FINAL GEOTECHNICAL DESIGN REPORT
KENNEBUNK BRIDGE
MAINE DOT PIN 15098.00
KENNEBUNK, MAINE

PREPARED FOR:
Maine Department of Transportation
Augusta, Maine

PREPARED BY:
GZA GeoEnvironmental, Inc.
Portland, Maine

August 2010
File No. 09.0025597.10

GZA
GeoEnvironmental, Inc.

Engineers and Scientists

August 6, 2010

File No. 09.0025597.10
Maine DOT PIN 15098.00

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

Re: Final Geotechnical Design Report
Kennebunk Bridge Replacement
Kennebunk, Maine

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Dear Laura:

GZA GeoEnvironmental, Inc. (GZA) is pleased to provide you with this Final Geotechnical Design Report for the Kennebunk Bridge in Kennebunk, Maine. Our work was completed in accordance with GZA's General Contract Agreement (GCA U1210060627) with Maine Department of Transportation (MaineDOT), GZA Work Plan dated March 30, 2010, Contract Modification 1, executed on July 28, 2010, and the attached Limitations contained in **Appendix A** of this report.

It has been a pleasure serving the MaineDOT project team 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 design-phase subsurface exploration and geotechnical evaluation for replacement of the Kennebunk Bridge in Kennebunk, Maine. Our services were provided in accordance with GZA's General Contract Agreement (GCA U1210060627) with MaineDOT, GZA Work Plan dated March 30, 2010, Contract Modification 1, dated June 25, 2010, and GZA's Limitations contained in **Appendix A** of the report.

1.1 BACKGROUND

Kennebunk Bridge carries US Route 1 over the Mousam River in Kennebunk, Maine, as shown in **Figure 1, Locus Plan**. The current bridge consists of a single-span, steel girder, concrete deck superstructure supported on a hybrid foundation system that includes stone masonry gravity walls as primary support for the roadway and a series of reinforced concrete piers that buttress the stone masonry and support the sidewalk on each side. The stone masonry and reinforced concrete footings bear directly on bedrock.

GZA completed a preliminary geotechnical evaluation for the Kennebunk bridge replacement project and presented findings in a June 20, 2009 report. That report was prepared to provide geotechnical recommendations for a replacement bridge at the current bridge location, re-using portions of the existing stone-masonry substructures, and foundations.

HNTB Corporation, of Westbrook, Maine (HNTB) has since conducted final design evaluations and prepared construction documents for the project. Our current understanding of the project is based on the 99 Percent Plans dated July 14, 2010 and subsequent correspondence with HNTB. A replacement bridge is proposed that will be 90 feet long and include full-height, cast-in-place concrete abutments; a flared wing wall on the southwest corner; and 90-degree return retaining walls on three corners, including along Rotary Park. The proposed abutments, wingwall and retaining walls were labeled by HNTB in accordance with the following table.

PROPOSED SUBSTRUCTURE ELEMENTS	
Location	**Designation**
Southwest Abutment	Abutment 1
Northeast Abutment	Abutment 2
Southeast Wingwall	Wingwall 1
Southwest Retaining Wall	Retaining Wall 1
Northwest Retaining Wall	Retaining Wall 2
Northeast Retaining Wall	Retaining Wall 3

The new abutments, wing walls and retaining walls will be founded on spread footings bearing on bedrock. The proposed spread footing locations are shown on **Figure 2, Boring Location Plan**. The new bridge deck and existing approaches will be raised by less than 1 foot, and the roadway will be reconstructed between Water Street and Brown Street.

The replacement bridge is planned to be constructed along the current bridge alignment. A temporary detour will be used to allow full closure of Route 1 between Water and Brown Streets (except for access to the Cumberland Farms parking lot and fuel island) during bridge construction. A temporary bridge will be constructed for the detour, crossing the Mousam River about 200 feet south of the existing bridge and approaches. The proposed temporary bridge will be approximately 200 feet long and will be supported by two abutments and a central pier. The temporary bridge alignment has been developed by HNTB, but the bridge will be designed by an engineer retained by the Contractor. The proposed alignment, abutments and pier locations are shown on **Figure 2**.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted site visits to observe surficial conditions; and reviewed existing bridge plans, and mapped surficial and bedrock geology of the site;
- Coordinated and observed a design phase subsurface exploration program consisting of six test borings for the replacement bridge, three borings for the temporary bridge, and two pavement probes for the temporary detour;
- Conducted a laboratory testing program to evaluate engineering properties of the site soils and bedrock;
- Reviewed available historical data and evaluated seepage potential through approach embankments;
- 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; and
- Prepared this final report summarizing our findings and design recommendations.

GZA is also collecting additional geophysical data to develop final seepage mitigation alternatives and design details associated with an abandoned wooden sluiceway and other potential voids beneath the south approach embankment, in accordance with the Work Plan presented in Contract Modification 2, dated June 29, 2010. As indicated in Contract Modification 2, the results of that work will be provided to the MaineDOT / HNTB design team as the data becomes available. The results of that study are not expected to influence the geotechnical design recommendations provided herein for the proposed bridge because the sluiceway and potential voids are beyond the anticipated limits of excavation for the bridge replacement. The data collected from this work and associated modifications to the Contract Documents, if any, will be provided to the bidders as an addendum.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary subsurface exploration program in 2008 and 2009 consisting of six test borings and a Ground Penetration Radar (GPR) survey of the existing stone masonry abutment walls. GZA recently completed a design-phase exploration program consisting of nine test borings and two pavement borings.

Previous explorations were conducted at the southwest approach to explore sinkholes. In 2004, the Kennebunk Public Works Department (KPWD) solicited a geotechnical investigation and a GPR survey. Details of these exploration programs are discussed below.

2.1 PRELIMINARY TEST BORINGS

Six test borings (designated BB-KMR-101 through -106) were completed for the preliminary exploration. One boring was completed through the soil behind each abutment (BB-KMR-101 and BB-KMR-106) and two were completed in the river approximately 5 to 10 feet in front of each abutment. All of the test borings were drilled from the roadway surface using a truck-mounted drill rig. River borings were completed through existing bridge deck drains and were cased through the air to the riverbed. The borings were laid out approximately in the field by taping from existing features shown on bridge plans. The boring locations are shown on **Figure 2.**

Approximate ground surface elevation at borings drilled behind the abutments was estimated by GZA from contours on the existing bridge survey shown on MicroStation drawings provided electronically via email on December 16, 2008 by Laura Krusinski of the MaineDOT[1]. Approximate mudline elevation at river borings was estimated using deck elevations from the existing bridge drawings and subtracting the measured distance from the bridge deck to the mudline at each location. Elevations referenced in this report are in feet and refer to North American Vertical Datum (NAVD 1988). Boring locations and ground surface elevations at the borings are approximate and should be considered accurate only to the degree implied by the methods used to establish them.

The borings were drilled to depths of 10 to 30.5 feet below ground surface and were terminated in bedrock. Two-inch diameter bedrock cores were obtained at each boring location. Core lengths of 5 to 11.7 feet were drilled to assess the nature of the bedrock. New Hampshire Boring, Inc. of Derry, New Hampshire coordinated utility clearance and provided drilling services. Their work was completed between December 16, 2008 and January 5, 2009. 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 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 of the theoretical SPT. 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.

2.2 DESIGN PHASE EXPLORATIONS

A total of nine test borings (designated BB-KMR-201 through -203, BB-KMR-301 through -303, and BB-KMR-401 through -403) and two pavement probes (PC-1 and PC-2) were completed for this exploration. Each series of borings was conducted to provide data for a different element of the project, as summarized below:

- BB-KMR-200 series: Foundation design for proposed northeast and southwest retaining walls;

[1] MicroStation files received in the email correspondence include: CONTOURS_26AUG08.dgn, ORIGTOPO_26AUG08.dgn, 001_Title.dgn, Alignments.dgn, Profile.dgn.

- BB-KMR-300 series: Evaluation of seepage-related potential for South approach settlement/sinkholes;
- BB-KMR-400 series: Data for Contractor's engineer to design replacement bridge; and
- PC-series pavement probes: Data for evaluation of pavement section along the proposed Brown and Water Streets detour.

All of the test borings except BB-KMR-402 were drilled using a truck-mounted drill rig. The BB-KMR-200 and -300 series borings were drilled along the bridge approaches. BB-KMR-202 was drilled through a hole cored in the sidewalk, which is cantilevered from the existing bridge retaining wall. BB-KMR-401 and -403 were drilled in the parking lot behind Cumberland Farms and in a work area at the south end of Rotary Park, respectively. BB-KMR-402 was drilled with portable tripod-mounted drilling equipment near the south shore of the Mousam River. The locations and ground surface elevation of the borings were surveyed by MaineDOT after drilling, and surface elevations and coordinates were provided to GZA on June 10, 2010. The surveyed boring locations are shown on **Figure 2.**

The borings were drilled to depths of approximately 5 to 42 feet below ground surface and were terminated in bedrock. Two-inch diameter bedrock cores were obtained at five boring locations (BB-KMR-201 through -203, BB-KMR-401 and BB-KMR-403). Core lengths of 9.1 to 10.8 feet were collected to assess the nature of the bedrock.

Maine Test Boring of Brewer, Maine coordinated utility clearance and provided drilling services. Their work was completed between May 25 and June 8, 2010. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix C**.

The borings were drilled using 3-inch and 4-inch casing and drive-and-wash drilling techniques. SPT and split-spoon sampling were performed at 5-foot typical intervals in the borings using a rope-and-cathead pulley system and a safety hammer. Therefore, a standard energy transfer efficiency of 60 percent was assumed for the hammer-pulley system. No correction was necessary since the field N-values represent N_{60}, the standard energy of a rope and cathead system.

The pavement probes were drilled for the proposed temporary detour, to a depth of 5 feet below the ground surface using a solid-stem auger. The conditions encountered in these probes are summarized in **Section 4.6** of this report.

2.3 EXPLORATIONS BY OTHERS

R.W. Gillespie and Associates (RWG) conducted a subsurface exploration program consisting of five borings (B1 through B5). Their results were presented in a report entitled, "Sinkhole Evaluation, Route 1 between Brown Street and Mousam River Bridge, Kennebunk, Maine," dated November 9, 2004. Details of their borings are presented in their geotechnical report, which is included in **Appendix D**. The borings were drilled to depths ranging from approximately 1 to 25 feet below the ground surface. All but boring B1 were reportedly terminated in either glacial till or bedrock.

GZA scaled the locations of RWG's test borings from their Exploration Location Sketch (Figure 2); and has shown the approximate boring locations on **Figure 2**.

2.4 GROUND PENETRATING RADAR SURVEYS

In 2004, the Town of Kennebunk hired NDT Corporation of Worcester, Massachusetts to complete a GPR survey to assess the presence and extent of soil settlement indicative of developing sinkholes. The GPR study was conducted on the travel lanes of Route 1 South approach between the Mousam River Bridge and Brown Street. GPR data was collected from the street surface along transverse and longitudinal grid lines spaced approximately 5-feet on-center. The reported depth of penetration of the GPR was approximately 10 to 15 feet.

During the preliminary geotechnical exploration by GZA in 2008, the abutment face and the wing walls were surveyed with GPR to assess the extent of the existing stone masonry. The data was collected along transverse and longitudinal lines from the street surface, and along the vertical faces of the masonry wells using an Under Bridge Inspection Vehicle provided by the MaineDOT.

Reports of the 2004 and 2008 NDT Corporation GPR surveys are included in **Appendix E**.

An additional GPR and multichannel analysis of surface waves (MASW) survey is ongoing at the south approach embankment. The results of this work will provided under separate cover when available.

3.0 LABORATORY TESTING

GZA completed a laboratory soil and bedrock testing program at the GZA Laboratory in Hopkinton, Massachusetts to support visual soil classifications, evaluate frost classifications, and estimate the engineering properties of the soils and rock. The program for the preliminary borings included four gradation analysis/AASHTO Classification/Frost Classification assessments on soil samples and two unconfined compression and modulus determinations on selected bedrock samples. The program for the design-phase borings, detour pavement borings and temporary bridge borings included 17 gradation analysis/AASHTO Classification/Frost Classification assessments on soil samples. Results of the testing are included in **Appendix F**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available literature, surficial geologic units mapped in the Kennebunk Bridge area include Presumpscot Formation deposits and marine regressive sand deposits. The following are brief descriptions of the geologic units.

- The marine regressive sand deposits are described as massive to stratified and cross-stratified, well-sorted brown to gray-brown sand. This deposit is found generally with gradational basal contact to the Presumpscot Formation and is generally between 3 and 15 feet thick. These sediments were deposited during the regressive phase of marine submergence.
- 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.

Bedrock at the site is mapped as the Kittery Formation. The Kittery Formation, part of the Merrimack Group, consists of dark gray phyllite, commonly found in graded beds with fine-grained medium gray feldspathic, micaceous and calcareous quartzite.

4.2 SUBSURFACE CONDITIONS–ROUTE 1 BRIDGE AND APPROACHES

Five subsurface units were encountered above bedrock in the Route 1 Bridge and approach test borings: Pavement, Fill, Marine Deposit, Glacial Till and Stone Masonry/Rubble. Overburden soils were not encountered in the river borings. The encountered thicknesses and generalized descriptions are presented below in descending order from ground surface downward.

Detailed descriptions of the materials encountered at specific locations are provided on the boring logs in **Appendices B through D**. The soil units are also shown in relation to the bridge alignment on **Figure 3, Interpretive Subsurface Profile**. Additional information on boring locations and strata thicknesses is provided in **Table 1, Summary of Subsurface Strata**.

The asphalt pavement directly behind both abutments was generally 1-foot thick, and it was generally about 6-inches thick in other borings drilled through Route 1, except for boring BB-KMR-301 (previous excavation/patch area), where the pavement was about 17-inches thick. The asphalt was typically underlain by granular base/subbase material.

Fill was encountered in all of the borings except the river borings. The fill generally consisted of very loose to very dense, brown, fine to coarse, SAND, some to little Gravel, little to trace Silt (USCS: SP-SM, SW-SM, SM). Layers of Silty CLAY, Sandy CLAY, and GRAVEL were also encountered in the fill. Brick fragments were observed in several samples. Approximate encountered thickness ranged from 6 to 21 feet.

A series of borings were focused on locating potential voids in the South approach roadway. Borings BB-KMR-301, B2 and B5 encountered a sequence of wood and voids between depths of about 11 and 20 feet in an area that reportedly contained an abandoned wooden sluiceway. Additional discussion of the sluiceway is provided in **Section 5.1** of this report. The upper 2 to 5 feet of fill in these borings was typically medium dense to very dense, and the lower 13 to 15 feet was typically very loose to loose.

Based on grain-size analysis tests performed, the AASHTO classification for the approach fill soils are typically A-1-a, A-1-b and A-2-4, and A-3, except in boring BB-KMR-301, where A-6 and A-4 soils were encountered above the wood layers. The MaineDOT Frost Classification for the near surface portions of the approach fill soils ranges from 0 to II.

Marine Deposit – A 2-foot thick marine deposit was encountered beneath the fill in boring BB-KMR-201. The marine deposit consisted of medium stiff, mottled gray/brown, Silty CLAY, little fine Sand, with rootlets (USCS: CL). This layer appeared to be a previous near-surface deposit based on the mottling and rootlets present, but it may have been reworked.

Glacial Till – Glacial till was encountered in all of the design-phase and RWG borings except B2, B3, B5 and BB-KMR-301. The glacial till generally consisted of medium dense to very dense, brown to gray, fine to coarse, SAND, little to some Silt with cobbles and boulders; to Sandy SILT, some to little Gravel with cobbles and boulders (USCS: SM, ML). An approximately 2-

foot boulder was encountered and cored at the top of the glacial till layer in boring BB-KMR-202. Overall thickness typically ranged from 4 to 7 feet; boring BB-KMR-302 encountered approximately 17 to 18 feet.

Based on grain-size analyses, the AASHTO classification for the glacial till is typically A-4.

Stone Masonry/Rubble – In the abutment test borings (BB-KMR-101 and -106A), a layer of stone masonry/rubble was encountered below the fill. The stone masonry/rubble generally consisted of granite masonry blocks and phyllite boulders and/or highly fractured bedrock fragments. Approximate encountered thickness ranged from 2.5 to 10 feet. Samples of the stone masonry/rubble were recovered during rock coring and are described on the boring logs in **Appendix B**.

The generalized descriptions above do not include the BB-KMR-400 series borings drilled for the temporary bridge. Those test boring logs are provided for informational purposes and are included in **Appendix C**.

4.3 BEDROCK

Bedrock was cored in all of the BB-KMR-100 and -200 series test borings. Cobbles and boulders and/or bedrock were encountered at the river bed surface at all boring locations in the Mousam River. Fractured rock was encountered overlying competent rock in borings BB-KMR-200 through -203; approximate encountered thickness of the fractured rock ranged from 1 to 3 feet. Estimated top of bedrock and competent bedrock depths and elevations are presented in **Table 1**.

The primary rock type encountered was very hard to hard, fresh to slightly weathered, fine to medium grained, dark gray to gray PHYLLITE. Joints were very close to closely spaced, low angle to moderately dipping with occasional high angle to vertical fractures, planar, smooth to rough, fresh to discolored, and tight to partly open, with occasional calcite stringers and occasional silt infilling.

The Rock Quality Designation (RQD) of the encountered bedrock material ranged from 0 to 85 percent, with an average of 47. A laboratory unconfined compressive test indicated an average unconfined compressive strength of 17 ksi and an average secant modulus of 5 ksi.

Based on a review of the literature[2], it is understood that the typical shear wave velocity for metamorphic rock exceeds 5,000 feet per second.

Based on the Rock Mass Rating System, the bedrock at the Kennebunk Bridge site has an RMR of 54, placing the bedrock in Class No. III, Fair Rock, based on the bedrock compressive strength, RQD, joint spacing, condition of joints, and groundwater conditions.

The condition of exposed bedrock was observed by a GZA engineer in areas adjacent to the existing bridge and dam foundations on July 27, 2010. The exposed bedrock visible during our site visit appeared competent and intact. There was no visual evidence that the condition of the rock beneath or adjacent to existing foundations had been scoured by water flow during the life of the dam or bridge.

[2] Literature review included the USGS Handbook of Physical Constants, ASTM Guide for Using the Seismic Refraction Method for Subsurface Investigation (ASTM D 5777-00), and ASTM Guide for using Seismic Reflection method for Shallow Subsurface Investigation (ASTM D 7128-05).

4.4 GROUNDWATER

Borings BB-KMR-102 through BB-KMR-105 were drilled in the Mousam River. The water level in these borings was controlled by the river level, which fluctuates depending on upstream dam activity.

Water was introduced into the remaining borings during the drilling operations. As a result, stabilized groundwater levels were not determined. Groundwater was observed approximately 22 feet from the ground surface at BB-KMR-106A at the completion of drilling. However, wet to saturated soil samples were encountered at depths of approximately 5 feet in both abutment borings. Based on these data, groundwater levels at the abutments were interpreted to be on the order of 5 feet below existing grade at the time borings BB-KMR-101 and -106 were drilled (December 2008/January 2009). The depth to wet soil samples in the design phase borings varied from approximately 5 to 15 feet.

Groundwater levels fluctuate due to season, precipitation, infiltration, and construction activity in the area as well as river level. The groundwater levels in the approach fills are also likely influenced by the water level upstream of the dam. Therefore, groundwater levels during and after construction may vary from those encountered at the time of the test borings.

4.5 STONE MASONRY ABUTMENTS

The 2008 GPR data indicate the face of each stone masonry abutment is approximately 8 to 10 feet thick with no indication of a tapered thickness from top to bottom. The abutment wing wall data indicated the wing walls are approximately 6 feet thick with no indication of a tapered thickness from top to bottom. Based on the GPR data it appears that the walls are constructed of approximately 2-foot deep stone blocks.

The GPR did not identify significant voids behind the masonry structures but did indicate that water was present in the joints between the blocks and the back of the abutments and wing walls.

The GPR report is included in this report as **Appendix F**.

4.6 PAVEMENT CONDITIONS - BROWN AND WATER STREET DETOUR

Probes were drilled through the existing pavement on Brown Street (PC-1) and Water Street (PC-2) to evaluate the existing pavement section along the proposed temporary detour route.

PC-1 was drilled approximately 130 feet east of the intersection at Brown Street and Route 1. The probe encountered approximately 3.5 inches of asphalt pavement overlying sand and gravel fill. Laboratory gradation analysis on a sample of the fill from 1 to 3 feet below top of pavement indicated the material consists of brown, fine to coarse SAND, some Gravel, trace SILT (USCS: SP-SM). The AASHTO classification is A-1-b, and the Maine DOT Frost Classification is 0.

PC-2 was drilled approximately 100 feet east of the intersection of Water Street and Route 1. The probe encountered approximately 11 inches of asphalt pavement overlying variable fill ranging from silty fine SAND, trace Gravel with ash and cinders; to silty coarse to fine SAND, trace Gravel (probable reworked Glacial Till). The material transitioned into olive-brown silty clay at a depth of approximately 4 to 5 feet below ground surface. Due to the non-homogeneous nature of the material a representative sample was not considered available and gradation analysis was not performed.

The materials encountered in the probes are representative of materials at those specific locations. Since the roadways have likely been reconstructed or impacted by utility or other excavation and repair activities, the pavement thickness and underlying materials are expected to vary at different locations along the roadway.

5.0 ENGINEERING EVALUATIONS

5.1 SEEPAGE CONSIDERATIONS

The northbound and southbound travel lanes of the northerly and southerly approaches to the existing bridge have a documented history of sinkhole formation and partial repair. In GZA's opinion, the sinkholes have resulted from piping of granular materials within the embankment combined with the collapse of historic buried structures. Groundwater seepage flow from the upstream dam is judged to be a possible factor driving the loss of ground (piping), subsurface structure collapses and sinkhole formation. Subsurface stormwater flow from abandoned utilities has also likely contributed to the piping. Previous sinkholes have typically been repaired by filling the holes with granular material, surficial compaction and replacement of pavement.

In 2006, a grouting program attempted to fill a suspected buried wooden sluiceway beneath the south approach roadway. The current exploration program was intended to assess the area. Boring BB-KMR-301 encountered a sequence of wood and voids similar to that encountered in the 2004 RWGA borings; no flowable fill was encountered. The conditions encountered in boring BB-KMR-301 indicate that the grouting program was not completely successful, and voids still exist that could result in future sinkhole formation in the south approach roadway. Considering that this potential seepage path is about 30 feet south of the south limit of work for Retaining Wall 1, the proposed bridge construction will not include work that could improve the seepage conditions, such as excavation and replacement, in the course of construction. In our opinion, additional seepage and sinkhole mitigation measures are warranted outside of the currently proposed bridge construction as part of the bridge replacement work.

Based on the currently available information, GZA has developed details for excavation and replacement of the buried sluiceway, which are included in the Contract Documents. The anticipated sluiceway removal limits have been developed based on available historical data and the borings and are described in the Contract Documents. Sluiceway removal would include excavation of sluiceway structural elements, nearby undocumented abandoned piping, debris and fill materials within the work area, under the observation of the Geotechnical Engineer, to expose naturally deposited soil or rock. If observations indicate additional potential for future sinkholes or seepage issues adjacent to the excavation area, the excavation would be extended to remediate potential problem areas as determined by the Geotechnical Engineer. It is GZA's opinion that the potential for future sinkholes would be reduced or mitigated by this process.

GZA is currently conducting geophysical work to further explore the conditions and evaluate possible alternate remediation options that would be more appropriate and/or cost-effective than excavation and replacement. The additional data will be presented under separate cover when available. If the geophysical work allows GZA to better identify the existing conditions and/or develop a different sinkhole mitigation approach, a contract Addendum would be issued to notify bidders of the updated information and/or approach.

5.2 SCOUR CONSIDERATIONS

The proposed abutment foundations will be founded on bedrock at the river bed. As discussed in **Section 4.3**, GZA observed the condition of the bedrock surface exposed at the river bed adjacent to existing foundation elements supporting the bridge and the dam. Based on our observations, some degradation of the foundation concrete has occurred along bedrock bearing surfaces, but the observed bedrock surface had no visible indication of rock scour. Therefore, it is our opinion that intact phyllite bedrock that will support the proposed foundations is not erodible or subject to scour.

5.3 SEISMIC CONSIDERATIONS

The new abutments will be supported on spread footings bearing on bedrock. Determination of the seismic Site Class for bedrock conditions was based on the typical shear wave velocity approach in accordance with AASHTO LRFD Table C3.10.3.1-1. As discussed in **Section 4.3**, it is understood that the typical shear wave velocity for metamorphic rock exceeds 5,000 feet per second. The site was therefore assigned to Site Class A.

The United States Geological Survey program seismic design parameters Version 2.10 was used to develop parameters for use in bridge design, based on the site address and Site Class A. The recommended AASHTO Response Spectrum for a 7 percent probability of exceedance in 75 years follows:

Site Class A - Fpga = 0.80, Fa = 0.80, Fv = 0.80
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.075	As– Site Class A
0.2	0.146	SDs - Site Class A
1.0	0.036	SD1 - Site Class A

5.4 RESISTANCE FACTORS

Resistance factors herein are based on LRFD Article 10.5.5.2.3. The following table presents the resistance factors recommended for the Route 1 Kennebunk Bridge.

RESISTANCE FACTORS		
Condition	Concrete on Intact Bedrock	AASHTO LRFD Table
Strength Limit State – Bearing, φ_b	0.45	10.5.5.2.2-1
Strength Limit State – Sliding, φ_τ	0.90	10.5.5.2.2-1
Strength Limit State – Sliding Passive Earth Pressure, φ_{ep}	0.50	10.5.5.2.2-1

5.5 EVALUATION OF ABUTMENT AND RETAINING WALL FOUNDATIONS

5.5.1 Abutment and Retaining Wall Type

We understand that the new bridge abutments, retaining walls and wing walls will consist of reinforced concrete walls supported on spread footings bearing on bedrock.

5.5.2 Footing Bearing Resistance on Intact Bedrock

The new bridge abutments, retaining walls and wing walls should be founded on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed for a nominal bearing resistance, q_n, at the service limit state of 70 kips per square foot (ksf), and should be at least 3 feet wide. At the strength limit state, spread footings should be designed for a factored bearing resistance of 31 ksf (resistance factor of 0.45 applied to q_n of 70 ksf).

An irregular bedrock surface is partially exposed within the limits of the proposed Retaining Wall 1 footing area, where it supports existing bridge and stone masonry wingwall foundations. GZA's observations indicate that a near-vertical step in the rock surface probably extends longitudinally beneath the limits of the proposed footing. It is our opinion that either the rock surface will need to be leveled or concrete fill with grouted dowels will be required in order to construct a stable footing at this location. Please refer to the recommendations for bedrock footing subgrade preparation provided in **Section 6.3** of this report.

5.5.3 Overturning

Footings founded on bedrock should be checked for overturning. In accordance with LRFD Article 10.6.3.3, the resultant reaction on the base of the footing should be no further than 3/8 L from the centerline of the footing, where L is the principal dimension of the footing perpendicular to the axis of rotation.

5.5.4 Abutment Settlement

Based on the recommended bearing resistance and rock classification guidelines outlined in LRFD Article 10.6.2.4.4, we anticipate bridge foundation settlements of less than ½-inch. Settlements are expected to occur elastically as loads are applied.

5.5.5 Frost Protection

Fill soils are present at the abutments behind the existing stone masonry walls. Based on the Maine DOT Bridge Design Guide (BDG), Section 5.2.1 the Freezing Index for the site is 1250, and with low-moisture content (<10%) soils, the estimated depth of frost penetration is 6 feet.

Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection.

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 August 2008, May 2010 and July 2010 site visits.

6.2 RECOMMENDED SOIL PROPERTIES FOR USE IN DESIGN

The design calls for new reinforced concrete abutment, retaining wall and wing wall structures to be constructed. Backfill for any new structures should consist of granular borrow for underwater backfill, Maine DOT Bridge Design Guide (BDG) Type 4 soil, in accordance with Maine DOT Standard Specification Section 703.19 Granular Borrow for Underwater Backfill. Recommended soil properties for Type 4 soils for use in foundation design are as follows:

- Internal Angle of Friction of Soil = 32°
- Soil Total Unit Weight = 125 pcf
- Coefficient of Friction, tan δ (Concrete to Soil) = 0.45
- Interface Friction Angle (Concrete to Soil) = 24°
- Coefficient of Active Earth Pressure, K_a = 0.31

Granular Borrow for Underwater Backfill should be placed to a distance of 12 feet behind the back face of abutments, retaining walls and wing walls and to backfill all excavations below El. 35.

6.3 SPREAD FOOTING FOUNDATIONS

- The proposed bridge abutments may be supported on spread footing foundations bearing on sound, intact bedrock. The footings at the strength limit state should be designed for a factored bearing resistance of 31 ksf and should be at least 3 feet wide. Eccentricity of the footing reaction at the strength limit state should not exceed three-eighths of the corresponding footing dimension.
- Foundation drainage should be provided in accordance with Section 5.4.1.4 of the BDG. We recommend the use of French drains or prefabricated drainage board on the uphill side of abutments and wing walls. The drains should outlet through a series of 4-inch diameter weep holes, spaced approximately 10-feet center-to-center.
- For footings bearing on bedrock, all existing concrete, soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the subgrade. The bearing surfaces should then be washed with high-pressure water and air. It is likely that the prepared surface of the bedrock will be irregular. Concrete fill may be used as necessary to raise and level the bedrock surface to the bottom of footing level.
- Estimated top of bedrock and top of competent bedrock levels are shown on **Figure 2** and in **Table 1**. Based on the boring results, we anticipate the top of sound intact bedrock to be in the following elevation ranges:

APPROXIMATE SOUND BEDROCK BEARING LEVELS	
Foundation Elements	Estimated Top of Sound Rock Elevation (feet, NAVD 88)
Abutment 1, Retaining Wall 1, and Wingwall 1	Approximately El. 23 (east) to El. 34 (west)
Abutment 2 and Retaining Wall 2	Approximately El. 19 to El. 24
Retaining Wall 3	Approximately El. 22 to El. 24

- Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4 horizontal to 1 vertical (4H:1V) in any direction. The bearing surfaces should be dry and clean when concrete is placed.
- Where near-vertical steps are present longitudinally along footing bearing surfaces with the lower bedrock level adjacent to the river, the bedrock surface should be made level at the lower elevation or may be prepared with grouted dowels. If the bedrock level extends above the design footing bearing level, the footing may be raised and vertical reinforcement shortened in the wall. The Geotechnical Engineer should be provided the opportunity to review the exposed bedrock surface and measures proposed to enhance sliding resistance.
- For spread footing foundations bearing directly on bedrock, the lateral loads may be resisted by friction between the footing bottoms and the bedrock. The sliding resistance between new footings and bedrock subgrades should be calculated using a nominal tan δ equal to 0.7 and the appropriate resistance factor given in **Section 5.3** of this report.

6.4 PAVEMENT DESIGN

It is anticipated that the approach fills will consist of a combination of imported fill (Maine DOT Standard Specification Section 703.19 Granular Borrow for Underwater Backfill) adjacent to new concrete walls and existing fill (very loose to very dense, brown, fine to coarse, SAND, some to little Gravel, little to trace Silt) in areas where excavation is not required. Given the potential variety of approach pavement subgrade materials, GZA recommends that a subgrade resilient modulus of 4,300 psi be used for pavement design, corresponding to a soil support value of 4.0 in accordance with the BDG.

7.0 CONSTRUCTION CONSIDERATIONS

Construction considerations are intended to provide a basis for design development and to identify geotechnical-related issues that are anticipated to impact bridge construction. These items are provided in the paragraphs that follow.

7.1 TEMPORARY LATERAL SUPPORT

The portion of Route 1 between Water Street and Brown Street will be closed during construction, except for a portion of Brown Street providing access to Cumberland Farms. The existing water main that crosses the bridge will also be decommissioned. We understand that the Contractor will design a structure to temporarily support the existing communications duct bank within the proposed excavation area. Since a temporary detour is proposed, the existing bridge can be removed and the proposed bridge constructed without staged construction.

The abutment foundations and portions of the wingwall and retaining wall foundations will be constructed at or near the river level. We anticipate that a braced sheet piling system with poured concrete seals is a feasible means of temporary lateral support.

