structural deficiency and should be replaced. The Federal Highway Administration (FHWA) states that treated marine piles last about 50 years in northern climates. Hydraulic analysis of the existing structure indicates that the current opening area is adequate to carry calculated flow.

It is understood that the existing bridge will be completely removed and replaced. The structure will be replaced with a single span (80 ft), pre-cast, prestressed, concrete butted box beam structure founded on integral abutments on a single row of driven H-piles. The horizontal alignment of the existing bridge will be maintained in the replacement. The vertical alignment will be modified slightly to raise the center of the bridge by approximately 4 inches.

2.0 GEOLOGIC SETTING

The Station 34 Bridge on US Route 103 in York, Maine crosses the Tidal Estuary approximately 0.28 miles south of Harris Island Road as shown on Sheet 1 - Location Map presented at the end of this Report. The Tidal Estuary flows into York Harbor.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits. Soils in the site area are generally comprised of silt, clay, sand and minor amounts of gravel. The most common component is the clayey silt known as the Presumpscot Formation. These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

According to the Bedrock Geologic Map of Maine (1985) the bedrock in the vicinity of the site consists of fine-grained, calcareous, feldspathic, sandstone of the Kittery Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling two (2) test borings (BB-YR34-101 and BB-YR34-102) behind the location of the existing abutments as show in Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this Report. The borings were drilled between March 3 and 10, 2004 using the MaineDOT drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and graphically on Sheet 3 - Boring Logs found at the end of this Report.

The borings were drilled using cased wash boring techniques. Soil samples were obtained at 5-ft or 10-ft intervals using Standard Penetration Test (STP) methods. In-situ vane shear tests were made at regular intervals in the soft soil deposits to measure the shear strength of the strata. The bedrock was cored in both borings using an NQ core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team Member selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field and laboratory testing requirements and logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion if the drilling program.

4.0 Laboratory Testing

Laboratory testing for samples obtained in the borings consisted of nine (9) grain size analyses with hydrometer, one (1) standard grain size analysis and five (5) Atterberg Limits tests. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this Report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this Report.

5.0 Subsurface Conditions

The general soil stratigraphy encountered at the site is as follows:

- **fill** underlain by
- **sand** underlain by
- **silt** underlain by
- **clayey silt** underlain by
- **silty sand** all of which is underlain by
- **bedrock**.

An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this Report. Results of the moisture content, grain size analyses and Atterberg Limits testing can be found in Appendix B - Laboratory Data. This information is also shown on the boring logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report. The following paragraphs discuss the soils encountered in detail:

Fill. Beneath the pavement, a layer of fill soils were encountered in both of the borings. This layer was found to be damp to wet, brown, fine to coarse sand with little gravel and silt. Four SPT N-values in the fill layer ranged from 7 to 27 blows per foot (bpf) indicating that the soil is loose to medium dense in consistency. The thickness of the fill layer ranged from approximately 13 ft in boring BB-YR34-101 to approximately 15 ft in boring BB-YR34-102. No laboratory testing was conducted on the soil samples collected from this layer.

Sand. Underlying the fill soils, a layer of sand was encountered. This layer was found to be wet, grey, fine to coarse sandy silt, with trace gravel and seashells. Two SPT N-values in the sand layer ranged from 4 to 13 bpf indicating that the soil is loose to medium dense in consistency. The thickness of the sand layer ranged from approximately 7 ft in boring BB-YR34-101 to approximately 5 ft in boring BB-YR34-102. No laboratory testing was conducted on the soil samples collected from this layer.

Silt. Underlying the sand, a layer of silt was encountered. This layer can be broken up into two sublayers. The upper layer was found to be grey, wet, very soft to soft, silt and sandy silt, with little clay, trace gravel and seashells. SPT N-values in the upper silt layer ranged from "weight of hammer" to 4 bpf indicating that the soil is very soft to soft in consistency. The thickness of the upper silt layer ranged from approximately 3 ft in boring BB-YR34-101 to approximately 7 ft in boring BB-YR34-102.

Water contents from three samples obtained within this upper silt layer range from approximately 32% to 45%. Three grain size analyses conducted on samples from this upper silt layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a CL-ML or ML by the Unified Soil Classification System. One Atterberg Limits test was made from a sample from the upper silt. The following table summarizes the test results:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-YR34-102 5D	45.4	37	33	4	3.1

The sample has a water content which exceeds the liquid limit indicating that the soil has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that have a high liquefaction potentially commonly referred to as "quick".

The lower silt layer was found to be olive, wet, silt, with little fine sand. This layer is typically described as the "upper crust" of the Presumpscot Formation. SPT N-values in the lower silt layer ranged from 11 to 17 bpf indicating that the lower silt is stiff in consistency. The thickness of the lower silt layer was approximately 5 ft in both of the borings. No laboratory testing was conducted on the soil samples collected from this sublayer.

One vane shear test conducted within the lower silt layer showed measured undrained shear strengths of the layer to exceed 1964 pounds per square foot (psf). When turning the vane in the field, the maximum torque reading was reached prior to shearing of the soil. No remolded shear strength was attempted. The shear strength value exceeding 1964 psf which was obtained indicates the soil is very stiff in consistency.

Clayey Silt. Underlying the silt, a layer of clayey silt was encountered. This layer was determined to consist of grey, wet, clayey silt with trace fine sand in layers and occasional black staining. The overall thickness of the clayey silt layer ranged from approximately 57 ft in boring BB-YR34-101 to approximately 49 ft in boring BB-YR34-102.

Vane shear testing conducted within the clayey silt layer showed measured undrained shear strengths of the layer to range from about 192 to 937 psf while the remolded shear strength ranged from about 49 to 304 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the silty clay has a sensitivity between 2 and 10 and is classified as moderately sensitive to very sensitive. Water contents from six samples obtained within this layer range from approximately 20% to 40%. Grain size analyses conducted on samples from the clayey silt layer indicate that the upper portion soil is classified as an A-4/A-6 by the AASHTO Classification System and a CL or CL-ML by the Unified Soil Classification System. The layer becomes more sandy with depth and is classified an A-4/A-2-4 by the AASHTO Classification System and a SC-SM or SM by the Unified Soil Classification System at these lower depths.

Four Atterberg Limits tests were made from samples throughout this layer. The following table summarizes these test results:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-YR34-101 6D	31.7	34	20	14	0.836
BB-YR34-101 8D	36.8	28	19	9	1.978
BB-YR34-101 11D	28.0	23	17	6	1.833
BB-YR34-102 8D	40.0	32	20	12	1.667

Interpretation of these results indicates that the upper portion of the layer is overconsolidated while the lower potion of the layer is on the verge of being a viscous liquid. The lower potion of the layer has water contents which exceed the liquid limit indicating that the soil has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that have a high liquefaction potentially commonly referred to as "quick".

Silty Sand. Underlying the clayey silt, a layer of silty sand was encountered. This layer was found to be grey, wet, fine to coarse silty sand with little gravel. SPT N-values obtained in the sand layer ranged from 18 to 41 bpf indicating that the soil is medium dense to dense consistency. The thickness of the silty sand layer ranged from approximately 9.8 ft in boring BB-YR34-101 to approximately 24.5 ft in boring BB-YR34-102. One water content from a sample obtained within this silty sand layer was approximately 18%. A grain size analysis conducted on a sample from this layer indicates that the soil is classified as an A-4 by the AASHTO Classification System and a CL-ML by the Unified Soil Classification System.

Bedrock. The bedrock surface was encountered and cored at a depth of 94.8 ft bgs in boring BB-YR34-101 and 105.5 ft bgs in boring BB-YR34-102. The bedrock at the site can be described as grey, medium grained, diorite with prominent orthoclase crystals. In Boring BB-YR34-102 the contact between the intrusive diorite and the country rock was observed. The country rock was encountered at a depth of approximately 108.3 ft bgs (El. -98.1) and is described as dark grey/green quenched andesite porphyry. The RQD of the bedrock was determined to range from 31 to 88% indicating a rock of poor to good quality.

Groundwater. Groundwater was observed at a depth of approximately 9 ft below ground surface in the borings. The water level reading was taken during drilling activities. Groundwater levels at the site are tidally influenced.

6.0 FOUNDATION ALTERNATIVES

Due to cost and the saltwater environment at the site, the use of integral abutments supported on driven H-piles was the only alternative evaluated in the Preliminary Design Report. The subsurface investigation indicates the presence of a significant clayey silt layer underlying the bridge site. Therefore, the use of a driven H-pile supported foundation integral abutment structure is a viable foundation alternative.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following subsections will discuss the foundation considerations and recommendations for an integral abutment structure supported on driven H-piles.

7.1 Driven H-Pile Foundations

The use of a stub abutment founded on a single row of driven H-piles has been determined to be the optimal foundation. The following piles were considered for use at the site: HP 12 x 53, HP 14 x 73 or HP 14 x 89. Grade 50 ksi steel piles should be specified.

Pile length at the abutments can be estimated based on the following data:

Location	Ground Elevation	Depth to Rock bgs	Top of Rock Elevation	Rock Quality Designation
BB-YR34-101 Abutment #1	10.2 ft	94.8 ft	-89.6 ft	31%
BB-YR34-102 Abutment # 2	10.2 ft	105.5 ft	-95.3 ft	88%

For integral abutment piles the MaineDOT Bridge Design Guide (BDG) recommends a Factor of Safety of 4.0 or 0.25Fy for the maximum structural design load. The geotechnical and structural capacities of the H-piles are summarized in the following table.

Pile Type	Allowable end bearing capacity, $Q_{t, allow}$ (Kulhaway & Goodman, Driven) FS = 2.25	Total Allowable $Q_{total, allow}$ Per Structural Capacity: 50 ksi FS = 4
HP 12 x 53	**152 kips**	194 kips
HP 14 x 73	**210 kips**	268 kips
HP 14 x 89	**257 kips**	326 kips

Calculations can be found in Appendix C at the end of this Report. Using the assumption that 50 ksi steel will be used; the allowable geotechnical capacity of the piles is less than the allowable structural capacity and therefore governs. Design axial loads should be shown on the plans. No downdrag should be considered.

The first pile driven at the site should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor. With this level of quality control, the piles shall be driven to an ultimate capacity of 2.25 times the design load.

The piles should be designed as end bearing and should be driven to refusal on or within the bedrock. Piles may penetrate up to 6 inches into fractured and weathered bedrock. The piles should be fitted with pile driving points to protect the tips and to improve penetration.

The soils encountered at the site will provide sufficient lateral support to assume the H-piles are fully braced against Euler buckling. The Designer should check that pile axial stresses from the dead loads, live loads, pile dead load and secondary thermal forces do not exceed the allowable axial pile loads shown in the table. The Designer should also check the live load rotation demand in accordance with BDG Design Procedure 5-4.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Engineer. Contract documents should require that the contractor perform a wave equation analysis of the proposed pile driving system, and the piles be driven to 2.25 times the design (working) load. This factor of safety assumes field dynamic testing will be performed. A hammer should be selected which provides the required geotechnical capacity when the penetration resistance for the final 3 to 6 inches is 8 to 13 blows per 1 inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows. Allowable pile stresses during driving shall be less than 0.90Fy, per AASHTO 4.5.11.

7.2 Stub Abutments and Wingwalls

Cast-in-place integral abutment sections shall be designed structurally for passive earth pressure. A passive earth pressure coefficient (K_p) of 7.3, calculated using Coulomb Theory, is recommended. If an approach slab is not specified, additional lateral earth pressure due to traffic surcharge is required and shall be approximated by an additional 2 ft of earth fill. This results in a traffic surcharge of 250 pcf. Use of an approach slab may be required per the BDG Sections 5.4.2.10 and 5.4.4.

Wingwalls, if oriented in-line with the abutment face, shall be designed for passive earth pressure. A passive earth pressure coefficient of K_p = 3.3 calculated using Rankine Theory can be used.

The Designer may assume Soil Typ~~e~~ 4 (BD~~M~~ G Section ~~300~~ 3.6.1) for retaining wall back fill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf, and a soil-concrete friction coefficient of 0.45.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 of the BDG.

7.3 Frost Protection

According to the MaineDOT design freezing index maps for the State of Maine, the site has a design-freezing index of approximately 1100 F-degree days. Grain size analyses conducted on soil samples in the upper layer of soils indicated that the soils are granular and have a water content of approximately 30%. These components correlate to a frost depth of approximately 4 ft. Therefore, any foundations placed on native subgrade soil should be founded a minimum of 4 ft below finished exterior grade for frost protection. See Appendix C - Calculations for supporting documentation. The final depth of embedment may be controlled by the scour susceptibility of the soil and may, in fact, be deeper than the depth required for frost protection.

7.4 Bearing Capacity

In the event that any foundation will be founded on the native soils the allowable bearing capacity of the layer should not exceed 4 ksf. See Appendix C- Calculations for supporting documentation. No footing shall be less than 2 ft wide regardless of the applied bearing pressure.

7.5 Settlement

It is understood that the horizontal alignment of the existing bridge will be maintained in the replacement of the Station 34 Bridge. The vertical alignment will be modified slightly to raise the center of the bridge by approximately 4 inches.

Any settlement of the bridge abutments will be due to the elastic compression of the piling. The roadway will be widened slightly and the widened approaches will be constructed using embankments sloped at 1V:3H. Settlements resulting from the placement of fills for the widened roadway may be on the order of 1 to 2 inches. This settlement will occur over a long period of time (years) and may require minor attention by a maintenance crew.

7.6 Backfill Material

Structure backfill within 10 ft of the structure and side-slope fill materials shall conform to MaineDOT Specification 703.19 - Granular Borrow for Underwater Backfill. This gradation specifies that 10 percent or less of the material may pass the No. 200 sieve. This material is also specified in order to reduce the amount of fines and to minimize frost action behind the structure. The structure design shall include a drainage system to intercept any groundwater. Drainage behind structure shall be in accordance with Section 5.4.1.4 of the BDG.

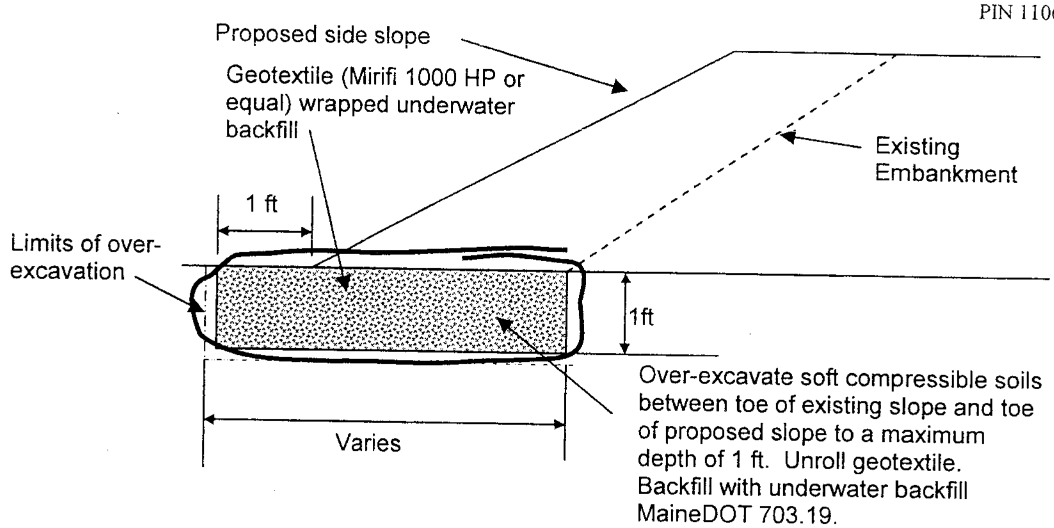
7.7 Embankment Widening Construction

At elevations below high tide plus wave run-up level, the embankment widening construction techniques will require special emphasis on preventing wave and current scour and the potential loss of soils due to piping. For portions of the work that will take place in the inter-tidal zone, the Contractor will need to work only as large a segment as can be completed in a single tide cycle, including final protective measures to prevent scour and piping. Two alternatives are considered feasible for embankment widening and reconstruction under these conditions:

- graded stone fill slopes and
- filter-protected granular fill slopes with riprap

The advantage of graded stone fill slopes is that the fills could be submerged repeatedly during construction and no cofferdams would be needed for the widening. The advantage of the filter-protected granular fill slopes with riprap is that they are less costly, however, it is difficult to protect the granular material during each tide cycle and it is difficult to compact lifts above saturated material.

Based on the borings located behind the abutments, it is understood that the existing approaches are built over soft compressible clayey silts. Due to the presence of these soft soils, it is recommended that the area of the embankment widening between the existing toe of slope and the proposed toe of slope be over-excavated to a depth of 1 ft and the material replaced with a geotextile wrapped granular mat. The geotextile should be rolled directly on the exposed subgrade along the existing embankment. The trench should be backfilled with compacted MaineDOT 703.19 Granular Borrow Material for Underwater Backfill. The geotextile should be wrapped up around the aggregate zone. The total area of the geotextile-wrapped trench should be sufficient to extend 1 ft beyond the toe of the widened embankment. The following figure illustrates the construction:



7.8 Seismic Design Considerations

Per BDG 3.7.1.1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09 are designated a Seismic Performance Category (SPC) classification of A, and should be designed in conformance with Section 5 of Division 1-A of the AASHTO Standard Specifications for Highway Bridges. For SPC A, no detailed analysis is required other than connection design and bearing seat length, except if the bridge is functionally important or classified as a major structure.

The horizontal bedrock acceleration coefficient (A) for York is less than 0.05g, based on Figure 3-4 of the BDG, Seismic Performance Categories for Maine, August 2003. Per Section 3.5 of Division 1-A of the AASHTO Standard Specifications for Highway Bridges Soil Profile Type III is applicable to the site and a site coefficient (S) of 1.5 would be used.

According to Figure 2-2 of the BDG, the Station 34 Bridge is not on the National Highway System (NHS) and is therefore not considered to be functionally important. Since the bridge construction costs do not exceed $10 million the bridge is not classified as a major structure. As a result, the bridge substructures will not be designed for seismic earth loads. The soils at the site are considered to be liquefaction-susceptible.

8.0 CLOSURE

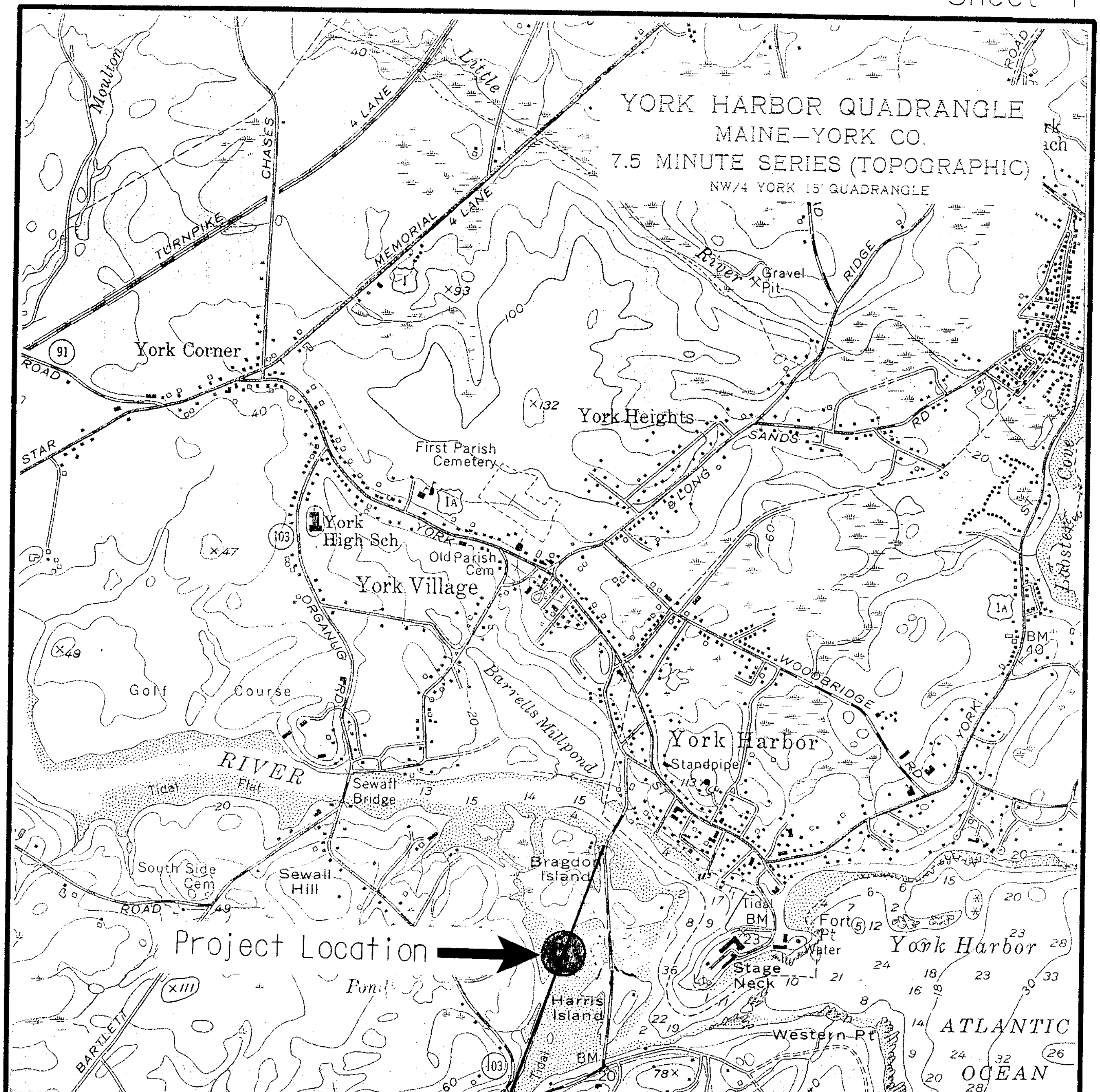
This Report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Station 34 Bridge in York, Maine in accordance with generally accepted soil and foundation engineering practices. No other intended use is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this Report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this Report.

We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Location Map

Sheet 1



York, Maine, Station 34 Bridge over Tidal Estuary, PIN. 11066.00

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

1:24000, 1" = 2000', 1 cm = 240 m

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine
Boring No.: BB-YR34-101
PIN: 11066.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/3/04-3/3/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	15+75, 9.2 Rt.	Casing ID/OD:	HW	Water Level*:	9.5' (Tidal)

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; R = Rock Core Sample; V = Insitu Vane Shear Test; SSA = Solid Stem Auger

Definitions: S_u = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_p = Unconfined Compressive Strength (ksf); $S_{u(lab)}$ = Lab Vane Shear Strength (psf); WOH = weight of 140lb. hammer; WOR = weight of rods

Definitions: 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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.78	Pavement ——0.42	
5	1D	24/24	5.00 - 7.00	7/15/12/16	27			Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/6	10.00 - 12.00	4/3/4/4	7	18, 19, 27		Brown, wet, loose, fine to coarse SAND, little gravel and silt, (Fill).	
						18, 26, 2	-2.80	——13.00	
15	3D	24/2	16.00 - 18.00	5/3/1/4	4	11, 10, 12, 12		Grey, wet, very loose, fine to coarse silty SAND, trace gravel and sea shells.	
20	4D	24/20	20.00 - 22.00	2/WOH/WOH/WOH	—	16, 11, 19	-9.80	——20.00 Grey, wet, very soft, SILT, some fine sand, little clay, trace gravel, medium to coarse sand and sea shells.	G#176641 A-4, CL-ML WC=36.8%
						30, 40	-12.80	- - - -23.00	
25	5D/MV	24/20	25.00 - 27.00	4/7/10/13	17	69, 69, 69, 63, 69		Olive, wet, very stiff, SILT, little fine sand, (Presumpscot upper crust). Attempt 55x110 mm vane: could not push	
							-17.80	- - - -28.00	
30	6D	24/23	30.00 - 32.00	3/2/3/3	5	78 aWA, 70, 64, 64, 50		Grey, wet, medium stiff, SILT, some clay, trace fine sand, mottled. aWashed ahead of casing.	G#176642 A-6, CL WC=31.7% LL=34 PL=20 PI=14
35	7D	24/22	35.00 - 37.00	2/2/1/WOH	3	52, 52, 50, 38, 32		Grey, wet, very loose, fine silty SAND, uniform, dilatent.	
40	8D V1	24/24	40.00 - 42.00 40.64	Push thru vane Su=259/125 psf		62		Grey, wet, soft, clayey SILT, trace fine sand layers, black staining. 55x110 mm vane raw torque readings: V1 = 5.8/2.8 ft-lbs V2 = 10.7/1.5 ft-lbs	G#176643 A-4, CL WC=36.8% LL=28 PL=19 PI=9
	V2		41.00 41.64 - 42.00	Su=478/67 psf		45, 44, 48, 50			
45	9D V3	24/24	45.00 - 47.00 45.64	Push thru vane Su=192/67 psf		87		Similar to above, very soft. 55x110 mm vane raw torque readings: V3 = 4.3/1.5 ft-lbs V4 = 5.3/1.6 ft-lbs	
	V4		46.00 46.64 - 47.00	Su=237/71 psf		68, 60, 40, 47			
50	10D V5	24/24	50.00 - 52.00 50.64	Push thru vane Su=437/80 psf		73		Grey, wet, soft to medium stiff, clayey SILT with occasional fine sand layers, black staining. 55x110 mm vane raw torque readings: V5 = 9.8/1.8 ft-lbs V6 = 13.5/1.7 ft-lbs	
	V6		51.00 51.64 - 52.00	Su=603/76 psf		84, 85, 69, 69			
55	MD V7	24/0	55.00 - 57.00 55.64	Push thru vane Su=545/76 psf		94		No recovery, similar soils on spoon, medium stiff. 55x110 mm vane raw torque readings: V7 = 12.2/1.7 ft-lbs V8 = 12.8/1.3 ft-lbs	
	V8		56.00 56.64 - 57.00	Su=571/58 psf		57, 47, 34, 31			
60	11D V9	24/24	60.00 - 62.00 60.64	Push thru vane Su=518/89 psf		49		Grey, wet, soft to medium stiff, SILT, some clay, with little fine sand layers, black staining. 55x110 mm vane raw torque readings: V9 = 11.6/2.0 ft-lbs V10 = 11.0/2.1 ft-lbs	G#176644 A-4, CL-ML WC=28.0% LL=23 PL=17 PI=6
	V10		61.00 61.64 - 62.00	Su=491/94 psf		40, 41, 38, 32			
65	12D V11	24/24	65.00 - 67.00 65.64	Push thru vane Su=603/143 psf		52		Similar to above, medium stiff. 55x110 mm vane raw torque readings: V11 = 13.5/3.2 ft-lbs V12 = 13.9/3.7 ft-lbs	
	V12		66.00 66.64 - 67.00	Su=621/165 psf		41, 46, 40, 50			
70	13D V13	24/24	70.00 - 72.00 70.64	Push thru vane Su=763/89 psf		74		Grey, wet, medium stiff, clayey SILT, little fine sand with fine sand layers. 55x110 mm vane raw torque readings: V13 = 17.1/2.0 ft-lbs V14 = 14.4/4.8 ft-lbs	
	V14		71.00 71.64 - 72.00	Su=643/214 psf		61, 61, 58, 53			
75									

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.

Page 1 of 2 — Boring No.: BB-YR34-101

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary — Location: York, Maine — Boring No.: BB-YR34-101 — PIN: 11066.00

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	14D V15	24/3	75.00 - 77.00 75.64	WOR/4/5/6 Su=608/304 psf	9		Similar to above, medium stiff. 55x110 mm vane raw torque readings: V15 = 18.1/6.8 ft-lbs MV = could not push	
	MV		76.00 - 76.20					
80	15D	24/20	80.00 - 82.00	28/58/54/24	112		Grey, wet, very stiff, SILT, some fine sand, trace clay. aWashed ahead of Casing.	G#176645 A-2-4, SM WC=21.5%
85	16D	24/24	85.00 - 87.00	WOR/8/11/10	19	-74.80	——85.00 Grey, wet, medium dense, fine silty SAND, uniform.	
90	17D	24/6	90.00 - 92.00	18/4/14/9	18		Grey, wet, medium dense, fine to coarse silty SAND, little gravel.	G#176646 A-4, CL-ML WC=18.2%
95	R1	60/60	94.80 - 99.80	RQD = 31%		-84.60	a156 blows for 0.4'. ——94.80 Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. R1:Core Times (min:sec) 94.8' - 95.8' (5:05) 95.8' - 96.8' (5:50) 96.8' - 97.8' (6:10) 97.8' - 98.8' (6:05) 98.8' - 99.8' (5:15) Recovery=100%	
100						-89.60	——99.80 Bottom of Exploration at 99.80 feet below ground surface.	

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.

Page 2 of 2 — Boring No.: BB-YR34-101

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine
Boring No.: BB-YR34-102
PIN: 11066.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/9/04-3/9/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	16+68.6, 10.0 Lt.	Casing ID/OD:	HW	Water Level*:	9.0' (Tidal)

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; R = Rock Core Sample; V = Insitu Vane Shear Test; SSA = Solid Stem Auger

Definitions: S_u = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_p = Unconfined Compressive Strength (ksf); $S_{u(lab)}$ = Lab Vane Shear Strength (psf); WOH = weight of 140lb. hammer; WOR = weight of rods

Definitions: 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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.62	Pavement ——0.58	
5	1D	24/4	5.00 - 7.00	6/6/17/22	23			Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/9	10.00 - 12.00	2/6/11/7	17			Brown, wet, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
15	3D	24/17	15.00 - 17.00	6/8/5/7	13	24, 14, 18, 17, 17	-4.80	——15.00 Grey, wet, medium dense, fine silty SAND with sea shells.	
20	4D	24/16	20.00 - 22.00	2/2/2/2	4	37, 24, 23, 23, 24	-9.80	——20.00 Grey, wet, soft, fine sandy SILT, little clay, trace gravel and medium to coarse sand with sea shells.	G#176647 A-4, CL-ML WC=31.6%
25	5D	24/24	25.00 - 27.00	1/WOH/WOH/2	—	32, 31		Grey, wet, soft, SILT, little clay, trace fine sand.	G#176648 A-4, ML WC=45.5% LL=37 PL=33 PI=4
	V1		27.64 - 28.00	aSu=1964/- psf		36, 88, 126	-16.80	- - - -27.00 55x110 mm vane raw torque readings: V1 = 44/— ft-lbs aVane reached maximum torque reading without shearing, no remolded was attempted.	
30	6D	24/14	30.00 - 32.00	8/4/7/9	11	45, 44		Olive, wet, stiff, SILT, (Presumpscot upper crust).	
						38, 39, 34, 34, 38, 38, 32, 30	-21.80	- - - -32.00	
40	7D V2	24/24	40.00 - 42.00 40.64	Push thru vane Su=446/67 psf		62		Grey, wet, soft, clayey SILT, trace fine sand, black staining. 55x110 mm vane raw torque readings: V2 = 10.0/1.5 ft-lbs V3 = 8.8/1.1 ft-lbs	
	V3		41.00 41.64 - 42.00	Su=393/49 psf		50, 37, 34, 30, 34, 30, 32, 31, 30			
50	8D V4	24/24	50.00 - 52.00 50.64	Push thru vane Su=540/71 psf		54		Grey, wet, soft, silty CLAY with trace fine sand layers and black staining. 55x110 mm vane raw torque readings: V4 = 12.1/1.6 ft-lbs V5 = 7.4/1.2 ft-lbs	G#176649 A-6, CL WC=40.0% LL=32 PL=20 PI=12
	V5		51.00 51.64 - 52.00	Su=330/54 psf		41, 39, 35, 33, 37, 35, 33, 33, 34			
60	9D V6	24/24	60.00 - 62.00 60.64	Push thru vane Su=241/112 psf		73		Similar to above, soft to medium stiff. 55x110 mm vane raw torque readings: V6 = 5.4/2.5 ft-lbs V7 = 12.0/2.2 ft-lbs	
	V7		61.00 61.64 - 62.00	Su=536/98 psf		56, 53, 45, 42, 48, 40, 45, 38, 34			
70	10D V8	24/24	70.00 - 72.00 70.64	Push thru vane Su=937/250 psf		52		Grey, wet, medium stiff, clayey SILT, little fine sand. 55x110 mm vane raw torque readings: V8 = 21.0/5.6 ft-lbs V9 = 12.5/3.0 ft-lbs	
	V9		71.00 71.64 - 72.00	Su=558/134 psf		47, 47, 46, 41			
75									

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.

Page 1 of 2 — Boring No.: BB-YR34-102

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary — Location: York, Maine — Boring No.: BB-YR34-102 — PIN: 11066.00

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	11D MV	24/14	75.00 - 77.00 75.20	1/1/8/13	9		Grey, wet, loose, fine SAND, some silt, little clay, trace medium sand, uniform. Attempt 55x110 mm vane: could not push	G#176650 A-4, SC-SM WC=20.3%
80	12D	24/24	80.00 - 82.00	WOR/WOR/7/11	7	-70.80	Grey, wet, soft, silty CLAY with black staining from 80.0-81.0' bgs. ——81.00 Grey, wet, loose, silty fine SAND, uniform from 81.0-82.0' bgs.	
90	13D	24/24	90.00 - 92.00	11/7/11/8	18		Grey, wet, medium dense, fine silty SAND, uniform. aWashed ahead of Casing to 98.1' bgs.	
100	14D	24/6	99.00 - 101.00	13/28/13/11	41		Grey, wet, dense, fine to coarse silty SAND. a214 blows for 0.6'. aWashed ahead to 105.5' bgs.	
105	R1	46/48	105.50 - 109.30	RQD = 88%		-95.30	——105.50 Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. At 108.3 ft bgs (El. -98.1) change to dark grey/green quenched andesite porphyry. R1:Core Times (min:sec) 105.5' - 106.5' (5:05) 106.5' - 107.5' (7:30) 107.5' - 108.5' (7:20) 108.5' - 109.3' (8:20) Recovery=100% Core Blocked	
110						-99.10	——109.30 Bottom of Exploration at 109.30 feet below ground surface.	

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.

Page 2 of 2 — Boring No.: BB-YR34-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1106(600)X

BRIDGE PLANS

PIN 11066.00

BRIDGE NO. 5848

	BY	DATE
PROJ. MANAGER	J. Wentworth	
DESIGN-DETAILED	K. MAGUIRE	MAR2004
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

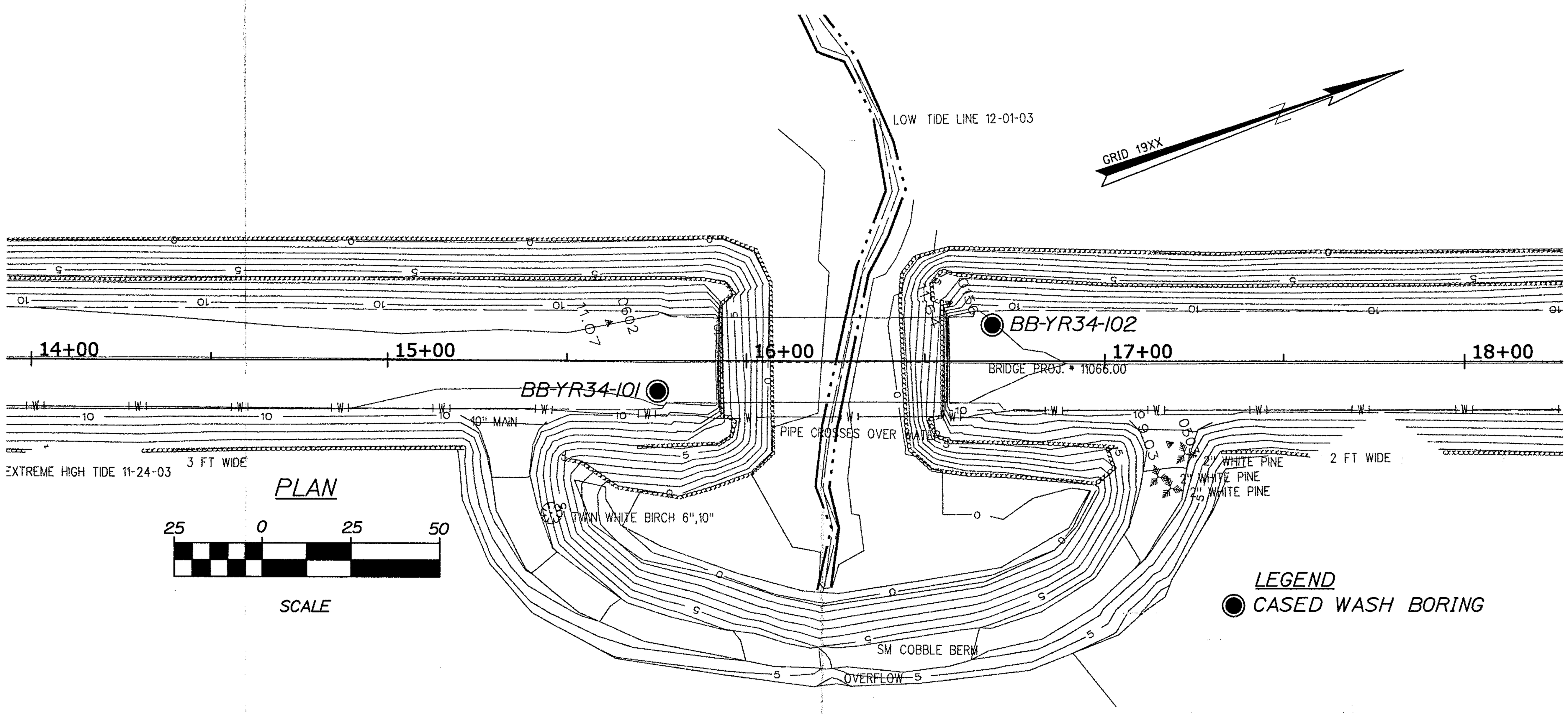
P.E. NUMBER

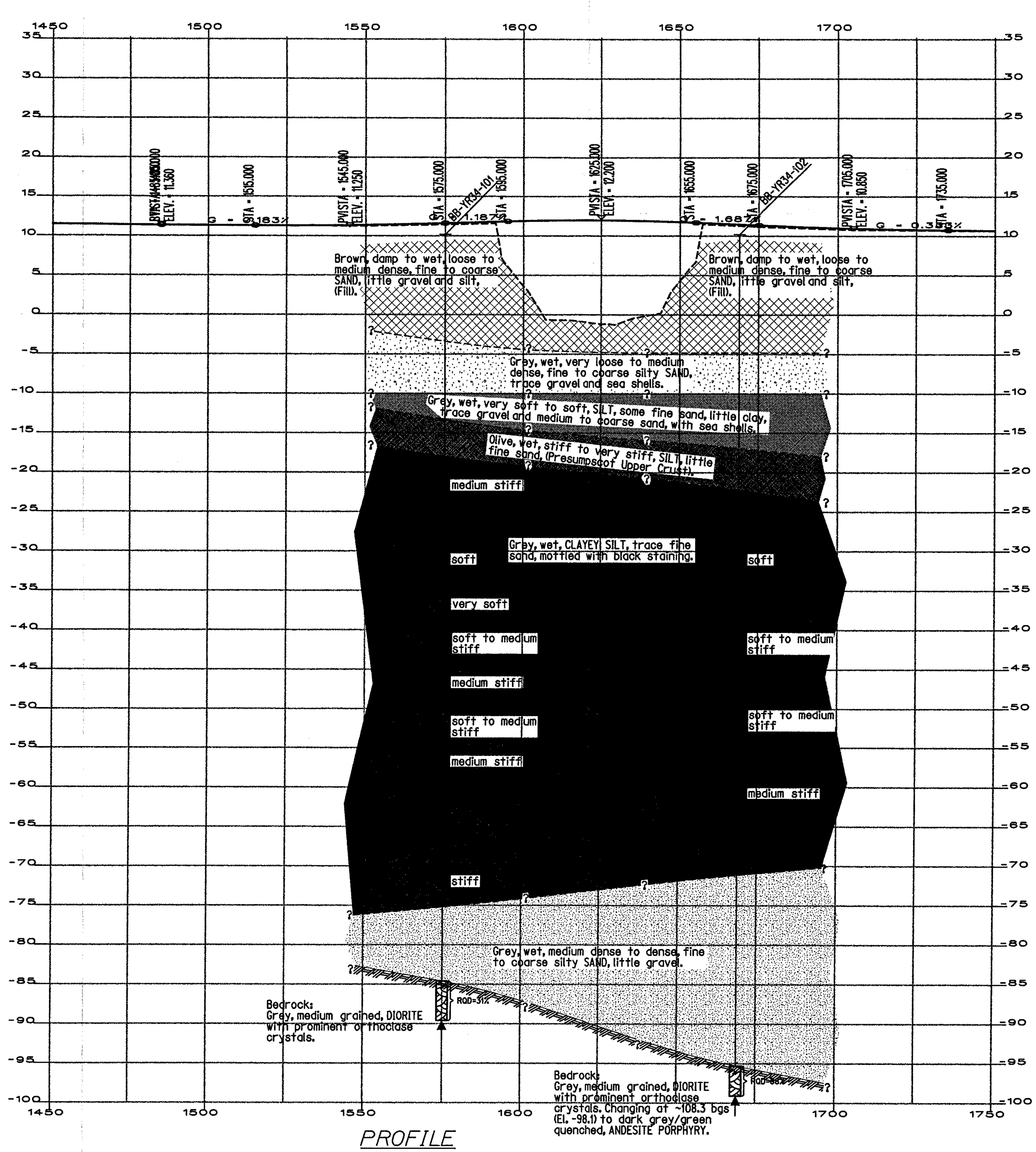
DATE

STATION 34
TIDAL ESTUARY
YORK COUNTY
YORK

BORING LOGS

SHEET NUMBER 3 OF 3





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

PROJ. MANAGER	J. Wentworth	BY	DATE
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	MAR2004
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

STATION 34
TIDAL ESTUARY
YORK YORK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1106(600)X

BRIDGE NO. 5848 PIN 11066.00 BRIDGE PLANS

SHEET NUMBER
2
OF 3

APPENDIX A

Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Station 34 Bridge over Tidal Estuary Location: York, Maine	Boring No.: BB-YR34-101 PIN: 11066.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/3/04-3/3/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	15+75, 9.2 Rt.	Casing ID/OD:	HW	Water Level*:	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.78		Pavement	
									0.4	
5	1D	24/24	5.0 - 7.0	7/15/12/16	27				Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/6	10.0 - 12.0	4/3/4/4	7	18			Brown, wet, loose, fine to coarse SAND, little gravel and silt, (Fill).	
						19				
						27				
						18	-2.80		13.0	
						26				
15						2				
	3D	24/2	16.0 - 18.0	5/3/1/4	4	11			Grey, wet, very loose, fine to coarse silty SAND, trace gravel and sea shells.	
						10				
						12				
						12	-9.80		20.0	
20	4D	24/20	20.0 - 22.0	2/WOH/WOH/WOH	---	16			Grey, wet, very soft, SILT, some fine sand, little clay, trace gravel, medium to coarse sand and sea shells.	G#176641 A-4, CL-ML WC=36.8%
						11				
						19				
						30	-12.80		23.0	
25						40				

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: Station 34 Bridge over Tidal Estuary		**Boring No.:**	BB-YR34-101
Location: York, Maine		**PIN:**	11066.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2
Operator:	C. Mann	**Datum:**	NGVD
Logged By:	K. Maguire	**Rig Type:**	CME 45C
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring
Boring Location:	15+75, 9.2 Rt.	**Casing ID/OD:**	HW
Auger ID/OD:	4.5" SSA	**Sampler:**	Standard Split Spoon
Hammer Wt./Fall:	140#/30"	**Core Barrel:**	NQ
Water Level*:	9.5' (Tidal)		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D/MV	24/20	25.0 - 27.0	4/7/10/13	17	69			Olive, wet, very stiff, SILT, little fine sand, (Presumpscot upper crust). Attempt 55x110 mm vane: could not push	
						69				
						69				
						63	-17.80		-28.0	
						69				
30	6D	24/23	30.0 - 32.0	3/2/3/3	5	78 aWA			Grey, wet, medium stiff, SILT, some clay, trace fine sand, mottled. aWashed ahead of casing.	G#176642 A-6, CL WC=31.7% LL=34 PL=20 PI=14
						70				
						64				
						64				
						50				
35	7D	24/22	35.0 - 37.0	2/2/1/WOH	3	52			Grey, wet, very loose, fine silty SAND, uniform, dilatent.	
						52				
						50				
						38				
						32				
40	8D V1	24/24	40.0 - 42.0 40.6 - 41.0	Push thru vane Su=259/125 psf		62			Grey, wet, soft, clayey SILT, trace fine sand layers, black staining. 55x110 mm vane raw torque readings: V1 = 5.8/2.8 ft-lbs V2 = 10.7/1.5 ft-lbs	G#176643 A-4, CL WC=36.8% LL=28 PL=19 PI=9
	V2		41.6 - 42.0	Su=478/67 psf		45				
						44				
						48				
						50				
45	9D V3	24/24	45.0 - 47.0 45.6 - 46.0	Push thru vane Su=192/67 psf		87			Similar to above, very soft. 55x110 mm vane raw torque readings: V3 = 4.3/1.5 ft-lbs V4 = 5.3/1.6 ft-lbs	
	V4		46.6 - 47.0	Su=237/71 psf		68				
						60				
						40				
50						47				

Remarks:

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* 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: Station 34 Bridge over Tidal Estuary Location: York, Maine	Boring No.: BB-YR34-101 PIN: 11066.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/3/04-3/3/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	15+75, 9.2 Rt.	Casing ID/OD:	HW	Water Level*:	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D V5	24/24	50.0 - 52.0 50.6 - 51.0	Push thru vane Su=437/80 psf		73			Grey, wet, soft to medium stiff, clayey SILT with occasional fine sand layers, black staining. 55x110 mm vane raw torque readings: V5 = 9.8/1.8 ft-lbs V6 = 13.5/1.7 ft-lbs	
	V6		51.6 - 52.0	Su=603/76 psf		84				
						85				
						69				
						69				
55	MD V7	24/0	55.0 - 57.0 55.6 - 56.0	Push thru vane Su=545/76 psf		94			No recovery, similar soils on spoon, medium stiff. 55x110 mm vane raw torque readings: V7 = 12.2/1.7 ft-lbs V8 = 12.8/1.3 ft-lbs	
	V8		56.6 - 57.0	Su=571/58 psf		57				
						47				
						34				
						31				
60	11D V9	24/24	60.0 - 62.0 60.6 - 61.0	Push thru vane Su=518/89 psf		49			Grey, wet, soft to medium stiff, SILT, some clay, with little fine sand layers, black staining. 55x110 mm vane raw torque readings: V9 = 11.6/2.0 ft-lbs V10 = 11.0/2.1 ft-lbs	G#176644 A-4, CL-ML WC=28.0% LL=23 PL=17 PI=6
	V10		61.6 - 62.0	Su=491/94 psf		40				
						41				
						38				
						32				
65	12D V11	24/24	65.0 - 67.0 65.6 - 66.0	Push thru vane Su=603/143 psf		52			Similar to above, medium stiff. 55x110 mm vane raw torque readings: V11 = 13.5/3.2 ft-lbs V12 = 13.9/3.7 ft-lbs	
	V12		66.6 - 67.0	Su=621/165 psf		41				
						46				
						40				
						50				
70	13D V13	24/24	70.0 - 72.0 70.6 - 71.0	Push thru vane Su=763/89 psf		74			Grey, wet, medium stiff, clayey SILT, little fine sand with fine sand layers. 55x110 mm vane raw torque readings: V13 = 17.1/2.0 ft-lbs V14 = 14.4/4.8 ft-lbs	
	V14		71.6 - 72.0	Su=643/214 psf		61				
						61				
						58				
75						53				

Remarks:

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* 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: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 11066.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	15+75, 9.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	14D V15	24/3	75.0 - 77.0 75.6 - 76.0	WOR/4/5/6 Su=808/304 psf	9	64			Similar to above, medium stiff. 55x110 mm vane raw torque readings: V15 = 18.1/6.8 ft-lbs MV = could not push	
	MV		76.2 - 76.2			55				
						62				
						73				
						128				
80	15D	24/20	80.0 - 82.0	28/58/54/24	112	78 bWA			Grey, wet, very stiff, SILT, some fine sand, trace clay. bWashed Ahead of Casing.	G#176645 A-2-4, SM WC=21.5%
						52				
						39				
						25				
						26	-74.80		85.0	
85	16D	24/24	85.0 - 87.0	WOR/8/11/10	19	62			Grey, wet, medium dense, fine silty SAND, uniform.	
						66				
						55				
						55				
						98				
90	17D	24/6	90.0 - 92.0	18/4/14/9	18	115			Grey, wet, medium dense, fine to coarse silty SAND, little gravel.	G#176646 A-4, CL-ML WC=18.2%
						79				
						82				
						93				
	R1	60/60	94.8 - 99.8	RQD = 31%		c156 NQ	-84.60		c156 blows for 0.4'. 94.8	
95									Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. R1:Core Times (min:sec) 94.8' - 95.8' (5:05) 95.8' - 96.8' (5:50) 96.8' - 97.8' (6:10) 97.8' - 98.8' (6:05) 98.8' - 99.8' (5:15) Recovery=100%	
100							-89.60		99.8	

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: Station 34 Bridge over Tidal Estuary Location: York, Maine	Boring No.: BB-YR34-101 PIN: 11066.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/3/04-3/3/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	15+75, 9.2 Rt.	Casing ID/OD:	HW	Water Level*:	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
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Definitions:
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information								Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log		
100									**Bottom of Exploration at 99.8 feet below ground surface.**	
105										
110										
115										
120										
125										

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.

Boring No.: BB-YR34-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-102
Location:	York, Maine			**PIN:**	11066.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	16+68.8, 10.0 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.62		Pavement — 0.6	
5	1D	24/4	5.0 - 7.0	6/6/17/22	23				Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/9	10.0 - 12.0	2/6/11/7	17				Brown, wet, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
15	3D	24/17	15.0 - 17.0	6/8/5/7	13	24	-4.80		15.0 — Grey, wet, medium dense, fine silty SAND with sea shells.	
						14				
						18				
						17				
						17				
20	4D	24/16	20.0 - 22.0	2/2/2/2	4	37	-9.80		20.0 — Grey, wet, soft, fine sandy SILT, little clay, trace gravel and medium to coarse sand with sea shells.	G#176647 A-4, CL-ML WC=31.6%
						24				
						23				
						23				
						24				
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary		**Boring No.:**	BB-YR34-102	
Location:	York, Maine		**PIN:**	11066.00	
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	16+68.8, 10.0 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/24	25.0 - 27.0	1/WOH/WOH/2	---	32			Grey, wet, soft, SILT, little clay, trace fine sand.	G#176648 A-4, ML WC=45.5% LL=37 PL=33 PI=4
						31				
	V1		27.6 - 28.0	aSu=1964/- psf		36	-16.80		—27.0 55x110 mm vane raw torque readings: V1 = 44/-- ft-lbs aVane reached maximum torque reading without shearing, no remolded was attempted.	
						88				
						126				
30	6D	24/14	30.0 - 32.0	8/4/7/9	11	45			Olive, wet, stiff, SILT, (Presumpscot upper crust).	
						44				
						38	-21.80		—32.0	
						39				
						34				
35						34				
						38				
						38				
						32				
						30				
40	7D V2	24/24	40.0 - 42.0 40.6 - 41.0	Push thru vane Su=446/67 psf		62			Grey, wet, soft, clayey SILT, trace fine sand, black staining. 55x110 mm vane raw torque readings: V2 = 10.0/1.5 ft-lbs V3 = 8.8/1.1 ft-lbs	
	V3		41.6 - 42.0	Su=393/49 psf		50				
						37				
						34				
						30				
45						34				
						30				
						32				
						31				
						30				
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: Station 34 Bridge over Tidal Estuary Location: York, Maine	Boring No.: BB-YR34-102 PIN: 11066.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/9/04-3/9/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	16+68.8, 10.0 Lt.	Casing ID/OD:	NW	Water Level*:	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	8D	24/24	50.0 - 52.0	Push thru vane		54			Grey, wet, soft, silty CLAY with trace fine sand layers and black staining. 55x110 mm vane raw torque readings: V4 = 12.1/1.6 ft-lbs V5 = 7.4/1.2 ft-lbs	G#176649 A-6, CL WC=40.0% LL=32 PL=20 PI=12
	V4		50.6 - 51.0	Su=540/71 psf						
	V5		51.6 - 52.0	Su=330/54 psf		41				
						39				
						35				
						33				
55						37				
						35				
						33				
						33				
						34				
60	9D	24/24	60.0 - 62.0	Push thru vane		73			Similar to above, soft to medium stiff. 55x110 mm vane raw torque readings: V6 = 5.4/2.5 ft-lbs V7 = 12.0/2.2 ft-lbs	
	V6		60.6 - 61.0	Su=241/112 psf						
	V7		61.6 - 62.0	Su=536/98 psf		56				
						53				
						45				
						42				
65						48				
						40				
						45				
						38				
						34				
70	10D	24/24	70.0 - 72.0	Push thru vane		52			Grey, wet, medium stiff, clayey SILT, little fine sand. 55x110 mm vane raw torque readings: V8 = 21.0/5.6 ft-lbs V9 = 12.5/3.0 ft-lbs	
	V8		70.6 - 71.0	Su=937/250 psf						
	V9		71.6 - 72.0	Su=558/134 psf		47				
						47				
						46				
75						41				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary	**Boring No.:**	BB-YR34-102
Location:	York, Maine	**PIN:**	11066.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	16+68.8, 10.0 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample Information									
75	11D MV	24/14	75.0 - 77.0 75.2 - 75.2	1/1/8/13	9	44			Grey, wet, loose, fine SAND, some silt, little clay, trace medium sand, uniform. Attempt 55x110 mm vane: could not push	G#176650 A-4, SC-SM WC=20.3%
						46				
						48				
						56				
						53				
80	12D	24/24	80.0 - 82.0	WOR/WOR/7/11	7	87			Grey, wet, soft, silty CLAY with black staining from 80.0-81.0' bgs.	
						76	-70.80		81.0 Grey, wet, loose, silty fine SAND, uniform from 81.0-82.0' bgs.	
						72				
						71				
						67				
85						81				
						166				
						138				
						157				
						184				
90	13D	24/24	90.0 - 92.0	11/7/11/8	18	90 bWA			Grey, wet, medium dense, fine silty SAND, uniform. bWashed Ahead of Casing to 98.1' bgs.	
						81				
						58				
						53				
						82				
95						103				
						66				
						64				
						70				
100	14D	24/6	99.0 - 101.0	13/28/13/11	41	64			Grey, wet, dense, fine to coarse silty SAND.	

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Station 34 Bridge over Tidal Estuary **Location:** York, Maine	**Boring No.:** BB-YR34-102 **PIN:** 11066.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	16+68.8, 10.0 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100						51				
						86				
						194				
						c214			c214 blows for 0.8'.	
						dWA			dWashed Ahead to 105.5' bgs.	
105	R1	46/46	105.5 - 109.3	RQD = 88%		NQ	-95.30		105.5 Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. At 108.3 ft bgs (El. -98.1) change to dark grey/green quenched andesite porphyry. R1:Core Times (min:sec) 105.5' - 106.5' (5:05) 106.5' - 107.5' (7:30) 107.5' - 108.5' (7:20) 108.5' - 109.33' (8:20) Recovery=100% Core Blocked	
							-99.10		109.3	
110									**Bottom of Exploration at 109.3 feet below ground surface.**	
115										
120										
125										

Remarks:

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

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APPENDIX B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): York **Project Number: 11066.00**

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-YR34-101, 4D	15+75	9.2 Rt.	20.0-22.0	176641	1	36.8			CL-ML	A-4	IV
BB-YR34-101, 6D	15+75	9.2 Rt.	30.0-32.0	176642	1	31.7	34	14	CL	A-6	III
BB-YR34-101,8D	15+75	9.2 Rt.	40.0-42.0	176643	1	36.8	28	9	CL	A-4	IV
BB-YR34-101, 11D	15+75	9.2 Rt.	60.0-62.0	176644	1	28.0	23	6	CL-ML	A-4	IV
BB-YR34-101, 15D	15+75	9.2 Rt.	80.0-82.0	176645	1	21.5			SM	A-2-4	II
BB-YR34-101, 17D	15+75	9.2 Rt.	90.0-92.0	176646	1	18.2			CL-ML	A-4	IV
BB-YR34-102, 4D	16+68.8	10.0 Lt.	20.0-22.0	176647	2	31.6			CL-ML	A-4	IV
BB-YR34-102, 5D	16+68.8	10.0 Lt.	25.0-27.0	176648	2	45.4	37	4	ML	A-4	IV
BB-YR34-102, 8D	16+68.8	10.0 Lt.	50.0-52.0	176649	2	40.0	32	12	CL	A-6	III
BB-YR34-102, 11D	16+68.8	10.0 Lt.	75.0-77.0	176650	2	20.3			SC-SM	A-4	III

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

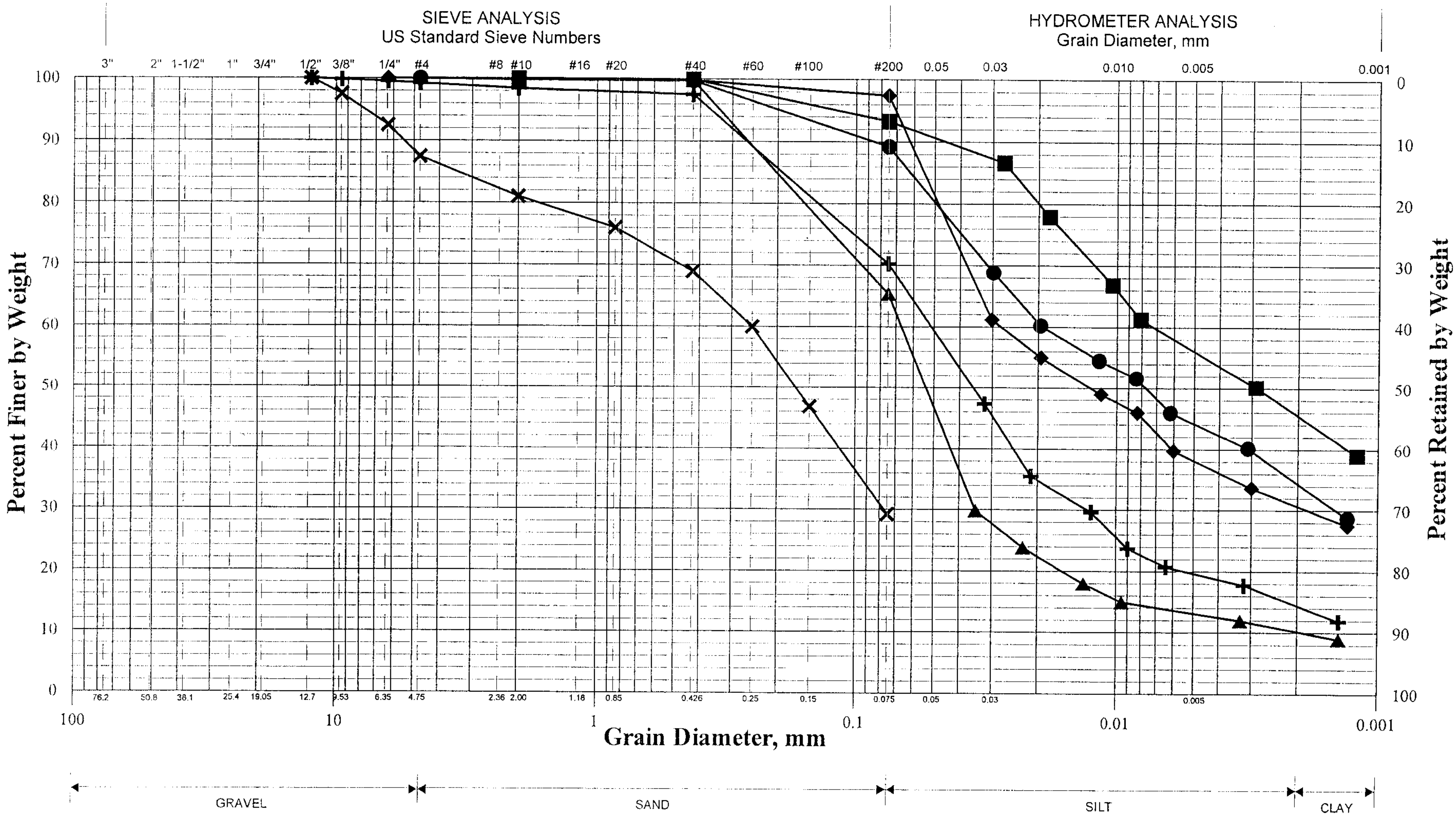
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

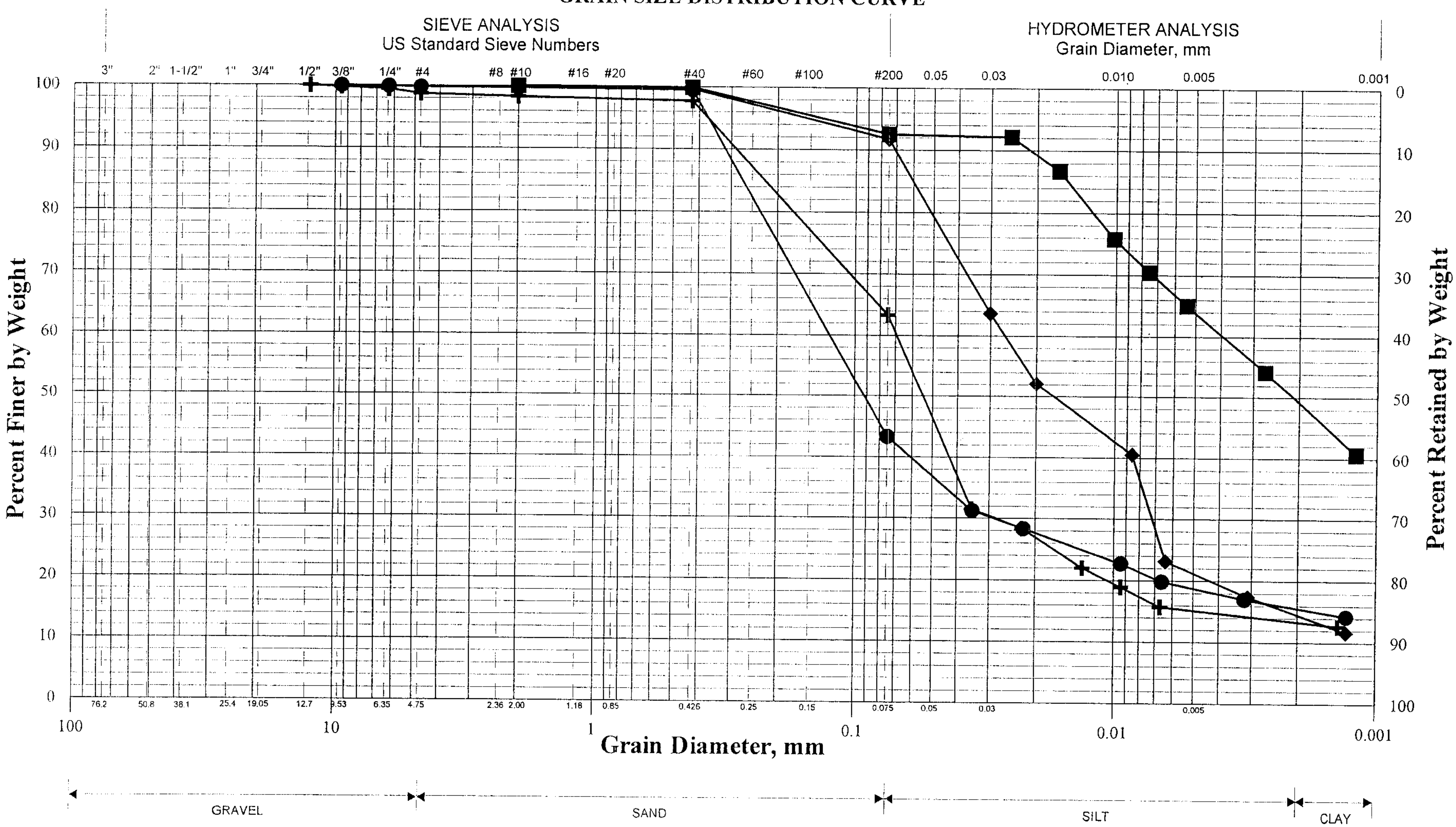


UNIFIED CLASSIFICATION

	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR34-101	4D	20.0-22.0	SILT, some fine sand, little clay, trace gravel and medium to coarse sand.	36.8			
◆	BB-YR34-101	6D	30.0-32.0	SILT, some clay, trace fine sand.	31.7	34	20	14
■	BB-YR34-101	8D	40.0-42.0	Clayey SILT, trace fine sand.	36.8	28	19	9
●	BB-YR34-101	11D	60.0-62.0	SILT, some clay, little sand.	28.0	23	17	6
▲	BB-YR34-101	15D	80.0-82.0	SILT, some fine sand, trace clay.	21.5			
×	BB-YR34-101	17D	90.0-92.0	Fine to coarse silty SAND, little gravel.	18.2			

PIN: 11066.00
Town: York
Reported by: T. White
Date: 3/26/04

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

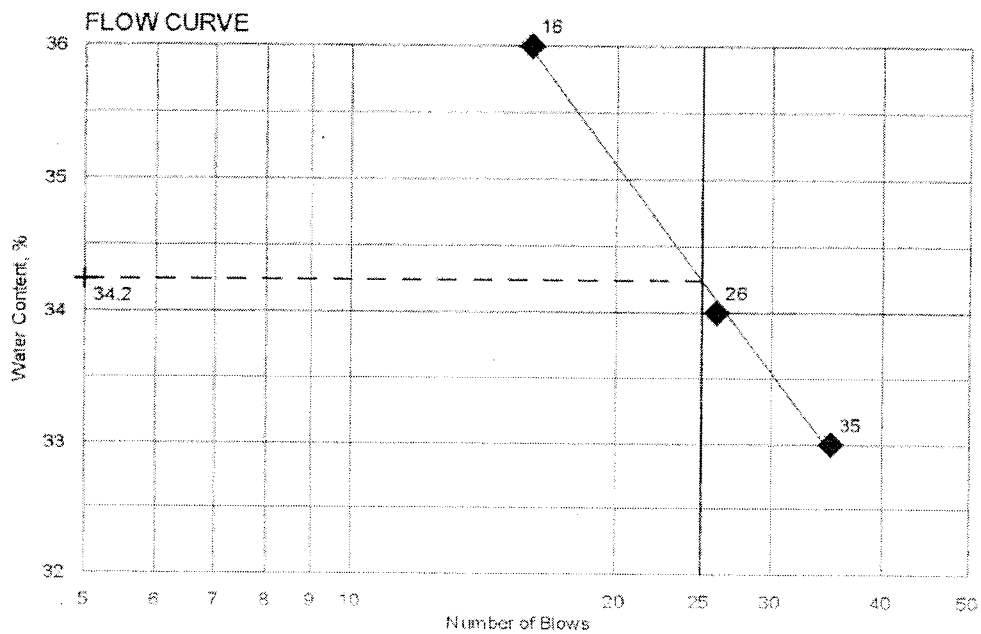
	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR34-102	4D	20.0-22.0	Fine sandy SILT, little clay, trace gravel and medium to coarse sand.	31.6			
◆	BB-YR34-102	5D	25.0-27.0	SILT, little clay, trace fine sand.	45.5	37	33	4
■	BB-YR34-102	8D	50.0-52.0	Silty CLAY, trace fine sand.	40.0	32	20	12
●	BB-YR34-102	11D	75.0-77.0	Fine SAND, some silt, little clay, trace medium sand.	20.3			
▲	---							
✕	---							