We anticipate that portions of the excavations for abutments, retaining walls and wing walls within the current roadway may be feasible using sloped open cut techniques. Excavation support may be needed in areas where sloping is not feasible due to proximity of existing structures or utilities. It is anticipated that temporary lateral support systems in these areas could consist of cantilever or braced steel sheet piling, depending on the required excavation height.

7.2 DEWATERING

Mousam River water levels may be near or above the bottom of footing levels for the abutments. We anticipate that pumping from sumps in conjunction with concrete seals could be sufficient to control seepage inflow and precipitation entering the abutment excavations. It may also be possible to use a temporary diversion of the river flow, if it is allowed by project permits. Where proposed foundations are located at greater distance from the river or above the river level, dewatering is anticipated to be feasible using sumps and open pumping.

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.

7.3 REUSE OF EXISTING EMBANKMENT FILL

Based on the test boring results and gradation analyses, the existing approach fill is heterogeneous and varies significantly in grain size distribution. If the contractor wishes to reuse excavated material as embankment fill or structural backfill, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project. In general, we anticipate that the excavated soil will be suitable for reuse as Common Borrow in accordance with Maine DOT Standard Specification Section 703.18, assuming unsuitable material is removed and moisture contents allow for compaction of the material.

TABLES

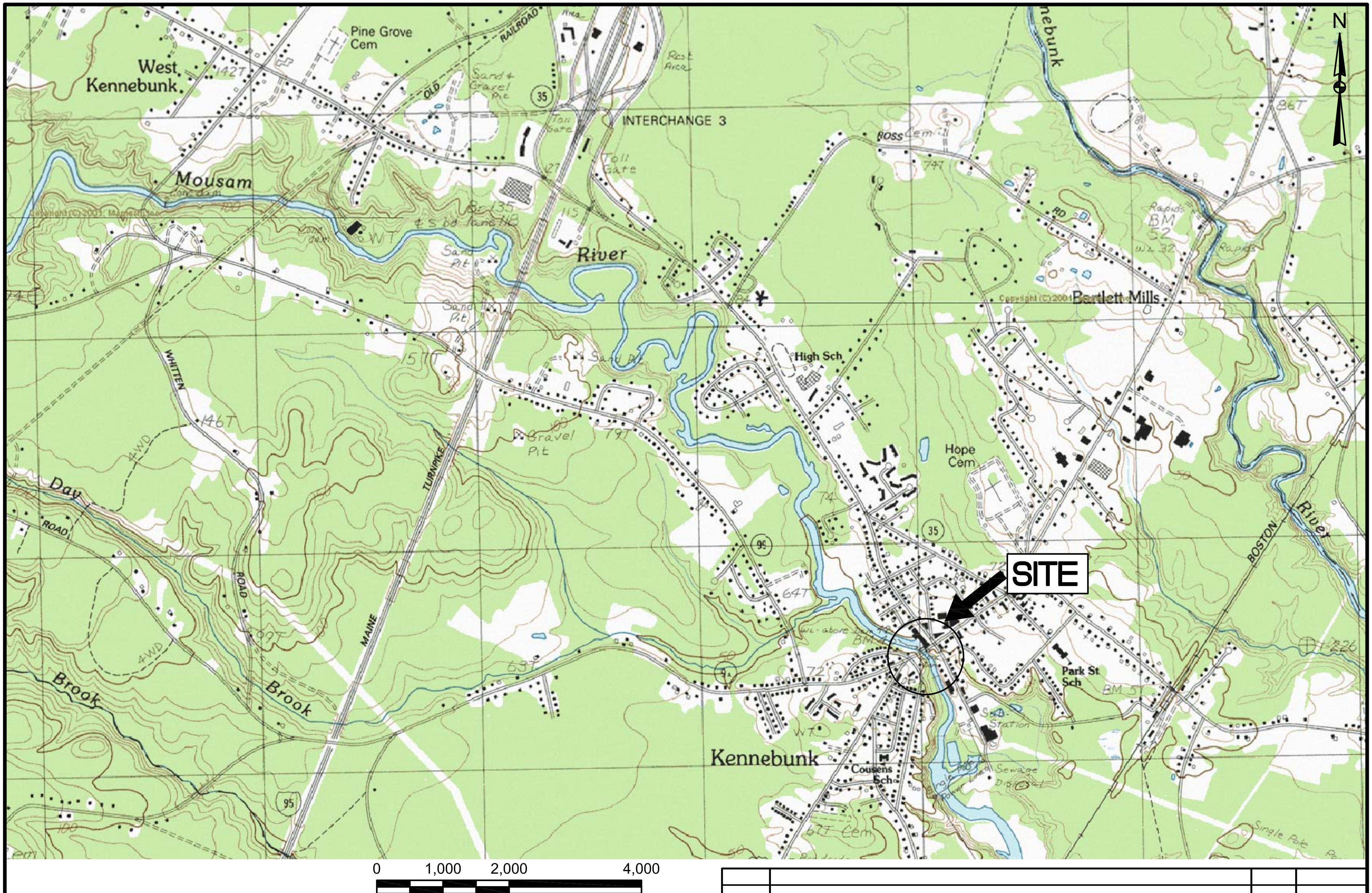
Table 1 - Summary of Subsurface Strata
Kennebunk Bridge over the Mousam River
MaineDOT PIN 15098.00

Boring Designation	Existing Ground Surface / Mudline Elevation	Location			Encountered Thickness of Strata (feet)					Estimated Top of Rock		Estimated Top of Competent Rock	
		Station, Offset	Northing	Easting	Fill	Marine Deposit	Glacial Till	Stone Masonry	Boulders / Fractured Rock	Depth (feet)	Elevation	Depth (feet)	Elevation
BB-KMR-101	47.0	15+99, 14.0' L	201,171	939,763	14.8	NE	NE	5.4	4.2	24.4	22.6	24.4	22.6
BB-KMR-102	22.2	15+78, 20.8' L	201,166	939,741	NE	NE	NE	NE	2.5	2.5	19.7	2.5	19.7
BB-KMR-103	23.2	15+79, 18.9' R	201,133	939,764	NE	NE	NE	NE	NE	0.0	23.2	0.0	23.2
BB-KMR-104	22.0	15+34, 18.6' L	201,140	939,705	NE	NE	NE	NE	NE	0.0	22.0	0.0	22.0
BB-KMR-105	22.5	15+35, 21.0' L	201,108	939,728	NE	NE	NE	NE	4.0	4.0	18.5	4.0	18.5
BB-KMR-106/106A	47.7	15+14, 13.1' R	201,103	939,705	20.5	NE	NE	4.2	NE	24.7	23.0	24.7	23.0
BB-KMR-201	48.3	14+70, 10.5' L	201,102.5	939,655.4	8.0	2.0	4.7	NE	1.0	14.7	33.6	15.7	32.6
BB-KMR-202	42.6	16+55, 20.8' R	201,174.6	939,828.5	12.0	NE	7.0	NE	1.2	19.0	23.6	20.2	22.4
BB-KMR-203	46.7	17+15, 24.7' R	201,206.9	939,879.7	6.7	NE	3.8	NE	2.7	10.5	36.2	13.2	33.5
BB-KMR-301	49.2	14+40, 12.5' L	201,089.8	939,627.2	19.6	NE	NE	NE	1.4 *	19.6	29.6	--	--
BB-KMR-302	49.6	14+25, 13.1' R	201,060.5	939,626.4	8.1	NE	17.5	NE	2.5 *	--	--	--	--
BB-KMR-303	48.3	14+25, 13.1' R	201,086.2	939,673.6	10.0	NE	4.0	NE	2.5 *	14.0	34.3	--	--

General Notes:

1. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
2. Approximate ground surface elevations at BB-KMR-100 series borings were estimated by measuring the distance from the bridge deck to the ground surface and determining bridge deck elevations based on the plans.
3. Locations of BB-KMR-100 series borings were determined approximately in the field by taping from existing site features. Coordinates were estimated from positioning of explorations in electronic files and should be considered approximate.
4. Ground surface elevations and locations of BB-KMR-200 and BB-KMR-300 series borings were surveyed after drilling by MaineDOT using GPS equipment.
5. Station and offset reference the project baseline shown on Microstation files provided by HNTB ("001_Plan.dgn, received on June 4, 2010). Coordinates West Zone coordinate system. reference the NAD83 (96) ME2000
6. "NE" indicates strata not encountered; "--" indicates rock or competent rock not confirmed in test borings.
7. Thickness of fractured rock in BB-KMR-300 series borings corresponds to estimated thickness of rock penetrated by roller cone (marked with *).
8. Prepared rock surface elevation will vary from the elevations noted in this table depending on local variation in the weathering and discontinuities in the rock, depending on the equipment used to prepare the rock surface.

FIGURES



NO.	ISSUE/DESCRIPTION	BY	DATE

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
4 FREE STREET
PORTLAND, MAINE 04101
(207) 879-9190

PREPARED FOR:
MAINE
DEPARTMENT OF TRANSPORTATION

KENNEBUNK BRIDGE REPLACEMENT

KENNEBUNK, MAINE

LOCUS PLAN

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

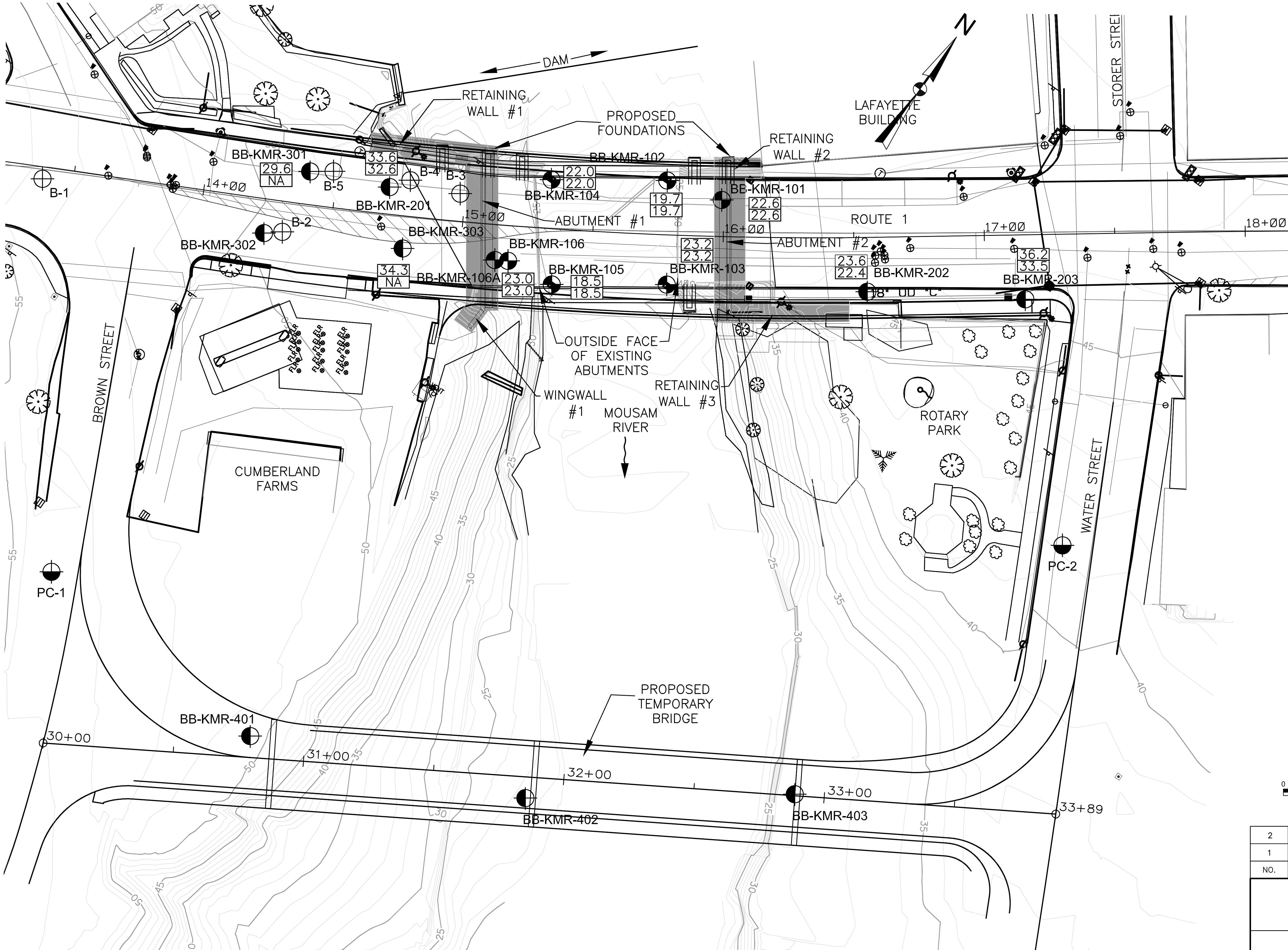
DATE
JULY 2010

PROJECT NO.
09.0025597.10

REVISION NO.

FIGURE
1

SHEET NO.



NOTES:

1) BASE MAP DEVELOPED FROM ELECTRONIC MICROSTATION FILES PROVIDED BY DONALD ETTINGER OF HNTB, TRANSMITTED VIA EMAIL ON MAY 24, 2010 AND JUNE 15, 2010 (FILES INCLUDED: 3DTOPO_10JUNE10.dgn, 001_PLAN.dgn, TOPO.dgn, ALIGNMENTS.dgn, APPROACH.dgn, CONTOURS.dgn, and 001_DETOURPLAN.dgn).

2) THE LOCATION OF THE BB-KMR-100 SERIES TEST BORINGS WERE APPROXIMATELY DETERMINED BY TAPE MEASUREMENTS FROM EXISTING TOPOGRAPHIC AND BRIDGE STRUCTURE FEATURES. THESE DATA SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

3) THE AS-DRILLED LOCATION OF THE BB-KMR-200 THROUGH THE BB-KMR-400 SERIES TEST BORINGS WERE DETERMINED BY MAINE DOT USING GPS EQUIPMENT.

4) THE LOCATION OF THE B-SERIES BORINGS WERE ESTIMATED USING A SITE PLAN IN THE R.W. GILLESPIE GEOTECHNICAL REPORT AND SHOULD BE CONSIDERED APPROXIMATE.

5) BB-KMR-200 SERIES BORINGS WERE CONDUCTED FOR FOUNDATION EVALUATION. BB-KMR-300 SERIES BORINGS WERE CONDUCTED FOR SINKHOLE EVALUATION. BB-KMR-400 SERIES BORINGS WERE CONDUCTED FOR THE PROPOSED TEMPORARY BRIDGE.

6) THE BB-KMR-100 SERIES TEST BORINGS WERE PERFORMED BY NEW HAMPSHIRE BORING, INC OF LONDONDERRY, NEW HAMPSHIRE BETWEEN DECEMBER 16, 2008 AND JANUARY 5, 2009 AND OBSERVED BY GZA PERSONNEL.

7) THE B-SERIES BORINGS WERE DRILLED BY GREAT WORKS PUMP AND TEST BORING INC. OF BERWICK, MAINE ON OCTOBER 18, 2004.

8) THE BB-KMR-200 SERIES THROUGH THE BB-KMR-400 SERIES TEST BORINGS AND PC-SERIES PAVEMENT PROBE WERE PERFORMED BY MAINE TEST BORING OF BREWER, MAINE BETWEEN MAY 25, 2010 AND JUNE 8, 2010 AND OBSERVED BY GZA PERSONNEL.

LEGEND:

BB-KMR-105 — PRELIMINARY BORING LOCATION AND DESIGNATION (DRILLED BY NHB FOR GZA)

B-2 — BORING LOCATION AND DESIGNATION (DRILLED BY GREAT WORKS FOR R.W. GILLESPIE)

BB-KMR-302 — DESIGN-PHASE BORING LOCATION AND DESIGNATION (DRILLED BY MTB FOR GZA)

PC-2 — DESIGN-PHASE PAVEMENT PROBE LOCATION AND DESIGNATION (DRILLED BY MTB FOR GZA)

33.6 — ESTIMATED TOP OF BEDROCK ELEVATION (FEET NAVD 88)

32.6 — ESTIMATED TOP OF COMPETENT ROCK ELEVATION (FEET NAVD 88); CORED BORINGS ONLY

NO.	ISSUE/DESCRIPTION	BY	DATE
2	FINAL GEOTECHNICAL REPORT	GZA	8/3/10
1	DRAFT GEOTECHNICAL REPORT	GZA	7/2/10

KENNEBUNK BRIDGE REPLACEMENT

KENNEBUNK, MAINE

BORING LOCATION PLAN

PREPARED BY:
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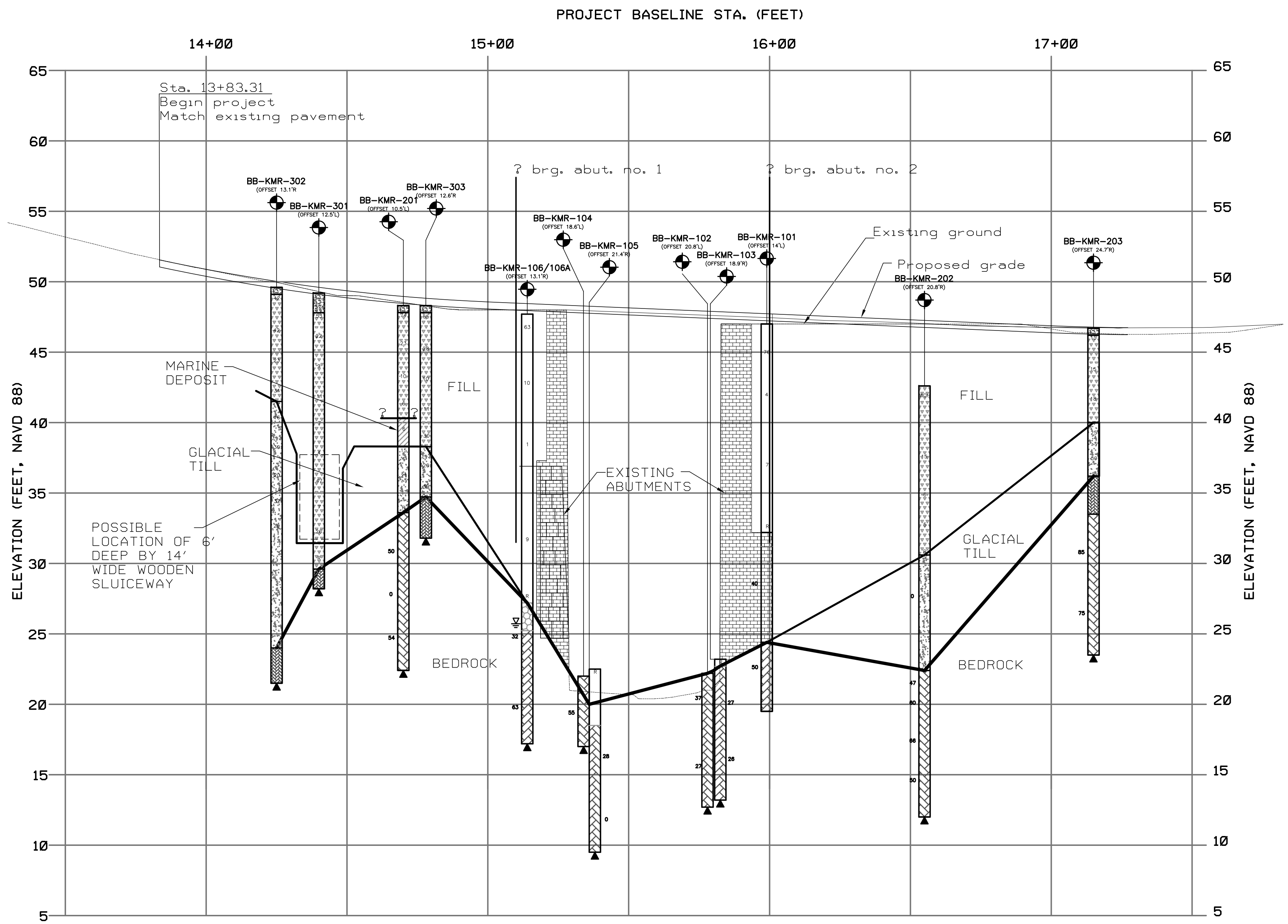
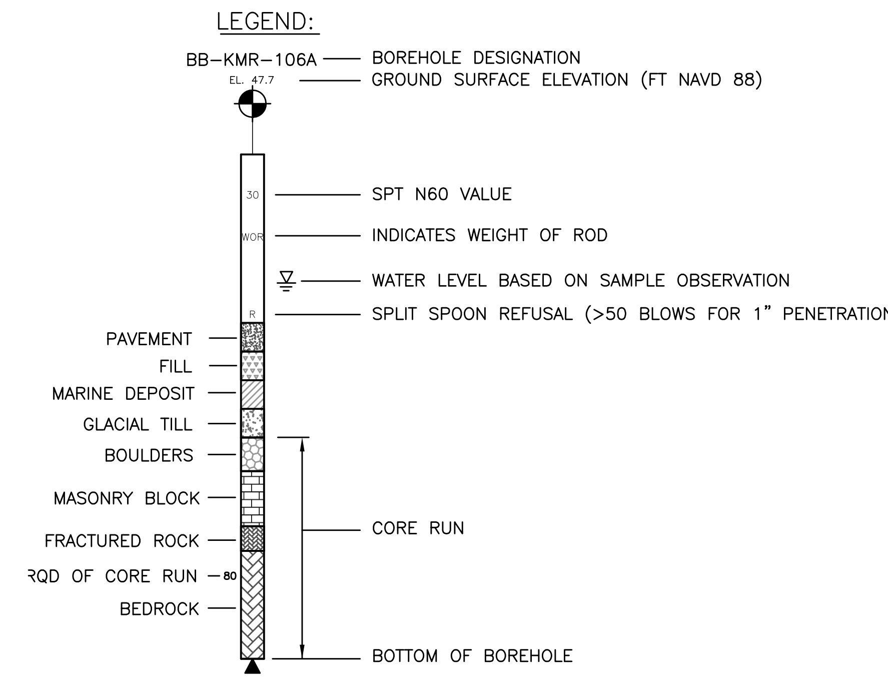
(207) 879-9190

PREPARED FOR:
MAINE DEPARTMENT OF TRANSPORTATION

PROJ MGR: CLS	REVIEWED BY: ARB	CHECKED BY: JVE
DESIGNED BY: ARB	DRAWN BY: MJD	SCALE: 1"=20'
DATE: JUNE 2010	PROJECT NO. 09.0025597.10	REVISION NO.

FIGURE 2
SHEET NO.





NOTES:

1) 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 TRANSACTIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO THE EXPLORATION LOGS.

2) UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

NO.	ISSUE/DESCRIPTION	BY	DATE
2	FINAL GEOTECHNICAL REPORT	GZA	8/3/10
1	DRAFT GEOTECHNICAL REPORT	GZA	7/2/10

KENNEBUNK MOUSAM RIVER BRIDGE

KENNEBUNK, MAINE

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY:
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(207) 879-9190

PREPARED FOR:
MAINE DEPARTMENT OF TRANSPORTATION

PROJ MGR: CLS	REVIEWED BY: ARB	CHECKED BY: JVE	FIGURE
DESIGNED BY: EJB	DRAWN BY: EJB	SCALE: AS SHOWN	3
DATE JUNE 2010	PROJECT NO. 09.0025597.10	REVISION NO.	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 Kennebunk Bridge Replacement in Kennebunk, Maine in accordance with generally accepted soil and foundation engineering practices. No Warranty, express or implied, is made.

APPENDIX B

PRELIMINARY BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-101
PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	47.0	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/19/08-01/05/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+99, 14.0 L	**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							12			Asphalt	
	1D	24/16	1.0 - 3.0	76-58-43-23	101	76	8	46.0		1.0 — Very dense, brown, fine to coarse SAND and GRAVEL, trace silt. Dry.	A-1-b, SP-SM WC=4.4%
							10				
							6				
	2D	24/10	4.0 - 6.0	7-3-2-2	5	4	10			Very loose, brown, fine to coarse SAND, Trace Gravel, Trace Silt. Moist.	A-3, SW-SM WC=10.2%
5							13				
							15				
							14				
							13				
	3D	24/0	9.0 - 11.0	WOH-3-6-18	9	7	6			No Recovery.	
10							11				
							30				
							29			See Remark 1.	
							43				
	4D	9/0	14.0 - 14.8	49-53/3"-100/0"			NQ	32.2		Split spoon refusal at 14.8 feet. No Recovery.	
15	R1	48/12	15.8 - 19.8							14.8 — 15.8' to 16.8': Hard, fresh, medium to coarse grained GRANITE. Bottom 2" Hard, fresh, fine grained gray PHYLLITE. See Remark 2. (Probable Stone Masonry) R1: Core Times (min) 15.8-16.8 (3) 16.8-17.8 (1) 17.8-18.8 (2) 18.8-19.8 (1)	
20	R2	60/40	20.2 - 25.2	RQD = 0%			NQ	26.8		20.2 — 22.6' to 24.4': Highly fractured PHYLLITE fragments. Probable top of bedrock at 24.4'.	
								22.6		24.4 — 24.4' to 25.2': Hard, fresh, fine grained, highly fractured PHYLLITE with low angle to near-vertical fractures. See Remark 4.	
25	R3	27/27	25.2 - 27.5	RQD = 50%							

Remarks:

1. Advanced roller cone through probable cobble or boulders.
2. Advanced casing to 15.0 feet; advanced roller cone 15.0' to 15.8' through possible granite block.
3. Resumed drilling on 1/5/09; roller coned bore hole to 20.2 feet to clear hole to resume rock coring.
4. R2 RQD based only on bedrock; does not include masonry block rock lengths.

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: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-101
PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	47.0	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/19/08-01/05/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+99, 14.0 L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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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.
								19.5		R2: Core Times (min) 20.2-21.2 (6) 21.2-22.2 (2) 22.2-23.2 (5) 23.2-24.2 (4) 24.2-25.2 (8) Hard, fresh, fine-grained, gray PHYLLITE. Joints are close, low angle, planar, smooth, fresh and tight to partially open. Highly fractured zone with some rust discoloration 25.2' to 25.7'. Occasional calcite stringers and banding noticeable throughout core.	
										27.5 **Bottom of Exploration at 27.50 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

1. Advanced roller cone through probable cobble or boulders.
2. Advanced casing to 15.0 feet; advanced roller cone 15.0' to 15.8' through possible granite block.
3. Resumed drilling on 1/5/09; roller coned bore hole to 20.2 feet to clear hole to resume rock coring.
4. R2 RQD based only on bedrock; does not include masonry block rock lengths.

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: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-102
PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	22.2	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	01/05/09-01/05/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+78, 20.8 L	**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							RC			Probable boulder.	
	R1	54/54	2.5 - 7.0	RQD = 37%			NQ	19.7		2.5 Hard, fresh, fine to medium grained, dark gray PHYLLITE. Joints are closely spaced, primarily low angle with occasional vertical fractures, planar, smooth to rough, fresh to slightly discolored, and partially open to moderately open. Some silt in filling. Highly fractured zone from 3.25 to 3.75 feet. See Remark 2. R1: Core Times (min) 2.5-3.5 (5) 3.5-4.5 (5) 4.5-5.5 (5) 5.5-6.5 (5) 6.5-7.0 (10)	
5											
	R2	30/30	7.0 - 9.5	RQD = 27%				12.7		Hard, fresh to slightly weathered, fine to medium grained, dark gray PHYLLITE. Joints are very closely spaced, primarily low angle with occasional vertical fractures, planar, smooth to rough, fresh to slightly discolored and partially open. Some Silt in filling. Highly fractured zone from approximately 8.0-9.0 feet. See Remark 2. R2: Core Times (min) 7.0-7.5 (2.5) 7.5-8.5 (5) 8.5-9.5 (5) 9.5	
10										**Bottom of Exploration at 9.50 feet below ground surface.**	
15											
20											
25											

Remarks:

1. Rock at 25 feet from bridge deck; advanced casing 2.0 feet into bedrock; roller cone to 2.5 feet (probable boulder from 0 to 2 feet.)
2. Highly fractured section likely the result of rock coring; the driller had difficulty with rock core and likely caused rock to become fractured.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-103
PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	23.2	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/16/08-12/18/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+79, 18.9 R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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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	R1	60/54	0.5 - 5.5	RQD = 27%			NQ			See Remark 1. Hard, fresh, fine grained gray PHYLLITE. Joints and fractures are very close to close, low angle to moderately dipping, planar, smooth to rough, fresh to discolored and tight to partially open. Thin calcite stringers throughout core. Area of larger calcite veins at approximately 3.2 to 3.5 feet. Banding noticeable throughout core. R1: Core Times (min) 0.5-1.5 (6) 1.5-2.5 (5) 2.5-3.5 (8) 3.5-4.5 (8) 4.5-5.5 (9)	
5	R2	54/42	5.5 - 10.0	RQD = 26%						5.5' to 7.5': Hard, fresh, fine grained gray PHYLLITE. Joints and fractures are very close to close, low angle to moderately dipping, planar, smooth to rough, fresh to discolored and tight to moderately wide. Banding noticeable throughout core. 7.5' to 10.0': Moderately weathered, fine grained, gray PHYLLITE. Highly fractured with discolored and decomposed rock fragments. R2: Core Times (min) 5.5-6.5 (8) 6.5-7.5 (8) 7.5-8.5 (9) 8.5-9.5 (8) 9.5-10.0 (5)	
10								13.2		10.0 **Bottom of Exploration at 10.00 feet below ground surface.**	
15											
20											
25											

Remarks:

1. Advanced roller cone into rock to seat casing for rock core.

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: Kennebunk Bridge Replacement Location: Kennebunk, ME	Boring No.: BB-KMR-104 PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	22.0	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	01/05/09-01/05/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+34, 18.6 L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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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.
0	R1	60/35	0.0 - 5.0	RQD = 55%			NQ			Hard, fresh, fine-grained, gray, PHYLLITE. Joints and fractures are close to moderately spaced, primarily low angle with occasional vertical fractures, planar, smooth, slightly discolored (rust colored near surface) to fresh and partially open to tight. Calcite stringers throughout core. R1: Core Times (min) 0-1.0 (6) 1.0-2.0 (5) 2.0-3.0 (6) 3.0-4.0 (7) 4.0-5.0 (7)	
5								17.0		5.0 **Bottom of Exploration at 5.00 feet below ground surface.**	
10											
15											
20											
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: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-105
PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	22.5	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/16/08-12/16/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+35, 21.0 L	**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	1D	0/0	0.0 - 0.0				OH			Attempted spoon sample. Encountered probable boulders or cobbles. See Remark 1. OH=Open Hole	
	R1	54/48	4.0 - 8.5	RQD = 28%			NQ	18.5		4.0 Hard, fresh to slightly weathered, fine grained, gray PHYLLITE. Joints and fractures are very close to close, low angle to moderately dipping, planar, rough, discolored, partially open to moderately wide. At approximately 5.75 to 6.5 feet, weathered zone with pieces discolored, rough and approximately 1/2 to 2 inches in size. At approximately 7.5 to 8.5, highly weathered gravel size rock and silt pieces. Occasional calcite veins throughout core. Rust discoloration in top 6 inches at joints. R1: Core Times (min) 4-5 (9) 5-6 (6) 6-7 (9) 7-8 (8) 8-8.5 (5)	
5											
	R2	54/54	8.5 - 13.0	RQD = 0%						8.5' to 10.0': Apparent open joint filled with rock fragments and sandy silt seams up to 2" thick. 10.0' to 13.0': Hard, fresh to slightly weathered, fine grained, gray, PHYLLITE. Joints are very close to close, moderately dipping to vertical, planar, smooth, fresh and tight to partially open, with continuous vertical fracture throughout. Occasional calcite stringers. R2: Core Times (mins) 8.5-9.5 (8) 9.5-10.5 (6) 10.5-11.5 (6) 11.5-12.5 (5) 12.5-13.0 (2)	
10											
								9.5		13.0 **Bottom of Exploration at 13.00 feet below ground surface.**	
15											
20											
25											

Remarks:

1. Advanced bore hole from 0 to 4 feet by roller cone and advanced the casing in 1-2 foot increments. Encountered probable boulders or cobbles. OH= Open Hole

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-106
PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	47.7	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/08-12/18/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+17, 13.1 R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.45	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
D = Split Spoon 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.
0							20			Asphalt.	
	1D	24/18	1.0 - 3.0	118-50-34-15	84	63	18	46.7		1.0 Very dense, brown, fine to medium SAND, some Gravel, Trace Silt. Dry.	A-1-b, SP-SM WC=3.5%
							12				
							12				
	2D	24/10	4.0 - 6.0	2-5-8-9	13	10	11			Loose, brown, fine to coarse SAND, trace Silt. Wet.	
5							10				
							8				
							10				
							6				
	3D	24/4	9.0 - 11.0	WOH-1/12"-45	1	1	2			Very loose, brown, fine to coarse SAND, little Gravel, trace Silt. Wet.	
10							2				
							4				
							38				
							42				
								33.7		14.0 **Bottom of Exploration at 14.00 feet below ground surface.** See Remark 1.	
15											
20											
25											

Remarks:

1. While advancing boring to 14 feet the lead casing broke off. Unable to retrieve casing and the hole was abandoned. Moved boring location south approximately 5 feet. Advanced new boring (BB-KMR-106A) to 14 feet with no sampling. See Boring NO. BB-KMR-106A for additional subsurface data.

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: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-106A
PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	47.7	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/08-12/19/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+14, 13.1 R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	22'
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)
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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.
0							RC			For upper 14 feet: See BB-KMR-106 for soil descriptions. See Remark 1.	
5											
10											
15	4D	24/5	15.0 - 17.0	WOH-4-5-8	9	7	8	32.7		15.0 Loose, brown, fine to coarse SAND, little Gravel, little Silt. Organic odor. Pockets and/or clumps of Silt.	A-2-4, SM WC=16.5%
							10				
							3				
							5				
	5D	15/5	19.0 - 20.3	20-21-10/3"-50/0			10			Medium dense, fine to coarse SAND and GRAVEL, some Silt. Split Spoon refusal at 20.3 feet.	
20	R1	60/34	20.5 - 25.5	RQD = 32%			NQ	27.2		20.5 20.5' to 22.0': Hard, fresh, fine grained, gray PHYLLITE. Joints are very close to close, low angle to moderately dipping, planar, smooth, fresh and tight with calcite veins and stringers . (Probable Stone Masonry) 22.0' to 23.0': Hard, slightly weathered, medium to coarse grained, pink GRANITE. (Probable Stone Masonry) 23.0' to 24.7': Fractured rock fragments. (Probable Stone Masonry)	
								23.0		24.7' to 25.5': Hard, fresh to slightly weathered, fine-grained PHYLLITE fragments. (Probable top of bedrock at 24.7'.) R1: Core Times (min)	
25	R2	60/54	25.5 - 30.5	RQD = 63%							

Remarks:

1. Advanced casing to 12 feet; casing refusal; roller cone from 12 to 13 feet; void under boulder or block caused rods to drop to 15 feet.
2. Water level taken at completion of drilling prior to backfilling bore 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: Kennebunk Bridge Replacement Location: Kennebunk, ME	Boring No.: BB-KMR-106A PIN: 15098.00

Driller:	New Hampshire Boring	**Elevation (ft.)**	47.7	**Auger ID/OD:**	NA
Operator:	Greg/Gerry Michael	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/08-12/19/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	St. 15+14, 13.1 R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	22'
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.
										20.5-21.5 (5) 21.5-22.5 (5) 22.5-23.5 (2) 23.5-24.5 (3) 24.5-25.5 (2)	
										24.7	
										Hard, fresh, fine to medium grained, gray PHYLLITE. Joints are close, low angle to moderately dipping, planar, smooth, fresh to discolored, tight to partially open. Calcite stringers and veins throughout core. Occasional fine sand and silt in filling (mid core depth). Upper 6 inches and bottom 12 inches: Fractures and joints surfaces are rust colored.	
30								17.2		R2: Core Times (min) 25.5-26.5 (6) 26.5-27.5 (5) 27.5-28.5 (5) 28.5-29.5 (5) 29.5-30.5 (10)	
										30.5	
										Bottom of Exploration at 30.50 feet below ground surface.	
35											
40											
45											
50											

Remarks:

1. Advanced casing to 12 feet; casing refusal; roller cone from 12 to 13 feet; void under boulder or block caused rods to drop to 15 feet.
2. Water level taken at completion of drilling prior to backfilling bore 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

DESIGN PHASE BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Kennebunk Bridge Replacement	**Location:**	Kennebunk, ME	**Boring No.:**	BB-KMR-201
				PIN:	15098.00
Driller:	Maine Test Boring	**Elevation (ft.)**	48.3	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Eric Baron	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/01/10-06/01/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 14+70, 10.5' L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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	1D	18/18	0.5 - 2.0	40-38-36	74	74	Auger	47.8		Asphalt. 0.5	SP-SM/A-1-b/0
										Brown, dry, very dense, gravelly fine to coarse SAND, trace silt. -FILL-	
	2D	24/16	2.0 - 4.0	13-16-7-6	23	23				Top 12": Same as 1D.	
								45.3		3.0 Bottom 4": Black/brown, dry, dense, fine to coarse Sandy GRAVEL, coal/wood. -FILL-	
	3D	24/14	4.0 - 6.0	6-5-5-7	10	10	12			Brown/gray, layered, moist, loose fine to coarse SAND, some Gravel, trace Silt.	
5							13				
	4D	24/15	6.0 - 8.0	5-3-3-3	6	6	13			Top 2": Brown, moist, loose, silty fine to coarse SAND, trace Gravel, nested. Bottom 14": Brown, moist, loose, fine to coarse SAND, little Gravel, little Silt, areas with fine SAND only then fine to coarse SAND, layered.	SM/A-2-4/0
							14				
	5D	24/18	8.0 - 10.0	2-2-4-6	6	6	18	40.3		8.0 Gray/brown, mottled, moist, medium stiff, Silty CLAY, little fine Sand, fine sand lenses, rootlets present. (Former surface layer/possibly reworked.) -MARINE DEPOSIT-	CL/A-6/IV
							14				
10	6D	24/21	10.0 - 12.0	6-7-8-13	15	15	14	38.3		10.0 Gray/brown, mottled, moist, medium dense, sandy SILT, little Gravel, trace Clay. -GLACIAL TILL-	ML/A-4/IV
							15				
	7D	24/22	12.0 - 14.0	21-10-11-35	21	21	22			Gray/brown, moist, medium dense, silty fine to medium SAND, trace Gravel.	
							21				
	8D	8/8	14.0 - 14.7	45-50/0.2			RC	33.6		Same as 7D with weathered rock fragments. 14.7	
15	R1	60/58	15.7 - 20.7	RQD = 50%				32.6		-FRACTURED ROCK- 15.7 Rolled to 15.7' to set casing in sound rock for coring. Gray, fine grained, metamorphic PHYLLITE, hard fresh, weak along foliation. Primary joints are low angle, close to moderate, partially open, undulating, rough, fresh, quartz/ calcite banding. Secondary joints are moderately dipping, wide, partially open. Rock Mass Quality= Fair.	
20	R2	14/13	20.7 - 21.9	RQD = 0%						Same as R1. Primary joints are very close to close.	
	R3	48/48	21.9 - 25.9	RQD = 54%						Gray, fine grained, metamorphic PHYLLITE, hard, fresh, weak along foliation. Primary joints are low angle, close to moderate spacing, partially open, undulating, rough, fresh, quartz/ calcite banding. Secondary joints are moderately dipping to high angle, close to moderate spacing, partially open, undulating, rough, fresh. Highly fractured zone from approximately 24.8- 25.9' due to breaks when removing from core barrel.	
25								22.4		25.9	

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:	Kennebunk Bridge Replacement
Location:	Kennebunk, ME
Boring No.:	BB-KMR-201
PIN:	15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	48.3	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Eric Baron	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/01/10-06/01/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 14+70, 10.5' L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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.
										Bottom of Exploration at 25.90 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-202
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	42.6	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Pisani/J. Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/26/10-05/26/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 16+55, 20.8' R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/0	0.0 - 2.0	1-WOH/18"			Auger		Dry, loose, SAND. See Note 1. -FILL-	
5	2D	24/8	5.0 - 7.0	11-7-8-3	15	15			Dark brown/black, wet, medium dense, fine to coarse SAND, trace Silt.	
	3D	24/0	7.0 - 9.0	4-3-3-10	6	6			No Recovery.	
10	4D	24/5	9.0 - 11.0	5-3-2-25	5	5			Dark brown/gray, wet, loose, GRAVEL, some fine to coarse Sand, trace Silt. Piece of wood and large gravel in spoon tip, potential wood layer between 10.5-11'. -FILL- See Note 2.	GP-GM/A-1a/0
	5D	6/6	11.5 - 12.0	30-50/0"				30.6	Dark brown, wet, fine to coarse SAND and WOOD, some Gravel, trace Silt. See Note 3. (12.0)	
	R1	60/0	12.4 - 17.4	RQD = 0%					Advanced NQ core barrel from 12.4 to 17.4 feet. No recovery. Last 3' of wash water changed to light brown. R1 Core Time (mins): 12.4-13.4 (1) 13.4-14.4 (1) 14.4-15.4 (.5) 15.4-16.4 (.5) 16.4-17.4 (.5) See Note 4. 12.4 to 14.4': Probable boulder. 14.4 to 17.4': Light brown with rust color and gray mottling, damp, medium stiff, SILT, trace Sand. -GLACIAL TILL-	
15	6D	24/20	15.0 - 17.0	2-2-6-15	8	8				
	7D	7/7	17.0 - 17.6	48-50/0.1"				23.6	Light brown with rust color and gray mottling, damp, hard, SILT, some Gravel, little Sand. Split spoon refusal at 19.0'. (19.0)	
20	MD	0/0	19.0 - 19.0	50/0"				22.4	No Recovery. Drove casing to refusal. Rolled to 20.2' through fractured rock to set casing for coring. (20.2)	
	R2	17/14	20.2 - 21.6	RQD = 47%					Gray, fine grained, metamorphic PHYLLITE, very hard, fresh. Primary joints are horizontal to low angle, close to moderate spacing, partially open, discolored, rust staining, undulating, rough. Secondary joints are steep, moderate spacing, partially open, undulating, rough, discolored, iron staining. Rock Mass Quality= Poor Core Time (mins): 20.2-21.2 (2)	
	R3	10/11	21.6 - 22.4	RQD = 60%						
	R4	61/63	22.4 - 27.5	RQD = 66%						
25										

Remarks:

1. Elevation is at ground surface. Cored through elevated sidewalk (3' thick of asphalt and concrete) to drill boring, located 4.7' above ground surface..
2. Casing refusal at 11.2' below ground surface, rolled ahead to 11.5' for sample 5D.
3. Split spoon refusal on apparent bedrock at 12.0'. Advanced roller bit from 12.0-12.4' below ground surface.
4. Borehole collapsed to 15.0' after core barrel pulled from hole. Took samples 6D and 7D in disturbed material, blow counts not representative due to disturbance.

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: Kennebunk Bridge Replacement Location: Kennebunk, ME	Boring No.: BB-KMR-202 PIN: 15098.00

Driller:	Maine Test Boring	Elevation (ft.)	42.6	Auger ID/OD:	NA
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Standard Split
Logged By:	Jennifer Pisani/J. Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	05/26/10-05/26/10	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	Sta. 16+55, 20.8' R	Casing ID/OD:	4"/4.5"	Water Level*:	
Hammer Efficiency Factor:	0.6	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.
										21.2-22.2 (3) Same as R2 with no Secondary joints. Rock Mass Quality= Fair Core Time (min): 21.6-22.6 (6) Gray, fine grained, metamorphic PHYLLITE, very hard, slightly weathered. Primary joints are low angle to moderately dipping, close, partially open, undulating, rough, discolored, Calcite present. Secondary joints are steep, close to moderate spacing, partially open, undulating, rough, fresh, Calcite present. Rock Mass Quality= Fair Core Time (min): 22.4-25.4 (3) 25.4-26.4 (2) 26.4-27.4 (3) 27.4-28.4 (2) 28.4-29.4 (2) Gray, fine grained, metamorphic PHYLLITE, very hard, fresh with slightly weathered zone at 28.2'. Primary joints are horizontal, very close to close, partially open, undulating, rough, fresh. Secondary joints are steep, wide, partially open, undulating, rough. Rock Mass Quality= Poor	
	R5	36/36	27.6 - 30.6	RQD = 50%							
30								12.0			
										30.6	
										Bottom of Exploration at 30.60 feet below ground surface.	
35											
40											
45											
50											

Remarks:

1. Elevation is at ground surface. Cored through elevated sidewalk (3' thick of asphalt and concrete) to drill boring, located 4.7' above ground surface..
2. Casing refusal at 11.2' below ground surface, rolled ahead to 11.5' for sample 5D.
3. Split spoon refusal on apparent bedrock at 12.0'. Advanced roller bit from 12.0-12.4' below ground surface.
4. Borehole collapsed to 15.0' after core barrel pulled from hole. Took samples 6D and 7D in disturbed material, blow counts not representative due to disturbance.

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: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-203
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	46.7	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Pisani/Eric Baron	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/27/10-05/27/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 17+15, 24.7' R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐	Hydraulic ☐	Rope & Cathead ☒

Definitions:
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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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
1D	18/14	0.5 - 2.0	6-7-8	15	15	Auger
2D	24/14	2.0 - 4.0	8-7-8-13	15	15	
3D	24/4	4.0 - 6.0	11-7-11-12	18	18	
4D	24/20	6.0 - 8.0	10-14-11-11	25	25	
5D	24/22	8.0 - 10.0	29-32-18-18	50	50	
6D	7/6	10.0 - 10.6	56-50/0.1'			OH
7D	2/2	12.0 - 12.2	75/0.2'			
R1	60/60	13.2 - 18.2	RQD = 85%			
R2	60/60	18.2 - 23.2	RQD = 75%			

Visual Description and Remarks

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
46.2	0.5	Asphalt.	
43.8	2.9	Brown, dry, medium dense, fine to coarse SAND, some Gravel, trace Silt. -FILL- Top 11": Brown, dry, medium dense, fine to coarse SAND, little Gravel, trace Silt. Bottom 3": Black, dry, GRAVEL/COAL. -FILL-	SP-SM/A-1-b/0
40.0	6.7	Brown, dry, medium dense, fine to medium SAND, some Gravel, little Silt. -FILL- Top 8": Brown, dry, medium dense, fine to coarse SAND, some Gravel, little Silt. -FILL-	SM/A-1-b/0
36.2	10.5	Bottom 12": Brown/gray, mottled, moist, medium dense, Silty fine SAND, trace coarse Sand/Gravel. -GLACIAL TILL- Brown/gray, mottled, dense, fine to medium SAND, some Silt, little Gravel. -GLACIAL TILL- Top 6": Brown/gray, mottled, very dense, Silty fine to medium SAND, some Gravel. Bottom: Fractured Bedrock.	
33.5	13.2	Rolled through fractured bedrock; began coring at 13.2'.	
23.5	23.2	Gray, fine grained, metamorphic PHYLLITE, very hard, fresh with moderate to severe weathered zone at 19.5'. Primary joints are horizontal, very close, partially open, undulating, rough, discolored. Secondary joints are steep, close, open, undulating, rough, discolored. Rock Mass Qualtiy=Good Core Time (min): 13.2-14.2 (2), 14.2-15.2 (3), 15.2-16.2 (2), 16.2-17.2 (2), 17.2-18.2 (2) Gray, fine grained, metamorphic PHYLLITE, very hard, fresh. Primary joints are horizontal, very close, partially open, undulating, rough, fresh. Secondary joints are steep, close, partially open, undulating, rough, discolored. Rock Mass Quality= Fair Core Time (min): 18.2-19.2 (2), 19.2-20.2 (3), 20.2-21.2 (3), 21.2-22.2 (3), 22.2-23.2 (3)	

Bottom of Exploration at 23.20 feet below ground surface.

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: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-301
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	49.2	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Eric Baron	**Rig Type:**	Mobile B 53 Truck Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/02/10-06/02/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 14+40, 12.5' L	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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							Auger			Asphalt.	
	1D	18/14	1.4 - 2.9	20-29-24	53	53		47.8		1.4 Brown, dry, very dense, gravelly fine to coarse SAND, trace Silt. -FILL-	
	2D	24/24	3.0 - 5.0	13-13-9-9	22	22				Brown/dark brown, layered, dry, medium dense, fine to coarse SAND, little Gravel, trace Silt, layering of fine to coarse sand and fine sand. -FILL-	
5	3D	24/21	5.0 - 7.0	4-5-4-5	9	9	8	44.2		5.0 Top 3": Brown, moist, loose, fine to medium SAND, layered with Sandy Clay.	
							6			Bottom 18": Gray/brown, moist, stiff, lean CLAY, some fine to medium Sand, trace Gravel, appeared reworked. -FILL-	
	4D	24/16	7.0 - 9.0	4-3-8-9	11	11	10			Gray/brown, moist, stiff, fine to medium Sandy CLAY. -FILL-	CL/A-6/IV
							13				
	5D	24/8	9.0 - 11.0	5-4-3-4	7	7	15	39.9		Top 4": Gray/brown, moist, medium stiff, fine to coarse Sandy CLAY.	
10							9			9.3 Bottom 4": Gray, moist, loose, silty fine SAND, poorly graded, non plastic, organic fibers within.	
	6D	24/9	11.0 - 13.0	4-2-2-3	4	4	10	37.5 37.3		Top 7": Blue/gray, medium stiff, fine to coarse silty SAND, brick fragments within.	SM/A-4/II
							12			11.7 Horizontal grained wood in tip.	
	7D	19/9	13.0 - 14.6	2-1-46-50/0.1'	47	47	15	36.2 36.0		11.9 Apparent void from 11.9' to 13.0'. 13.0 Top: Horizontal grained wood.	
15	8D	24/8	15.0 - 17.0	8-4-3-3	7	7	18			13.2 Bottom: Light gray/white, moist, medium SAND, little Silt, probable voids. -FILL-	SM/A-1-b/0
							20			Gray, wet, medium dense, fine to coarse SAND, some Gravel, little Silt. -FILL-	
	9D	24/10	17.0 - 19.0	1-1-25-7	26	26	21	31.5 31.3		Top 8": Gray, wet, very loose, medium SAND, little Silt, trace Gravel. Probable voids from 17 to 17.7'. 1911 penny in recovery.	SM/A-1-b/0
							35			17.7 Bottom 2": Wood, horizontal grained.	
	10D	7/1	19.0 - 19.6	30-50/0.1'			RC	29.6		17.9 Piece of Gravel/ledge.	
20								28.2		19.6 Rolled to 21' below ground surface. Consistent resistance indicates probable bedrock from 19.6 to 21.0'.	
										21.0 **Bottom of Exploration at 21.00 feet below ground surface.**	
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-302
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	49.6	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Eric Baron	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/03/10-06/03/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 14+25, 13.1' R	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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	1D	18/10	0.5 - 2.0	11-22-20	42	42	Auger	49.1		Asphalt. —0.5— Brown, dense, fine to coarse SAND, some Gravel, trace Silt. -FILL-	SP-SM/A-1-b/0
	2D	24/10	2.0 - 4.0	8-16-26-28	42	42				Top 6": Brown, dry, fine to medium SAND, little Gravel, trace Silt.	
										Bottom 4": Brown, dry, dense, Sandy GRAVEL, little Silt.	
	3D	24/2	4.0 - 6.0	18-13-10-13	23	23		45.6		—4.0— Brown, dry, dense, fine to medium SAND, trace Silt.	
5											
	4D	24/18	6.0 - 8.0	16-18-16-23	34	34	22	42.9		Top 8": Brown, moist, dense, fine to coarse SAND, some Silt, little Gravel. —6.7—	SM/A-2-4/II
							14			Bottom 10": Brown/gray, wet, dense, GRAVEL, little Silt, little Sand, angular. -FILL-	
	5D	24/24	8.0 - 10.0	8-10-6-6	16	16	18	41.5		Top 1": Brown/gray, wet, GRAVEL, little Sand and Silt, angular.	
							42			—8.1—	
10	6D	24/14	10.0 - 12.0	16-17-13-13	30	30	21			Bottom 23": Brown/gray, mottled, wet, very stiff, fine to coarse Silty SAND, little Gravel. -GLACIAL TILL-	SM/A-4/II
							26			Olive, wet, medium dense, fine to coarse silty SAND, little Gravel.	
	7D	24/24	12.0 - 14.0	10-10-10-12	20	20	OH			Olive/brown, wet, medium dense, fine to coarse SAND, some Silt and Clay, trace Gravel. Transition of color to Gray.	
	8D	24/24	14.0 - 16.0	11-12-20-20	32	32				Gray, wet, dense, fine to coarse SAND, some Silt and Clay, trace Gravel.	
15											
	9D	24/24	16.0 - 18.0	14-13-16-17	29	29	RC			Gray, wet, medium dense, fine to coarse SAND, some Silt and Clay, trace Gravel. -GLACIAL TILL-	
										Rolled ahead to 28.1'. Consistent resistance to 25.6'.	
20											
25								24.0		—25.6—	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement

Location: Kennebunk, ME

Boring No.: BB-KMR-302

PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	49.6	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Eric Baron	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/03/10-06/03/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 14+25, 13.1' R	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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.
										Probable Fractured rock (based on drill action and cuttings). Consistent resistance in probable bedrock from 25.6 to 28.1'	
								21.5		28.1	
										Bottom of Exploration at 28.10 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Kennebunk Bridge Replacement **Location:** Kennebunk, ME	**Boring No.:** BB-KMR-303 **PIN:** 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	48.3	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Eric Baron	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/03/10-06/03/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 14+78, 12.6' R	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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
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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	1D	18/14	0.6 - 2.1	27-30-35	65	65	Auger	47.7		Asphalt. (0.6) Brown, dry, very dense, gravelly fine to coarse SAND, trace Silt. -FILL- Top 6": Same as 1D.	
	2D	24/18	2.0 - 4.0	21-16-12-6	28	28		45.8		(2.5) Bottom 12": Dark brown, dry, fine to coarse SAND, some Silt, little Gravel. -FILL-	
5	3D	24/16	4.0 - 6.0	3-5-5-4	10	10				Brown, dry, loose, fine to coarse SAND, some Gravel, little Silt. -FILL-	
	4D	24/24	6.0 - 8.0	3-4-7-10	11	11	9			Brown, moist, medium dense, fine to medium SAND, some Silt, little Gravel. Chunks of Silt/loam. -FILL-	SM/A-2-4/II
							18				
	5D	24/9	8.0 - 10.0	17-19-11-6	30	30	34			Top 3": Brown, moist, dense, fine to coarse SAND, some Silt, some Gravel. Bottom 6": Gray, moist, silty fine to medium SAND, trace Gravel. Very small, horizontally grained Wood pieces within.	SM/A-2-4/II
							41				
10	6D	24/9	10.0 - 12.0	11-10-10-9	20	20	58	38.3		(10.0) Brown/gray, mottled, moist, medium dense, silty fine to medium SAND, little Gravel. -GLACIAL TILL-	
							26				
	7D	24/20	12.0 - 14.0	10-12-47-56	59	59	29			Top 13": Brown/gray, moist, medium dense, fine to medium SAND, little Silt, little Gravel. Bottom 7": Gray, wet, very dense, GRAVEL, trace Sand, trace Silt, probable fractured rock.	
							36				
	8D	5/5	14.0 - 14.4	75/0.4			RC	34.3		(14.0) Fractured Rock fragments. Rolled from 14.4 to 16.5' with consistent resistance through probable fractured rock.	
15											
								31.8		(16.5) **Bottom of Exploration at 16.50 feet below ground surface.**	
20											
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-401
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	50.4	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Pisani	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/07/10-06/07/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 30+79, 8.2' L	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
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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.
0	1D	24/20	0.0 - 2.0	1-4-3-3	7	7	AUGER			Brown, moist, loose, fine to medium SAND, some Gravel, trace Silt, top is topsoil. -FILL-	
	2D	24/8	2.0 - 4.0	5-3-3-5	6	6				Brown, damp, loose, fine to medium SAND, some Gravel, some Silt.	
	3D	24/6	4.0 - 6.0	3-2-4-15	6	6				Dark brown, moist, loose, fine to medium SAND, little Gravel, some Silt. -FILL-	
5											
	4D	24/4	6.0 - 8.0	5-4-5-8	9	9				Brown, dry, loose, fine to medium SAND, some Gravel, concrete pieces.	
	5D	24/6	8.0 - 10.0	5-5-5-11	10	10				Dark brown, moist, loose, fine to medium SAND, little Gravel, trace Silt, concrete piece in tip.	
10								40.4		10.0	
	6D	24/18	10.0 - 12.0	5-9-6-9	15	15				Black, moist, medium dense, fine to medium SAND, some Gravel, little Silt, potentially former topsoil layer. -FILL-	
								38.4		12.0	
	7D	24/14	12.0 - 14.0	7-6-5-4	11	11				Brown and yellow-brown, wet, medium dense, fine SAND, some Gravel, some Silt. -FILL-	
								36.4		14.0	
	8D	24/18	14.0 - 16.0	1-2-3-1	5	5				Brown to dark brown, wet, loose, fine Silty SAND, some Gravel, brick fragments. -FILL-	
15											
	9D	24/20	16.0 - 18.0	2-0-1-2	1	1				Brown to dark brown, very loose, fine to coarse Silty SAND, some Gravel, trace brick fragments.	
	10D	24/24	18.0 - 20.0	4-7-9-12	16	16		31.7		Top 8": Brown, saturated, medium dense, fine to coarse SAND, some Gravel, some Silt, trace brick fragments.	
										18.7	
										Bottom 12": Gray, wet, medium dense, fine to medium Silty SAND, little Gravel. -GLACIAL TILL-	
20	11D	24/24	20.0 - 22.0	12-12-8-9	20	20				Top 6": Brown to gray, wet, medium dense, fine to medium Silty SAND, some gravel, sandy silt lenses. -GLACIAL TILL-	
25	12D	24/14	25.0 - 27.0	31-65-40-50	105	105				Gray, wet, very dense, fine to coarse Silty SAND, some Gravel.	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-401
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	50.4	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Pisani	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/07/10-06/07/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 30+79, 8.2' L	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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.
										-GLACIAL TILL-	
	R1	60/13	28.7 - 33.7	RQD = 15%				21.7		28.7	
30										Casing refusal at 28.7 below ground surface; begin coring. 28.7 to 29.6'- Boulder. 29.6' to 32.0'- Soil. Gray, wet, Silty SAND, some Gravel. -GLACIAL TILL-	
								18.4		32.0	
	R2	29/5	33.7 - 36.1	RQD = 14%						Bottom 20": Gray, medium grained, metamorphic PHYLLITE, hard, fresh. Primary joints are low angle, very close to close, open, undulating, rough, fresh. Secondary joints are high angle, moderately spaced, open, undulating, rough, fresh. R1 Core Times (mins) 28.7-29.7 (2), 29.7-30.7 (2), 30.7-31.7 (1), 31.7-32.7 (2), 32.7-33.7 (2)	
35										Gray, medium grained, metamorphic PHYLLITE, hard, fresh. Primary joints are low angle, very close to close, open, undulating, rough, fresh to slightly weathered. Secondary joints are high angle, close to very close, partially open to open, undulating, rough, fresh to slightly weathered. R2 Core Times (mins) 33.7-34.7 (2), 34.7-35.7 (2), 35.7-36.1 (2)	
	R3	17/0	36.1 - 37.5	RQD = 0%						Gray, medium grained, metamorphic PHYLLITE, hard, fresh, primary joints are low angle, close to very close, partially open to open, undulating, rough, fresh to slightly weathered. Secondary joints are same, high angle. R3 Core Times (mins): 36.1-37.1 (2), 37.1-37.1 (3), 37.1-37.5 (3)	
	R4	33/16	37.5 - 40.3	RQD = 36%						R4: Same description as R2. R4 Core Times (mins): 37.5-38.5 (1), 38.5-39.5 (2), 39.5-40.3 (3)	
40											
	R5	21.5/5	40.3 - 42.1	RQD = 0%				8.3		R5: Same description as R3. R5 Core Times (mins): 40.3-42.1 (3)	
										42.1	
45										**Bottom of Exploration at 42.10 feet below ground surface.**	
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-402
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	22.6	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Pisani	**Rig Type:**	Tripod	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/07/10-06/07/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	
Boring Location:	Sta. 31+86, 8.1' R	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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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
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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	1D	24/8	0.0 - 2.0	7-4-3-7	7	7				Dark brown, damp, loose, fine SAND, some Gravel, little Silt, with organics. -TOPSOIL/FILL-	
	2D	24/18	2.0 - 4.0	10-19-23-27	42	42				Top 16": Brown, wet, medium dense SAND, some Gravel, trace Silt, brick fragments. -FILL-	
								19.3		3.3	
	3D	14/14	4.0 - 5.2	32-33-50/2"						Bottom 2": Gray, wet, very dense, GRAVEL, little Sand. -FRACTURED ROCK- Gray-brown, wet, very dense, GRAVEL, some Sand, little Silt. -FRACTURED ROCK-	
5								17.4		5.2	
										Bottom of Exploration at 5.20 feet below ground surface.	
10											
15											
20											
25											

Remarks:

1. Reached refusal resistance at 2.4', moved hole to attempt sampling again. Samples 2D and 3D were collected at second location.

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: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-403
PIN: 15098.00

Driller:	Maine Test Boring	**Elevation (ft.)**	32.4	**Auger ID/OD:**	NA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	Jennifer Pisani	**Rig Type:**	Tripod	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	06/07/10-06/07/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	Sta. 32+89, 0.6' L	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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	1D	24/20	0.0 - 2.0	1-9-4-1	13	13	Auger			Top 3": Gray-brown, moist, medium dense, Sandy SILT. -FILL- Bottom 17": Brown, damp, medium dense, fine to medium SAND, some Gravel, trace Silt. -FILL-	
	2D	24/6	2.0 - 4.0	1-1-7-2	8	8				Brown, damp, loose, fine to medium SAND, some Gravel, trace Silt.	
	3D	24/6	4.0 - 6.0	2-5-5-9	10	10				Brown and black, moist, loose, fine to medium SAND, some Gravel, little Silt, some burnt wood chips.	
5											
	4D	14/8	6.0 - 7.2	4-7-50/0.2'						Dark brown to gray, wet, medium dense, fine SAND, some Gravel, little Silt. -FILL-	
	5D	14/6	8.0 - 9.2	24-13/2"				24.2		Caved in. Top 1.5": Gray-brown, wet, dense, fine SAND, some Gravel, trace Silt. -FILL- (8.2)	
	R1	57.6/31	9.2 - 14.0	RQD = 54%				23.2		Bottom 4": Gray, dry, dense, GRAVEL, some Sand, trace Silt, fractured rock. (9.2)	
10										Gray, fine grained, metamorphic, PHYLLITE, very hard, fresh. Primary joints are horizontal, very dense, open, undulating, smooth, fresh. Secondary joints are steep, close, open, undulating, rough, fresh to slightly discolored. R1 Core Times (min): 9.2-10.2 (2), 10.2-11.2 (2), 11.2-12.2 (2), 12.2-13.2 (2), 13.2-14.2 (1)	
	R2	12/7	14.0 - 15.0	RQD = 58%						Gray, fine grained, metamorphic, PHYLLITE, very hard, fresh. Primary joints are horizontal, close, open, undulating, smooth, fresh. Secondary joints are steep, moderately close, open, rough, fresh.	
15	R3	60/20.5	15.0 - 20.0	RQD = 34%						Gray, fine grained, metamorphic, PHYLLITE, very hard, fresh. Primary joints are horizontal, very close, open, undulating, smooth, fresh, fractured zone at 19' below ground surface. R3 Core Times (min): 15-16 (2), 16-17 (2), 17-18 (2), 18-19 (2), 19-20 (1)	
20								12.4		(20.0) **Bottom of Exploration at 20.00 feet below ground surface.**	
25											

Remarks:

1. Casing encountered obstruction between 8 and 9.2'. Moved approximately 1' away from retaining wall and advanced casing to top of bedrock at 9.2' below ground surface.

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 D

GEOTECHNICAL REPORT BY R.W. GILLESPIE & ASSOCIATES, INC.:
"Sinkhole Evaluation, U.S. Route 1 Between Brown Street and Mousam River Bridge, Kennebunk, Maine," dated November 9, 2004.