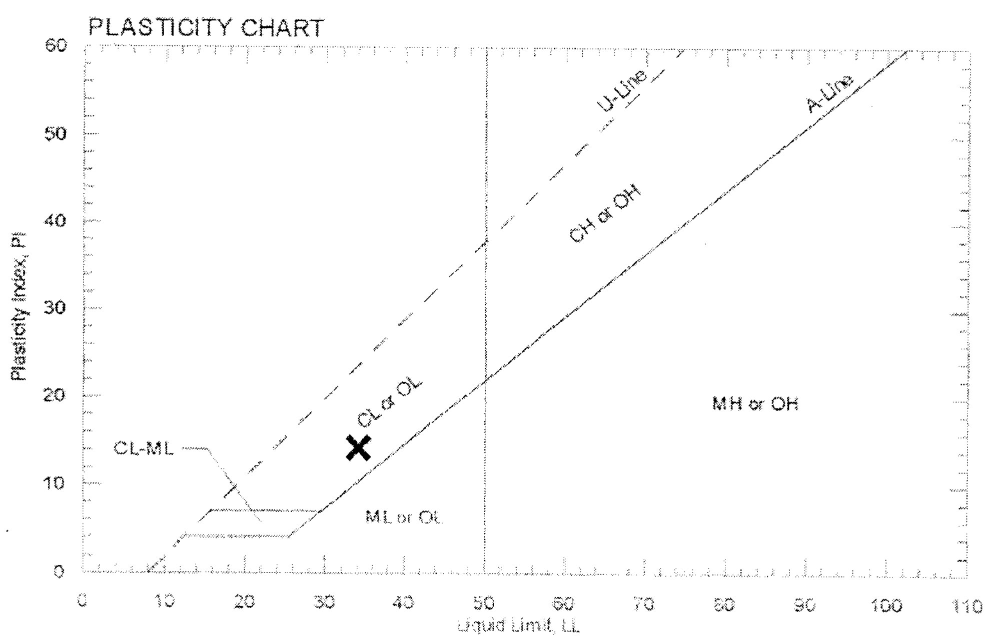
PIN: 11066.00
Town: York
Reported by: T. White
Date: 3/26/04

SHEET NO. 2

State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

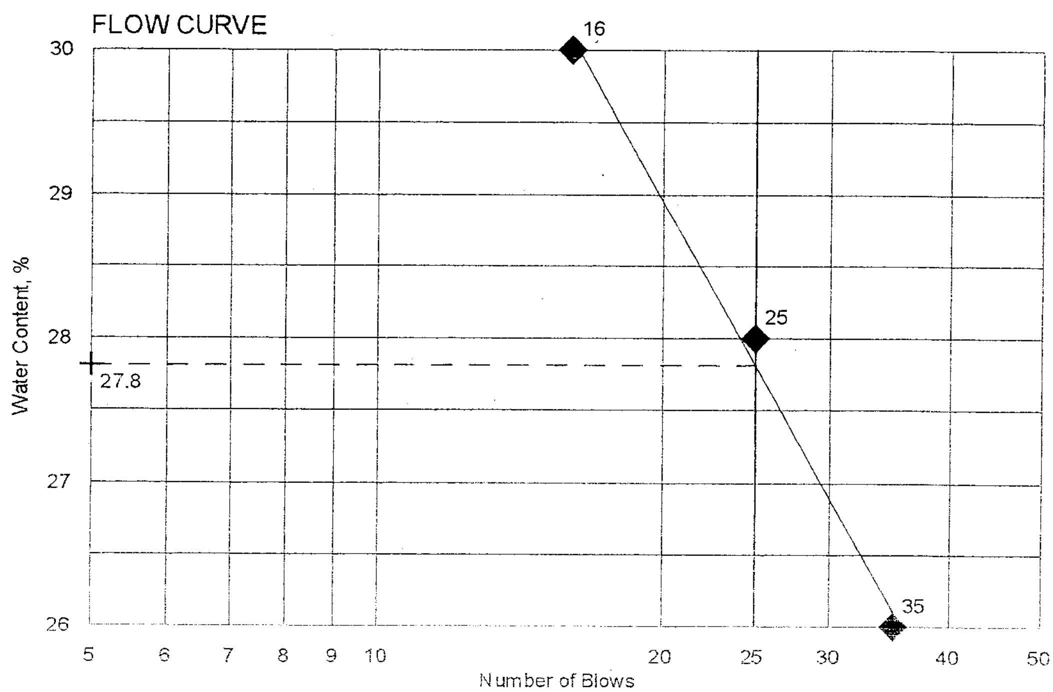
TOWN	York	Reference No.	176642
PIN	11066.00	Natural water content (%)	31.7
Date	4/21/2004	Plastic limit	20
Boring No.	BB-YR34-101	Liquid limit	34
Station 15+75, 9.2Rt		Plasticity index	14
Depth/Sample No.	30-32'/6D	Reported by	KLD

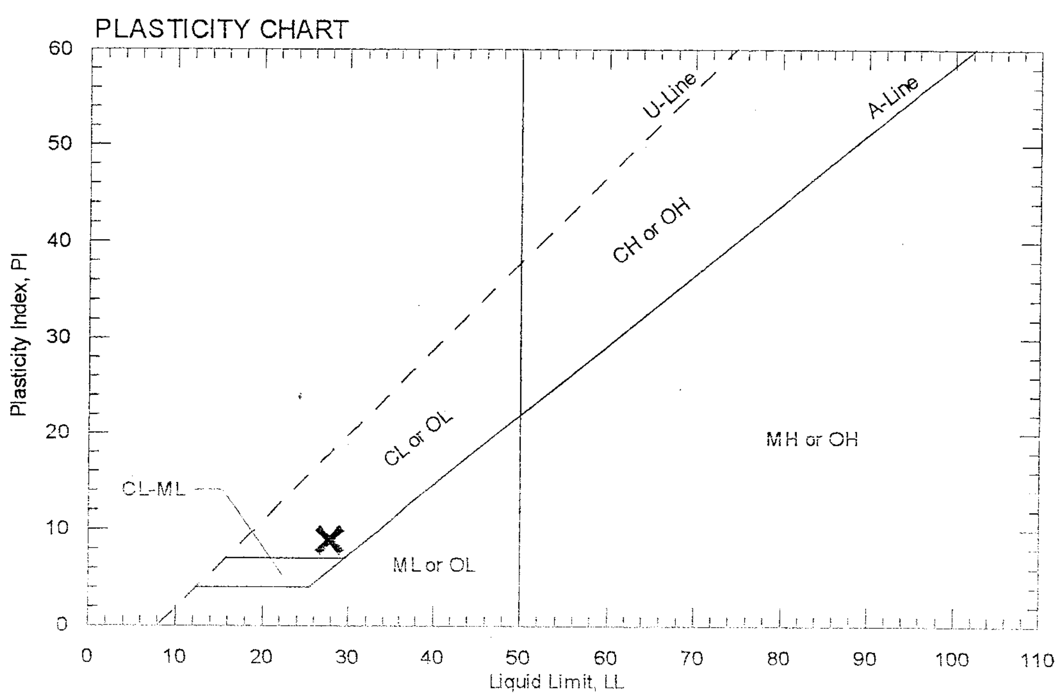




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

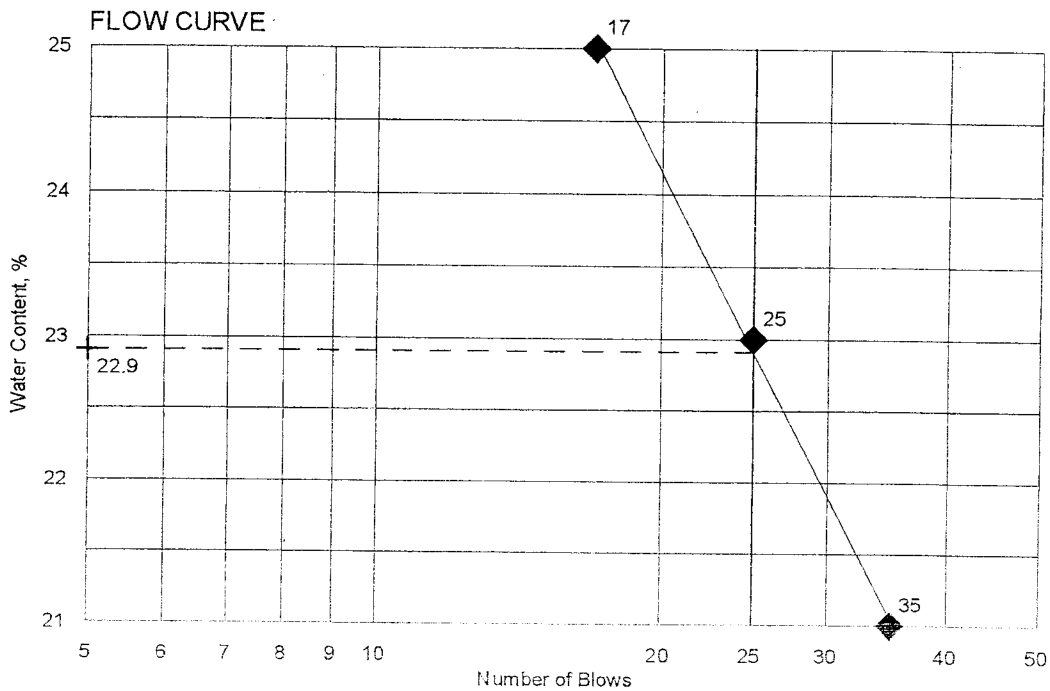
TOWN	YORK	Reference No.	176643
PIN	11066.00	Natural water content (%)	36.8
Date	4/23/2004	Plastic limit	19
Boring No.	BB-YR34-101	Liquid limit	28
Station	15+75, 9.2 Rt	Plasticity index	9
Depth/Sample No.	40.0-42.0/8D	Reported by	B. D. FOGG

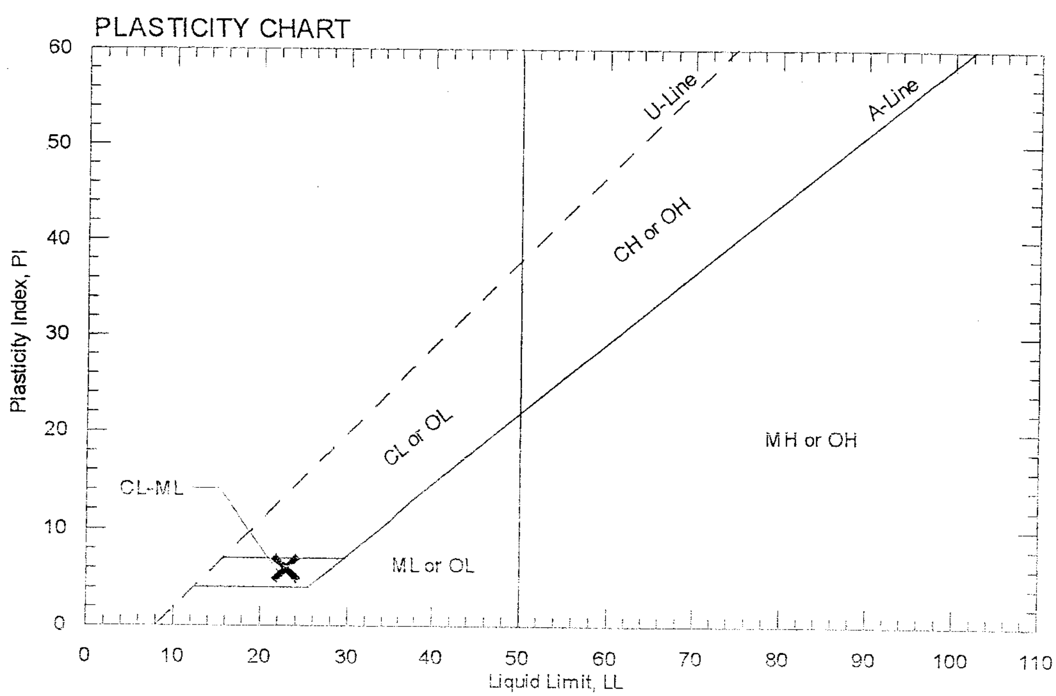




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

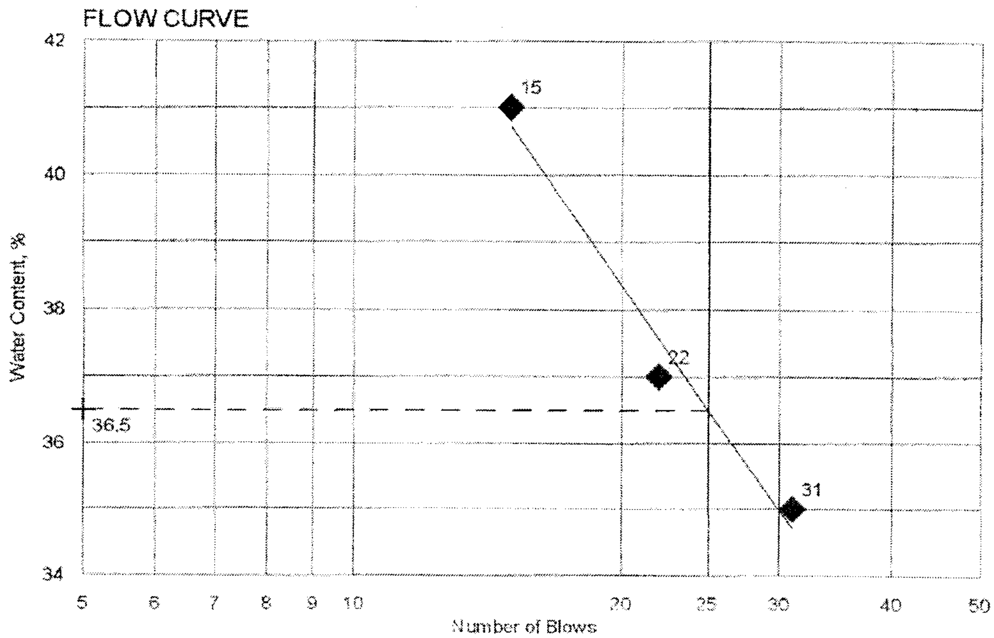
TOWN	YORK	Reference No.	176644
PIN	11066.00	Natural water content (%)	28.0
Date	4/23/2004	Plastic limit	17
Boring No.	BB-YR34-101	Liquid limit	23
Station	15+75, 9.2 Rt	Plasticity index	6
Depth/Sample No.	60.0-62.0/11D	Reported by	B. D. FOGG

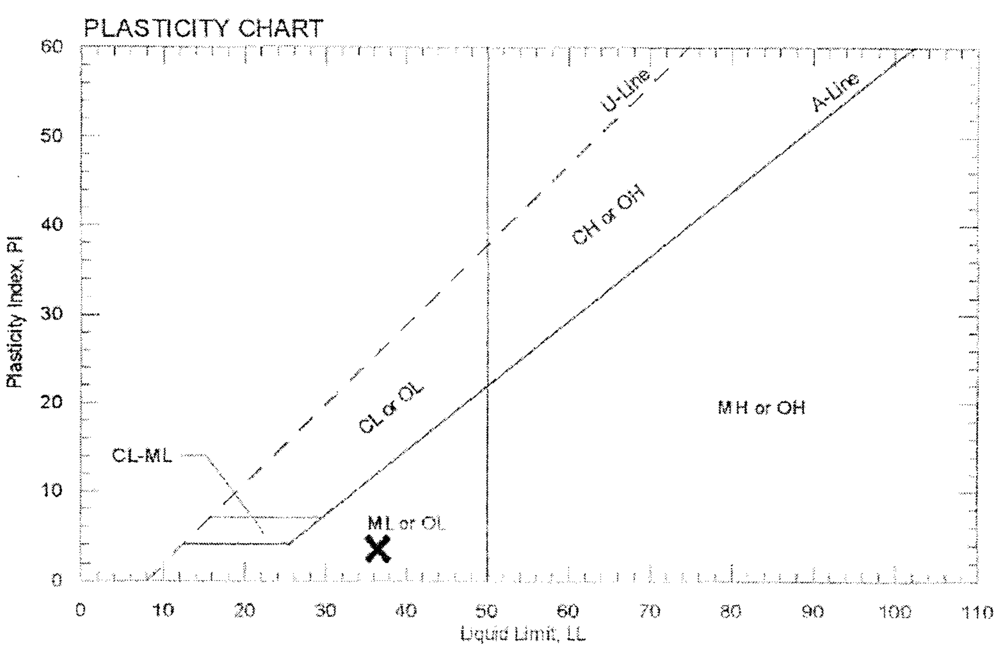




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

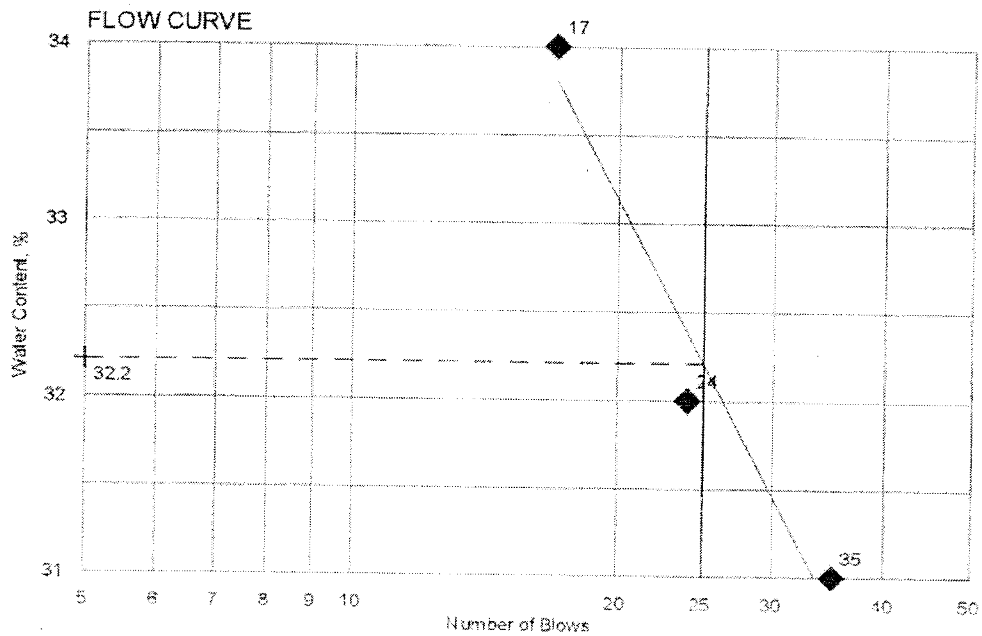
TOWN	York	Reference No.	176648
PIN	11066.00	Natural water content (%)	45.5
Date	4/21/2004	Plastic limit	33
Boring No.	BB-YR34-102	Liquid limit	36
Station	16+68.8, 10.0 Rt	Plasticity index	3
Depth/Sample No.	25-27'/5D	Reported by	KLD

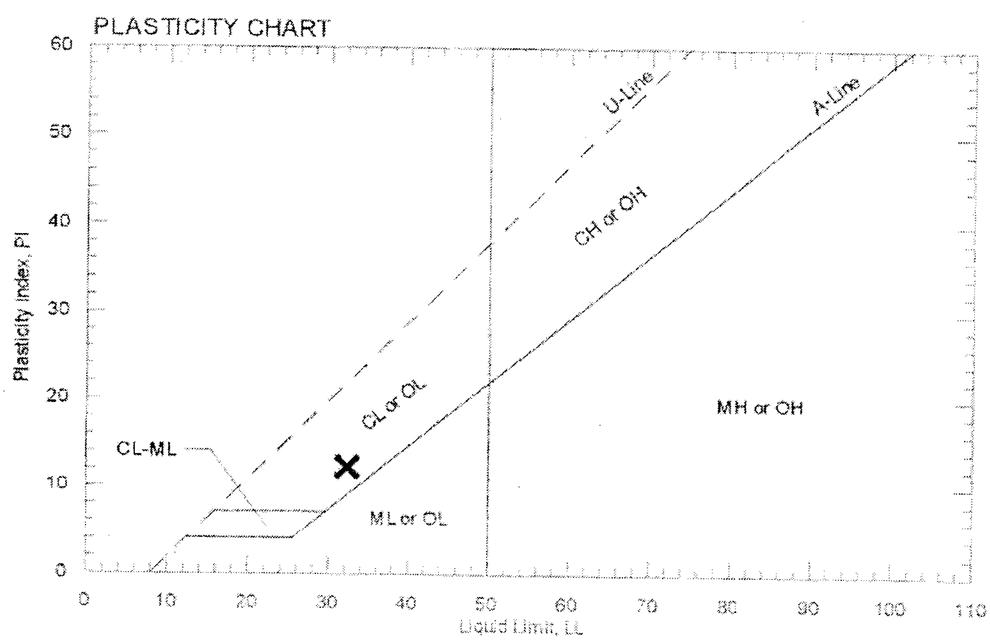




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	York	Reference No.	176649
PIN	11066.00	Natural water content (%)	40.0
Date	4/21/2004	Plastic limit	20
Boring No.	BB-YR34-102	Liquid limit	32
Station	16+68.8, 10.0 Rt	Plasticity index	12
Depth/Sample No.	50-52'/8D	Reported by	KLD





APPENDIX C

Calculations

Definition of Units:

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := g \cdot \left(\frac{ton}{ft^2}\right)$

$ksf := \frac{kip}{ft^2}$ $ft = 0.305\ m$ $in = 0.025\ m$ $MPa := 1000 \cdot kPa$ $ksi := \frac{kip}{in^2}$ $kip := 1000 \cdot lbf$

LIQUIDITY INDEX

wc is close to LL	Soil is normally consolidated
wc is close to PL	Soil is some-to-heavily overconsolidated
wc is intermediate	Soil is overconsolidated
wc is greater than LL	Soil is on the verge of being a viscous liquid

BB-YR34-102 Sample 5D Lab Results: $wc := 45.4$ $PL := 33$ $LL_5 := 37$

$$LI := \frac{wc - PL}{LL_5 - PL} \qquad LI = 3.1$$

BB-YR34-101 Sample 6D Lab Results: $wc := 31.7$ $PL := 20$ $LL_6 := 34$

$$LI := \frac{wc - PL}{LL_6 - PL} \qquad LI = 0.836$$

BB-YR34-101 Sample 8D Lab Results: $wc := 36.8$ $PL := 19$ $LL_8 := 28$

$$LI := \frac{wc - PL}{LL_8 - PL} \qquad LI = 1.978$$

BB-YR34-101 Sample 11D Lab Results: $wc := 28$ $PL := 17$ $LL_{11} := 23$

$$LI := \frac{wc - PL}{LL_{11} - PL} \qquad LI = 1.833$$

BB-YR34-102 Sample 8D Lab Results: $wc := 40$ $PL := 20$ $LL_8 := 32$

$$LI := \frac{wc - PL}{LL_8 - PL} \qquad LI = 1.667$$

Frost Protection:

MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
York, Maine
DFI = 1100 degree-days

From the lab testing: soils are coarse grained with a water content = ~30%

From Table 5-1 MaineDOT BDG

$\text{Frost\_depth} := 49.8\text{in} \qquad \text{Frost\_depth} = 4.15\,\text{ft}$

Use 4.0 feet

Note: The final depth of footing embedment may be controlled by the scour susceptibility of the foundation material and may, in fact, be deeper than the depth required for frost protection.

COMPUTE STRUCTURAL CAPACITY OF H-PILES

Using 50 ksi steel and FS = 4 for integral piles per BDG (0.25Fy)

HP 12 x 53
HP 14 x 73 Note: All matrices set up in this order
HP 14 x 89

$\sigma_a := 0.25 \cdot 50 \cdot \text{ksi}$

$\sigma_a = 12.5\,\text{ksi}$

$$\text{Area}_l := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix} \cdot \text{in}^2$$

$$Q_{all} := \sigma_a \cdot \text{Area}_l \qquad Q_{all} = \begin{pmatrix} 193.75 \\ 267.5 \\ 326.25 \end{pmatrix} \text{kip}$$

COMPUTE GEOTECHNICAL CAPACITY OF H-PILES

H pile Capacity - end bearing on rock assume driven through compressive soils and to bedrock

HP 12 x 53
HP 14 x 73 Note: All matrices set up in this order
HP 14 x 89

Method 1: Geotechnical Capacity
Based on Unconfined Compressive Strength of bedrock

From Fang Second Edition Table 3.8:
sandstone compressive strength = 100 - 1,800 kg/cm2
use q_{uc} = 1,200 kg/cm2 = 17,000 psi

$$1200\cdot\frac{kgf}{cm^2} = 1.707\cdot10^4 \, psi$$

$$Area_1 := \begin{bmatrix}15.5\\21.4\\26.1\end{bmatrix}\cdot in^2 \qquad Area_1 = \begin{bmatrix}0.108\\0.149\\0.181\end{bmatrix} ft^2$$

$$q_{uc} := 17000\cdot psi$$

$$Q_{ult1} := q_{uc}\cdot Area_1 \qquad Q_{ult1} = \begin{bmatrix}263.5\\363.8\\443.7\end{bmatrix} kip$$

$$Q_{all\_tip} := \frac{Q_{ult1}}{2.25} \qquad Q_{all\_tip} = \begin{bmatrix}117.111\\161.689\\197.2\end{bmatrix} kip$$

Method 2: Geotechnical Capacity by Goodman's Method
Based on Unconfined Compressive Strength of Bedrock
Reference: Principles of Foundation Engineering, BM Das, Second Edition

$$\phi := 32\cdot deg \qquad N_\phi := \left(\tan\left(45\cdot deg + \frac{\phi}{2}\right)\right)^2 \qquad N_\phi = 3.255$$

$$q_{pt\_ult} := \left(\frac{q_{uc}}{5}\right)\cdot\left(N_\phi + 1\right) \qquad q_{pt\_ult} = 14.466\, ksi$$

$$q_{pt\_all} := \frac{q_{pt\_ult}}{2.25} \qquad q_{pt\_all} = 6.429\, ksi$$

$$Q_{pt\_all} := q_{pt\_all}\cdot Area_1 \qquad Q_{pt\_all} = \begin{bmatrix}99.652\\137.584\\167.801\end{bmatrix} kip$$

Method 3: Geotechnical Capacity by Goodman's Method
Based on bedrock condition - in this case poor RQD (0 - 31%) Mudstone
Reference: Pile Design and Construction Practice 4th Edition MJ Tomlinson

Low friction: 20-27 for schists, shales
Medium Friction 27-34 for sandstone, siltstone, gneiss, slate
High Friction: 34-40 for granite

$\phi_2 := 32 \cdot deg$ $\quad N_\phi := \left(\tan\left(45 \cdot deg + \frac{\phi_2}{2}\right)\right)^2$ $\quad N_\phi = 3.255$

$q_b := (2 \cdot N_\phi) \cdot \frac{q_{uc}}{5}$ $\quad q_b = 22.131 \cdot ksi$

$Q_{ult2} := q_b \cdot Area_1$ $\quad Q_{ult2} = \begin{bmatrix} 343.034 \\ 473.608 \\ 577.624 \end{bmatrix} \cdot kip$

$Q_{all\_tip2} := \frac{Q_{ult2}}{2.25}$ $\quad Q_{all\_tip2} = \begin{bmatrix} 152.459 \\ 210.492 \\ 256.722 \end{bmatrix} \cdot kip$

Method 4: Geotechnical Capacity
Allowable End Bearing Capacity $Q_{t,\ allow}$:
(Kulhway & Goodman, Fs = 2.25)

Reference: Pile Design and Construction Practice, M.J. Tomlinson, Fourth Edition

Method ignores side resistance - use Driven to assess side friction

Corrrections for wedge failure under a strip footing -
multiply the cN_c factor by 1.25 for a square pile
multiply γBN_γ factor by 0.8 for a square pile

Case I	Case II
For RQD of 0 - 70%:	For RQD of 70 - 100%:
$q_c = 0.33 \times Q_{uc}$	$q_c = 0.33$ to $0.88 \times Q_{uc}$
$c = 0.1 \times Q_{uc}$	$c = 0.1 \times Q_{uc}$
$\phi = 30$ degrees	$\phi = 30$ to 60 degrees

Assume Case I: as RQD = 31 to 88%

$q_{uc} = 1.7 \cdot 10^4 \cdot psi$

$c := 0.1 \cdot q_{uc}$ $\quad c = 1.7 \cdot 10^3 \cdot psi$

$\gamma := 150 \cdot pcf$ $\quad B := \begin{bmatrix} 12.05 \\ 14.59 \\ 14.70 \end{bmatrix} in$ $\quad B = \begin{bmatrix} 1.004 \\ 1.216 \\ 1.225 \end{bmatrix} \cdot ft$

based on Pells & Turner from Tomlinson page 140 - $\phi = 32$ $\quad N_c := 15.35$ $\quad N_q := 10.59$ $\quad N_\gamma := 17.31$

$D := 3\cdot in$ Depth of penetration into bedrock

$q_c := 0.33\cdot q_{uc}$ $\qquad q_c = 5.61 \times 10^3\, psi$

$$q_{ub} := 1.25\cdot c\cdot N_c + 0.8\cdot 0.5\cdot \gamma\cdot B\cdot N_\gamma + \gamma\cdot D\cdot N_q$$

$$q_{ub} = \begin{pmatrix} 32.629 \\ 32.63 \\ 32.63 \end{pmatrix} ksi \qquad Area_1 := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix}\cdot in^2$$

$$Q_{ult3} := \overrightarrow{\left(q_{ub}\cdot Area_1\right)} \qquad Q_{ult3} = \begin{pmatrix} 505.746 \\ 698.288 \\ 851.652 \end{pmatrix} kip$$

$$Q_{all\_tip4} := \frac{Q_{ult3}}{2.25} \qquad Q_{all\_tip4} = \begin{pmatrix} 224.776 \\ 310.35 \\ 378.512 \end{pmatrix} kip$$

Use DRIVEN to calculate the skin friction (Q_{side}) and apply FS = 2

$$Q_{side} := \begin{pmatrix} 94.77 \\ 127.78 \\ 139.42 \end{pmatrix}\cdot kip$$

$$Q_T := Q_{all\_tip4} + \frac{Q_{side}}{2.0} \qquad Q_T = \begin{pmatrix} 272.161 \\ 374.24 \\ 448.222 \end{pmatrix} kip$$

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\Y34-1253.DVN
Project Name: Station 34 Project Date: 05/12/2004
Project Client: York
Computed By: KMaguire
Project Manager: JWentworth

PILE INFORMATION

Pile Type: H Pile - HP12X53
Top of Pile: 0.00 ft
Perimeter Analysis: Pile
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	9.00 ft
	- Driving/Restrike	9.00 ft
	- Ultimate:	9.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	85.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	13.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesionless	7.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
3	Cohesive	3.00 ft	10.00%	118.00 pcf	500.00 psf	T-80 Clay
4	Cohesive	5.00 ft	10.00%	110.00 pcf	1900.00 psf	T-80 Clay
5	Cohesive	57.00 ft	5.00%	100.00 pcf	400.00 psf	T-80 Clay
6	Cohesionless	9.80 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
8.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
12.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
13.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
19.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
20.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
22.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
23.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
27.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
28.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
37.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
46.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
55.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
64.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
73.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
82.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
84.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
84.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
85.00 ft	0.00 Kips	0.39 Kips	0.39 Kips
85.01 ft	0.09 Kips	3.55 Kips	3.64 Kips
94.01 ft	86.78 Kips	3.55 Kips	90.34 Kips
94.79 ft	94.77 Kips	3.55 Kips	98.32 Kips

6

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\Y34-1473.DVN
Project Name: Station 34 Project Date: 05/12/2004
Project Client: York
Computed By: KMaguire
Project Manager: JWentworth

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 0.00 ft
Perimeter Analysis: Pile
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	9.00 ft
	- Driving/Restrike	9.00 ft
	- Ultimate:	9.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	85.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	13.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesionless	7.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
3	Cohesive	3.00 ft	10.00%	118.00 pcf	500.00 psf	T-80 Clay
4	Cohesive	5.00 ft	10.00%	110.00 pcf	1900.00 psf	T-80 Clay
5	Cohesive	57.00 ft	5.00%	100.00 pcf	400.00 psf	T-80 Clay
6	Cohesionless	9.80 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
8.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
12.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
13.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
19.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
20.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
22.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
23.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
27.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
28.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
37.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
46.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
55.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
64.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
73.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
82.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
84.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
84.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
85.00 ft	0.00 Kips	0.54 Kips	0.54 Kips
85.01 ft	0.12 Kips	4.90 Kips	5.03 Kips
94.01 ft	117.02 Kips	4.90 Kips	121.92 Kips
94.79 ft	127.78 Kips	4.90 Kips	132.68 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\Y34-1489.DVN
Project Name: Station 34 Project Date: 05/12/2004
Project Client: York
Computed By: KMaguire
Project Manager: JWentworth

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Pile
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	9.00 ft
	- Driving/Restrike	9.00 ft
	- Ultimate:	9.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	85.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	13.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesionless	7.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
3	Cohesive	3.00 ft	10.00%	118.00 pcf	500.00 psf	T-80 Clay
4	Cohesive	5.00 ft	10.00%	110.00 pcf	1900.00 psf	T-80 Clay
5	Cohesive	57.00 ft	5.00%	100.00 pcf	400.00 psf	T-80 Clay
6	Cohesionless	9.80 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
8.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
12.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
13.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
19.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
20.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
22.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
23.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
27.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
28.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
37.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
46.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
55.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
64.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
73.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
82.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
84.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
84.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
85.00 ft	0.00 Kips	0.65 Kips	0.65 Kips
85.01 ft	0.13 Kips	5.98 Kips	6.11 Kips
94.01 ft	127.68 Kips	5.98 Kips	133.66 Kips
94.79 ft	139.42 Kips	5.98 Kips	145.40 Kips

Earth Pressures:

Cast-in-place integral abutments shall be designed to withstand a maximum lateral applied load equal to the passive earth pressure. A passive earth presure coefficient (K_p) should be calculated using Coulomb Theory.

Coulomb Theory From AASHTO Standard Specifications for Highway Bridges Sixteenth Edition 1996 on Figure 5.5.2.A page 122.

$\phi := 32 \cdot deg$ $\quad \delta := \left(\frac{2}{3} \cdot \phi\right)$ $\quad \beta := 90 \cdot deg$ $\quad \alpha := 0 \cdot deg$

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

$$K_p = 7.333$$

Rankine Theory from Das Principles of Foundation Engineering Second Edition Eq. 5.23

$\phi := 32 \cdot deg$

$$K_{pr} := \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^2$$

$$K_{pr} = 3.255$$

Bearing Capacity: Native Soils

For any footing founded on native sands or fill.

Method I. Presumptive Bearing Capacity

Based on NavFac DM 7.2 pg 142-143 Table 1 - "Presumptive Values of Allowable Bearing Pressures for Spread Foundations"

Type of Bearing Material:	Consistency In Place:	Allowable Bearing Pressure tons per square foot:	Recommended value:
Coarse to medium, sand	Very compact	4 to 6	4 tsf
sand with little gravel	Medium to compact	2 to 4	3 tsf
	loose	1 to 3	1.5 tsf

Assume medium dense conditions $bearing\_capacity := 3 \cdot tsf$ $bearing\_capacity = 6 \cdot ksf$
Say 3 tsf

Method II. Bearing Capacity by Terzaghi

Assumed parameters for the native sand and fill layer:

$\gamma := 120 \cdot pcf$ $\phi := 32 \cdot deg$ from Foundation Analysis and Design, Bowles 4th Edition Table 3-4 pg 141

$c := 0\ psf$

$\gamma_1 := 120 \cdot pcf - 62.4 \cdot pcf$

$\gamma_1 = 57.6 \cdot pcf$ unit weight of native granular soils = 120 pcf less 62.4 pcf unit weight of water for effective unit weight

Assume footing width of 5.0 ft

$B := 5 \cdot ft$

From Bowles 4th Edition Table 4-2 for $\phi = 0$

$N_q := 29.5$ $N_c := 44.9$ $N_\gamma := 27.85$

From Bowles 4th Edition Table 4-1
Assume strip footing:

$s_c := 1.0$ $s_\gamma := 1.0$

Assume footing embedment, D_f of 4 feet

$D_f := 4 \cdot ft$ $q_{bar} := \gamma_1 \cdot D_f$ $q_{bar} = 230.4 \cdot psf$

$$q_{ult} := c \cdot N_c \cdot s_c + q_{bar} \cdot N_q + 0.5 \cdot \gamma_1 \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{ult} = 1.081 \cdot 10^4 \cdot psf$$

$q_{all} := \frac{q_{ult}}{3}$ $q_{all} = 3.602 \cdot 10^3 \cdot psf$ $q_{all} = 3.602 \cdot ksf$ $q_{all} = 1.801 \cdot tsf$

Settlement:

Elastic and Consolidation (total) settlement for embankment widening

Placement of up to 4 ft of fill soils on sand overlying silt and clayey silt

Elev. 1 ft

Compacted Fill Layer

$\gamma_{fill} := 120 \cdot pcf$ $\quad H_{fill} := 4 \cdot ft$ $\quad N_{fill} := 20$

Elev. -3 ft

Layer 1 - Sand   Water table at top of sand

$\gamma_{sand} := 120 \cdot pcf$ $\quad H_{sand} := 7 \cdot ft$ $\quad N_{sand} := 9$

Elev. -10 ft

Layer 2 - Silt (Presumpscot Upper Crust)

$\gamma_{silt} := 118 \cdot pcf$ $\quad H_{silt} := 8 \cdot ft$

Elev. -18 ft

Layer 3 - Clayey Silt S_u ranges from 300 to 900 pcf

57.0 ft clay $\phi = 0$

$H_{claya} := 7 \cdot ft$

Elev. -25 ft

Break layer up into 6 sublayers
Layers 3a, 3b, 3c, 3d, 3e and 3f

$H_{clayb} := 10 \cdot ft$

Elev. -35 ft

$\gamma_{clay} := 110 \cdot pcf$ $\quad H_{clay} := 42.0 \cdot ft$

$H_{clayc} := 10 \cdot ft$

Elev. -45 ft

Assume C_c and C_r values

e_o = (wc x SG)/100 use avg wc = 33.9%

$H_{clayd} := 10 \cdot ft$

Elev. -55 ft

C_c = 1.3 x wc - 0.2

$H_{claye} := 10 \cdot ft$

Cr = 0.1 x C_c

Elev. -65 ft

$e_0 := 0.9$ $\quad C_c := 0.24$ $\quad C_r := 0.024$

$H_{clayf} := 10 \cdot ft$

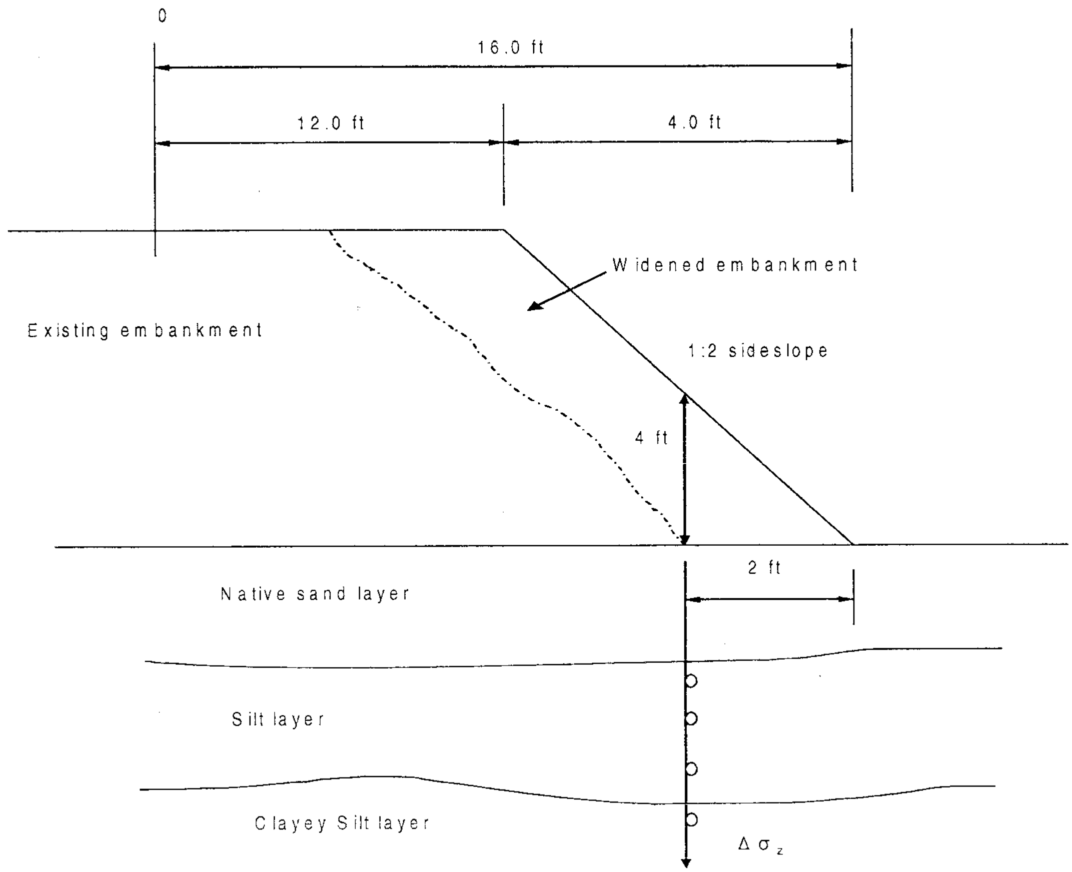
Elev. -75 ft

Layer 4 - Silty sand $\gamma_{ssand} := 120 \cdot pcf$ $\quad H_{ssand} := 10 \cdot ft$ $\quad N_{ssand} := 24$

Top of Bedrock

Elev. -85 ft

$\gamma_w := 62.4 \cdot pcf$ $\quad D_w := 13.7 \cdot ft$



Geometry for calculating $\Delta\sigma_z$
(not to scale)

Calculate the change in vertical streess with depth using STRESS
Reference: Productivity Tools For Geotechnical Engineers Volume 1
by J.T. Christian and A. Urzua

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

Project Name : Station 34 Client : York
Project Number : 11066.00 Project Manager : JWentworth
Date : 5/13/10 Computed by : km

Embank. slope a = 2.00(ft)
Embank. width b = 14.00(ft)
p load/unit area = 500.00(psf)

INCREMENT OF STRESSES FOR X-Z PLANE
Vert. σz
(psf)

X(ft)	10.00	11.00	12.00	13.00	14.00
Z(ft)	(psf)	(psf)	(psf)	(psf)	(psf)
0.00	500.00	500.00	500.00	500.00	250.00
1.00	498.31	496.43	489.79	454.52	249.95
2.00	488.75	479.05	453.95	386.97	249.62
3.00	470.44	451.95	415.04	347.39	248.76
4.00	446.95	423.04	383.00	322.94	247.20
5.00	421.68	395.96	357.40	305.63	244.84
6.00	396.65	371.48	336.26	291.87	241.67
7.00	372.80	349.42	318.06	279.97	237.73
8.00	350.51	329.43	301.88	269.11	233.13
9.00	329.88	311.20	287.16	258.89	227.98
10.00	310.89	294.50	273.60	249.15	222.42
11.00	293.45	279.16	260.99	239.79	216.56
12.00	277.48	265.03	249.24	230.80	210.53
13.00	262.83	252.00	238.25	222.17	204.42
14.00	249.41	239.97	227.97	213.90	198.31
15.00	237.10	228.85	218.36	206.01	192.27
16.00	225.79	218.57	209.36	198.49	186.34
17.00	215.38	209.04	200.93	191.34	180.56
18.00	205.79	200.20	193.04	184.54	174.96
19.00	196.94	192.00	185.65	178.10	169.55
20.00	188.74	184.36	178.72	171.99	164.34
21.00	181.15	177.25	172.22	166.20	159.34
22.00	174.10	170.62	166.12	160.72	154.54
23.00	167.54	164.42	160.39	155.53	149.96
24.00	161.43	158.62	154.99	150.61	145.57
25.00	155.71	153.19	149.91	145.95	141.38
26.00	150.37	148.09	145.12	141.53	137.37
27.00	145.36	143.30	140.60	137.33	133.55
28.00	140.66	138.78	136.33	133.35	129.90
29.00	136.24	134.53	132.30	129.57	126.41
30.00	132.08	130.52	128.47	125.98	123.08
31.00	128.16	126.73	124.85	122.57	119.90
32.00	124.45	123.14	121.42	119.31	116.86
33.00	120.94	119.74	118.15	116.22	113.95
34.00	117.62	116.51	115.05	113.26	111.17
35.00	114.47	113.45	112.10	110.44	108.51
36.00	111.48	110.53	109.28	107.75	105.96
37.00	108.64	107.76	106.60	105.18	103.51
38.00	105.94	105.12	104.04	102.72	101.17
39.00	103.36	102.60	101.60	100.37	98.92
40.00	100.90	100.19	99.26	98.11	96.76

$\Delta\sigma z_1 := 335 \cdot psf$

$\Delta\sigma z_2 := 235 \cdot psf$

$\Delta\sigma z_{3a} := 178 \cdot psf$

$\Delta\sigma z_{3b} := 135 \cdot psf$

$\Delta\sigma z_{3c} := 104 \cdot psf$

X(ft) Z(ft)	10.00 (psf)	11.00 (psf)	12.00 (psf)	13.00 (psf)	14.00 (psf)
41.00	98.56	97.90	97.03	95.95	94.69
42.00	96.31	95.70	94.88	93.88	92.70
43.00	94.17	93.59	92.83	91.89	90.79
44.00	92.12	91.58	90.86	89.98	88.94
45.00	90.15	89.64	88.97	88.15	87.17
46.00	88.26	87.79	87.16	86.38	85.46
47.00	86.45	86.01	85.41	84.68	83.82
48.00	84.72	84.29	83.74	83.05	82.23
49.00	83.04	82.65	82.12	81.47	80.70
50.00	81.44	81.06	80.57	79.95	79.23
51.00	79.89	79.53	79.07	78.49	77.80
52.00	78.40	78.06	77.62	77.07	76.42
53.00	76.96	76.64	76.22	75.71	75.09
54.00	75.57	75.27	74.88	74.38	73.80
55.00	74.24	73.95	73.58	73.11	72.55
56.00	72.95	72.68	72.32	71.87	71.35
57.00	71.70	71.44	71.10	70.68	70.18
58.00	70.49	70.25	69.92	69.52	69.05
59.00	69.32	69.09	68.78	68.40	67.95
60.00	68.19	67.97	67.68	67.32	66.88
61.00	67.10	66.89	66.61	66.26	65.85
62.00	66.04	65.84	65.57	65.24	64.85
63.00	65.01	64.82	64.57	64.25	63.88
64.00	64.02	63.84	63.59	63.29	62.93
65.00	63.05	62.88	62.65	62.36	62.01
66.00	62.11	61.95	61.73	61.45	61.12
67.00	61.20	61.04	60.83	60.57	60.25
68.00	60.32	60.17	59.96	59.71	59.41
69.00	59.46	59.31	59.12	58.88	58.59
70.00	58.63	58.48	58.30	58.07	57.79
71.00	57.81	57.68	57.50	57.28	57.01
72.00	57.02	56.89	56.72	56.51	56.25
73.00	56.25	56.13	55.96	55.76	55.51
74.00	55.50	55.39	55.23	55.03	54.79
75.00	54.78	54.66	54.51	54.32	54.09
76.00	54.06	53.95	53.81	53.62	53.41
77.00	53.37	53.27	53.12	52.95	52.74
78.00	52.70	52.59	52.46	52.29	52.09
79.00	52.04	51.94	51.81	51.65	51.45
80.00	51.40	51.30	51.18	51.02	50.83
81.00	50.77	50.68	50.56	50.41	50.22
82.00	50.16	50.07	49.95	49.81	49.63

$\Delta\sigma z_{3d} := 84 \cdot psf$

$\Delta\sigma z_{3e} := 70 \cdot psf$

$\Delta\sigma z_{3f} := 60 \cdot psf$

$\Delta\sigma z_{4} := 52 \cdot psf$

Calculate the stresses at mid-layer

N'/N - Ratio of Corrected blow count to SPT Value
Bazaraa 1967 - FHWA Soils and Foundation Workshop Manual page 6-8

Bearing Capacity Index = C1 for granular soils per FHWA Soils and Foundation Workshop Manual page 6-9

Layer 1: Sand

$$\sigma 1_0 := \gamma_{fill} \cdot H_{fill} + \frac{H_{sand}}{2} \cdot (\gamma_{sand} - \gamma_w) \qquad \sigma 1_0 = 681.6 \cdot psf$$

SPT N-value (bpf) $\quad N_{sand} := 9$

AT P_o = 1000 psf $\quad$ N'/N = r1 = 1.3 $\quad r1 := 1.3$

Corrected Blow Count $\quad N_{cor} := r1 \cdot N_{sand} \quad N_{cor} = 11.7$

From Figure 6-6 using the "clean well graded fine to coasrse sand" curve

Bearing Capacity Index: $\quad C1 := 50$

Layer 2: Silt

$$\sigma 2_0 := \sigma 1_0 + \frac{H_{sand}}{2} \cdot (\gamma_{sand} - \gamma_w) + \frac{H_{silt}}{2} \cdot (\gamma_{silt} - \gamma_w) \qquad \sigma 2_0 = 1.106 \cdot 10^3 \cdot psf$$

Layer 3: Silty Clay layer

$$\sigma 3a_0 := \sigma 2_0 + \frac{H_{silt}}{2} \cdot (\gamma_{silt} - \gamma_w) + \frac{H_{claya}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3a_0 = 1494.6 \cdot psf$$

$$\sigma 3b_0 := \sigma 3a_0 + \frac{H_{claya}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{clayb}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3b_0 = 1899.2 \cdot psf$$

$$\sigma 3c_0 := \sigma 3b_0 + \frac{H_{clayb}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{clayc}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3c_0 = 2375.2 \cdot psf$$

$$\sigma 3d_0 := \sigma 3c_0 + \frac{H_{clayc}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{clayd}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3d_0 = 2851.2 \cdot psf$$

$$\sigma 3e_0 := \sigma 3d_0 + \frac{H_{clayd}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{claye}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3e_0 = 3327.2 \cdot psf$$

$$\sigma 3f_0 := \sigma 3e_0 + \frac{H_{claye}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{clayf}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3f_0 = 3803.2 \cdot psf$$

Layer 4: Loose Sand layer

$$\sigma 4_0 := \sigma 3f_0 + \frac{H_{clayf}}{2}\cdot(\gamma_{clay} - \gamma_w) + \frac{H_{ssand}}{2}\cdot(\gamma_{ssand} - \gamma_w) \qquad \sigma 4_0 = 4329.2\,\text{psf}$$

SPT N-value (bpf) $\quad N_{ssand} := 24$

AT P_o = 4300 psf $\quad$ N'/N = r1 = .72 $\quad r1 := .72$

Corrected Blow Count $\quad N_{cor} := r1\cdot N_{ssand} \qquad N_{cor} = 17.28$

From Figure 6-6 using the "clean well graded fine to medium sand" curve

Bearing Capacity Index: $\quad C2 := 62$

Settlement per layer

$$\Delta H_1 := H_{sand}\cdot\frac{1}{C1}\cdot\log\left(\frac{\sigma 1_0 + \Delta\sigma z_1}{\sigma 1_0}\right) \qquad \Delta H_1 = 0.292\,\text{in}$$ Elastic settlement

$$\Delta H_2 := H_{silt}\cdot\frac{C_r}{1 + e_0}\cdot\log\left(\frac{\sigma 2_0 + \Delta\sigma z_2}{\sigma 2_0}\right) \qquad \Delta H_2 = 0.101\,\text{in}$$ Consolidation settlement

$$\Delta H_{3a} := H_{claya}\cdot\frac{C_r}{1 + e_0}\cdot\log\left(\frac{\sigma 3a_0 + \Delta\sigma z_{3a}}{\sigma 3a_0}\right) \qquad \Delta H_{3a} = 0.052\,\text{in}$$ Consolidation settlement

$$\Delta H_{3b} := H_{clayb}\cdot\frac{C_c}{1 + e_0}\cdot\log\left(\frac{\sigma 3b_0 + \Delta\sigma z_{3b}}{\sigma 3b_0}\right) \qquad \Delta H_{3b} = 0.452\,\text{in}$$ Consolidation settlement

$$\Delta H_{3c} := H_{clayc}\cdot\frac{C_c}{1 + e_0}\cdot\log\left(\frac{\sigma 3c_0 + \Delta\sigma z_{3c}}{\sigma 3c_0}\right) \qquad \Delta H_{3c} = 0.282\,\text{in}$$ Consolidation settlement

$$\Delta H_{3d} := H_{clayd}\cdot\frac{C_c}{1 + e_0}\cdot\log\left(\frac{\sigma 3d_0 + \Delta\sigma z_{3d}}{\sigma 3d_0}\right) \qquad \Delta H_{3d} = 0.191\,\text{in}$$ Consolidation settlement

$$\Delta H_{3e} := H_{claye}\cdot\frac{C_c}{1 + e_0}\cdot\log\left(\frac{\sigma 3e_0 + \Delta\sigma z_{3e}}{\sigma 3e_0}\right) \qquad \Delta H_{3e} = 0.137\,\text{in}$$ Consolidation settlement

$$\Delta H_{3f} := H_{clayf}\cdot\frac{C_c}{1 + e_0}\cdot\log\left(\frac{\sigma 3f_0 + \Delta\sigma z_{3f}}{\sigma 3f_0}\right) \qquad \Delta H_{3f} = 0.103\,\text{in}$$ Consolidation settlement

$$\Delta H_4 := H_{ssand} \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma 4_0 + \Delta\sigma z_4}{\sigma 4_0}\right) \qquad \Delta H_4 = 0.01\ \text{in}$$

Elastic settlement

$$\text{Total\_Settlement} := \Delta H_1 + \Delta H_2 + \Delta H_{3a} + \Delta H_{3b} + \Delta H_{3c} + \Delta H_{3d} + \Delta H_{3e} + \Delta H_{3f} + \Delta H_4$$

$$\text{Total\_Settlement} = 1.62\ \text{in}$$

$$\text{Elastic\_Settlement} := \Delta H_1 + \Delta H_4 \qquad \text{Elastic\_Settlement} = 0.302\ \text{in}$$

$$\text{Consolidation\_Settlement} := \Delta H_2 + \Delta H_{3a} + \Delta H_{3b} + \Delta H_{3c} + \Delta H_{3d} + \Delta H_{3e} + \Delta H_{3f}$$

$$\text{Consolidation\_Settlement} = 1.319\ \text{in}$$

Check Consolidation Settlement with SAF-I
Reference: Productivity Tools For Geotechnical Engineers Volume 1
by J.T. Christian and A. Urzua

```
ONE DIMENSIONAL SETTLEMENT ANALYSIS/PROTOTYPE ENGINEERING INC.
³           STRIP FOOTING VERTICAL EMBANKMENT LOADING
³
³ Project Name   : Station 34       Client          : York
³ Project Number : 11066.00         Project Manager : JWentworth
³ Date           : 5/13/10          Computed by     : km
³       Increment of stresses obtained using : Boussinesq
³               Settlement for X-Direction
³
³ Embank. slope a       =  2.00 (ft)    p load/unit area   = 500.00 (psf)
³ Embank. width         = 14.00 (ft)    Foundation Elev.   =  -3.00 (ft)
³ Ground Surface Elev.  = -3.00 (ft)
³ Water table Elev.     = -3.00 (ft)    Unit weight of Wat. = 62.40 (pcf)
³
³       LAYER              COEFFICIENT         UNIT   SPECIFIC  VOID
³   NO. TYPE  THICK. COMP. RECOMP. SWELL.  WEIGHT  GRAVITY  RATIO
³               (ft)                        (pcf)
³    1 INCOMP.  7.0   -----  -----   -----  120.00    ----     ----
³    2 COMP.   65.0   0.240  0.024   0.024  120.00    2.65     0.90
³    3 INCOMP. 10.0   -----  -----   -----  120.00    ----     ----
³
³       SUBLAYER                 SOIL STRESSES
³   NO.   THICK.      ELEV.      INITIAL     MAX.PAST PRESS.
³          (ft)        (ft)       (psf)          (psf)
³    1   INCOMP.
³    2   65.00       -42.50     2275.20        2275.20
³    3   INCOMP.
³
³          X =  0.00     X =  1.00     X =  2.00     X =  3.00     X =  4.00
³ Layer   Stress Sett.  Stress Sett.  Stress Sett.  Stress Sett.  Stress Sett.
³         (psf)  (in.)  (psf)  (in.)  (psf)  (in.)  (psf)  (in.)  (psf)  (in.)
³  1        INCOMP.       INCOMP.       INCOMP.       INCOMP.       INCOMP
³  2      96.27  1.77   97.86  1.80   99.26  1.83  100.44  1.85  101.40  1.87
³  3        INCOMP.       INCOMP.       INCOMP.       INCOMP.       INCOMP.
³               ------        ------        ------        ------        ------
³                1.77          1.80          1.83          1.85          1.87
³
³          X =  5.00     X =  6.00     X =  7.00     X =  8.00     X =  9.00
³ Layer   Stress Sett.  Stress Sett.  Stress Sett.  Stress Sett.  Stress Sett.
³         (psf)  (in.)  (psf)  (in.)  (psf)  (in.)  (psf)  (in.)  (psf)  (in.)
³  1        INCOMP.       INCOMP.       INCOMP.       INCOMP.       INCOMP.
³  2     102.13  1.88  102.62  1.89  102.86  1.89  102.86  1.89  102.61  1.89
³  3        INCOMP.       INCOMP.       INCOMP.       INCOMP.       INCOMP.
³               ------        ------        ------        ------        ------
³                1.88          1.89          1.89          1.89          1.89
³                                    ³
³          X = 10.00     X = 11.00
³ Layer   Stress Sett.  Stress Sett.
³         (psf)  (in.)  (psf)  (in.)
³  1        INCOMP.       INCOMP.
³  2     102.12  1.88  101.38  1.87
³  3        INCOMP.       INCOMP.
³               ------        ------
³                1.88          1.87
```

Settlement =
Approx. 2"

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 6/16/04

To: Mike Babb **Dept:** Reproduction Room

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: York, Station 34 Bridge over Tidal Estuary, PIN. 11066.00

Attached is one (1) copy of Soils Report 2004-21, entitled "GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: STATION 34 BRIDGE OVER TIDAL ESTUARY, YORK, MAINE" dated: June 2004.

Please forward your copy to Kate Maguire after report has been scanned.

taw
att: 1 of 2004-21

COPY

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 6/15/04

To: Matthew Steele **Dept:** Environment Office

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: York, Station 34 Bridge over Tidal Estuary, PIN. 11066.00

Attached is one (1) copy of Soils Report 2004-21, entitled "GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: STATION 34 BRIDGE OVER TIDAL ESTUARY, YORK, MAINE" dated: June 2004.

taw
att: 1 of 2004-21

COPY

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 6/23/04

To: Kevin Cummings **Dept:** Urban and Federal Bridge Program

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: York, Station 34 Bridge over Tidal Estuary, PIN. 11066.00

Attached is one (1) copy of Soils Report 2004-21, entitled "GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: STATION 34 BRIDGE OVER TIDAL ESTUARY, YORK, MAINE" dated: June 2004.

taw
att: 1 of 2004-21

COPY

MEMORANDUM

TO: Ms. Laura Krusinski, P.E.
Maine Department of Transportation (MDOT)
Bridge Program
State House Station 16, Transportation Building
Augusta, Maine 04333-0016

FROM: Eric J. Baron, E.I.T.
Christopher L. Snow, P.E., Senior Project Manager
James V. Errico, P.E., Principal
GZA GeoEnvironmental, Inc. (GZA)

STATE OF MAINE
CHRISTOPHER L. SNOW
No. 7275
LICENSED
PROFESSIONAL ENGINEER

DATE: December 3, 2008

FILE NO.: 09.0025577.00

SUBJECT: Subsurface Investigation and Settlement Evaluation
Station 34 Bridge
York, Maine
MDOT PIN 15110.00

This Memorandum presents the results of the subsurface explorations, laboratory testing, and settlement estimates completed by GZA GeoEnvironmental for proposed replacement of the Station 34 Bridge over the tidal estuary in York, Maine. This work was preformed in accordance with GZA's September 2, 2008, Work Plan Contract Modification No. 2, associated with Specific Period Contract GCA Number U1210060627, and the attached limitations. This report contains the work product for the Station 34 Bridge. New Bridge is discussed in a separate report.

PROJECT BACKGROUND

Station 34 Bridge is located on Route 103 in York, Maine. The existing bridge consists of a three span, timber-pile supported structure crossing a tidal estuary. The existing structure has a total span of less than 70 feet. MDOT is considering three design alternatives for the replacement of the existing structure including: a single span pile supported bridge, a pile supported embankment and an earth fill embankment. This report includes a settlement evaluation for the earth fill embankment option.

The soil profile used in GZA's evaluation is based on subsurface conditions encountered in test borings BB-YR34-101 and BB-YR34-102 drilled by MDOT in March 2004. The profile was included in the Geotechnical Design Report entitled, "Station 34 Bridge Over Tidal Estuary, York, Maine," PIN 11066.00, dated June 2004 and provided to GZA by the MDOT.

SUBSURFACE INVESTIGATION

PREVIOUS EXPLORATIONS

MDOT drilled two test borings at the Station 34 Bridge for the bridge replacement. Borings BB-YR34-101 and BB-YR34-102 were drilled on the north and south sides, respectively, of Station 34 Bridge, at distances of approximately 14 feet from each bridge abutment.

The borings encountered compressible deposits to depths of approximately 74.8 and 70.8 feet below ground surface; sand deposits to depths of 84.6 and 95.3 feet below ground surface and were terminated in rock at 89.6 and 99.1 feet below ground surface. Standard penetration testing and split-spoon sampling were performed at 5 foot typical intervals in the borings. Insitu vane shear tests were performed in the compressible deposits below the standard penetration tests at 5 foot typical intervals. Boring logs are provided in Appendix B. A site location plan, boring location plan and interpretive soil profile are also included in Appendix B for reference.

RECENT EXPLORATIONS

GZA performed two additional test borings at the Station 34 Bridge for use in evaluating settlement of the earth fill embankment option. GZA retained the services of New Hampshire Borings, Inc of Londonderry, New Hampshire for two days of drilling services on September 15, 2008 and October 17, 2008. A GZA engineer monitored the drilling activities and prepared logs of the borings that are included in Appendix B.

Borings BB-YR34-201 and BB-YR34-201A were drilled at distances of about 13.6 feet and 16.6 feet south of the bridge abutment, respectively and 3.6 feet east of the west edge of the pavement.

Test borings were drilled into the compressible deposits to depths of approximately 78 and 61 feet below ground surface for the purpose of collecting samples of the compressible soils. Standard penetration testing and split-spoon sampling were performed at 10 foot typical intervals in the borings. A series of 3-inch diameter thin wall tube samples were obtained from the borings at various depths for laboratory testing to evaluate the index properties and the consolidation parameters of the compressible soils. Laboratory test results are provided in Appendix C.

LABORATORY TESTING

A laboratory testing program was performed for use in soil classification and evaluation of site-specific geotechnical engineering properties of the compressible deposits. The program consisted of three (3) one-dimensional consolidation tests, six (6) sets of Atterberg Limits, twenty-five (25) natural water content determinations, six (6) unit weight determinations, sixteen (16) torvane tests and eight (8) vane shear tests performed on thin-wall tube samples from borings BB-YR34-201 and BB-YR34-201A.

Results of the consolidation testing are presented graphically in Figure 1 – Atterberg Limits, Shear Strength, and Stress History; Figure 2 – Compressibility Parameters; and Figure 3 – Consolidation Coefficient. Complete Testing data are included in Appendix C, Laboratory Test Results. Laboratory testing was performed at GZA's Hopkinton, Massachusetts soil testing laboratory for boring BB-YR34-201; and at R.W. Gillespie & Associates in Saco, Maine for BB-YR34-201A.

SUBSURFACE CONDITIONS

Five soil units were encountered above bedrock in the explorations: Fill, silty SAND, SILT, clayey SILT and silty SAND. The encountered thicknesses and generalized descriptions of the units are summarized in the following table. Refer to logs of the test borings included in Appendix B for conditions at specific locations.

Unit	Encountered Thickness (Ft)	Description
Fill	13 to 15	Loose to medium dense, brown, fine to coarse SAND, little Silt.
Silty SAND	5 to 7	Medium dense, gray-brown, fine to coarse silty SAND, trace Gravel.
SILT & CLAY to Clayey Silt	54 to 59	Variable from: stiff, gray SILT & CLAY trace Gravel with black staining to: soft, Gray, clayey SILT.
Silty SAND	24.5 to 9.8	Medium dense, gray-brown, fine to coarse silty SAND, trace Gravel.
Depth to Rock	94.8 to 105.5	Top rock estimated based on refusal and 5 foot confirmatory rock core.

STRESS HISTORY

The stress-strain characteristics of a clay deposit are highly dependent on its stress history. If the soil is stressed within the limits of the maximum past pressure, the settlement will be a function of the recompression ratio (RR) determined from consolidation testing results. If the soil is stressed beyond the limits of the maximum past pressure, the settlement will be a function of the virgin compression ratio (CR). The magnitude of the virgin compression ratio is frequently on the order of 10 to 15 times the recompression ratio for sensitive clays.

Laboratory test results indicate that the maximum past pressures ranged from 11.8 ksf in the stiff crust to 2.6 ksf in the soft material at greater depths. The test results and correlations of undrained strength and effective overburden stress indicate that at the abutment, the deposit is over consolidated to depths of about 39 feet below ground surface, and normally consolidated below.

Since the settlement analysis will be performed at mid-span where no embankment loads have been imposed, the stress history is likely to differ from that encountered at the abutment. For the purposes of estimating consolidation settlement, the normally consolidated portion of the stress history curve is assumed to be equal to the calculated vertical effective stress – without embankment stress.

The maximum previous stress which each specimen has experienced was estimated from vertical strain versus consolidation stresses and is presented in Figure 1 relative to the existing effective overburden stress, estimated historic effective overburden stress prior to fill placement and depth below ground surface.

Because consolidation testing was performed on samples from locations beneath existing earth embankments, the stress history there reflects the consolidation from the embankment fill. To make settlement estimations for the new embankment it was necessary to modify the stress history profile. At shallow depths where the existing deposits were over consolidated, the maximum past pressure was not exceeded by the effective stress increase from the earth fill embankment. However, where the soil was normally consolidated, the past pressure is likely to have been lower than the test results indicate. At these locations, the past pressure was approximated as the existing effective overburden pressure prior to the placement of the fill. A graphical representation of the estimated past pressure curve is located in Appendix D entitled, Stress History.

SETTLEMENT EVALUATION- EARTH FILL OPTION

In order to evaluate the proposed earth-fill option bearing on the clay stratum, it is necessary to consider the settlement which would result from the loading of the new earth fill embankment. The fill material was assumed to consist of Granular Borrow for Underwater Backfill as specified by MaineDOT Specification 703.19, with a unit weight of 125 pcf. Furthermore, it is assumed that the existing Bridge abutments will be buried in place and the fill will taper from approximately 7 feet at the abutments to 14 feet at the center of the existing span.

The maximum estimated settlement due to the earth fill embankment replacement for the Station 34 Bridge is in the order of 12 inches of primary settlement and 5 inches of secondary settlement occurring approximately at station 32+50. Lower estimated settlements are expected in the fill areas to the north and to the south of Station 32+50 due to the tapering of the fill toward the existing abutments. Furthermore the lower settlement is expected in the fill to the east and west of the centerline due to the reduction in the stress effects across the fill area and the tapering of the fill at the limits of the roadway.

Assuming double-drainage of the full layer thickness, it is estimated that the primary settlement will occur over a period of about nineteen years. Based on the review of the boring logs from both MDOT and GZA, it is judged that the few intermediate sand lenses described in the borings are not continuous horizontally, and that the sand will not provide a significant drainage path for relief of excess pore pressure. The estimated time to achieve primary consolidation is based on the information available. In the event that the sand deposits are continuous and provide significant drainage for the excess pore pressure, the settlement could occur at a more rapid rate.

This information is provided for use in evaluating the impacts of settlement under the proposed earth fill embankment. The evaluations are only valid for the conditions outlined above. In the event that changes are proposed in the fill materials, embankment configuration or other pertinent aspects of the project, these settlement evaluations will need to be reviewed for continued validity.

If you have any questions or require additional information, please feel free to contact Mr. Snow at (207) 879-9190, or by e-mail at *csnow@gza.com*.