R. W. Gillespie & Associates, Inc.
Geotechnical Engineering • Geohydrology • Materials Testing Services

09 November 2004

Mr. Michael Claus, P.E.
Town of Kennebunk Public Works Department
1 Summer Street
Kennebunk, Maine 04043

Subject: Sinkhole Evaluation
U.S. Route 1 Between Brown Street and Mousam River Bridge
Kennebunk, Maine
RWG&A Project No. 317-04

Dear Mr. Claus:

As requested, R.W. Gillespie & Associates, Inc., (RWG&A) has conducted a subsurface investigation at the subject site in Kennebunk, Maine. This work was undertaken in accordance with our discussions with you in October 2004. The purpose of this investigation was to obtain subsurface information in order to evaluate causal relationships between subgrade soil conditions and sinkholes which have occurred in the paved portion of U.S. Route 1 between Brown Street and the Mousam River Bridge abutment. In addition, this report provides recommendations for improvements to reduce the potential for future sinkholes to occur in this area.

Background

In September 2004, we understand that a hole occurred in the pavement near the southbound lane of Route 1 approximately 50 feet north of the intersection of Brown Street and Route 1 (see Figure 1, *Locus Map*). The hole occurred in an area of the roadway which had been patched with asphalt in the past due to subsidence of the pavement surface. Town of Kennebunk personnel excavated a test pit through the asphalt-patched area into the underlying soils to observe subsurface conditions. Town personnel reportedly observed the following features in the test pit: two 10- to 14-inch diameter, north-south oriented clay pipes in the southern sidewall of the test pit; and, a cavity in the soil (or sinkhole) which was oriented in the same general east-west direction as the pavement patch (the pavement patch being located in the area of recurring subsidence). One of the clay pipes was broken, and water introduced into the pipe was reportedly observed outletting from a pipe located at the Mousam River Bridge abutment north of the test pit.

Research by Town personnel suggested that a former wooden flume structure related to past water power activities in the area might be buried beneath the sinkhole observed in the September test pit excavation. Ground penetrating radar (GPR) measurements were conducted by NDT

Corporation for the Town on 06 October 2004. Two areas with subsurface features interpreted by NDT Corporation to be consistent with soil subsidence and possible sinkhole development were delineated between Brown Street and the Mousam River Bridge abutment. The Town decided to conduct test boring explorations with split-spoon sampling in these two areas to further explore subsurface conditions in the area of the suspected former wooden flume.

Subsurface Exploration

The subsurface exploration program for this sinkhole evaluation consisted of five test borings (B1 through B5) advanced to depths of 0.7 to 25 feet below local ground surface; test boring B1 was terminated at 0.7 feet below grade due to the potential for underground utilities near that location. Refusal was encountered in two of the borings at depths ranging from 14.9 feet (B4) to 15.9 feet (B3). The borings were advanced by Great Works Pump & Test Boring, Inc., using a truck-mounted drill rig. Approximate, as-completed test boring locations are shown on Figure 2, *Exploration Location Sketch.*

The test boring locations were selected by the Town of Kennebunk and RWG&A in the field with reference to the two areas of subsidence indicated by the GPR data, as described above, and GPR survey grid markings. Exploration locations shown on Figure 2 were located by taping from existing physical features, and these locations, as well as the Location Sketch, should be considered accurate only to the degree implied by the methods used to locate them and create the Sketch.

An RWG&A geotechnical engineer was present to log and classify the soils and prepare the exploration logs appended hereto. Soils were described using the procedures of ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Stratification lines shown on the exploration logs represent the approximate boundaries between soil types encountered; the actual transitions will be more gradual and will vary over short distances.

Subsurface Conditions

A layer of asphalt pavement which ranged in thickness from about 6 to 8 inches was encountered at the roadway surface in borings B1 through B4. An asphalt patch penetrated at boring B5 was about 2 inches thick. A layer of gravelly sand was observed in borings B2, B3, and B4 beneath the pavement layer which corresponded to the road base fill section. This layer varied from 23 inches to 38 inches thick and was very dense, with Standard Penetration Test (SPT) N-values ranging greater than 50 blows per foot (bpf). A layer of gravelly sand was also observed in boring B5; however, this layer extended to about 5.5 feet below grade and likely corresponds to backfill in the test pit made by the Town in September 2004. Below the roadway base fill layer, the subsoils generally consisted of miscellaneous silt and sand fill with gravel which contained layers of cinders and pieces of brick and organics. This layer of miscellaneous fill ranged from approximately 10.8 feet thick at B4 to 17.4 feet thick at B2. The miscellaneous fill ranged from very loose to medium dense in relative density terms, with SPT N-values ranging from 2 to 23 bpf.

Anomalous zones were observed in the miscellaneous fill from about 14 to 20 feet depth at B2 and from about 11.6 to 17.7 feet depth at B5. Drilling equipment and measuring devices dropped through these zones with little or no resistance. At the top and bottom of the anomalous zone at B5, resistance on the split-spoon sampler suggested a wood layer was present, and pieces of wood resembling lumber were observed in the recovered samples.

A layer of naturally deposited glacial till soil was observed underlying the miscellaneous fill at borings B2, B4, and B5. The glacial till was comprised of silty sand with gravel. This layer ranged from approximately 0.5 feet thick at B4 to 4 feet thick at B2. An SPT N-value of 25 bpf was obtained in the till layer at B5, indicating the till is medium dense to dense. The till overlaid a thin layer of weathered rock at B2 and B4. A thin layer of weathered rock was also observed directly underlying the miscellaneous fill at B3 (the till layer was not observed at B3). Refusal to drilling advance on probable bedrock was encountered at 14.9 feet at B4 and 15.9 feet at B3. Free water was observed at depths ranging from 9.5 to 14 feet below local ground surface in all the test borings except for B3, where free water was not apparent, and B1, which was not advanced below 0.7 feet. Refer to the exploration logs presented in Appendix A for details at specific locations.

Evaluation

After initially terminating boring B2 at 17 feet depth, the driller had difficulty backfilling the borehole; placement of the drill cuttings and three 50-lb. bags of sand down the hole did not raise it. A tape measure was placed down the hole, and, after meeting some resistance near 14 feet depth, the tape penetrated to 20 feet depth without difficulty. The borehole was then continued to its final termination depth of 25 feet. At B5, after advancing the hole to 12.5 feet depth with hollow stem augers, the drill rods inside the augers advanced beyond the bottom of the augers to approximately 17 feet depth.

The observed thickness of the zones observed at B2 and B5, approximately 6 feet thick, corresponds to the thickness, or depth, of the former wooden flume structure that the Town's research suggested might be in the area. Pieces of wood were observed in samples near the top and bottom of the zone. The Town's research suggested that the former wooden flume had been backfilled with soil; loose, wet soil was sampled in the zone. The Town's research also suggested that the wooden flume structure had been covered with a protective layer of clayey soil; a thin layer of clay was observed from about 11.3 to 11.6 feet depth in sample number S4 at B5 directly above the zone.

These observations suggest the anomalous zones observed at B2 and B5 are voids (or one continuous void) in the subsurface. Evidence observed at B5 suggests the void is bounded top and bottom by a wood layer, and evidence from both B2 and B5 suggests the void contains soil. Based on the above, we suspect the former wooden flume structure is present beneath the locations of borings B2 and B5, corresponding to the area of past pavement subsidence. The presence of a subsurface void would allow subsurface erosion of soil such as that which likely caused the pavement hole observed in September 2004.

The south abutment of the Mousam River Bridge is formed by dry laid granite blocks resting on bedrock without apparent mechanical connection to bedrock. In turn, the granite blocks support precast, prestressed concrete Tees which form the deck. Resistance to lateral movement is generated through a combination of arching in the dry laid blocks, base frictional resistance, and fixity at the deck-abutment interface. Brief observations of the abutment and deck were made as part of our work and show that the abutment does not does appear to have base slippage or to have an accentuated arch over its face. However, a sudden or significant change of subsurface conditions in the approach could result in a re-orientation of stresses in the abutment with the potential for movement of an unknown magnitude.

In view of past performance of the road; a plethora of utilities, both old and recent, and known and unknown; the erodability of subsurface soils; and the presence of voids in the flume area, appropriate remedial actions are considered important to stabilize both the road and bridge approaches. Discussions among the senior staff at this office and with you suggest that reconstruction of the road from Brown Street to the south abutment provides a positive preventative action.

Recommendations

The following recommendations are presented for your review and use in planning for remedial actions and for subsequent design.

1. Remove and replace the flume and its appurtenances, if any, and fill the resulting void with compacted structural fill or cellular concrete.

2. Re-route and/or re-connect existing utilities such that discharge is to a common point. Clay tiles which extend beyond the area of work should be closed by grouting provided they are inactive or can be made so by new connections. Grouting should consist of 1 sack of portland cement and 1 cup of powdered bentonite per five gallons of water.

3. Existing fill should be replaced with material meeting MeDOT specification 703.06 Type D or E.

4. The south abutment should be evaluated for stability since the excavation for flume removal will approach its base. Ground penetrating radar and our borings suggest the north edge of the flume may be proximal to the abutment itself.

5. Dewatering should be anticipated for most excavations below a depth of about 10 feet.

Closure

This report has been prepared for specific application to U.S. Route 1 between Brown Street and Mousam River Bridge in Kennebunk, Maine, and for the exclusive use of the Town of Kennebunk Public Works Department. This work has been completed in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made. In the event any changes are made in the nature or location of the warehouse, the conclusions and recommendations of this report should be reviewed by RWG&A.

The recommendations presented are based on the results of widely spaced explorations. The nature of variations between the explorations may not become evident until construction. If variations are encountered, it will be necessary for RWG&A to re-evaluate the recommendations presented in this report. RWG&A requests an opportunity for a general review of the final design and specifications in order to determine that earthwork and foundation recommendations have been interpreted in the manner in which they were intended.

If you have any questions, please contact us.

Very truly yours,
R. W. GILLESPIE & ASSOCIATES, INC.

Scott R. Dixon, P.E., C.G.
Geotechnical Engineer

Robert W. Gillespie, P. E.
Chairman

STATE OF MAINE
ROBERT W. GILLESPIE
4975
REGISTERED
PROFESSIONAL ENGINEER

SRD/RWG:ci
In four copies plus electronic copy

Attachments: Figure 1. Locus Map
Figure 2. Exploration Location Sketch
Appendix A. Test Boring Logs

G:\PROJECTS\0300\0317\0317-004\Reports\09Nov04SinkholeEvaluation.wpd

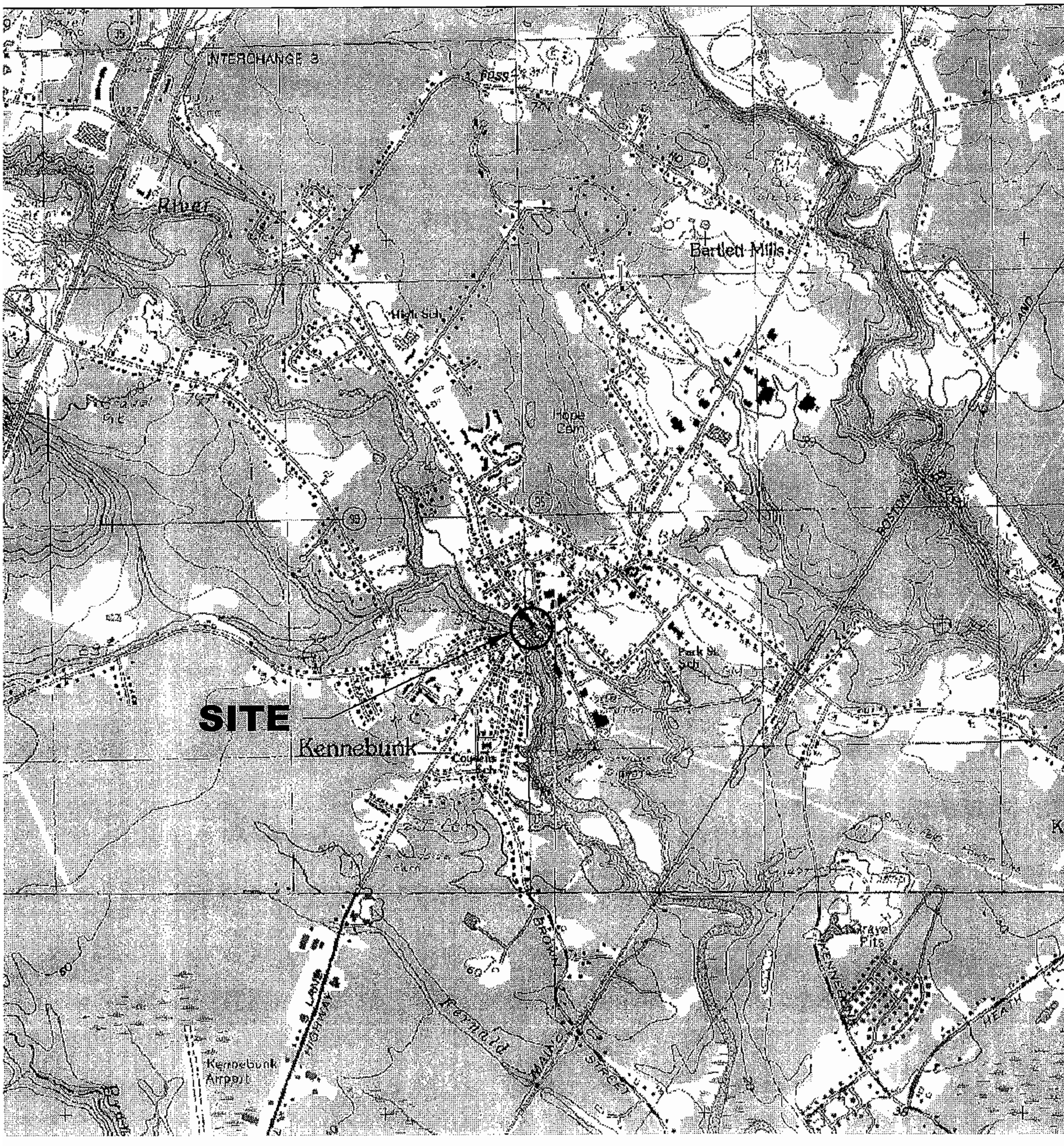

FIGURE 1
LOCUS MAP
ROUTE 1 SINK HOLE EVALUATION
KENNEBUNK, MAINE

N

0 2000 3000 4000

SCALE, FEET

SOURCE:

USGS 7.5-MINUTE TOPOGRAPHIC QUADRANGLE OF KENNEBUNK AND WELLS, DATED 1990.

NOVEMBER 2004

PROJECT NO. 317-04

R.W. Gillespie & Associates, Inc.
CONSULTING GEOTECHNICAL & ENVIRONMENTAL SPECIALISTS
86 Industrial Park Rd., Suite 4 Saco, Maine 04072 (207) 286-8008
Fax: (207) 286-2882 E-mail: rwg-a@rwg-a.com

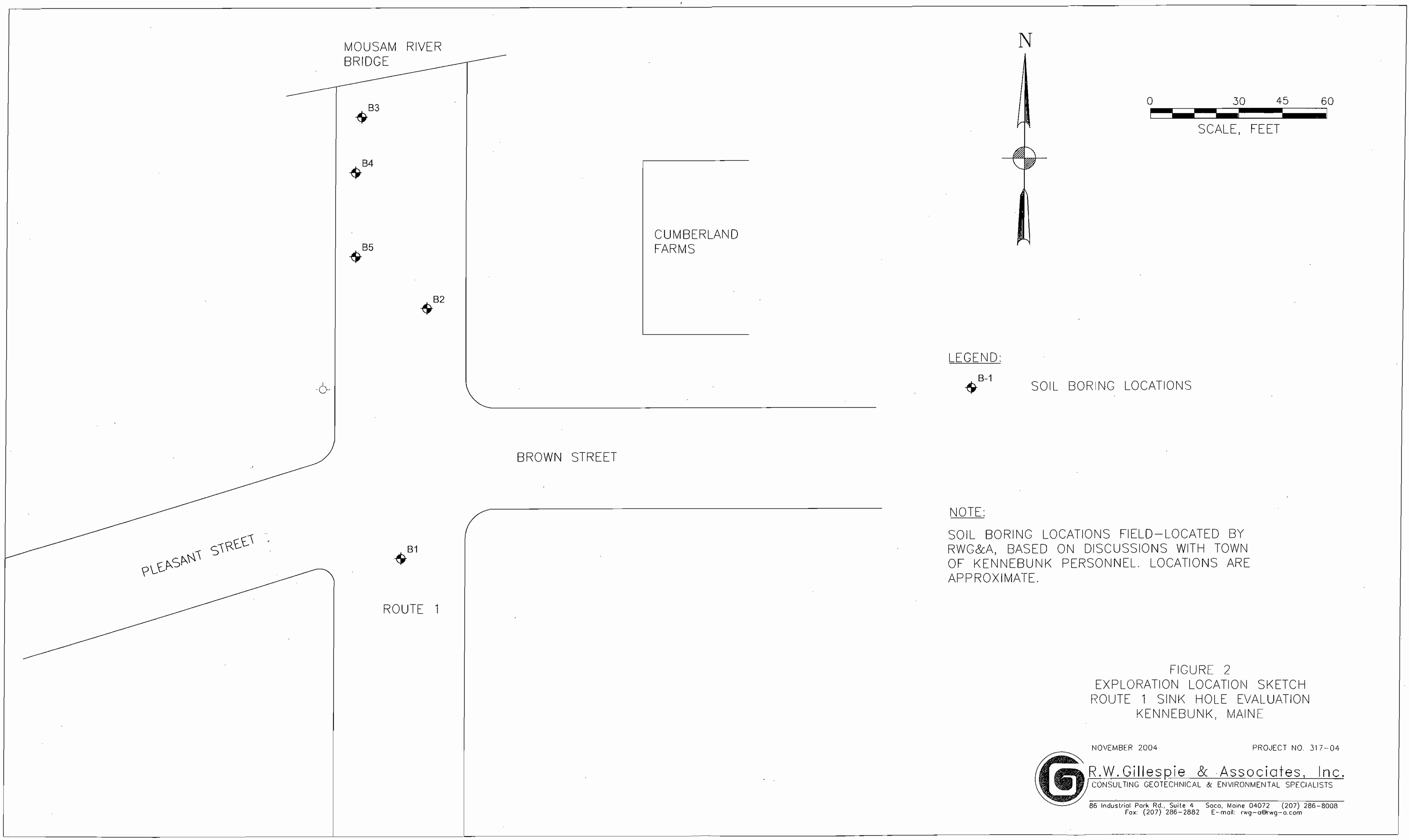

FIGURE 2
EXPLORATION LOCATION SKETCH
ROUTE 1 SINK HOLE EVALUATION
KENNEBUNK, MAINE

NOVEMBER 2004 PROJECT NO. 317-04

R.W. Gillespie & Associates, Inc.
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86 Industrial Park Rd., Suite 4 Saco, Maine 04072 (207) 286-8008
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APPENDIX A

TEST BORING LOGS

Sinkhole Evaluation
U.S. Route 1 Between Brown Street and Mousam River Bridge
Kennebunk, Maine

R.W. Gillespie & Associates, Inc.
Geotechnical Engineering• Geohydrology•Materials Testing Services

Project:	Sink Hole Evaluation	Boring Log:	B-1
Location:	Kennebunk, Maine	Surface Elevation:	
Client:	Town of Kennebunk	Observed Water Depth:	Not Obs.
Project No.	317-04	Date Completed:	10/18/04

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
0				ASPHALTIC PAVEMENT.					
				Bottom of Exploration at 0.7'; Terminated due to proximity of underground utilities. Borehole backfilled with granular soil, tamped, and layer of cold patch asphalt placed to ground surface.					
5									
10									
15									
20									
25									
30									
35									

R.W. Gillespie & Associates, Inc.
Geotechnical Engineering• Geohydrology•Materials Testing Services

Project:	Sink Hole Evaluation	Boring Log:	B-2
Location:	Kennebunk, Maine	Surface Elevation:	
Client:	Town of Kennebunk	Observed Water Depth:	10
Project No.	317-04	Date Completed:	10/18/04

DEPTH, FT.	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
0		ASPHALTIC PAVEMENT (8 inches).					
	S-1	GRAVELLY SAND (SW); Very dense, damp, gravelly sand, trace silt, brown.	19	18 38 21 17	59		
	S-2	GRAVELLY SAND (SW); Very dense, damp, gravelly sand, little to trace silt, lenses of brown and dark gray-brown soil.	16	23 30 29 24	59		
5	S-3	SAND WITH SILT AND GRAVEL (SM); Medium dense, damp, sand, little silt and gravel, brown.					
	S-4	-FILL-	14	9 6 5 6	4		
	S-5	SILTY SAND (SM); Medium dense to loose, moist, medium to fine sand, some silt, trace gravel, gray-brown.	21	3 3 2 2	5		
10	S-6	SILTY SAND WITH CLAY (SM); Very loose, wet, silty sand, little clay and gravel, with pieces of brick, trace organics, gray.	7	2 1 1 1	2		
15	S-7	SILTY SAND WITH CLAY (SM); Loose, wet, silty sand, little clay, trace gravel, gray with pieces of angular rock and light-colored wood fibers. -FILL IN PROBABLE VOID-	4	1 2 3 2	5		
20		Auger resistance indicates coarse-grained soil at 20'. -PROBABLE GLACIAL TILL-					
24		Very hard auger resistance at 24'. -PROBABLE WEATHERED ROCK-					
25		Bottom of Exploration at 25'; Not refusal. Borehole backfilled with granular soil, tamped, and layer of cold patch asphalt place to ground surface.					
30							
35							

R.W. Gillespie & Associates, Inc.
Geotechnical Engineering•Geohydrology•Materials Testing Services

Project: Sink Hole Evaluation
Location: Kennebunk, Maine
Client: Town of Kennebunk
Project No. 317-04

Boring Log: B-3
Surface Elevation:
Observed Water Depth: Not Obs.
Date Completed: 10/18/04

DEPTH, FT.	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
0	S-1	ASPHALTIC PAVEMENT (6 inches). GRAVELLY SAND (SW); Very dense grading to medium dense, damp, gravelly sand, trace silt, brown.	18	33 44 38 30	82		
	S-2		16	23 12 9 10	21		
5	S-3	FILL; Medium dense, damp, CINDERS, trace amount of medium to fine sand, black and gray. SAND WITH GRAVEL AND SILT (SP); Medium dense grading to loose, dry to damp, medium to fine sand, trace gravel and silt, with trace organic fibers and angular gravel, brown and dark brown.	15	6 15 8 8	23		
	S-4		16	6 4 3 4	7		
10	S-5	-FILL-	8	13 8 6 10	14		
	S-6		6	12 9 13 8	22		
15	S-7	Moist. WEATHERED ROCK; Very dense, rock fragments, dry, dark gray.	4	26/ 0.4" 50/0"	26+		
		Bottom of Exploration at 15.9'; Auger refusal on probable bedrock.					
		Borehole backfilled with granular soil, tamped, and layer of cold patch asphalt placed to ground surface.					
20							
25							
30							
35							

R.W. Gillespie & Associates, Inc.
Geotechnical Engineering• Geohydrology •Materials Testing Services

Project:	Sink Hole Evaluation	Boring Log:	B-4
Location:	Kennebunk, Maine	Surface Elevation:	
Client:	Town of Kennebunk	Observed Water Depth:	9.5
Project No.	317-04	Date Completed:	10/18/04

DESCRIPTION OF MATERIAL

ASPHALTIC PAVEMENT (6 inches).

GRAVELLY SAND (SW); Very dense, dry to damp, gravelly sand, trace silt, brown.

Medium dense, damp, CINDERS, black and gray.

SAND WITH GRAVEL AND SILT (SM); Medium dense to loose, damp, medium to fine sand, trace to little silt and gravel, brown, light brown, and orange-brown.

SAND WITH SILT (SM); Very loose to loose, moist, medium to fine sand, some silt, trace clay, brown and light orange-brown, with angular rock fragments, plant stem, and wood fibers, stratified.

-FILL-

SILTY SAND WITH GRAVEL (SM); Dense, wet, sand, some silt and angular rock fragments, brown mottled.

-GLACIAL TILL-

WEATHERED ROCK; Very dense, dry, rock fragments, dark gray and orange-brown.

Bottom of Exploration at 14.9'; Spoon refusal on probable bedrock.

Borehole backfilled with granular soil, tamped, and layer of cold patch asphalt placed to ground surface.

SAMPLE NUMBER	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
S-1	18	35, 62, 47, 51	108		
S-2		35, 15, 11, 9	26		
S-3	8	7, 13, 8, 5	21		
S-4	12	3, 2, 2, 2	4		
S-5	6	11, 3, 3, 2	6		
S-6	6	3, 4, 3, 2	7		
S-7	10	42, 52/ 0.4", 50/0"	94+		

R.W. Gillespie & Associates, Inc.
Geotechnical Engineering • Geohydrology • Materials Testing Services

Project:	Sink Hole Evaluation	Boring Log:	B-5
Location:	Kennebunk, Maine	Surface Elevation:	
Client:	Town of Kennebunk	Observed Water Depth:	14
Project No.	317-04	Date Completed:	10/18/04

DEPTH, FT.	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
0	S-1	ASPHALTIC PAVEMENT (2 inches). GRAVELLY SAND (SW); Medium dense, damp, gravelly sand, trace silt, brown.	19	11 18 13 11	21		
5	S-2		5	1 1 2 2	3		
	S-3	SILTY SAND (SP); Very loose, wet, medium to fine sand, little silt, dark brown, occasional rock fragments and seams of black soil, brown, grading at 7.5' to very loose, wet, silty fine sand, little clay, dark brown and gray, with particles of ceramic pipe. -FILL-	12	2 1 1 1	2		
10	S-4		8	48 11 5 13/3" 50/0"	16+		
		SILTY CLAY (CL); Stiff, wet, silty clay, gray. Wood layer from approximately 11.6' (Spoon refusal at 11.7').					
15		SAND (SP); Very loose, wet, medium to fine sand, brown and dark brown. -FILL IN PROBABLE VOID-					
	S-5	Wood layer from approximately 17.7' to 17.8'.	16	2 10 15 13	25		
20		SILTY SAND WITH GRAVEL (SM); Medium dense to dense, wet, silty sand, little gravel and clay. -GLACIAL TILL-					
		Bottom of Exploration at 19'; Not refusal. Borehole backfilled with granular soil, tamped, and layer of cold patch asphalt placed to ground surface.					
25							
30							
35							

APPENDIX E

GEOPHYSICAL INVESTIGATIONS BY NDT CORPORATION:

1. "GPR Sinkhole Investigation, US Rt 1, Kennebunk, Maine," dated October 11, 2004.
2. "Ground Penetrating Radar, Masonary Bridge BR#2431, US Rt 1 over the Mousam River, Kennebunk, Maine," December 29, 2008.

F C4292

GPR SINKHOLE INVESTIGATION

US RT 1
KENNEBUNK, MAINE

Prepared for

KENNEBUNK PUBLIC WORKS

OCTOBER, 2004

NDT CORPORATION

NDT CORPORATION

October 11, 2004

Mr. Michael Clause
Kennebunk Public Works
1 Summer Street
Kennebunk, Maine 04043

Dear Mr. Clause:

In accordance with authorization to proceed, NDT Corporation conducted ground penetrating radar (GPR) measurements in both the North and Southbound lanes of US Route 1 between Brown Street and the Mousam River Bridge. The Purpose of the GPR investigation was to identify the presence and extents of soil settlement indicative of developing sinkholes that may exist in this area. Fieldwork was conducted on October 6, 2004. This report presents the results and findings of our investigation.

METHOD OF INVESTIGATION
Survey Control
The general location of the GPR survey is shown on Figure 1. Figure 2 is a sketch plan of the site showing the location of GPR lines and results of the survey. GPR lines were referenced to a fire hydrant along the western curb/sidewalk of Route 1 across from the intersection of Brown Street. Forty-eight cross lines were collected(24 at 60 nanoseconds and 24 at 120 nanoseconds); the first of these was located 15 feet south of the hydrant and subsequent lines were collected at a 5 foot spacing for 100 feet North of the hydrant. Cross lines began at the west edge of the western sidewalk and ended near the eastern edge of the eastern sidewalk. Five longitudinal lines (western curb, middle of southbound lane, centerline, middle of northbound lane and western curb) of data were also collected along Route 1 from North to South (100 feet North of the hydrant to 15 feet South of the hydrant) beginning 100 feet north of the hydrant and ending 15 feet south of the hydrant.

Ground Penetrating Radar (GPR)
GPR data were acquired using a digital system coupled with a 400 MHz antenna. The GPR method uses a pulsed electromagnetic signal that is transmitted to and reflected by a target back to the point of transmission. The electromagnetic wave transmission and reflection is dependent on the dielectric constant and conductivity (electrical) properties of the material(s) being investigated. These electrical properties are highly dependent on moisture content, saturated or concentrated moist conditions provide both strong reflections and high attenuation. A detailed discussion of the GPR Survey Method is included in Appendix 1.

DISCUSSION OF RESULTS
Indicators of sinkhole and/or soil settlement using GPR are: 1) sloping or draped marker layers, 2) broken or disturbed marker layers, 3) areas of high conductivity/high moisture content relative to sandy host materials, and/or 4) areas of low conductivity/low moisture content relative to silty/clay host materials. Filled utility trenches and old excavations may have similar characteristics; therefore the data was correlated to known utility locations painted on the road previous to the investigation. A highly conductive layer at approximately 15 nanoseconds (2.5 to 3 feet in depth) was used as a marker layer for this investigation. At this site the GPR investigation had an approximate depth of penetration of 10 to 15 feet, and detected the buried water line, and indications of subsidence.

Figure 2 shows two types of anomalies associated with soil subsidence and possible sinkhole development marked on each individual GPR line;

> 1) Marked as purple ovals on Figure 2, GPR data at these locations indicate disturbances in the soil layering, such as dipping or broken layers. These disturbances may also be abandoned utility trenches, reworked soil for road construction or previously filled sinkholes.
>
> 2) Marked as orange squares on Figure 2, GPR data at these locations indicate higher moisture content in soils at depths at or greater than 7.5 feet. Higher moisture conditions may be the result of loose soil conditions caused by soil settlement or may be saturated timbers (sluiceway).

Two areas, marked as red on Figure 3, have been delineated as areas GPR data indicates there may be soil subsidence and possible sinkhole development. Area 1 extends from approximately 10 feet north of the fire hydrant to approximately 35 feet north of the fire hydrant, and extends the width of the road. This area, in the general location of the clay drain pipe and encompasses the previous sinkhole, is characterized by dipping and broken layering. Area 2 extends from 50 feet north of the fire hydrant to approximately 80 feet north of the fire hydrant and extends the width of the road. This area is characterized by dipping and broken layering over areas of high moisture content. It is believed due to the location and characteristics of the anomalies in this area that the old wooden sluiceway may be located in this area as delineated by the dashed black lines on Figure 3.

The results of the GPR investigation should be verified and therefore NDT recommends that several locations be sampled with a split spoon probe.