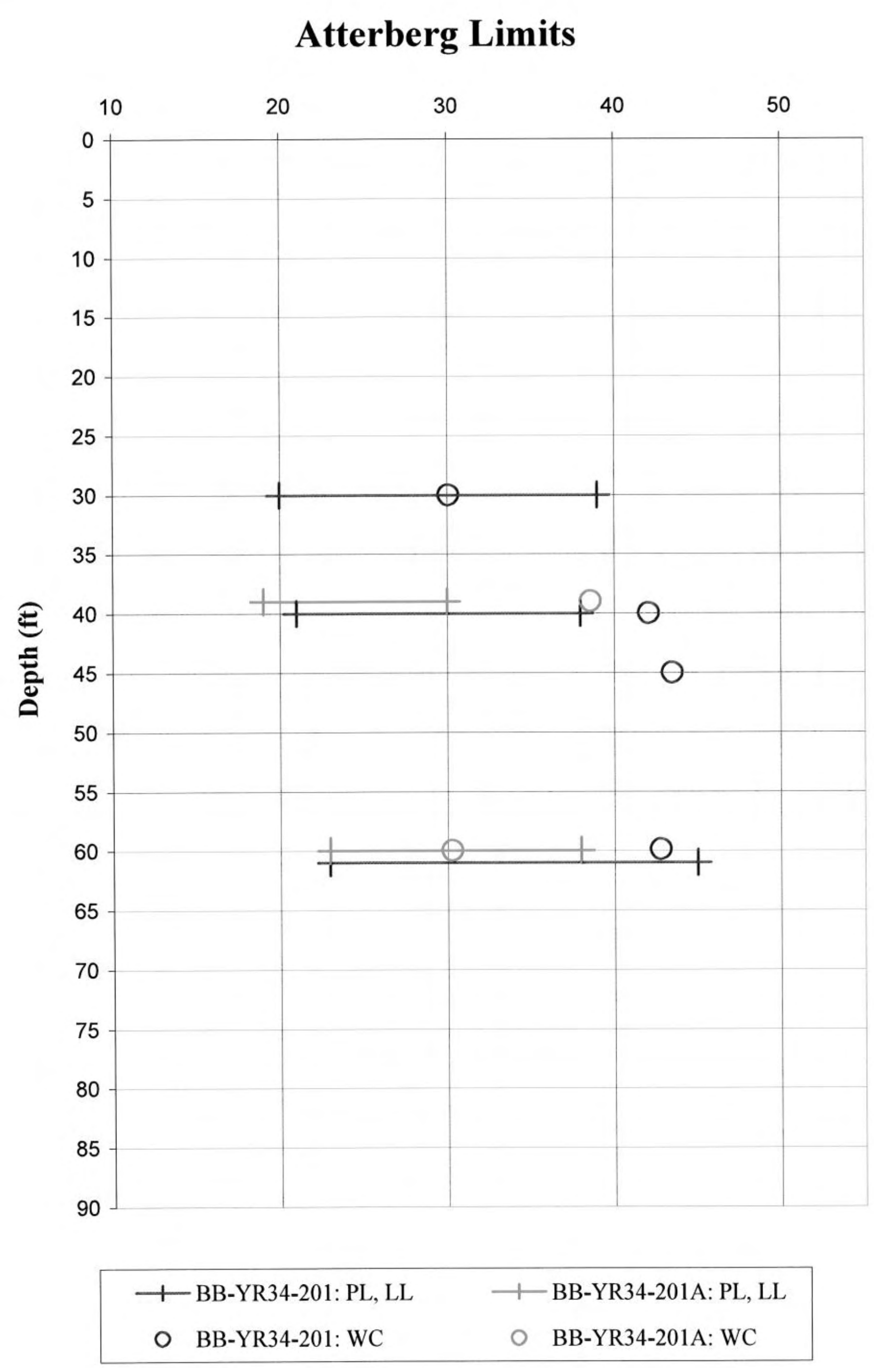
Attachments:
- Figure 1 – Atterberg Limits, Shear Strength and Stress History
- Figure 2 – Compressibility Parameters
- Figure 3 – Consolidation Coefficient
- Appendix A – Limitations
- Appendix B – Boring Logs
- Appendix C – Lab Test Results
- Appendix D –Calculations

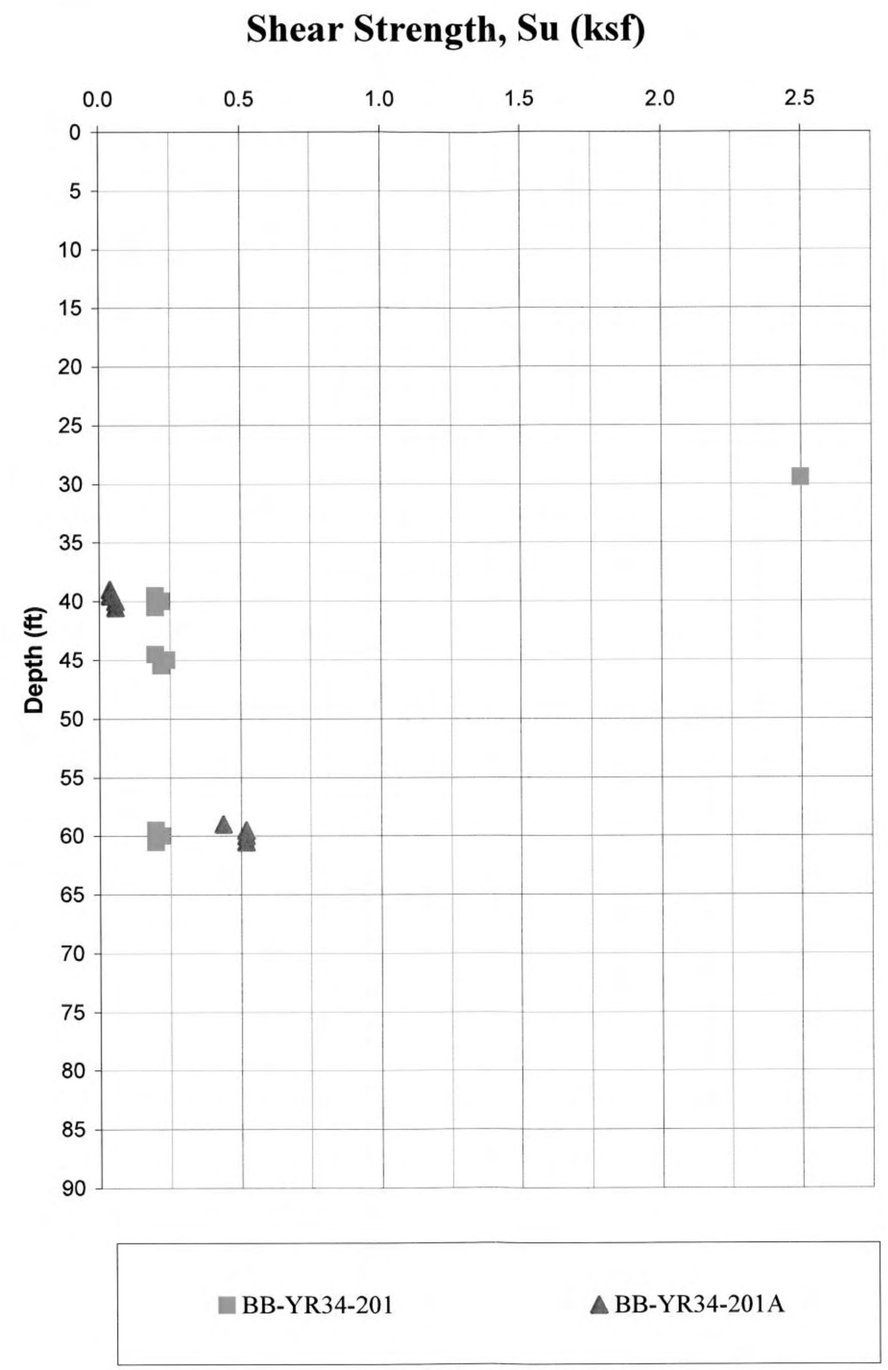
FIGURES

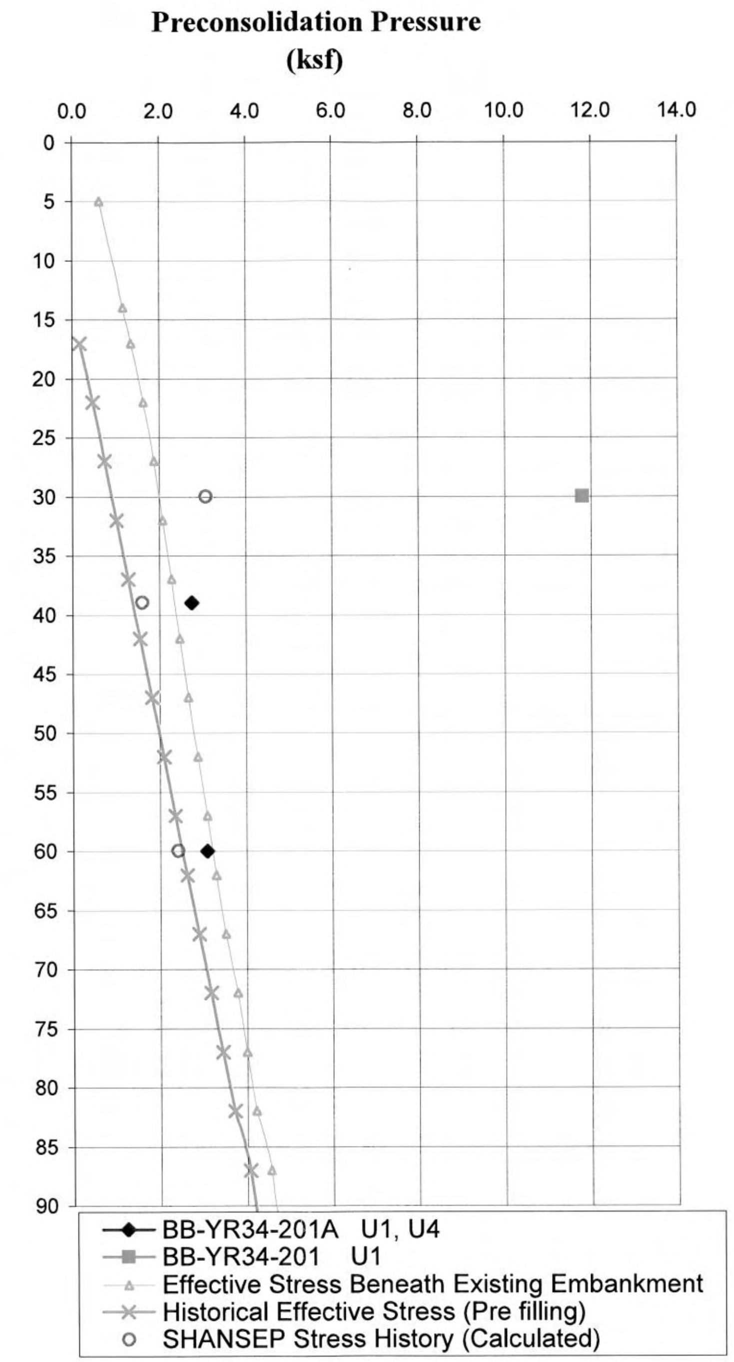
GZA
GeoEnvironmental, Inc.
Engineers and Scientists

Figure 1: Atteberg Limits, Shear Strength, and Stress History
Borings BB-YR34-201 and BB-YR34-201A
Station 34 Bridge Over Tidal Estuary(PIN11066.00
York, Maine

GZA Job No. 25577.00



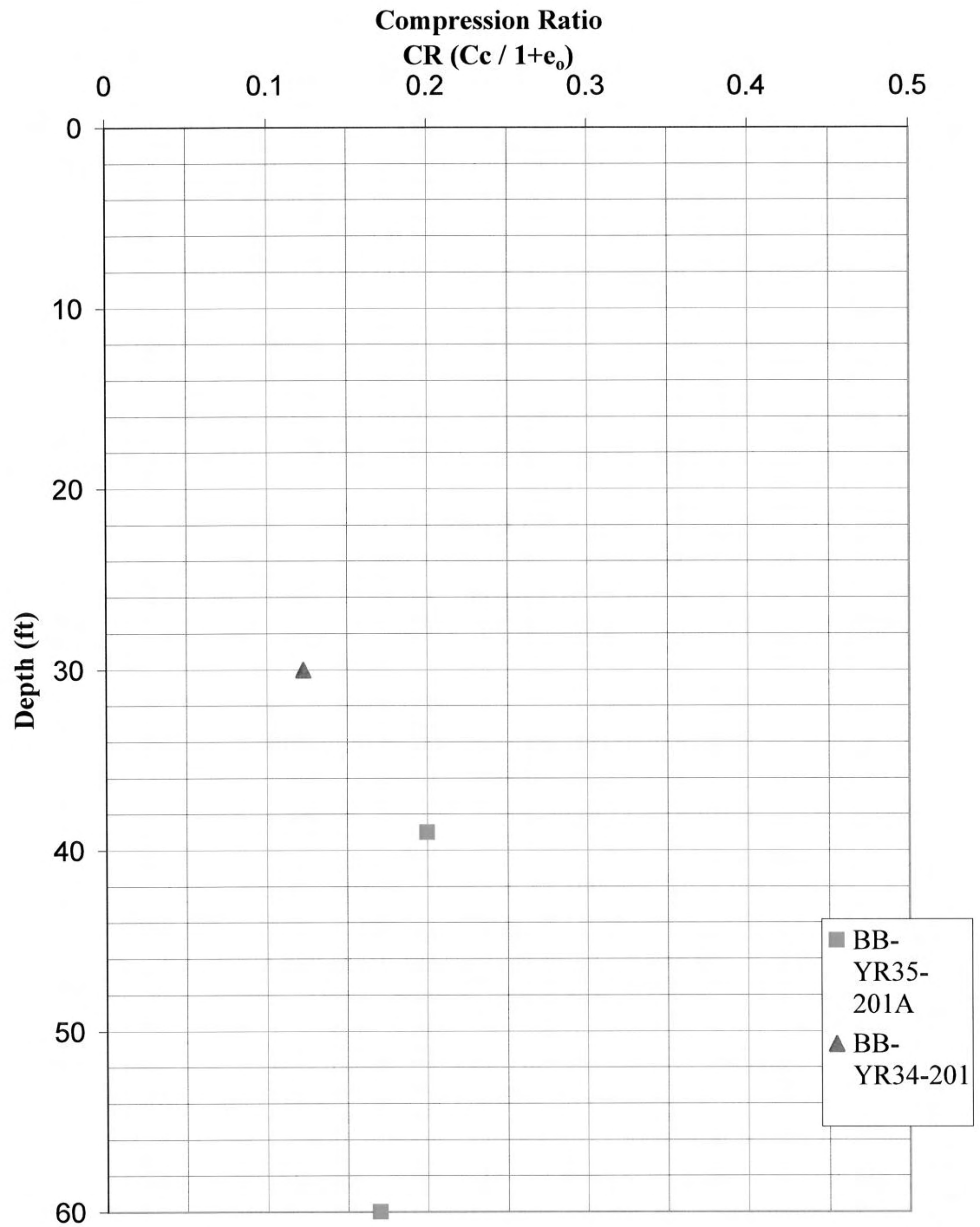




GZA
GeoEnvironmental, Inc.
Engineers and Scientists

Figure 2: Compressibility Parameters
Borings BB-YR34-201 and BB-YR34-201A
Station 34 Bridge Over Tidal Estuary(PIN11066.00
York, Maine

GZA Job No. 25577.00



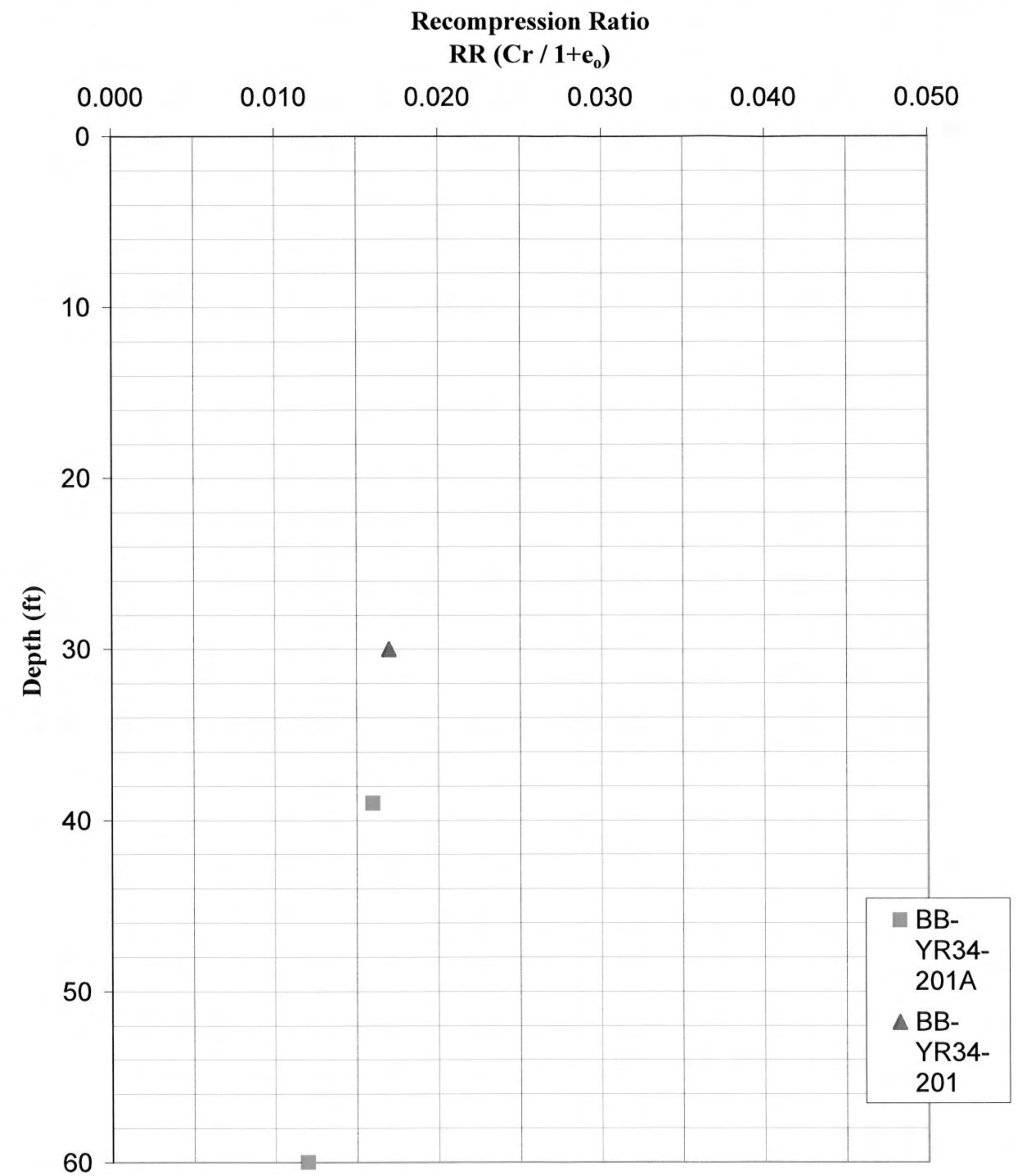
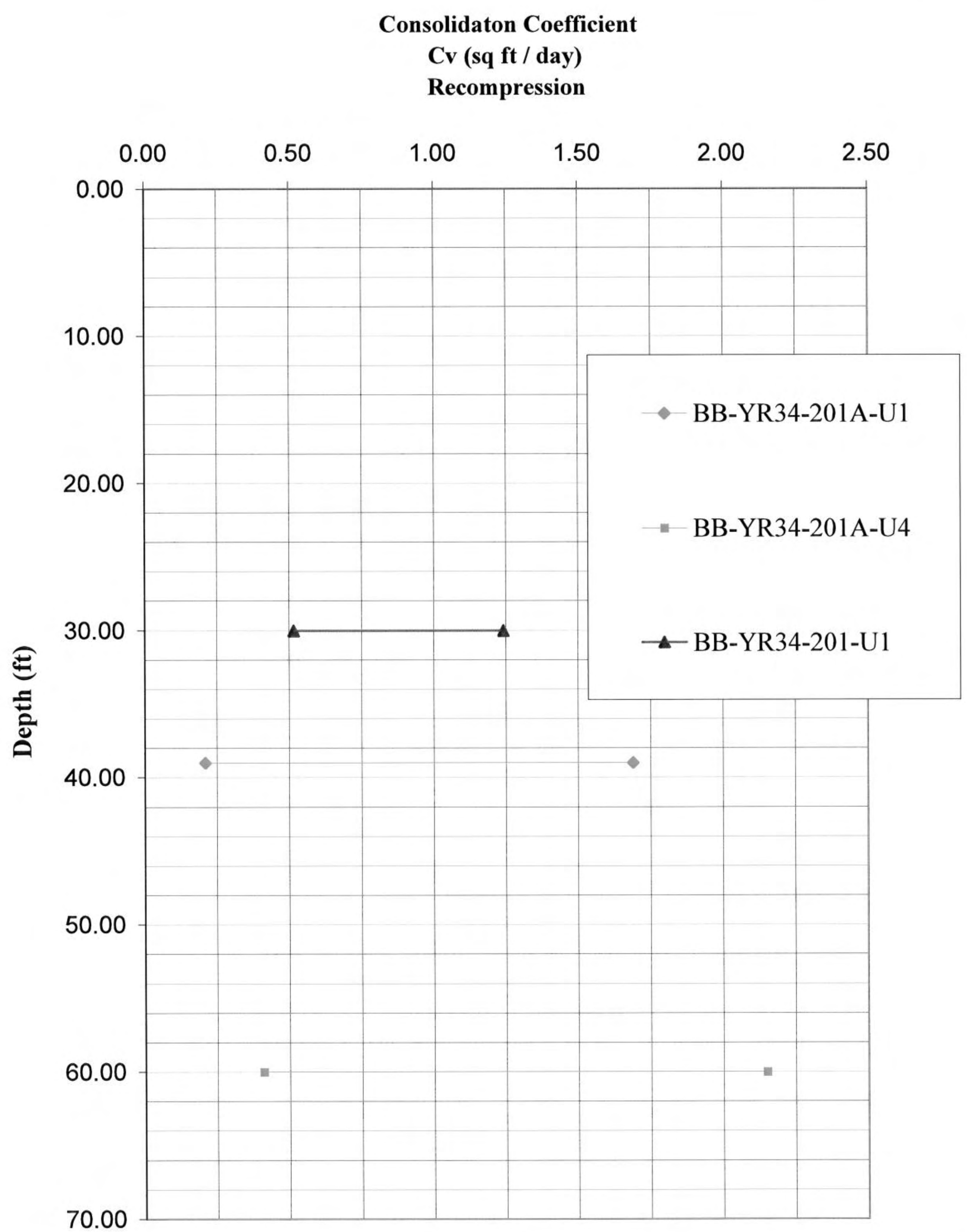
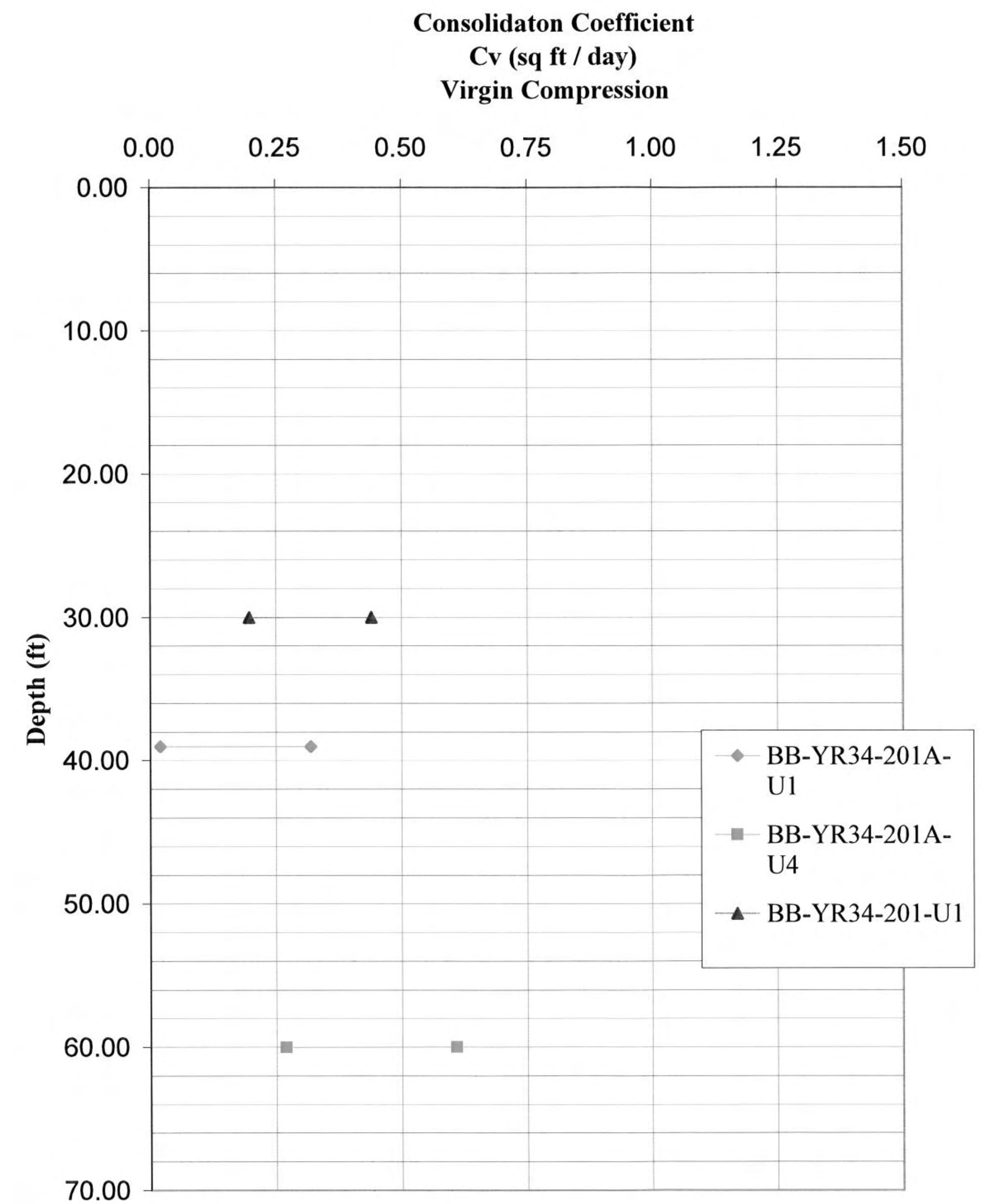


Figure 3: Consolidation Coefficient
Borings BB-YR34-201 and BB-YR34-201A
Station 34 Bridge Over Tidal Estuary(PIN11066.00
York, Maine





This plot shows selected data from one-dimensional consolidation testing over the specific load ranges stated. Complete data are provided in the laboratory results section of the report.

APPENDIX A

LIMITATIONS

LIMITATIONS

Explorations

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

2. The generalized soil profile described in the text 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 are probably more erratic. For specific information, refer to the boring logs.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, and other factors occurring since the time measurements were made.

Review

4. In the event that any changes in the nature, design or location of the proposed building are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by GZA GeoEnvironmental, Inc. It is recommended that this firm be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

Construction

5. It is recommended that this firm be retained to provide soil engineering services during construction of the excavation and foundation phases of the work. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

Use of Report

6. This soil and foundation engineering report has been prepared for this project by GZA GeoEnvironmental, Inc. This report is for design purposes only and is not sufficient to prepare an accurate bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to design considerations only.

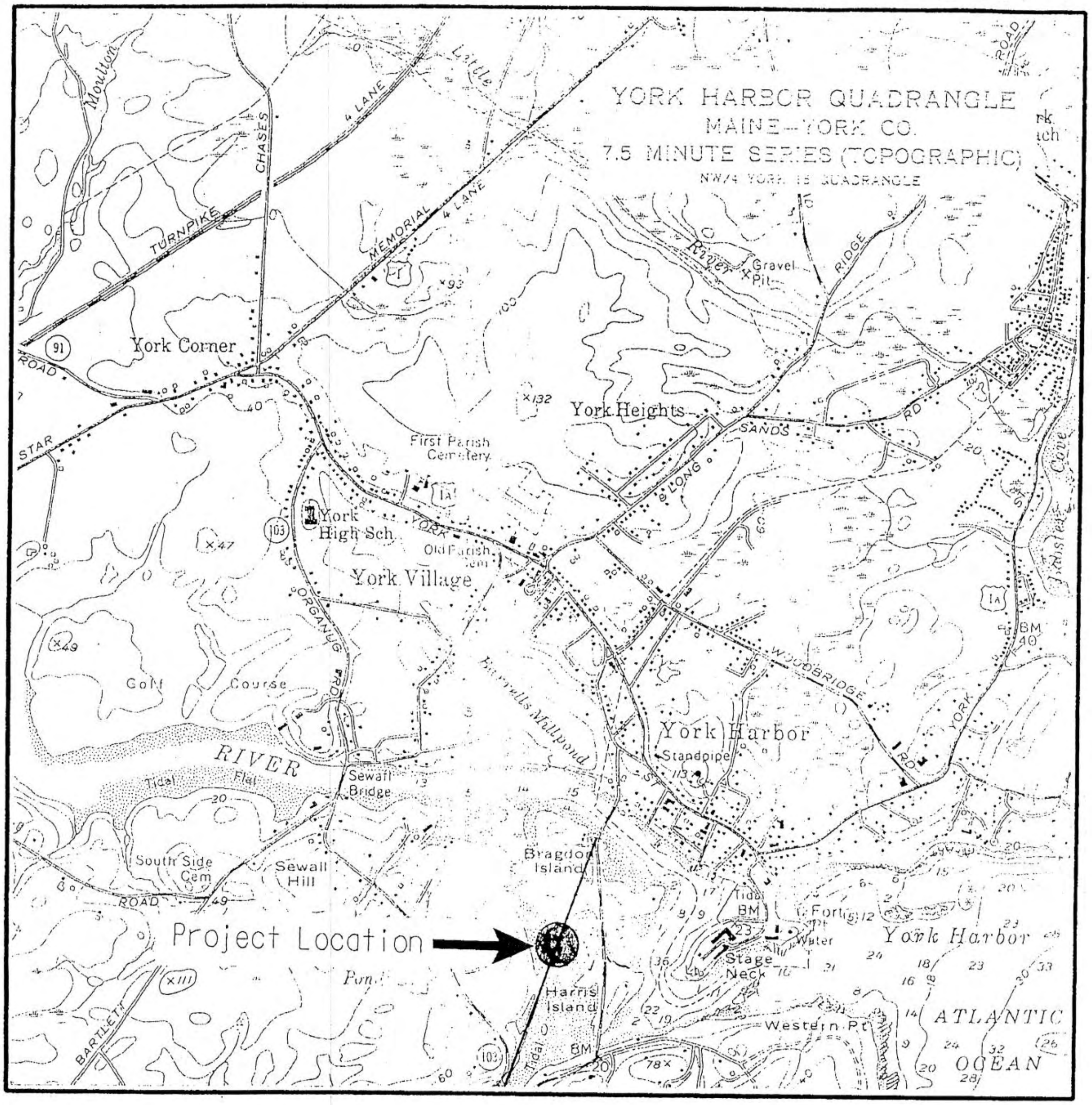
7. This report has been prepared for this project by GZA GeoEnvironmental, Inc. for the exclusive use of the Maine Department of Transportation for specific application to the Station 34 Bridge Earth Fill Option in York, Maine in accordance with generally accepted soil and foundation engineering practices. No Warranty, express or implied, is made.

APPENDIX B

BORING LOGS

Location Map

Sheet



15111.00

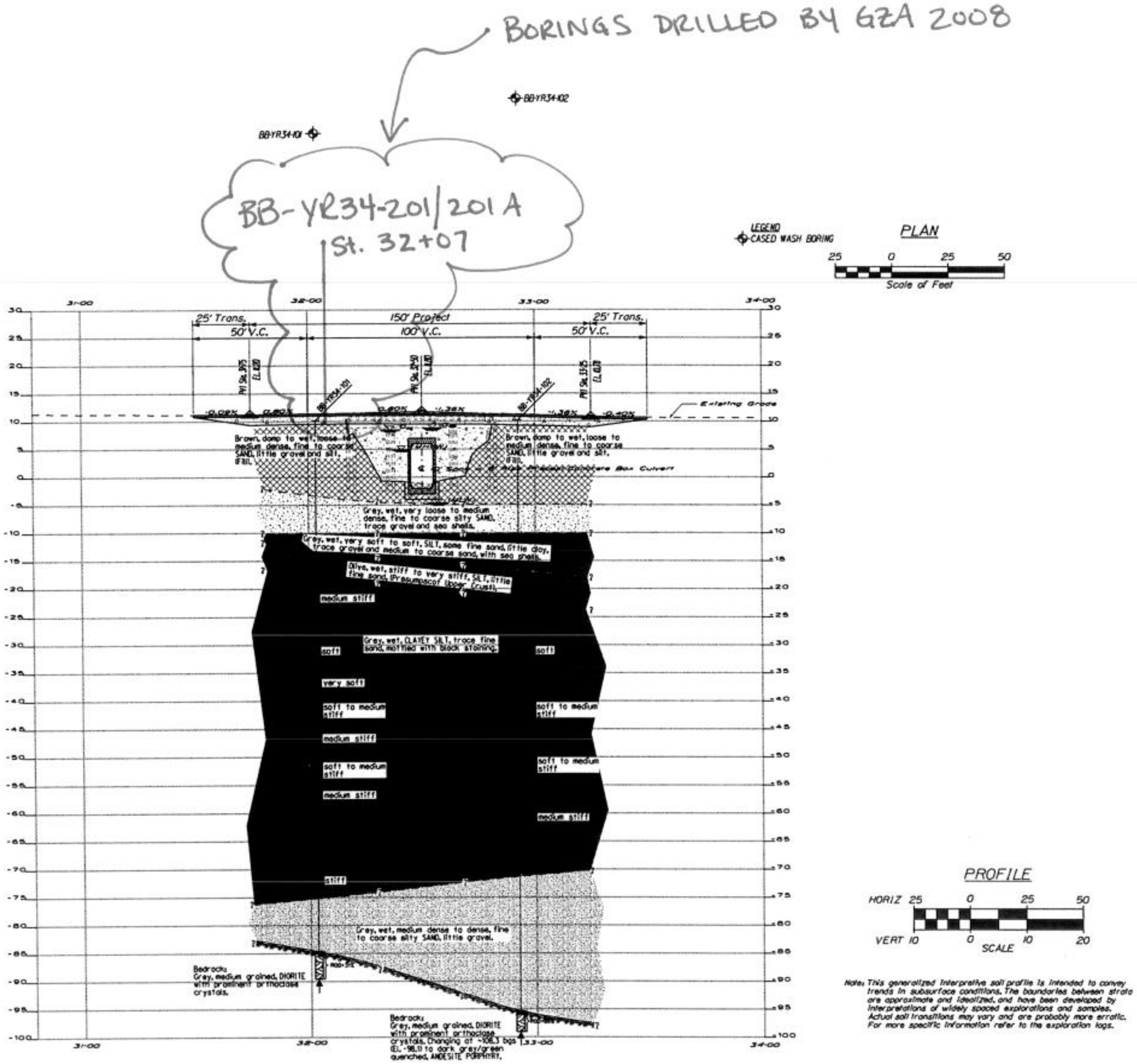
York, Maine, Station 34 Bridge over Tidal Estuary, PIN. ~~11066.00~~

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

1:24000, 1" = 2000', 1 cm = 240 m



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1511(100)X

BRIDGE NO. 5648 | PIN 15111.00 | BRIDGE PLANS

PROJ. MANAGER	J. Wentworth	BY	DATE
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	JAN 2009
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

STATION 34 BRIDGE
TIDAL ESTUARY
YORK COUNTY
YORK

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
4
OF 9

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary	**Location:**	York, Maine	**Boring No.:**	BB-YR34-101
				PIN:	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.78		Pavement 0.420	
5	1D	24/24	5.00 - 7.00	7/15/12/16	27				Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/6	10.00 - 12.00	4/3/4/4	7	18			Brown, wet, loose, fine to coarse SAND, little gravel and silt, (Fill).	
						19				
						27				
						18	-2.80		13.000	
						26				
15						2				
	3D	24/2	16.00 - 18.00	5/3/1/4	4	11			Grey, wet, very loose, fine to coarse silty SAND, trace gravel and sea shells.	
						10				
						12				
						12				
20	4D	24/20	20.00 - 22.00	2/WOH/WOH/WOH	---	16	-9.80		20.000 Grey, wet, very soft, SILT, some fine sand, little clay, trace gravel, medium to coarse sand and sea shells.	G#176641 A-4, CL-ML WC=36.8%
						11				
						19				
						30	-12.80		23.000	
						40				
25										

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Station 34 Bridge over Tidal Estuary **Location:** York, Maine	**Boring No.:** BB-YR34-101 **PIN:** 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
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V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D/MV	24/20	25.00 - 27.00	4/7/10/13	17	69			Olive, wet, very stiff, SILT, little fine sand, (Presumpscot upper crust). Attempt 55x110 mm vane: could not push	
						69				
						69				
						63	-17.80		28.000	
						69				
30	6D	24/23	30.00 - 32.00	3/2/3/3	5	78 aWA			Grey, wet, medium stiff, SILT, some clay, trace fine sand, mottled. aWashed ahead of casing.	G#176642 A-6, CL WC=31.7% LL=34 PL=20 PI=14
						70				
						64				
						64				
						50				
35	7D	24/22	35.00 - 37.00	2/2/1/WOH	3	52			Grey, wet, very loose, fine silty SAND, uniform, dilatent.	
						52				
						50				
						38				
						32				
40	8D ~~V1~~	24/24	40.00 - 42.00 ~~40.64 - 41.00~~	Push thru vane ~~Su=259/125 psf~~		62			Grey, wet, soft, clayey SILT, trace fine sand layers, black staining. 55x110 mm vane raw torque readings: V1 = 5.8/2.8 ft-lbs V2 = 10.7/1.5 ft-lbs	G#176643 A-4, CL WC=36.8% LL=28 PL=19 PI=9
	V2		41.64 - 42.00	Su=478/67 psf		45				
						44				
						48				
						50				
45	9D ~~V3~~	24/24	45.00 - 47.00 ~~45.64 - 46.00~~	Push thru vane ~~Su=192/67 psf~~		87			Similar to above, very soft. 55x110 mm vane raw torque readings: V3 = 4.3/1.5 ft-lbs V4 = 5.3/1.6 ft-lbs	
	V4		46.64 - 47.00	Su=237/71 psf		68				
						60				
						40				
50						47				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/24	50.00 - 52.00	Push thru vane		73			Grey, wet, soft to medium stiff, clayey SILT with occasional fine sand layers, black staining. 55x110 mm vane raw torque readings: V5 = 9.8/1.8 ft-lbs V6 = 13.5/1.7 ft-lbs	
	V5		50.64 - 51.00	Su=437/80 psf						
	V6		51.64 - 52.00	Su=603/76 psf		84				
						85				
						69				
						69				
55	MD	24/0	55.00 - 57.00	Push thru vane		94			No recovery, similar soils on spoon, medium stiff. 55x110 mm vane raw torque readings: V7 = 12.2/1.7 ft-lbs V8 = 12.8/1.3 ft-lbs	
	V7		55.64 - 56.00	Su=545/76 psf						
	V8		56.64 - 57.00	Su=571/58 psf		57				
						47				
						34				
						31				
60	11D	24/24	60.00 - 62.00	Push thru vane		49			Grey, wet, soft to medium stiff, SILT, some clay, with little fine sand layers, black staining. 55x110 mm vane raw torque readings: V9 = 11.6/2.0 ft-lbs V10 = 11.0/2.1 ft-lbs	G#176644 A-4, CL-ML WC=28.0% LL=23 PL=17 PI=6
	V9		60.64 - 61.00	Su=518/89 psf						
	V10		61.64 - 62.00	Su=491/94 psf		40				
						41				
						38				
						32				
65	12D	24/24	65.00 - 67.00	Push thru vane		52			Similar to above, medium stiff. 55x110 mm vane raw torque readings: V11 = 13.5/3.2 ft-lbs V12 = 13.9/3.7 ft-lbs	
	V11		65.64 - 66.00	Su=603/143 psf						
	V12		66.64 - 67.00	Su=621/165 psf		41				
						46				
						40				
						50				
70	13D	24/24	70.00 - 72.00	Push thru vane		74			Grey, wet, medium stiff, clayey SILT, little fine sand with fine sand layers. 55x110 mm vane raw torque readings: V13 = 17.1/2.0 ft-lbs V14 = 14.4/4.8 ft-lbs	
	V13		70.64 - 71.00	Su=763/89 psf						
	V14		71.64 - 72.00	Su=643/214 psf		61				
						61				
						58				
						53				
75										

Remarks:

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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SSA = Solid Stem Auger

Definitions:
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T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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Definitions:
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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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	14D	24/3	75.00 - 77.00	WOR/4/5/6	9	64			Similar to above, medium stiff.	
	V15		75.64 - 76.00	Su=808/304 psf					55x110 mm vane raw torque readings:	
	MV		76.20 - 76.20			55			V15 = 18.1/6.8 ft-lbs MV = could not push	
						62				
						73				
						128				
80	15D	24/20	80.00 - 82.00	28/58/54/24	112	78			Grey, wet, very stiff, SILT, some fine sand, trace clay.	G#176645 A-2-4, SM WC=21.5%
						bWA			bWashed Ahead of Casing.	
						52				
						39				
						25				
						26				
85	16D	24/24	85.00 - 87.00	WOR/8/11/10	19	62	-74.80		85.000 Grey, wet, medium dense, fine silty SAND, uniform.	
						66				
						55				
						55				
						98				
90	17D	24/6	90.00 - 92.00	18/4/14/9	18	115			Grey, wet, medium dense, fine to coarse silty SAND, little gravel.	G#176646 A-4, CL-ML WC=18.2%
						79				
						82				
						93			c156 blows for 0.4'.	
	R1	60/60	94.80 - 99.80	RQD = 31%		c156				
95						NQ	-84.60		94.800 Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. R1:Core Times (min:sec) 94.8' - 95.8' (5:05) 95.8' - 96.8' (5:50) 96.8' - 97.8' (6:10) 97.8' - 98.8' (6:05) 98.8' - 99.8' (5:15) Recovery=100%	
100							-89.60		99.800	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-101
Location:	York, Maine			**PIN:**	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100									**Bottom of Exploration at 99.80 feet below ground surface.**	
105										
110										
115										
120										
125										

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: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-102
PIN: 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.62		Pavement — 0.580	
5	1D	24/4	5.00 - 7.00	6/6/17/22	23				Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/9	10.00 - 12.00	2/6/11/7	17				Brown, wet, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
15	3D	24/17	15.00 - 17.00	6/8/5/7	13	24	-4.80		15.000 — Grey, wet, medium dense, fine silty SAND with sea shells.	
						14				
						18				
						17				
						17				
20	4D	24/16	20.00 - 22.00	2/2/2/2	4	37	-9.80		20.000 — Grey, wet, soft, fine sandy SILT, little clay, trace gravel and medium to coarse sand with sea shells.	G#176647 A-4, CL-ML WC=31.6%
						24				
						23				
						23				
						24				
25										

Remarks:

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Boring No.: BB-YR34-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-102
Location:	York, Maine			**PIN:**	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/24	25.00 - 27.00	1/WOH/WOH/2	---	32			Grey, wet, soft, SILT, little clay, trace fine sand.	G#176648 A-4, ML WC=45.5% LL=37 PL=33 PI=4
						31				
	V1		27.64 - 28.00	aSu=1964/- psf		36	-16.80		27.000 55x110 mm vane raw torque readings: V1 = 44/-- ft-lbs aVane reached maximum torque reading without shearing, no remolded was attempted.	
						88				
						126				
30	6D	24/14	30.00 - 32.00	8/4/7/9	11	45			Olive, wet, stiff, SILT, (Presumpscot upper crust).	
						44				
						38	-21.80		32.000	
						39				
						34				
35						34				
						38				
						38				
						32				
						30				
40	7D V2	24/24	40.00 - 42.00 40.64 - 41.00	Push thru vane Su=446/67 psf		62			Grey, wet, soft, clayey SILT, trace fine sand, black staining. 55x110 mm vane raw torque readings: V2 = 10.0/1.5 ft-lbs V3 = 8.8/1.1 ft-lbs	
	V3		41.64 - 42.00	Su=393/49 psf		50				
						37				
						34				
						30				
45						34				
						30				
						32				
						31				
						30				
50										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-102
Location:	York, Maine			**PIN:**	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	8D	24/24	50.00 - 52.00	Push thru vane		54			Grey, wet, soft, silty CLAY with trace fine sand layers and black staining. 55x110 mm vane raw torque readings: V4 = 12.1/1.6 ft-lbs V5 = 7.4/1.2 ft-lbs	G#176649 A-6, CL WC=40.0% LL=32 PL=20 PI=12
	V4		50.64 - 51.00	Su=540/71 psf						
	V5		51.64 - 52.00	Su=330/54 psf		41				
						39				
						35				
						33				
55						37				
						35				
						33				
						33				
						34				
60	9D	24/24	60.00 - 62.00	Push thru vane		73			Similar to above, soft to medium stiff. 55x110 mm vane raw torque readings: V6 = 5.4/2.5 ft-lbs V7 = 12.0/2.2 ft-lbs	
	V6		60.64 - 61.00	Su=241/112 psf						
	V7		61.64 - 62.00	Su=536/98 psf		56				
						53				
						45				
						42				
65						48				
						40				
						45				
						38				
						34				
70	10D	24/24	70.00 - 72.00	Push thru vane		52			Grey, wet, medium stiff, clayey SILT, little fine sand. 55x110 mm vane raw torque readings: V8 = 21.0/5.6 ft-lbs V9 = 12.5/3.0 ft-lbs	
	V8		70.64 - 71.00	Su=937/250 psf						
	V9		71.64 - 72.00	Su=558/134 psf		47				
						47				
						46				
						41				
75										

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-102
Location:	York, Maine			**PIN:**	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	11D ~~MV~~	24/14	75.00 - 77.00 ~~75.20 - 75.20~~	1/1/8/13	9	44			Grey, wet, loose, fine SAND, some silt, little clay, trace medium sand, uniform. Attempt 55x110 mm vane: could not push	G#176650 A-4, SC-SM WC=20.3%
						46				
						48				
						56				
						53				
80	12D	24/24	80.00 - 82.00	WOR/WOR/7/11	7	87			Grey, wet, soft, silty CLAY with black staining from 80.0-81.0' bgs.	
						76	-70.80		81.000 Grey, wet, loose, silty fine SAND, uniform from 81.0-82.0' bgs.	
						72				
						71				
						67				
85						81				
						166				
						138				
						157				
						184				
90	13D	24/24	90.00 - 92.00	11/7/11/8	18	90 bWA			Grey, wet, medium dense, fine silty SAND, uniform. bWashed Ahead of Casing to 98.1' bgs.	
						81				
						58				
						53				
						82				
95						103				
						66				
						64				
						70				
100	14D	24/6	99.00 - 101.00	13/28/13/11	41	64			Grey, wet, dense, fine to coarse silty SAND.	

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Station 34 Bridge over Tidal Estuary **Location:** York, Maine	**Boring No.:** BB-YR34-102 **PIN:** 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100						51				
						86				
						194				
						c214			c214 blows for 0.8'.	
						dWA			dWashed Ahead to 105.5' bgs.	
105	R1	46/46	105.50 - 109.33	RQD = 88%		NQ	-95.30		105.500	
									Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. At 108.3 ft bgs (El. -98.1) change to dark grey/green quenched andesite porphyry.	
									R1:Core Times (min:sec) 105.5' - 106.5' (5:05) 106.5' - 107.5' (7:30) 107.5' - 108.5' (7:20) 108.5' - 109.33' (8:20) Recovery=100% Core Blocked	
							-99.10		109.300	
110									**Bottom of Exploration at 109.30 feet below ground surface.**	
115										
120										
125										

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 103, Station 34 Bridge
Location: York, Maine

Boring No.: BB-YR34-201
PIN: 11066.00

Driller:	New Hamshire Boring	**Elevation (ft.)**		**Auger ID/OD:**	NA
Operator:	Greg Leavitt	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/15/08-09/16/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	na
Boring Location:	St. 32+07	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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			0.0 - 0.5				HP	-0.5		Asphalt (approximately 6") HP=Hydraulic Push — 0.5	
5											
	1D	24/20	9.0 - 11.0	13-12-13-23	25	19	10			Medium Dense, Brown, fine to coarse SAND, little Gravel and Silt.	
10							12				
							11				
							10				
							12				
							22				
15							24				
							15				
							11				
							12	-19.0		19.0	
	2D	24/24	19.0 - 21.0	1-2-1-2	3	2	HP			Soft, Gray, SILT, with Organics, trace Gravel, Shells; Hydrogen Sulfide odor.	
20											
	3D	24/20	24.0 - 26.0	WOH-4-8-9	12	9				Top 10": Light brown, SILT with Organics. Bottom 10": Stiff, Gray, Clayey Silt.	
25											

Remarks:

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Boring No.: BB-YR34-201

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Route 103, Station 34 Bridge **Location:** York, Maine	**Boring No.:** BB-YR34-201 **PIN:** 11066.00

Driller:	New Hamshire Boring	**Elevation (ft.)**		**Auger ID/OD:**	NA
Operator:	Greg Leavitt	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/15/08-09/16/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	na
Boring Location:	St. 32+07	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							HP				
	U1	24/24	29.0 - 31.0							Gray, Clayey Silt.	A-6, CL WC=29% LL=39, PL=20 PI=19 Tv=1.4 tsf UW=118 pcf
30							NR			NR=Not Recorded	
	4D	24/24	31.0 - 33.0	1-1-1-1	2	2				Soft, Gray, Clayey Silt.	
	5D	24/0	34.0 - 36.0	5-4-3-3	7	5				No Recovery.	
35											
	U2	24/23	39.0 - 41.0							Gray, SILT and CLAY.	A-6, CL WC=42% LL=40, PL=21 PI=19 UW=107 pcf
40											
	6D	24/24	41.0 - 43.0	WOH-2-1-1	3	2				Soft, Gray, SILT and CLAY with Black Staining, trace Gravel..	
	U3	24/24	44.0 - 46.0							Gray, SILT and CLAY	A-6, CL WC=44% LL=38, PL=21 PI=17 UW=110 pcf
45											
	7D	24/24	46.0 - 48.0	WOR						Very Soft, Gray, SILT and CLAY	
50											

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Route 103, Station 34 Bridge Location: York, Maine	Boring No.: BB-YR34-201 PIN: 11066.00

Driller:	New Hamshire Boring	Elevation (ft.)		Auger ID/OD:	NA
Operator:	Greg Leavitt	Datum:		Sampler:	Standard Split Spoon
Logged By:	Keith Rudman	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/15/08-09/16/08	Drilling Method:	Cased Wash Boring	Core Barrel:	na
Boring Location:	St. 32+07	Casing ID/OD:	4"/4.5"	Water Level*:	
Hammer Efficiency Factor:	0.45	Hammer Type:	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							NR			NR=Not Recorded	
55											
	U4	24/24	59.0 - 61.0							Gray, SILT and CLAY	A-7-6, CL WC=43% LL=45, PL=23 PI=22 UW=118 pcf
60											
	8D	24/8	61.0 - 63.0	WOR/12"-WOH/12"	<2					Very Soft, Gray, SILT and CLAY	
65											
	MU ~~9D~~	24/0 ~~24/1~~	69.0 - 71.0 ~~69.0 - 71.0~~	WOR						Very Soft, Gray, SILT and CLAY.	
70											
	MU ~~10D~~	24/0 ~~24/10~~	72.0 - 74.0 ~~72.0 - 74.0~~	WOR						Very soft, Gray, SILT and fine SAND, little Clay	
	5U	24/24	74.0 - 76.0								
75											

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 103, Station 34 Bridge		**Boring No.:**	BB-YR34-201
Location: York, Maine		**PIN:**	11066.00

Driller:	New Hamshire Boring	**Elevation (ft.)**		**Auger ID/OD:**	NA
Operator:	Greg Leavitt	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/15/08-09/16/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	na
Boring Location:	St. 32+07	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒	Rope & Cathead ☐	

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person

S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75							NR			NR=Not Recorded	
	11D	24/12	76.0 - 78.0	WOR-3-7-13	10	8				Medium Dense, Gray, fine SAND, little Silt	
								-78.0		78.0	
										Bottom of Exploration at 78.00 feet below ground surface. No Refusal.	
80											
85											
90											
95											
100											

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.

Boring No.: BB-YR34-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 103, Station 34 Bridge
Location: York, Maine

Boring No.: BB-YR34-201A
PIN: 11066.00

Driller:	New Hamshire Boring	**Elevation (ft.)**		**Auger ID/OD:**	NA
Operator:	Greg Leavitt	**Datum:**		**Sampler:**	Thin Wall Tube
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	
Date Start/Finish:	10/17/08-10/17/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	
Boring Location:	St. 32+07	**Casing ID/OD:**	4"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**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
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							NR			See Note 1. NR=Not Recorded	
5											
10											
15											
20											
25											

Remarks:

1. Advanced casing without sampling to first tube sample interval. (39- 41 feet.)

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 103, Station 34 Bridge			**Boring No.:**	BB-YR34-201A
Location:	York, Maine			**PIN:**	11066.00
Driller:	New Hamshire Boring	**Elevation (ft.)**		**Auger ID/OD:**	NA
Operator:	Greg Leavitt	**Datum:**		**Sampler:**	Thin Wall Tube
Logged By:	Keith Rudman	**Rig Type:**	Truck	**Hammer Wt./Fall:**	
Date Start/Finish:	10/17/08-10/17/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	
Boring Location:	St. 32+07	**Casing ID/OD:**	4"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR = weight of rods
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S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							NR			NR=Not Recorded	
30											
35											
								-39.0		-39.0	
	U-1	24/24	39.0 - 41.0							Soft, Gray, SILT and CLAY. 39 feet: Tv=285 psf 41 feet: Tv=300 psf	A-6, CL WC=38.6% LL=30, PL=19 PI=11 Su=320/40 psf UW=132 pcf
40											
45											
50	U-2	24/21	49.0 - 51.0							Soft, Gray, SILT and CLAY 51 feet: Tv=500 psf	

Remarks:

1. Advanced casing without sampling to first tube sample interval. (39- 41 feet.)

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 103, Station 34 Bridge Location: York, Maine	Boring No.: BB-YR34-201A PIN: 11066.00

Driller:	New Hamshire Boring	Elevation (ft.)		Auger ID/OD:	NA
Operator:	Greg Leavitt	Datum:		Sampler:	Thin Wall Tube
Logged By:	Keith Rudman	Rig Type:	Truck	Hammer Wt./Fall:	
Date Start/Finish:	10/17/08-10/17/08	Drilling Method:	Cased Wash Boring	Core Barrel:	
Boring Location:	St. 32+07	Casing ID/OD:	4"	Water Level*:	
Hammer Efficiency Factor:	0.45	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
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
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S_u = Insitu Field Vane Shear Strength (psf)
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q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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							NR			NR=Not Recorded	
	U-3	24/22	54.0 - 56.0							Soft, Gray, SILT and fine SAND.	
55											
	U-4	24/24	59.0 - 61.0							Soft, Gray, SILT and CLAY. 61 feet: Tv=460 psf	A-6, CL WC=30.3% LL=38, PL=23 PI=15 Su=520/80 psf UW=137 pcf
60											
								-61.0		61.0 **Bottom of Exploration at 61.00 feet below ground surface.** No Refusal.	
65											
70											
75											

Remarks:

1. Advanced casing without sampling to first tube sample interval. (39- 41 feet.)

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

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

APPENDIX C

LAB TEST RESULTS

LABORATORY TESTING DATA SHEET

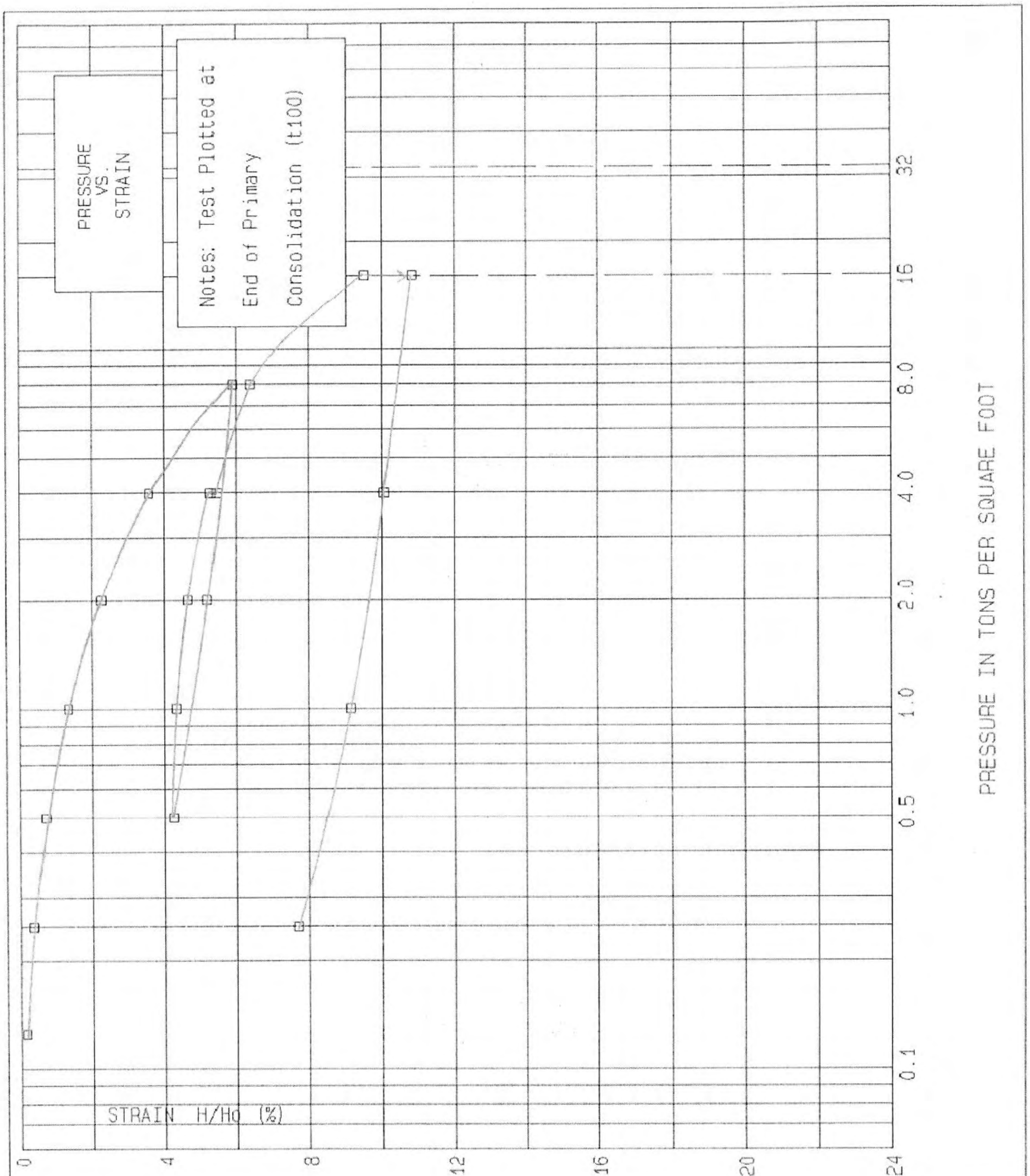
Project Name Rte 103 - Station 34 Bridge
Project No. 09.0025577.00
Project Engineer J. Tooley

Location York, ME
Assigned By K. Rudman
Report Date 10/9/2008

Reviewed By [signature]
Date Reviewed 10/9/08

Boring No.	Sample No.	Depth ft.	Lab No.	Identification Tests: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	ORG %	Dry unit wt. pcf	Strength Tests: Perme-ability cm/sec	Torvane or Type Test	σ_c psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	Consol. $\frac{C_c}{1+e_0}$	Laboratory Log and Soil Description
BB-YR34-201	U-1	29-31	1		Average Total Unit Weight (29.0-31.0') = 118.2 Pcf													
		29.0-29.5		Sample Saved														Gray Lean Clay trace Silt Lenses (CL) Very Stiff Consistency
		29.5		28.5								Tv= 1.25 tsf						
		29.8-29.9		28.8	39	20												
		29.9-30.0		30.1						92.3							0.12	
		30.1		28.1								Tv= 1.40 tsf						
		30.1-30.6		Sample Saved														
		30.6		27.3								Tv= 1.42 tsf						

GZA GeoEnvironmental, Inc.



SOIL DESCRIPTION: Gray Lean Clay (CL)

SAMPLE DIAM 2.5 IN. LIQUID LIMIT 39 % PLASTIC LIMIT 20 % SPECIFIC GRAVITY 2.67

	WATER CONTENT, %	DRY UNIT WEIGHT, pcf	VOID RATIO	SATURATION, %	SAMPLE HEIGHT, INCH
INITIAL	30.10	92.26			0.80
FINAL	26.80	99.87			0.74

Rte 103-Station 34 Bridge

CONSOLIDATION TEST

BORING NO. BBYR3420I
SAMPLE U-1
DEPTH 29.9-30.0'
TECH. MBP
REVIEWER DAS

TEST SERIES NO. C1.1
DATE 10/9/08
FILE 25577.00

FIGURE

Job Number 25577.00
Project Name .... Rte 103-Station 34 Bridge
Test Number C1.1
Sample Description .. Gray Lean Clay (CL)
Boring Number BBYR3420
Sample Number U-1
Depth .... 29.9-30.0'

Technician .... MBP
Reviewer .... DAS

NOTES: Test Plotted at
End of Primary
Consolidation (t100)

Specific Gravity .... 2.6700E+00
Sample Dry Weight .... 9.5100E+01
Sample Diameter .... 2.5000E+00
Initial Void Ratio .... 0.0000E+00
Initial Sample Height 8.0000E-01
Final Sample Height 7.3900E-01
Initial Water Content .... 3.0100E+01
Final Water Content ...... 2.6800E+01
Liquid Limit .... 3.9000E+01
Plastic Limit .... 2.0000E+01
Initial Saturation .... 0.0000E+00
Final Saturation .... 0.0000E+00
T50 data excluded
T90 data included

Increment umber	Pressure (TSF)	Final Dial	Percent Strain	Void Ratio	T90 (min)	Cv (T90) (/10000) cm²/sec	Cα
1	0.000	0.0	0.00	0.806			
2	0.125	12.0	0.15	0.803			
3	0.250	28.0	0.35	0.800			
4	0.500	57.0	0.71	0.793	2.6	55.52	
5	1.000	107.0	1.34	0.782	1.7	84.07	
6	2.000	181.0	2.26	0.765	2.3	61.17	
7	4.000	288.0	3.60	0.741	2.9	47.40	
8	8.000	473.0	5.91	0.699	4.0	33.09	
9	2.000	416.0	5.20	0.712			
10	0.500	340.0	4.25	0.729			
11	1.000	347.0	4.34	0.728	1.0	133.63	
12	2.000	374.0	4.67	0.721	2.0	66.52	
13	4.000	422.0	5.27	0.711	2.3	57.28	
14	4.000	437.0	5.46	0.707			0.0055
15	8.000	511.0	6.39	0.691	3.6	35.87	
16	16.000	760.0	9.50	0.634	5.8	21.32	
17	16.000	867.0	10.84	0.610			0.0051
18	4.000	805.0	10.06	0.624			
19	1.000	730.0	9.12	0.641			
20	0.250	615.0	7.69	0.667			

LABORATORY TESTING DATA SHEET

Project Name Rte 103 - Station 34 Bridge
Project No. 09.0025577.00
Project Engineer J. Tooley

Location York, ME
Assigned By K. Rudman
Report Date 10/9/2008

Reviewed By [signature]
Date Reviewed 10/9/08

				Identification Tests							Strength Tests						Consol.	
Boring No.	Sample No.	Depth ft.	Lab No.	Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	ORG %	Dry unit wt. pcf	Perme-ability cm/sec	Torvane or Type Test	σ_c psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	$\frac{C_c}{1 + e_0}$	Laboratory Log and Soil Description
BB-YR34-201	U-2	39-41	2		Average Total Unit Weight (39.0-40.9') = 106.6 Pcf													
		39.5		35.5								Tv= 0.10 tsf						**Entire Sample Disturbed** Gray Lean Clay trace Silt Lenses (CL) Very Soft Consistency
		39.8-40.0		42.1	40	21												
		40.1		39.5								Tv= 0.11 tsf						
		40.6		41.2								Tv= 0.10 tsf						

GZA GeoEnvironmental, Inc.

LABORATORY TESTING DATA SHEET

Project Name Rte 103 - Station 34 Bridge
Project No. 09.0025577.00
Project Engineer J. Tooley

Location York, ME
Assigned By K. Rudman
Report Date 10/9/2008

Reviewed By [signature]
Date Reviewed 10/9/08

Boring No.	Sample No.	Depth ft.	Lab No.	Identification Tests: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	ORG %	Dry unit wt. pcf	Strength Tests: Perme-ability cm/sec	Torvane or Type Test	σ_c psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	Consol. $\frac{C_c}{1+e_0}$	Laboratory Log and Soil Description
BB-YR34-201	U-3	44-46	3		Average Total Unit Weight (44.0-46.0') = 110.0 Pcf													
		44.5		43.8								Tv= 0.10 tsf						**Entire Sample Disturbed**
		44.8-45.0		43.5	38	21												Gray Lean Clay
		45.1		41.8								Tv= 0.12 tsf						(CL) Very Soft Consistency
		45.6		47.0								Tv= 0.11 tsf						

GZA GeoEnvironmental, Inc.

LABORATORY TESTING DATA SHEET

Project Name Rte 103 - Station 34 Bridge
Project No. 09.0025577.00
Project Engineer J. Tooley

Location York, ME
Assigned By K. Rudman
Report Date 10/9/2008

Reviewed By [signature]
Date Reviewed 10/9/08

				Identification Tests							Strength Tests						Consol.	
Boring No.	Sample No.	Depth ft.	Lab No.	Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	ORG %	Dry unit wt. pcf	Perme-ability cm/sec	Torvane or Type Test	σ_c psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	$\frac{C_c}{1 + e_0}$	Laboratory Log and Soil Description
BB-YR34-201	U-4	59-61	4		Average Total Unit Weight (59.0-61.0') = 117.5 Pcf													
		59.5		37.2								Tv= 0.10 tsf						**Entire Sample Disturbed**
		59.7-59.9		42.8	45	23												Gray Lean Clay trace fine Sand trace Silt Lenses (CL) Very Soft Consistency
		60.1		34.0								Tv= 0.11 tsf						
		60.6		28.4								Tv= 0.10 tsf						

GZA GeoEnvironmental, Inc.

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Route 103 Bridge

MDOT Project Number:

Town(s): York

GZA Project Number: 09.0025577.00

Boring Identification Number	Sample Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
										Unified	AASHTO	Frost
BB-YR34-201	S-2			19-21			31.2	30	6	ML	A-4	IV
BB-YR34-201	S-3			24-26			35.7	37	16	CL	A-6	IV
BB-YR34-201	S-10			72-74			18.1					
BB-YR34-201	S-11			76-78			19.4					

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

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

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

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

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

R. W. Gillespie & Associates, Inc.

86 Industrial Park Road, Suite 4, Saco, ME 04072 207-286-8008
P.O. Box 289, Augusta, ME 04332 207-623-4914
200 Int'l Drive, Suite 170, Portsmouth, NH 03801 603-427-0244

LETTER OF TRANSMITTAL

Date: October 30, 2008	Project No.: 876-007
Attention: Chris Snow, P.E. (csnow@gza.com)	
Re: Laboratory Testing Misc Services	

GZA GeoEnvironmental, Inc.

Four Free St

Portland, ME 04101

We are sending you attached laboratory test results.