1) 12 Feet North of hydrant and 15 feet East of west edge of sidewalk
2) 22 Feet North of hydrant and 36 feet East of west edge of sidewalk
3) 72 Feet North of hydrant and 36 feet East of west edge of sidewalk
4) 80 Feet North of hydrant and 25 feet East of west edge of sidewalk

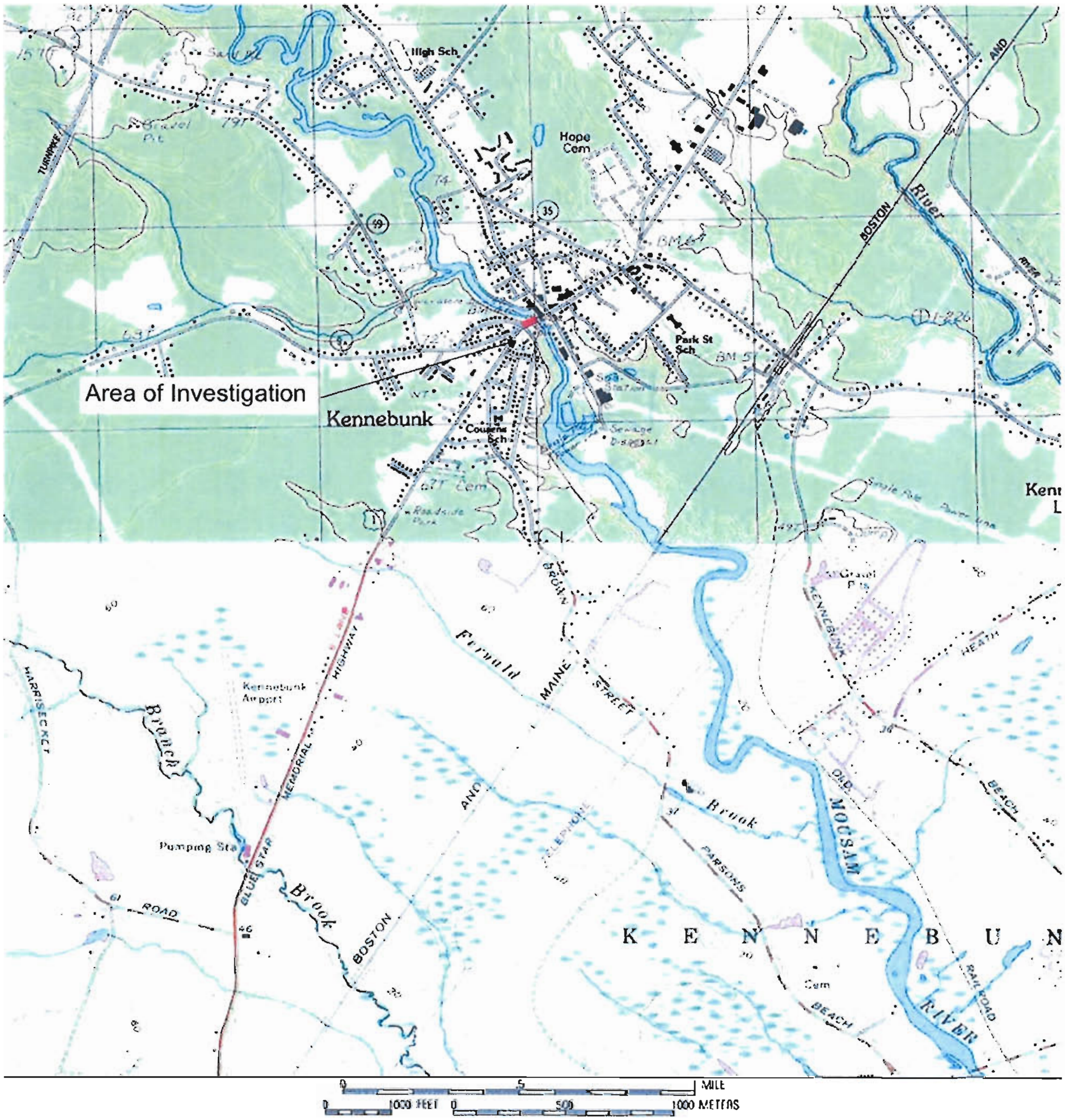
A separate location should also be sampled outside of the reported settlement areas to use as a baseline for the other test locations. It is recommended that these probes extend for at least 10 feet of depth.

If you have any questions, please contact the undersigned at 508-754-0417.

Sincerely,
NDT CORPORATION

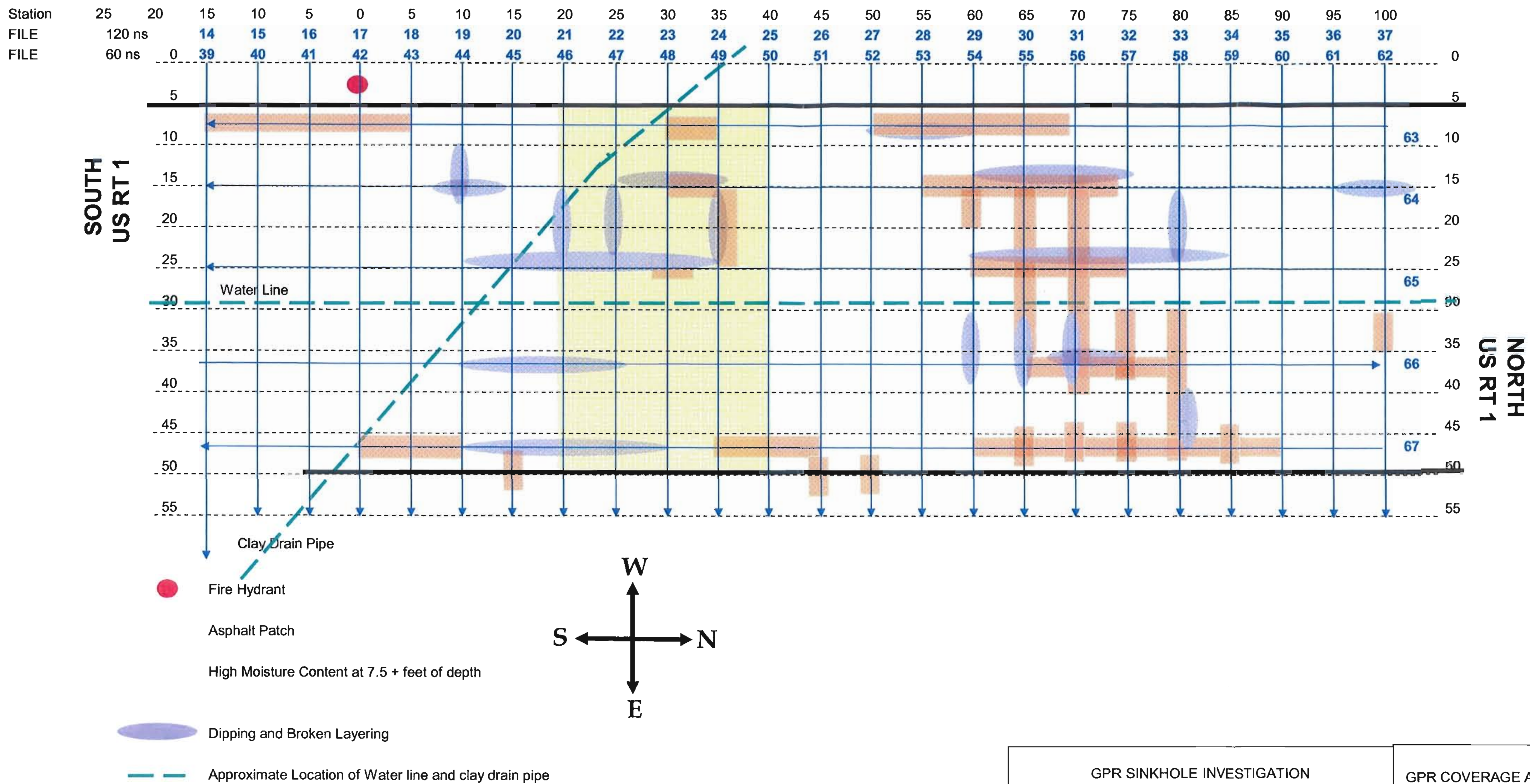
Paul S. Fisk

FIGURES

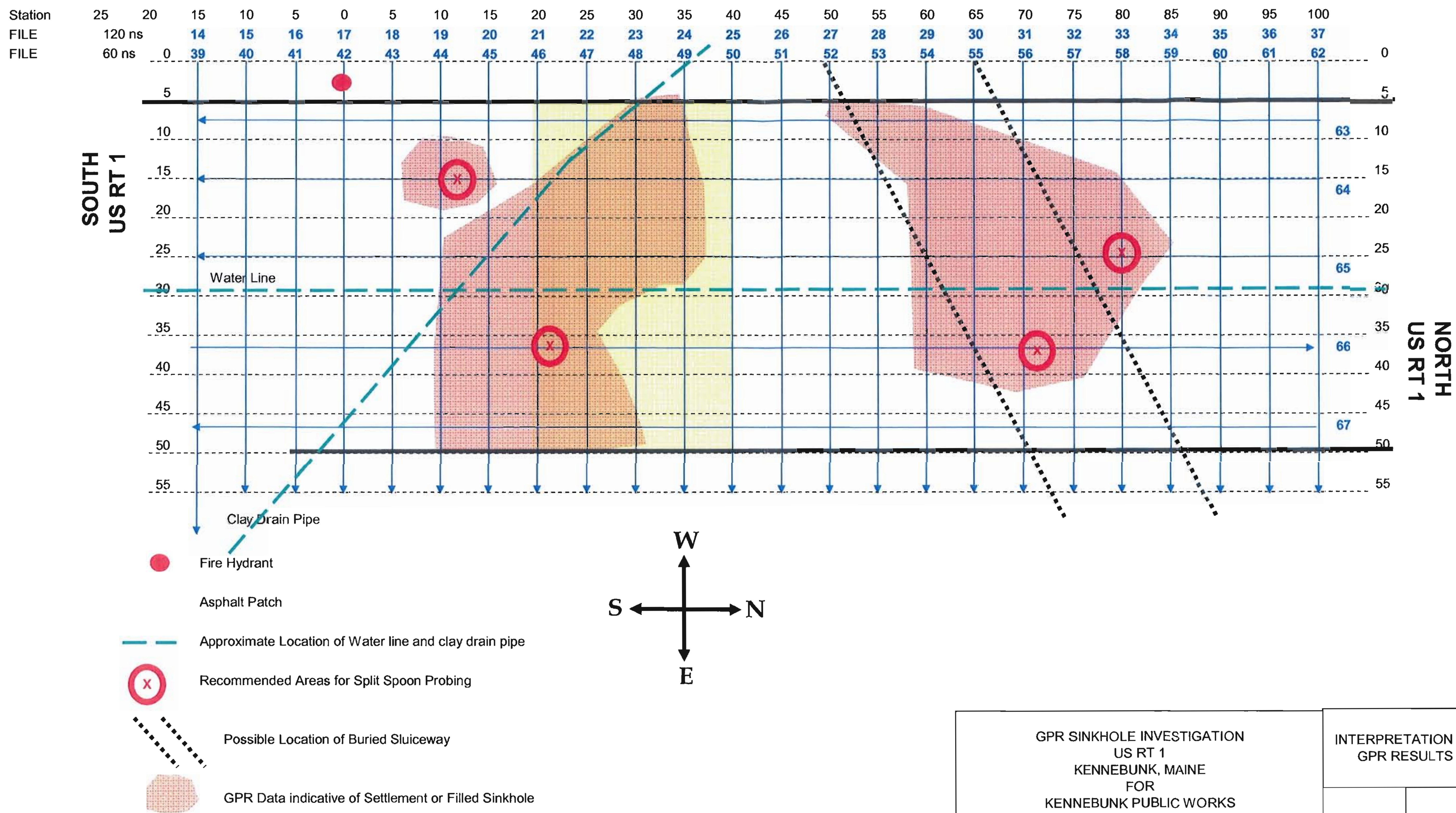


Map created with TOPO!® ©2003 National Geographic (www.nationalgeographic.com/topo)

GPR SINKHOLE INVESTIGATION US RT 1 KENNEBUNK, MAINE FOR KENNEBUNK PUBLIC WORKS By NDT CORPORATION	AREA OF INVESTIGATION	
	Oct. 2004	Figure 1

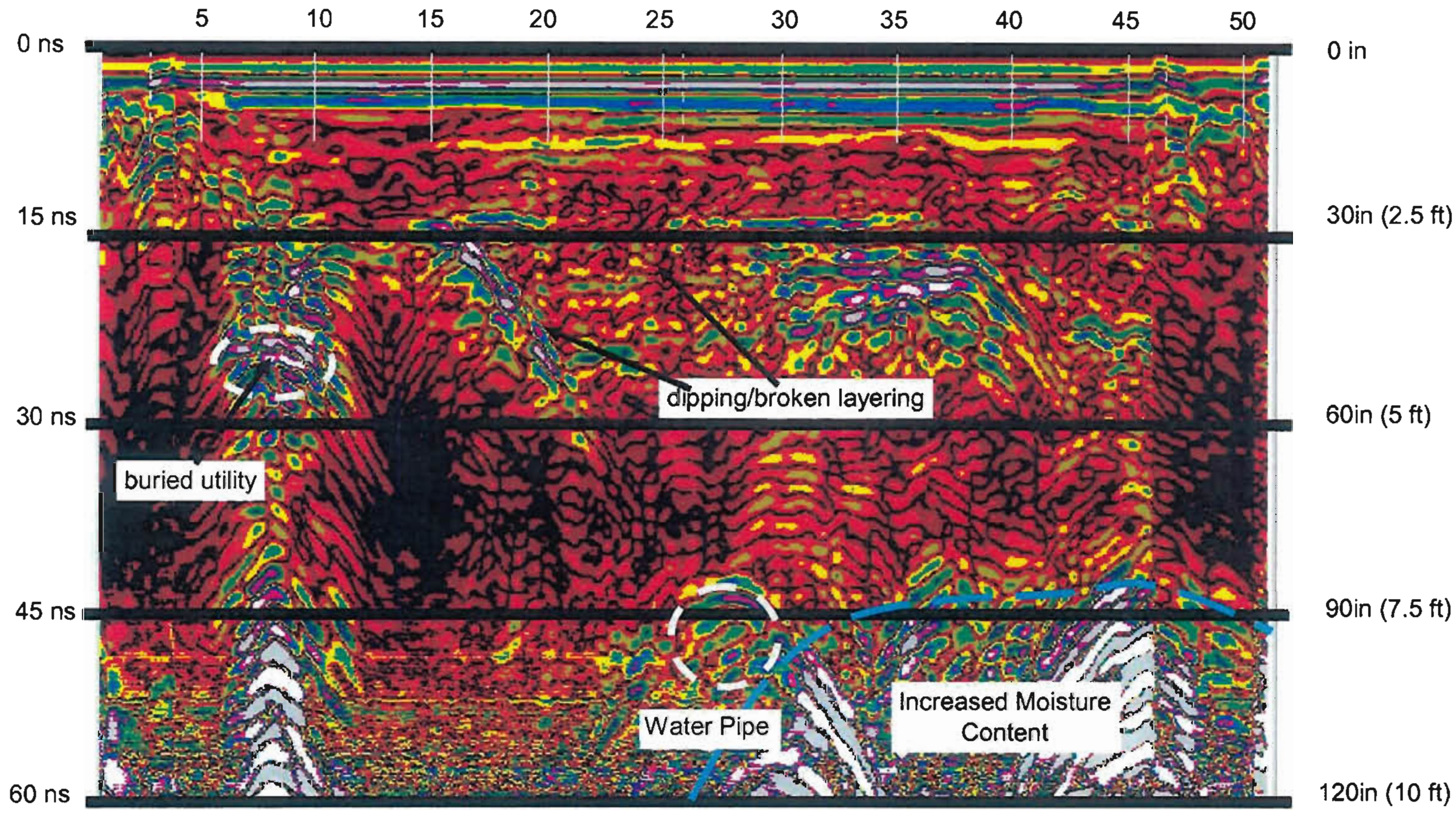


GPR SINKHOLE INVESTIGATION US RT 1 KENNEBUNK, MAINE FOR KENNEBUNK PUBLIC WORKS By NDT CORPORATION	GPR COVERAGE AND RESULTS	
	Oct. 2004	Figure 2

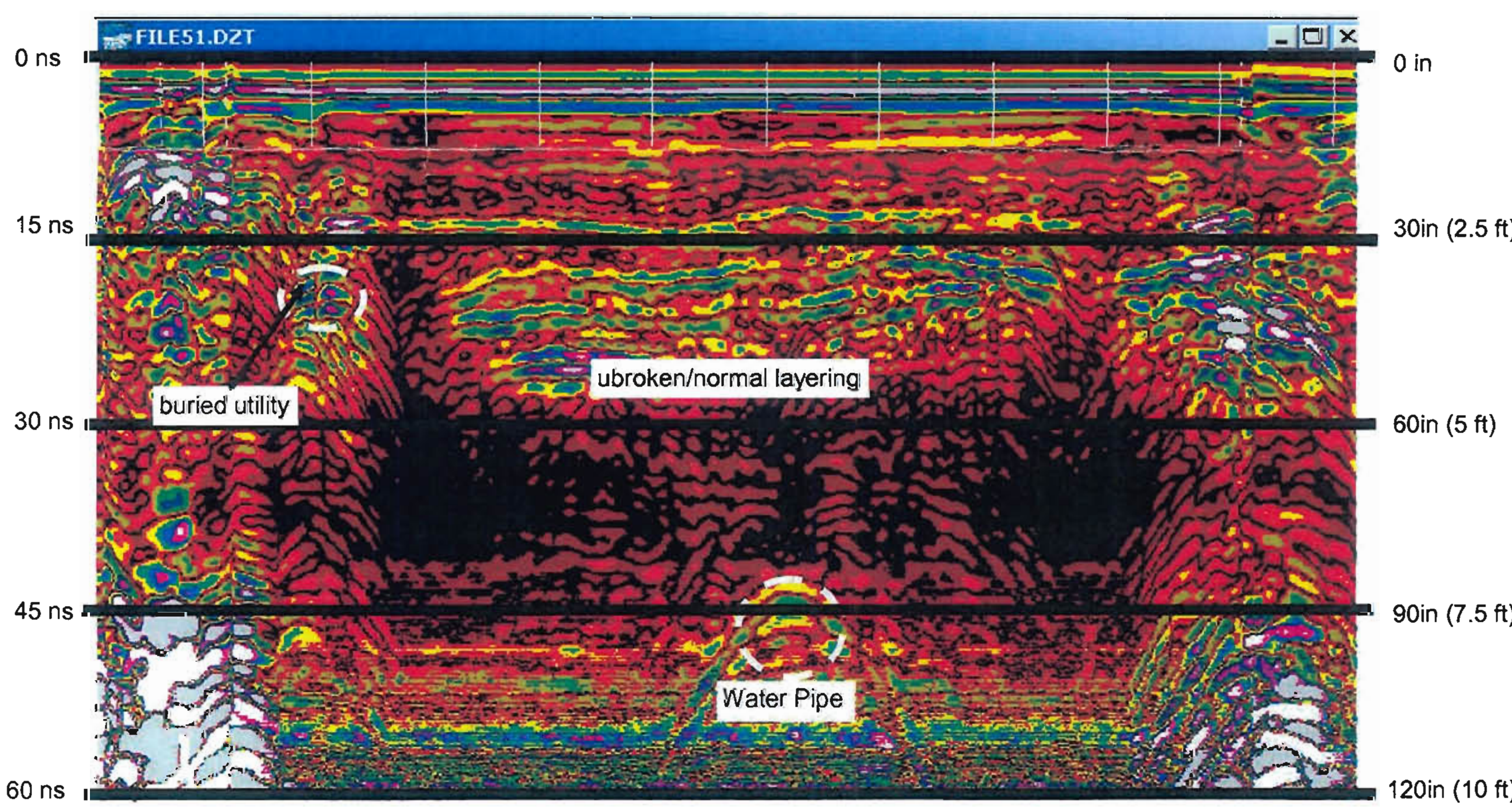


GPR SINKHOLE INVESTIGATION US RT 1 KENNEBUNK, MAINE FOR KENNEBUNK PUBLIC WORKS By NDT CORPORATION	INTERPRETATION OF GPR RESULTS
	Oct. 2004 / Figure 3

File 58 West to East Cross Line
80 Feet North of Hydrant



File 51 West to East Cross Line
45 Feet North of Hydrant



GPR SINKHOLE INVESTIGATION US RT 1 KENNEBUNK, MAINE FOR KENNEBUNK PUBLIC WORKS By NDT CORPORATION	ANNOTATED GPR RECORDS	
	Oct. 2004	Figure 4

APPENDIX

GROUND PENETRATING RADAR

APPENDIX: GROUND PENETRATING RADAR

Ground Penetrating Radar (GPR) is an electrical geophysical method for evaluating subsurface conditions by transmitting high frequency electromagnetic waves into the ground and detects the energy reflected back to the surface. Electromagnetic signals are transmitted from the antenna (transmitter and receiver) at ground surface and reflected back to the antenna from interfaces with differing electrical (dielectric constant and conductivity) properties. The greater the contrast in the electrical properties between two materials, the more energy that is reflected to the surface and the more defined results are.

GPR reflections typically occur at subsurface discontinuities such as:

- Buried metal objects (utilities, tanks, reinforcing)
- Open and Water filled voids
- Water table
- Soil stratification
- Seepage paths
- Bedrock Fractures

The depth of penetration of GPR is site specific, limited by the attenuation of the electromagnetic energy. Signal attenuation is controlled by four different mechanisms:

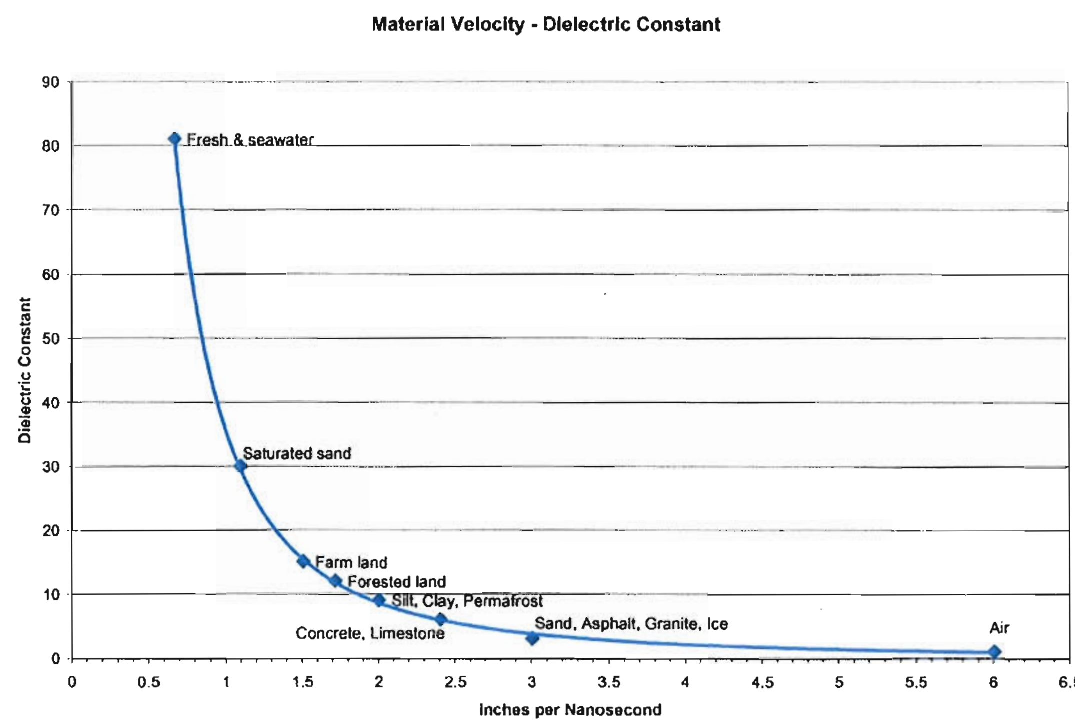
- Scattering: energy losses due to scattering occur when signals are dispersed in random direction, away from the receiving antenna, by large irregular shaped objects, such as boulders, tree stumps and closely spaced rebar.
- High conductivity layers: the greater the conductivity values of materials at a site, the more signal attenuation or less penetration. (mineral content, high moisture content, water table, metal plates, etc.)
- Water/Moisture Content: water molecules polarize in the presence of the applied electromagnetic field. Electromagnetic energy is lost to the radar system when it is converted to kinetic and thermal energy.
- Clays, (Ion content): ions along clay surfaces polarize in the presence of the applied electromagnetic field. Electromagnetic energy is lost to the radar when it is converted to kinetic and thermal energy.

An onsite calibration should be conducted so that the velocity for the materials and the depth of penetration can be determined. Sites can be electrically variable so it may be necessary to conduct multiple onsite calibrations.

Signal penetration is also dependent on the frequency of the antenna. High frequency antennas have shallow penetration and high resolution. A 1500 MHz high frequency antenna has an approximate depth of penetration of 1.5 feet and is able to identify wire mesh. Low frequency antennas have lower resolution and deeper depth of investigation. A 400 MHz antenna is capable of penetrating 10 to 15 feet in dry soils.

Ground Penetrating Radar (GPR) can be used to locate underground pipes, buried drums, foundations, voids in rock and concrete, soil settlement, determine stratigraphy, depth to water table, buried artifacts, filled excavations, and locate voids/settlement behind walls and under floor slabs. GPR is also a good tool for evaluating concrete structures such as bridges, walls, beams, ceilings, etc where the GPR can locate rebar and conduits, quantify rebar spacing, cover variability over reinforcing, and concrete thickness.

Laterally GPR can cover large areas relatively quickly. Using a grid pattern of survey lines it is very effective for mapping the lateral extents of subsurface features as well as calculating the depth to the features of interest. Depth of investigation can be estimated using material dielectric constants and the diagram shown below. Accurate depth calculations require an onsite calibration, to determine the electrical properties (speed of the signal) of the materials at the site. Depth calibrations typically consist of collecting GPR data over a metal target with a known depth. Known utilities, and buried metal plates are good targets for calibrations. GPR surveys can be very effective when coupled with other geophysical surveys and/or ground truth methods to verify, correlate and extrapolate GPR results. GPR surveys are a fast and cost effective method to collect data over large or obstructed sites, and isolate anomalies and areas where borings or other methods can be focused for the best interest of a project.



GPR systems consist of: Control unit (pulse transmitter, digital recorder, data storage, monitor); Antenna(s); Coaxial Cable and Printer

GPR Control Unit is a computer which control data acquisition parameters, such as sampling rate, range, gain control, filtering, etc. The control unit also visually displays the data, digitally archives the data, and allows for play back of the data.

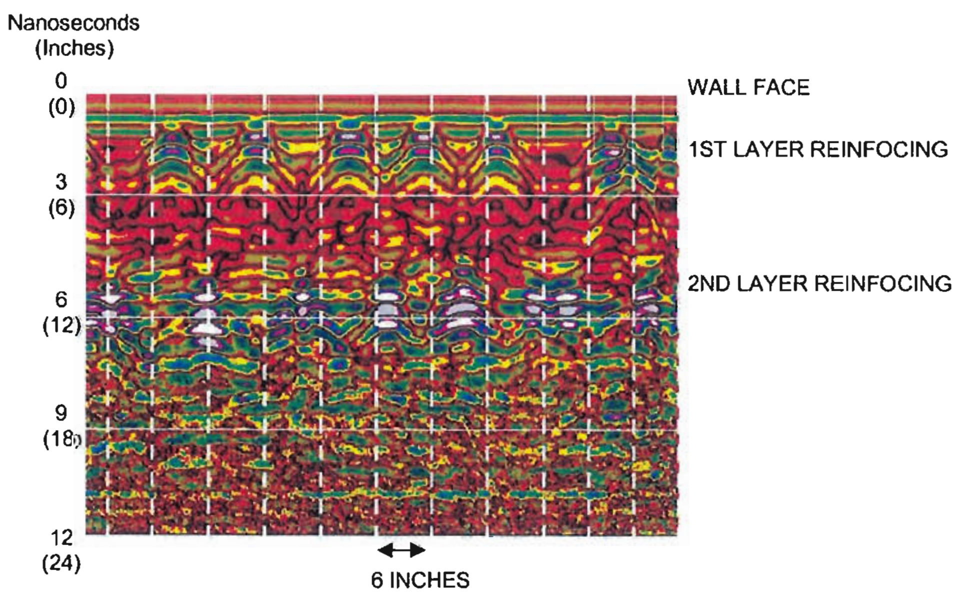
The coaxial cable connects the control unit to the Antenna. The Antenna(s) are sealed and shielded fiberglass housing for the transmitter and receiver. Selection of the antenna is dictated by the requirements of the survey. For high resolution, near-surface data, a high frequency antenna is used; for deeper penetration investigation, a lower frequency antenna is used. Typically the 80 to 300 MHz antennas are used for geologic surveys; 300 to 900 are used for utility, near surface voiding settlement, foundation, etc surveys while the high frequency antenna 900 to 1500 is used for concrete assessment.

ACQUISTION AND INTERPETATION:

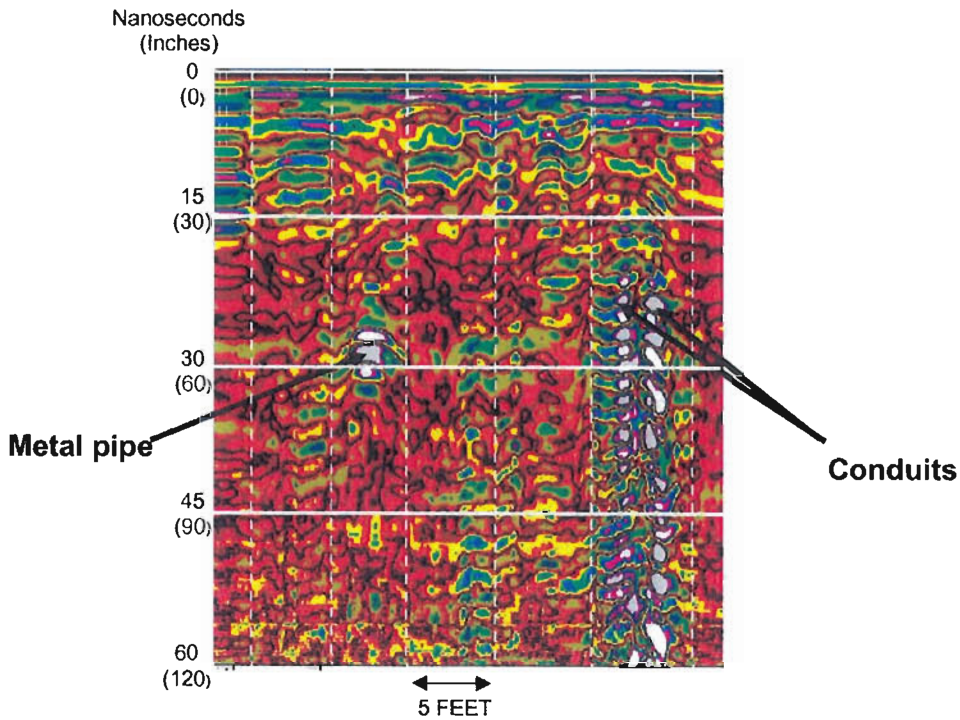
Radar signals propagate from the antenna in a 15 to 45 degree cone, thus the slower the speed of the antenna the greater the horizontal resolution. Radar data are typically acquired at a slow walking speed. Data are printed and digitally saved. Station markers and any field notes are written right on the printed copy and the digitally saved data can be used to reprint or to use with post processing software.

Interpretation of GPR data is subjective, even among experienced interpreters. GPR results should be verified with borings or test pits. The strength of a reflected signal and/or the continuity of the reflector across the record may be indicative of a stratigraphic contact. Point targets, such as reinforcing, buried utilities, boulders, create a distinctive parabolic feature on GPR records. Annotated GPR records of reinforcing and buried metal utilities are shown below. Positive identification of point targets is subjective, as the GPR signature of a pipe is similar to that of a large boulder.
Computer processing is available though it is somewhat costly and in most cases not necessary, except for presentation purposes.

GPR RECORD
12" THICK WALL WITH REINFORCING



UNDER GROUND UTILITY LOCATION/MAPPING



GROUND PENETRATING RADAR

MASONARY BRIDGE BR#2431
US RT 1 OVER THE MOUSAM RIVER

KENNEBUNK, MAINE

Prepared for

GZA GEOENVIRONMENTAL, INC

December, 2008

NDT CORPORATION

NDT CORPORATION

December 29, 2008

Mr. Christopher L. Snow, P.E.
GZA GeoEnvironmental, Inc.
4 Free Street
Portland, Maine 04101

Dear Mr. Snow:

In accordance with your authorization to proceed NDT Corporation conducted ground penetrating radar (GPR) geophysical measurements to determine the thickness of the masonry abutments and wing walls of the US RT 1 Bridge over the Mousam River in Kennebunk. The Maine DOT assisted NDT by providing a Under Bridge Inspection Vehicle and operator. Fieldwork was conducted on December 15th, 2008.