Laboratory No. (s)	Test (s) Performed
10516	2- Atterberg, Lab Vanes, Consol

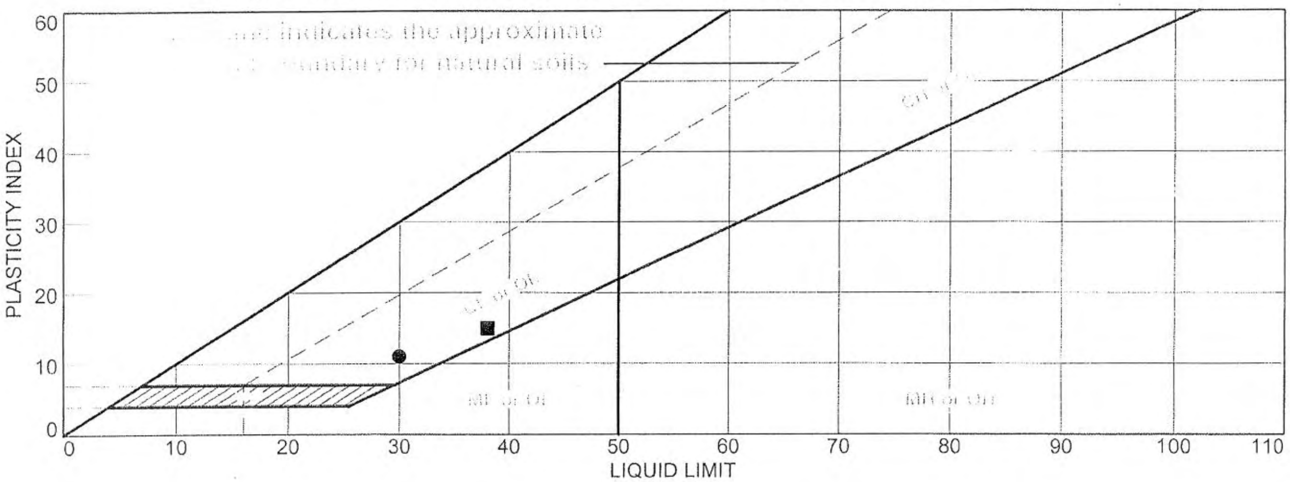
Remarks:

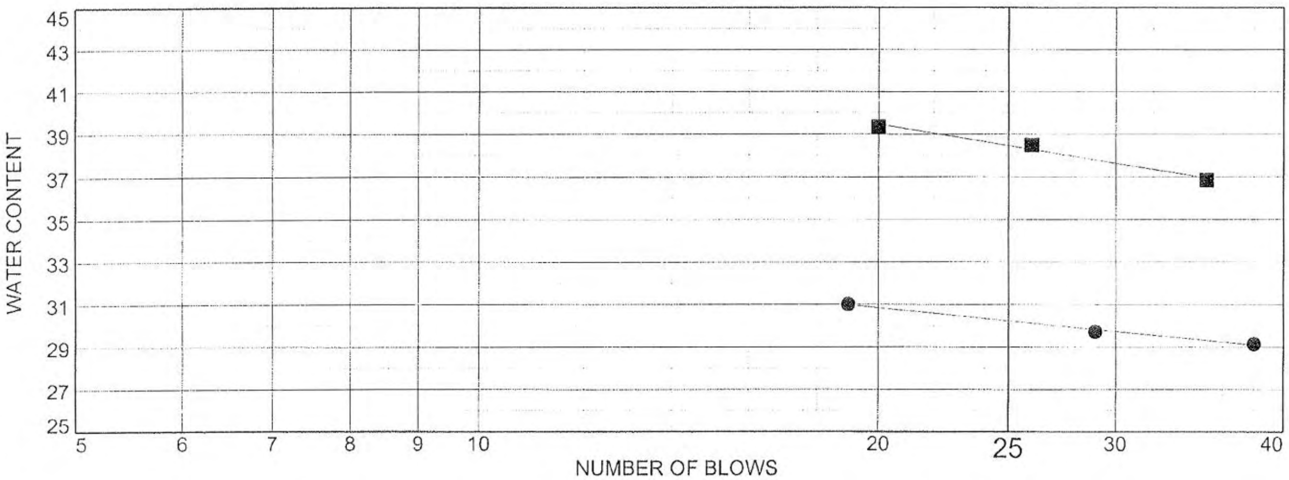
Copy To: none

Signed: Suzan A. Michaud

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

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	silty clay	30	19	11			CL
■	silty clay	38	23	15			CL

Project No. 876-07 **Client:** GZA GeoEnvironmental, Inc.

Project: Route 103 Bridge

● **Sample Source:** BB-YR34-201A **Depth:** 39'-41' **Sample No.:** U-1

■ **Sample Source:** BB-YR34-201A **Depth:** 59'-61' **Sample No.:** U-4

Remarks:

● Natural Moisture: 32.6%

■ Natural Moisture: 41.3%

R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 10516

Tested By: DCH **Checked By:** MTG MTG

Laboratory Vane Shear Test Results

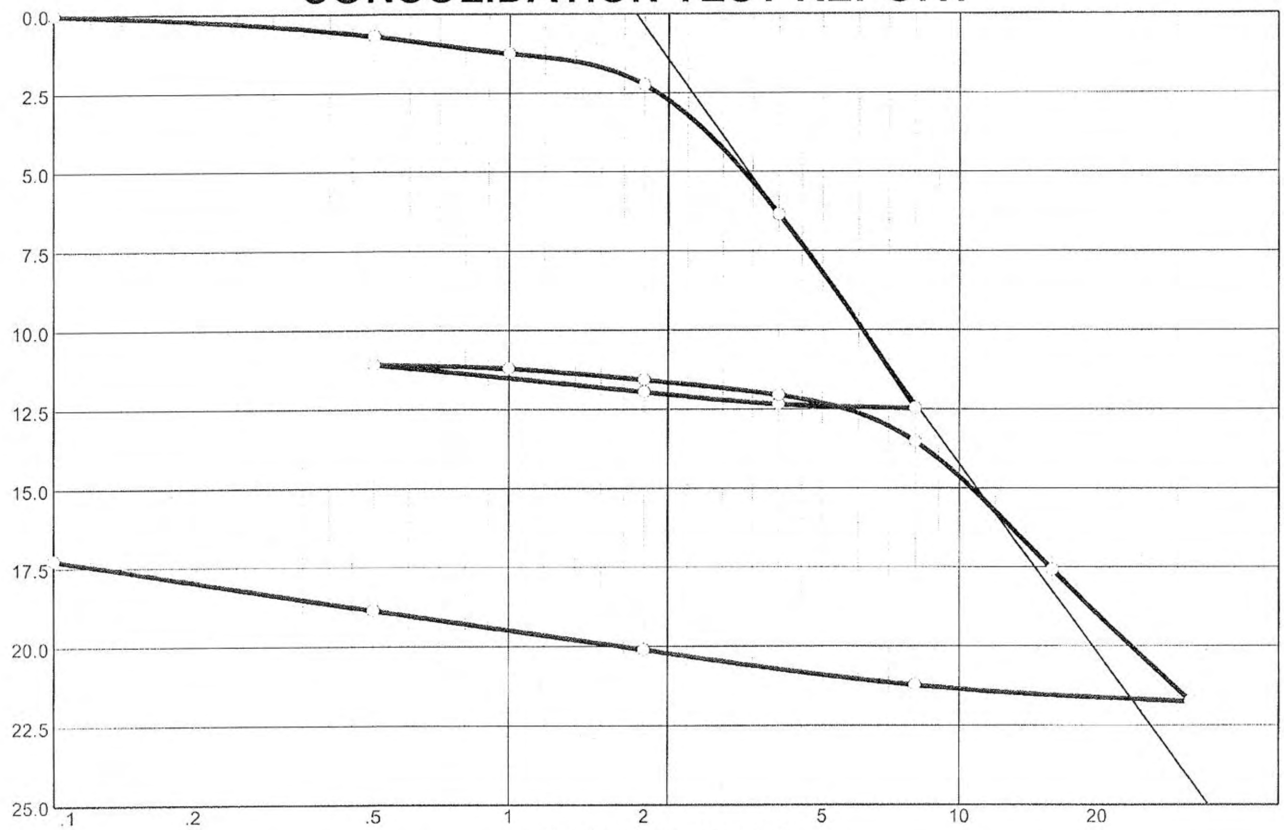
Project:	Route 103 Bridge	**Client:**	GZA
Project No.:	876-07	**Location:**	York, Maine

Boring No.	BB-YR34-201A	**Lab No.**	10516a
Sample No.	U-1 (39'-41')		
Test No.	**S_u (Undisturbed)**	**S_u (Residual)**	**Moisture Content**
1	280 psf	40 psf	34.7%
2	320 psf	40 psf	33.7%
3	500 psf	60 psf	29.3%
4	500 psf	60 psf	32.6%

Tube Wt.	11.875 lbs
Tube Vol.	0.0896 cu. ft.
Wet Density	132.5 pcf
Dry Density	99.0 pcf

MTL

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.10	1.69		11	2.00	0.46	0.001				
2	0.50	0.38		12	4.00	0.44					
3	1.00	0.46		13	8.00	0.27					
4	2.00	0.32		14	16.00	0.16	0.010				
5	4.00	0.13		15	32.00	0.19					
6	8.00	0.11		16	8.00	0.32					
7	4.00	0.98		17	2.00	0.16					
8	2.00	0.48		18	0.50	0.06					
9	0.50	0.21		19	0.10	0.02					
10	1.00	0.32	0.000								

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
93.0 %	38.6 %	80.5	30	11	2.77							

MATERIAL DESCRIPTION	USCS	AASHTO
silty clay		

Project No. 876-07 **Client:** GZA GeoEnvironmental, Inc.

Project: Route 103 Bridge

Source: BB-YR34-201A **Sample No.:** U-1 **Elev./Depth:** 39'-41'

Remarks:
Tested by: DCH

R.W. Gillespie & Associates, Inc.
Saco, Maine

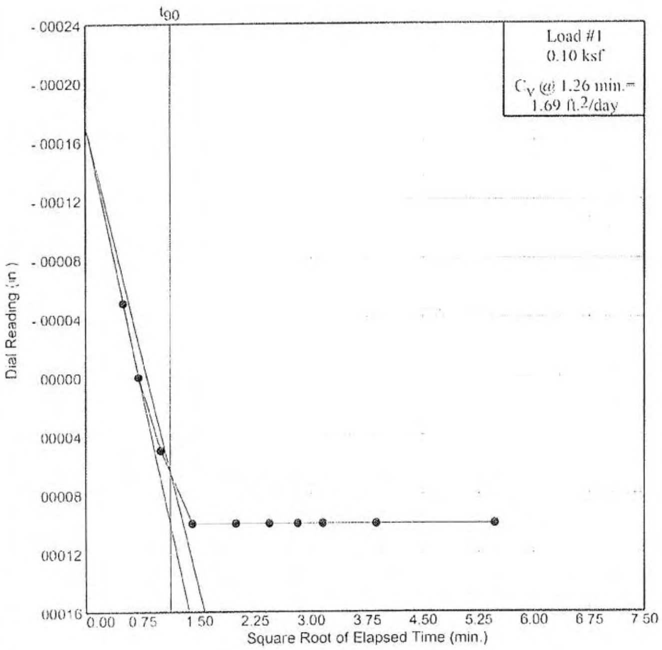
MTG

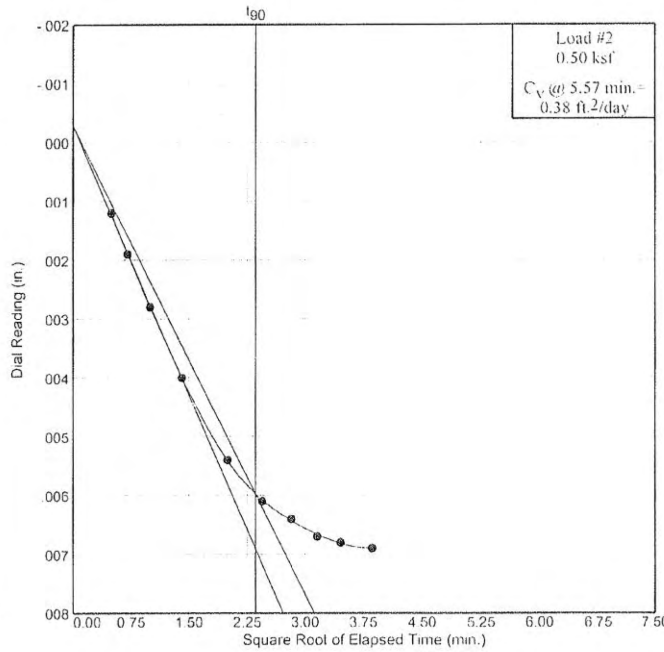
Lab No. 10516a

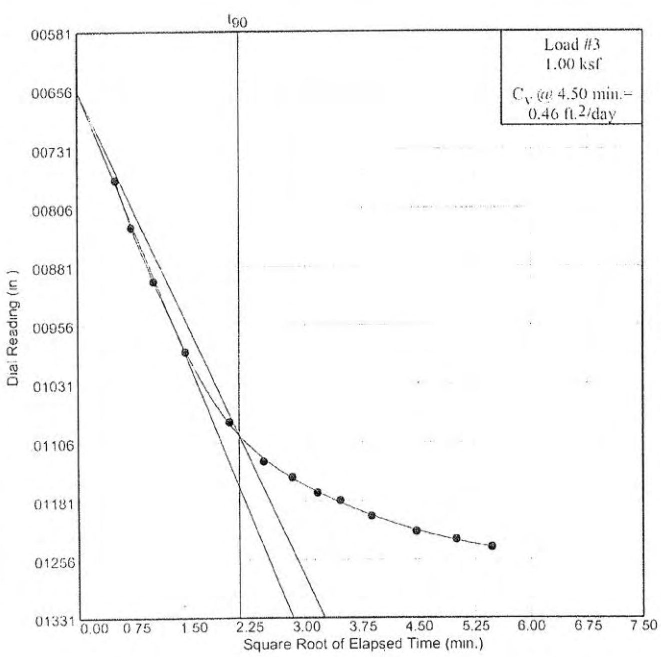
Dial Reading vs. Time

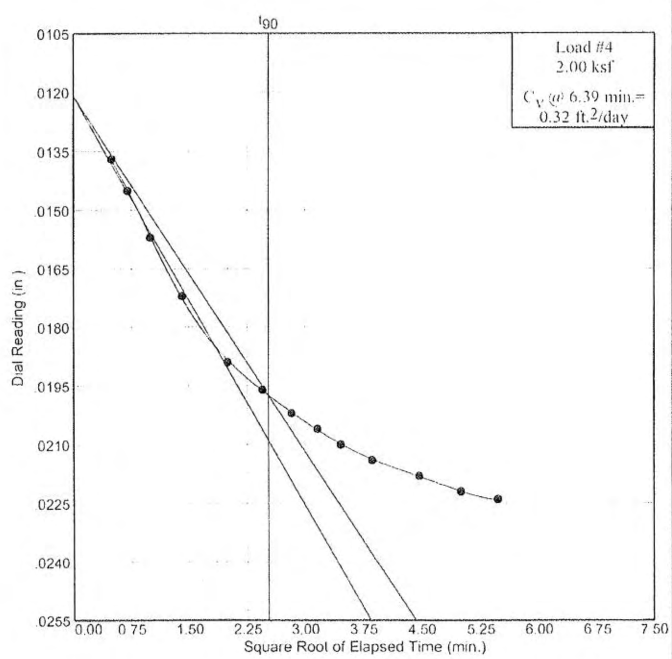
Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A   Sample No.: U-1   Elev./Depth: 39'-41'









R.W. Gillespie & Associates, Inc.
Saco, Maine

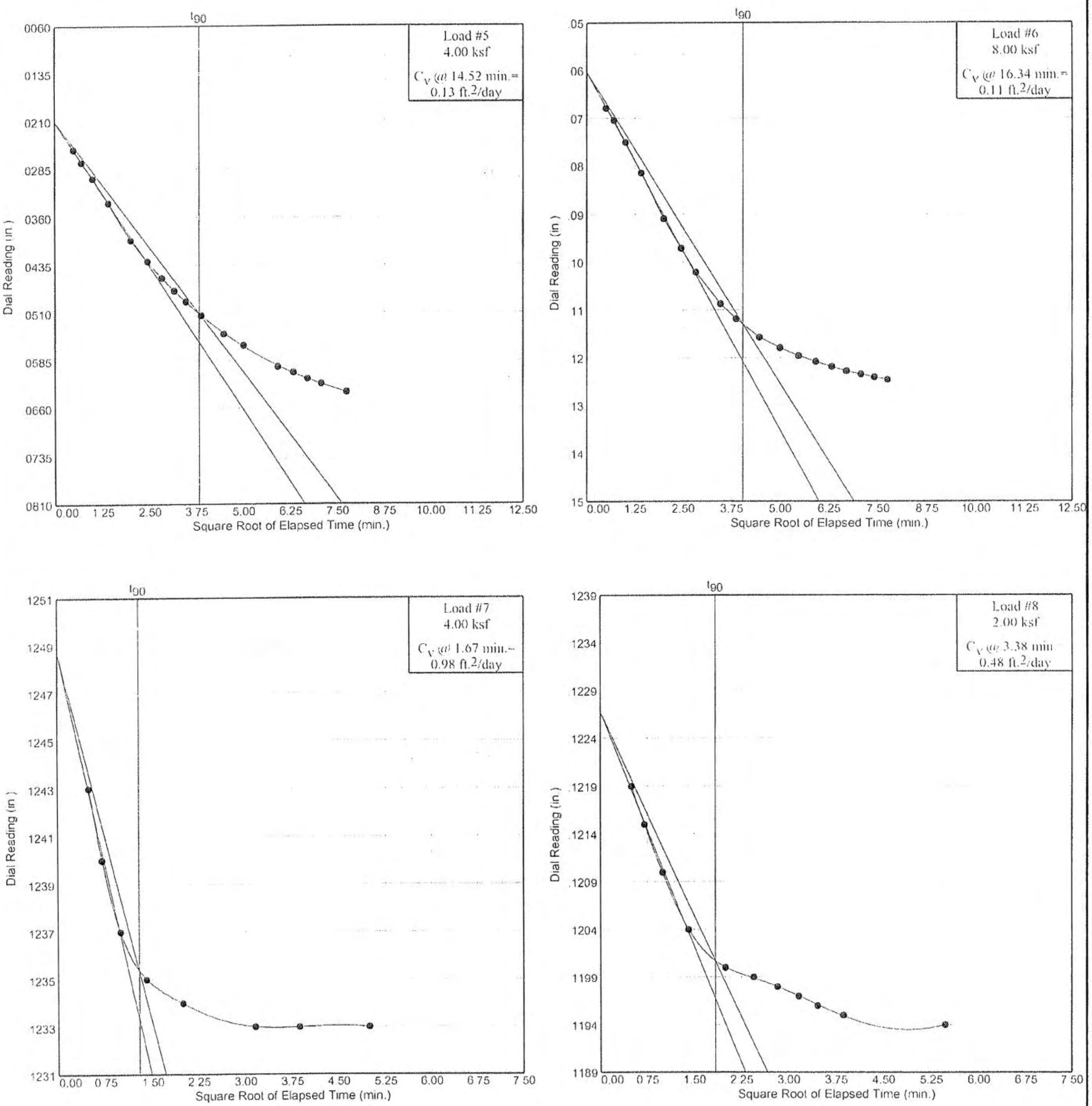
MJC

Lab No. 10516a

Dial Reading vs. Time

Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A   Sample No.: U-1   Elev./Depth: 39'-41'



R.W. Gillespie & Associates, Inc.
Saco, Maine

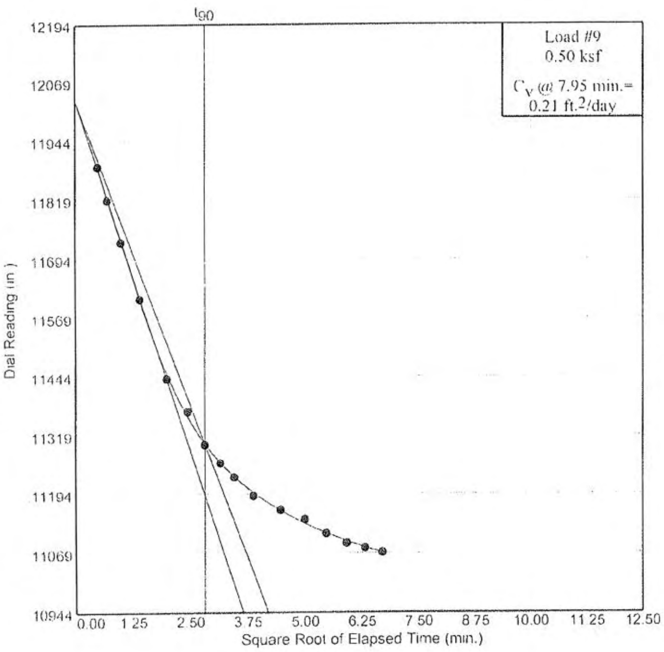
MTG

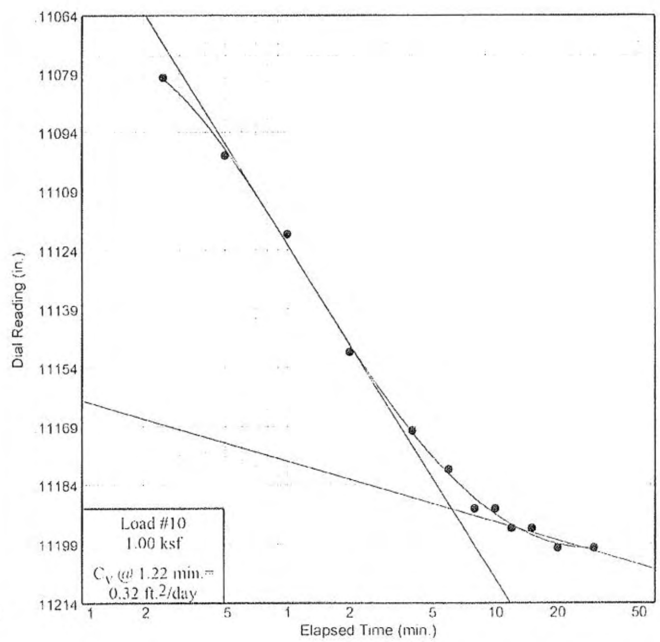
Lab No. 10516a

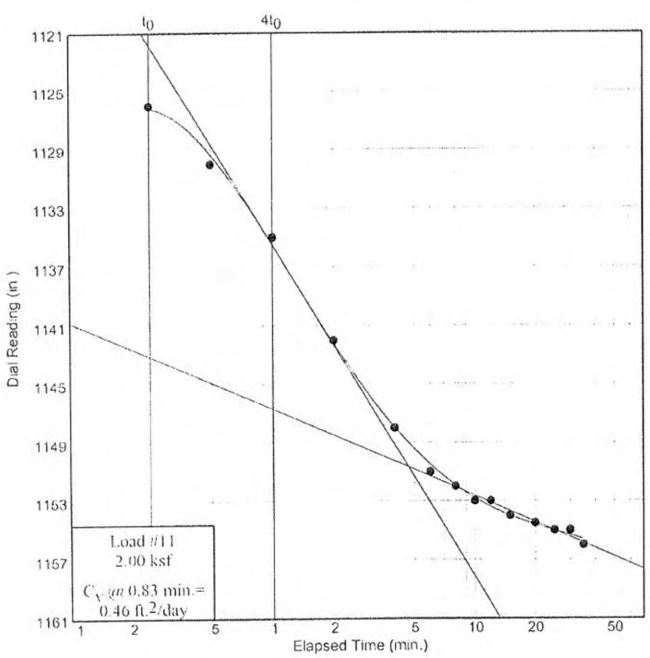
Dial Reading vs. Time

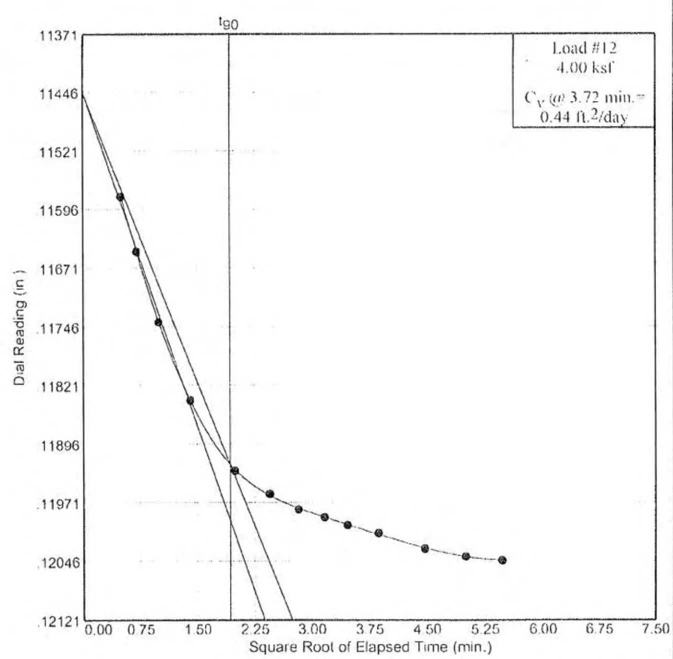
Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A   Sample No.: U-1   Elev./Depth: 39'-41'









R.W. Gillespie & Associates, Inc.
Saco, Maine

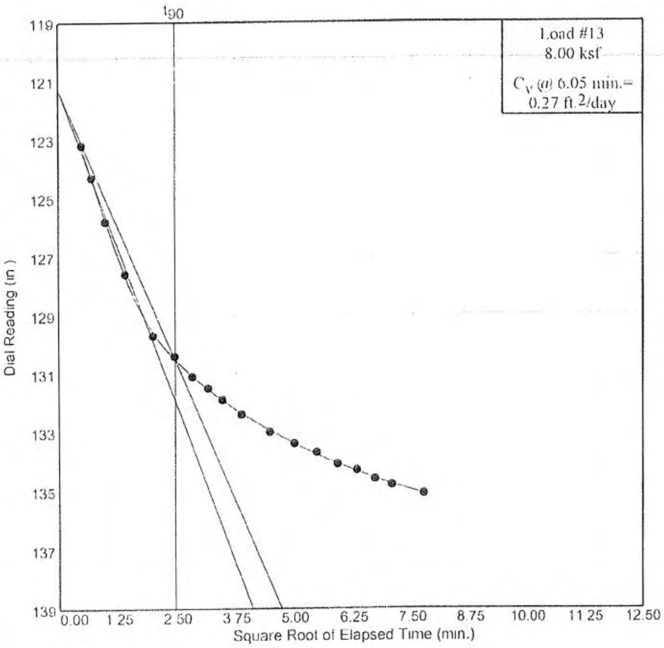
MTZ

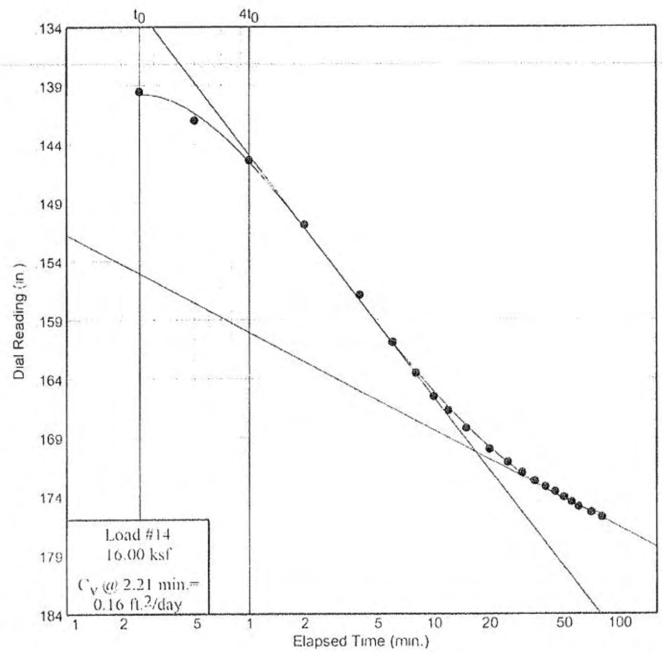
Lab No. 10516a

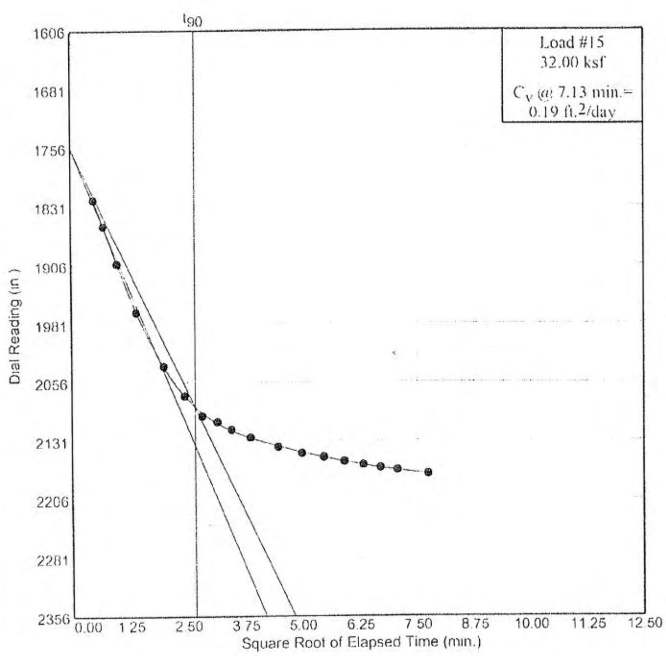
Dial Reading vs. Time

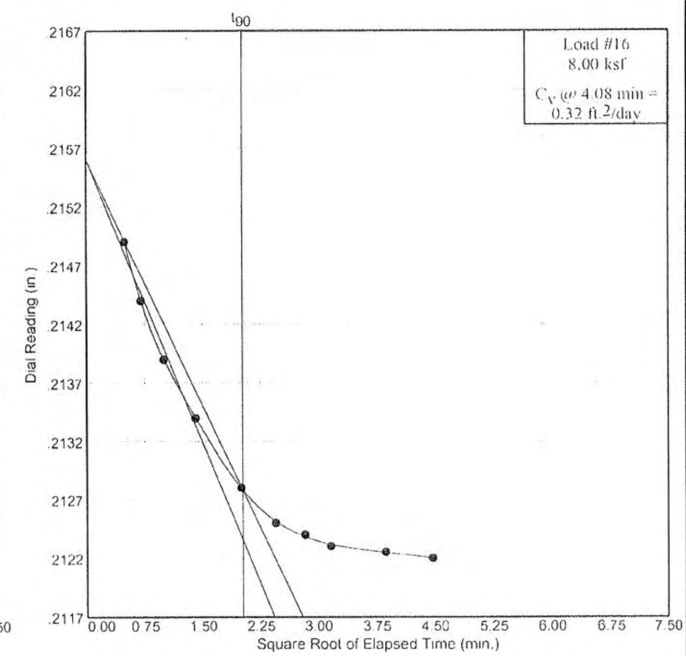
Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A | Sample No.: U-1 | Elev./Depth: 39'-41'







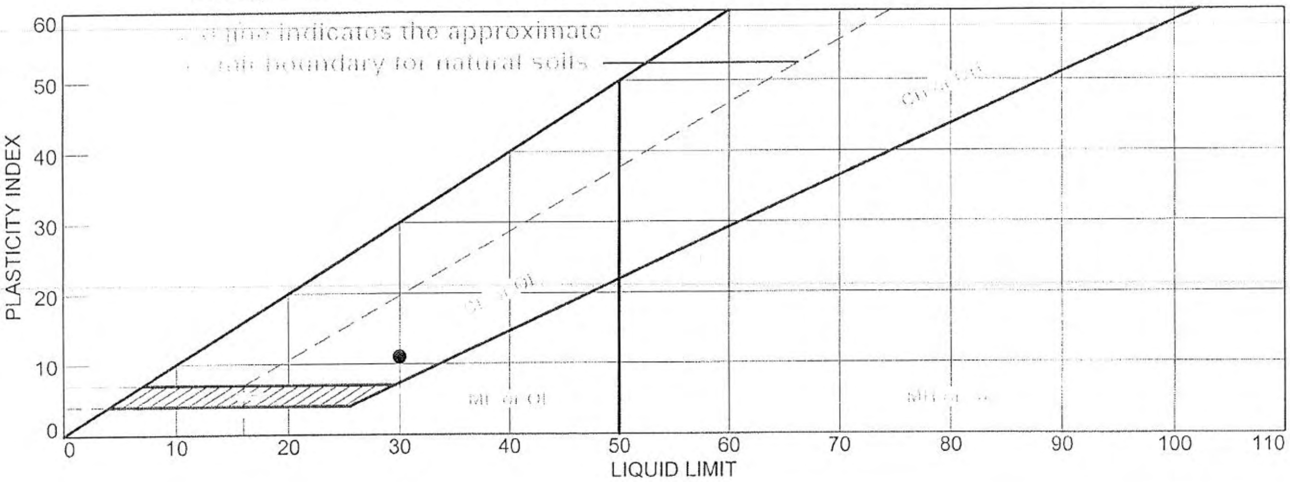


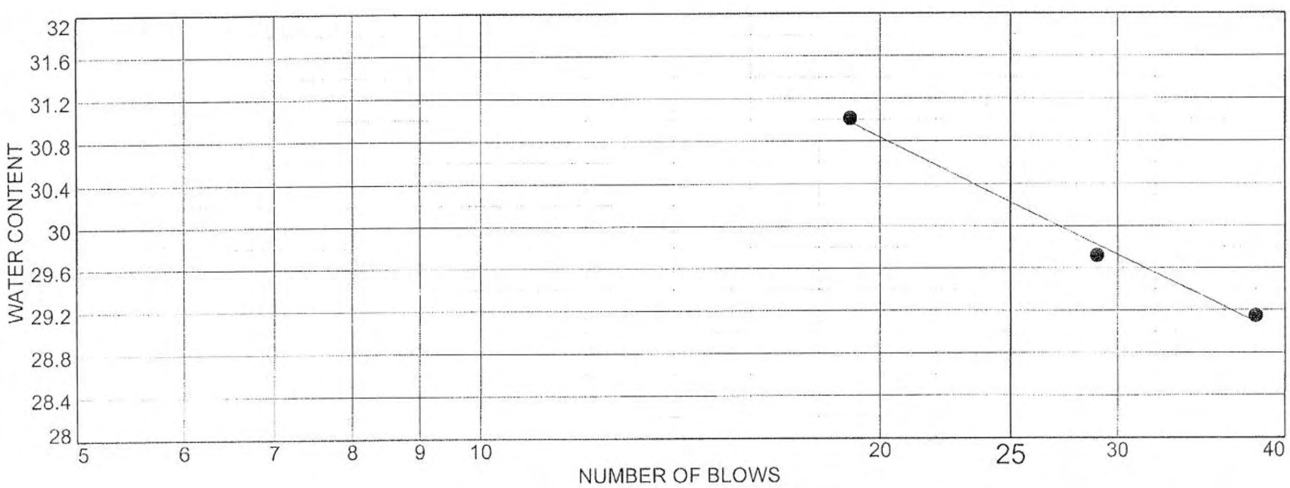
R.W. Gillespie & Associates, Inc.
Saco, Maine

MTG

Lab No. 10516a

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	silty clay	30	19	11			

Project No. 876-07 **Client:** GZA GeoEnvironmental, Inc.

Project: Route 103 Bridge

● **Sample Source:** BB-YR34-201A **Depth:** 39'-41' **Sample No.:** U-1

Remarks:

● Natural Moisture: 32.6%

R.W. Gillespie & Associates, Inc.

Saco, Maine

MTG

Lab No. 10516a

Tested By: DCH **Checked By:** MTG

Laboratory Vane Shear Test Results

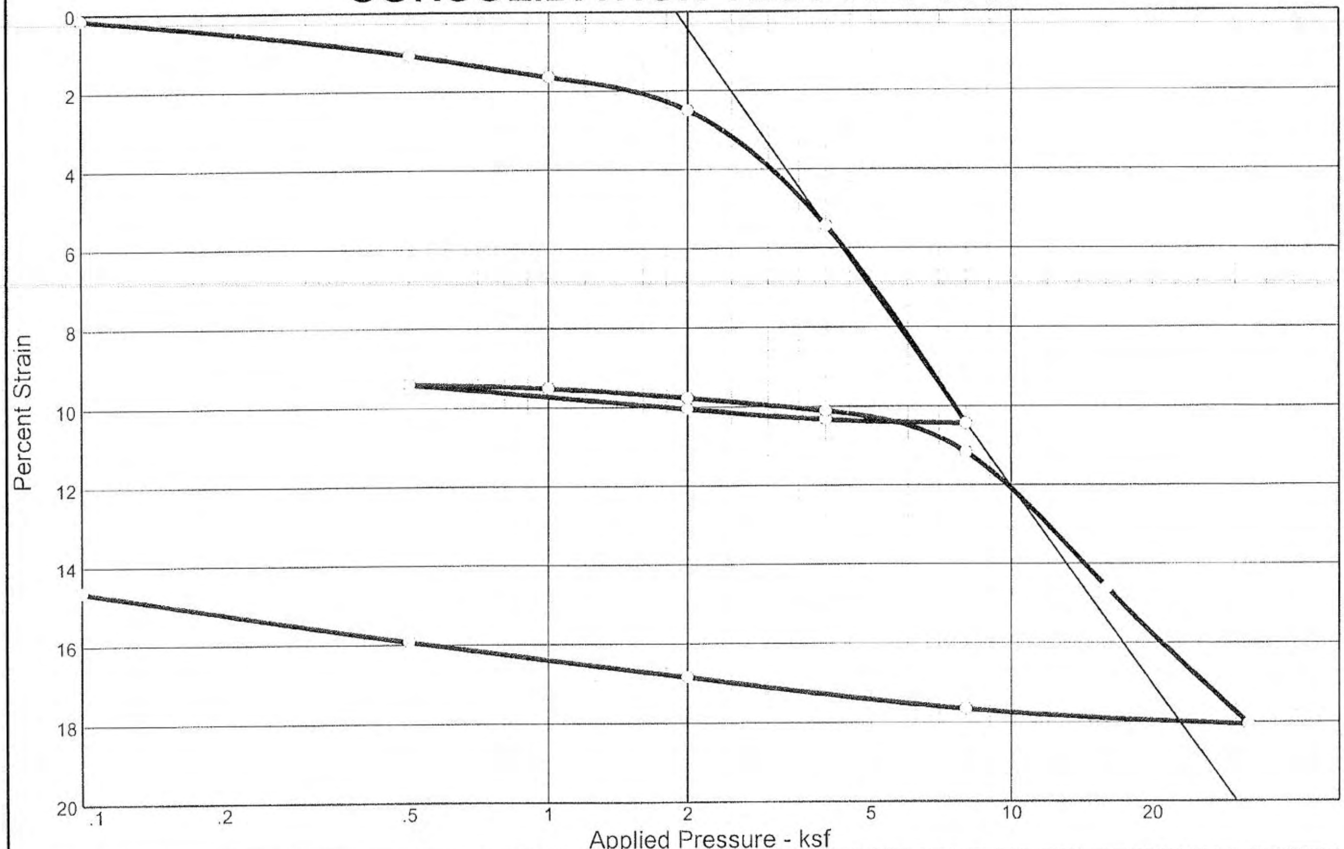
Project:	Route 103 Bridge	**Client:**	GZA
Project No.:	876-07	**Location:**	York, Maine

Boring No.	BB-YR34-201A	**Lab No.**	10516b
Sample No.	U-4 (59'-61')		
Test No.	**S_u (Undisturbed)**	**S_u (Residual)**	**Moisture Content**
1	440 psf	60 psf	44.1%
2	520 psf	80 psf	42.5%
3	520 psf	80 psf	41.3%
4	520 psf	80 psf	30.8%

Tube Wt.	12.295 lbs
Tube Vol.	0.0896 cu. ft.
Wet Density	137.2 pcf
Dry Density	99.6 pcf

MTL

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.^{2}/day)	C_α	No.	Load (ksf)	C_v (ft.^{2}/day)	C_α	No.	Load (ksf)	C_v (ft.^{2}/day)	C_α
1	0.10	1.72		11	2.00	1.24	0.000				
2	0.50	0.52		12	4.00	0.97					
3	1.00	0.51		13	8.00	0.46					
4	2.00	0.82		14	16.00	0.44	0.008				
5	4.00	0.30		15	32.00	0.47					
6	8.00	0.22		16	8.00	0.54					
7	4.00	2.15		17	2.00	0.55					
8	2.00	0.58		18	0.50	0.61					
9	0.50	0.41		19	0.10	0.27					
10	1.00	0.85	0.000								

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
92.6 %	30.3 %	90.8	38	15	2.77							

MATERIAL DESCRIPTION	USCS	AASHTO
silty clay	cl	

Project No. 876-07 **Client:** GZA GeoEnvironmental, Inc.

Project: Route 103 Bridge

Source: BB-YR34-201A **Sample No.:** U-4 **Elev./Depth:** 59'-61'

R.W. Gillespie & Associates, Inc.

Saco, Maine

Remarks:

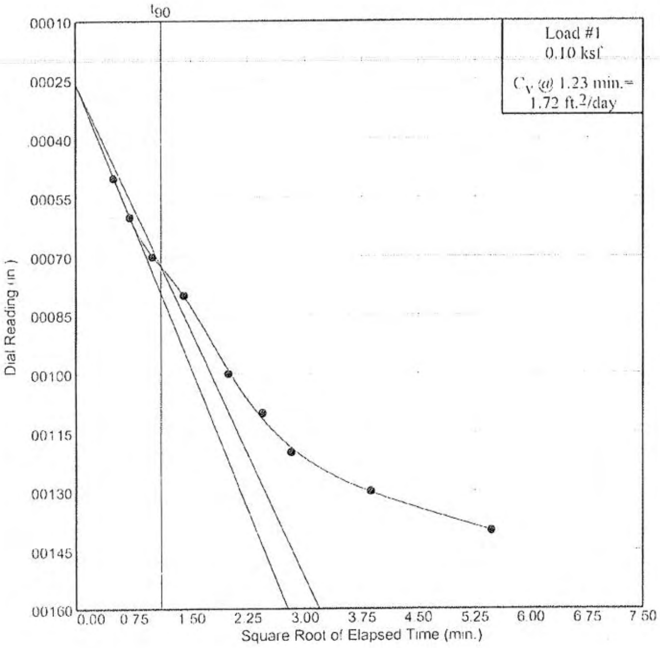
Tested by: DCH

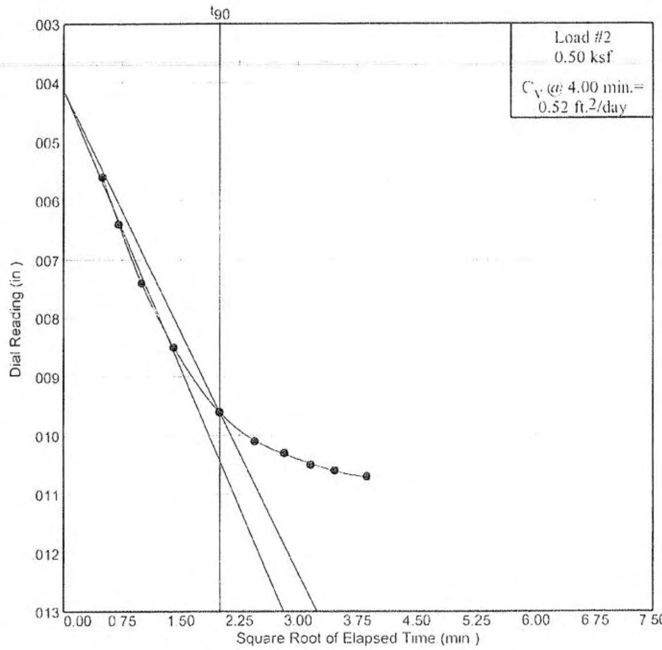
Lab No. 10516b

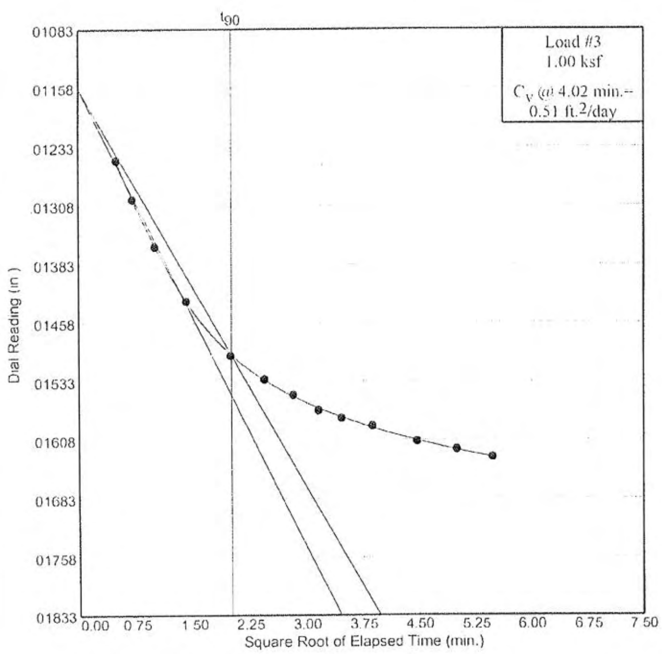
Dial Reading vs. Time

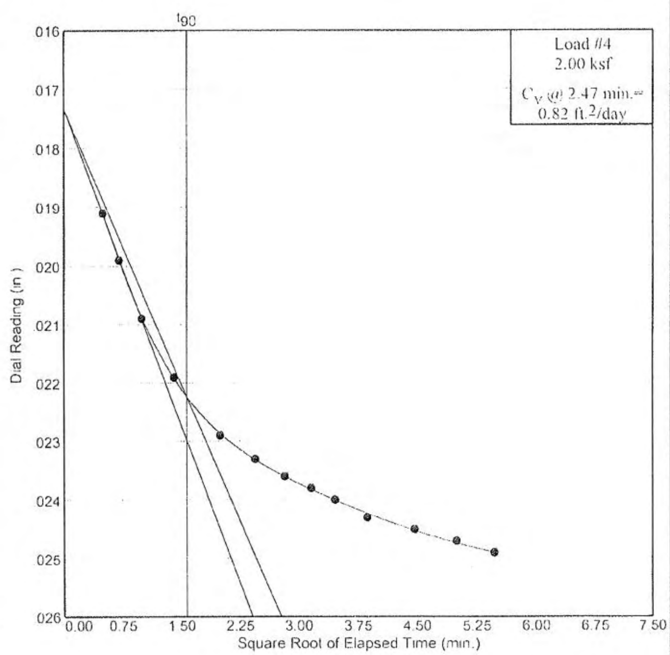
Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A   Sample No.: U-4   Elev./Depth: 59'-61'









R.W. Gillespie & Associates, Inc.
Saco, Maine

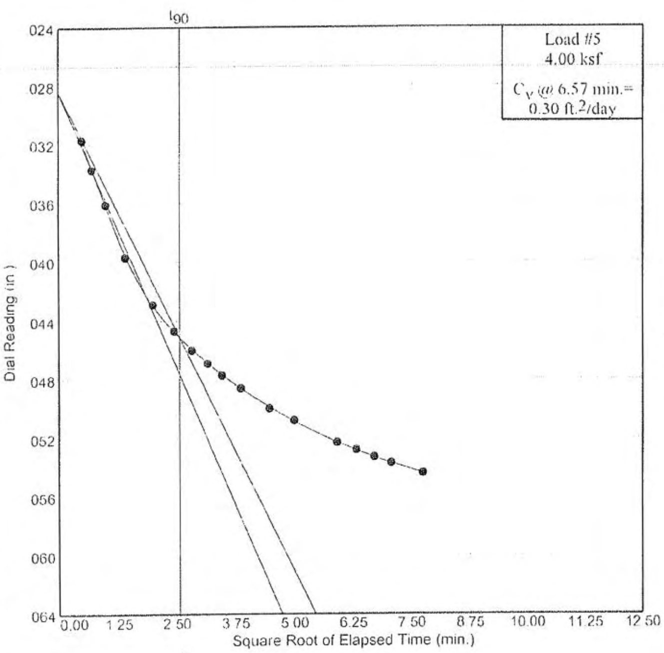
MTB

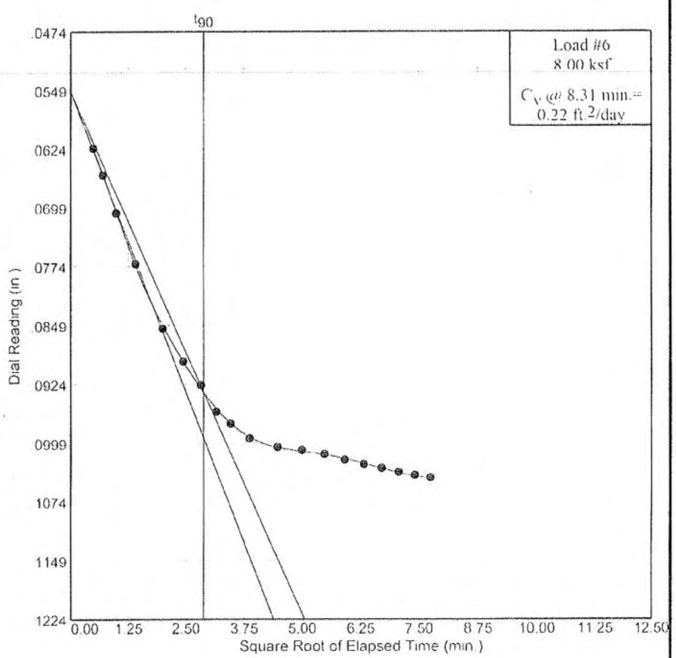
Lab No. 10516b

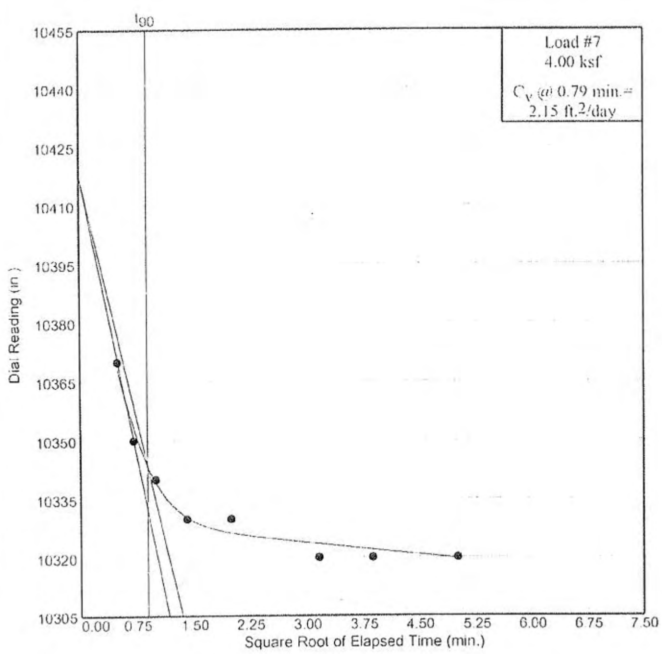
Dial Reading vs. Time

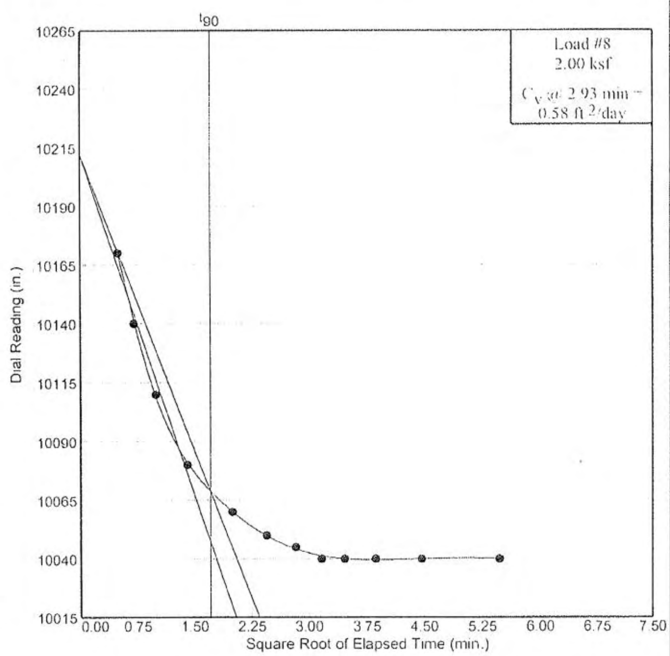
Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A　　Sample No.: U-4　　Elev./Depth: 59'-61'









R.W. Gillespie & Associates, Inc.
Saco, Maine

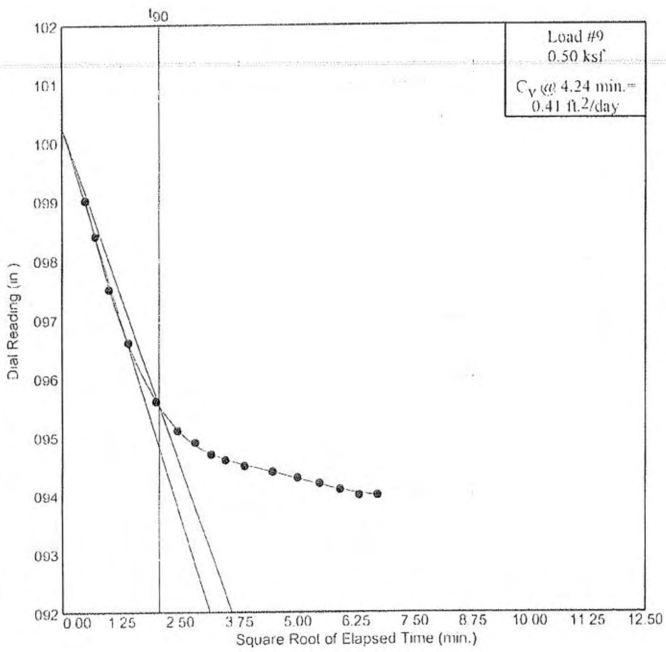
MTG

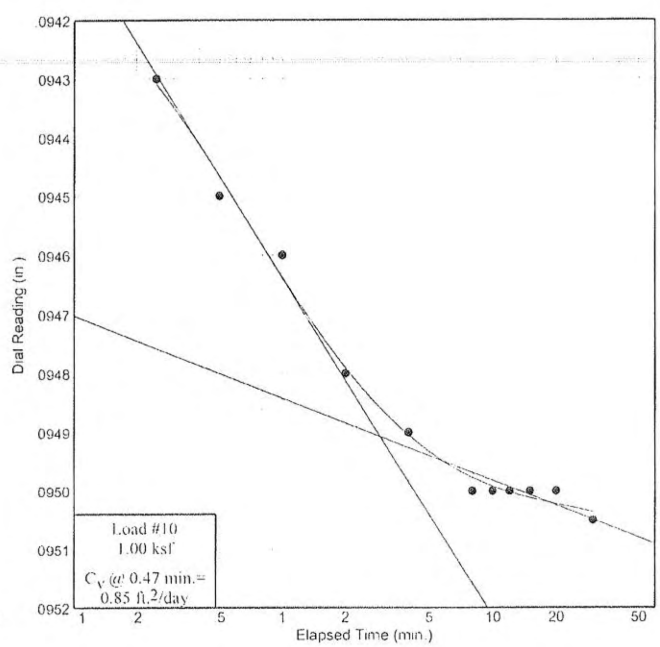
Lab No. 10516b

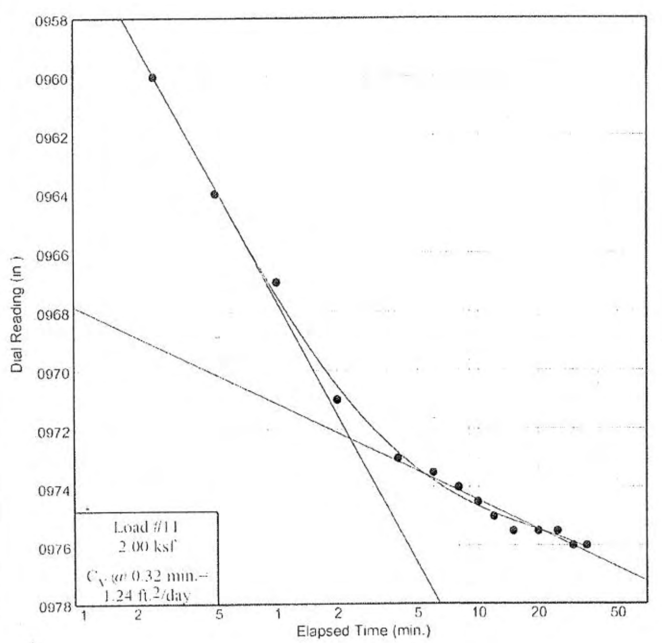
Dial Reading vs. Time

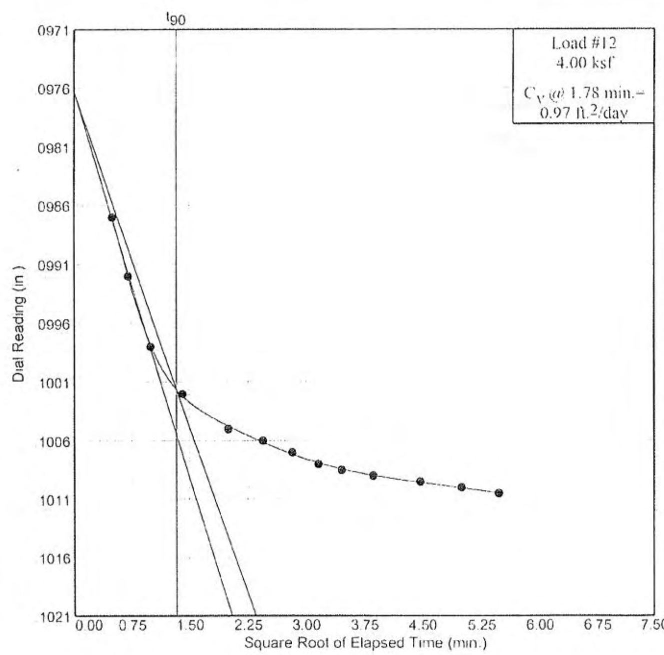
Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A | Sample No.: U-4 | Elev./Depth: 59'-61'









R.W. Gillespie & Associates, Inc.
Saco, Maine

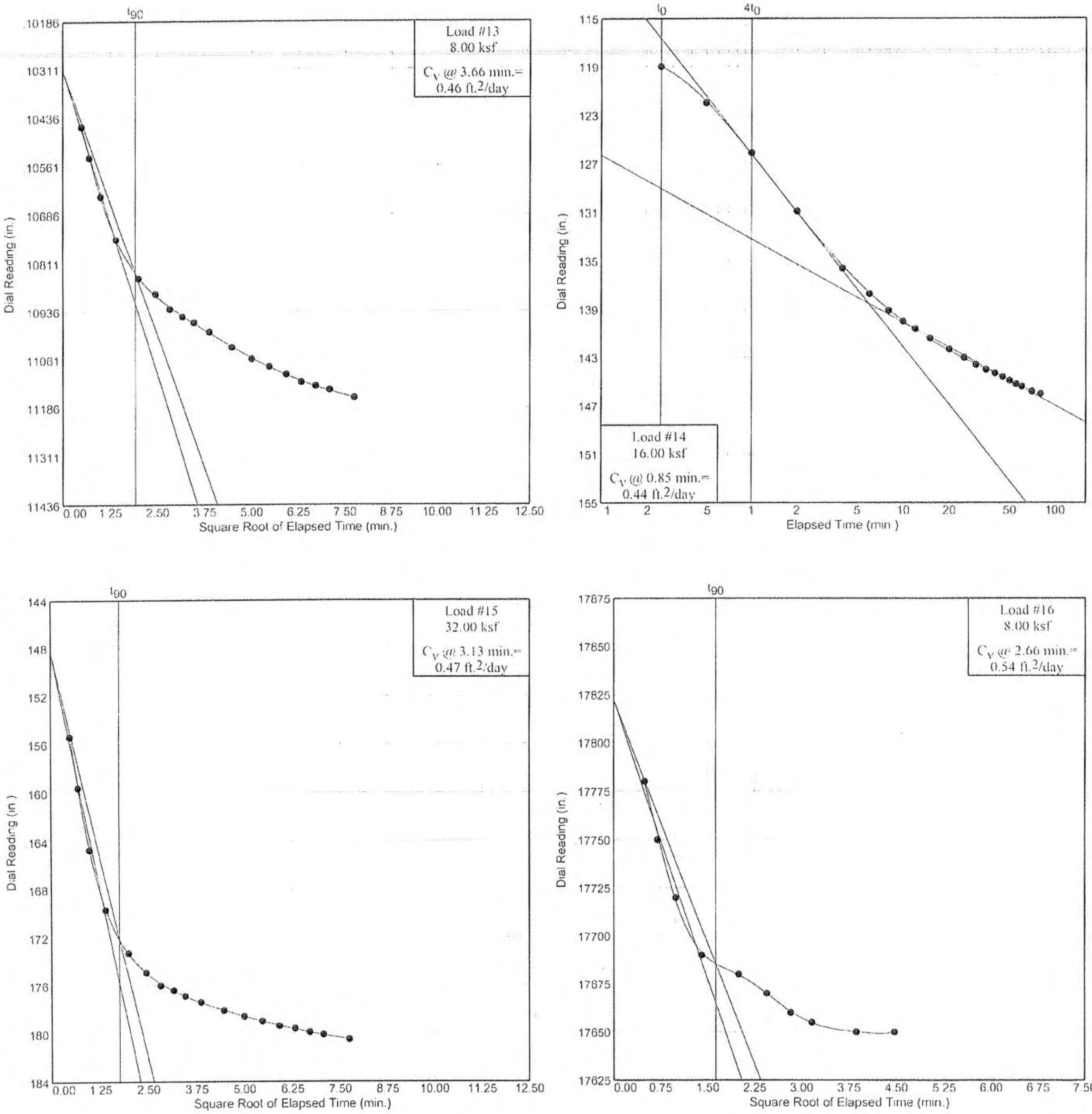
MTB

Lab No. 10516b

Dial Reading vs. Time

Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A | Sample No.: U-4 | Elev./Depth: 59'-61'



R.W. Gillespie & Associates, Inc.
Saco, Maine

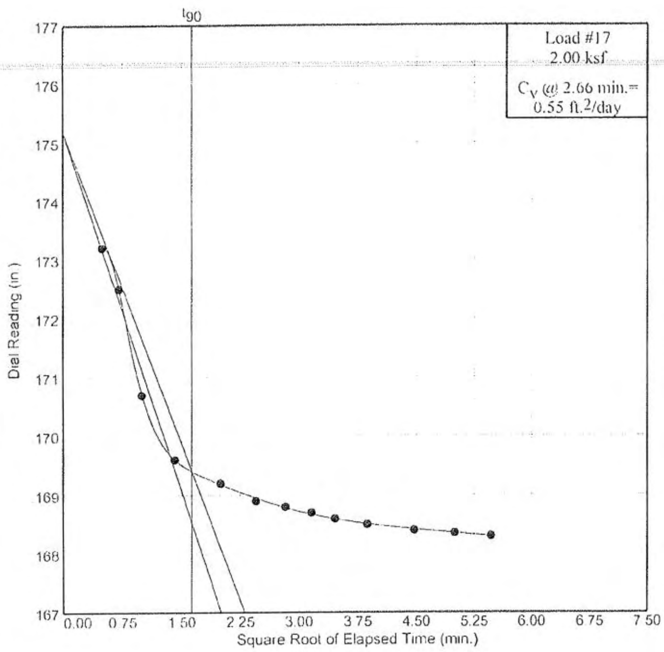
MTB

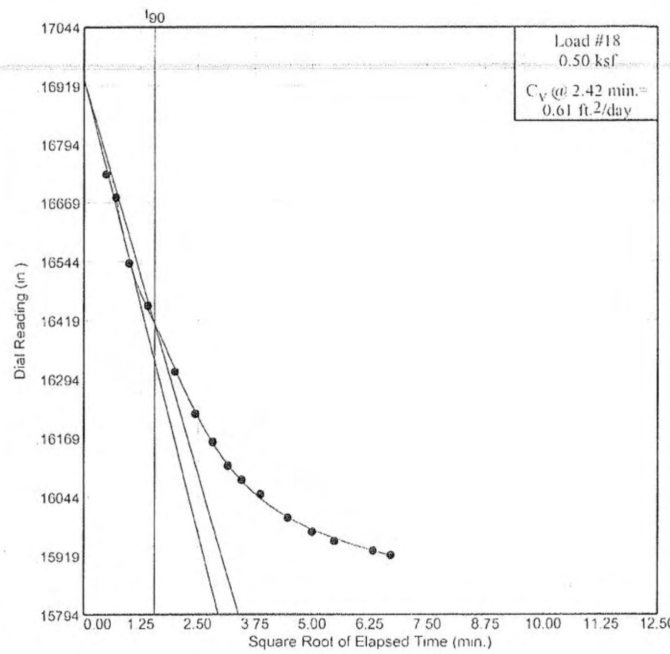
Lab No. 10516b

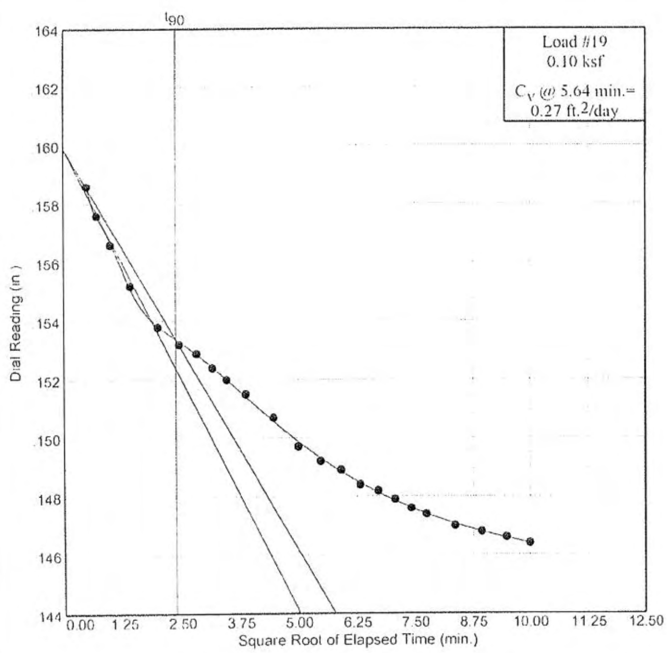
Dial Reading vs. Time

Project No.: 876-07
Project: Route 103 Bridge

Source: BB-YR34-201A   Sample No.: U-4   Elev./Depth: 59'-61'







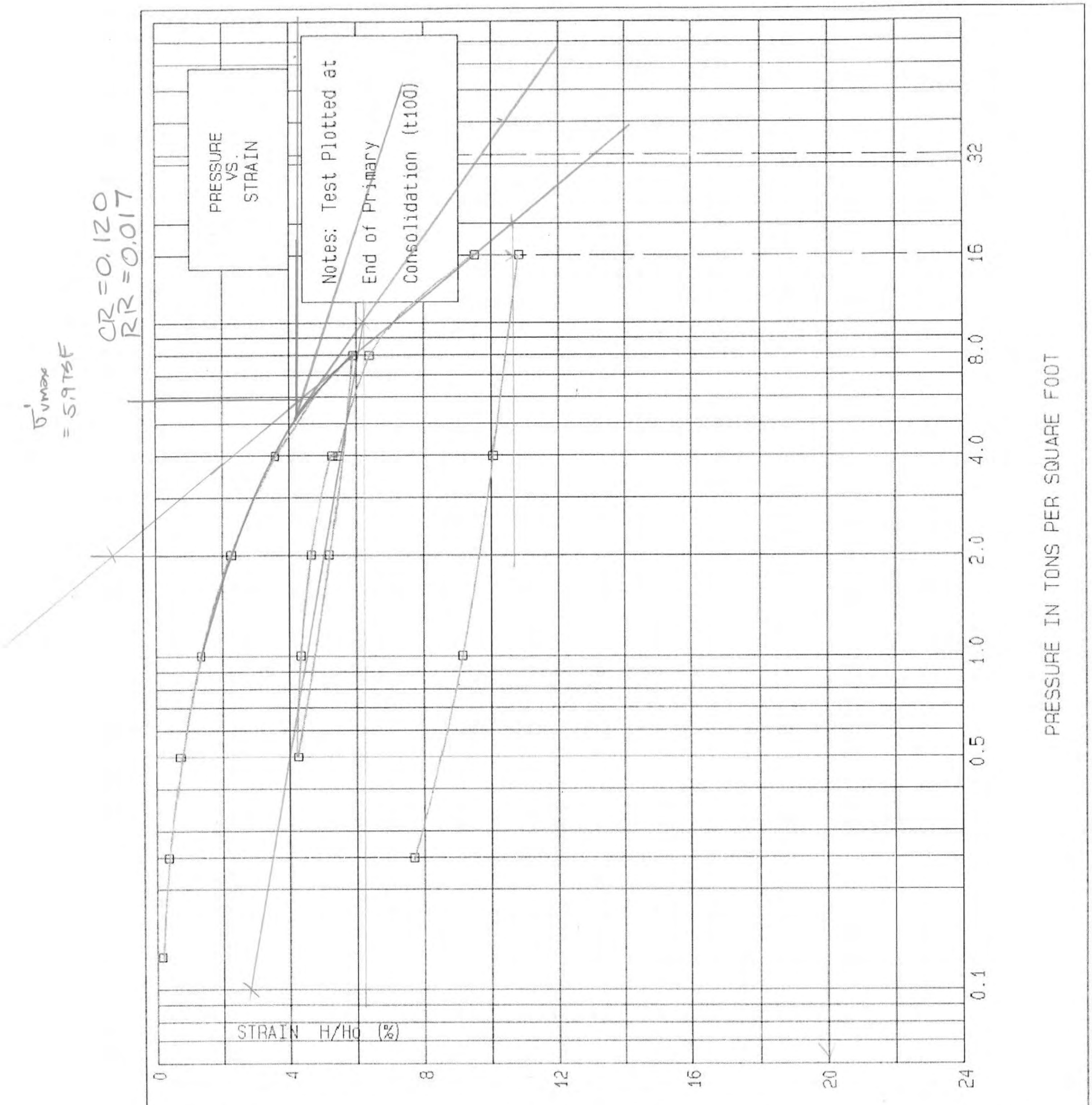
R.W. Gillespie & Associates, Inc.
Saco, Maine

MTB

Lab No. 10516b

APPENDIX D

CALCULATIONS



SOIL DESCRIPTION: Gray Lean Clay (CL)			
SAMPLE DIAM 2.5 IN.	LIQUID LIMIT 39 %	PLASTIC LIMIT 20 %	SPECIFIC GRAVITY 2.67

	WATER CONTENT, %	DRY UNIT WEIGHT, pcf	VOID RATIO	SATURATION, %	SAMPLE HEIGHT, INCH
INITIAL	30.10	92.26			0.80
FINAL	26.80	99.87			0.74

Rte 103-Station 34 Bridge

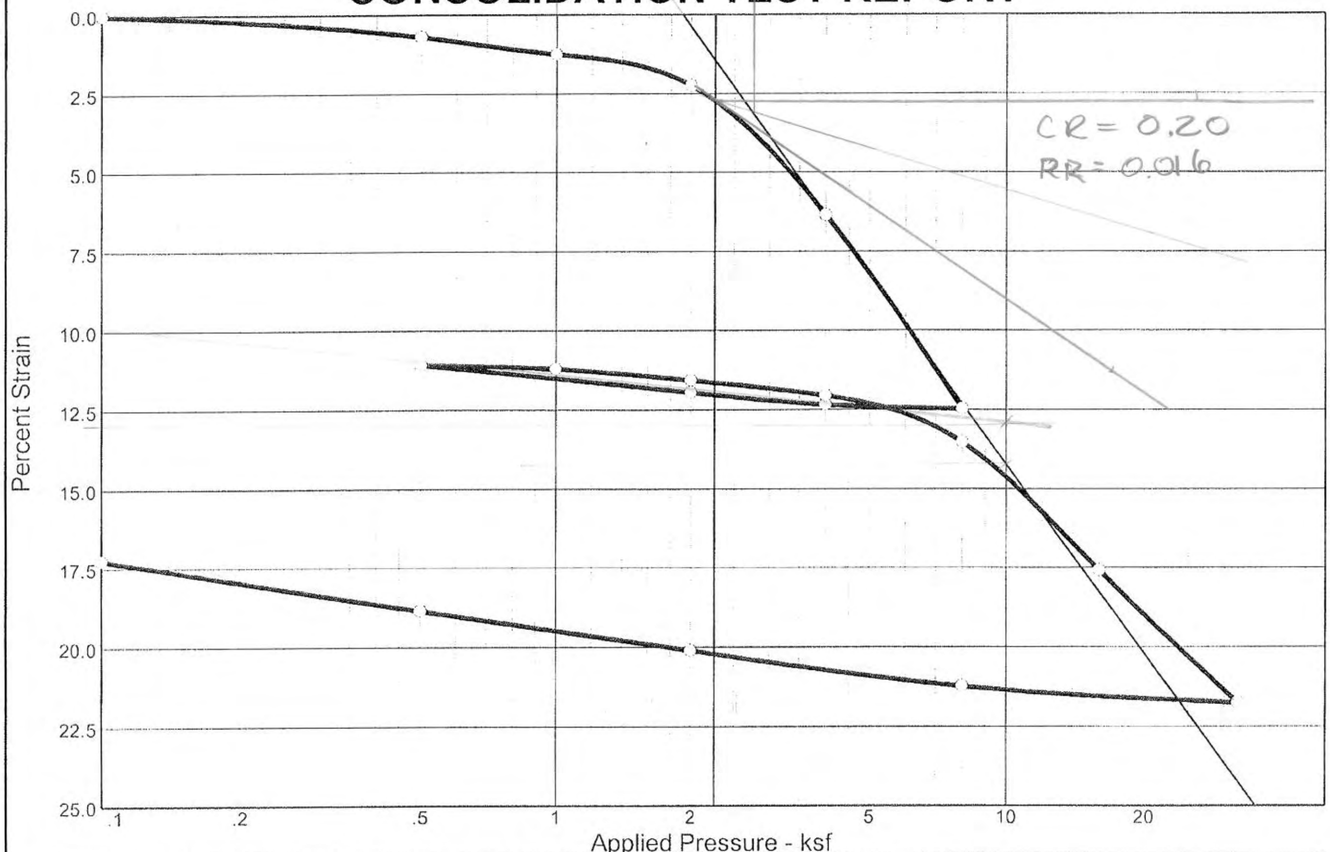
CONSOLIDATION TEST

BORING NO.	BBYR3420I	TEST SERIES NO.	C1.1
SAMPLE	U-1	DATE	10/9/08
DEPTH	29.9-30.0'		
TECH.	MBP		
REVIEWER	DAS	FILE	25577.00

FIGURE

σ'_{vmax} = 2.8 ksf

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (ksf)	C_v (ft.^{2}/day)	C_α	No.	Load (ksf)	C_v (ft.^{2}/day)	C_α	No.	Load (ksf)	C_v (ft.^{2}/day)	C_α
1	0.10	1.69		11	2.00	0.46	0.001				
2	0.50	0.38		12	4.00	0.44					
3	1.00	0.46		13	8.00	0.27					
4	2.00	0.32		14	16.00	0.16	0.010				
5	4.00	0.13		15	32.00	0.19					
6	8.00	0.11		16	8.00	0.32					
7	4.00	0.98		17	2.00	0.16					
8	2.00	0.48		18	0.50	0.06					
9	0.50	0.21		19	0.10	0.02					
10	1.00	0.32	0.000								

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
93.0 %	38.6 %	80.5	30	11	2.77							

MATERIAL DESCRIPTION	USCS	AASHTO
silty clay		

Project No. 876-07 **Client:** GZA GeoEnvironmental, Inc.