This report presents the results and findings of our investigation. If you have any questions or require additional information contact the undersigned at 508-754-0417

Sincerely,
NDT CORPORATION

Paul S. Fisk

Table of Contents

1.0 Summary of Results:

Ground Penetrating Radar (GPR) results indicated the north and south masonry abutments are approximately 8-10+/- feet thick while the wing walls are approximately 6+/- feet thick. The GPR data had no indications of voiding behind the abutment and wing walls but did indicate moisture/water entrapment in joints between masonry blocks at the back of abutments and walls.

2.0 Introduction and Purpose:

NDT Corporation conducted geophysical measurements to determine the thickness of the masonry abutments and wing walls of the US RT 1 Bridge over the Mousam River in Kennebunk, Maine (Figure 1). The Maine DOT provided an Under Bridge Inspection Vehicle and operators to assist with data acquisition. Fieldwork was conducted on December 15th 2008.

3.0 Testing Method:

3.1 Ground Penetrating Radar (GPR)

GPR uses a pulsed electromagnetic signal that is transmitted to and reflected by "targets" back to the point of transmission. The electromagnetic wave transmission and reflection is dependent on the dielectric constant and conductivity (electrical properties) of the material(s) being investigated. Saturated or moist conditions and metal reinforcing are highly reflective of radar signals; dry concrete and stone are relatively transparent to radar signals. As a result, reflections from moist soils behind abutments can be distinguished by the GPR profiling and are used to determine the masonry abutment thickness.

GPR data were acquired with a 400 MHz antenna. The 400 MHz antenna has a depth of investigation of 15 or 20 feet or greater in dry materials. Given the average time to a reflector, an average signal velocity is used to calculate the depth/thickness of the masonry wall or abutment. Typically 2 inches/nanosecond is used when an onsite calibration is not available.

4.0 Discussion of Results

GPR data was collected on vertical and horizontal lines on both the north and south abutment faces, and where accessible on the north abutment east and west wing walls, and the south abutment east wing wall (Figure 2). Data could not be collected on the south-west wing wall because it could not be accessed by the under bridge inspection vehicle.

Data collected on the abutment faces has reflections at approximately 12, 24, 36, 48 and (60) nanoseconds which indicate thickness of 2, 4, 6, 8, and (10) feet. It is believed the average block thickness to be approximately 2 feet which would indicate the abutments are 4 to 5 blocks thick which gives a thickness of 8-10+/- feet. Moisture entrapment in the joint between blocks near the back of the wall make it difficult to determine if the abutment is 4 blocks, 8+/- feet or if the abutment is 5 blocks thick or 10+/- feet thick.

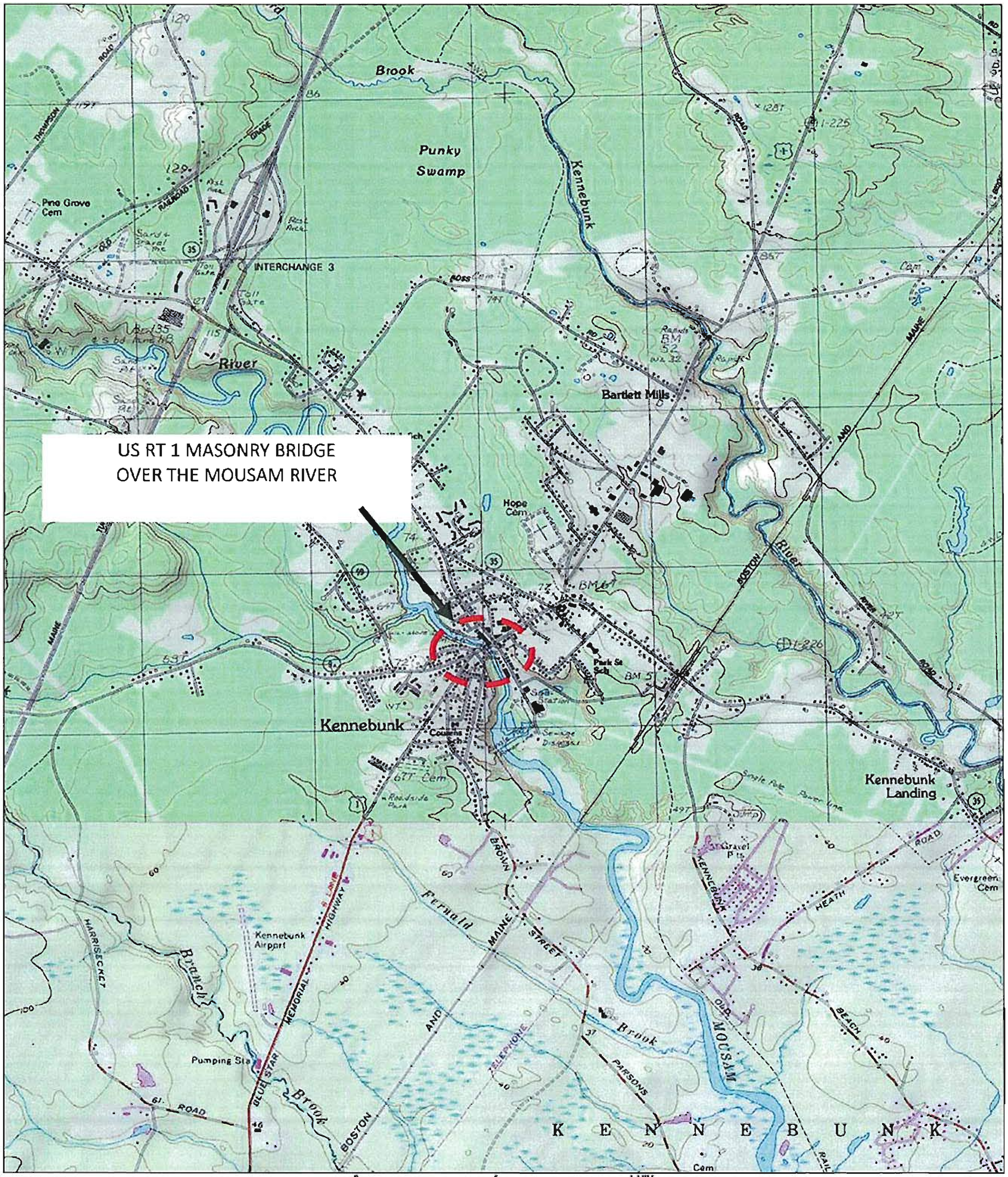
Vertical lines and horizontal lines at different levels did not indicate a tapered thickness; data indicated a consistent thickness of 8-10+/- feet.

Data collected on the wing wall locations indicated reflectors only at 12, 24, and 36 nanoseconds, indicating the wing walls are 3 blocks thick or approximately 6+/- feet thick.

Data was also collected along transverse line and longitudinal lines at the street surface (Figure 2) to correlate with data collected on the wing wall and abutment faces.

Figure 3 is a compilation of the wall, abutment and street GPR results. Figure 4 has an annotated wall and abutment GPR record.

FIGURES

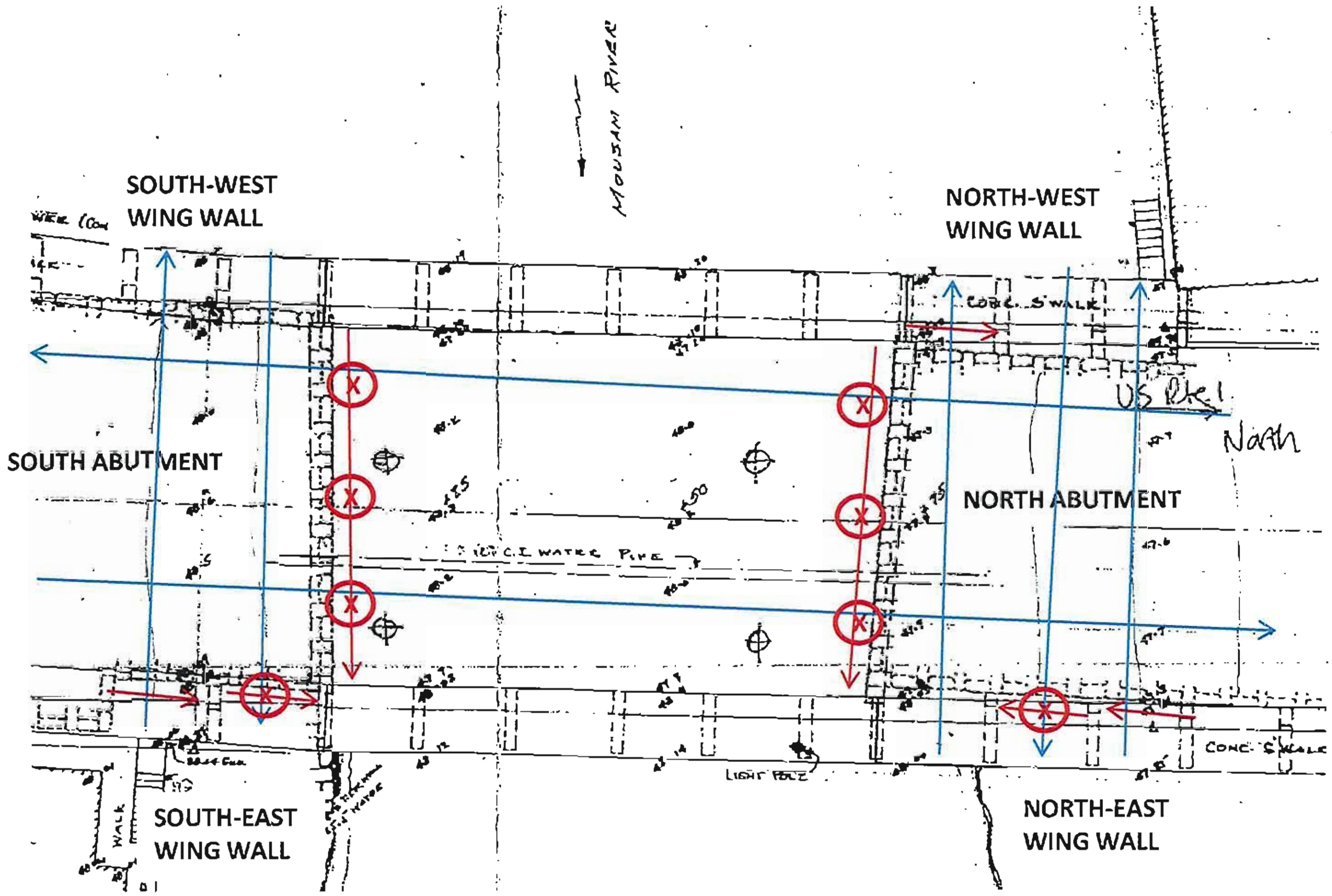


MN TN 16°

0 .5 1 MILE
0 1000 FEET 0 500 1000 METERS

Map created with TOPO!® ©2003 National Geographic (www.nationalgeographic.com/topo)

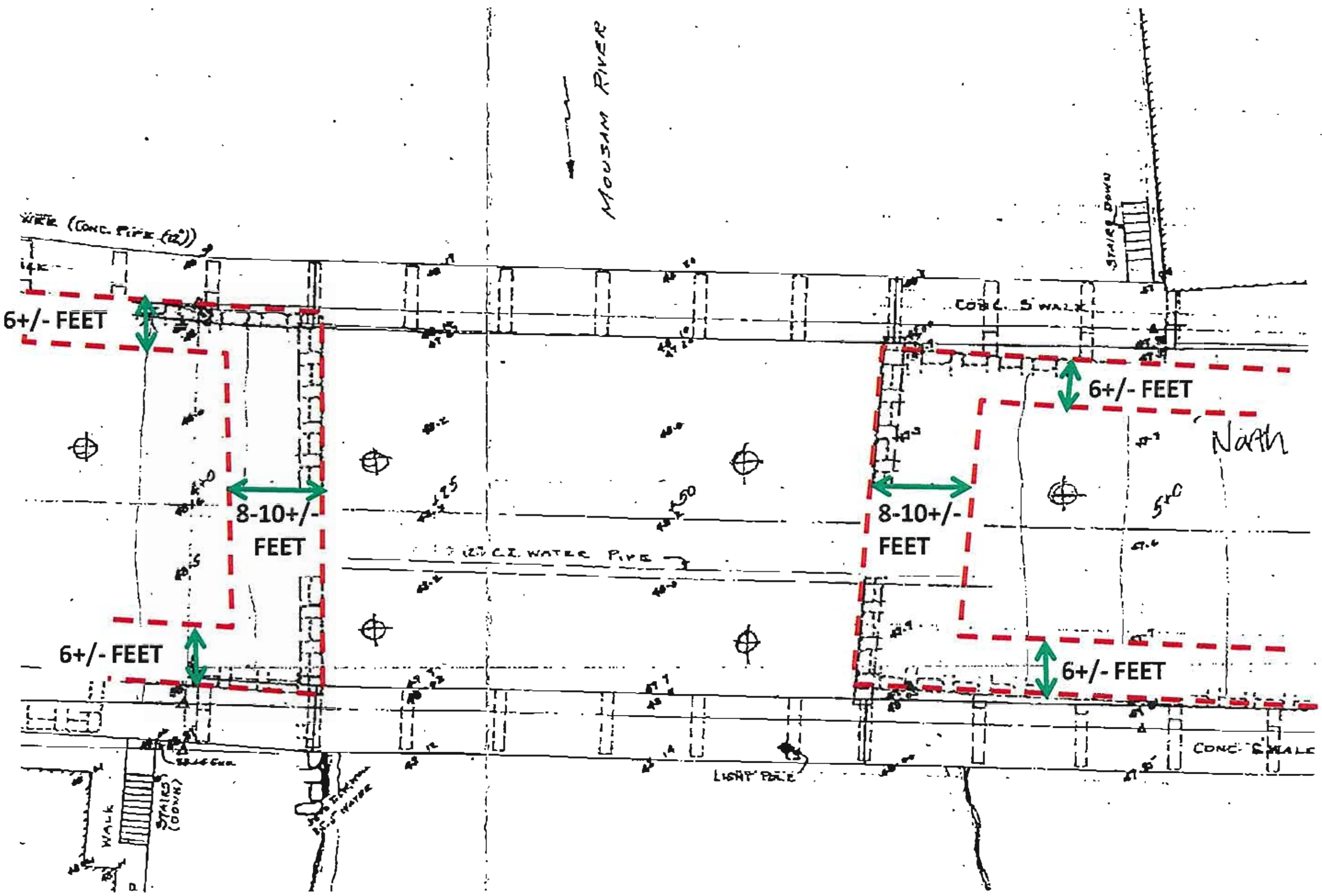
GROUND PENETRATING RADAR US RT 1 MASONRY BRIDGE OVER THE MOUSAM RIVER Prepared for GZA GEOENVIRONMENTAL, INC KENNEBUNK, MAINE by NDT Corporation	AREA OF INVESTIGATION	
	Dec-08	Figure 1



Surface (street level) GPR Lines

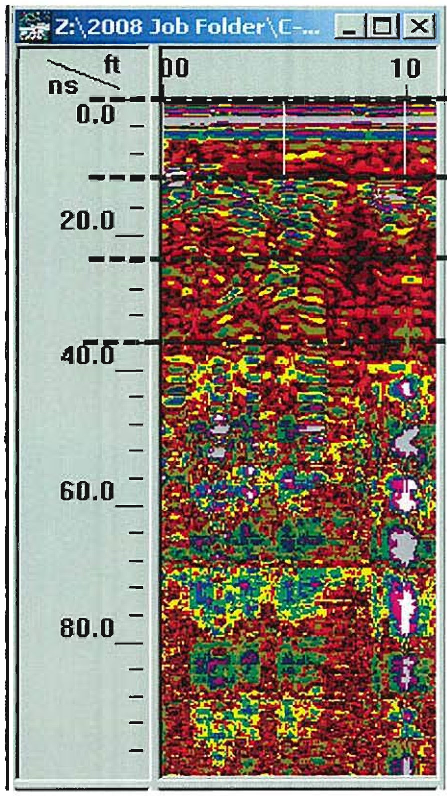
GPR Lines on Abutments and Wing Walls

GROUND PENETRATING RADAR US RT 1 MASONRY BRIDGE OVER THE MOUSAM RIVER Prepared for GZA GEOENVIRONMENTAL, INC KENNEBUNK, MAINE by NDT Corporation	LINES OF COVERAGE	
	Dec-08	Figure 2



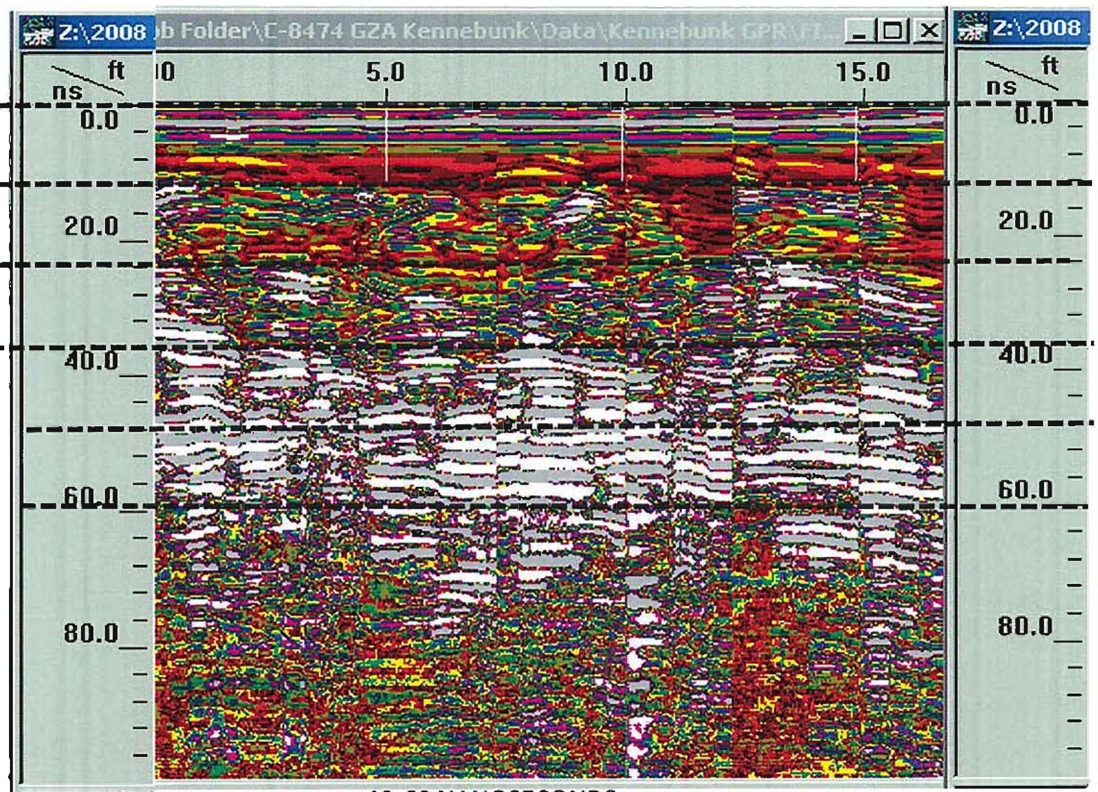
GROUND PENETRATING RADAR US RT 1 MASONRY BRIDGE OVER THE MOUSAM RIVER Prepared for GZA GEOENVIRONMENTAL, INC KENNEBUNK, MAINE by NDT Corporation	RESULTS	
	Dec-08	Figure 3

WING WALL RECORD



36 NANOSECONDS
APPROXIMATELY 6 +/- FEET
3 TIERS OF 2 +/- FT BLOCKS

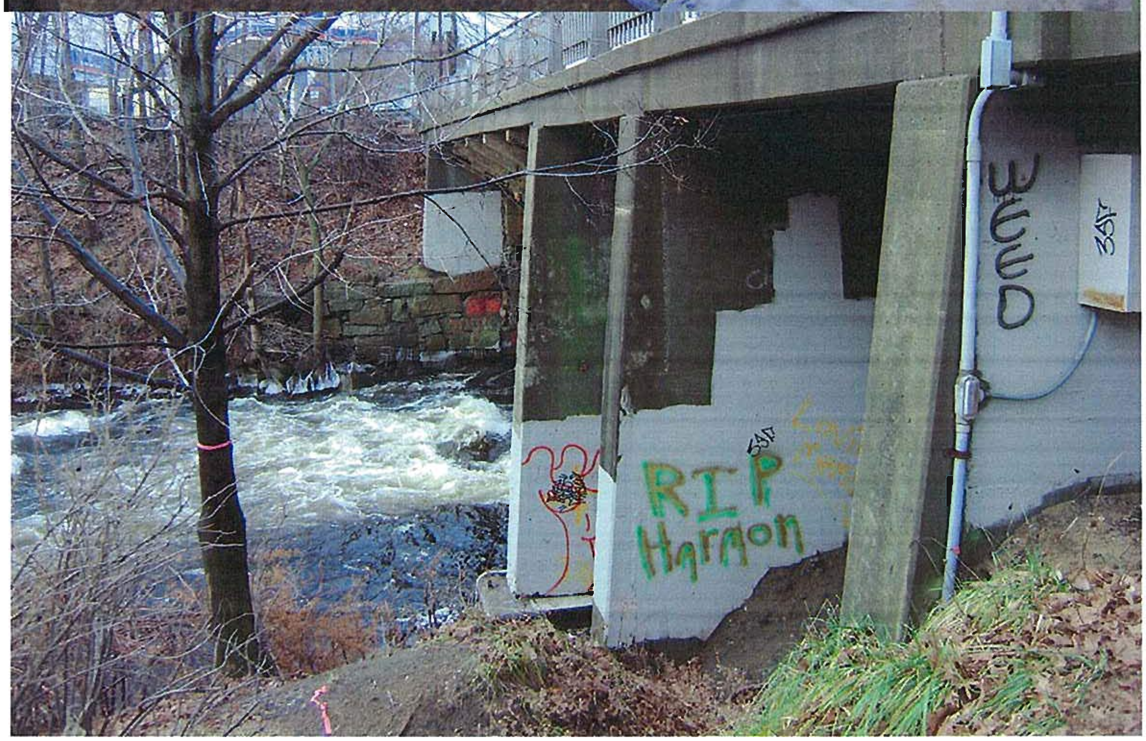
ABUTMENT FACE RECORD



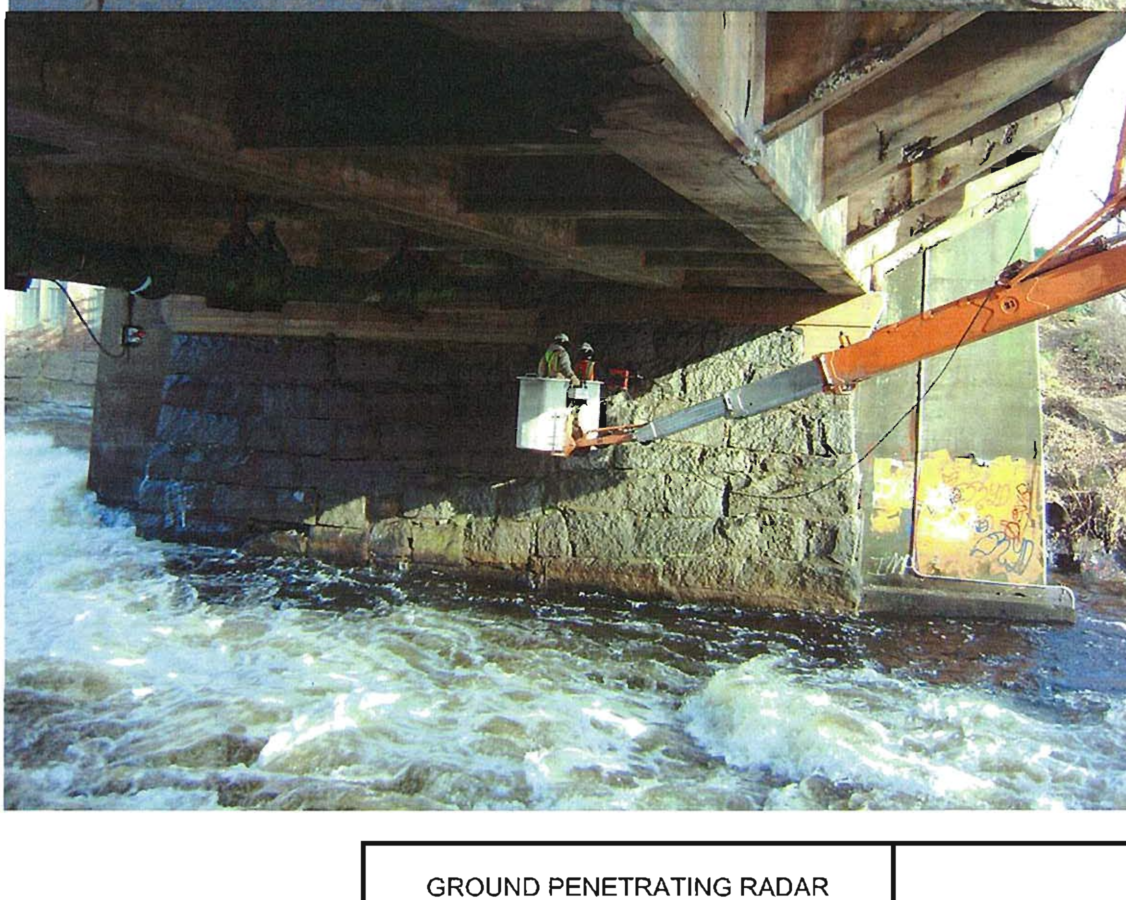
48-60 NANOSECONDS
APPROXIMATELY 8-10 +/- FEET
4-5 TIERS OF 2 +/- FT BLOCKS

GROUND PENETRATING RADAR US RT 1 MASONRY BRIDGE OVER THE MOUSAM RIVER Prepared for GZA GEOENVIRONMENTAL, INC KENNEBUNK, MAINE by NDT Corporation	TYPICAL RECORDS WING WALL ABUTMENT FACE	
	Dec-08	Figure 4

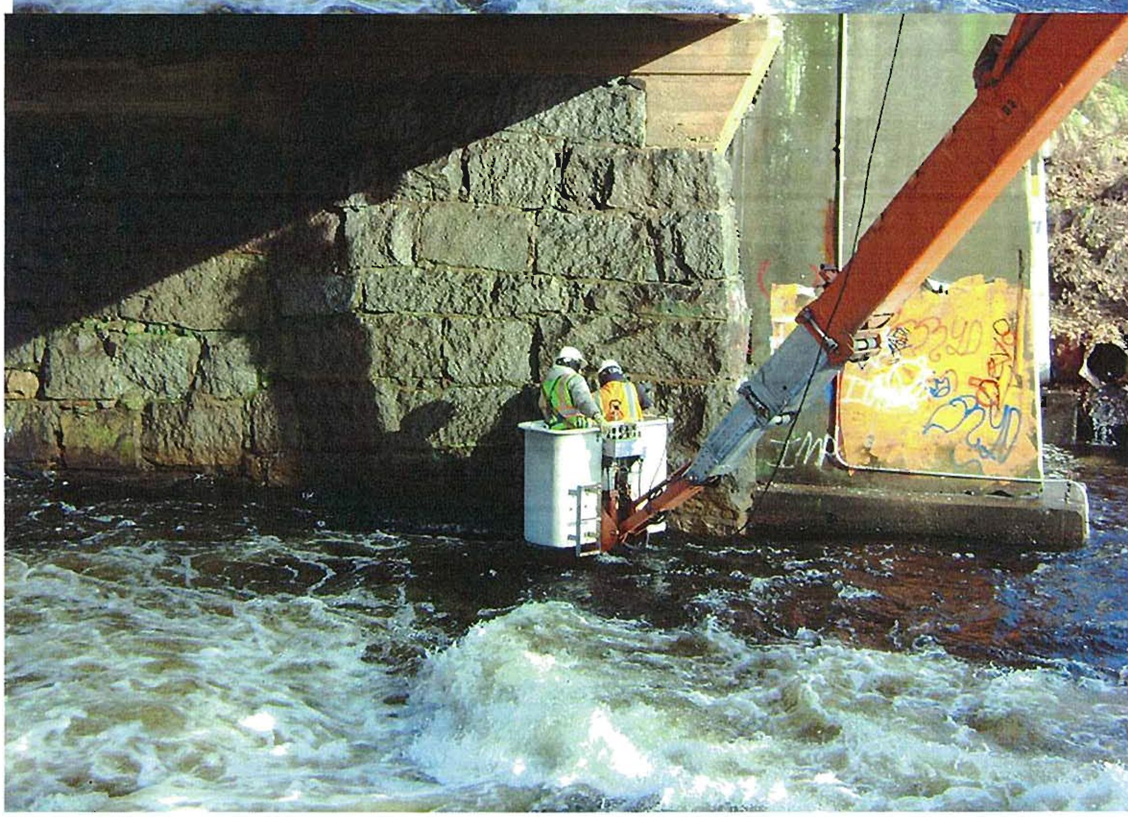
PHOTOGRAPHS



GROUND PENETRATING RADAR US RT 1 MASONRY BRIDGE OVER THE MOUSAM RIVER Prepared for GZA GEOENVIRONMENTAL, INC KENNEBUNK, MAINE by NDT Corporation	Photos	
	Dec-08	Photos



GROUND PENETRATING RADAR US RT 1 MASONRY BRIDGE OVER THE MOUSAM RIVER Prepared for GZA GEOENVIRONMENTAL, INC KENNEBUNK, MAINE by NDT Corporation	Photos	
	Dec-08	Photos



GROUND PENETRATING RADAR
US RT 1 MASONRY BRIDGE
OVER THE MOUSAM RIVER
Prepared for
GZA GEOENVIRONMENTAL, INC
KENNEBUNK, MAINE
by
NDT Corporation

Photos

Dec-08

Photos

APPENDIX 1

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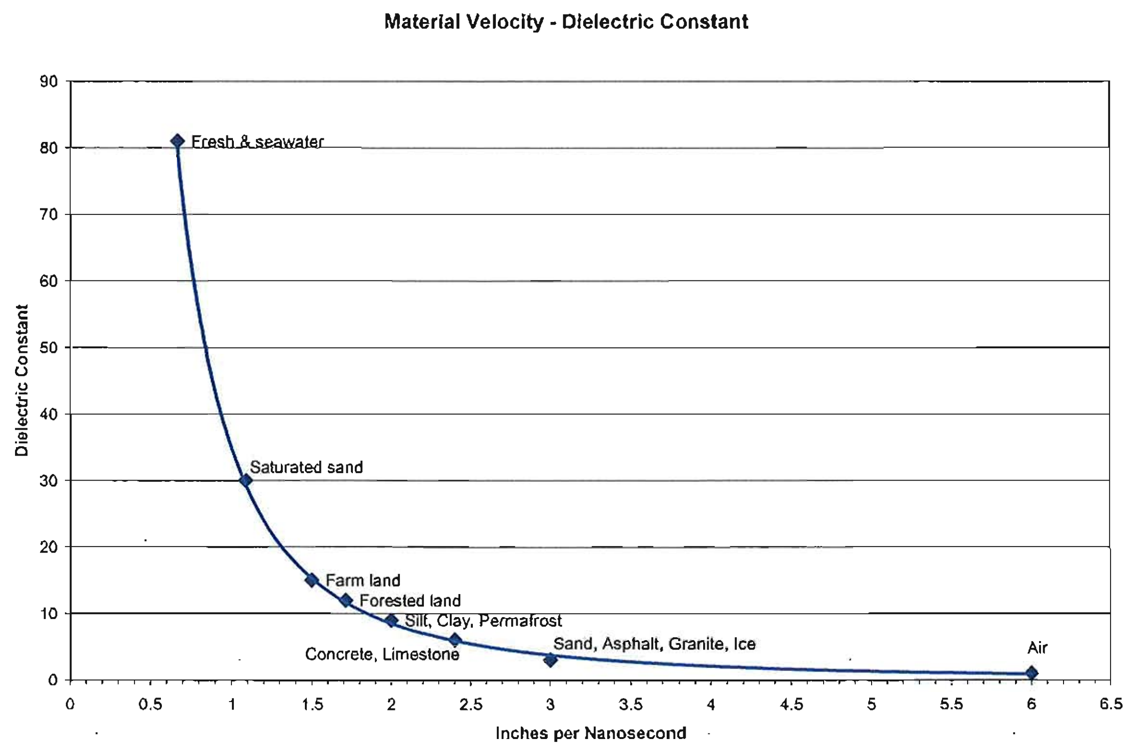

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ACQUISITION AND INTERPRETATION:

Radar signals propagate from the antenna in a 15 to 45 degree cone, thus the slower the speed of the antenna the greater the horizontal resolution. Radar data are typically acquired at a slow walking speed. Data are printed and digitally saved. Station markers and any field notes are written right on the printed copy and the digitally saved data can be used to reprint or to use with post processing software.

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GPR RECORD
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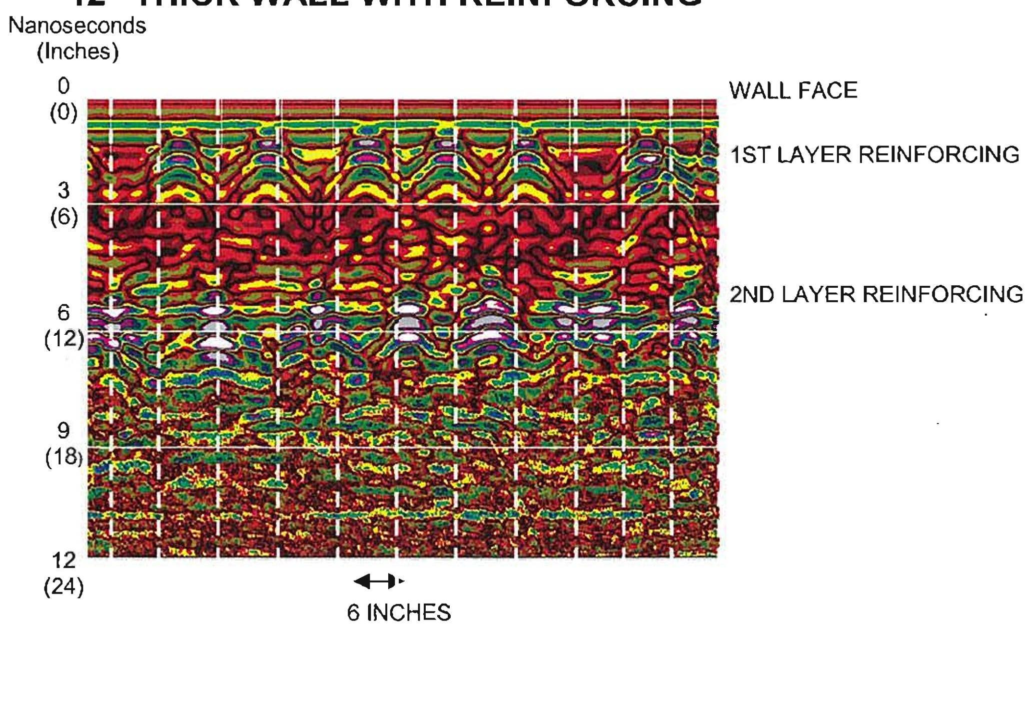
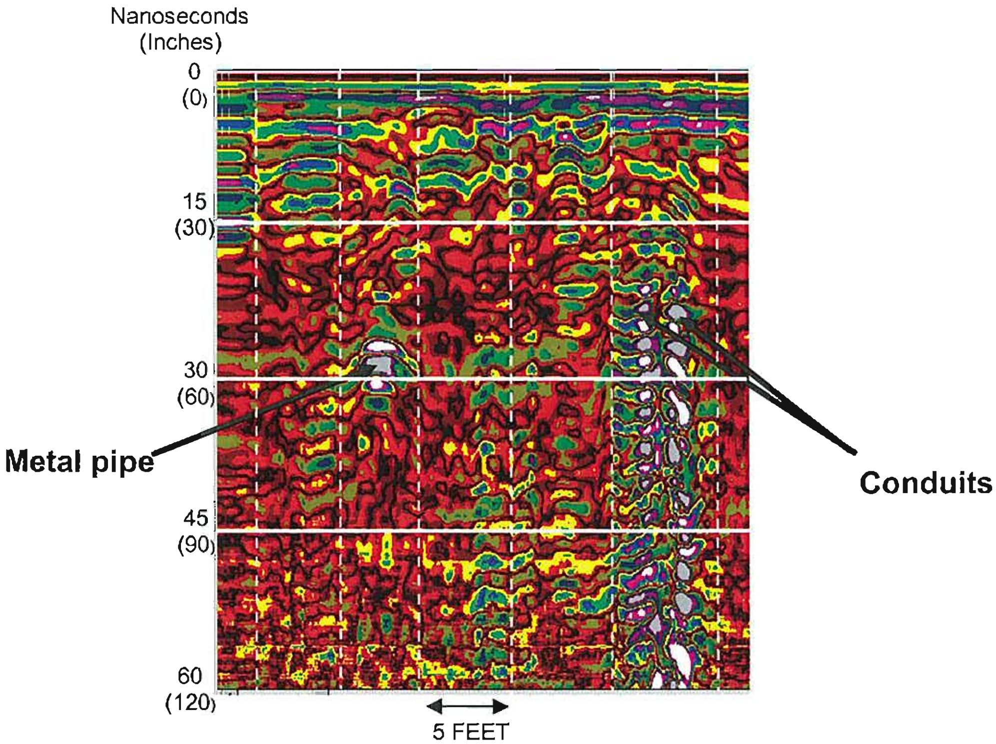


UNDER GROUND UTILITY LOCATION/MAPPING



APPENDIX F

LABORATORY TESTING RESULTS, 2008 AND 2010

GZA GeoEnvironmental, Inc.

State of Maine - Department of Transportation

<u>Laboratory Testing Summary Sheet</u>

Kennebunk Bridge

Town(s): Kennebunk

MDOT Project Number: 15098

GZA Project Number: 09.0025597.00

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-KMR-101	1D			1-3			4.4			SP-SM	A-1-b	II
BB-KMR-101	2D			4-6			10.2			SW-SM	A-3	II
BB-KMR-106	1D			1-3			3.5			SP-SM	A-1-b	II
BB-KMR-106A	4D			15-17			16.5			SM	A-2-4	II

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.

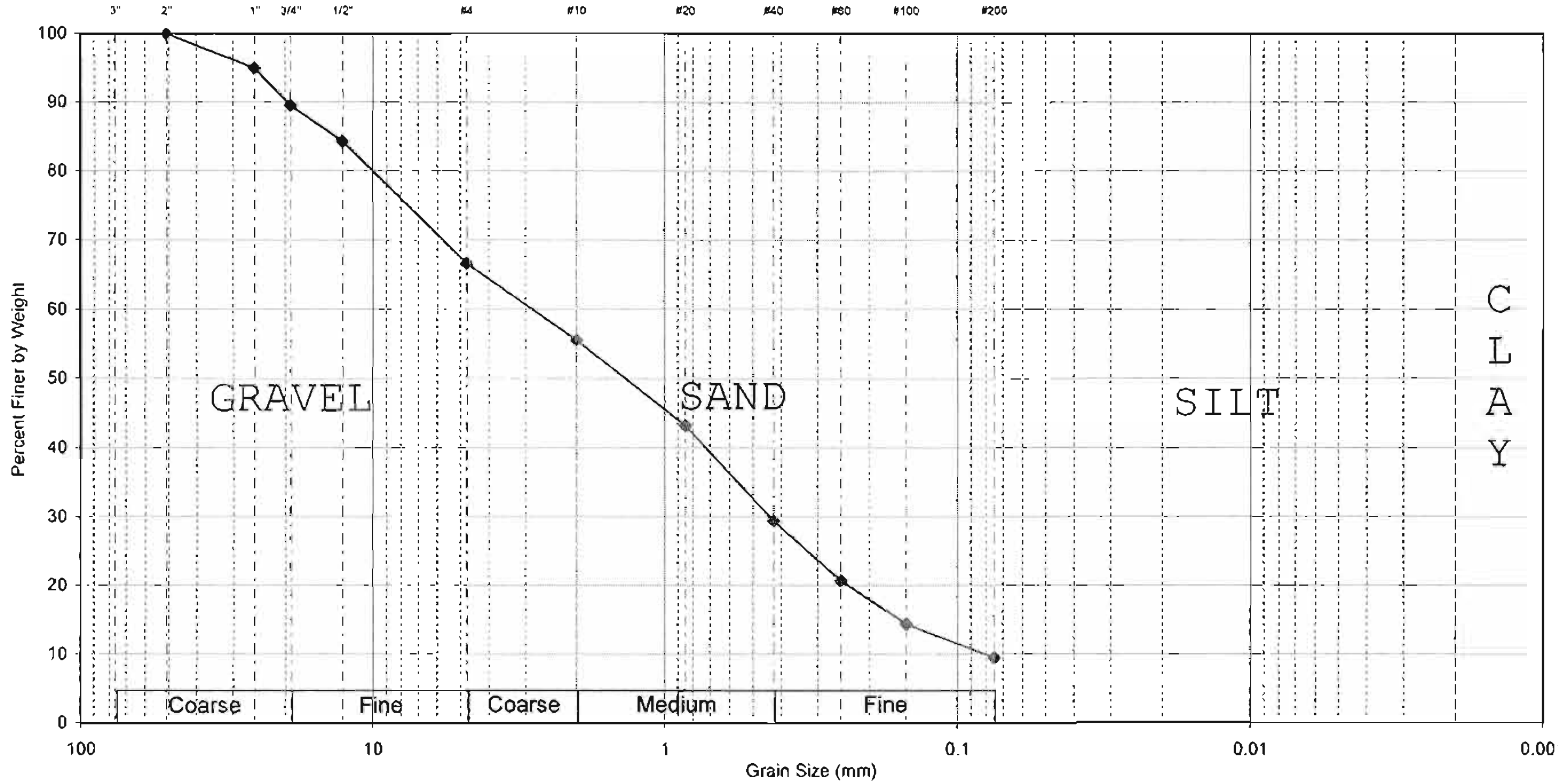
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



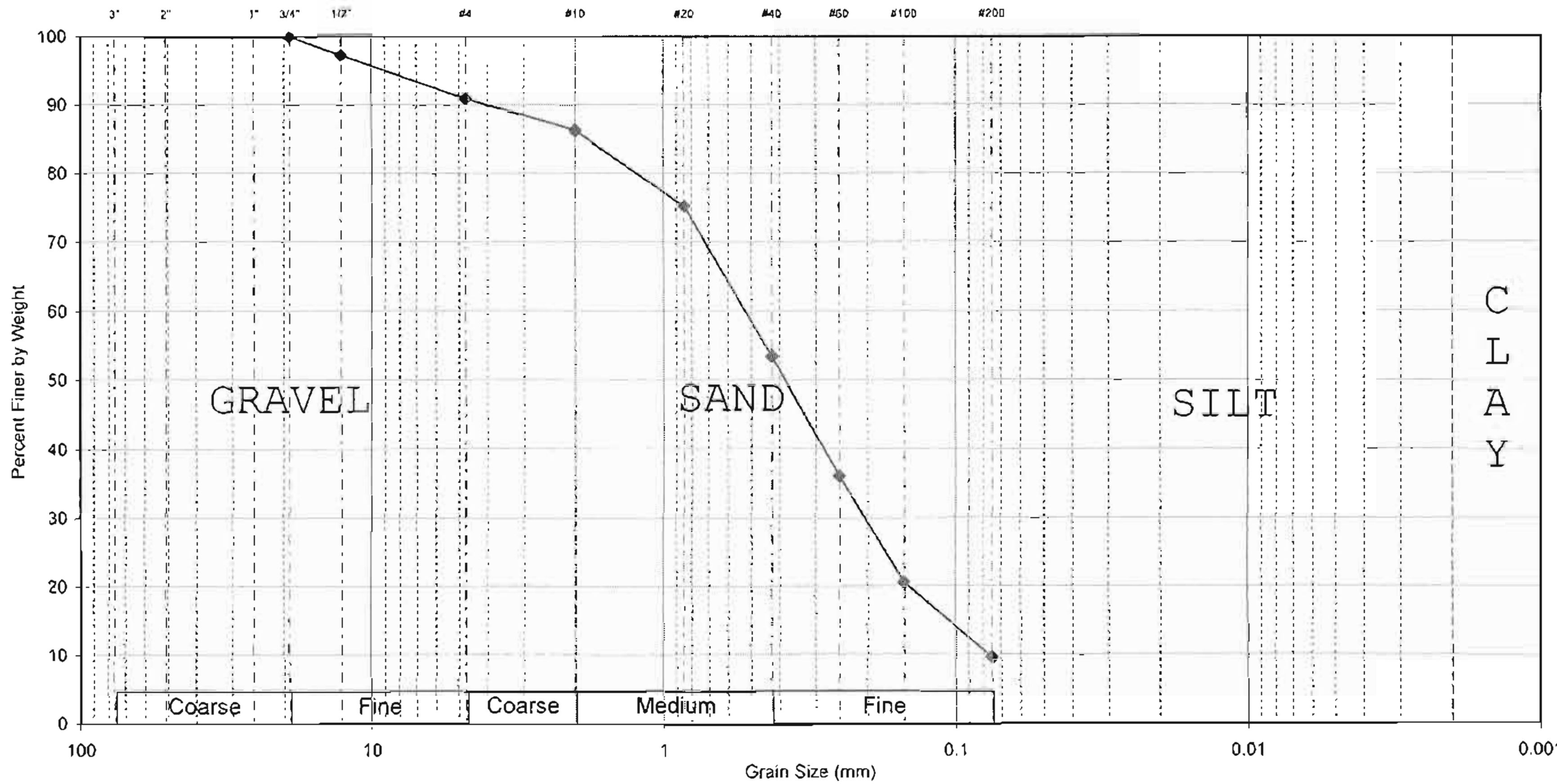
Gravel	Sand	Fines
33.4%	57.1%	9.5%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
1	BB-KMR-101	1D	1-3'	Brown Poorly-graded Sand with Silt & Gravel (SP-SM)	4.4			

Kennebunk Bridge
Kennebunk, ME
GZA File # 09.0025597.00

Tested by:	JMN	Date:	1/11/09
Reviewed by:	MBP	Date:	1/12/09

U.S. STANDARD SIEVE AND HYDROMETER

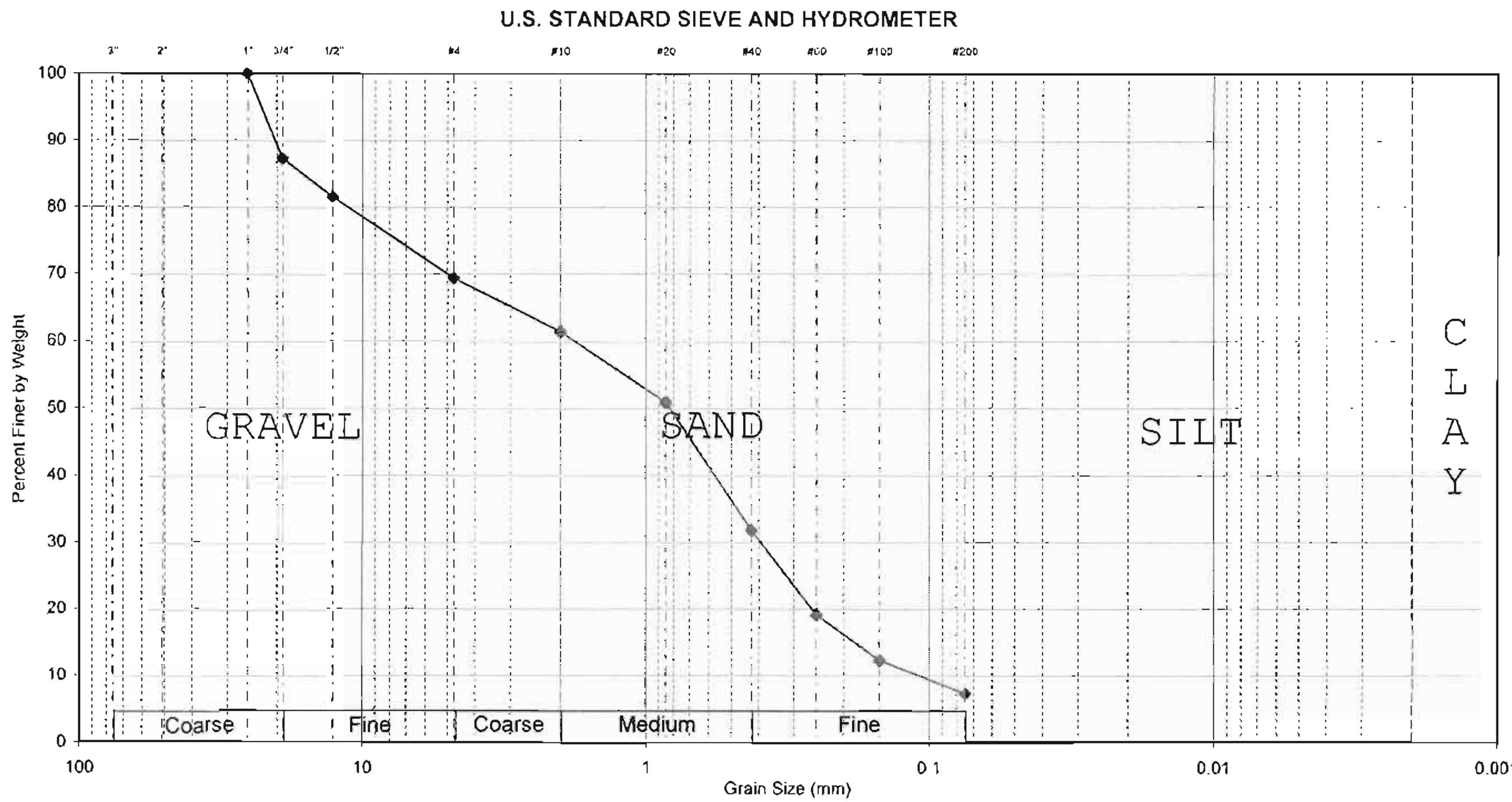


Gravel	Sand	Fines
9.1%	81.3%	9.6%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
2	BB-KMR-101	2D	4-6'	Brown Well-graded Sand with Silt (SW-SM)	10.2			

Kennebunk Bridge
Kennebunk, ME
GZA File # 09.0025597.00

Tested by:	JMN	Date:	1/11/09
Reviewed by:	MBP	Date:	1/12/09



Gravel 30.6%

Sand 62.0%

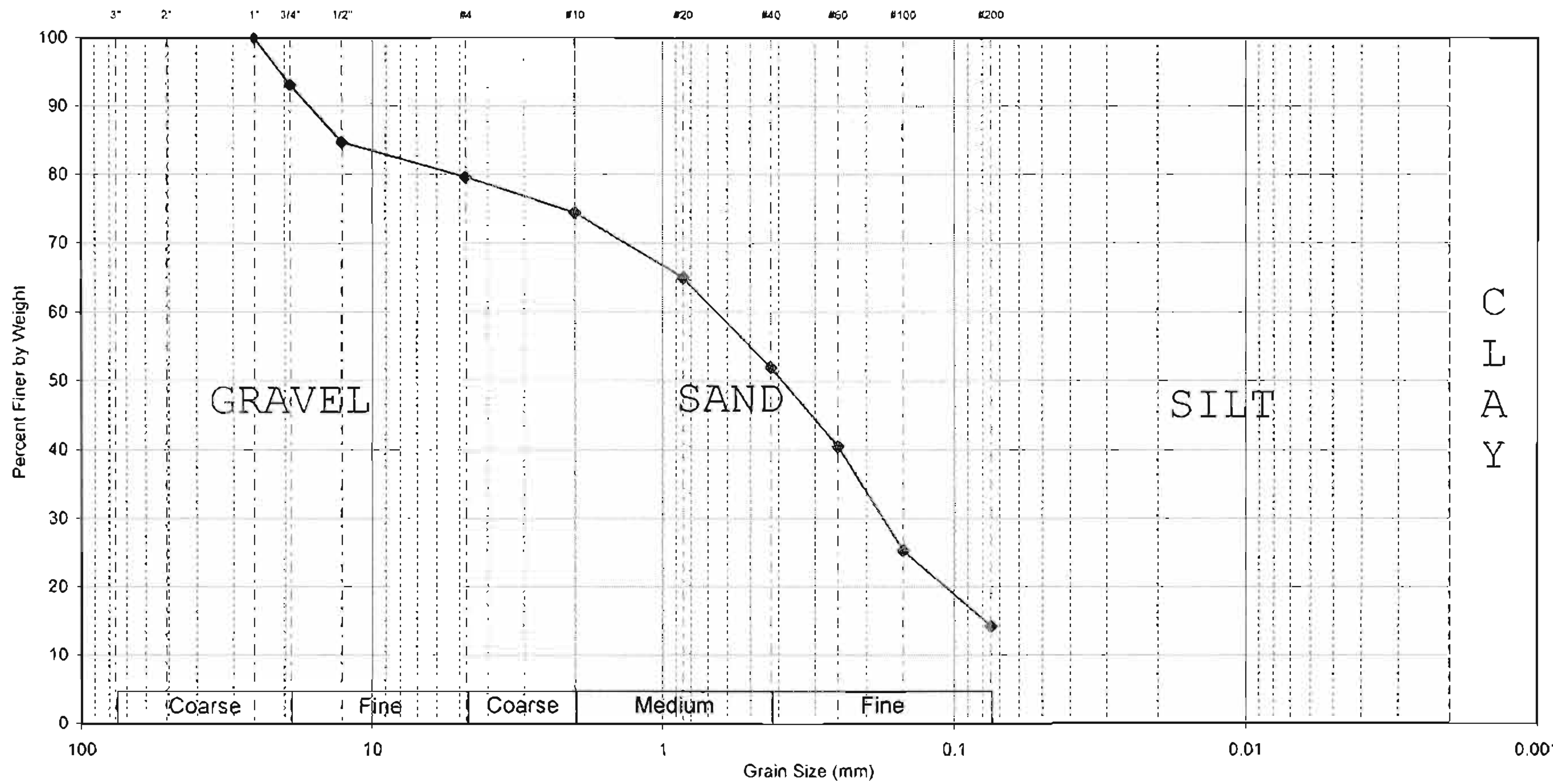
Fines 7.3%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
3	BB-KMR-106	1D	1-3'	Brown Poorly-graded Sand with Silt & Gravel (SP-SM)	3.5			

Kennebunk Bridge
Kennebunk, ME
GZA File # 09.0025597.00

Tested by: JMN Date: 1/11/09
Reviewed by: MBP Date: 1/12/09

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
20.4%	65.4%	14.2%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
4	BB-KMR-106A	4D	15-17'	Brown Silty Sand with Gravel (SM)	16.5			

Kennebunk Bridge
Kennebunk, ME
GZA File # 09.0025597.00

Tested by: JMN Date: 1/11/09
Reviewed by: MBP Date: 1/12/09

LABORATORY TESTING DATA SHEET

Project Name Kennebunk Bridge
Project No. 09.0025597.00
Project Manager J. Tooley

Location Kennebunk, ME
Assigned By J. Tooley
Report Date 1/15/2009

Reviewed By [signature]
Date Reviewed 1/15/09

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	στ KSI	Is_{50} KSI	Rock Formation or Description or Remarks
BB-KMR-103	R1	25.5-25.9	5		1.966	4.471	172.3			U	17.95	0.32		4.96				Gray fine grained Siltstone
BB-KMR-102	R1	29.9-30.3	6		1.966	4.591	181.6			U	16.80	0.34		5.44				Gray f-m grained Sandstone

(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 Peak 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

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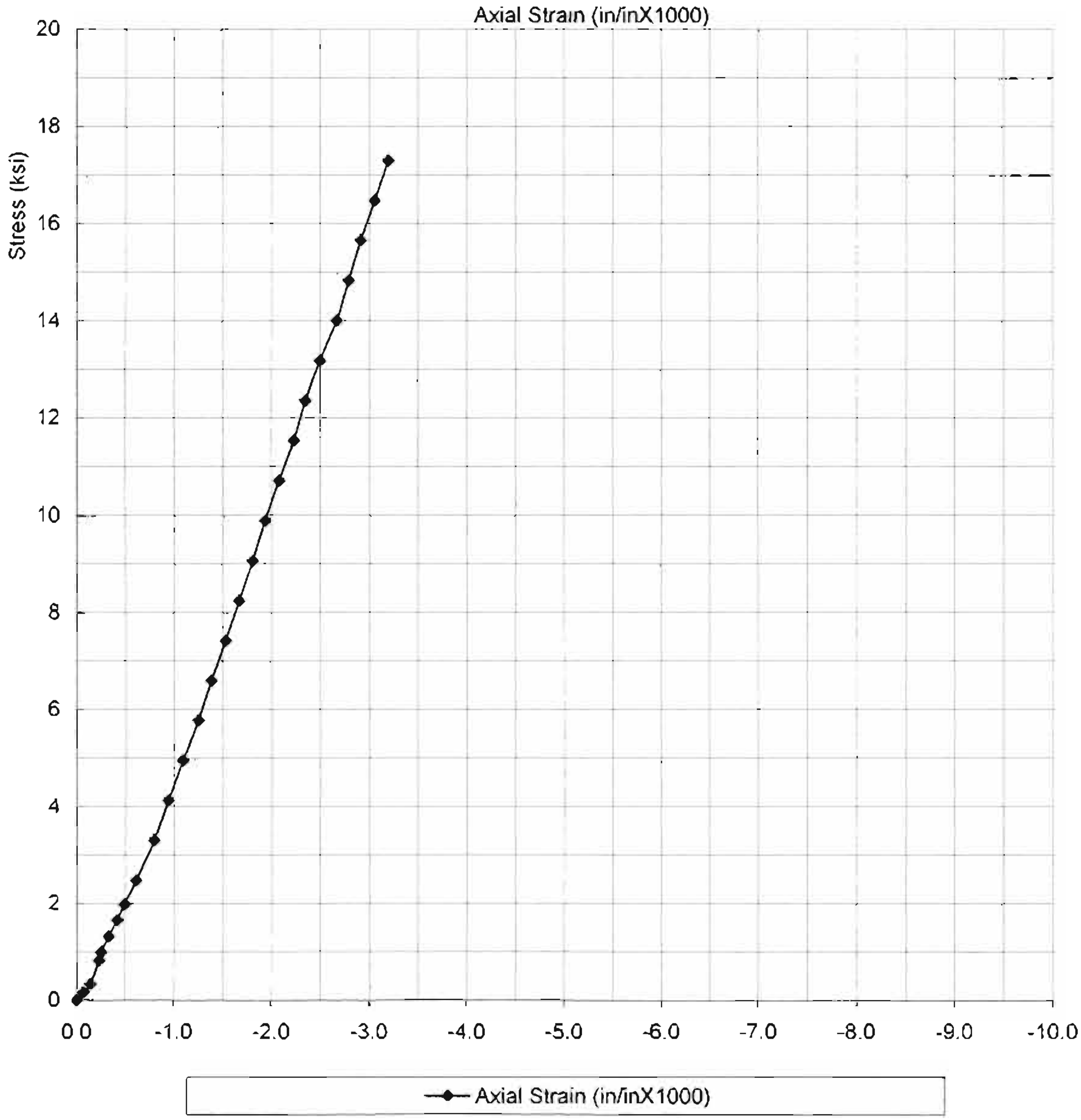
Kennebunk Bridge
Kennebunk, ME

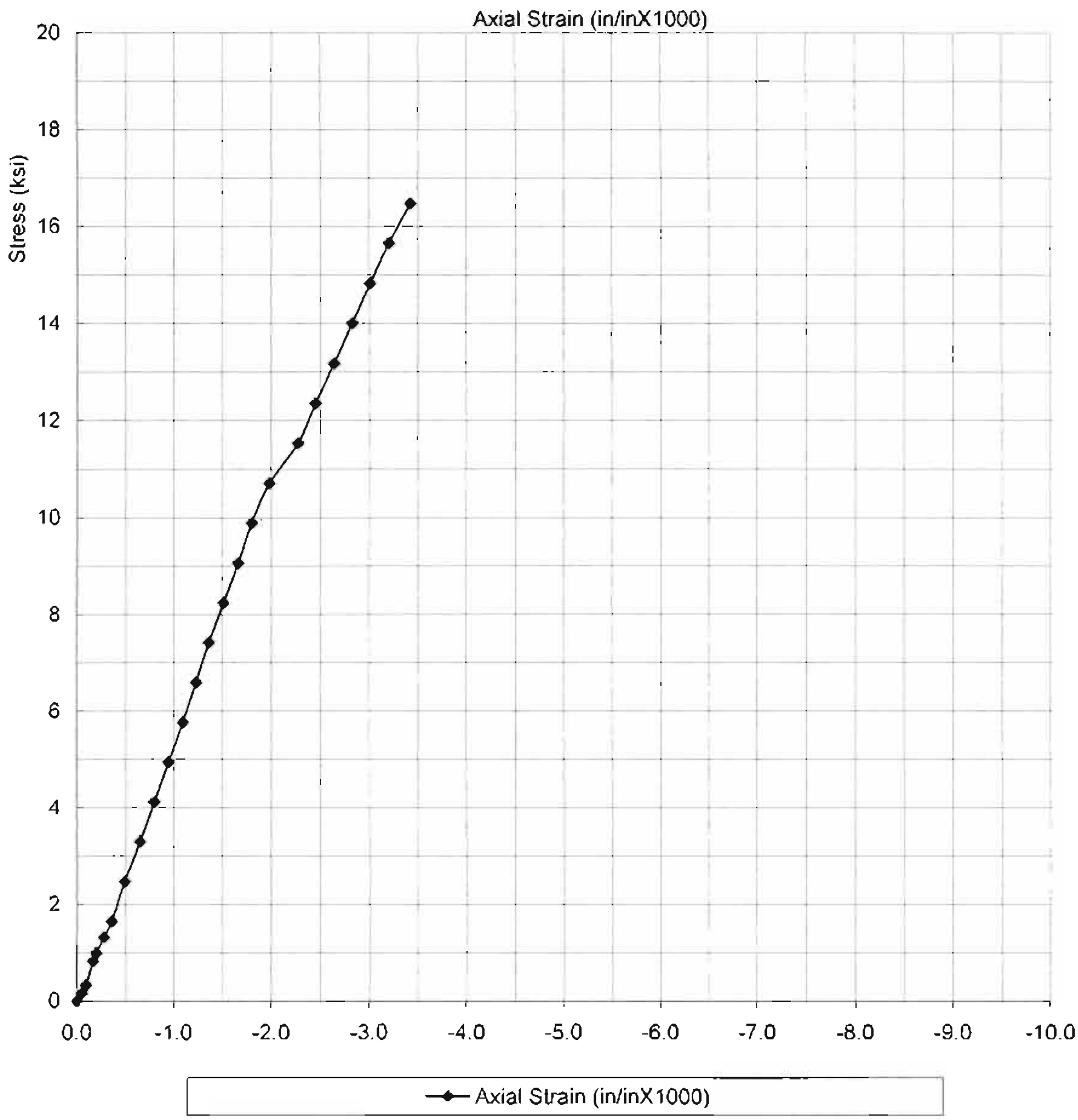


Rock Testing

Boring No. BB-KMR-103
Sample No. R1
Depth: 25.5-25.9'

File No. 09.0025597.00
Date: 1/14/2009
Test No. U 5

Kennebunk Bridge
Kennebunk, ME



Rock Testing

Boring No.	BB-KMR-102	File No	09.0025597.00
Sample No.	R1	Date:	1/14/2009
Depth:	29.9-30.3'	Test No.	U 6

GZA GeoEnvironmental, Inc.

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Kennebunk Mousam River Bridge

MDOT Project Number:

Town(s): Portland

GZA Project Number: 09.0025597.10

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-KMR-201		1D	0.5-2						SP-SM	A-1-b	0
BB-KMR-201		4D	6-8						SM	A-2-4	0
BB-KMR-201		5D	8-10						CL	A-6	IV
BB-KMR-201		6D	10-12						ML	A-4	IV
BB-KMR-202		4D	9-11						GP-GM	A-1-a	0
BB-KMR-203		1D	0.5-2						SP-SM	A-1-b	0
BB-KMR-203		3D	4-6						SM	A-1-b	0
BB-KMR-301		4D	7-9						CL	A-6	IV
BB-KMR-301		6D	11-13						SM	A-4	II
BB-KMR-301		8D	15-17						SM	A-1-b	0
BB-KMR-301		9D	17-19						SM	A-1-b	0
BB-KMR-302		1D	0.5-2						SP-SM	A-1-b	0
BB-KMR-302		4D	6-8						SM	A-2-4	II
BB-KMR-302		6D	10-12						SM	A-4	II
BB-KMR-303		4D	6-8						SM	A-2-4	II
BB-KMR-303		5D	8-10						SM	A-2-4	II
PC-1			1-3						SP-SM	A-1-b	0

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.