Project: Route 103 Bridge

Source: BB-YR34-201A **Sample No.:** U-1 **Elev./Depth:** 39'-41'

R.W. Gillespie & Associates, Inc.

Saco, Maine

Remarks:

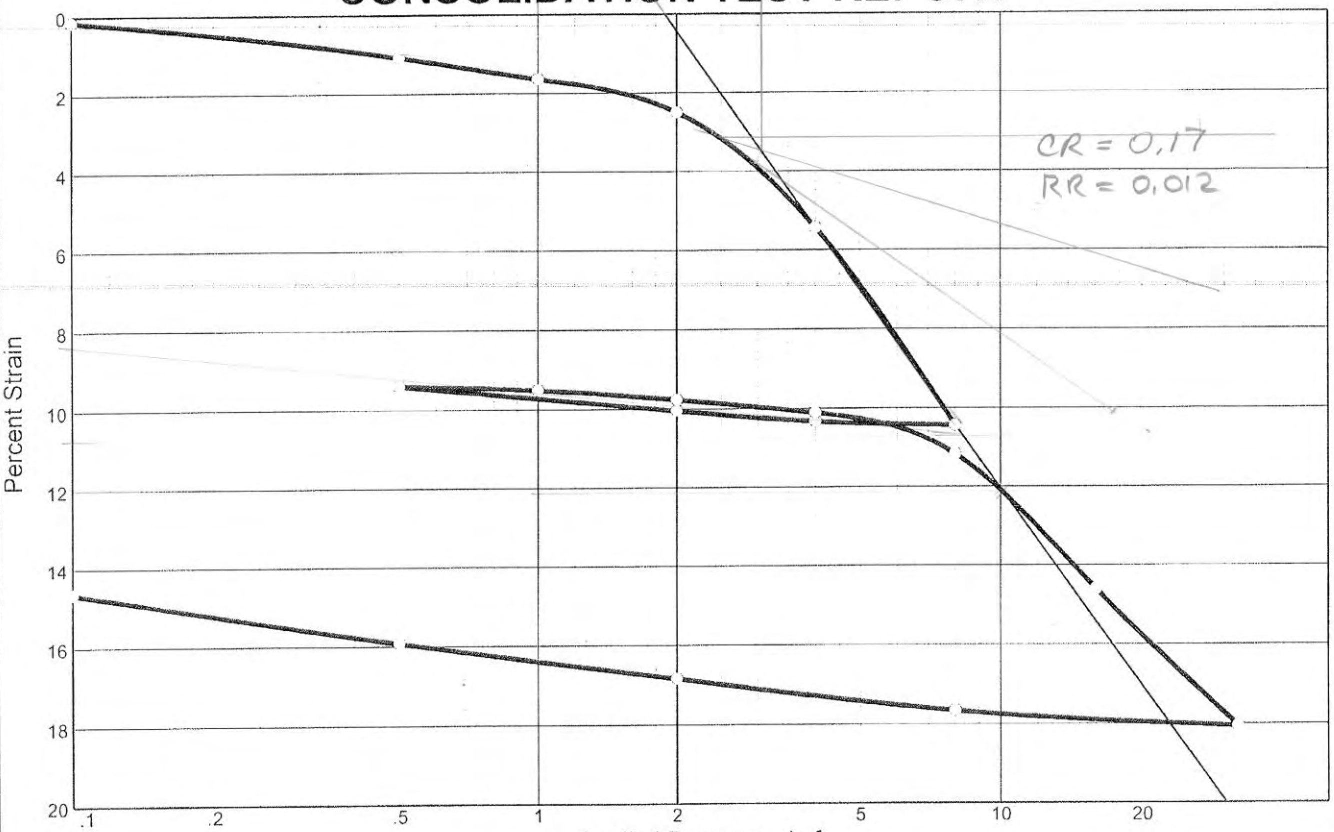
Tested by: DCH

MJG

Lab No. 10516a

σ'_{vmax} = 3.0 ksf

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (ksf)	C_v ($ft.^2/day$)	C_α	No.	Load (ksf)	C_v ($ft.^2/day$)	C_α	No.	Load (ksf)	C_v ($ft.^2/day$)	C_α
1	0.10	1.72		11	2.00	1.24	0.000				
2	0.50	0.52		12	4.00	0.97					
3	1.00	0.51		13	8.00	0.46					
4	2.00	0.82		14	16.00	0.44	0.008				
5	4.00	0.30		15	32.00	0.47					
6	8.00	0.22		16	8.00	0.54					
7	4.00	2.15		17	2.00	0.55					
8	2.00	0.58		18	0.50	0.61					
9	0.50	0.41		19	0.10	0.27					
10	1.00	0.85	0.000								

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
92.6 %	30.3 %	90.8	38	15	2.77							

MATERIAL DESCRIPTION	USCS	AASHTO
silty clay	cl	

Project No. 876-07 **Client:** GZA GeoEnvironmental, Inc.

Project: Route 103 Bridge

Source: BB-YR34-201A **Sample No.:** U-4 **Elev./Depth:** 59'-61'

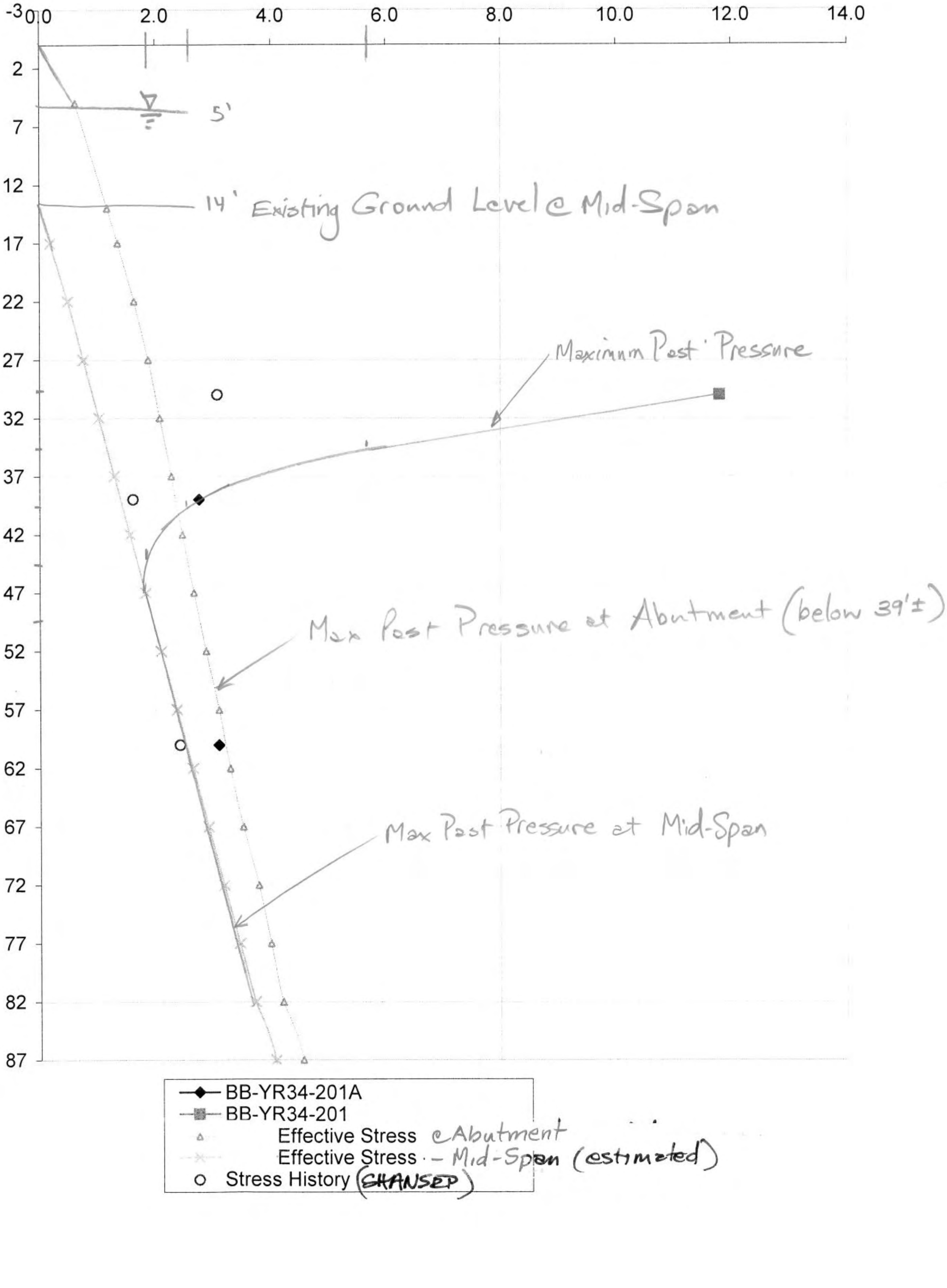
Remarks:
Tested by: DCH

R.W. Gillespie & Associates, Inc.
Saco, Maine

Lab No. 10516b

STRESS HISTORY

Preconsolidation Pressure
(ksf)



MDOT - Station 34
File: PIN15110.00
GZA File No. 25577.00
11-Nov-08
calc: E. Baron
ck: c. Snow

SETTLEMENT ESTIMATE
Location: Sta 34 mid-span
Existing Grade:
Design GWT Depth 5 ft
Footing Type: N/A
Contact Pressure: 5ftx125psf+14ftx125-64psf 1174 psf
Stress Estimate by: Boussinesq charts

Layer	Depth (ft) top/mid/bot (below bot of load)	Gamma Strata	Gamma Strata Buoyant	Thick. (ft)	Initial Stress (psf)	Surcharge Stress Increase (psf)	Fill Stress Increase (psf)	Total Stress Increase (psf)	Final Stress (psf)	Max. Prev. Stress (psf)	RR	CR	RECOMPRESSION SETTLEMENT RR H log(Po+dP/Po) (in)	VIRGIN COMPR. SETTLEMENT CR H log(Po+dP/Pmax) (in)	TOTAL CONSOL. SETTLEMENT (in)	Stress Ratio	Calpha	Second Settlement (in)
FILL	0 2.5 5	125	NA	5	0	0		0	0	0	0	0	0.00	0.00	0.00	0.00	0	0
FILL	5 9.5 14	125	61	9	0	0		0	0	0	0	0	0.00	0.00	0.00	0.00	0	0
BR SW	14 15.5 17	125	61	3	91.5	0	1174	1174	1266	0	0	0	0.00	0.00	0.00	0.00	0	0
GRY SW-SM	17 19.5 22	125	61	5	335.5	0	1150	1150	1486	11800	0.017	0.12	0.66	0.00	0.66	0.13	0.00055	0.033
GRY CL & SI	22 24.5 27	118	54	5	623	0	1127	1127	1750	11800	0.016	0.19	0.43	0.00	0.43	0.15	0.00055	0.033
GRY CL & SI	27 29.5 32	118	54	5	893	0	1056	1056	1949	11800	0.016	0.19	0.33	0.00	0.33	0.17	0.00055	0.033
GRY CL & SI	32 34.5 37	118	54	5	1163	0	986	986	2149	5700	0.016	0.19	0.26	0.00	0.26	0.38	0.00055	0.033
GRY CL & SI	37 39.5 42	118	54	5	1433	0	903	903	2336	2600	0.016	0.19	0.20	0.00	0.20	0.90	0.001	0.06
GRY CL & SI	42 49.5 57	118	54	15	1973	0	774	774	2747	1973	0.016	0.19	0.00	4.92	4.92	1.39	0.01	1.8
GRY CL & SI	57 64.5 72	118	54	15	2783	0	598	598	3381	2783	0.016	0.19	0.00	2.89	2.89	1.21	0.008	1.44
GRY CL & SI	72 79.5 87	118	54	15	3593	0	481	481	4074	3593	0.016	0.19	0.00	1.87	1.87	1.13	0.008	1.44
GRY SW-SM	87 94.5 102	125	61	15	4536.5	0	364	364	4901	4537	0	0	0.00	0.00	0.00	0.00	0	0
													1.87	9.67	11.55			4.87

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Engineers and Scientists

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SHEET NO. ______ OF ______
CALCULATED BY E Baron DATE 11-11-08
CHECKED BY JRT DATE 12-3-08
SCALE ______

Calculation of σ'_{vc} for analysis check of Stress history.

For depth = 30ft
$S_u = 2.8$ ksf ← From lab data.

$\sigma'_{vmax} = 11.8$ ksf

$\sigma'_v = 1.89 + 3/5\,(2.08 - 1.89) = 2.0$ Linear Interpolation from insitu de stress.

Use equation 7.1 from Ladd

$S_u/\sigma'_{vc} = S\,(OCR)^m$ Where $m = 0.8$ and $S = 0.22$ and $OCR = \frac{\sigma'_{vmax}}{\sigma'_v}$

Solving for σ'_{vc} gives

$\sigma'_v = S_u/(S\,(OCR)^m)$

$\sigma'_{vc} = 2.8/\left((0.22)\left(\frac{11.8}{2.0}\right)^{0.8}\right) = 3.08$ ksf @ 30ft

σ'_{vc} @ 39'
$\sigma'_{vmax} = 2.76$ KSF $S_u = 0.4$ ksf

$\sigma'_v = 2.28 + 3/5\,(2.47 - 2.08) = 2.39$ ksf

$\sigma'_{vc} = 1.62$ ksf @ 39ft

σ'_{vc} @ 60'
$\sigma'_{vmax} = 3.10$ ksf $S_u = 0.520$ ksf

$\sigma'_v = 3.11 + 3/5\,(3.29 - 3.11) = 3.22$ ksf

$\sigma'_{vc} = 2.43$ ksf @ 60'

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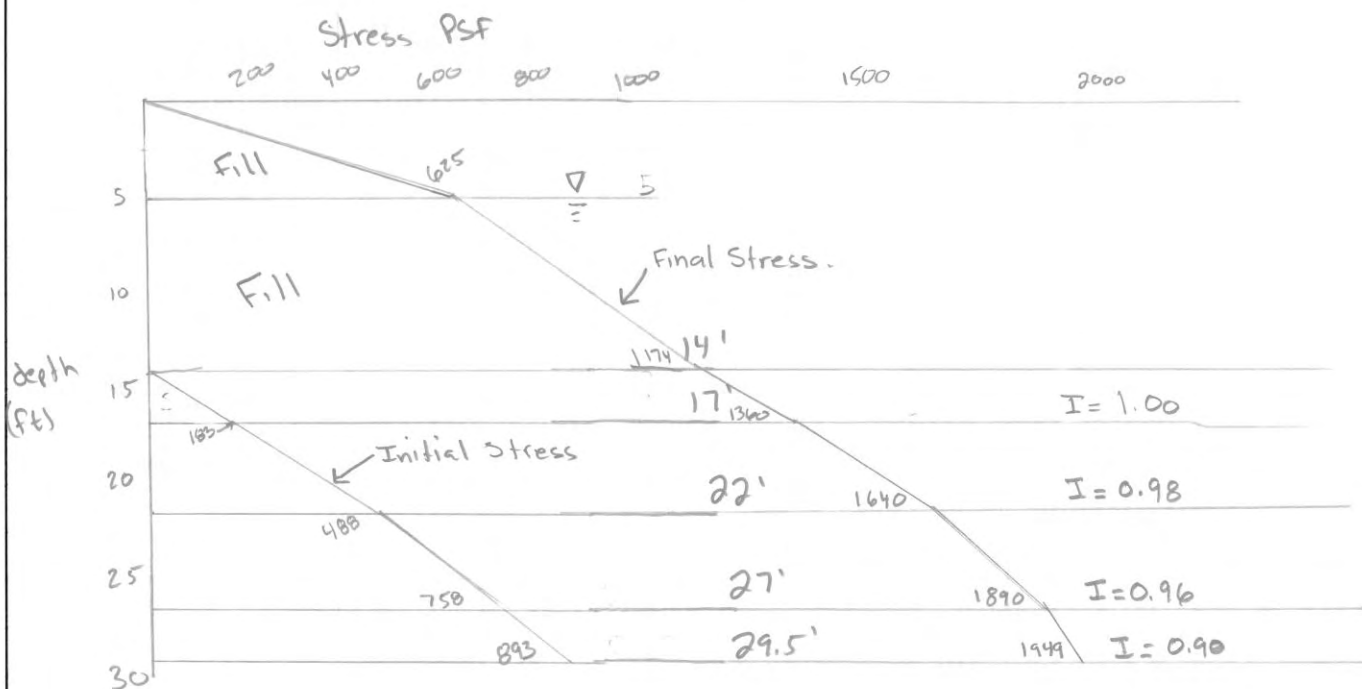
Sample Calculation of Settlement Spreadsheet

Check Settlement of the 27 to 32 foot Soil Layer.



0-14 ft Fill - γ = 125 PCF.

14-17 ft SW - γ = 125 PCF

17-22 ft SW-SM - γ = 125 PCF

22-27 ft CL & SI - γ = 118 PCF.

The Initial stress σ_1 is calculated as $\Sigma(\gamma - \gamma_{H_2O}) H_o$

where H_o = thickness of soil layer and γ_{H_2O} is 64 PCF - Salt water.

The final stress is calculated as $\sigma_1 + ((\gamma_{fill_1} \cdot 5) + (\gamma_{fill_2} - 64) \cdot 9) I$

Where I is the stress influence value at b/z by Boussinesq Where b = 50 ft.

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At 29.5' (center of Soil Layer) the stress in the soil is 1949 PSF.

From Max past Pressure Curve Fit Plot attached find the max past Pressure at 29.5'

$$\delta_p = 11800 \text{ psf} > \delta_{final} = 1949 \text{ psf}$$

There forthe soil is in Recompression.

From Lab data RR= 0.017 at shallow depths. and CR= 0.12

Calculate settlement.

$$S_c = RR\, H_o \log\left(\frac{\delta'_p}{\delta'_{vo}}\right) + \cancel{CR\, H_o\left(\log \frac{\delta_{vo} + \Delta\delta}{\delta'_p}\right)} \; 0$$

$$S_c = 0.017\,(5\,\text{ft} \cdot 12\text{''}/\text{ft}) \log\left(\frac{1949 \text{ psf}}{893}\right) = 0.35 \text{ in}$$

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CALCULATED BY E Baron DATE 11-13-08
CHECKED BY JKT DATE 12.3.08
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Time Rate of Consolidation

$T = C_v \frac{t}{H_{dr}^2}$ $\quad t = \frac{H_{dr}^2 T}{C_v}$

Layer is doubly drained

check 2 cases -

Case one → Assume that layer has No significant sandseams to carry drainage from pores.

$H_{dr} = \frac{H}{2} = \frac{65'}{2} = 32.5'$

from Lab data @ Load = 4.0 ksf.

$C_v = 0.13\ ft^2/day$ $\quad t = \frac{(32.5 ft)^2 (0.848)}{0.13\ ft^2/day} = 6890\ days \approx 19\ Years.$

$T_{90} = 0.848$

Case 2 → Sand seams are significant enough to allow drainage at Shorter distances.

Based on borings, sand seams are present at > 10 foot intervals.
assume $H_{dr} = \frac{10 ft}{2} = 5 ft$

$t = \frac{5 ft^2 (0.848)}{0.13\ ft^2/day}$ $\quad t = 163\ days =$

Maine Department of Transportation
Bridge Program
Geotechnical Section
Augusta, Maine

Geotechnical Design Report

For the Replacement of:

Station 44 Bridge
Over Tidal Estuary
York, Maine

Prepared by:

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Geotechnical Engineer

State of Maine
Kathleen
Maguire
7120
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York County

Soils Report No. 2004-23

PIN 11067.00
Fed No. AC-BR-1106(700)X
July 2004

Table of Contents

GEOTECHNICAL DESIGN SUMMARY

The purpose of this Design Report is to make geotechnical recommendations for the Station 44 Bridge over the Tidal Estuary in York, Maine in order to facilitate the replacement of the existing bridge. The proposed bridge will consist of pre-cast, prestressed, concrete butted box beams with a bituminous wearing surface founded on driven, integral H-piles. During the site subsurface investigation, a significant, compressible, silty clay layer was encountered.

The following design recommendations are discussed in detail in the attached Report:

Integral Abutment H-Piles - Piles for use at abutments may be HP 12 x 53, HP 14 x 73 or HP 14 x 89. Grade 50 ksi steel is assumed. The first pile driven at the site should be dynamically tested for capacity. The piles should be driven to refusal on or within the bedrock. The piles should be fitted with pile driving points to protect the tips and to improve the penetration of the piles. Design axial loads should be shown on the plans.

Pile Capacities - Using 50 ksi steel, the geotechnical capacity of the piles is less than the structural capacity and therefore governs.

Pile Type	Allowable end bearing capacity, $Q_{t, allow}$ (Goodman's Bedrock Condition) FS = 2.25	Total Allowable $Q_{total, allow}$ Per Structural Capacity: 50 ksi FS = 4
HP 12 x 53	**152 kips**	194 kips
HP 14 x 73	**210 kips**	268 kips
HP 14 x 89	**257 kips**	326 kips

Frost Protection - All foundations placed on native subgrade soils or fill should be founded a minimum of 4.0 ft below finished exterior grade for frost protection.

Settlement - Any settlement of the bridge abutments will be due to the elastic compression of the piling. The roadway will be widened and settlements resulting from the placement of fills for the widened roadway may be on the order of 1 to 2 inches. This settlement will occur over a long period of time (years) and may require minor attention by a maintenance crew.

1.0 INTRODUCTION

A subsurface investigation and geotechnical design for the replacement of the Station 44 Bridge over the Tidal Estuary in York, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This Report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing bridge was constructed in 1957 and consists of a timber pile supported, two-span, steel girder superstructure with a concrete deck. The existing structure has a span of length 46 ft. Physical inspection indicates that the bridge is approaching structural deficiency and should be replaced. The Federal Highway Administration (FHWA) states that treated marine piles last about 50 years in northern climates. Hydraulic analysis of the existing structure indicates that the current opening area is adequate to carry calculated flow.

It is understood that the existing bridge will be completely removed and replaced. The structure will be replaced with a single span (60 ft), pre-cast, prestressed, concrete butted box beam structure founded on integral abutments on a single row of driven H-piles. The horizontal alignment of the existing bridge will be maintained in the replacement. The vertical alignment will be modified slightly to raise the center of the bridge by approximately 10 inches to improve drainage.

2.0 GEOLOGIC SETTING

The Station 44 Bridge on US Route 103 in York, Maine crosses the Tidal Estuary approximately 0.47 miles south of Harris Island Road as shown on Sheet 1 - Location Map presented at the end of this Report. The Tidal Estuary flows into York Harbor.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits. Soils in the site area are generally comprised of silt, clay, sand and minor amounts of gravel. The most common component is the silty clay known as the Presumpscot Formation. These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

According to the Bedrock Geologic Map of Maine (1985) the bedrock in the vicinity of the site consists of fine-grained, calcareous, feldspathic, sandstone of the Kittery Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling two (2) test borings (BB-YR44-101 and BB-YR44-102) behind the location of the existing abutments as show in Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this Report. The borings were drilled between March 1 and 2, 2004 using the MaineDOT drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and graphically on Sheet 3 - Boring Logs found at the end of this Report.

The borings were drilled using cased wash boring techniques. Soil samples were obtained at 5-ft intervals using Standard Penetration Test (STP) methods. In-situ vane shear tests were made at regular intervals in the soft soil deposits to measure the shear strength of the strata. The bedrock was cored in both borings using an NQ core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field and laboratory testing requirements and logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion if the drilling program.

4.0 Laboratory Testing

Laboratory testing for samples obtained in the borings consisted of eight (8) grain size analyses with hydrometer and three (3) Atterberg Limits tests. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this Report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this Report.

5.0 Subsurface Conditions

The general soil stratigraphy encountered at the site is as follows:

- **fill** underlain by
- **silt** underlain by
- **silty clay** underlain by
- **interbedded sand and silt layers** all of which is underlain by
- **bedrock**.

An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this Report. Results of the moisture content, grain size analyses and Atterberg Limits testing can be found in Appendix B - Laboratory Data. This information is also shown on the boring logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report. The following paragraphs discuss the soils encountered in detail:

Fill. Beneath the pavement, a layer of fill soils were encountered in both of the borings. This layer was found to be damp to wet, brown, fine to coarse sand with little gravel and silt. Five SPT N-values in the fill layer ranged from 7 to 43 blows per foot (bpf) indicating that the soil is loose to dense in consistency. The thickness of the fill layer ranged from approximately 15 ft in boring BB-YR44-101 to approximately 14 ft in boring BB-YR44-102. No laboratory testing was conducted on the soil samples collected from this layer.

Silt. Underlying the fill, a layer of silt was encountered. This layer was found to be grey, wet, soft, silt, with little fine sand and seashells. Two SPT N-values in the silt layer were both 4 bpf indicating that the soil is soft in consistency. The thickness of the silt layer was approximately 5 ft in both of the borings. No laboratory testing was conducted on the soil samples collected from this layer.

Silty Clay. Underlying the silt, a layer of silty clay was encountered. This layer was determined to consist of grey, wet, silty clay with trace fine sand in layers, trace gravel, and occasional black staining. The overall thickness of the silty clay layer ranged from approximately 15.5 ft in boring BB-YR44-101 to approximately 12 ft in boring BB-YR44-102.

Vane shear testing conducted within the silty clay layer showed measured undrained shear strengths of the layer to range from about 402 to 527 psf while the remolded shear strength ranged from about 49 to 125 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the silty clay has a sensitivity between 4 and 8 and is classified as moderately sensitive to sensitive. Water contents from five samples obtained within this layer range from approximately 31% to 47%. Grain size analyses conducted on samples from the silty clay layer indicate that the soil is classified as an A-6 by the AASHTO Classification System and a CL by the Unified Soil Classification System.

Two Atterberg Limits tests were made from samples throughout this layer. The following table summarizes these test results:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-YR44-101 7D	47.2	33	19	14	2.014
BB-YR44-102 4D	31.3	34	20	14	0.807

Interpretation of these results indicates that the upper portion of the layer is overconsolidated while the lower potion of the layer is on the verge of being a viscous liquid. The lower potion of the layer has a water content which exceeds the liquid limit indicating that the soil has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that have a high liquefaction potentially commonly referred to as "quick".

Interbedded Silt and Sand. Underlying the silty clay, a layer of interbedded silt and sand was encountered. This layer was found to be grey, wet, layers of fine silty sand with trace medium to coarse sand interbedded with silt layers. SPT N-values obtained in the sand layers ranged from 13 to 22 bpf indicating that the sand is medium dense in consistency. SPT N-values obtained in the silt layers ranged from 3 to 11 bpf indicating that the silt is soft to stiff in consistency. The thickness of the individual silt and sand layers ranged from approximately 4 ft to approximately 10.5 ft. The overall thickness of the layer ranged from 19 ft in boring BB-YR44-101 to approximately 5.9 ft in boring BB-YR44-102. Water contents from three samples obtained within this layer range from approximately 10% to 28%.

One Atterberg Limits test was made from a sample from this layer. The following table summarizes these test results:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-YR44-1026D	28.3	19	15	4	3.325

Interpretation of these results indicates that the layer is on the verge of being a viscous liquid. The layer has a water content which exceeds the liquid limit indicating that the soil has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that have a high liquefaction potentially commonly referred to as "quick".

Three grain size analyses conducted on samples from this layer indicate that the silt layers are classified as an A-4 by the AASHTO Classification System and a CL-ML by the Unified Soil Classification System while the sand layers are classified as an A-2-4 by the AASHTO Classification System and a SC-SM by the Unified Soil Classification System.

Bedrock. The bedrock surface was encountered and cored at a depth of 54.5 ft bgs in boring BB-YR44-101 and 36.9 ft bgs in boring BB-YR44-102. Different bedrock types were encountered in both of the borings providing a timeline picture of the geologic events that formed the area. In Boring BB-YR44-102 a grey/brown fractured brecciated and metamorphosed phyllite with extensive limonite alteration along most fractured surfaces was cored. This rock can be identified as the "country rock" of the area. Using classical dating techniques, this rock appears to be to oldest in the area. Sometime later this "country rock" was covered by the intrusive rock cored in Boring BB-YR44-101. This rock is identified as dark grey/green andesite porphyry. At other locations in this area a second intrusive rock (diorite) is also seen with in the bedrock column. The diorite was not encountered at this location. The RQD of the bedrock was determined to range from 46 to 68% indicating a rock of poor to fair quality.

Groundwater. Groundwater was observed at a depth of approximately 10 to 10.5 ft below ground surface in the borings. The water level readings were taken during drilling activities. Groundwater levels at the site are tidally influenced.

6.0 FOUNDATION ALTERNATIVES

Due to cost and the saltwater environment at the site, the use of integral abutments supported on driven H-piles was the only alternative evaluated in the Preliminary Design Report. The subsurface investigation indicates the presence of a significant silty clay layer underlying the bridge site. Therefore, the use of a driven H-pile supported foundation integral abutment structure is a viable foundation alternative.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following subsections will discuss the foundation considerations and recommendations for an integral abutment structure supported on driven H-piles.

7.1 Driven H-Pile Foundations

The use of a stub abutment founded on a single row of driven H-piles has been determined to be the optimal foundation. The following piles were considered for use at the site: HP 12 x 53, HP 14 x 73 or HP 14 x 89. Grade 50 ksi steel piles should be specified.

Pile length at the abutments can be estimated based on the following data:

Location	Ground Elevation	Depth to Rock bgs	Top of Rock Elevation	Rock Quality Designation
BB-YR44-101 Abutment #1	10.4 ft	54.5 ft	-44.10 ft	68%
BB-YR44-102 Abutment # 2	10.4 ft	36.9 ft	-26.5 ft	46%

For integral abutment piles the MaineDOT Bridge Design Guide (BDG) recommends a Factor of Safety of 4.0 or 0.25Fy for the maximum structural design load. The geotechnical and structural capacities of the H-piles are summarized in the following table. Calculations can be found in Appendix C at the end of this Report. Using the assumption that 50 ksi steel will be used; the allowable geotechnical capacity of the piles is less than the allowable structural capacity and therefore governs. Design axial loads should be shown on the plans. No downdrag should be considered.

Pile Type	Allowable end bearing capacity, $Q_{t,\,allow}$ (Goodman's Bedrock Condition) FS = 2.25	Total Allowable $Q_{total,\,allow}$ Per Structural Capacity: 50 ksi FS = 4
HP 12 x 53	**152 kips**	194 kips
HP 14 x 73	**210 kips**	268 kips
HP 14 x 89	**257 kips**	326 kips

The first pile driven at the site should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor. With this level of quality control, the piles shall be driven to an ultimate capacity of 2.25 times the design load.

The piles should be designed as end bearing and should be driven to refusal on or within the bedrock. Piles may penetrate up to 6 inches into fractured and weathered bedrock. The piles should be fitted with pile driving points to protect the tips and to improve penetration.

The soils encountered at the site will provide sufficient lateral support to assume the H-piles are fully braced against Euler buckling. The Designer should check that pile axial stresses from the dead loads, live loads, pile dead load and secondary thermal forces do not exceed the allowable axial pile loads shown in the table. The Designer should also check the live load rotation demand in accordance with BDG Design Procedure 5-4.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Engineer. Contract documents should require that the contractor perform a wave equation analysis of the proposed pile driving system, and the piles be driven to 2.25 times the design (working) load. This factor of safety assumes field dynamic testing will be performed. A hammer should be selected which provides the required geotechnical capacity when the penetration resistance for the final 3 to 6 inches is 8 to 13 blows per 1 inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows. Allowable pile stresses during driving shall be less than 0.90Fy, per AASHTO 4.5.11.

7.2 Stub Abutments and Wingwalls

Cast-in-place integral abutment sections shall be designed structurally for passive earth pressure. A passive earth pressure coefficient (K_p) of 7.3, calculated using Coulomb Theory, is recommended. If an approach slab is not specified, additional lateral earth pressure due to traffic surcharge is required and shall be approximated by an additional 2 ft of earth fill. This results in a traffic surcharge of 250 pcf. Use of an approach slab may be required per the BDG Sections 5.4.2.10 and 5.4.4.

Wingwalls, if oriented in-line with the abutment face, shall be designed for passive earth pressure. A passive earth pressure coefficient of $K_p = 3.3$ calculated using Rankine Theory can be used.

The Designer may assume Soil Type 4 (BDM Section 700) for retaining wall back fill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf, and a soil-concrete friction coefficient of 0.45.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 of the BDG.

7.3 Frost Protection

According to the MaineDOT design freezing index maps for the State of Maine, the site has a design-freezing index of approximately 1100 F-degree days. Grain size analyses conducted on soil samples in the upper layer of soils indicated that the soils are granular and have a water content of approximately 30%. These components correlate to a frost depth of approximately 4 ft. Therefore, any foundations placed on native subgrade soil should be founded a minimum of 4 ft below finished exterior grade for frost protection. See Appendix C - Calculations for supporting documentation. The final depth of embedment may be controlled by the scour susceptibility of the soil and may, in fact, be deeper than the depth required for frost protection.

7.4 Bearing Capacity

In the event that any foundation will be founded on the native soils the allowable bearing capacity of the layer should not exceed 4 ksf. See Appendix C- Calculations for supporting documentation. No footing shall be less than 2 ft wide regardless of the applied bearing pressure.

7.5 Settlement

It is understood that the horizontal alignment of the existing bridge will be maintained in the replacement of the Station 44 Bridge. The vertical alignment will be modified slightly to raise the center of the bridge by approximately 10 inches to improve drainage.

Any settlement of the bridge abutments will be due to the elastic compression of the piling. The roadway will be widened slightly and the widened approaches will be constructed using embankments sloped at 1V:3H. Settlements resulting from the placement of fills for the widened roadway may be on the order of 1 to 2 inches. This settlement will occur over a long period of time (years) and may require minor attention by a maintenance crew.

7.6 Backfill Material

Structure and head wall backfill within 10 ft of the structure and side-slope fill materials shall conform to MaineDOT Specification 703.19 - Granular Borrow for Underwater Backfill. This gradation specifies that 10 percent or less of the material may pass the No. 200 sieve. This material is also specified in order to reduce the amount of fines and to minimize frost action behind the structure. The structure design shall include a drainage system to intercept any groundwater. Drainage behind structure shall be in accordance with Section 5.4.1.4 of the BDG.

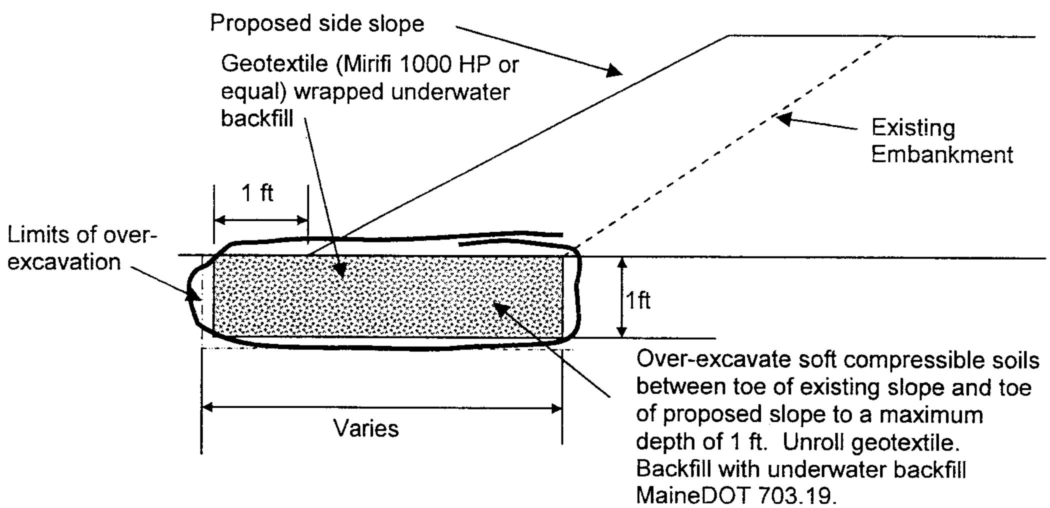
7.7 Embankment Widening Construction

At elevations below high tide plus wave run-up level, the embankment widening construction techniques will require special emphasis on preventing wave and current scour and the potential loss of soils due to piping. For portions of the work that will take place in the inter-tidal zone, the Contractor will need to work only as large a segment as can be completed in a single tide cycle, including final protective measures to prevent scour and piping. Two alternatives are considered feasible for embankment widening and reconstruction under these conditions:

- graded stone fill slopes and
- filter-protected granular fill slopes with riprap

The advantage of graded stone fill slopes is that the fills could be submerged repeatedly during construction and no cofferdams would be needed for the widening. The advantage of the filter-protected granular fill slopes with riprap is that they are less costly, however, it is difficult to protect the granular material during each tide cycle and it is difficult to compact lifts above saturated material.

Based on the borings located behind the abutments, it is understood that the existing approaches are built over soft compressible clayey silts. Due to the presence of these soft soils, it is recommended that the area of the embankment widening between the existing toe of slope and the proposed toe of slope be over-excavated to a depth of 1 ft and the material replaced with a geotextile wrapped granular mat. The geotextile should be rolled directly on the exposed subgrade along the existing embankment. The trench should be backfilled with compacted MaineDOT 703.19 Granular Borrow Material for Underwater Backfill. The geotextile should be wrapped up around the aggregate zone. The total area of the geotextile-wrapped trench should be sufficient to extend 1 ft beyond the toe of the widened embankment. The following figure illustrates the construction:



7.8 Seismic Design Considerations

Per BDG 3.7.1.1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09 are designated a Seismic Performance Category (SPC) classification of A, and should be designed in conformance with Section 5 of Division 1-A of the <u>AASHTO Standard Specifications for Highway Bridges.</u> For SPC A, no detailed analysis is required other than connection design and bearing seat length, except if the bridge is functionally important or classified as a major structure.

The horizontal bedrock acceleration coefficient (A) for York is less than 0.05g, based on Figure 3-4 of the BDG, Seismic Performance Categories for Maine, August 2003. Per

Section 3.5 of Division 1-A of the AASHTO Standard Specifications for Highway Bridges Soil Profile Type III is applicable to the site and a site coefficient (S) of 1.5 would be used.

According to Figure 2-2 of the BDG, the Station 44 Bridge is not on the National Highway System (NHS) and is therefore not considered to be functionally important. Since the bridge construction costs do not exceed $10 million the bridge is not classified as a major structure. As a result, the bridge substructures will not be designed for seismic earth loads. The soils at the site are considered to be liquefaction-susceptible.

8.0 CLOSURE

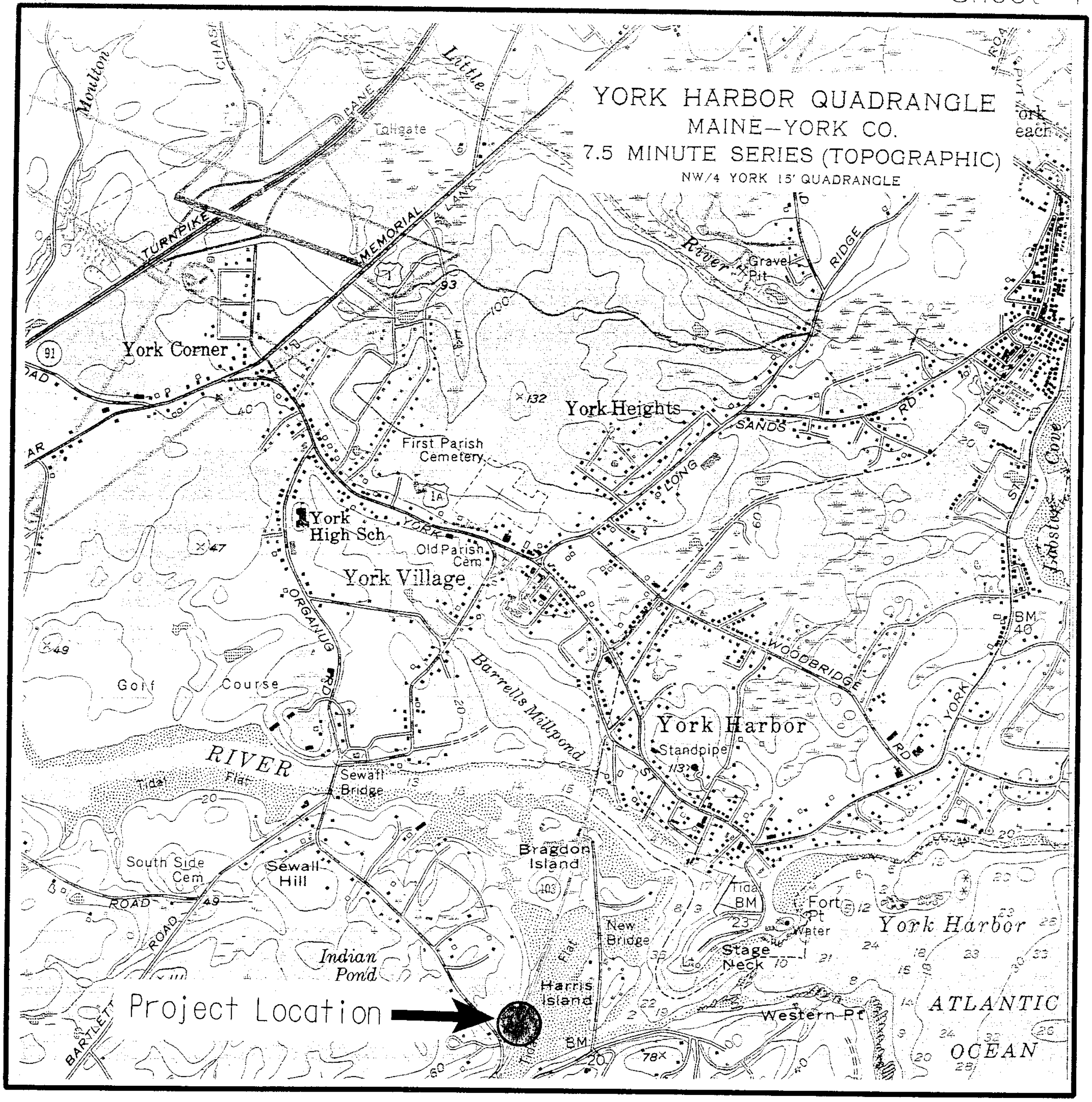
This Report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Station 44 Bridge in York, Maine in accordance with generally accepted soil and foundation engineering practices. No other intended use is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this Report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this Report.

We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Location Map

Sheet 1



York, Maine, Station 44 Bridge over Tidal Estuary, PIN. 11067.00

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

1:24000, 1"= 2000', 1 cm = 240 m

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 44 Bridge over Tidal Estuary		Location: York, Maine		Boring No.:	BB-YR44-101
				PIN:	11067.00
Driller:	MaineDOT	Elevation (ft.)	10.4	Auger ID/OD:	4.5" SSA
Operator:	G. Lidstone	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/1/04-3/1/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	5+23.9, 9.1 Rt.	Casing ID/OD:	HW	Water Level*:	10.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98	Pavement ——— 0.42	
	1D-A	24/20	2.00 - 4.00	16/31/12/9	43			(1D-A) (2.0-3.0) Brown, damp, dense, fine to coarse SAND, little silt, trace gravel (Fill).	
	1D-B							(1D/B) (3.0-4.0) Brown, damp, dense, fine to coarse SAND, some silt, little gravel (Fill).	
5	2D	24/16	5.00 - 7.00	3/3/8/15	11			Brown, damp, medium dense, fine to coarse SAND, some silt, little gravel, old pavement, (Fill).	
10	3D	24/17	10.00 - 12.00	5/5/3/7	8	25			
						36	-0.60	——— 11.00 Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	
						33			
						30			
						28			
15	4D	24/16	15.00 - 17.00	4/2/2/4	4	15	-4.60	——— 15.00 Grey, wet, soft, fine sandy SILT with sea shells.	
						15			
						28			
						29			
						19			
20	5D	24/20	20.00 - 22.00	1/1/WOH/1	1	35	-9.60	——— 20.00 Grey, wet, very soft, silty CLAY, trace fine sand and fine sand layers, trace gravel. Stiffer at top of sample, softer with depth.	G#176633 A-6, CL WC=37.8%
						35			
	MV		22.40 - 22.40			28		55x110 mm vane raw torque readings: MV= could not push.	
						22			
						19			
25	6D	24/16	25.00 - 27.00	7/1/WOH/1	1	40		Grey, wet, very soft, silty CLAY, varved with sand lenses, trace gravel, trace coarse sand.	G#176634 A-6, CL WC=38.3%
						27			
	MV		27.00 - 27.00			23		55x110 mm vane raw torque readings: MV= could not push.	
						21			
						19			
30	7D	24/10	30.00 - 32.00	Push thru vane		42		Grey, wet, soft, silty CLAY, trace fine sand.	G#176635 A-6, CL WC=47.2% LL=33 PL=19 PI=14
	V1		31.00 - 31.36	Su=402/49 psf		33		55x110 mm vane raw torque readings: V1= 9.0/1.1 ft-lbs MV = could not push	
	MV		32.00 - 32.00			29			
						31			
						36			
35	8D	24/3	35.00 - 37.00	1/5/8/6	13	55	-25.10	——— 35.50 Grey, wet, medium dense, fine silty SAND, trace coarse sand. 55x110 mm vane raw torque readings: MV= could not push.	
	MV		36.00 - 36.00			38			
						27			
						23			
						38			
40	9D	24/6	40.00 - 42.00	3/WOH/3/7	3	48	-29.60	——— 40.00 Grey, wet, soft, SILT, little gravel, sand and clay, dilatent.	G#176636 A-4, CL-ML WC=24.2%
						36			
						28			
						48			
						124	-33.60	——— 44.00	
45	10D	24/6	45.00 - 47.00	22/10/12/11	22	93		Grey, wet, medium dense, fine to coarse SAND, some gravel and silt, trace clay.	G#176637 A-2-4, SC-SM WC=10.2%
						79			
						116			
						132			
						260			
50	MD	2/0	50.00 - 50.17	60(0.2")	—	a WR		Wash material in spoon a Washed ahead and Roller coned ahead to 54.5' bgs through very dense grey till.	
	R1	60/52	54.50 - 59.50	RQD = 68%		NQ	-44.10	——— 54.50 Bedrock: Dark grey/green, fine grained, ANDESITE porphyry.	
55						Core		R1:Core Times (min:sec) 54.5' - 55.5' (18:20) 55.5' - 56.5' (10:00) 56.5' - 57.5' (10:10) 57.5' - 58.5' (9:10) 58.5' - 59.5' (9:05) Recovery=97%	
							-49.10	——— 59.50 Bottom of Exploration at 59.50 feet below ground surface.	
60									
65									
70									
75									

Remarks:

MV = Unsuccessful vane shear test attempt.

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

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

Page 1 of 1

Boring No.: BB-YR44-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 44 Bridge over Tidal Estuary		Location: York, Maine		Boring No.:	BB-YR44-102
				PIN:	11067.00
Driller:	MaineDOT	Elevation (ft.)	10.4	Auger ID/OD:	4.5" SSA
Operator:	G. Lidstone	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/1/04-3/1/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	5+50.8, 10.3 Lt.	Casing ID/OD:	HW	Water Level*:	10.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98	Pavement ——— 0.42 Brown, damp, dense, fine to coarse SAND, little gravel, trace silt. (Fill).	
5	1D	24/10	5.00 - 7.00	4/4/5/6	9			Brown, damp, loose, fine to coarse SAND, some gravel, little silt. (Fill).	
10	2D	24/2	10.00 - 12.00	6/3/4/11	7	46		Brown, wet, loose, fine silty SAND, little gravel, trace medium to coarse sand. (Fill).	
						27			
						208		Obstruction at 13.0' bgs. Roller coned through.	
						191			
						41	-3.60	——— 14.00	
15	3D	24/16	15.00 - 17.00	2/2/2/1	4	34		Grey, wet, soft, sandy SILT with broken shells.	
						30			
						27			
						73			
						43	-8.60	——— 19.00	
20	4D/MV	24/18	20.00 - 22.00	5/3/4/7	7	45		Grey, wet, medium stiff, Silty CLAY, trace gravel and sand, mottled. 55x110 mm and 25.4x50.8 mm vane raw torque readings: MV = could not push	G#176638 A-6, CL WC=31.3% LL=34 PL=20 PI=14
						43			
						41			
						34			
						34			
25	5D	24/24	25.00 - 27.00	Push thru vane	—	46		Grey, wet, soft, silty CLAY with black staining, occasional black fine to coarse silty sand layers, trace gravel. 55x110 mm vane raw torque readings: V1 = 10.0/2.4 ft-lbs V2 = 11.8/2.6 ft-lbs	G#176639 A-6, CL WC=42.6%
	V1		26.00 - 26.36	Su=446/107 psf		29			
	V2		27.00 - 27.36	Su=527/116 psf		32			
						30			
						33			
30	6D	24/18	30.00 - 32.00	WOH/WOH/11/12	11	59		Washed ahead of Casing.	
	V3		31.00 - 31.36	Su=424/125 psf		47	-20.60	——— 31.00 Grey, wet, stiff, clayey SILT, trace gravel and sand. 55x110 mm vane raw torque readings: V3 = 9.5/2.8 ft-lbs MV = could not push Wash water changed to brown ± 33.0' bgs.	G#176640 A-4, CL-ML WC=26.3% LL=19 PL=15 PI=4
	MV		32.00 - 32.00			54			
						126		Grey, wet, dense, fine to coarse silty SAND, some gravel.	
35	7D	1/1	35.00 - 35.08	50(0.1")	—				
	8D R1	1/0 50/50	36.00 - 36.08 36.50 - 41.07	50(0.1") RQD = 46%	—	NQ Core	-25.90	——— 36.50 Bedrock: Grey/brown, fine grained, fractured, brecciated and metamorphosed PHYLLITE. R1:Core Times (min:sec) 36.5' - 37.5' (6:00) 37.5' - 38.5' (6:30) 38.5' - 39.5' (12:15) 39.5' - 40.5' (3:35) 40.5' - 41.07' (1:51) Recovery=100%	
40							-30.70	——— 41.07 Bottom of Exploration at 41.07 feet below ground surface.	
45									
50									

Remarks:

MV = Unsuccessful vane shear test attempt.

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

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

Page 1 of 1

Boring No.: BB-YR44-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1106(700)X

BRIDGE PLANS

PIN 11067.00

BRIDGE NO. 5849

		BY	DATE
PROJ. MANAGER	J Wentworth		
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	MAR2004
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

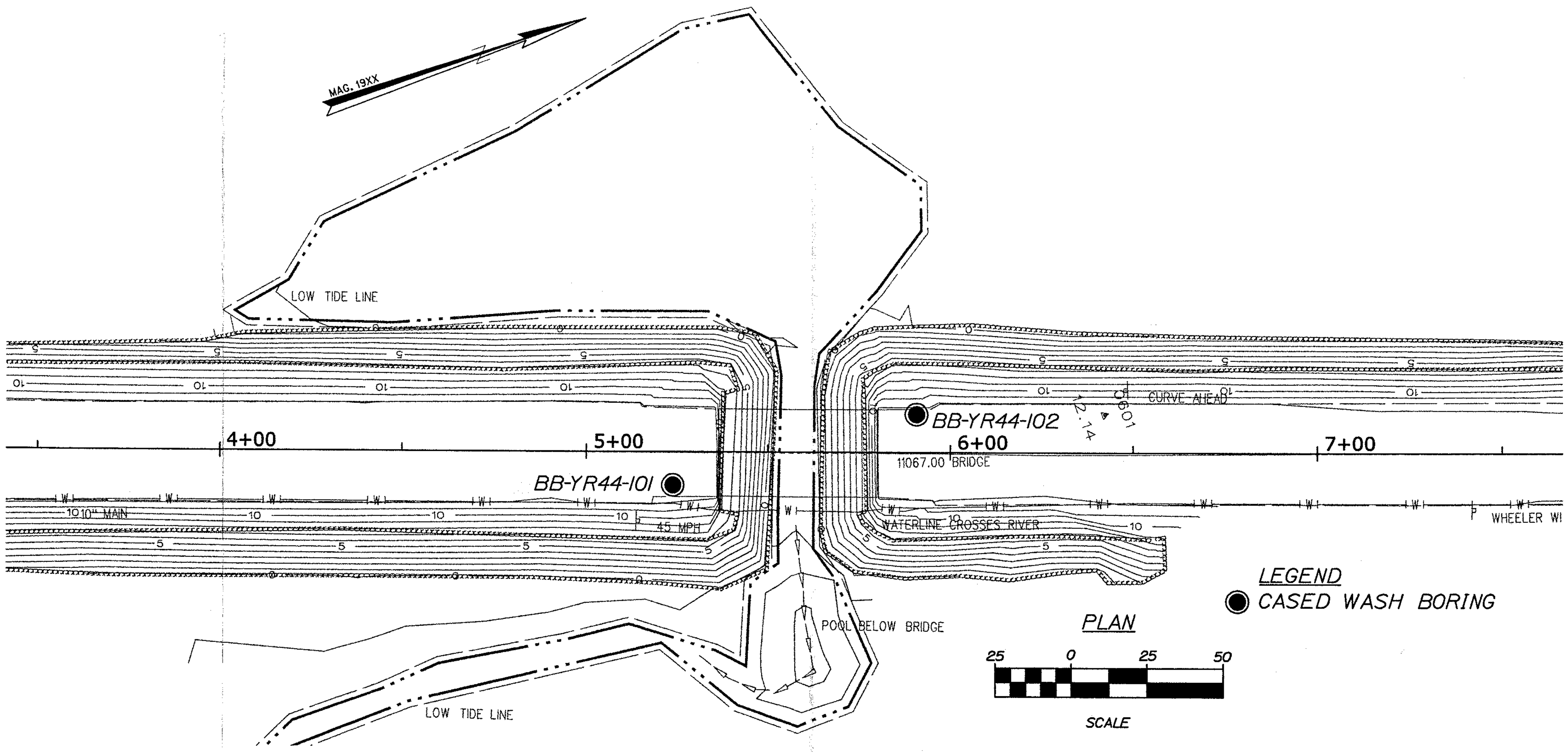
P.E. NUMBER

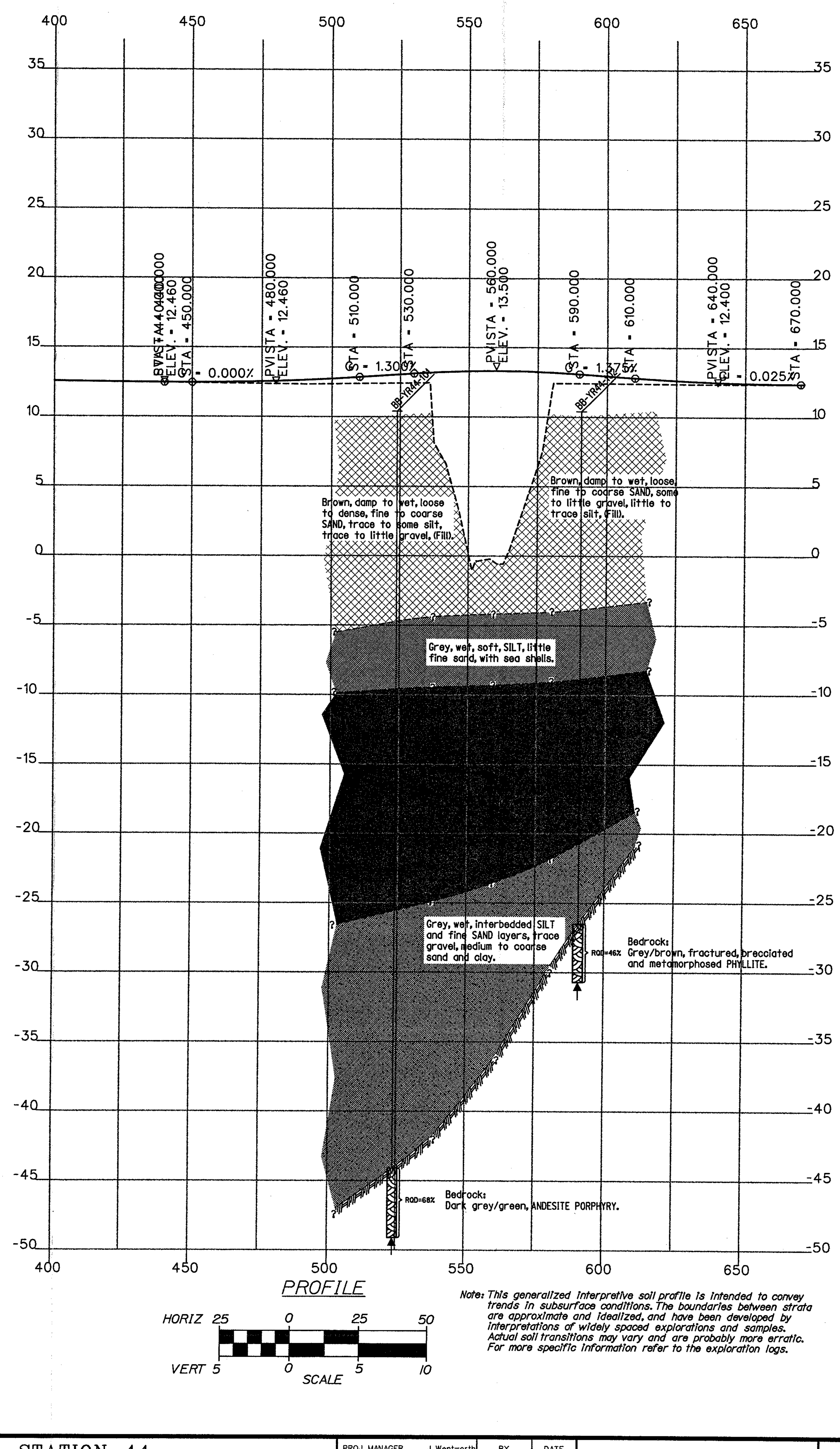
DATE

STATION 44
TIDAL ESTUARY
YORK VILLAGE YORK COUNTY

BORING LOGS

SHEET NUMBER
3
OF _





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

SHEET NUMBER 2 OF _

STATION 44
TIDAL ESTUARY
YORK VILLAGE YORK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

PROJ. MANAGER	J Wentworth	BY	DATE
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	MAR2004
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
AC-BR-1106(700)X
BRIDGE NO. 5849 PIN 11067.00 BRIDGE PLANS

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 44 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR44-101
PIN: 11067.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	5+23.9, 9.1 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98		Pavement 0.4	
	1D-A	24/20	2.0 - 4.0	16/31/12/9	43				(1D-A) (2.0-3.0) Brown, damp, dense, fine to coarse SAND, little silt, trace gravel (Fill).	
	1D-B								(1D/B) (3.0-4.0) Brown, damp, dense, fine to coarse SAND, some silt, little gravel (Fill).	
5	2D	24/16	5.0 - 7.0	3/3/8/15	11				Brown, damp, medium dense, fine to coarse SAND, some silt, little gravel, old pavement, (Fill).	
10	3D	24/17	10.0 - 12.0	5/5/3/7	8	25				
						36	-0.60		11.0 Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	
						33				
						30				
						28				
15	4D	24/16	15.0 - 17.0	4/2/2/4	4	15	-4.60		15.0 Grey, wet, soft, fine sandy SILT with sea shells.	
						15				
						28				
						29				
						19				
20	5D	24/20	20.0 - 22.0	1/1/WOH/1	1	35	-9.60		20.0 Grey, wet, very soft, silty CLAY, trace fine sand and fine sand layers, trace gravel. Stiffer at top of sample, softer with depth.	G#176633 A-6, CL WC=37.8%
						35				
	MV		22.4 - 22.4			28			55x110 mm vane raw torque readings: MV= could not push.	
						22				
25						19				

Remarks:

MV = Unsuccessful vane shear test attempt.

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:	Station 44 Bridge over Tidal Estuary			**Boring No.:**	BB-YR44-101
Location:	York, Maine			**PIN:**	11067.00
Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	5+23.9, 9.1 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/16	25.0 - 27.0	7/1/WOH/1	1	40			Grey, wet, very soft, silty CLAY, varved with sand lenses, trace gravel, trace coarse sand.	G#176634 A-6, CL WC=38.3%
						27				
	MV		27.0 - 27.0			23			55x110 mm vane raw torque readings: MV= could not push.	
						21				
						19				
30	7D	24/10	30.0 - 32.0	Push thru vane		42			Grey, wet, soft, silty CLAY, trace fine sand.	G#176635 A-6, CL WC=47.2% LL=33 PL=19 PI=14
	V1		31.0 - 31.4	Su=402/49 psf		33			55x110 mm vane raw torque readings: V1= 9.0/1.1 ft-lbs MV = could not push	
	MV		32.0 - 32.0			29				
						31				
						36				
35	8D	24/3	35.0 - 37.0	1/5/8/6	13	55	-25.10		35.5 Grey, wet, medium dense, fine silty SAND, trace coarse sand. 55x110 mm vane raw torque readings: MV= could not push.	
	MV		36.0 - 36.0			38				
						27				
						23				
						38				
40	9D	24/6	40.0 - 42.0	3/WOH/3/7	3	48	-29.60		40.0 Grey, wet, soft, SILT, little gravel, sand and clay, dilatent.	G#176636 A-4, CL-ML WC=24.2%
						36				
						28				
						48				
						124	-33.60		44.0	
45	10D	24/6	45.0 - 47.0	22/10/12/11	22	93			Grey, wet, medium dense, fine to coarse SAND, some gravel and silt, trace clay.	G#176637 A-2-4, SC-SM WC=10.2%
						79				
						116				
						132				
50						260				

Remarks:

MV = Unsuccessful vane shear test attempt.

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:	Station 44 Bridge over Tidal Estuary			**Boring No.:**	BB-YR44-101
Location:	York, Maine			**PIN:**	11067.00
Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	5+23.9, 9.1 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	MD	2/0	50.0 - 50.2	60(0.2")	---	aWR			Wash material in spoon a Washed ahead and Roller coned ahead to 54.5' bgs through very dense grey till.	
	R1	60/52	54.5 - 59.5	RQD = 68%		NQ	-44.10		54.5	
55						Core			Bedrock: Dark grey/green, fine grained, ANDESITE porphyry. R1:Core Times (min:sec) 54.5' - 55.5' (18:20) 55.5' - 56.5' (10:00) 56.5' - 57.5' (10:10) 57.5' - 58.5' (9:10) 58.5' - 59.5' (9:05) Recovery=97%	
							-49.10		59.5	
60									**Bottom of Exploration at 59.5 feet below ground surface.**	
65										
70										
75										

Remarks:

MV = Unsuccessful vane shear test attempt.

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: Station 44 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR44-102
PIN: 11067.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	5+90.8, 10.3 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98		Pavement	
									0.4	
									Brown, damp, fine to coarse SAND, little gravel, trace silt, (Fill).	
5	1D	24/10	5.0 - 7.0	4/4/5/6	9				Brown, damp, loose, fine to coarse SAND, some gravel, little silt, (Fill).	
10	2D	24/2	10.0 - 12.0	6/3/4/11	7	46			Brown, wet, loose, fine silty SAND, little gravel, trace medium to coarse sand, (Fill).	
						27				
						208				
						191			Obstruction at 13.0' bgs. Roller coned through.	
						41	-3.60		14.0	
15	3D	24/16	15.0 - 17.0	2/2/2/1	4	34			Grey, wet, soft, sandy SILT with broken shells.	
						30				
						27				
						73				
						43	-8.60		19.0	
20	4D/MV	24/18	20.0 - 22.0	5/3/4/7	7	45			Grey, wet, medium stiff, Silty CLAY, trace gravel and sand, mottled. 55x110 mm and 25.4x50.8 mm vane raw torque readings: MV = could not push	G#176638 A-6, CL WC=31.3% LL=34 PL=20 PI=14
						43				
						41				
						34				
						34				
25										

Remarks:

MV = Unsuccessful vane shear test attempt.

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:	Station 44 Bridge over Tidal Estuary			**Boring No.:**	BB-YR44-102
Location:	York, Maine			**PIN:**	11067.00
Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	5+90.8, 10.3 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/24	25.0 - 27.0	Push thru vane.	---	46			Grey, wet, soft, silty CLAY with black staining, occasional black fine to coarse silty sand layers, trace gravel. 55x110 mm vane raw torque readings: V1 = 10.0/2.4 ft-lbs V2 = 11.8/2.6 ft-lbs	G#176639 A-6, CL WC=42.8%
	V1		26.0 - 26.4	Su=446/107 psf		29				
	V2		27.0 - 27.4	Su=527/116 psf		32				
						30				
						33				
30	6D	24/18	30.0 - 32.0	WOH/WOH/11/12	11	59 WA	-20.60		Washed ahead of Casing.	G#176640 A-4, CL-ML WC=28.3% LL=19 PL=15 PI=4
	V3		31.0 - 31.4	Su=424/125 psf		47			31.0 Grey, wet, stiff, clayey SILT, trace gravel and sand. 55x110 mm vane raw torque readings: V3 = 9.5/2.8 ft-lbs MV = could not push Wash water changed to brown ± 33.0' bgs.	
	MV		32.0 - 32.0			54				
						126				
35	7D	1/1	35.0 - 35.1	80(0.1")	---				Grey, wet, dense, fine to coarse silty SAND, some gravel.	
	8D	1/0	36.0 - 36.1	60(0.1")	---	NQ				
	R1	50/50	36.9 - 41.1	RQD = 46%			-26.50		36.9 Bedrock: Grey/brown, fine grained, fractured, brecciated and metamorphosed PHYLLITE. R1:Core Times (min:sec) 36.9' - 37.9' (6:00) 37.9' - 38.9' (6:30) 38.9' - 39.9' (12:15) 39.9' - 40.9' (3:35) 40.9' - 41.07' (1:51) Recovery=100%	
						Core				
40										
							-30.70		41.1 **Bottom of Exploration at 41.1 feet below ground surface.**	
45										
50										

Remarks:

MV = Unsuccessful vane shear test attempt.

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

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

APPENDIX B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): York

Project Number: 11067.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	GSDC Sheet	WC	LL	PI	Classification		
									Unified	AASHTO	Frost
BB-YR44-101, 5D	5+23.9	9.1 Rt.	20.0-22.0	176633	1	37.8			CL	A-6	III
BB-YR44-101, 6D	5+23.9	9.1 Rt.	25.0-27.0	176634	1	38.3			CL	A-6	III
BB-YR44-101, 7D	5+23.9	9.1 Rt.	30.0-32.0	176635	1	47.2	33	14	CL	A-6	III
BB-YR44-101, 9D	5+23.9	9.1 Rt.	40.0-42.0	176636	1	24.2			CL-ML	A-4	IV
BB-YR44-101, 10D	5+23.9	9.1 Rt.	45.0-47.0	176637	1	10.2			SC-SM	A-2-4	III
BB-YR44-102, 4D	5+90.8	10.3 Lt.	20.0-22.0	176638	2	31.3	34	14	CL	A-6	III
BB-YR44-102, 5D	5+90.8	10.3 Lt.	25.0-27.0	176639	2	42.8			CL	A-6	III
BB-YR44-102, 6D	5+90.8	10.3 Lt.	30.0-32.0	176640	2	28.3	19	4	CL-ML	A-4	IV

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

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

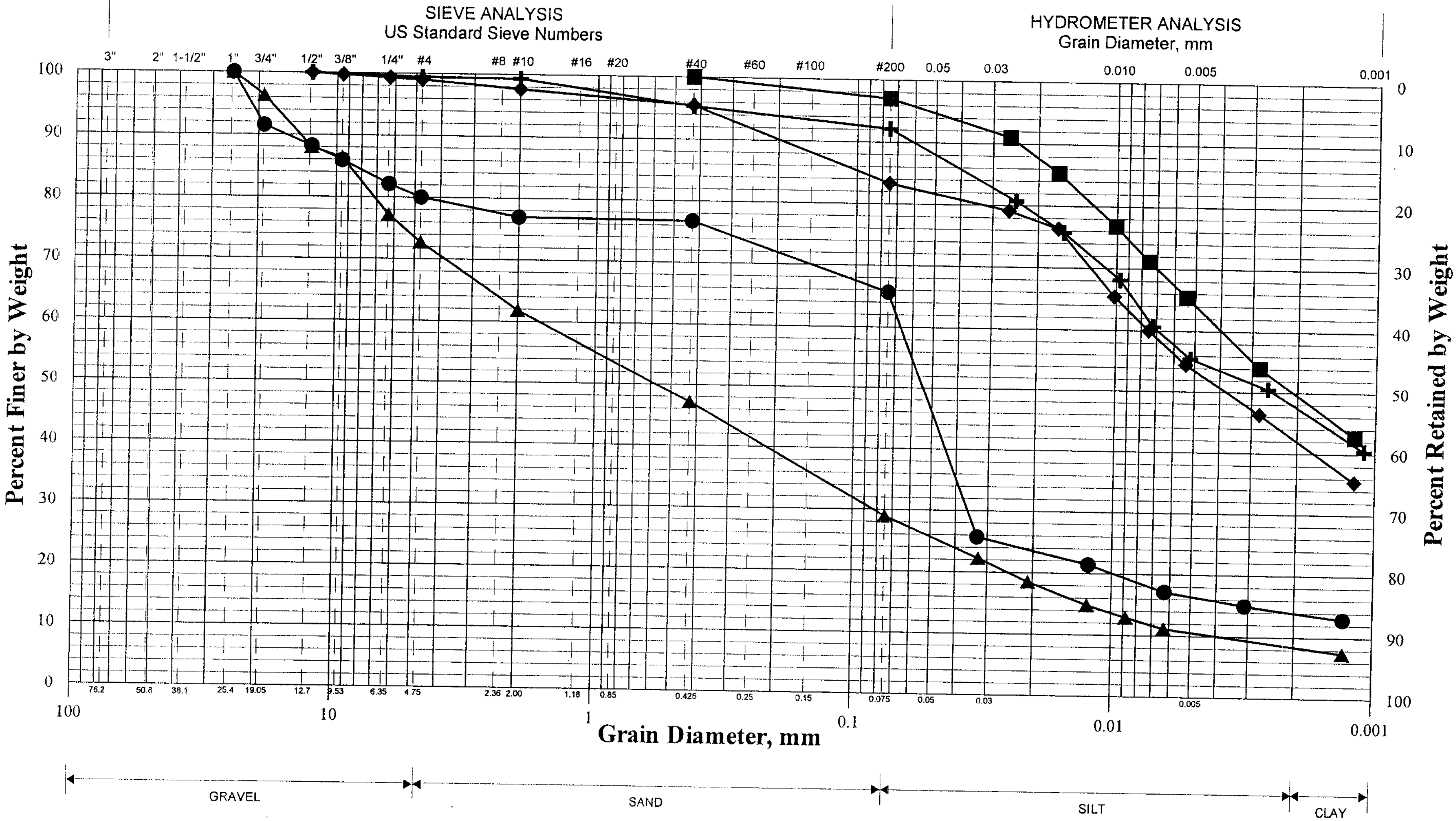
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE

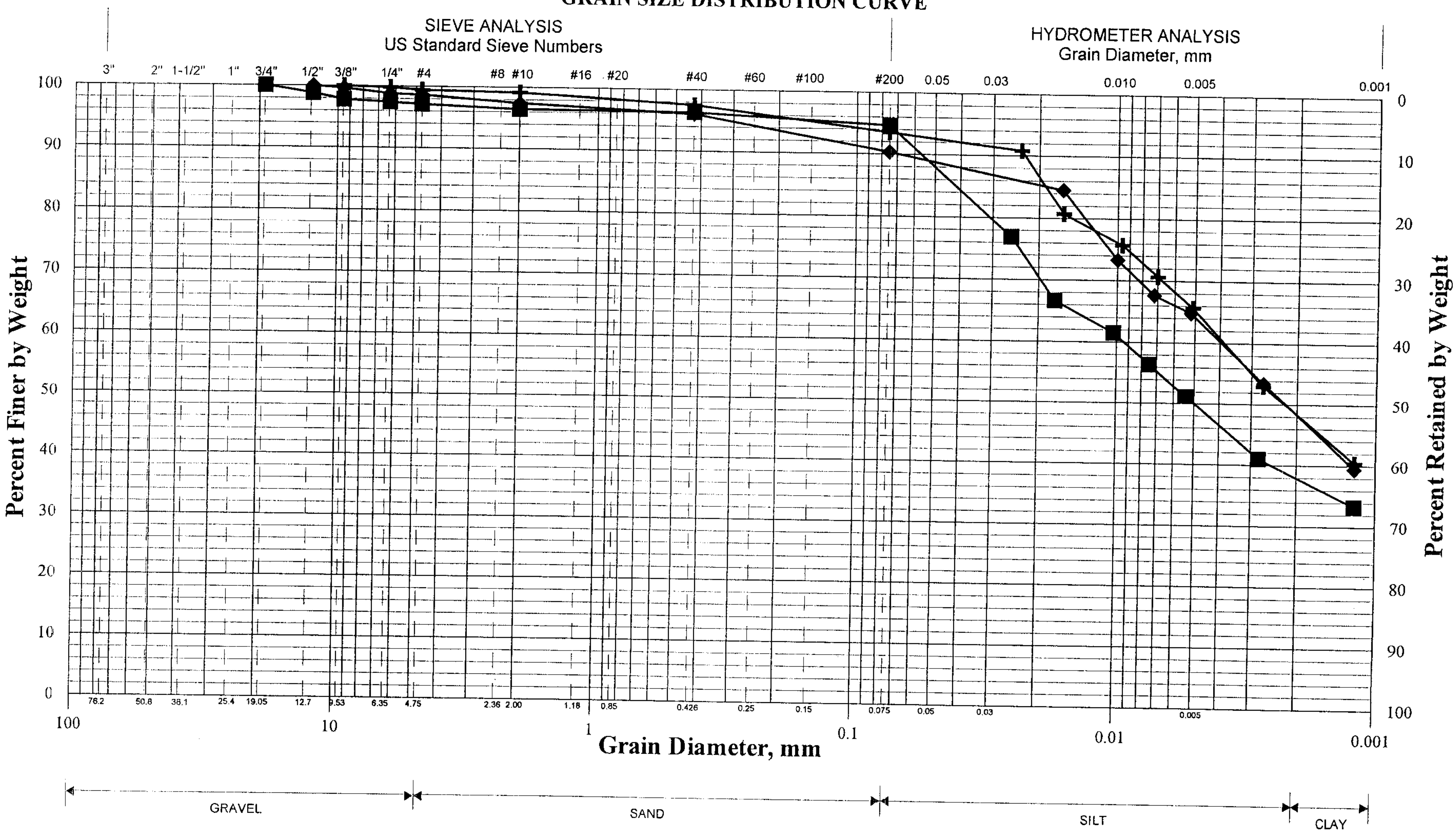


UNIFIED CLASSIFICATION

	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR44-101	5D	20.0-22.0	Silty CLAY, trace sand, trace gravel.	37.8			
◆	BB-YR44-101	6D	25.0-27.0	Silty CLAY, little sand, trace gravel.	38.3			
■	BB-YR44-101	7D	30.0-32.0	Silty CLAY, trace sand.	47.2	33	19	14
●	BB-YR44-101	9D	40.0-42.0	SILT, little gravel, sand and clay.	24.2			
▲	BB-YR44-101	10D	45.0-47.0	Fine to coarse SAND, some gravel and silt, trace clay.	10.2			
×	---							

PIN: 11067.00
Town: York
Reported by: T. White
Date: 3/29/04

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



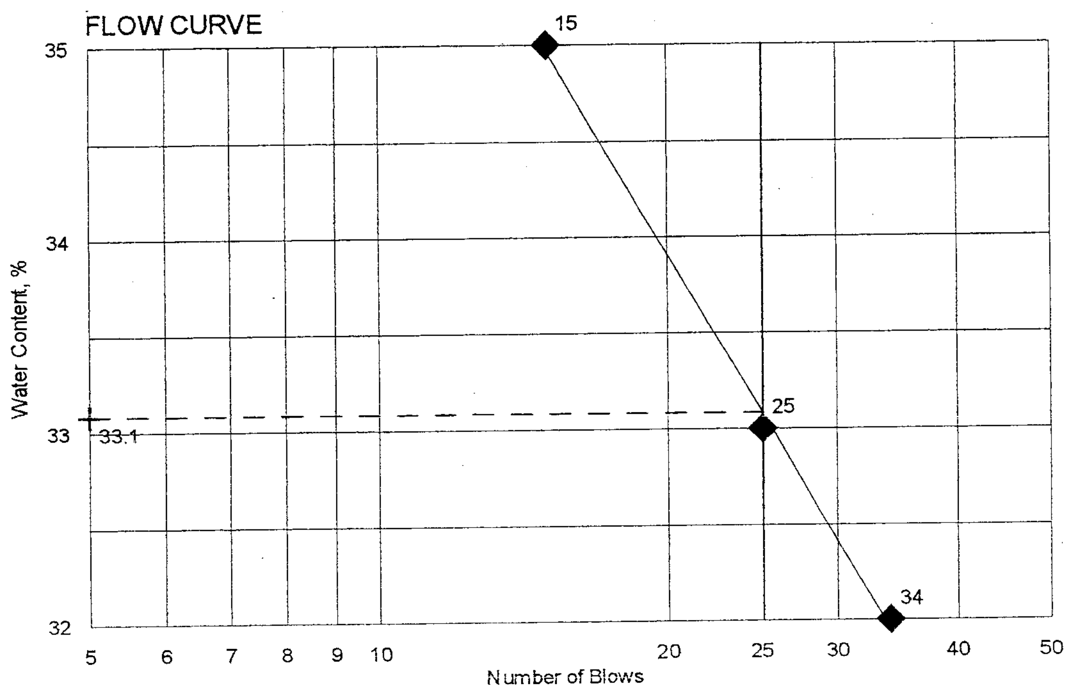
UNIFIED CLASSIFICATION

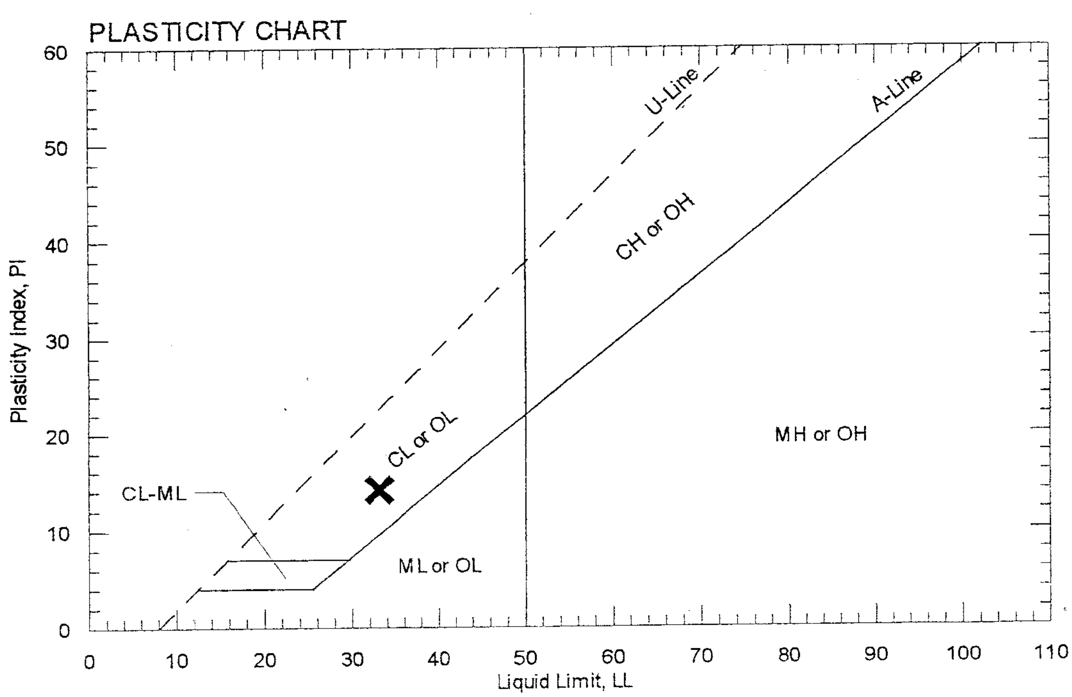
	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR44-102	4D	20.0-22.0	Silty CLAY, trace gravel and sand.	31.3	34	20	14
◆	BB-YR44-102	5D	25.0-27.0	Silty CLAY, trace gravel and sand.	42.8			
■	BB-YR44-102	6D	30.0-32.0	Clayey SILT, trace gravel and sand.	28.3	19	15	4
●	---							
▲	---							
×	---							

PIN: 11067.00
Town: York
Reported by: T. White
Date: 3/29/04

State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	York	Reference No.	176635
PIN	11067.00	Natural water content (%)	47.2
Date	4/21/2004	Plastic limit	19
Boring No.	BB-YR44-101	Liquid limit	33
Station	5+23.9, 9.1 RT	Plasticity index	14
Depth/Sample No.	30-32'/7D	Reported by	KLD

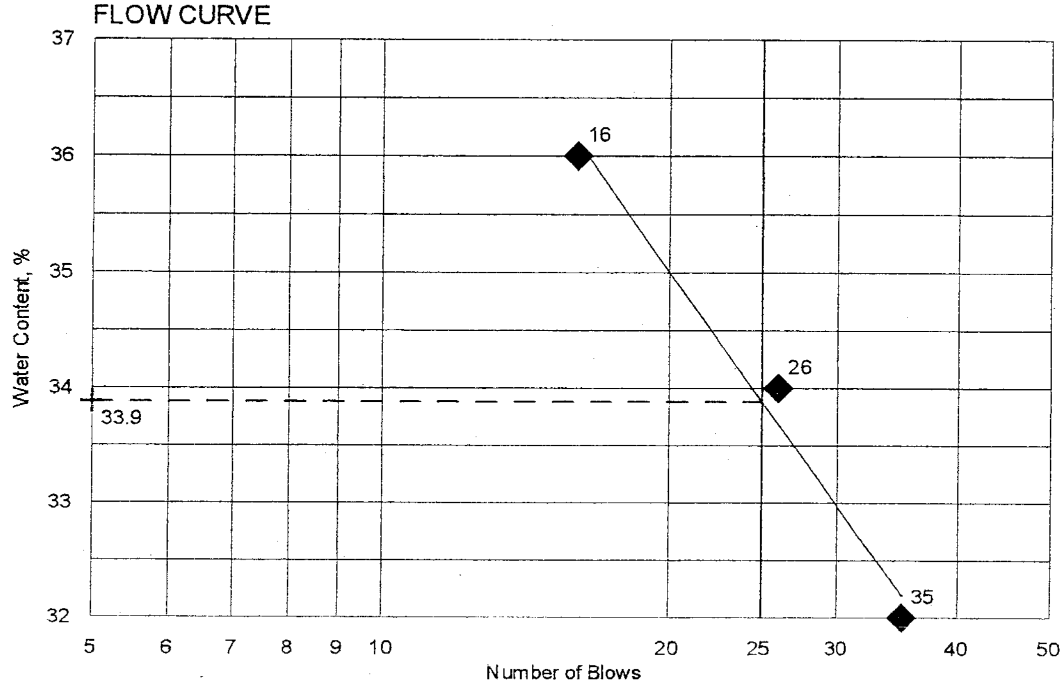




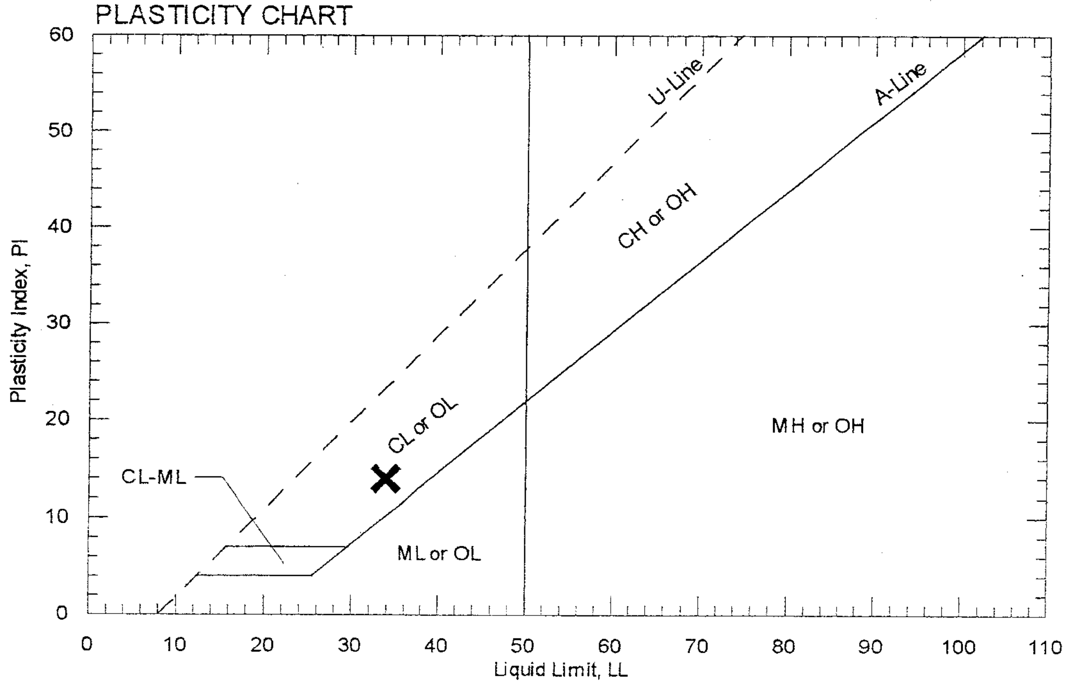
State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	YORK	Reference No.	176638
PIN	11067.00	Natural water content (%)	31.3
Date	4/23/2004	Plastic limit	20
Boring No.	BB-YR44-102	Liquid limit	34
Station	5+90.8, 10.3 LT	Plasticity index	14
Depth/Sample No.	20.0-22.0/4D	Reported by	B. D. FOGG

FLOW CURVE

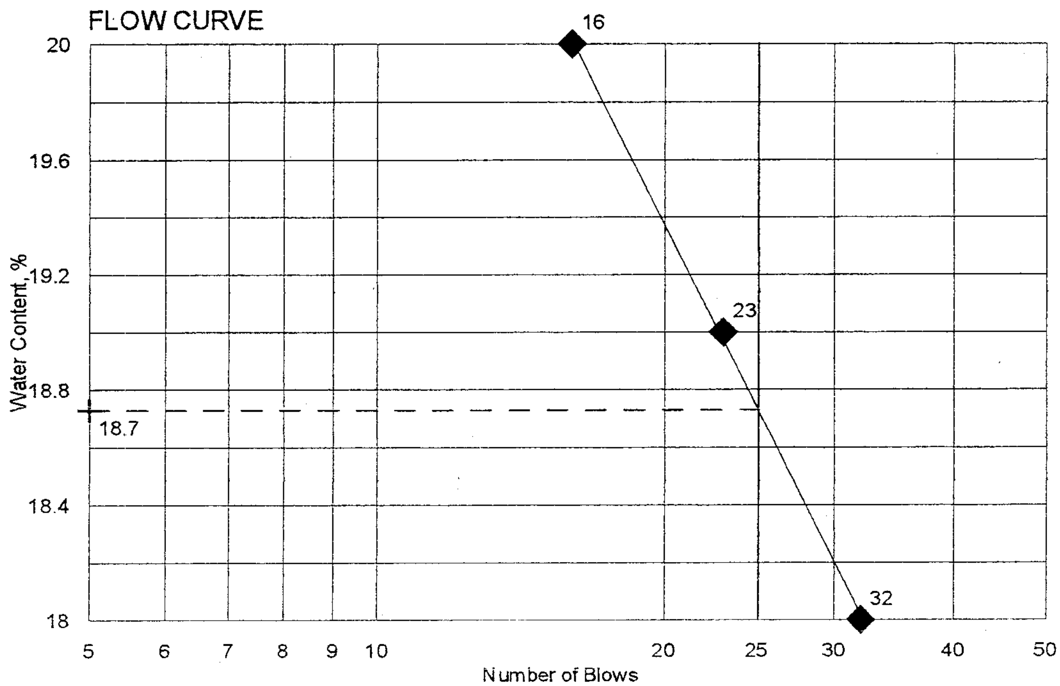


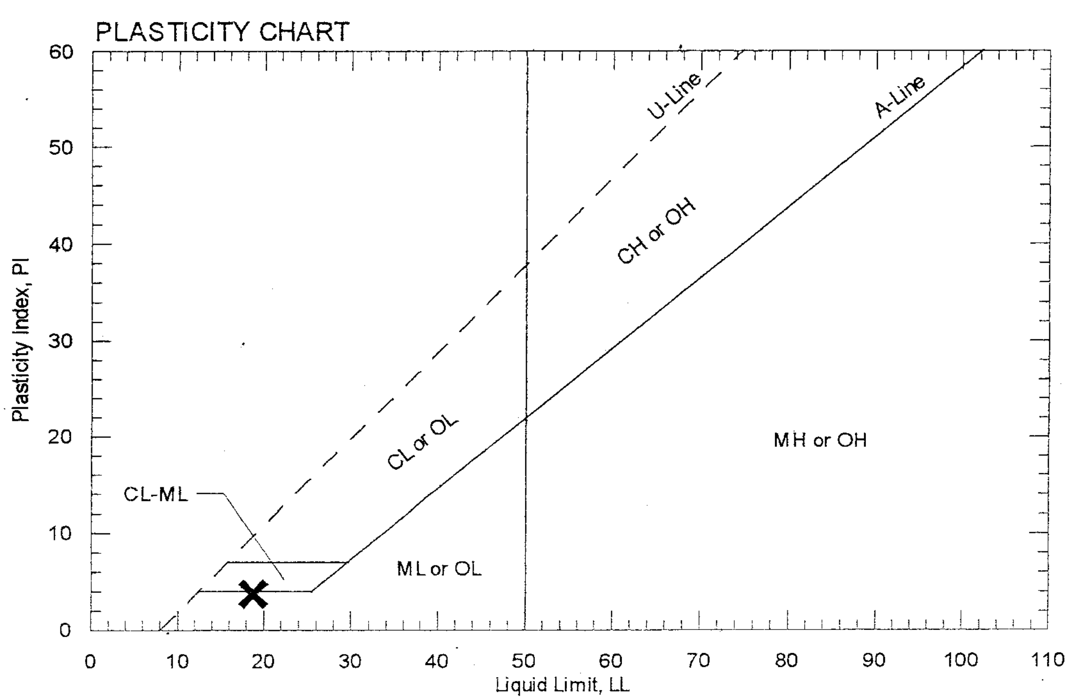
PLASTICITY CHART



State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	York	Reference No.	176640
PIN	11067.00	Natural water content (%)	28.3
Date	4/21/2004	Plastic limit	15
Boring No.	BB-YR44-102	Liquid limit	19
Station	5+90.8, 10.3 LT	Plasticity index	4
Depth/Sample No.	30-32'/6D	Reported by	KLD





APPENDIX C

Calculations

Definition of Units:

$$psf := \frac{lbf}{ft^2} \quad pcf := \frac{lbf}{ft^3} \quad tsf := g\cdot\left(\frac{ton}{ft^2}\right) \quad kip := 1000\cdot lbf$$

$$ksf := \frac{kip}{ft^2} \quad ksi := \frac{kip}{in^2}$$

LIQUIDITY INDEX

wc is close to LL	Soil is normally consolidated
wc is close to PL	Soil is some-to-heavily overconsolidated
wc is intermediate	Soil is overconsolidated
wc is greater than LL	Soil is on the verge of being a viscous liquid

BB-YR44-101 Sample 7D Lab Results: $wc := 47.2 \quad PL := 19 \quad LL_7 := 33$

$$LI := \frac{wc - PL}{LL_7 - PL} \qquad LI = 2.014$$

BB-YR44-102 Sample 4D Lab Results: $wc := 31.3 \quad PL := 20 \quad LL_4 := 34$

$$LI := \frac{wc - PL}{LL_4 - PL} \qquad LI = 0.807$$

BB-YR44-102 Sample 6D Lab Results: $wc := 28.3 \quad PL := 15 \quad LL_6 := 19$

$$LI := \frac{wc - PL}{LL_6 - PL} \qquad LI = 3.325$$

Frost Protection:

MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
York, Maine
DFI = 1100 degree-days

From the lab testing: soils are coarse grained with a water content = ~30%

From Table 5-1 MaineDOT BDG

$Frost\_depth := 49.8in \qquad Frost\_depth = 4.15\,ft$

Use 4.0 feet

Note: The final depth of footing embedment may be controlled by the scour susceptibility of the foundation material and may, in fact, be deeper than the depth required for frost protection.

COMPUTE STRUCTURAL CAPACITY OF H-PILES

Using 50 ksi steel and FS = 4 for integral piles per BDG (0.25Fy)

HP 12 x 53
HP 14 x 73 Note: All matrices set up in this order
HP 14 x 89

$\sigma_a := 0.25 \cdot 50 \cdot \text{ksi}$

$\sigma_a = 12.5\,\text{ksi}$

$$\text{Area}_1 := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix} \cdot \text{in}^2$$

$$Q_{all} := \sigma_a \cdot \text{Area}_1 \qquad Q_{all} = \begin{pmatrix} 193.75 \\ 267.5 \\ 326.25 \end{pmatrix} \text{kip}$$

COMPUTE GEOTECHNICAL CAPACITY OF H-PILES

H pile Capacity - end bearing on rock assume driven through compressive soils and to bedrock

HP 12 x 53
HP 14 x 73 Note: All matrices set up in this order
HP 14 x 89

Method 1: Geotechnical Capacity
Based on Unconfined Compressive Strength of bedrock

From Fang Second Edition Table 3.8:
sandstone compressive strength = 100 - 1,800 kg/cm2
use q_{uc} = 1,200 kg/cm2 = 17,000 psi

$$1200 \cdot \frac{\text{kgf}}{\text{cm}^2} = 1.707 \times 10^4\,\text{psi}$$

$$\text{Area}_1 := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix} \cdot \text{in}^2 \qquad \text{Area}_1 = \begin{pmatrix} 0.108 \\ 0.149 \\ 0.181 \end{pmatrix} \text{ft}^2$$

$q_{uc} := 17000 \cdot \text{psi}$

$$Q_{ult1} := q_{uc} \cdot \text{Area}_1 \qquad Q_{ult1} = \begin{pmatrix} 263.5 \\ 363.8 \\ 443.7 \end{pmatrix} \text{kip}$$

$$Q_{all\_tip} := \frac{Q_{ult1}}{2.25} \qquad Q_{all\_tip} = \begin{pmatrix} 117.111 \\ 161.689 \\ 197.2 \end{pmatrix} \text{kip}$$

Method 2: Geotechnical Capacity by Goodman's Method
Based on Unconfined Compressive Strength of Bedrock
Reference: Principles of Foundation Engineering, BM Das, Second Edition

$\phi := 32 \cdot deg \qquad N_\phi := \left(\tan\left(45 \cdot deg + \frac{\phi}{2}\right)\right)^2 \qquad N_\phi = 3.255$

$q_{pt\_ult} := \left(\frac{q_{uc}}{5}\right) \cdot \left(N_\phi + 1\right) \qquad q_{pt\_ult} = 14.466\,ksi$

$q_{pt\_all} := \frac{q_{pt\_ult}}{2.25} \qquad q_{pt\_all} = 6.429\,ksi$

$Q_{pt\_all} := q_{pt\_all} \cdot Area_1 \qquad Q_{pt\_all} = \begin{pmatrix} 99.652 \\ 137.584 \\ 167.801 \end{pmatrix} kip$

Method 3: Geotechnical Capacity by Goodman's Method
Based on bedrock condition - in this case poor RQD (0 - 31%) Mudstone
Reference: Pile Design and Construction Practice 4th Edition MJ Tomlinson

Low friction: 20-27 for schists, shales
Medium Friction 27-34 for sandstone, siltstone, gneiss, slate
High Friction: 34-40 for granite

$\phi_2 := 32 \cdot deg \qquad N_\phi := \left(\tan\left(45 \cdot deg + \frac{\phi_2}{2}\right)\right)^2 \qquad N_\phi = 3.255$

$q_b := \left(2 \cdot N_\phi\right) \cdot \frac{q_{uc}}{5} \qquad q_b = 22.131\,ksi$

$Q_{ult2} := q_b \cdot Area_1 \qquad Q_{ult2} = \begin{pmatrix} 343.034 \\ 473.608 \\ 577.624 \end{pmatrix} kip$

$Q_{all\_tip2} := \frac{Q_{ult2}}{2.25} \qquad Q_{all\_tip2} = \begin{pmatrix} 152.459 \\ 210.492 \\ 256.722 \end{pmatrix} kip$

Method 4: Geotechnical Capacity
Allowable End Bearing Capacity $Q_{t,\ allow}$:
(Kulhway & Goodman, Fs = 2.25)

Reference: Pile Design and Construction Practice, M.J. Tomlinson, Fourth Edition

Method ignores side resistance - use Driven to assess side friction

Corrrections for wedge failure under a strip footing -
multiply the cN_c factor by 1.25 for a square pile
multiply γBN_γ factor by 0.8 for a square pile

Case I
For RQD of 0 - 70%:
$q_c = 0.33 \times Q_{uc}$
$c = 0.1 \times Q_{uc}$
$\phi = 30$ degrees

Case II
For RQD of 70 - 100%:
$q_c = 0.33$ to $0.88 \times Q_{uc}$
$c = 0.1 \times Q_{uc}$
$\phi = 30$ to 60 degrees

Assume Case I: as RQD = 46 to 68%

$q_{uc} = 1.7 \times 10^4\,\text{psi}$

$c := 0.1 \cdot q_{uc}$ $\qquad c = 1.7 \times 10^3\,\text{psi}$

$\gamma := 150 \cdot \text{pcf}$ $\qquad B := \begin{pmatrix} 12.05 \\ 14.59 \\ 14.70 \end{pmatrix} \text{in}$ $\qquad B = \begin{pmatrix} 1.004 \\ 1.216 \\ 1.225 \end{pmatrix} \text{ft}$

based on Pells & Turner from Tomlinson page 140 - $\phi = 32$ $\qquad N_c := 15.35$ $\qquad N_q := 10.59$ $\qquad N_\gamma := 17.31$

$D := 6 \cdot \text{in}$ Depth of penetration into bedrock

$q_c := 0.33 \cdot q_{uc}$ $\qquad q_c = 5.61 \times 10^3\,\text{psi}$

$$q_{ub} := 1.25 \cdot c \cdot N_c + 0.8 \cdot 0.5 \cdot \gamma \cdot B \cdot N_\gamma + \gamma \cdot D \cdot N_q$$

$q_{ub} = \begin{pmatrix} 32.632 \\ 32.633 \\ 32.633 \end{pmatrix} \text{ksi}$ $\qquad \text{Area}_1 := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix} \cdot \text{in}^2$

$Q_{ult3} := \overrightarrow{(q_{ub} \cdot \text{Area}_1)}$ $\qquad Q_{ult3} = \begin{pmatrix} 505.788 \\ 698.347 \\ 851.724 \end{pmatrix} \text{kip}$

$Q_{all\_tip4} := \dfrac{Q_{ult3}}{2.25}$ $\qquad Q_{all\_tip4} = \begin{pmatrix} 224.795 \\ 310.376 \\ 378.544 \end{pmatrix} \text{kip}$

Use DRIVEN to calculate the skin friction (Q_{side}) and apply FS = 2.25 (output attached)

$$Q_{side} := \begin{pmatrix} 55.22 \\ 65.96 \\ 70.74 \end{pmatrix} \cdot kip$$

$$Q_T := Q_{all\_tip4} + \frac{Q_{side}}{2.25} \qquad Q_T = \begin{pmatrix} 249.337 \\ 339.692 \\ 409.984 \end{pmatrix} kip$$

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\Y44-1253.DVN
Project Name: Station 44 Project Date: 05/17/2004
Project Client: York
Computed By: KMaguire
Project Manager: JWentworth

PILE INFORMATION

Pile Type: H Pile - HP12X53
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	14.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesive	5.00 ft	10.00%	118.00 pcf	500.00 psf	T-80 Clay
3	Cohesive	12.00 ft	5.00%	100.00 pcf	400.00 psf	T-80 Clay
4	Cohesionless	5.90 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	6.98 Kips	2.94 Kips	9.92 Kips
9.99 ft	8.58 Kips	3.26 Kips	11.84 Kips
10.01 ft	8.61 Kips	3.27 Kips	11.88 Kips
13.99 ft	16.11 Kips	3.55 Kips	19.66 Kips
14.01 ft	16.14 Kips	0.48 Kips	16.62 Kips
18.99 ft	21.28 Kips	0.48 Kips	21.76 Kips
19.01 ft	21.29 Kips	0.39 Kips	21.68 Kips
28.01 ft	28.90 Kips	0.39 Kips	29.29 Kips
30.99 ft	32.55 Kips	0.39 Kips	32.94 Kips
31.01 ft	32.60 Kips	3.55 Kips	36.15 Kips
36.89 ft	52.22 Kips	3.55 Kips	55.77 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\Y44-1473.DVN
Project Name: Station 44 Project Date: 05/17/2004
Project Client: York
Computed By: KMaguire
Project Manager: JWentworth

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	14.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesive	5.00 ft	10.00%	118.00 pcf	500.00 psf	T-80 Clay
3	Cohesive	12.00 ft	5.00%	100.00 pcf	400.00 psf	T-80 Clay
4	Cohesionless	5.90 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	9.33 Kips	4.06 Kips	13.39 Kips
9.99 ft	11.47 Kips	4.50 Kips	15.97 Kips
10.01 ft	11.52 Kips	4.51 Kips	16.03 Kips
13.99 ft	21.55 Kips	4.90 Kips	26.45 Kips
14.01 ft	21.59 Kips	0.67 Kips	22.26 Kips
18.99 ft	27.66 Kips	0.67 Kips	28.33 Kips
19.01 ft	27.69 Kips	0.54 Kips	28.22 Kips
28.01 ft	36.69 Kips	0.54 Kips	37.22 Kips
30.99 ft	39.67 Kips	0.54 Kips	40.20 Kips
31.01 ft	39.72 Kips	4.90 Kips	44.62 Kips
36.89 ft	65.96 Kips	4.90 Kips	70.86 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\Y44-1489.DVN
Project Name: Station 44 Project Date: 05/17/2004
Project Client: York
Computed By: KMaguire
Project Manager: JWentworth

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	14.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesive	5.00 ft	10.00%	118.00 pcf	500.00 psf	T-80 Clay
3	Cohesive	12.00 ft	5.00%	100.00 pcf	400.00 psf	T-80 Clay
4	Cohesionless	5.90 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	~~0.01 Kips~~	~~0.01 Kips~~
9.01 ft	10.22 Kips	~~4.95 Kips~~	~~15.17 Kips~~
9.99 ft	12.57 Kips	~~5.49 Kips~~	~~18.06 Kips~~
10.01 ft	12.62 Kips	~~5.50 Kips~~	~~18.12 Kips~~
13.99 ft	23.60 Kips	~~5.98 Kips~~	~~29.58 Kips~~
14.01 ft	23.64 Kips	~~0.82 Kips~~	~~24.46 Kips~~
18.99 ft	29.79 Kips	~~0.82 Kips~~	~~30.61 Kips~~
19.01 ft	29.81 Kips	~~0.65 Kips~~	~~30.47 Kips~~
28.01 ft	38.92 Kips	~~0.65 Kips~~	~~39.57 Kips~~
30.99 ft	41.93 Kips	~~0.65 Kips~~	~~42.59 Kips~~
31.01 ft	41.99 Kips	~~5.98 Kips~~	~~47.97 Kips~~
36.89 ft	70.74 Kips	~~5.98 Kips~~	~~76.72 Kips~~

Earth Pressures:

Cast-in-place integral abutments shall be designed to withstand a maximum lateral applied load equal to the passive earth pressure. A passive earth presure coefficient (K_p) should be calculated using Coulomb Theory.

Coulomb Theory From AASHTO Standard Specifications for Highway Bridges Sixteenth Edition 1996 on Figure 5.5.2.A page 122.

$$\phi := 32 \cdot deg \qquad \delta := \left(\frac{2}{3} \cdot \phi\right) \qquad \beta := 90 \cdot deg \qquad \alpha := 0 \cdot deg$$

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

$$K_p = 7.333$$

Rankine Theory from Das Principles of Foundation Engineering Second Edition Eq. 5.23

$$\phi := 32 \cdot deg$$

$$K_{pr} := \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^2$$

$$K_{pr} = 3.255$$

Bearing Capacity: Native Soils

For any footing founded on native sands or fill.

Method I. Presumptive Bearing Capacity

Based on NavFac DM 7.2 pg 142-143 Table 1 - "Presumptive Values of Allowable Bearing Pressures for Spread Foundations"

Type of Bearing Material:	Consistency In Place:	Allowable Bearing Pressure tons per square foot:	Recommended value:
Coarse to medium, sand sand with little gravel	Very compact	4 to 6	4 tsf
	Medium to compact	2 to 4	3 tsf
	loose	1 to 3	1.5 tsf

Assume very medium dense conditions
Say 3 tsf

$\text{bearing\_capacity} := 3\cdot\text{tsf}$ $\quad$ $\text{bearing\_capacity} = 6\,\text{ksf}$

Method II. Bearing Capacity by Terzaghi

Assumed parameters for the native sand and fill layer:

$\gamma := 120\cdot\text{pcf}$ $\quad$ $\phi := 32\cdot\text{deg}$ $\quad$ from Foundation Analysis and Design, Bowles 4th Edition Table 3-4 pg 141

$c := 0\text{psf}$

$\gamma_1 := 120\cdot\text{pcf} - 62.4\cdot\text{pcf}$

$\gamma_1 = 57.6\,\text{pcf}$ $\quad$ unit weight of native granular soils = 120 pcf less 62.4 pcf unit weight of water for effective unit weight

Assume footing width of 5.0 ft

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

From Bowles 4th Edition Table 4-2 for $\phi = 0$

$N_q := 29.5 \quad N_c := 44.9 \quad N_\gamma := 27.85$

From Bowles 4th Edition Table 4-1
Assume strip footing:

$s_c := 1.0 \quad s_\gamma := 1.0$

Assume footing embedment, D_f of 4 feet

$D_f := 4\cdot\text{ft} \quad q_{bar} := \gamma_1\cdot D_f \quad q_{bar} = 230.4\,\text{psf}$

$$q_{ult} := c\cdot N_c\cdot s_c + q_{bar}\cdot N_q + 0.5\cdot\gamma_1\cdot B\cdot N_\gamma\cdot s_\gamma$$

$$q_{ult} = 1.081\times10^4\,\text{psf}$$

$q_{all} := \dfrac{q_{ult}}{3} \quad q_{all} = 3.602\times10^3\,\text{psf} \quad q_{all} = 3.602\,\text{ksf} \quad q_{all} = 1.801\,\text{tsf}$

Settlement:

Elastic and Consolidation (total) settlement for embankment widening

Placement of up to 4 ft of fill soils on sand overlying silt and silty clay

Elev. 3 ft

Compacted Fill Layer

$\gamma_{fill} := 120 \cdot pcf$ $H_{fill} := 4 \cdot ft$ $N_{fill} := 20$

Elev. -1 ft

Layer 1 - Sand Water table at top of sand

$\gamma_{sand} := 120 \cdot pcf$ $H_{sand} := 15 \cdot ft$ $N_{sand} := 9$

Elev. -16 ft

Layer 2 - Silt

$\gamma_{silt} := 118 \cdot pcf$ $H_{silt} := 5 \cdot ft$

Elev. -21 ft

Layer 3 - Clayey Silt S_u ranges from 400 to 525 pcf

15.5 ft clay

Break layer up into 3 sublayers
Layers 3a, 3b, and 3c

$H_{claya} := 5 \cdot ft$

$\gamma_{clay} := 110 \cdot pcf$ $H_{clay} := 42.0 \cdot ft$

Elev. -26 ft

C_c and C_r values

$H_{clayb} := 5 \cdot ft$

e_o = (wc x SG)/100 use avg wc = 40%

Elev. -31 ft

C_c = 1.3 x wc - 0.2

$H_{clayc} := 5.5 \cdot ft$

Cr = 0.1 x C_c

$e_0 := 1.1$ $C_c := 0.32$ $C_r := 0.032$

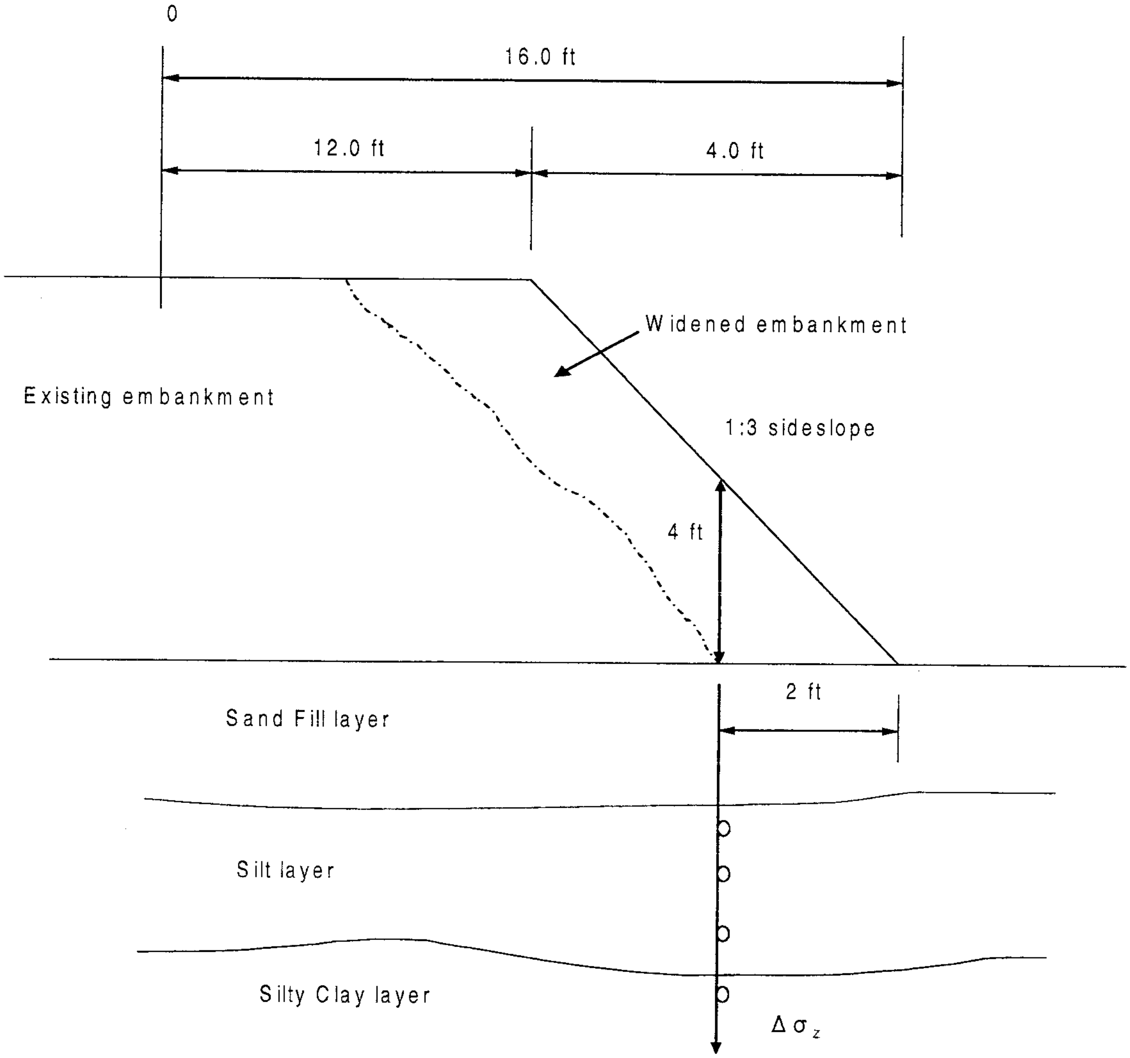
Elev. -36.5 ft

Layer 4 - Silt and sand $\gamma_{ssand} := 120 \cdot pcf$ $H_{ssand} := 19 \cdot ft$ $N_{ssand} := 12$

Top of Bedrock

Elev. -55.5 ft

$\gamma_w := 62.4 \cdot pcf$ $D_w := 10.0 \cdot ft$



Geometry for calculating $\Delta \sigma_z$
(not to scale)

Calculate the change in vertical streess with depth using STRESS
Reference: Productivity Tools For Geotechnical Engineers Volume 1 by J.T. Christian and A. Urzua

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING
Project Name : Station 44 Client : York
Project Number : 11067.00 Project Manager : JWentworth
Date : 5/17/10 Computed by : km
Embank. slope a = 2.00(ft)
Embank. width b = 14.00(ft)
p load/unit area = 500.00(psf)
INCREMENT OF STRESSES FOR X-Z PLANE

Vert. âz
(psf)

X(ft)	10.00	11.00	12.00	13.00	14.00
Z(ft)	(psf)	(psf)	(psf)	(psf)	(psf)
0.00	500.00	500.00	500.00	500.00	250.00
1.00	498.31	496.43	489.79	454.52	249.95
2.00	488.75	479.05	453.95	386.97	249.62
3.00	470.44	451.95	415.04	347.39	248.76
4.00	446.95	423.04	383.00	322.94	247.20
5.00	421.68	395.96	357.40	305.63	244.84
6.00	396.65	371.48	336.26	291.87	241.67
7.00	372.80	349.42	318.06	279.97	237.73
8.00	350.51	329.43	301.88	269.11	233.13
9.00	329.88	311.20	287.16	258.89	227.98
10.00	310.89	294.50	273.60	249.15	222.42
11.00	293.45	279.16	260.99	239.79	216.56
12.00	277.48	265.03	249.24	230.80	210.53
13.00	262.83	252.00	238.25	222.17	204.42
14.00	249.41	239.97	227.97	213.90	198.31
15.00	237.10	228.85	218.36	206.01	192.27
16.00	225.79	218.57	209.36	198.49	186.34
17.00	215.38	209.04	200.93	191.34	180.56
18.00	205.79	200.20	193.04	184.54	174.96
19.00	196.94	192.00	185.65	178.10	169.55
20.00	188.74	184.36	178.72	171.99	164.34
21.00	181.15	177.25	172.22	166.20	159.34
22.00	174.10	170.62	166.12	160.72	154.54
23.00	167.54	164.42	160.39	155.53	149.96
24.00	161.43	158.62	154.99	150.61	145.57
25.00	155.71	153.19	149.91	145.95	141.38
26.00	150.37	148.09	145.12	141.53	137.37
27.00	145.36	143.30	140.60	137.33	133.55
28.00	140.66	138.78	136.33	133.35	129.90
29.00	136.24	134.53	132.30	129.57	126.41
30.00	132.08	130.52	128.47	125.98	123.08
31.00	128.16	126.73	124.85	122.57	119.90
32.00	124.45	123.14	121.42	119.31	116.86
33.00	120.94	119.74	118.15	116.22	113.95
34.00	117.62	116.51	115.05	113.26	111.17
35.00	114.47	113.45	112.10	110.44	108.51
36.00	111.48	110.53	109.28	107.75	105.96
37.00	108.64	107.76	106.60	105.18	103.51
38.00	105.94	105.12	104.04	102.72	101.17
39.00	103.36	102.60	101.60	100.37	98.92
40.00	100.90	100.19	99.26	98.11	96.76
41.00	98.56	97.90	97.03	95.95	94.69
42.00	96.31	95.70	94.88	93.88	92.70
43.00	94.17	93.59	92.83	91.89	90.79
44.00	92.12	91.58	90.86	89.98	88.94
45.00	90.15	89.64	88.97	88.15	87.17
46.00	88.26	87.79	87.16	86.38	85.46
47.00	86.45	86.01	85.41	84.68	83.82
48.00	84.72	84.29	83.74	83.05	82.23
49.00	83.04	82.65	82.12	81.47	80.70
50.00	81.44	81.06	80.57	79.95	79.23
51.00	79.89	79.53	79.07	78.49	77.80
52.00	78.40	78.06	77.62	77.07	76.42
53.00	76.96	76.64	76.22	75.71	75.09
54.00	75.57	75.27	74.88	74.38	73.80
55.00	74.24	73.95	73.58	73.11	72.55

$\Delta\sigma z_1 := 275 \cdot psf$

$\Delta\sigma z_2 := 185 \cdot psf$

$\Delta\sigma z_{3a} := 156 \cdot psf$

$\Delta\sigma z_{3b} := 133 \cdot psf$

$\Delta\sigma z_{3c} := 116 \cdot psf$

$\Delta\sigma z_4 := 87 \cdot psf$

Calculate the stresses at mid-layer

N'/N - Ratio of Corrected blow count to SPT Value
Bazaraa 1967 - FHWA Soils and Foundation Workshop Manual page 6-8

Bearing Capacity Index = C1 for granular soils per FHWA Soils and Foundation Workshop Manual page 6-9

Layer 1: Sand

$$\sigma 1_0 := \gamma_{fill} \cdot H_{fill} + \frac{H_{sand}}{2} \cdot (\gamma_{sand} - \gamma_w) \qquad \sigma 1_0 = 912\,\text{psf}$$

SPT N-value (bpf) $N_{sand} := 9$

AT P_o = 900 psf N'/N = r1 = 1.4 $r1 := 1.4$

Corrected Blow Count $N_{cor} := r1 \cdot N_{sand}$ $N_{cor} = 12.6$

From Figure 6-6 using the "clean well graded fine to coasrse sand" curve

Bearing Capacity Index: $C1 := 55$

Layer 2: Silt

$$\sigma 2_0 := \sigma 1_0 + \frac{H_{sand}}{2} \cdot (\gamma_{sand} - \gamma_w) + \frac{H_{silt}}{2} \cdot (\gamma_{silt} - \gamma_w) \qquad \sigma 2_0 = 1.483 \times 10^3\,\text{psf}$$

Layer 3: Silty Clay layer

$$\sigma 3a_0 := \sigma 2_0 + \frac{H_{silt}}{2} \cdot (\gamma_{silt} - \gamma_w) + \frac{H_{claya}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3a_0 = 1741\,\text{psf}$$

$$\sigma 3b_0 := \sigma 3a_0 + \frac{H_{claya}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{clayb}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3b_0 = 1979\,\text{psf}$$

$$\sigma 3c_0 := \sigma 3b_0 + \frac{H_{clayb}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{clayc}}{2} \cdot (\gamma_{clay} - \gamma_w) \qquad \sigma 3c_0 = 2228.9\,\text{psf}$$

Layer 4: Loose Sand layer

$$\sigma 4_0 := \sigma 3c_0 + \frac{H_{clayc}}{2} \cdot (\gamma_{clay} - \gamma_w) + \frac{H_{ssand}}{2} \cdot (\gamma_{ssand} - \gamma_w) \qquad \sigma 4_0 = 2907\,\text{psf}$$

SPT N-value (bpf) $N_{ssand} := 12$

AT P_o = 2900 psf N'/N = r1 = 0.84 $r1 := 0.84$

Corrected Blow Count $N_{cor} := r1 \cdot N_{ssand}$ $N_{cor} = 10.08$

From Figure 6-6 using the "clean well graded fine to medium sand" curve

Bearing Capacity Index: $C2 := 48$

Settlement per layer

$$\Delta H_1 := H_{sand} \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma 1_0 + \Delta\sigma z_1}{\sigma 1_0}\right)$$ $\Delta H_1 = 0.375\,in$ Elastic settlement

$$\Delta H_2 := H_{silt} \cdot \frac{C_r}{1 + e_0} \cdot \log\left(\frac{\sigma 2_0 + \Delta\sigma z_2}{\sigma 2_0}\right)$$ $\Delta H_2 = 0.047\,in$ Consolidation settlement

$$\Delta H_{3a} := H_{claya} \cdot \frac{C_r}{1 + e_0} \cdot \log\left(\frac{\sigma 3a_0 + \Delta\sigma z_{3a}}{\sigma 3a_0}\right)$$ $\Delta H_{3a} = 0.034\,in$ Consolidation settlement

$$\Delta H_{3b} := H_{clayb} \cdot \frac{C_c}{1 + e_0} \cdot \log\left(\frac{\sigma 3b_0 + \Delta\sigma z_{3b}}{\sigma 3b_0}\right)$$ $\Delta H_{3b} = 0.258\,in$ Consolidation settlement

$$\Delta H_{3c} := H_{clayc} \cdot \frac{C_c}{1 + e_0} \cdot \log\left(\frac{\sigma 3c_0 + \Delta\sigma z_{3c}}{\sigma 3c_0}\right)$$ $\Delta H_{3c} = 0.222\,in$ Consolidation settlement

$$\Delta H_4 := H_{ssand} \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma 4_0 + \Delta\sigma z_4}{\sigma 4_0}\right)$$ $\Delta H_4 = 0.061\,in$ Elastic settlement

$Total\_Settlement := \Delta H_1 + \Delta H_2 + \Delta H_{3a} + \Delta H_{3b} + \Delta H_{3c} + \Delta H_4$

$Total\_Settlement = 0.996\,in$

$Elastic\_Settlement := \Delta H_1 + \Delta H_4$ $Elastic\_Settlement = 0.435\,in$

$Consolidation\_Settlement := \Delta H_2 + \Delta H_{3a} + \Delta H_{3b} + \Delta H_{3c}$

$Consolidation\_Settlement = 0.561\,in$

Check Consolidation Settlement with SAF-I
Reference: Productivity Tools For Geotechnical Engineers Volume 1
by J.T. Christian and A. Urzua

```
ONE DIMENSIONAL SETTLEMENT ANALYSIS/PROTOTYPE ENGINEERING INC.
3            STRIP FOOTING VERTICAL EMBANKMENT LOADING
3
3 Project Name   : Station 44           Client         : York
3 Project Number : 11067.00             Project Manager : JWentworth
3 Date           : 5/17/10              Computed by    : km
3
3        Increment of stresses obtained using : Boussinesq
3                  Settlement for X-Direction
3
3 Embank. slope a        =   2.00 (ft)     p load/unit area   =  500.00 (psf)
3 Embank. width          =  14.00 (ft)     Foundation Elev.   =  -1.00 (ft)
3 Ground Surface Elev.   =  -1.00 (ft)
3 Water table Elev.      =  -1.00 (ft)     Unit weight of Wat. =  62.40 (pcf)
3
3            LAYER              COEFFICIENT         UNIT    SPECIFIC  VOID
3    No. TYPE   THICK.  COMP. RECOMP. SWELL.  WEIGHT  GRAVITY  RATIO
3                (ft)                          (pcf)
3    1  INCOMP. 15.0    -----   -----   -----   120.00   ----     ----
3    2  COMP.    5.0    0.320   0.032   0.032   120.00   2.65     1.10
3    3  COMP.   15.5    0.320   0.032   0.032   110.00   2.65     1.10
3    4  INCOMP. 19.0    -----   -----   -----   120.00   ----     ----
3
3            SUBLAYER               SOIL STRESSES
3    No.  THICK.      ELEV.      INITIAL    MAX.PAST PRESS.
3          (ft)        (ft)       (psf)         (psf)
3    1   INCOMP.
3    2    5.00       -18.50     1008.00       1008.00
3    3   15.50       -28.75     1520.90       1520.90
3    4   INCOMP.
3
3          X =   4.00     X =   5.00     X =   6.00     X =   7.00
3 Layer   Stress   Sett.  Stress   Sett.  Stress   Sett.  Stress   Sett.
3          (psf)   (in.)  (psf)    (in.)  (psf)    (in.)  (psf)    (in.)
3  1         INCOMP.        INCOMP.        INCOMP.        INCOMP.
3  2      204.60   0.73   210.53   0.75   214.60   0.77   216.67   0.77
3  3      139.93   1.08   141.84   1.10   143.14   1.11   143.79   1.11
3  4         INCOMP.        INCOMP.        INCOMP.        INCOMP.
3                 ------         ------         ------         ------
3                  1.82           1.85           1.87           1.89
3
3          X =   8.00     X =   9.00     X =  10.00     X =  11.00
3 Layer   Stress   Sett.  Stress   Sett.  Stress   Sett.  Stress   Sett.
3          (psf)   (in.)  (psf)    (in.)  (psf)    (in.)  (psf)    (in.)
3  1         INCOMP.        INCOMP.        INCOMP.        INCOMP.
3  2      216.66   0.77   214.58   0.77   210.49   0.75   204.54   0.73
3  3      143.78   1.11   143.12   1.11   141.81   1.10   139.89   1.08
3  4         INCOMP.        INCOMP.        INCOMP.        INCOMP.
3                 ------         ------         ------         ------
3                  1.88           1.87           1.85           1.82
```

Settlement =
Approx. 2"

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 7/6/04

To: Mike Babb **Dept:** Reproduction Room

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: York, Station 44 Bridge over Tidal Estuary, PIN. 11067.00

Attached is one (1) copy of Soils Report 2004-23, entitled "GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: STATION 44 BRIDGE OVER TIDAL ESTUARY, YORK, MAINE" dated: July 2004.

Please forward your copy to Kate Maguire after report has been scanned.

taw
att: 1 of 2004-23

COPY

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 7/6/04

To: Kevin Cummings **Dept:** Urban and Federal Bridge Program

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: York, Station 44 Bridge over Tidal Estuary, PIN. 11067.00

Attached is one (1) copy of Soils Report 2004-23, entitled "GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: STATION 44 BRIDGE OVER TIDAL ESTUARY, YORK, MAINE" dated: July 2004.

taw
att: 1 of 2004-23

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 7/6/04

To: Matthew Steele | **Dept:** Environment Office

From: Kate Maguire | **Dept:** Urban and Federal Bridge Program Geotechnical Section

Subject: York, Station 44 Bridge over Tidal Estuary, PIN. 11067.00

Attached is one (1) copy of Soils Report 2004-23, entitled "GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: STATION 44 BRIDGE OVER TIDAL ESTUARY, YORK, MAINE" dated: July 2004.

taw
att: 1 of 2004-23

COPY

State of Maine

Department of Transportation
Bridge Program - Geotechnical
16 State House Station
Augusta, Maine 04333-0016

Kate Maguire, PE
Phone: 624-3415
FAX: 624-3491
email: kate.maguire@maine.gov

Addendum #1

To: File
cc: TEDOCS
From: Kate Maguire, PE
Date: January 14, 2009
Re: Soils Report No. 2004-21
Geotechnical Design Report
For the Replacement of
Station 34 Bridge
Over Tidal Estuary
York, Maine
PIN: 15111.00

==

The following changes are made to the Geotechnical Design Report for the Replacement of Station 34 Bridge Over Tidal Estuary York, Maine Soils Report No. 2004-21:

1. Throughout the document, replace Project Identification Number (PIN) 11066.00 with PIN 15111.00.

2. Throughout the document, replace Fed No. AC-BH-1106(600)X with Fed No. BH-1511(100)X.

3. Replace the Table on page 6 of the report text with the following table which gives the correct Top of Rock Elevation for boring BB-YR34-101:

Location	Ground Elevation	Depth to Rock bgs	Top of Rock Elevation	Rock Quality Designation
BB-YR34-101 Abutment #1	10.2 ft	94.8 ft	-84.6 ft	31%
BB-YR34-102 Abutment # 2	10.2 ft	105.5 ft	-95.3 ft	88%

4. Replace the third paragraph of Section 7.2 on page 8 of the report text with the following text which corrects the Soil Type reference given:

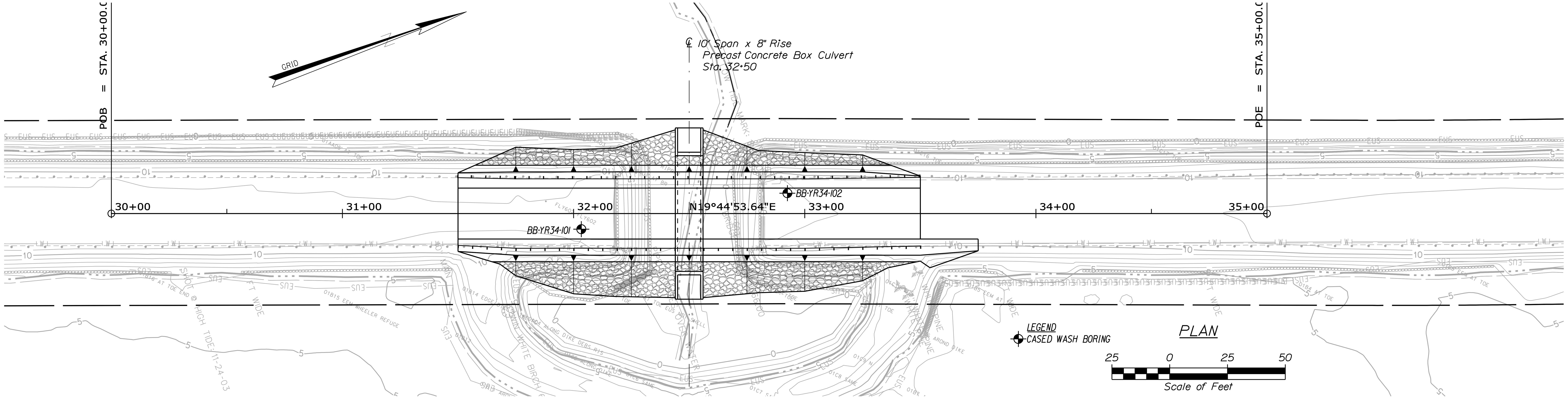
The Designer may assume Soil Type 4 (BDG Section 3.6.1) for retaining wall backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf, and a soil-concrete friction coefficient of 0.45.

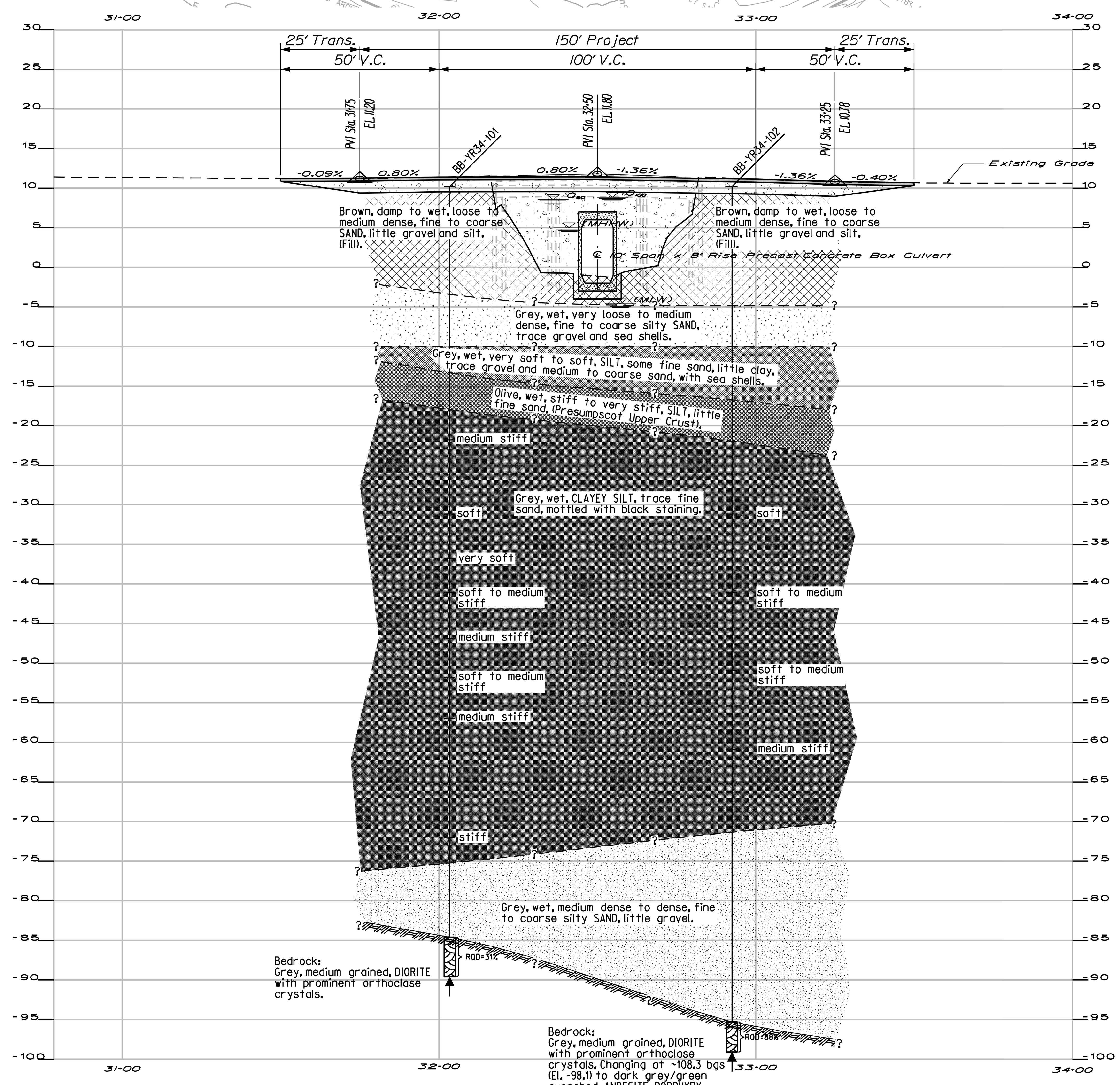
5. Replace Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile with the attached Sheet 2 which has been updated with current station information.

6. Replace Sheet 3 - Boring Logs with the attached Sheet 3 which has been updated with current station information.

7. Replace Appendix A - Boring Logs with the attached pages which have been updated with current station information.

8. Replace Appendix B – Laboratory Data with the attached pages which have been updated with current station information.





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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1511(100)X

PIN 15111.00

BRIDGE PLANS

BRIDGE NO. 5848

	BY	DATE
PROJ. MANAGER	J. Wentworth	
DESIGN-DETAILED	T. WHITE	JAN 2009
CHECKED-REVIEWED	K. MAGUIRE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

STATION 34 BRIDGE
TIDAL ESTUARY
YORK COUNTY
YORK

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\GEOTECH\MSTA\004_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:1/12/2009

Filename: ...\MSTA\005_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:1/12/2009

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/3/04-3/3/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	32+03.4, 6.8 Rt.	Casing ID/OD:	HW	Water Level*:	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.78	Pavement —0.42	
5	1D	24/24	5.00 - 7.00	7/15/12/16	27			Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/6	10.00 - 12.00	4/3/4/4	7	18		Brown, wet, loose, fine to coarse SAND, little gravel and silt, (Fill).	
11						19			
12						27			
13						18	-2.80	—13.00	
14						26			
15						2			
16	3D	24/2	16.00 - 18.00	5/3/1/4	4	11		Grey, wet, very loose, fine to coarse silty SAND, trace gravel and sea shells.	
17						10			
18						12			
19						12			
20	4D	24/20	20.00 - 22.00	2/WOH/WOH/WOH	---	16	-9.80	—20.00 Grey, wet, very soft, SILT, some fine sand, little clay, trace gravel, medium to coarse sand and sea shells.	G#176641 A-4, CL-ML WC=36.8%
21						11			
22						19			
23						30	-12.80	—23.00	
24						40			
25	5D/MV	24/20	25.00 - 27.00	4/7/10/13	17	69		Olive, wet, very stiff, SILT, little fine sand, (Presumpscot upper crust). Attempt 55x110 mm vane: could not push	
26						69			
27						69			
28						63	-17.80	—28.00	
29						69			
30	6D	24/23	30.00 - 32.00	3/2/3/3	5	78 aWA		Grey, wet, medium stiff, SILT, some clay, trace fine sand, mottled. aWashed ahead of casing.	G#176642 A-6, CL WC=31.7% LL=34 PL=20 PI=14
31						70			
32						64			
33						64			
34						50			
35	7D	24/22	35.00 - 37.00	2/2/1/WOH	3	52		Grey, wet, very loose, fine silty SAND, uniform, dilatent.	
36						52			
37						50			
38						38			
39						32			
40	8D V1	24/24	40.00 - 42.00 40.64 - 41.00	Push thru vane Su=259/125 psf		62		Grey, wet, soft, clayey SILT, trace fine sand layers, black staining. 55x110 mm vane raw torque readings: V1 = 5.8/2.8 ft-lbs V2 = 10.7/1.5 ft-lbs	G#176643 A-4, CL WC=36.8% LL=28 PL=19 PI=9
41	V2		41.64 - 42.00	Su=478/67 psf		45			
42						44			
43						48			
44						50			
45	9D V3	24/24	45.00 - 47.00 45.64 - 46.00	Push thru vane Su=192/67 psf		87		Similar to above, very soft. 55x110 mm vane raw torque readings: V3 = 4.3/1.5 ft-lbs V4 = 5.3/1.6 ft-lbs	
46	V4		46.64 - 47.00	Su=237/71 psf		68			
47						60			
48						40			
49						47			
50	10D V5	24/24	50.00 - 52.00 50.64 - 51.00	Push thru vane Su=437/80 psf		73		Grey, wet, soft to medium stiff, clayey SILT with occasional fine sand layers, black staining. 55x110 mm vane raw torque readings: V5 = 9.8/1.8 ft-lbs V6 = 13.5/1.7 ft-lbs	
51	V6		51.64 - 52.00	Su=603/76 psf		84			
52						85			
53						69			
54						69			
55	MD V7	24/0	55.00 - 57.00 55.64 - 56.00	Push thru vane Su=545/76 psf		94		No recovery, similar soils on spoon, medium stiff. 55x110 mm vane raw torque readings: V7 = 12.2/1.7 ft-lbs V8 = 12.8/1.3 ft-lbs	
56	V8		56.64 - 57.00	Su=571/58 psf		57			
57						47			
58						34			
59						31			
60	11D V9	24/24	60.00 - 62.00 60.64 - 61.00	Push thru vane Su=518/89 psf		49		Grey, wet, soft to medium stiff, SILT, some clay, with little fine sand layers, black staining. 55x110 mm vane raw torque readings: V9 = 11.6/2.0 ft-lbs V10 = 11.0/2.1 ft-lbs	G#176644 A-4, CL-ML WC=28.0% LL=23 PL=17 PI=6
61	V10		61.64 - 62.00	Su=491/94 psf		40			
62						41			
63						38			
64						32			
65	12D V11	24/24	65.00 - 67.00 65.64 - 66.00	Push thru vane Su=603/143 psf		52		Similar to above, medium stiff. 55x110 mm vane raw torque readings: V11 = 13.5/3.2 ft-lbs V12 = 13.9/3.7 ft-lbs	
66	V12		66.64 - 67.00	Su=621/165 psf		41			
67						46			
68						40			
69						50			
70	13D V13	24/24	70.00 - 72.00 70.64 - 71.00	Push thru vane Su=763/89 psf		74		Grey, wet, medium stiff, clayey SILT, little fine sand with fine sand layers. 55x110 mm vane raw torque readings: V13 = 17.1/2.0 ft-lbs V14 = 14.4/4.8 ft-lbs	
71	V14		71.64 - 72.00	Su=643/214 psf		61			
72						61			
73						58			
74						53			

Remarks:

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Page 1 of 2
Boring No.: BB-YR34-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/3/04-3/3/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	32+03.4, 6.8 Rt.	Casing ID/OD:	HW	Water Level*:	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	14D V15	24/3	75.00 - 77.00 75.64 - 76.00	WOR/4/5/6 Su=808/304 psf	9	64		Similar to above, medium stiff. 55x110 mm vane raw torque readings: V15 = 18.1/6.8 ft-lbs MV = could not push	
76	MV		76.20 - 76.20			55			
77						62			
78						73			
79						128			
80	15D	24/20	80.00 - 82.00	28/58/54/24	112	78 bWA		Grey, wet, very stiff, SILT, some fine sand, trace clay. bWashed Ahead of Casing.	G#176645 A-2-4, SM WC=21.5%
81						52			
82						39			
83						25			
84						26			
85	16D	24/24	85.00 - 87.00	WOR/8/11/10	19	62	-74.80	—85.00 Grey, wet, medium dense, fine silty SAND, uniform.	
86						66			
87						55			
88						55			
89						98			
90	17D	24/6	90.00 - 92.00	18/4/14/9	18	115		Grey, wet, medium dense, fine to coarse silty SAND, little gravel.	G#176646 A-4, CL-ML WC=18.2%
91						79			
92						82			
93						93		c156 blows for 0.4'.	
94	R1	60/60	94.80 - 99.80	RQD = 31%		c156 NQ	-84.60	—94.80 Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. R1:Core Times (min:sec) 94.8' - 95.8' (5:05) 95.8' - 96.8' (5:50) 96.8' - 97.8' (6:10) 97.8' - 98.8' (6:05) 98.8' - 99.8' (5:15) Recovery=100%	
99							-89.60	—99.80 Bottom of Exploration at 99.80 feet below ground surface.	

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Page 2 of 2
Boring No.: BB-YR34-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-102
PIN: 15111.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/9/04-3/9/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	32+92.5, 9.1 Lt.	Casing ID/OD:	NW	Water Level*:	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.62	Pavement —0.58	
5	1D	24/4	5.00 - 7.00	6/6/17/22	23			Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/9	10.00 - 12.00	2/6/11/7	17			Brown, wet, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
15	3D	24/17	15.00 - 17.00	6/8/5/7	13	24	-4.80	—15.00 Grey, wet, medium dense, fine silty SAND with sea shells.	
16						14			
17						18			
18						17			
19						17			
20	4D	24/16	20.00 - 22.00	2/2/2/2	4	37	-9.80	—20.00 Grey, wet, soft, fine sandy SILT, little clay, trace gravel and medium to coarse sand with sea shells.	G#176647 A-4, CL-ML WC=31.6%
21						24			
22						23			
23						23			
24						24			
25	5D	24/24	25.00 - 27.00	1/WOH/WOH/2	---	32		Grey, wet, soft, SILT, little clay, trace fine sand.	G#176648 A-4, ML WC=45.5% LL=37 PL=33 PI=4
26						31			
27	V1		27.64 - 28.00	aSu=1964/- psf		36	-16.80	—27.00 55x110 mm vane raw torque readings: V1 = 44/-- ft-lbs aVane reached maximum torque reading without shearing, no remolded was attempted.	
28						88			
29						126			
30	6D	24/14	30.00 - 32.00	8/4/7/9	11	45		Olive, wet, stiff, SILT, (Presumpscot upper crust).	
31						44			
32						38	-21.80	—32.00	
33						39			
34						34			
35						34			
36						38			
37						38			
38						32			
39						30			
40	7D V2	24/24	40.00 - 42.00 40.64 - 41.00	Push thru vane Su=446/67 psf		62		Grey, wet, soft, clayey SILT, trace fine sand, black staining. 55x110 mm vane raw torque readings: V2 = 10.0/1.5 ft-lbs V3 = 8.8/1.1 ft-lbs	
41	V3		41.64 - 42.00	Su=393/49 psf		50			
42						37			
43						34			
44						30			
45						34			
46						30			
47						32			
48						31			
49						30			
50	8D V4	24/24	50.00 - 52.00 50.64 - 51.00	Push thru vane Su=540/71 psf		54		Grey, wet, soft, silty CLAY with trace fine sand layers and black staining. 55x110 mm vane raw torque readings: V4 = 12.1/1.6 ft-lbs V5 = 7.4/1.2 ft-lbs	G#176649 A-6, CL WC=40.0% LL=32 PL=20 PI=12
51	V5		51.64 - 52.00	Su=330/54 psf		41			
52						39			
53						35			
54						33			
55						37			
56						35			
57						33			
58						33			
59						34			
60	9D V6	24/24	60.00 - 62.00 60.64 - 61.00	Push thru vane Su=241/112 psf		73		Similar to above, soft to medium stiff. 55x110 mm vane raw torque readings: V6 = 5.4/2.5 ft-lbs V7 = 12.0/2.2 ft-lbs	
61	V7		61.64 - 62.00	Su=536/98 psf		56			
62						53			
63						45			
64						42			
65						48			
66						40			
67						45			
68						38			
69						34			
70	10D V8	24/24	70.00 - 72.00 70.64 - 71.00	Push thru vane Su=937/250 psf		52		Grey, wet, medium stiff, clayey SILT, little fine sand. 55x110 mm vane raw torque readings: V8 = 21.0/5.6 ft-lbs V9 = 12.5/3.0 ft-lbs	
71	V9		71.64 - 72.00	Su=558/134 psf		47			
72						47			
73						46			
74						41			

Remarks:

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Page 1 of 2
Boring No.: BB-YR34-102

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-102
PIN: 15111.00

Driller:	MaineDOT	Elevation (ft.)	10.2	Auger ID/OD:	4.5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/9/04-3/9/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	32+92.5, 9.1 Lt.	Casing ID/OD:	NW	Water Level*:	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	11D MV	24/14	75.00 - 77.00 75.20 - 75.20	1/1/8/13	9	44		Grey, wet, loose, fine SAND, some silt, little clay, trace medium sand, uniform. Attempt 55x110 mm vane: could not push	G#176650 A-4, SC-SM WC=20.3%
76						46			
77						48			
78						56			
79						53			
80	12D	24/24	80.00 - 82.00	WOR/WOR/7/11	7	87		Grey, wet, soft, silty CLAY with black staining from 80.0-81.0' bgs.	
81						76	-70.80	—81.00 Grey, wet, loose, silty fine SAND, uniform from 81.0-82.0' bgs.	
82						72			
83						71			
84						67			
85						81			
86						166			
87						138			
88						157			
89						184			
90	13D	24/24	90.00 - 92.00	11/7/11/8	18	90 bWA		Grey, wet, medium dense, fine silty SAND, uniform. bWashed Ahead of Casing to 98.1' bgs.	
91						81			
92						58			
93						53			
94						82			
95						103			
96						66			
97						64			
98						70			
99	14D	24/6	99.00 - 101.00	13/28/13/11	41	64		Grey, wet, dense, fine to coarse silty SAND.	
100						51			
101						86			
102						194		c214 blows for 0.8'.	
103						c214 dWA		dWashed Ahead to 105.5' bgs.	
104									
105	R1	46/46	105.50 - 109.30	RQD = 88%		NQ	-95.30	—105.50 Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. At 108.3 ft bgs (El. -98.1) change to dark grey/green quenched andesite porphyry. R1:Core Times (min:sec) 105.5' - 106.5' (5:05) 106.5' - 107.5' (7:30) 107.5' - 108.5' (7:20) 108.5' - 109.3' (8:20) Recovery=100% Core Blocked	
109							-99.10	—109.30 Bottom of Exploration at 109.30 feet below ground surface.	

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Page 2 of 2
Boring No.: BB-YR34-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1511(100)X

PIN 15111.00

BRIDGE PLANS

BRIDGE NO. 5848

	BY	DATE
PROJ. MANAGER	J. Wentworth	
DESIGN-DETAILED	K. MAGUIRE	
CHECKED-REVIEWED	T. WHITE	JAN 2009
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

STATION 34
TIDAL ESTUARY
YORK COUNTY

YORK

BORING LOGS

SHEET NUMBER
3
OF 3

APPENDIX A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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Definitions:
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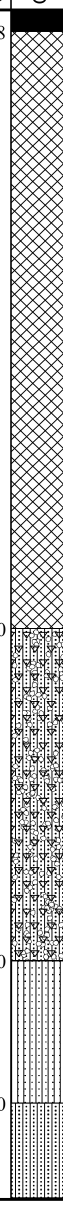
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.78	Pavement — 0.420	
5	1D	24/24	5.00 - 7.00	7/15/12/16	27			Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/6	10.00 - 12.00	4/3/4/4	7	18		Brown, wet, loose, fine to coarse SAND, little gravel and silt, (Fill).	
						19			
						27	-2.80	13.000	
						18			
						26			
15						2			
	3D	24/2	16.00 - 18.00	5/3/1/4	4	11		Grey, wet, very loose, fine to coarse silty SAND, trace gravel and sea shells.	
						10			
						12			
						12	-9.80	20.000	
20	4D	24/20	20.00 - 22.00	2/WOH/WOH/WOH	---	16		Grey, wet, very soft, SILT, some fine sand, little clay, trace gravel, medium to coarse sand and sea shells.	G#176641 A-4, CL-ML WC=36.8%
						11			
						19			
						30	-12.80	23.000	
						40			
25									

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D/MV	24/20	25.00 - 27.00	4/7/10/13	17	69			Olive, wet, very stiff, SILT, little fine sand, (Presumpscot upper crust). Attempt 55x110 mm vane: could not push	
						69				
						69				
						63	-17.80		28.000	
						69				
30	6D	24/23	30.00 - 32.00	3/2/3/3	5	78 aWA			Grey, wet, medium stiff, SILT, some clay, trace fine sand, mottled. aWashed ahead of casing.	G#176642 A-6, CL WC=31.7% LL=34 PL=20 PI=14
						70				
						64				
						64				
						50				
35	7D	24/22	35.00 - 37.00	2/2/1/WOH	3	52			Grey, wet, very loose, fine silty SAND, uniform, dilatent.	
						52				
						50				
						38				
						32				
40	8D V1	24/24	40.00 - 42.00 40.64 - 41.00	Push thru vane Su=259/125 psf		62			Grey, wet, soft, clayey SILT, trace fine sand layers, black staining. 55x110 mm vane raw torque readings: V1 = 5.8/2.8 ft-lbs V2 = 10.7/1.5 ft-lbs	G#176643 A-4, CL WC=36.8% LL=28 PL=19 PI=9
	V2		41.64 - 42.00	Su=478/67 psf		45				
						44				
						48				
						50				
45	9D V3	24/24	45.00 - 47.00 45.64 - 46.00	Push thru vane Su=192/67 psf		87			Similar to above, very soft. 55x110 mm vane raw torque readings: V3 = 4.3/1.5 ft-lbs V4 = 5.3/1.6 ft-lbs	
	V4		46.64 - 47.00	Su=237/71 psf		68				
						60				
						40				
50						47				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/24	50.00 - 52.00	Push thru vane		73			Grey, wet, soft to medium stiff, clayey SILT with occasional fine sand layers, black staining. 55x110 mm vane raw torque readings: V5 = 9.8/1.8 ft-lbs V6 = 13.5/1.7 ft-lbs	
	V5		50.64 - 51.00	Su=437/80 psf						
	V6		51.64 - 52.00	Su=603/76 psf		84				
						85				
						69				
						69				
55	MD	24/0	55.00 - 57.00	Push thru vane		94			No recovery, similar soils on spoon, medium stiff. 55x110 mm vane raw torque readings: V7 = 12.2/1.7 ft-lbs V8 = 12.8/1.3 ft-lbs	
	V7		55.64 - 56.00	Su=545/76 psf						
	V8		56.64 - 57.00	Su=571/58 psf		57				
						47				
						34				
						31				
60	11D	24/24	60.00 - 62.00	Push thru vane		49			Grey, wet, soft to medium stiff, SILT, some clay, with little fine sand layers, black staining. 55x110 mm vane raw torque readings: V9 = 11.6/2.0 ft-lbs V10 = 11.0/2.1 ft-lbs	G#176644 A-4, CL-ML WC=28.0% LL=23 PL=17 PI=6
	V9		60.64 - 61.00	Su=518/89 psf						
	V10		61.64 - 62.00	Su=491/94 psf		40				
						41				
						38				
						32				
65	12D	24/24	65.00 - 67.00	Push thru vane		52			Similar to above, medium stiff. 55x110 mm vane raw torque readings: V11 = 13.5/3.2 ft-lbs V12 = 13.9/3.7 ft-lbs	
	V11		65.64 - 66.00	Su=603/143 psf						
	V12		66.64 - 67.00	Su=621/165 psf		41				
						46				
						40				
						50				
70	13D	24/24	70.00 - 72.00	Push thru vane		74			Grey, wet, medium stiff, clayey SILT, little fine sand with fine sand layers. 55x110 mm vane raw torque readings: V13 = 17.1/2.0 ft-lbs V14 = 14.4/4.8 ft-lbs	
	V13		70.64 - 71.00	Su=763/89 psf						
	V14		71.64 - 72.00	Su=643/214 psf		61				
						61				
						58				
75						53				

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Station 34 Bridge over Tidal Estuary **Location:** York, Maine	**Boring No.:** BB-YR34-101 **PIN:** 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	14D	24/3	75.00 - 77.00	WOR/4/5/6	9	64			Similar to above, medium stiff.	
	V15		75.64 - 76.00	Su=808/304 psf					55x110 mm vane raw torque readings: V15 = 18.1/6.8 ft-lbs	
	MV		76.20 - 76.20			55			MV = could not push	
						62				
						73				
						128				
80	15D	24/20	80.00 - 82.00	28/58/54/24	112	78			Grey, wet, very stiff, SILT, some fine sand, trace clay.	G#176645 A-2-4, SM WC=21.5%
						bWA			bWashed Ahead of Casing.	
						52				
						39				
						25				
						26	-74.80		85.000	
85	16D	24/24	85.00 - 87.00	WOR/8/11/10	19	62			Grey, wet, medium dense, fine silty SAND, uniform.	
						66				
						55				
						55				
						98				
90	17D	24/6	90.00 - 92.00	18/4/14/9	18	115			Grey, wet, medium dense, fine to coarse silty SAND, little gravel.	G#176646 A-4, CL-ML WC=18.2%
						79				
						82				
						93			c156 blows for 0.4'.	
	R1	60/60	94.80 - 99.80	RQD = 31%		c156	-84.60		94.800	
95						NQ			Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. R1:Core Times (min:sec) 94.8' - 95.8' (5:05) 95.8' - 96.8' (5:50) 96.8' - 97.8' (6:10) 97.8' - 98.8' (6:05) 98.8' - 99.8' (5:15) Recovery=100%	
100							-89.60		99.800	

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Station 34 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR34-101
PIN: 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/04-3/3/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+03.4, 6.8 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100									**Bottom of Exploration at 99.80 feet below ground surface.**	
105										
110										
115										
120										
125										

Remarks:

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Boring No.: BB-YR34-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Station 34 Bridge over Tidal Estuary **Location:** York, Maine	**Boring No.:** BB-YR34-102 **PIN:** 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.62		Pavement ——0.580	
5	1D	24/4	5.00 - 7.00	6/6/17/22	23				Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
10	2D	24/9	10.00 - 12.00	2/6/11/7	17				Brown, wet, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	
15	3D	24/17	15.00 - 17.00	6/8/5/7	13	24	-4.80		——15.000 Grey, wet, medium dense, fine silty SAND with sea shells.	
						14				
						18				
						17				
						17				
20	4D	24/16	20.00 - 22.00	2/2/2/2	4	37	-9.80		——20.000 Grey, wet, soft, fine sandy SILT, little clay, trace gravel and medium to coarse sand with sea shells.	G#176647 A-4, CL-ML WC=31.6%
						24				
						23				
						23				
						24				
25										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-102
Location:	York, Maine			**PIN:**	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/24	25.00 - 27.00	1/WOH/WOH/2	---	32			Grey, wet, soft, SILT, little clay, trace fine sand.	G#176648 A-4, ML WC=45.5% LL=37 PL=33 PI=4
						31				
	V1		27.64 - 28.00	aSu=1964/- psf		36	-16.80		27.000 55x110 mm vane raw torque readings: V1 = 44/-- ft-lbs aVane reached maximum torque reading without shearing, no remolded was attempted.	
						88				
						126				
30	6D	24/14	30.00 - 32.00	8/4/7/9	11	45			Olive, wet, stiff, SILT, (Presumpscot upper crust).	
						44				
						38	-21.80		32.000	
						39				
						34				
35						34				
						38				
						38				
						32				
						30				
40	7D V2	24/24	40.00 - 42.00 40.64 - 41.00	Push thru vane Su=446/67 psf		62			Grey, wet, soft, clayey SILT, trace fine sand, black staining. 55x110 mm vane raw torque readings: V2 = 10.0/1.5 ft-lbs V3 = 8.8/1.1 ft-lbs	
	V3		41.64 - 42.00	Su=393/49 psf		50				
						37				
						34				
						30				
45						34				
						30				
						32				
						31				
						30				
50										

Remarks:

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Boring No.: BB-YR34-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-102
Location:	York, Maine			**PIN:**	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
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T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	8D	24/24	50.00 - 52.00	Push thru vane		54			Grey, wet, soft, silty CLAY with trace fine sand layers and black staining.	G#176649
	V4		50.64 - 51.00	Su=540/71 psf					55x110 mm vane raw torque readings:	A-6, CL
	V5		51.64 - 52.00	Su=330/54 psf		41			V4 = 12.1/1.6 ft-lbs	WC=40.0%
									V5 = 7.4/1.2 ft-lbs	LL=32
						39				PL=20
						35				PI=12
						33				
55						37				
						35				
						33				
						33				
						34				
60	9D	24/24	60.00 - 62.00	Push thru vane		73			Similar to above, soft to medium stiff.	
	V6		60.64 - 61.00	Su=241/112 psf					55x110 mm vane raw torque readings:	
	V7		61.64 - 62.00	Su=536/98 psf		56			V6 = 5.4/2.5 ft-lbs	
									V7 = 12.0/2.2 ft-lbs	
						53				
						45				
						42				
65						48				
						40				
						45				
						38				
						34				
70	10D	24/24	70.00 - 72.00	Push thru vane		52			Grey, wet, medium stiff, clayey SILT, little fine sand.	
	V8		70.64 - 71.00	Su=937/250 psf					55x110 mm vane raw torque readings:	
	V9		71.64 - 72.00	Su=558/134 psf		47			V8 = 21.0/5.6 ft-lbs	
									V9 = 12.5/3.0 ft-lbs	
						47				
						46				
						41				
75										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Station 34 Bridge over Tidal Estuary			**Boring No.:**	BB-YR34-102
Location:	York, Maine			**PIN:**	15111.00
Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	11D MV	24/14	75.00 - 77.00 75.20 - 75.20	1/1/8/13	9	44			Grey, wet, loose, fine SAND, some silt, little clay, trace medium sand, uniform. Attempt 55x110 mm vane: could not push	G#176650 A-4, SC-SM WC=20.3%
						46				
						48				
						56				
						53				
80	12D	24/24	80.00 - 82.00	WOR/WOR/7/11	7	87			Grey, wet, soft, silty CLAY with black staining from 80.0-81.0' bgs.	
						76	-70.80		81.000 Grey, wet, loose, silty fine SAND, uniform from 81.0-82.0' bgs.	
						72				
						71				
						67				
85						81				
						166				
						138				
						157				
						184				
90	13D	24/24	90.00 - 92.00	11/7/11/8	18	90 bWA			Grey, wet, medium dense, fine silty SAND, uniform. bWashed Ahead of Casing to 98.1' bgs.	
						81				
						58				
						53				
						82				
95						103				
						66				
						64				
						70				
100	14D	24/6	99.00 - 101.00	13/28/13/11	41	64			Grey, wet, dense, fine to coarse silty SAND.	

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:** Station 34 Bridge over Tidal Estuary **Location:** York, Maine	**Boring No.:** BB-YR34-102 **PIN:** 15111.00

Driller:	MaineDOT	**Elevation (ft.)**	10.2	**Auger ID/OD:**	4.5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/04-3/9/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	32+92.5, 9.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100						51				
						86				
						194				
						c214			c214 blows for 0.8'.	
						dWA			dWashed Ahead to 105.5' bgs.	
105	R1	46/46	105.50 - 109.33	RQD = 88%		NQ	-95.30		105.500 Bedrock: Grey, medium grained, diorite with prominent orthoclase crystals. At 108.3 ft bgs (El. -98.1) change to dark grey/green quenched andesite porphyry. R1:Core Times (min:sec) 105.5' - 106.5' (5:05) 106.5' - 107.5' (7:30) 107.5' - 108.5' (7:20) 108.5' - 109.33' (8:20) Recovery=100%	
							-99.10		Core Blocked 109.300 **Bottom of Exploration at 109.30 feet below ground surface.**	
110										
115										
120										
125										

Remarks:

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

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

APPENDIX B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): York Project Number: 15111.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-YR34-101, 4D	32+03.4	6.8 Rt.	20.0-22.0	176641	1	36.8			CL-ML	A-4	IV
BB-YR34-101, 6D	32+03.4	6.8 Rt.	30.0-32.0	176642	1	31.7	34	14	CL	A-6	III
BB-YR34-101,8D	32+03.4	6.8 Rt.	40.0-42.0	176643	1	36.8	28	9	CL	A-4	IV
BB-YR34-101, 11D	32+03.4	6.8 Rt.	60.0-62.0	176644	1	28.0	23	6	CL-ML	A-4	IV
BB-YR34-101, 15D	32+03.4	6.8 Rt.	80.0-82.0	176645	1	21.5			SM	A-2-4	II
BB-YR34-101, 17D	32+03.4	6.8 Rt.	90.0-92.0	176646	1	18.2			CL-ML	A-4	IV
BB-YR34-102, 4D	32+92.5	9.1 Lt.	20.0-22.0	176647	2	31.6			CL-ML	A-4	IV
BB-YR34-102, 5D	32+92.5	9.1 Lt.	25.0-27.0	176648	2	45.4	37	4	ML	A-4	IV
BB-YR34-102, 8D	32+92.5	9.1 Lt.	50.0-52.0	176649	2	40.0	32	12	CL	A-6	III
BB-YR34-102, 11D	32+92.5	9.1 Lt.	75.0-77.0	176650	2	20.3			SC-SM	A-4	III

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

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

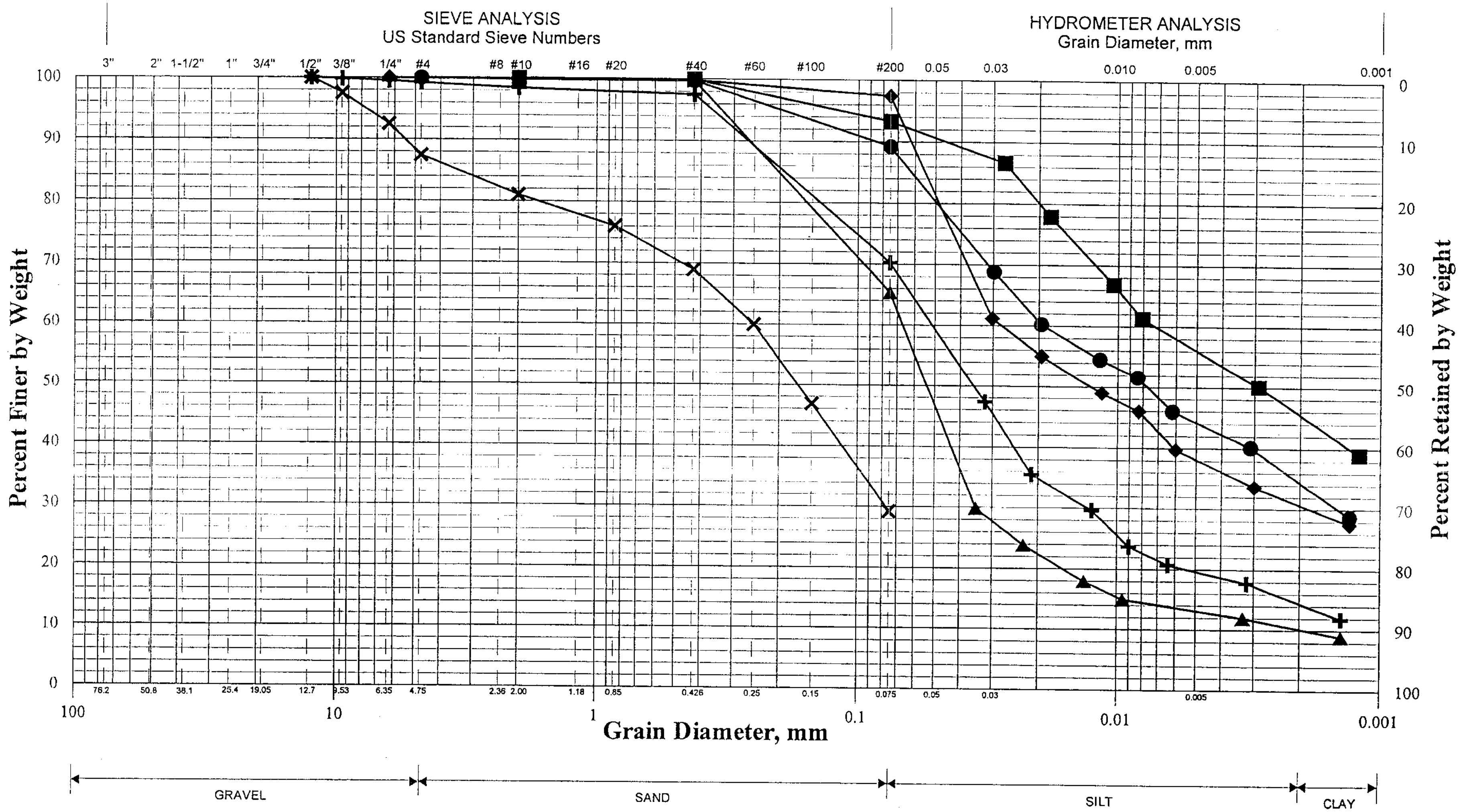
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

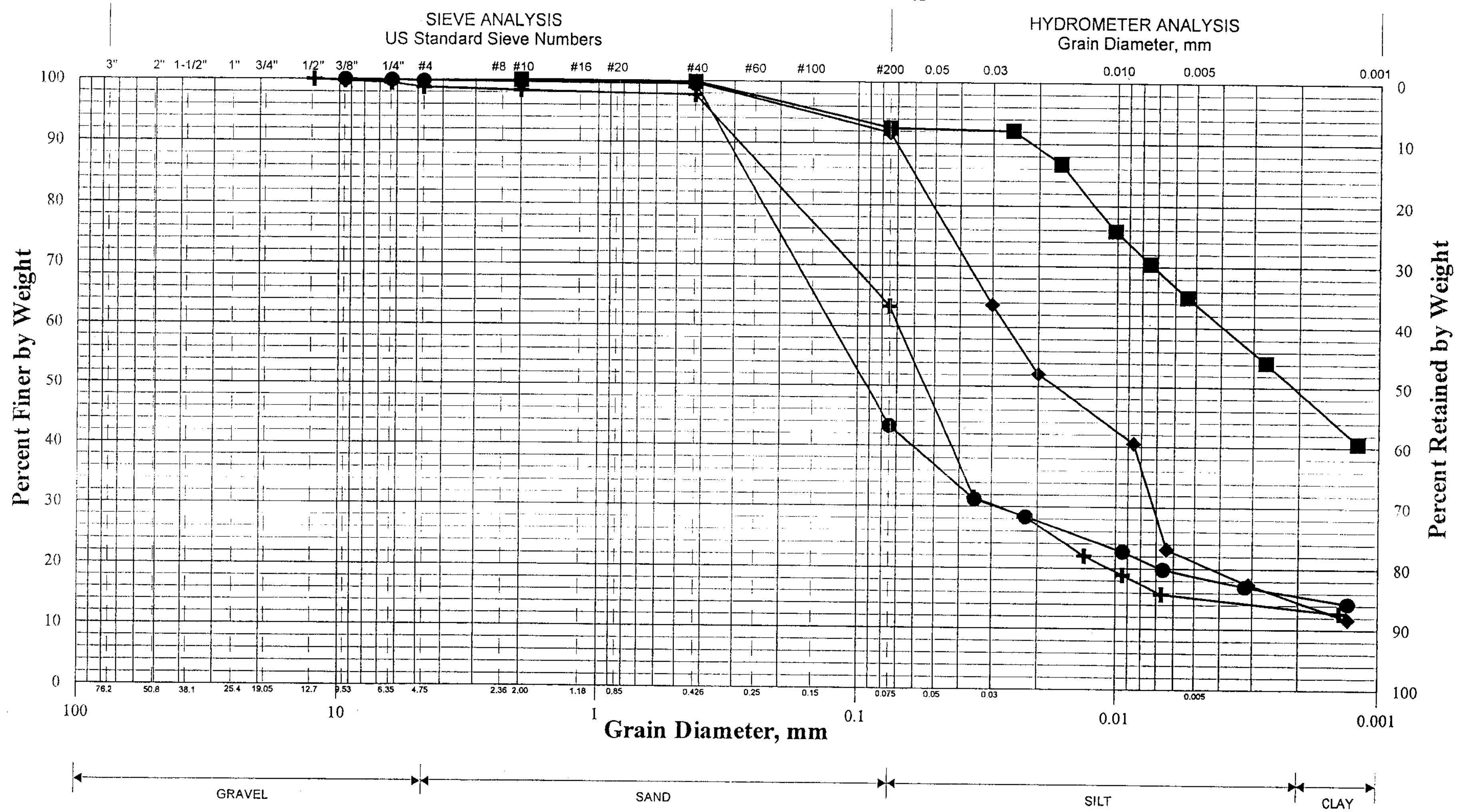
	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR34-101	4D	20.0-22.0	SILT, some fine sand, little clay, trace gravel and medium to coarse sand.	36.8			
◆	BB-YR34-101	6D	30.0-32.0	SILT, some clay, trace fine sand.	31.7	34	20	14
■	BB-YR34-101	8D	40.0-42.0	Clayey SILT, trace fine sand.	36.8	28	19	9
●	BB-YR34-101	11D	60.0-62.0	SILT, some clay, little sand.	28.0	23	17	6
▲	BB-YR34-101	15D	80.0-82.0	SILT, some fine sand, trace clay.	21.5			
×	BB-YR34-101	17D	90.0-92.0	Fine to coarse silty SAND, little gravel.	18.2			

PIN: 15111.00
Town: York
Reported by: T. White
Date: 3/26/04

SHEET NO. 1

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

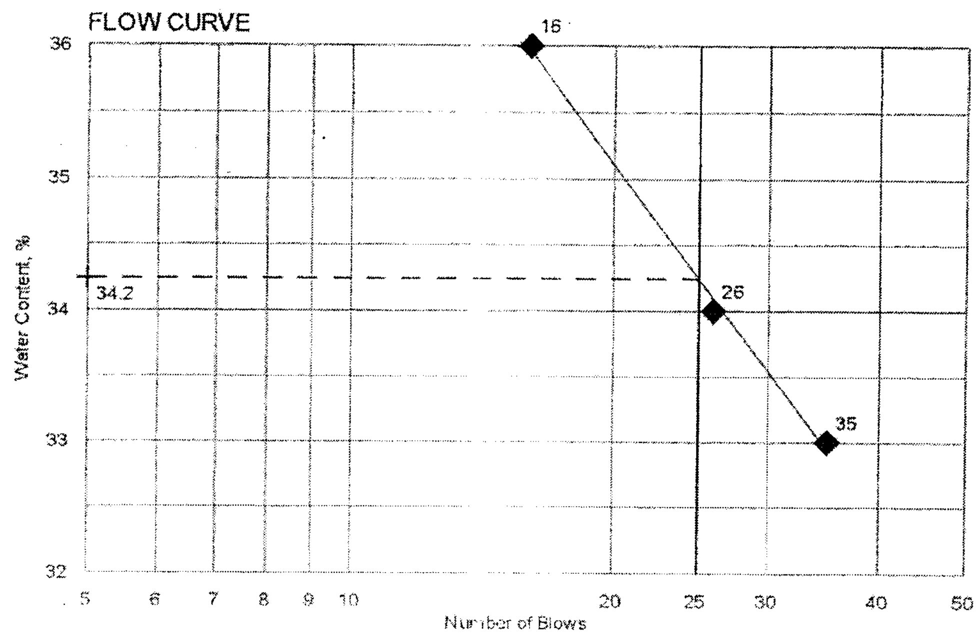
	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR34-102	4D	20.0-22.0	Fine sandy SILT, little clay, trace gravel and medium to coarse sand.	31.6			
◆	BB-YR34-102	5D	25.0-27.0	SILT, little clay, trace fine sand.	45.5	37	33	4
■	BB-YR34-102	8D	50.0-52.0	Silty CLAY, trace fine sand.	40.0	32	20	12
●	BB-YR34-102	11D	75.0-77.0	Fine SAND, some silt, little clay, trace medium sand.	20.3			
▲	---							
✕	---							

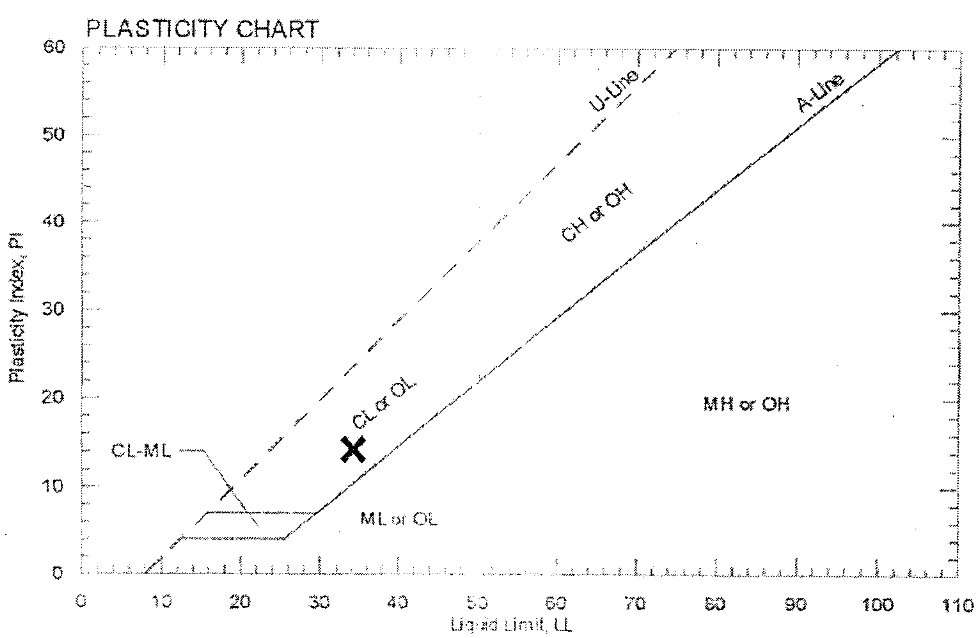
PIN: 15111.00
Town: York
Reported by: T. White
Date: 3/26/04

SHEET NO. 2

State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

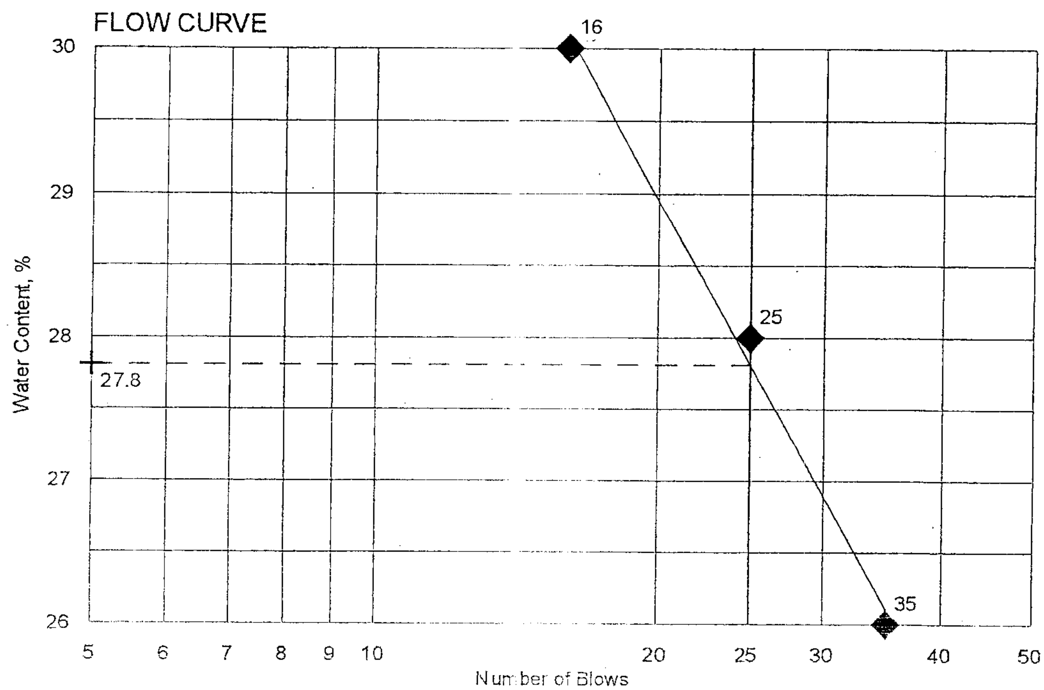
TOWN	York	Reference No.	176642
PIN	15111.00	Natural water content (%)	31.7
Date	4/21/2004	Plastic limit	20
Boring No.	BB-YR34-101	Liquid limit	34
Station	32+03.4, 6.8' RT	Plasticity index	14
Depth/Sample No.	30-32'/6D	Reported by	KLD

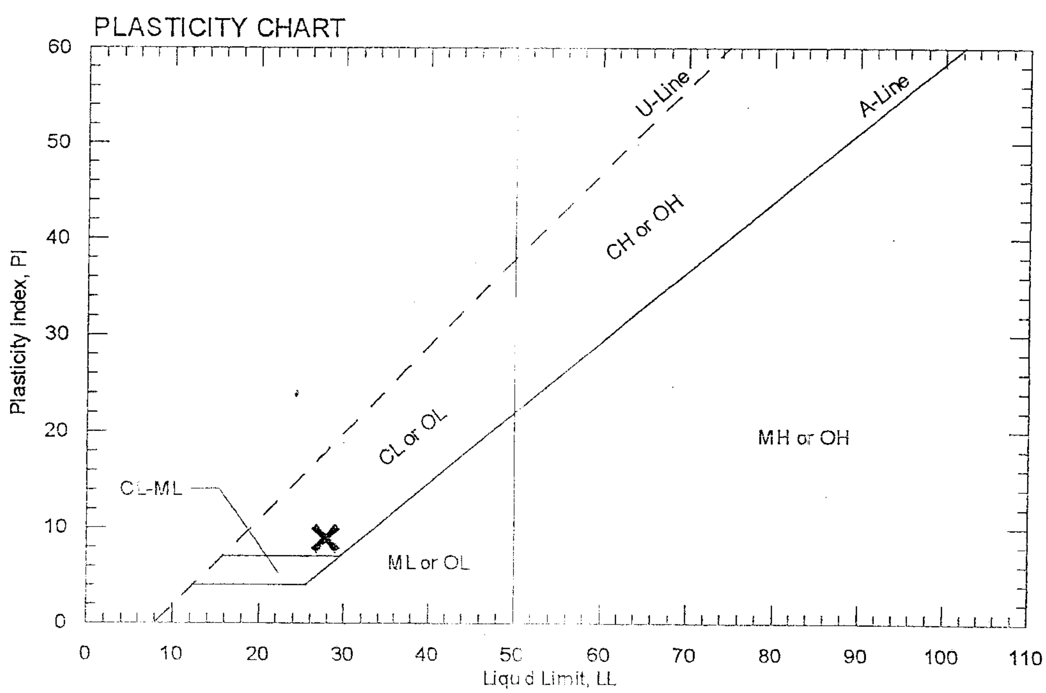




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

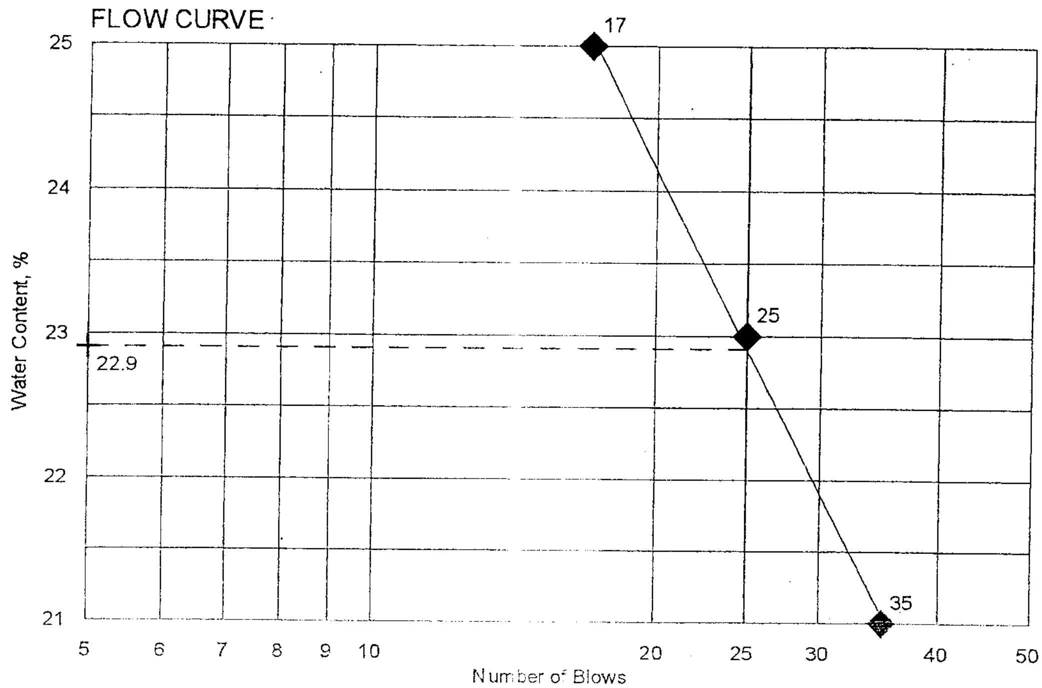
TOWN	YORK	Reference No.	176643
PIN	15111.00	Natural water content (%)	36.8
Date	4/23/2004	Plastic limit	19
Boring No.	BB-YR34-101	Liquid limit	28
Station	32+03.4, 6.8' RT	Plasticity index	9
Depth/Sample No.	40.0-42.0/8D	Reported by	B. D. FOGG

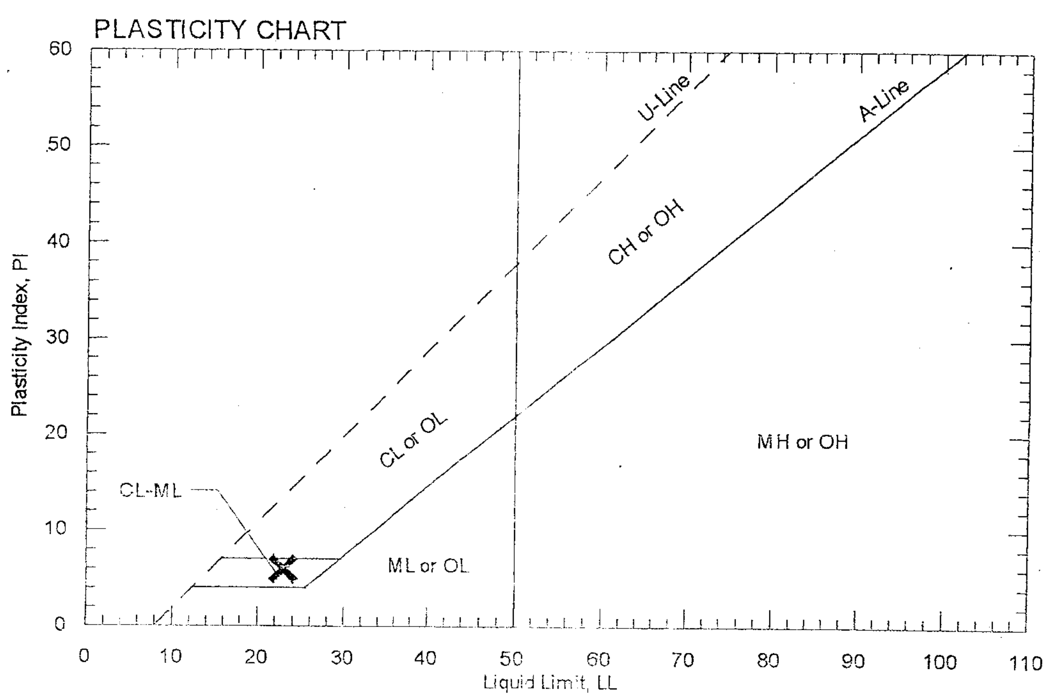




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	YORK	Reference No.	176644
PIN	15111.00	Natural water content (%)	28.0
Date	4/23/2004	Plastic limit	17
Boring No.	BB-YR34-101	Liquid limit	23
Station	32 + 034, 6.8' RT	Plasticity index	6
Depth/Sample No.	60.0-62.0/11D	Reported by	B. D. FOGG

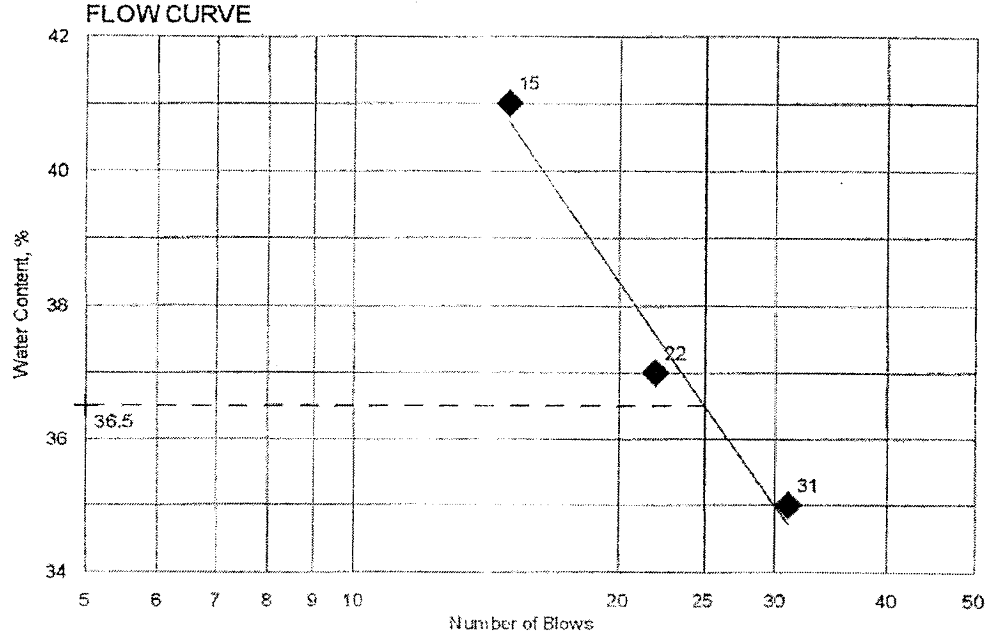




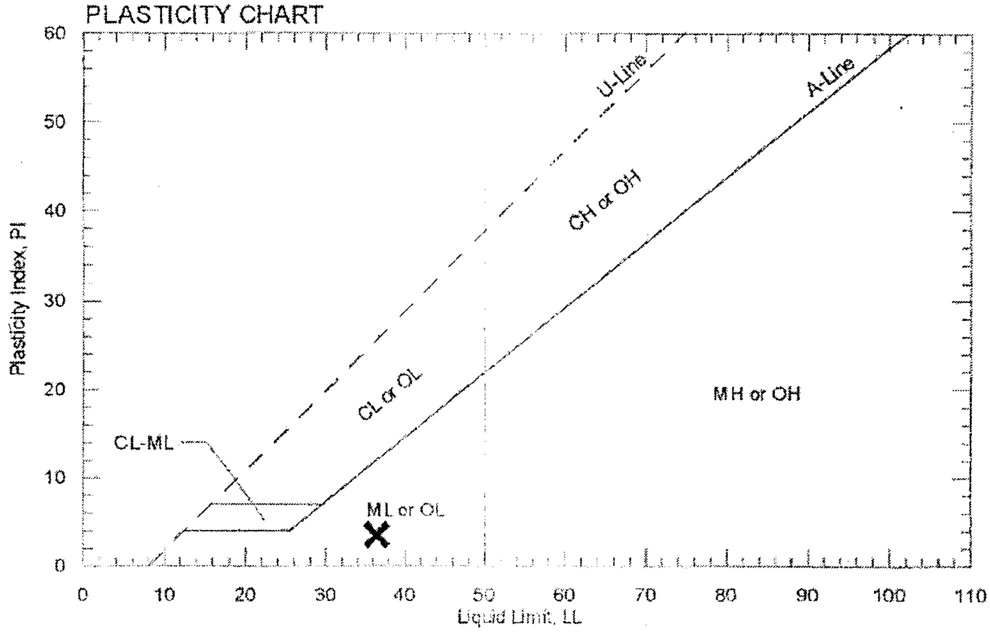
State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	York	Reference No.	176648
PIN	15111.00	Natural water content (%)	45.5
Date	4/21/2004	Plastic limit	33
Boring No.	BB-YR34-102	Liquid limit	36
Station	32+92.5, 9.1' LT	Plasticity index	3
Depth/Sample No.	25-27'/5D	Reported by	KLD

FLOW CURVE

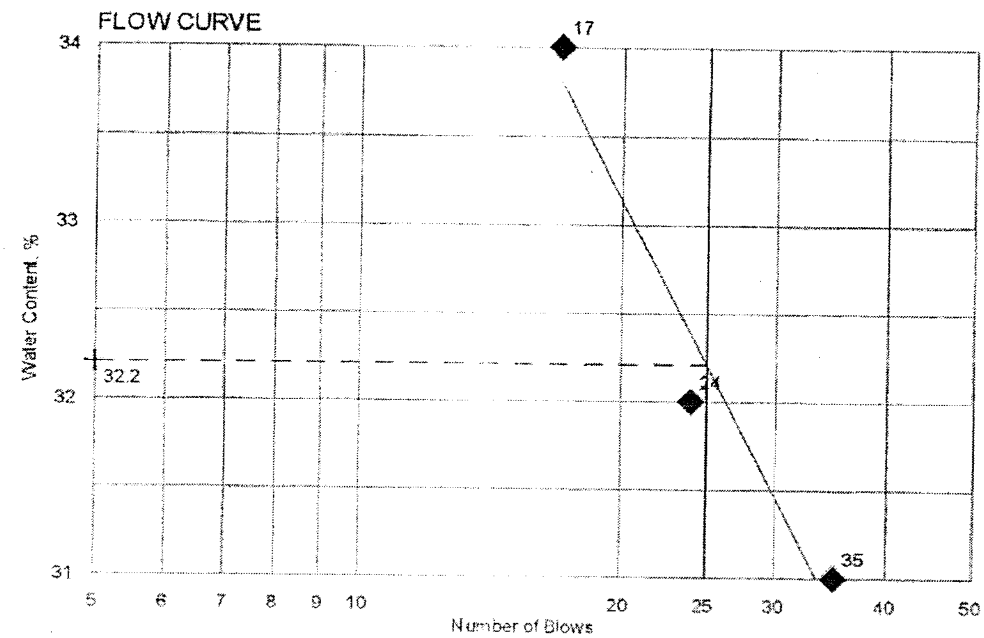


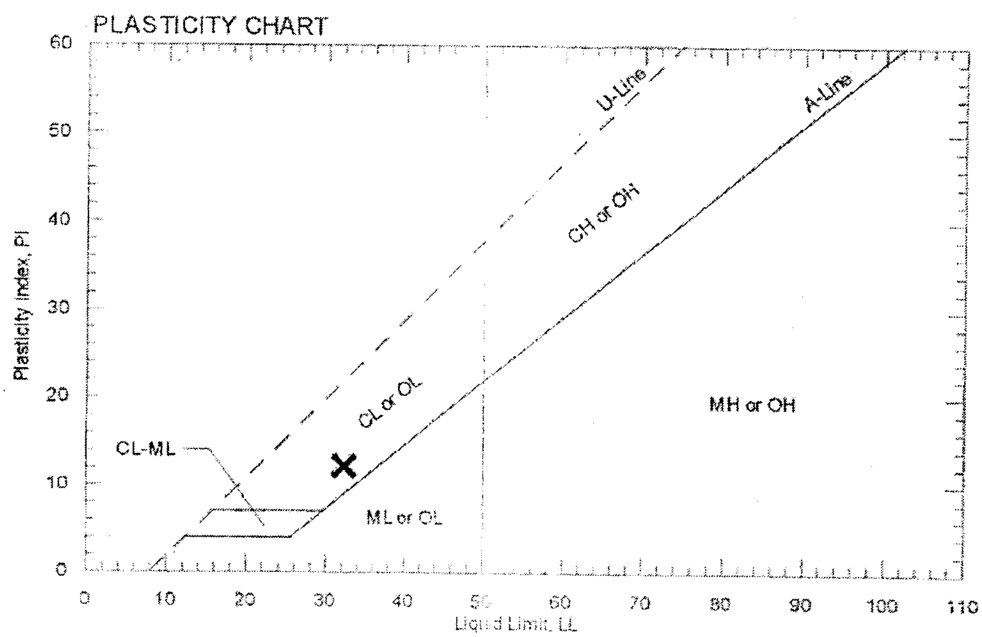
PLASTICITY CHART



State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	York	Reference No.	176649
PIN	15111.00	Natural water content (%)	40.0
Date	4/21/2004	Plastic limit	20
Boring No.	BB-YR34-102	Liquid limit	32
Station	32+92.5, 9.1' LT	Plasticity index	12
Depth/Sample No.	50-52/8D	Reported by	KLD





State of Maine

Department of Transportation
Bridge Program - Geotechnical
16 State House Station
Augusta, Maine 04333-0016

Kate Maguire, PE
Phone: 624-3415
FAX: 624-3491
email: kate.maguire@maine.gov

Addendum #1

To: File
cc: TEDOCS
From: Kate Maguire, PE
Date: January 14, 2009
Re: Soils Report No. 2004-23
Geotechnical Design Report
For the Replacement of
Station 44 Bridge
Over Tidal Estuary
York, Maine
PIN: 15112.00

==

The following changes are made to the Geotechnical Design Report for the Replacement of Station 44 Bridge Over Tidal Estuary York, Maine Soils Report No. 2004-23:

1. Throughout the document, replace Project Identification Number (PIN) 11067.00 with PIN 15112.00.

2. Throughout the document, replace Fed No. AC-BH-1106(700)X with Fed No. BH-1511(200)X.

3. Replace the third paragraph of Section 7.2 on page 7 of the report text with the following text which corrects the Soil Type reference given:

The Designer may assume Soil Type 4 (BDG Section 3.6.1) for retaining wall backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf, and a soil-concrete friction coefficient of 0.45.

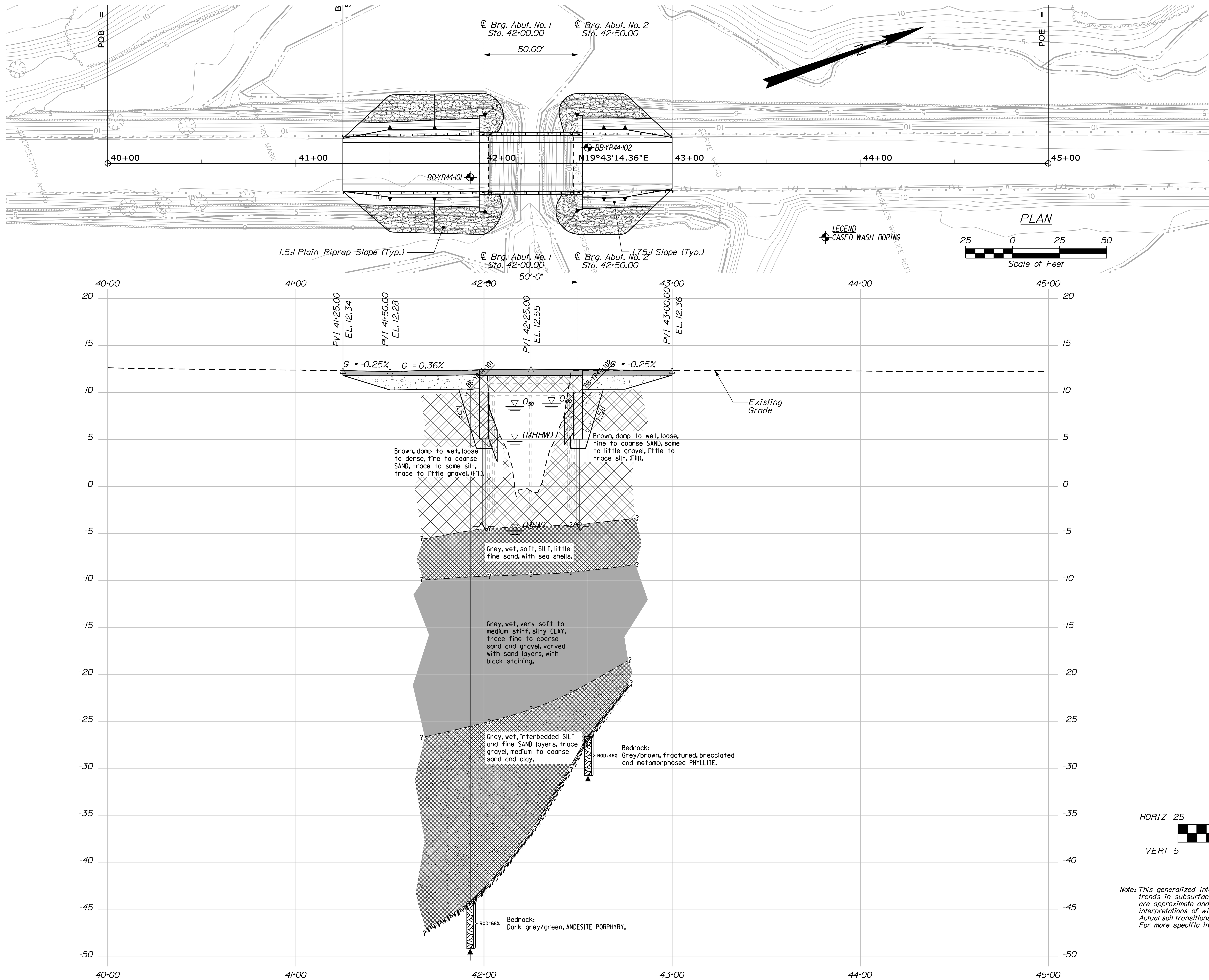
5. Replace Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile with the attached Sheet 2 which has been updated with current station information.

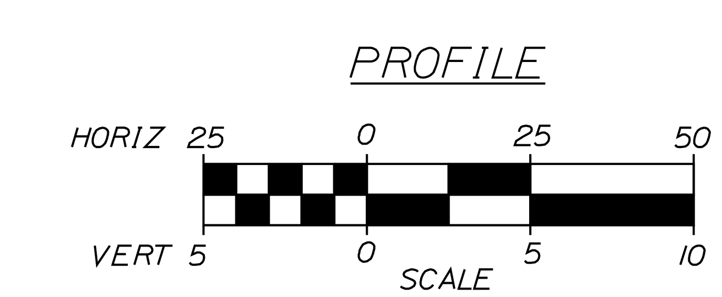
6. Replace Sheet 3 - Boring Logs with the attached Sheet 3 which has been updated with current station information.

7. Replace Appendix A - Boring Logs with the attached pages which have been updated with current station information.

8. Replace Appendix B – Laboratory Data with the attached pages which have been updated with current station information.

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:1/12/2009





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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1511(200)X

PIN 15112.00

BRIDGE NO. 5849

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	J. Wentworth	
DESIGN-DETAILED	K. MAGUIRE	JAN 2009
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

STATION 44 BRIDGE
TIDAL ESTUARY
YORK COUNTY
YORK

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\MSTA\009_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:1/12/2009

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Station 44 Bridge over Tidal Estuary	Location:	York, Maine	Boring No.:	BB-YR44-101
				PIN:	15112.00
Driller:	MaineDOT	Elevation (ft.)	10.4	Auger ID/OD:	4.5" SSA
Operator:	G. Lidstone	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/1/04-3/1/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	41+92.7, 7.4 Rt.	Casing ID/OD:	HW	Water Level*:	10.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98	Pavement 0.42	
	1D-A	24/20	2.00 - 4.00	16/31/12/9	43			(1D-A) (2.0-3.0) Brown, damp, dense, fine to coarse SAND, little silt, trace gravel (Fill). (1D/B) (3.0-4.0) Brown, damp, dense, fine to coarse SAND, some silt, little gravel (Fill).	
	1D-B								
5	2D	24/16	5.00 - 7.00	3/3/8/15	11			Brown, damp, medium dense, fine to coarse SAND, some silt, little gravel, old pavement, (Fill).	
10	3D	24/17	10.00 - 12.00	5/5/3/7	8	25			
						36	-0.60	11.00 Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	
						33			
						30			
						28			
15	4D	24/16	15.00 - 17.00	4/2/2/4	4	15	-4.60	15.00 Grey, wet, soft, fine sandy SILT with sea shells.	
						15			
						28			
						29			
						19			
20	5D	24/20	20.00 - 22.00	1/1/WOH/1	1	35	-9.60	20.00 Grey, wet, very soft, silty CLAY, trace fine sand and fine sand layers, trace gravel. Stiffer at top of sample, softer with depth.	G#176633 A-6, CL WC=37.8%
						35			
	MV		22.40 - 22.40			28		55x110 mm vane raw torque readings: MV= could not push.	
						22			
						19			
25	6D	24/16	25.00 - 27.00	7/1/WOH/1	1	40		Grey, wet, very soft, silty CLAY, varved with sand lenses, trace gravel, trace coarse sand.	G#176634 A-6, CL WC=38.3%
						27			
	MV		27.00 - 27.00			23		55x110 mm vane raw torque readings: MV= could not push.	
						21			
						19			
30	7D	24/10	30.00 - 32.00	Push thru vane		42		Grey, wet, soft, silty CLAY, trace fine sand.	G#176635 A-6, CL WC=47.2% LL=33 PL=19 PI=14
	V1		31.00 - 31.36	Su=402/49 psf		33		55x110 mm vane raw torque readings: V1= 9.0/1.1 ft-lbs MV = could not push	
	MV		32.00 - 32.00			29			
						31			
						36			
35	8D	24/3	35.00 - 37.00	1/5/8/6	13	55	-25.10	35.50 Grey, wet, medium dense, fine silty SAND, trace coarse sand. 55x110 mm vane raw torque readings: MV= could not push.	
	MV		36.00 - 36.00			38			
						27			
						23			
						38			
40	9D	24/6	40.00 - 42.00	3/WOH/3/7	3	48	-29.60	40.00 Grey, wet, soft, SILT, little gravel, sand and clay, dilatent.	G#176636 A-4, CL-ML WC=24.2%
						36			
						28			
						48			
						124	-33.60	44.00	
45	10D	24/6	45.00 - 47.00	22/10/12/11	22	93		Grey, wet, medium dense, fine to coarse SAND, some gravel and silt, trace clay.	G#176637 A-2-4, SC-SM WC=10.2%
						79			
						116			
						132			
						260			
50	MD	2/0	50.00 - 50.17	60(0.2")	---	a WR		Wash material in spoon a Washed ahead and Roller coned ahead to 54.5' bgs through very dense grey till.	
	R1	60/52	54.50 - 59.50	RQD = 68%		NQ	-44.10	54.50 Bedrock: Dark grey/green, fine grained, ANDESITE porphyry. R1:Core Times (min:sec) 54.5' - 55.5' (18:20) 55.5' - 56.5' (10:00) 56.5' - 57.5' (10:10) 57.5' - 58.5' (9:10) 58.5' - 59.5' (9:05) Recovery=97%	
55						Core			
							-49.10	59.50 Bottom of Exploration at 59.50 feet below ground surface.	
60									
65									
70									
75									

Remarks:

MV = Unsuccessful vane shear test attempt.

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

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

Page 1 of 1

Boring No.: BB-YR44-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Station 44 Bridge over Tidal Estuary	Location:	York, Maine	Boring No.:	BB-YR44-102
				PIN:	15112.00
Driller:	MaineDOT	Elevation (ft.)	10.4	Auger ID/OD:	4.5" SSA
Operator:	G. Lidstone	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/1/04-3/1/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	42+55.3, 8.4 Lt.	Casing ID/OD:	HW	Water Level*:	10.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98	Pavement 0.42 Brown, damp, fine to coarse SAND, little gravel, trace silt, (Fill).	
5	1D	24/10	5.00 - 7.00	4/4/5/6	9			Brown, damp, loose, fine to coarse SAND, some gravel, little silt, (Fill).	
10	2D	24/2	10.00 - 12.00	6/3/4/11	7	46		Brown, wet, loose, fine silty SAND, little gravel, trace medium to coarse sand, (Fill).	
						27			
						208		Obstruction at 13.0' bgs. Roller coned through.	
						191			
						41	-3.60	14.00	
15	3D	24/16	15.00 - 17.00	2/2/2/1	4	34		Grey, wet, soft, sandy SILT with broken shells.	
						30			
						27			
						73			
						43	-8.60	19.00	
20	4D/MV	24/18	20.00 - 22.00	5/3/4/7	7	45		Grey, wet, medium stiff, Silty CLAY, trace gravel and sand, mottled. 55x110 mm and 25.4x50.8 mm vane raw torque readings: MV = could not push	G#176638 A-6, CL WC=31.3% LL=34 PL=20 PI=14
						43			
						41			
						34			
						34			
25	5D	24/24	25.00 - 27.00	Push thru vane.	---	46		Grey, wet, soft, silty CLAY with black staining, occasional black fine to coarse silty sand layers, trace gravel. 55x110 mm vane raw torque readings: V1 = 10.0/2.4 ft-lbs V2 = 11.8/2.6 ft-lbs	G#176639 A-6, CL WC=42.8%
	V1		26.00 - 26.36	Su=446/107 psf		29			
	V2		27.00 - 27.36	Su=527/116 psf		32			
						30			
						33			
30	6D	24/18	30.00 - 32.00	WOH/WOH/11/12	11	59 WA		Washed ahead of Casing.	G#176640 A-4, CL-ML WC=28.3% LL=19 PL=15 PI=4
	V3		31.00 - 31.36	Su=424/125 psf		47	-20.60	31.00 Grey, wet, stiff, clayey SILT, trace gravel and sand. 55x110 mm vane raw torque readings: V3 = 9.5/2.8 ft-lbs MV = could not push Wash water changed to brown ± 33.0' bgs.	
	MV		32.00 - 32.00			54			
						126			
								Grey, wet, dense, fine to coarse silty SAND, some gravel.	
35	7D	1/1	35.00 - 35.08	80(0.1")	---				
	8D R1	1/0 50/50	36.00 - 36.08 36.90 - 41.07	60(0.1") RQD = 46%	---	NQ	-26.50	36.90 Bedrock: Grey/brown, fine grained, fractured, brecciated and metamorphosed PHYLLITE. R1:Core Times (min:sec) 36.9' - 37.9' (6:00) 37.9' - 38.9' (6:30) 38.9' - 39.9' (12:15) 39.9' - 40.9' (3:35) 40.9' - 41.07' (1:51) Recovery=100%	
						Core			
40									
							-30.70	41.07 Bottom of Exploration at 41.07 feet below ground surface.	
45									
50									

Remarks:

MV = Unsuccessful vane shear test attempt.

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

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

Page 1 of 1

Boring No.: BB-YR44-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1511(200)X

BRIDGE NO. 5849 | PIN 15112.00 | BRIDGE PLANS

PROJ. MANAGER	J Wentworth	BY	DATE
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	MAR2004
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

STATION 44
TIDAL ESTUARY
YORK VILLAGE YORK COUNTY

BORING LOGS

SHEET NUMBER

3

OF 3

APPENDIX A

Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Station 44 Bridge over Tidal Estuary **Location:** York, Maine	**Boring No.:** BB-YR44-101 **PIN:** 15112.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	41+92.7, 7.4 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98		Pavement 0.420	
	1D-A	24/20	2.00 - 4.00	16/31/12/9	43				(1D-A) (2.0-3.0) Brown, damp, dense, fine to coarse SAND, little silt, trace gravel (Fill).	
	1D-B								(1D/B) (3.0-4.0) Brown, damp, dense, fine to coarse SAND, some silt, little gravel (Fill).	
5	2D	24/16	5.00 - 7.00	3/3/8/15	11				Brown, damp, medium dense, fine to coarse SAND, some silt, little gravel, old pavement, (Fill).	
10	3D	24/17	10.00 - 12.00	5/5/3/7	8	25				
						36	-0.60		11.000 Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	
						33				
						30				
						28				
15	4D	24/16	15.00 - 17.00	4/2/2/4	4	15	-4.60		15.000 Grey, wet, soft, fine sandy SILT with sea shells.	
						15				
						28				
						29				
						19				
20	5D	24/20	20.00 - 22.00	1/1/WOH/1	1	35	-9.60		20.000 Grey, wet, very soft, silty CLAY, trace fine sand and fine sand layers, trace gravel. Stiffer at top of sample, softer with depth.	G#176633 A-6, CL WC=37.8%
						35				
	MV		22.40 - 22.40			28			55x110 mm vane raw torque readings: MV= could not push.	
						22				
25						19				

Remarks:

MV = Unsuccessful vane shear test attempt.

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: Station 44 Bridge over Tidal Estuary Location: York, Maine	Boring No.: BB-YR44-101 PIN: 15112.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	41+92.7, 7.4 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.5' (Tidal)

Definitions:	Definitions:	Definitions:
D = Split Spoon Sample	S_u = Insitu Field Vane Shear Strength (psf)	WC = water content, percent
MD = Unsuccessful Split Spoon Sample attempt	T_v = Pocket Torvane Shear Strength (psf)	LL = Liquid Limit
U = Thin Wall Tube Sample	q_p = Unconfined Compressive Strength (ksf)	PL = Plastic Limit
R = Rock Core Sample	$S_{u(lab)}$ = Lab Vane Shear Strength (psf)	PI = Plasticity Index
V = Insitu Vane Shear Test	WOH = weight of 140lb. hammer	G = Grain Size Analysis
SSA = Solid Stem Auger	WOR = weight of rods WOC = weight of casing	C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/16	25.00 - 27.00	7/1/WOH/1	1	40			Grey, wet, very soft, silty CLAY, varved with sand lenses, trace gravel, trace coarse sand.	G#176634 A-6, CL WC=38.3%
						27				
	MV		27.00 - 27.00			23			55x110 mm vane raw torque readings: MV= could not push.	
						21				
						19				
30	7D	24/10	30.00 - 32.00	Push thru vane		42			Grey, wet, soft, silty CLAY, trace fine sand.	G#176635
	V1		31.00 - 31.36	Su=402/49 psf		33			55x110 mm vane raw torque readings: V1= 9.0/1.1 ft-lbs	A-6, CL WC=47.2% LL=33
	MV		32.00 - 32.00			29			MV = could not push	PL=19 PI=14
						31				
						36				
35	8D	24/3	35.00 - 37.00	1/5/8/6	13	55	-25.10		35.500 Grey, wet, medium dense, fine silty SAND, trace coarse sand.	
	MV		36.00 - 36.00			38			55x110 mm vane raw torque readings: MV= could not push.	
						27				
						23				
						38				
40	9D	24/6	40.00 - 42.00	3/WOH/3/7	3	48	-29.60		40.000 Grey, wet, soft, SILT, little gravel, sand and clay, dilatent.	G#176636 A-4, CL-ML WC=24.2%
						36				
						28				
						48				
						124	-33.60		44.000	
45	10D	24/6	45.00 - 47.00	22/10/12/11	22	93			Grey, wet, medium dense, fine to coarse SAND, some gravel and silt, trace clay.	G#176637 A-2-4, SC-SM WC=10.2%
						79				
						116				
						132				
						260				
50										

Remarks:

MV = Unsuccessful vane shear test attempt.

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: Station 44 Bridge over Tidal Estuary Location: York, Maine	Boring No.: BB-YR44-101 PIN: 15112.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	41+92.7, 7.4 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.5' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
50	MD	2/0	50.00 - 50.17	60(0.2")	---	aWR			Wash material in spoon a Washed ahead and Roller coned ahead to 54.5' bgs through very dense grey till.		
	R1	60/52	54.50 - 59.50	RQD = 68%		NQ	-44.10			54.500	
55						Core			Bedrock: Dark grey/green, fine grained, ANDESITE porphyry. R1:Core Times (min:sec) 54.5' - 55.5' (18:20) 55.5' - 56.5' (10:00) 56.5' - 57.5' (10:10) 57.5' - 58.5' (9:10) 58.5' - 59.5' (9:05) Recovery=97%		
							-49.10			59.500	
60									**Bottom of Exploration at 59.50 feet below ground surface.**		
65											
70											
75											

Remarks:

MV = Unsuccessful vane shear test attempt.

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* 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:	Station 44 Bridge over Tidal Estuary			**Boring No.:**	BB-YR44-102
Location:	York, Maine			**PIN:**	15112.00
Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	42+55.3, 8.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	9.98	Pavement 0.420	
								Brown, damp, fine to coarse SAND, little gravel, trace silt, (Fill).	
5	1D	24/10	5.00 - 7.00	4/4/5/6	9			Brown, damp, loose, fine to coarse SAND, some gravel, little silt, (Fill).	
10	2D	24/2	10.00 - 12.00	6/3/4/11	7	46		Brown, wet, loose, fine silty SAND, little gravel, trace medium to coarse sand, (Fill).	
						27			
						208			
						191		Obstruction at 13.0' bgs. Roller coned through.	
						41	-3.60	14.000	
15	3D	24/16	15.00 - 17.00	2/2/2/1	4	34		Grey, wet, soft, sandy SILT with broken shells.	
						30			
						27			
						73			
						43	-8.60	19.000	
20	4D/MV	24/18	20.00 - 22.00	5/3/4/7	7	45		Grey, wet, medium stiff, Silty CLAY, trace gravel and sand, mottled. 55x110 mm and 25.4x50.8 mm vane raw torque readings: MV = could not push	G#176638 A-6, CL WC=31.3% LL=34 PL=20 PI=14
						43			
						41			
						34			
						34			
25									

Remarks:

MV = Unsuccessful vane shear test attempt.

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: Station 44 Bridge over Tidal Estuary
Location: York, Maine

Boring No.: BB-YR44-102
PIN: 15112.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	4.5" SSA
Operator:	G. Lidstone	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/1/04-3/1/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	42+55.3, 8.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' (Tidal)

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/24	25.00 - 27.00	Push thru vane.	---	46			Grey, wet, soft, silty CLAY with black staining, occasional black fine to coarse silty sand layers, trace gravel. 55x110 mm vane raw torque readings: V1 = 10.0/2.4 ft-lbs V2 = 11.8/2.6 ft-lbs	G#176639 A-6, CL WC=42.8%
	V1		26.00 - 26.36	Su=446/107 psf		29				
	V2		27.00 - 27.36	Su=527/116 psf		32				
						30				
						33				
30	6D	24/18	30.00 - 32.00	WOH/WOH/11/12	11	59			Washed ahead of Casing.	G#176640 A-4, CL-ML WC=28.3% LL=19 PL=15 PI=4
						WA	-20.60		31.000	
	V3		31.00 - 31.36	Su=424/125 psf		47			Grey, wet, stiff, clayey SILT, trace gravel and sand. 55x110 mm vane raw torque readings: V3 = 9.5/2.8 ft-lbs MV = could not push Wash water changed to brown ± 33.0' bgs.	
	MV		32.00 - 32.00			54				
						126				
35	7D	1/1	35.00 - 35.08	80(0.1")	---				Grey, wet, dense, fine to coarse silty SAND, some gravel.	
	8D	1/0	36.00 - 36.08	60(0.1")	---	NQ				
	R1	50/50	36.90 - 41.07	RQD = 46%		Core	-26.50		36.900 Bedrock: Grey/brown, fine grained, fractured, brecciated and metamorphosed PHYLLITE. R1:Core Times (min:sec) 36.9' - 37.9' (6:00) 37.9' - 38.9' (6:30) 38.9' - 39.9' (12:15) 39.9' - 40.9' (3:35) 40.9' - 41.07' (1:51) Recovery=100%	
40							-30.70		41.100 **Bottom of Exploration at 41.10 feet below ground surface.**	
45										
50										

Remarks:

MV = Unsuccessful vane shear test attempt.

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

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

APPENDIX B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): York   Project Number: 15112.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	GSDC Sheet	WC	LL	PI	Classification		
									Unified	AASHTO	Frost
BB-YR44-101, 5D	41+92.7	7.4 Rt.	20.0-22.0	176633	1	37.8			CL	A-6	III
BB-YR44-101, 6D	41+92.7	7.4 Rt.	25.0-27.0	176634	1	38.3			CL	A-6	III
BB-YR44-101, 7D	41+92.7	7.4 Rt.	30.0-32.0	176635	1	47.2	33	14	CL	A-6	III
BB-YR44-101, 9D	41+92.7	7.4 Rt.	40.0-42.0	176636	1	24.2			CL-ML	A-4	IV
BB-YR44-101, 10D	41+92.7	7.4 Rt.	45.0-47.0	176637	1	10.2			SC-SM	A-2-4	III
BB-YR44-102, 4D	42+55.3	8.4 Lt.	20.0-22.0	176638	2	31.3	34	14	CL	A-6	III
BB-YR44-102, 5D	42+55.3	8.4 Lt.	25.0-27.0	176639	2	42.8			CL	A-6	III
BB-YR44-102, 6D	42+55.3	8.4 Lt.	30.0-32.0	176640	2	28.3	19	4	CL-ML	A-4	IV

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

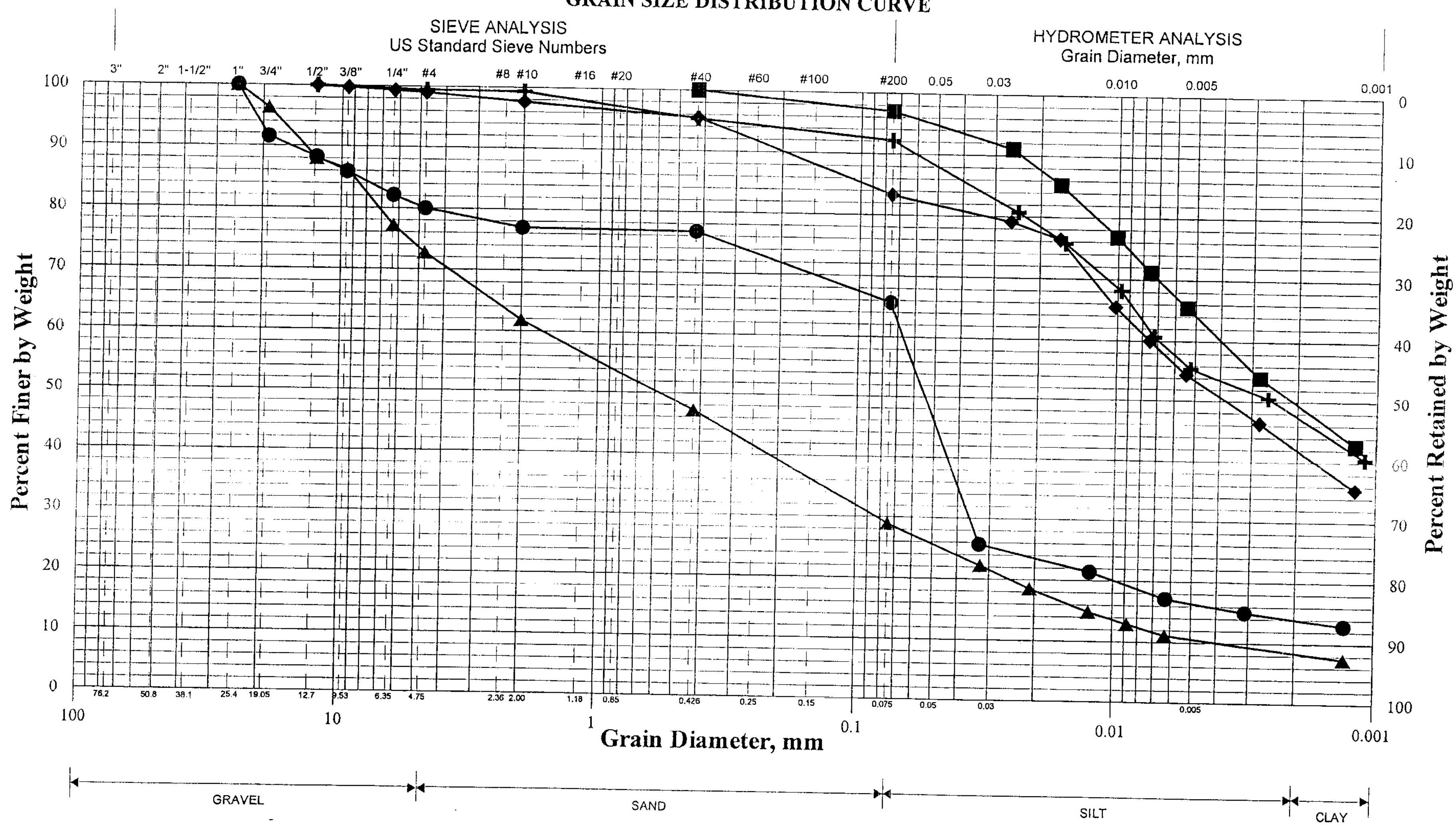
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

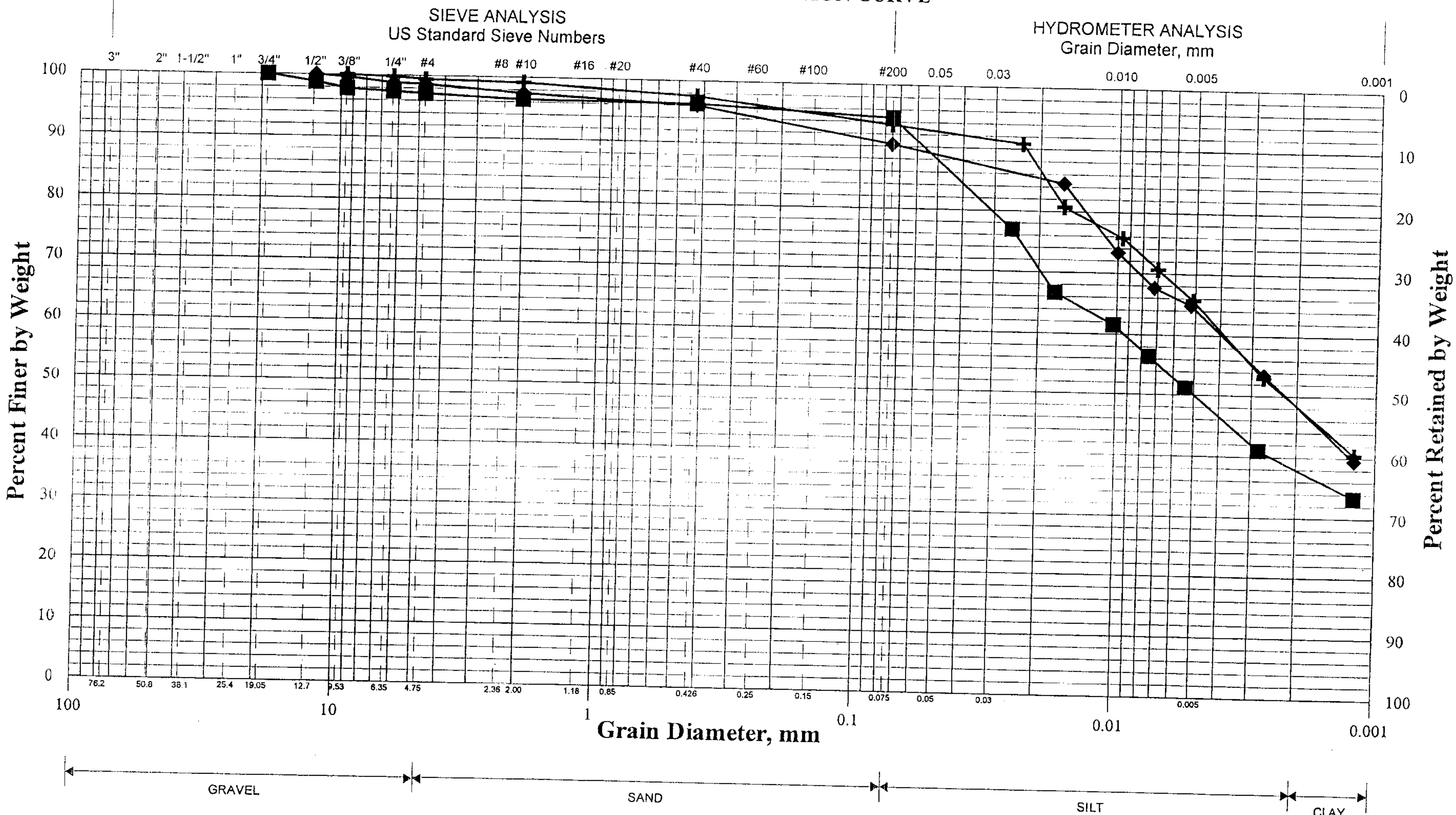
	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR44-101	5D	20.0-22.0	Silty CLAY, trace sand, trace gravel.	37.8			
◆	BB-YR44-101	6D	25.0-27.0	Silty CLAY, little sand, trace gravel.	38.3			
■	BB-YR44-101	7D	30.0-32.0	Silty CLAY, trace sand.	47.2	33	19	14
●	BB-YR44-101	9D	40.0-42.0	SILT, little gravel, sand and clay.	24.2			
▲	BB-YR44-101	10D	45.0-47.0	Fine to coarse SAND, some gravel and silt, trace clay.	10.2			
×	---							

PIN: 15112.00
Town: York
Reported by: T. White
Date: 3/29/04

SHEET NO. 1

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

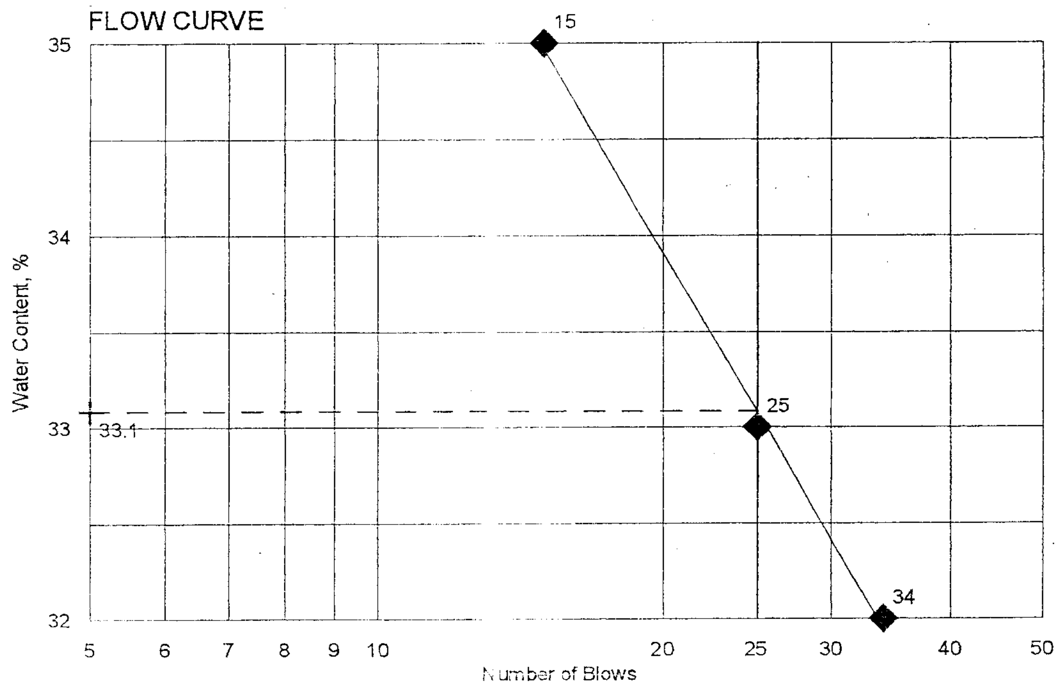
	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-YR44-102	4D	20.0-22.0	Silty CLAY, trace gravel and sand.	31.3	34	20	14
◆	BB-YR44-102	5D	25.0-27.0	Silty CLAY, trace gravel and sand.	42.8			
■	BB-YR44-102	6D	30.0-32.0	Clayey SILT, trace gravel and sand.	28.3	19	15	4
●	---							
▲	---							
✕	---							

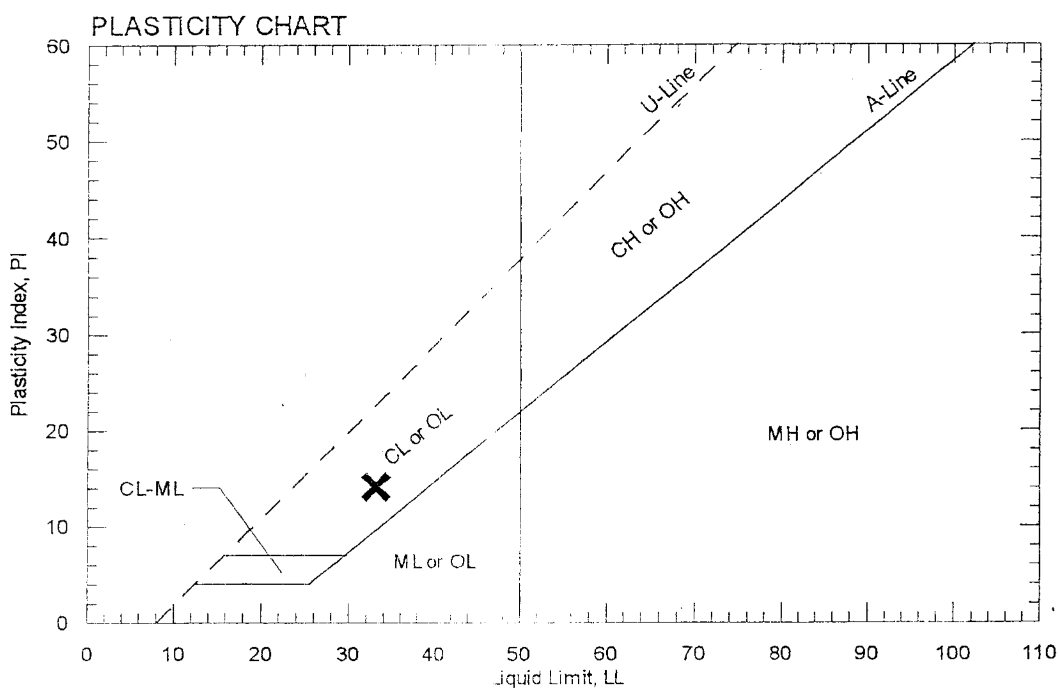
PIN: 15112.00
Town: York
Reported by: T. White
Date: 3/29/04

SHEET NO. 2

State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

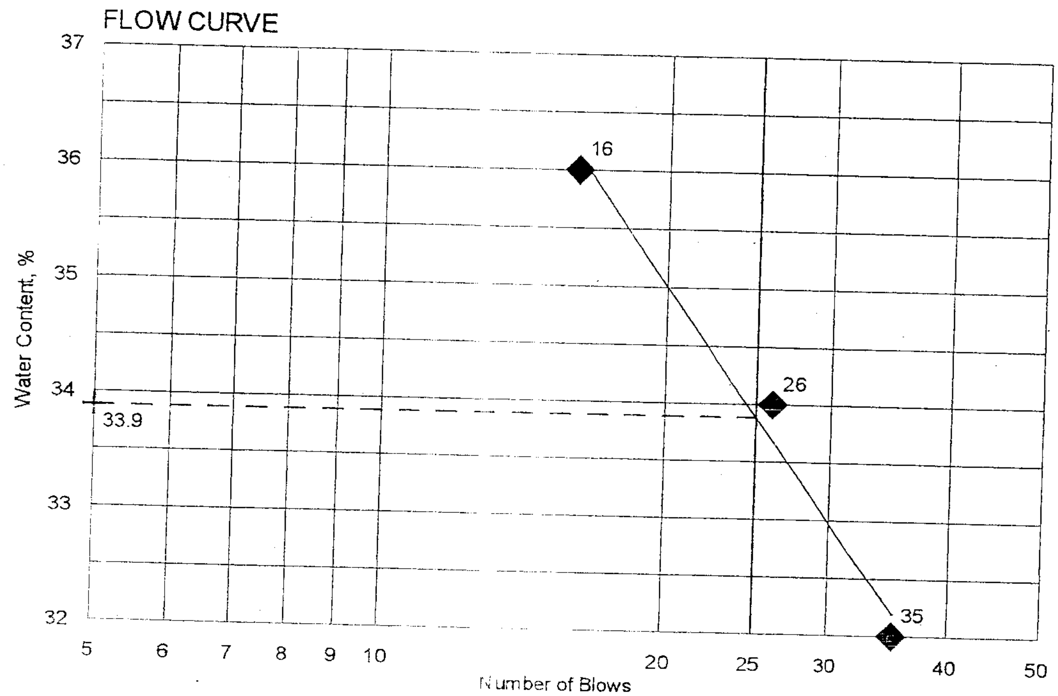
TOWN	York	Reference No.	176635
PIN	15112.00	Natural water content (%)	47.2
Date	4/21/2004	Plastic limit	19
Boring No.	BB-YR44-101	Liquid limit	33
Station	41+92.7, 7.4' RT	Plasticity index	14
Depth/Sample No.	30-32'/7D	Reported by	KLD

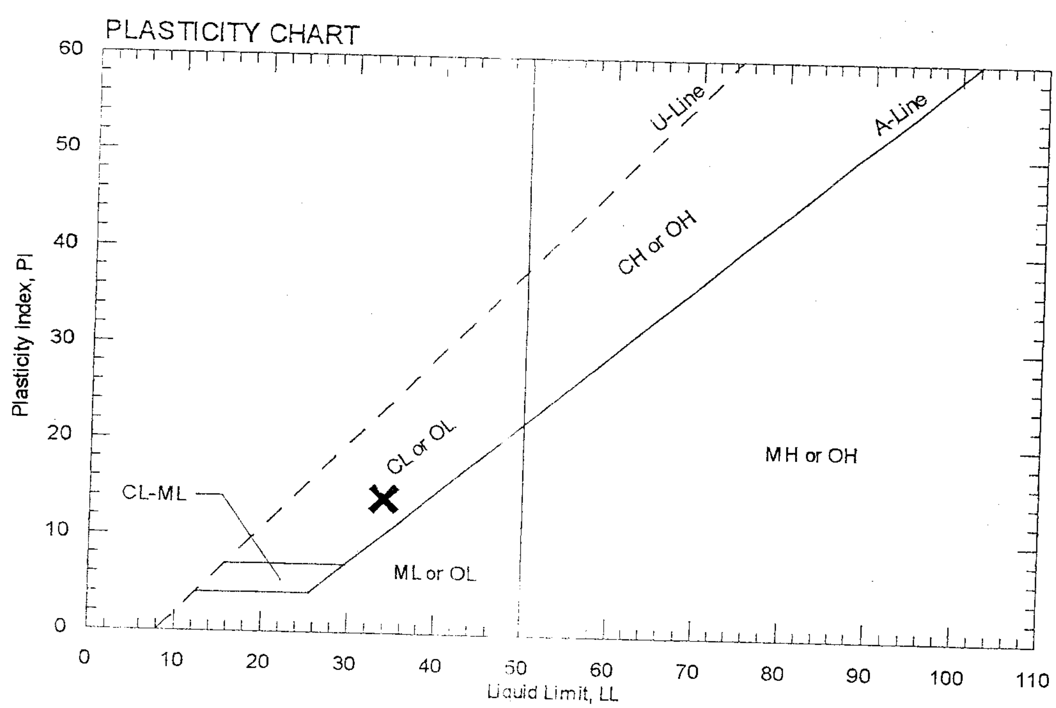




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

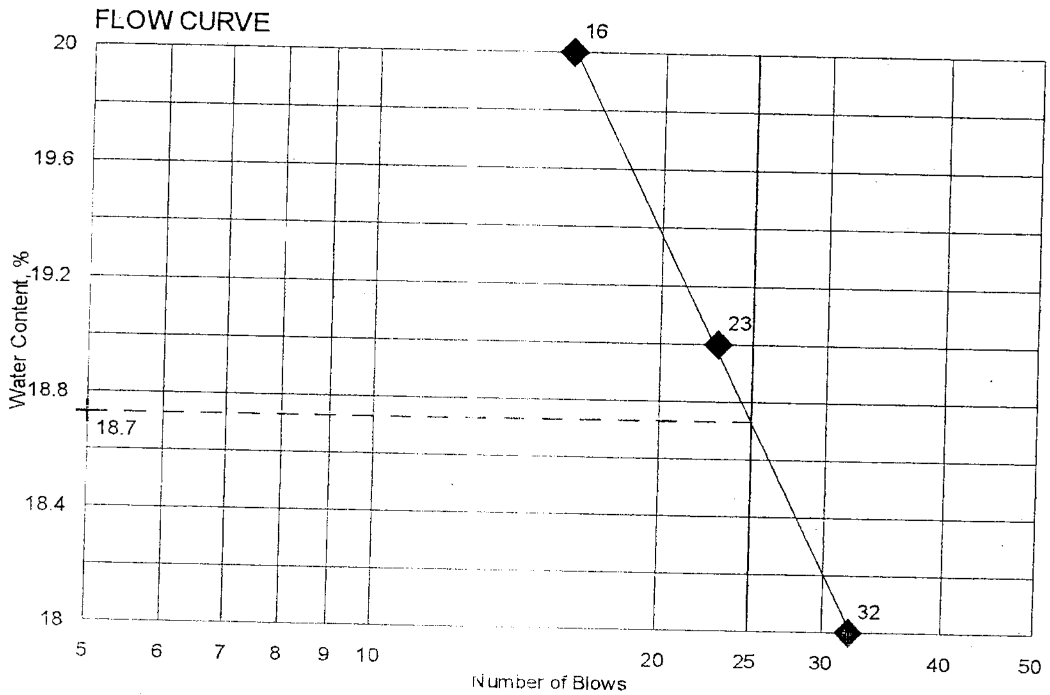
TOWN	YORK	Reference No.	176638
PIN	15112.00	Natural water content (%)	31.3
Date	4/23/2004	Plastic limit	20
Boring No.	BB-YR44-102	Liquid limit	34
Station	42+55.3, 8.4' LT	Plasticity index	14
Depth/Sample No.	20.0-22.0/4D	Reported by	B. D. FOGG

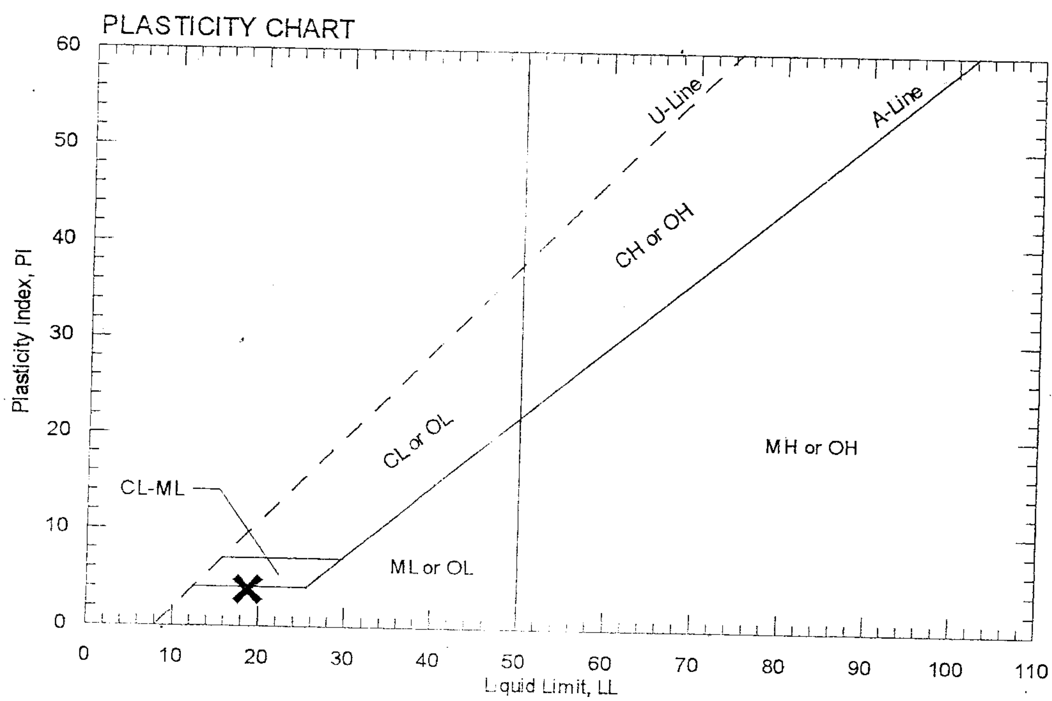




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	York	Reference No.	176640
PIN	15112.00	Natural water content (%)	28.3
Date	4/21/2004	Plastic limit	15
Boring No.	BB-YR44-102	Liquid limit	19
Station	42+55.3, 8.4' LT	Plasticity index	4
Depth/Sample No.	30-32'/6D	Reported by	KLD





State of Maine

Department of Transportation
Bridge Program - Geotechnical
16 State House Station
Augusta, Maine 04333-0016

Kate Maguire, PE
Phone: 624-3415
FAX: 624-3491
email: kate.maguire@maine.gov

Addendum #2

To: Jeff Folsom, PE
cc: Jim Wentworth, PE, TEDOCS
From: Kate Maguire, PE
Date: February 13, 2009
Re: York Station 44 Bridge
Estimated H-Pile Resistances and
LRFD Geotechnical Design Recommendations
PIN: 15112.00

The purpose of this addendum is to transmit estimated H-pile resistances and pile design recommendations for the replacement of the Station 44 Bridge in York, Maine. A geotechnical report was published for this project in July 2004 (MaineDOT Soils Report No. 2004-23). The design information included in the report was developed using Allowable Stress Design (ASD) methodology. In 2006 MaineDOT adopted the use of Load Factor and Resistance Design (LRFD) design methodology. The plans currently developed for design and construction of the Station 44 Bridge require LRFD design methods. This addendum should be used as a supplement to the original geotechnical report to aid in the design of the replacement structure.

INTEGRAL ABUTMENT H-PILES:

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 14x73, HP 14x89 or HP 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on Table 1 below:

Location	Depth to Bedrock From Ground Surface	Ground Surface Elevation	Top of Bedrock Elevation	Rock Quality Designation	Estimated Pile Length
BB-YR44-101 Abutment #1	54.5 feet	10.4 feet	-44.10 feet	68%	55 feet
BB-YR44-102 Abutment #2	36.9 feet	10.4 feet	-26.5 feet	46%	40 feet

Table 1 – Estimated Pile Lengths for Piles Installed to Bedrock Surface

These pile lengths do not take into account the additional five (5) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed below.

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. The design flood scour is defined in AASHTO LRFD Bridge Design Specifications 4th Edition (LRFD) Articles 2.6.4.4.2 and 3.7.5. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2.

Strength Limit State Design:

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. It is the responsibility of the structural engineer to calculate the column slenderness factor (λ) for the upper and lower portions of integral H-piles based on unbraced lengths and K-values from project specific L-Pile® analyses and determine the structural pile resistances. Preliminary estimates of the factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (good driving conditions) and a λ of 0.

The nominal geotechnical compressive resistances of the H-pile sections in the strength limit state were calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45.

The drivability of the four proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65. Per LRFD Article 10.5.5.2.3 the resistance factor 0.65 is reduced by 20% if it is applied to a nonredundant pile group, i.e., there are less than 5 piles in a group. In the event that fewer than 5 piles are used per abutment, the resistance factor of ϕ_{dyn}, = 0.52 shall be used.

The calculated factored axial compressive structural, geotechnical and drivability resistances of the four proposed H-pile sections for both abutments are summarized in Table 2 below. Supporting calculations are included at the end of this addendum.

Pile Section	Factored Axial Resistances for Abutment Piles at the Strength Limit State			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance ϕ=0.65	Drivability Resistance ϕ=0.52
HP 12 x 53	465 kips	236 kips	313 kips	251 kips
HP 14 x 73	642 kips	297 kips	379 kips	303 kips
HP 14 x 89	783 kips	361 kips	404 kips	323 kips
HP 14 x 117	1032 kips	473 kips	449 kips	359 kips

*based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure).

Table 2 – Factored Axial Resistances for Abutment Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial drivability resistance is less than the factored axial structural resistance and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the factored drivability resistances shown in Table 2 above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined nominal axial and flexural resistance of the pile in the interaction equation, (LRFD Eq. 6.12.2.2.1-2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.12.2. The structural designer should evaluate the capacity of the pile in combined axial load and flexure when the loads and moments are calculated.

Service and Extreme Limit States Design:

For the service and extreme limit states resistance factors of 1.0 are recommended for structural and geotechnical pile resistances. For preliminary analysis, the H-piles were assumed fully embedded and λ was taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances.

The calculated factored axial structural, geotechnical and drivability resistances of the four proposed H-pile sections for both abutments are summarized in Table 3 below. Supporting calculations are included at the end of this addendum.

Pile Section	Factored Axial Resistances for Abutment Piles at the Service/Extreme Limit States			
	Structural Resistance	Geotechnical Resistance	Drivability Resistance ϕ=0.65	Governing Resistance
HP 12 x 53	775 kips	524 kips	482 kips	482 kips
HP 14 x 73	1070 kips	660 kips	583 kips	583 kips
HP 14 x 89	1305 kips	803 kips	622 kips	622 kips
HP 14 x 117	1720 kips	1052 kips	690 kips	690 kips

*based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure).

Table 3 – Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial drivability resistance is less than the factored axial structural resistance and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the maximum factored axial pile load used in design for the service and extreme limit states should not exceed the factored drivability resistance shown in Table 3 above.

PILE INSTALLATION RECOMMENDATIONS:

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. Per LRFD Article 10.5.5.2.3 the resistance factor 0.65 is reduced by 20% if it is applied to a nonredundant pile group, i.e., there are less than 5 piles in a group. In the event that fewer than 5 piles are used per abutment, the resistance factor of ϕ_{dyn}, = 0.52 shall be used. The maximum factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Geotechnical Engineer. Driving stresses in the pile determined in a drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 8 to 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

STUB ABUTMENTS AND WINGWALLS:

Cast-in-place integral stub abutments and wingwalls shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Since the abutments will be pile supported, design for resistance against sliding and overturning is not required. The design of abutments at the strength limit state shall consider the pile group and structural failure. The design of independent return wings at the strength limit state shall consider nominal bearing resistance and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

A resistance factor of ϕ=1.0 shall be used to assess abutment design at the service limit state including: settlement, horizontal movement, overall stability and scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor,ϕ, of 0.65. Extreme limit state design checks for abutments supported on piles shall include pile structural resistance pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0. The unfactored strength limit state loads include any debris loads occurring during the flood event.

Integral abutments and wingwall sections that are integral with the abutment should be designed to withstand a passive earth pressure state. In designing for passive earth pressure associated with integral abutments, the Coulomb state is recommended. Experience in designing wingwalls and integral abutments has shown that the use of the Coulomb passive earth pressure K_p=6.89 may result in uneconomical wall sections. For this reason, consideration may be given to using a Rankine passive earth pressure, K_p=3.25 when designing integral abutments and integral wingwall extensions.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the abutments when traffic loads are located within a horizontal distance equal to one-half of the wall height behind the back of the wall. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads on abutments is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 4 below:

Abutment Height	h_{eq}
5 feet	4.0 feet
10 feet	3.0 feet
≥20 feet	2.0 feet

Table 4 – Equivalent Height of Soil for Vehicular Loading
on Abutments Perpendicular to Traffic

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Conventional wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using as active earth pressure coefficient, K_a of 0.307 calculated using Rankine Theory for cantilever wingwalls and K_a of 0.276 calculated using Coulomb Theory for gravity shaped structures. Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the wingwalls when traffic loads are located within a horizontal distance equal to one-half of the wall height behind the back of the wall. In the situation a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD Article 3.11.6.4. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken form Table 5 below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic = 0 feet	Distance from wall backface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥20	2.0	2.0

Table 5 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface.

All abutment designs shall include a drainage system behind the abutments to intercept any water. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. French drains or geocomposite drainage board applied to the backsides of the abutments and wingwalls with weep holes will provide adequate drainage. To avoid water intrusion behind the abutment, the approach slab should connect directly to the abutment. Traffic and seasonal movements of integral abutments cause the fill behind the abutments to shift and self compact. For this reason the approach slab should be supported on a sleeper slab placed outside the area expected to settle.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

BEARING RESISTANCE:

It is anticipated that the proposed abutment and wingwalls will be founded on driven H-piles. In the event that a spread footing founded on fill soils is proposed for the site these shall be designed to provide stability against bearing capacity failure. Applicable permanent and transient loads are specified in LRFD Articles 3.4.1 and 11.5.5. The soil distribution may be assumed to be uniformly distributed over the effective base as shown in LRFD Figure 11.6.3.2-1.

Bearing resistance for any structure founded on fill soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 7 ksf for footings on fill soils. The bearing resistance factor, $\phi_{b,}$ for spread footings on soil is 0.45 based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 5 ksf may be used when analyzing the service limit state and for preliminary sizing of footings based on presumptive bearing resistance values. The bearing resistance for spread footings shall be checked for the extreme limit state with a resistance factor of 1.0. Supporting calculations are included at the end of this addendum.

In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as 0.3*f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material. Any organic material encountered shall be removed to the full depth and replaced with compacted Granular Borrow, MaineDOT 703.19.

CLOSURE:

This addendum has been prepared to provide LRFD H-pile resistances and geotechnical design recommendations for the design of a replacement structure at the Station 44 Bridge site in York, Maine. It was prepared in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is implied. This addendum should be used as a supplement to the original geotechnical report to aid in the design of the replacement structure. In the event that any changes in the nature, design, or location of the proposed project are planned, the recommendations should be reviewed by a geotechnical engineer to assess the appropriateness and to modify the recommendations as appropriate to reflect the changes.

Abutment Foundations: Integral driven H-piles

Structural Resistance of H-piles by LRFD Design Methods

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at the following piles:

Note: All matrices set up in this order

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

yield strength: $F_y := 50 \cdot ksi$

Nominal Compressive Resistance $P_n=0.66^{\lambda}*F_y*A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

$\lambda=(Kl/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3 $\lambda := 0$ as l unbraced length is 0

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Resistance:

Strength Limit State Axial Resistance factor for piles in compression under good driving conditions:

From Article 6.5.4.2 $\phi_c := 0.6$ good driving conditions

Factored Compressive Resistance:

eq. 6.9.2.1-1 $P_f := \phi_c \cdot P_n$ $P_f = \begin{pmatrix} 465 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1 $P_f := \phi \cdot P_n$ $P_f = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

Geotechnical Resistance of H-piles by LRFD Design Methods

Assume piles will be end bearing on bedrock driven through overlying sand, silt and clay.

Bedrock Type:

Phyllite or Andesite RQD assume 55% and ϕ = 27 deg (Tomlinson 4th Ed. pg 139)

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

Steel area: $A_s = \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

Pile depth: $d := \begin{pmatrix} 11.78 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$

Pile width: $b := \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

Calculate pile box area:

$A_{box} := \overrightarrow{(d \cdot b)}$ $\qquad A_{box} = \begin{pmatrix} 141.89 \\ 198.502 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$

End bearing resistance of piles on bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

q_u for phyllite compressive strength ranges from 3500 to 35000 psi

q_u for andesite compressive strength ranges from 14000 to 26000 psi

use $\sigma_{cp} := 20000 \cdot psi$

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on knowledge of area bedrock

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are assumed tight

Footing width, b:

$b = \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \sqrt{1 + 300 \cdot \frac{\delta}{c}}} \qquad K_{sp} = \begin{pmatrix} 0.56333 \\ 0.51437 \\ 0.51263 \\ 0.50969 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot ft$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 0 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4 \left(\frac{L_s}{B_s} \right)$ $d_f = 1$ should be < or = 3 OK

$$q_a := \sigma_{cp} \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 1622 \\ 1481 \\ 1476 \\ 1468 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$$R_p := \overrightarrow{(3 q_a \cdot A_s)} \qquad R_p = \begin{pmatrix} 524 \\ 660 \\ 803 \\ 1052 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 236 \\ 297 \\ 361 \\ 473 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 524 \\ 660 \\ 803 \\ 1052 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

Drivability Resistance of H-piles by per LRFD Design Methods

For steel piles in compression or tension
σ_{dr} = 0.9 x ϕ_{da} x f_y (eq. 10.7.8-1)

Ref: LRFD Article 10.7.8

$f_y := 50 \cdot ksi$ yield strength of steel

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1
Pile Drivability Analysis, Steel piles

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

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored structural resistance from above as this governs) divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Strength Limit State:

Table 10.5.5.2.3-1 pg 10-37 gives resistance factor for dynamic test, ϕ_{dyn}: $\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium site variability.

LRFD Article 10.5.5.2.3 requires that if less than 5 piles are used in a group, the resistance factor should be reduced by 20% to reflect a higher target β value.

$\phi_{dyn.reduced} := 0.65 \cdot 0.8$ $\phi_{dyn.reduced} = 0.52$

Calculate resistances at strength limit state with both Φs for memo.

Service and Extreme Limit State:

LRFD Articles 10.5.5.1 (Service) and 10.5.5.3 (Extreme) resistance factors: $\phi := 1.0$

Assume Contractor will use a MKT DE 42 hammer to install 12 x 53 piles

Pile Size = 12 x 53

State of Maine Dept. Of Transportation — 03-Feb-2009
York Station 44 Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
480.0	39.54	2.06	14.7	10.00	25.65
481.0	39.53	2.06	14.9	10.00	25.67
482.0	39.53	2.06	15.0	10.00	25.68
483.0	39.52	2.06	15.1	10.00	25.70
484.0	39.48	1.97	15.4	10.00	25.64
485.0	39.47	1.97	15.5	10.00	25.66
486.0	39.47	1.95	15.6	10.00	25.67
487.0	39.45	1.95	15.8	10.00	25.68
488.0	39.45	1.95	15.9	10.00	25.70
489.0	39.41	1.86	16.2	10.00	25.65

Limited to blow count to 15 blows per inch

$R_{dr\_12x53\_nom} := 482 \cdot kip$

Strength Limit State:

$R_{dr\_12x53\_strength} := R_{dr\_12x53\_nom} \cdot \phi_{dyn}$

$R_{dr\_12x53\_strength} = 313 \cdot kip$

$R_{dr\_12x53\_strength\_red} := R_{dr\_12x53\_nom} \cdot \phi_{dyn.reduced}$

$R_{dr\_12x53\_strength\_red} = 251 \cdot kip$

Service and Extreme Limit State:

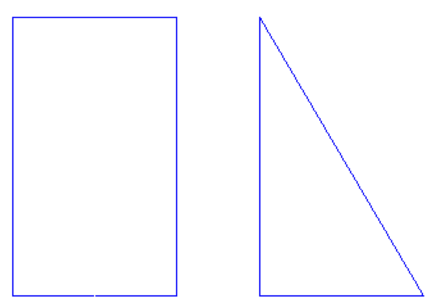
$R_{dr\_12x53\_servext} := R_{dr\_12x53\_nom} \cdot \phi$

$R_{dr\_12x53\_servext} = 482 \cdot kip$

MKT DE 42/35

Stroke	10.00	feet
Efficiency	0.800	
Helmet	1.60	kips
Hammer Cushion	40399	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	15.50	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 20 %
(Proportional)

Assume Contractor will use a MKT DE 42 hammer to install 14 x 73 piles

Pile Size = 14 x 73

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York Station 44 Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
580.0	44.82	2.21	14.8	10.18	25.15
581.0	44.82	2.22	14.8	10.19	25.19
582.0	44.85	2.19	15.0	10.20	25.15
583.0	44.88	2.23	15.0	10.19	25.21
584.0	44.87	2.19	15.2	10.20	25.18
585.0	44.86	2.19	15.3	10.20	25.17
586.0	44.89	2.20	15.3	10.21	25.20
587.0	44.92	2.21	15.4	10.21	25.23
588.0	44.89	2.22	15.5	10.21	25.23
589.0	44.93	2.22	15.6	10.22	25.26

Limited to blow count to 15 blows per inch

$R_{dr\_14x73\_nom} := 583 \cdot kip$

Strength Limit State:

$R_{dr\_14x73\_strength} := R_{dr\_14x73\_nom} \cdot \phi_{dyn}$

$R_{dr\_14x73\_strength} = 379 \cdot kip$

$R_{dr\_14x73\_strength\_red} := R_{dr\_14x73\_nom} \cdot \phi_{dyn.reduced}$

$R_{dr\_14x73\_strength\_red} = 303 \cdot kip$

Service and Extreme Limit State:

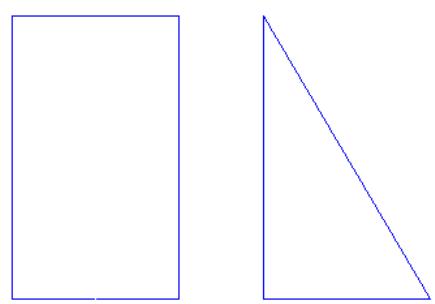
$R_{dr\_14x73\_servext} := R_{dr\_14x73\_nom} \cdot \phi$

$R_{dr\_14x73\_servext} = 583 \cdot kip$

MKT DE 42/35

Efficiency	0.800	
Helmet	1.60	kips
Hammer Cushion	40399	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a MKT DE 42 hammer to install 14 x 89 piles

Pile Size = 14 x 89

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York Station 44 Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
615.0	41.31	1.39	14.5	10.00	23.76
616.0	41.28	1.43	14.6	10.00	23.73
617.0	41.31	1.42	14.7	10.00	23.75
618.0	41.32	1.41	14.7	10.00	23.77
619.0	41.28	1.44	14.8	10.00	23.74
620.0	41.33	1.44	14.9	10.00	23.76
621.0	41.28	1.46	15.0	10.00	23.73
622.0	41.34	1.46	15.0	10.00	23.75
623.0	41.28	1.49	15.1	10.00	23.71
624.0	41.34	1.48	15.2	10.00	23.74

Limited to blow count to 15 blows per inch

$R_{dr\_14x89\_nom} := 622 \cdot kip$

Strength Limit State:

$R_{dr\_14x89\_strength} := R_{dr\_14x89\_nom} \cdot \phi_{dyn}$

$R_{dr\_14x89\_strength} = 404 \cdot kip$

$R_{dr\_14x89\_strength\_red} := R_{dr\_14x89\_nom} \cdot \phi_{dyn.reduced}$

$R_{dr\_14x89\_strength\_red} = 323 \cdot kip$

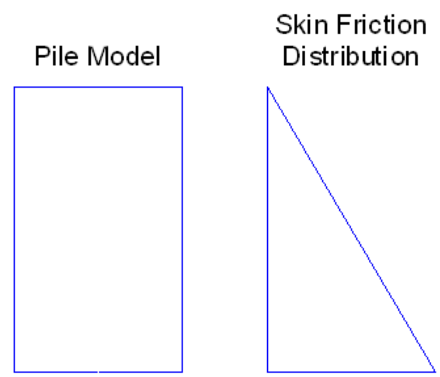
Service and Extreme Limit State:

$R_{dr\_14x89\_servext} := R_{dr\_14x89\_nom} \cdot \phi$

$R_{dr\_14x89\_servext} = 622 \cdot kip$

MKT DE 42/35

Stroke	10.00	feet
Efficiency	0.800	
Helmet	1.60	kips
Hammer Cushion	40399	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	26.10	in2



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a MKT DE 42 hammer to install 14 x 117 piles

Pile Size = 14 x 117

State of Maine Dept. Of Transportation — 03-Feb-2009
York Station 44 Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
630.0	35.19	0.71	12.4	9.63	21.69
640.0	35.32	0.84	12.8	9.66	21.74
650.0	35.61	0.91	13.2	9.70	21.83
660.0	35.77	0.96	13.6	9.73	21.93
670.0	35.86	1.03	14.1	9.77	21.98
680.0	36.13	1.10	14.5	9.80	22.08
690.0	36.19	1.19	15.0	9.82	22.13
700.0	36.38	1.29	15.5	9.85	22.19
710.0	36.51	1.40	16.1	9.88	22.24
720.0	36.90	1.56	16.3	10.00	22.56

Limited to blow count to 15 blows per inch

$R_{dr\_14x117\_nom} := 690 \cdot kip$

Strength Limit State:

$R_{dr\_14x117\_strength} := R_{dr\_14x117\_nom} \cdot \phi_{dyn}$

$R_{dr\_14x117\_strength} = 449 \cdot kip$

$R_{dr\_14x117\_strength\_red} := R_{dr\_14x117\_nom} \cdot \phi_{dyn.reduced}$

$R_{dr\_14x117\_strength\_red} = 359 \cdot kip$

Service and Extreme Limit State:

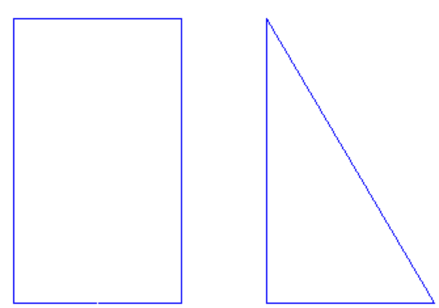
$R_{dr\_14x117\_servext} := R_{dr\_14x117\_nom} \cdot \phi$

$R_{dr\_14x117\_servext} = 690 \cdot kip$

MKT DE 42/35

Efficiency	0.800	
Helmet	1.60	kips
Hammer Cushion	40399	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Abutment and Wingwall Passive and Active Earth Pressure:

For cases where interface friction is considered (for gravity structures) use Coulomb Theory

Coulomb Theory - Passive Earth Pressure from Maine DOT Bridge Design Guide
Section 3.6.6 pg 3-8

Angle of back face of wall to the horizontal: $\alpha := 90 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 20 \cdot deg$

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

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

$K_p = 6.89$

Rankine Theory - Passive Earth Pressure from Bowles 5th Edition Section 11-5 pg 602

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

$$K_{p\_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$K_{p\_rank} = 3.25$

Bowles does not recommend the use of the Rankine Method for K_p when β>0.

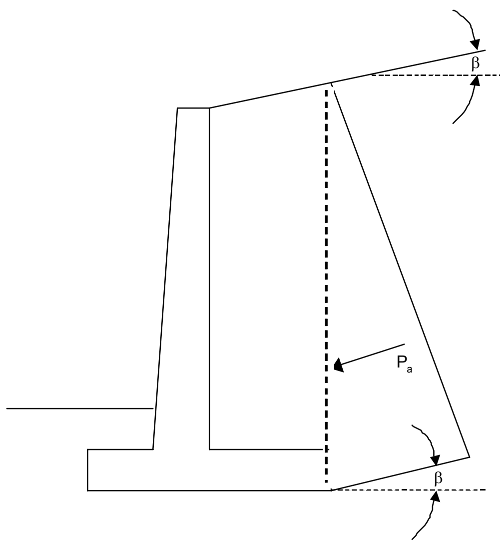
Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

unit weight: $\gamma_{type4} := 125 \cdot pcf$

Internal Friction Angle: $\phi_{type4} := 32 \cdot deg$

Cohesion: $c_{sand} := 0 \cdot psf$

Active Earth Pressure - Rankine Theory
from MaineDOT Bridge Design Guide Section 3.6.5.2 pg 3-7



Generally use Rankine for long heeled cantilever walls where the failure surface is un interrupted by the top of the wall system. The earth pressure is applied to a plane extending vertically up from the heel of the wall base and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or the backface of the wall.

For cantilever walls with horizontal backfill surface:

$$K_{a\_rankine} := \tan\left(45 \cdot deg - \frac{\phi_{type4}}{2}\right)^2 \qquad K_{a\_rankine} = 0.307$$

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

$\beta := 0 \cdot deg$

$$K_{a\_rankine\_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}} \qquad K_{a\_rankine\_slope} = 0.307$$

Pa is oriented at an angle of β to the vertical plane.

<u>Active Earth Pressure</u> - Coulomb Theory
from MaineDOT Bridge Design Guide Section 3.6.5.2 pg 3-5

For cases where the backface of the wall interferes with the development of a full sliding surface in the backfill use Coulomb Theory.

- Coulomb Theory applies for gravity, semi-gravity, and prefab modular walls with steep back faces
- Coulomb Theory applies to concrete cantilever wall with short heels where the sliding surface is restricted by the top of the wall - the wedge of soil does not move.
- Inter face friction is considered in Coulomb Theory

Angle of backface of wall to the horizontal: $\alpha := 90 \cdot deg$

Choosing Friction Angle between fill and wall:

i.) From LRFD Table 3.11.5.3-1 range from 17 to 22 - choose δ = 20 degrees
ii.) From MaineDOT BDG Table 3-3 δ = 24 degrees
iii.) From LRFD Figure C3.11.5.3-1 - δ = 1/3 to 2/3 * Internal Friction Angle = 21.33 degrees

Use Friction Angle between fill and wall = $\delta := 20 \cdot deg$

β = Angel of fill slope to the horizontal $\beta := 0 \cdot deg$

Internal Friction Angle: $\phi_{type4} := 32 \cdot deg$

$$K_{a\_coulomb} := \frac{\sin\left(\alpha + \phi_{type4}\right)^2}{\sin(\alpha)^2 \cdot \sin(\alpha - \delta) \cdot \left(1 + \sqrt{\frac{\sin\left(\phi_{type4} + \delta\right) \cdot \sin\left(\phi_{type4} - \beta\right)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)}}\right)^2}$$

$$K_{a\_coulomb} = 0.276$$

Orientation of Coulomb Pa :

- In the case of gravity shaped walls and prefab walls - Pa is oriented δ degrees up from a perpendicular line to the backface.
- In the case of short heeled cantilever walls where the top of the wall interferes with the failure surface - Pa is oriented at an angle of 1/3 to 2/3 Φ to the normal of a vertical line extending up from the heel of the wall.

Bearing Resistance - Fill Soils:

Part 1 - Strength Limit State **Nominal and factored Bearing Resistance - Spread footing on fill**

Assumptions:
1. Footings will be embedded 4.0 feet for frost protection. $D_f := 4.0 \cdot ft$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)
   - Saturated unit weight: $\gamma_s := 125 \cdot pcf$
   - Dry unit weight: $\gamma_d := 120 \cdot pcf$
   - Internal friction angle: $\phi_{ns} := 32 \cdot deg$
   - Undrained shear strength: $c_{ns} := 0 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 10 \cdot ft$ Based on boring logs

Unit Weight of water: $\gamma_w := 62.4 \cdot pcf$

Look at several footing widths

$$B := \begin{pmatrix} 2 \\ 4 \\ 6 \\ 8 \\ 10 \end{pmatrix} \cdot ft$$

Terzaghi Shape factors from Table 4-1 For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For ϕ=32 deg $N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$q := (D_f) \cdot (\gamma_s)$ $q = 0.5 \cdot ksf$ $q_{nominal} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s) B \cdot N_\gamma \cdot s_\gamma$

$$q_{nominal} = \begin{pmatrix} 14 \\ 17 \\ 20 \\ 23 \\ 25 \end{pmatrix} \cdot ksf$$

Resistance Factor: $\phi_b := 0.45$

AASHTO LRFD Table 10.5.5.2.2-1

$q_{factored} := q_{nominal} \cdot \phi_b$

$$q_{factored} = \begin{pmatrix} 6.5 \\ 7.7 \\ 8.9 \\ 10.2 \\ 11.4 \end{pmatrix} \cdot ksf$$

Based on these footing widths

$$B = \begin{pmatrix} 2 \\ 4 \\ 6 \\ 8 \\ 10 \end{pmatrix} \cdot ft$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 7 ksf for footings on fill soils

Part 2 - Service Limit State

Presumptive Bearing Resistance for Service Limit State ONLY spread footings on fill soils

Reference: AASHTO LRFD Bridge Design Specifications Third Edition
Table C10.6.2.6.1-1 "Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy *(1982)*"

Type of Bearing Material: Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)

Consistency In Place: Medium dense

Bearing Resistance Ordinary Range (ksf): 4 to 8 ksf

Recommended Value of Use (ksf): 5 ksf

Based on an average N-value in the sand fill of 16 - Soils are medium dense

Recommended Value of Use for Service Limit State:

$q_{pres} := 5 \cdot ksf$