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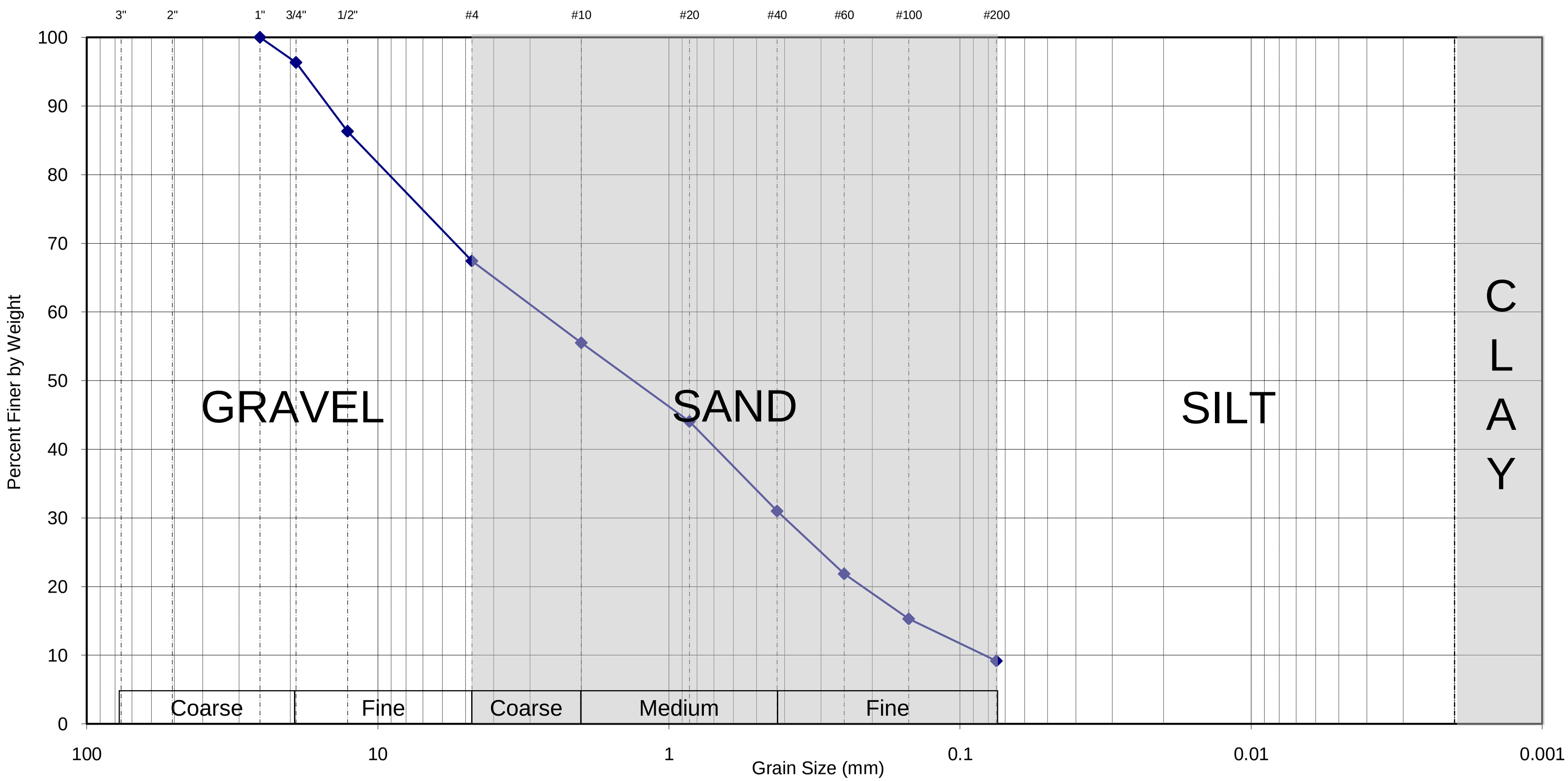
U.S. STANDARD SIEVE AND HYDROMETER



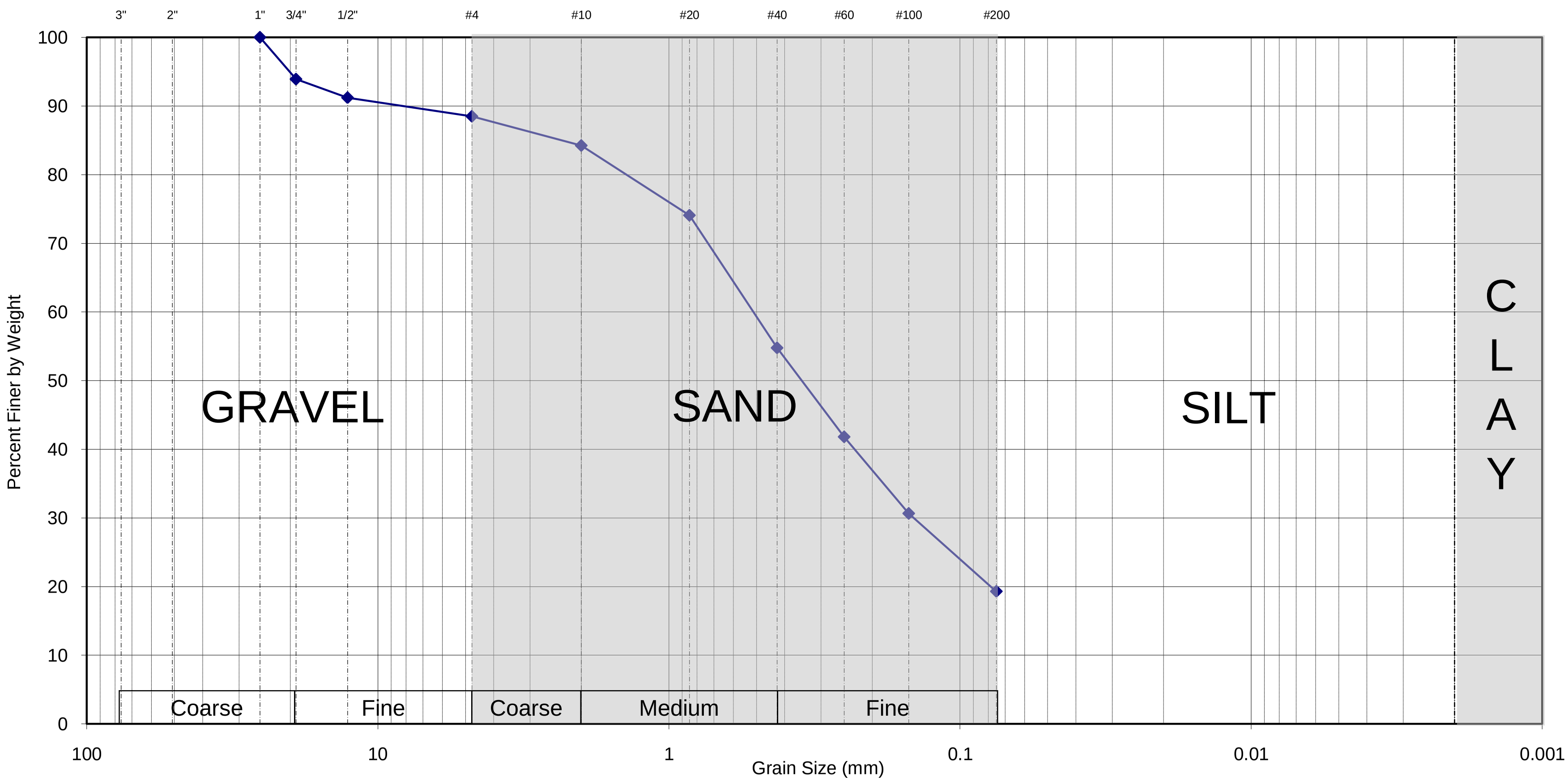
Gravel	Sand	Fines
32.6%	58.2%	9.2%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
1	BB-KMR-201	1D	0.5-2.0'	Brown Poorly-graded Sand with Silt and Gravel (SP-SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



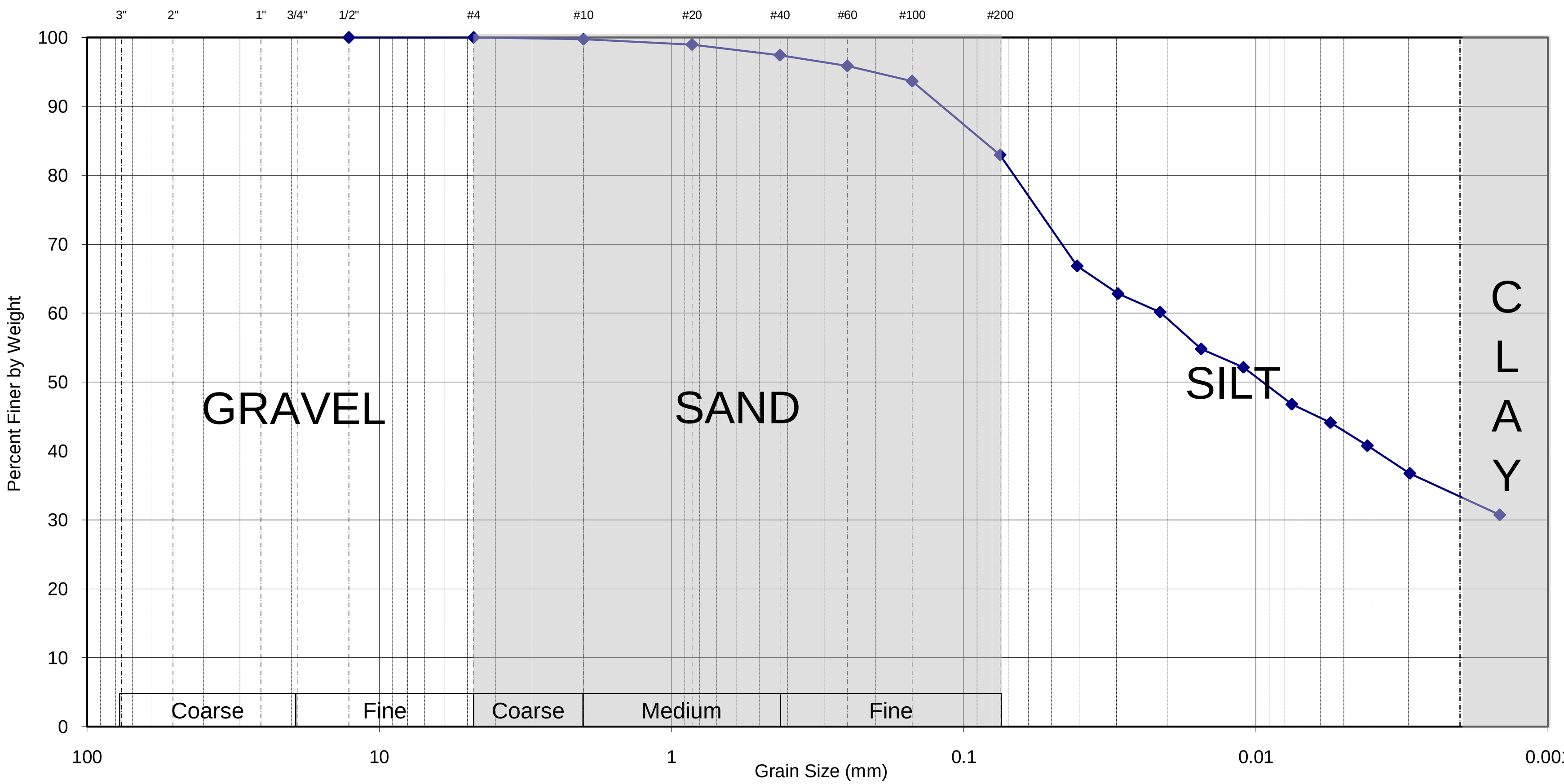
Gravel	Sand	Fines
11.5%	69.2%	19.3%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
2	BB-KMR-201	4D	6-8'	Brown Silty Sand (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



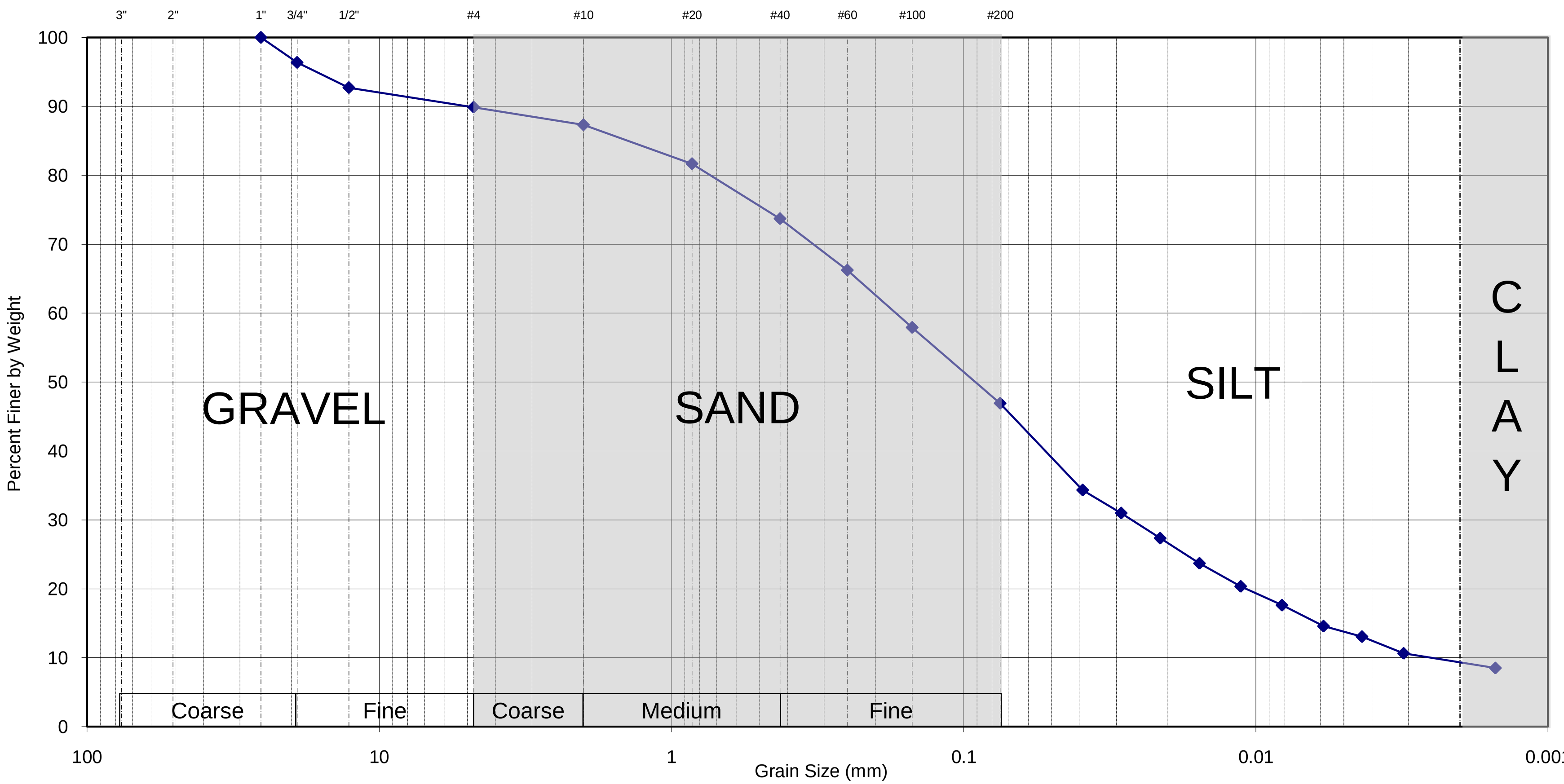
Gravel	Sand	Fines
0.0%	17.0%	83.0%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
3	BB-KMR-201	5D	8-10'	Brown Lean Clay with Sand (CL)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
Reviewed by:	MBP	Date:	6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



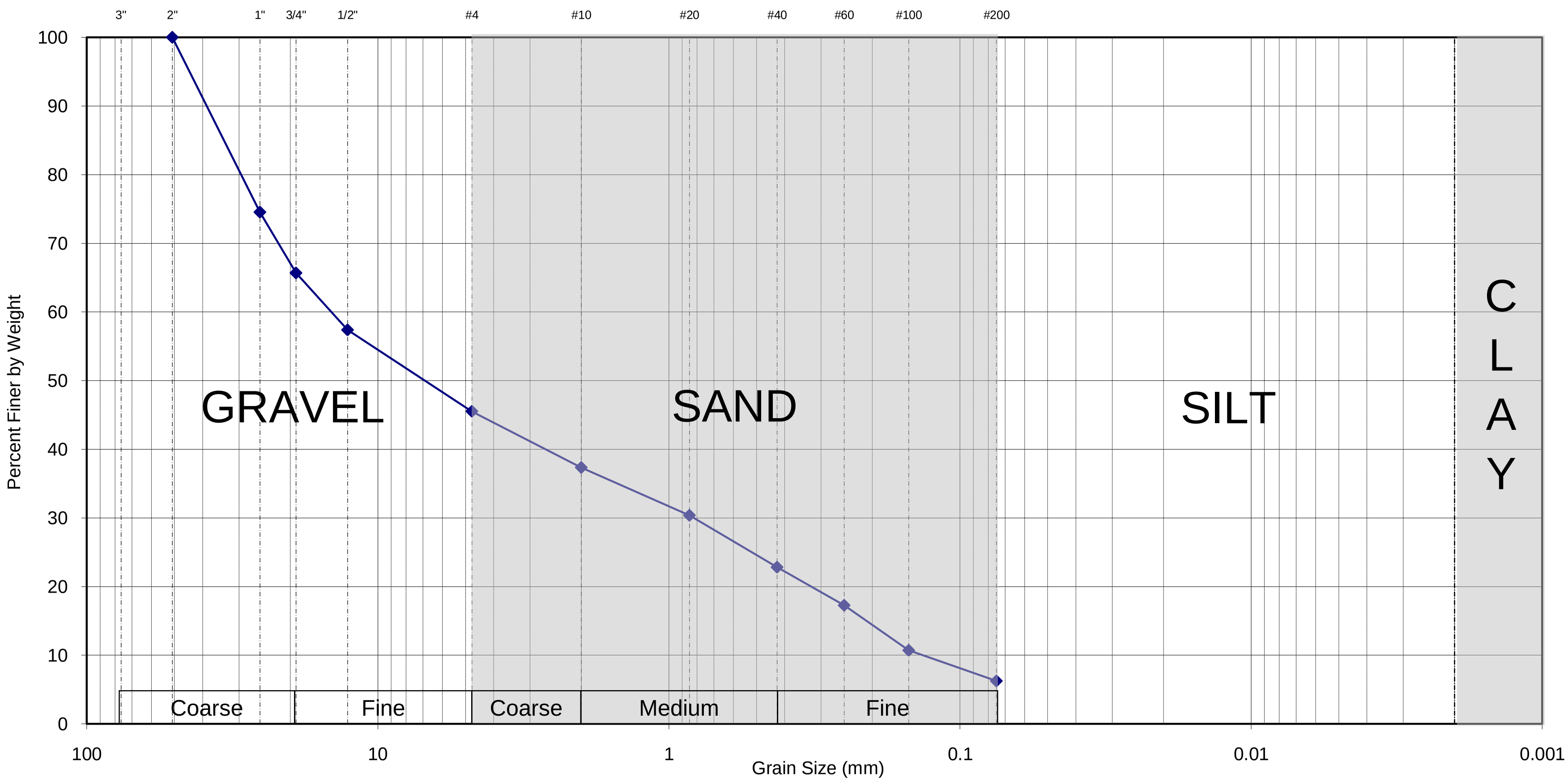
Gravel	Sand	Fines
10.1%	43.0%	46.9%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
4	BB-KMR-201	6D	10-12'	Brown Sandy Silt (ML)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
Reviewed by:	MBP	Date:	6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



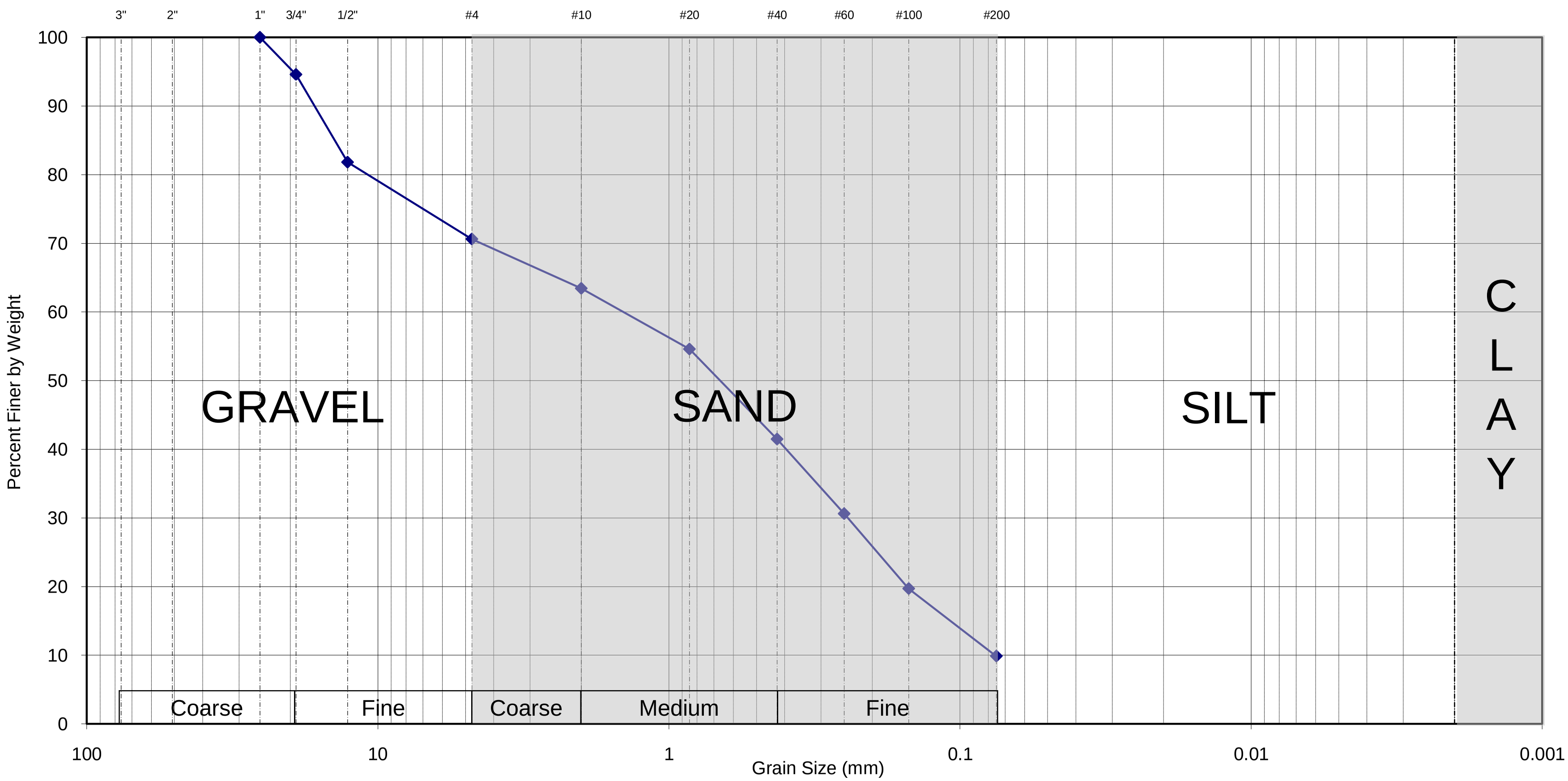
Gravel	Sand	Fines
54.5%	39.3%	6.2%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
5	BB-KMR-202	4D	9-11'	Brown Poorly-graded Gravel with Silt and Sand (GP-GM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



Gravel 29.4%

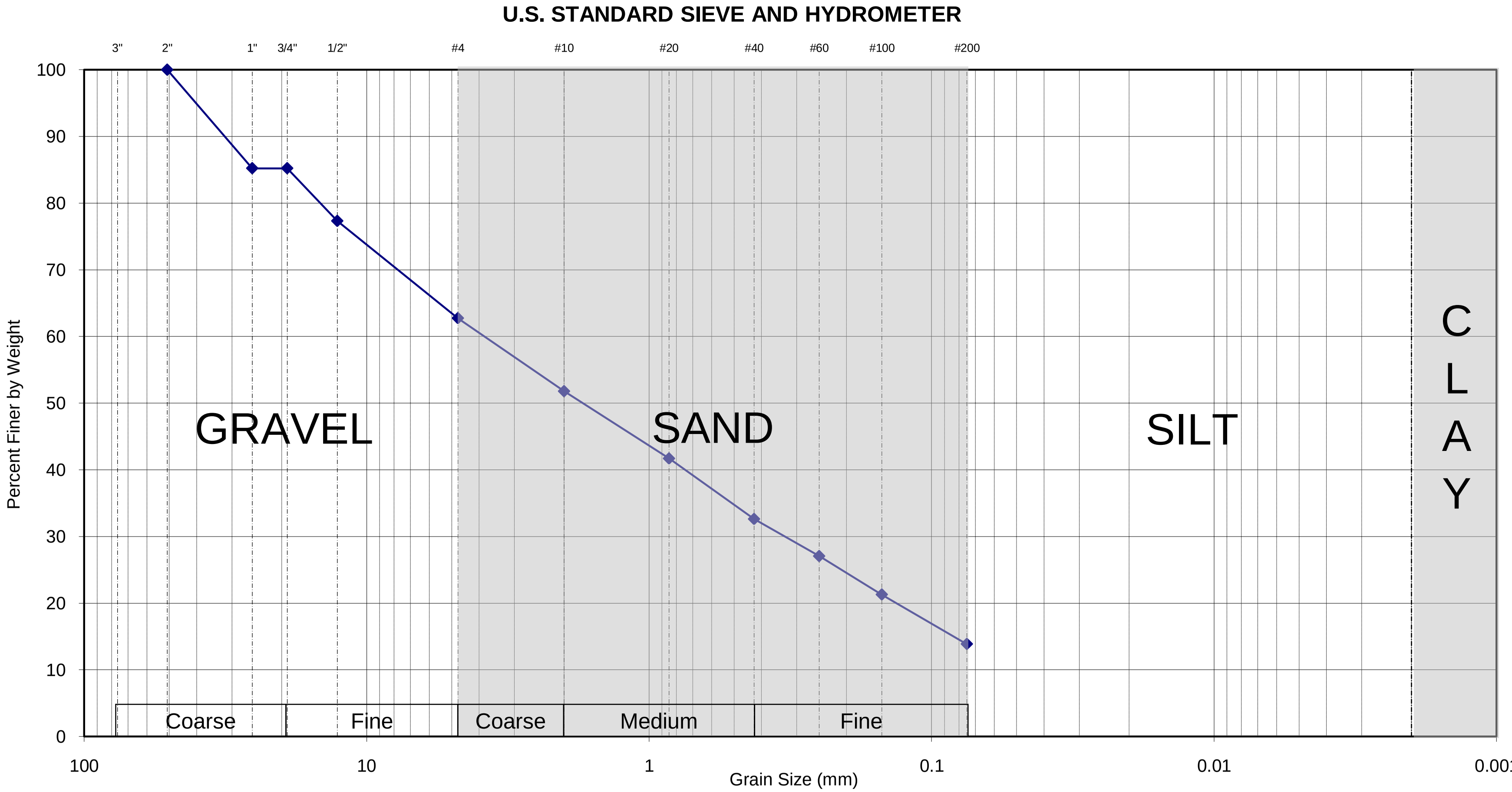
Sand 60.8%

Fines 9.9%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
6	BB-KMR-203	1D	0.5-2.0'	Brown Poorly-graded Sand with Silt and Gravel (SP-SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10

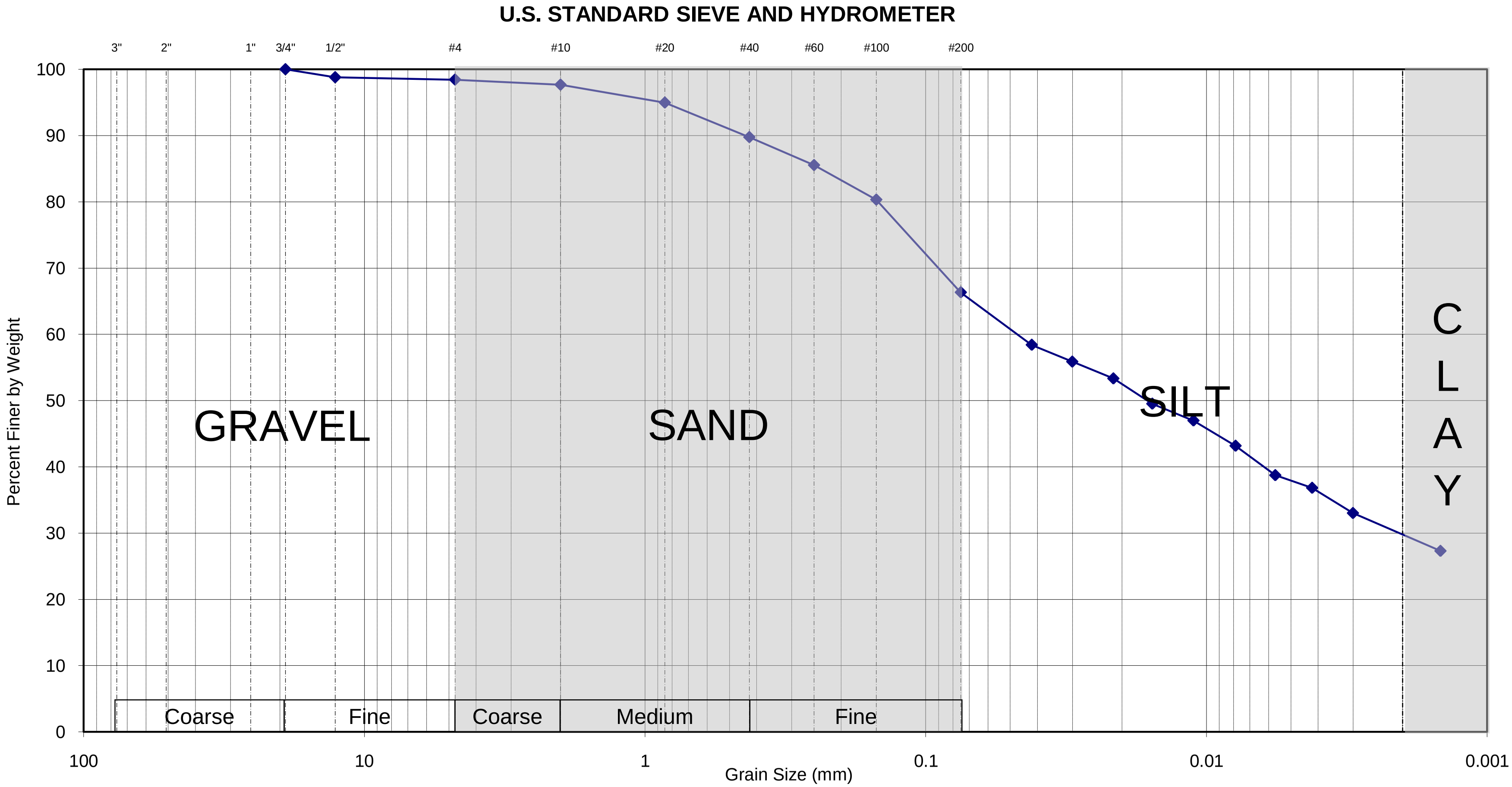


Gravel	Sand	Fines
37.3%	48.9%	13.9%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
7	BB-KMR-203	3D	4-6'	Brown Silty Sand with Gravel (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
Reviewed by:	MBP	Date:	6/22/10



Gravel 1.6%

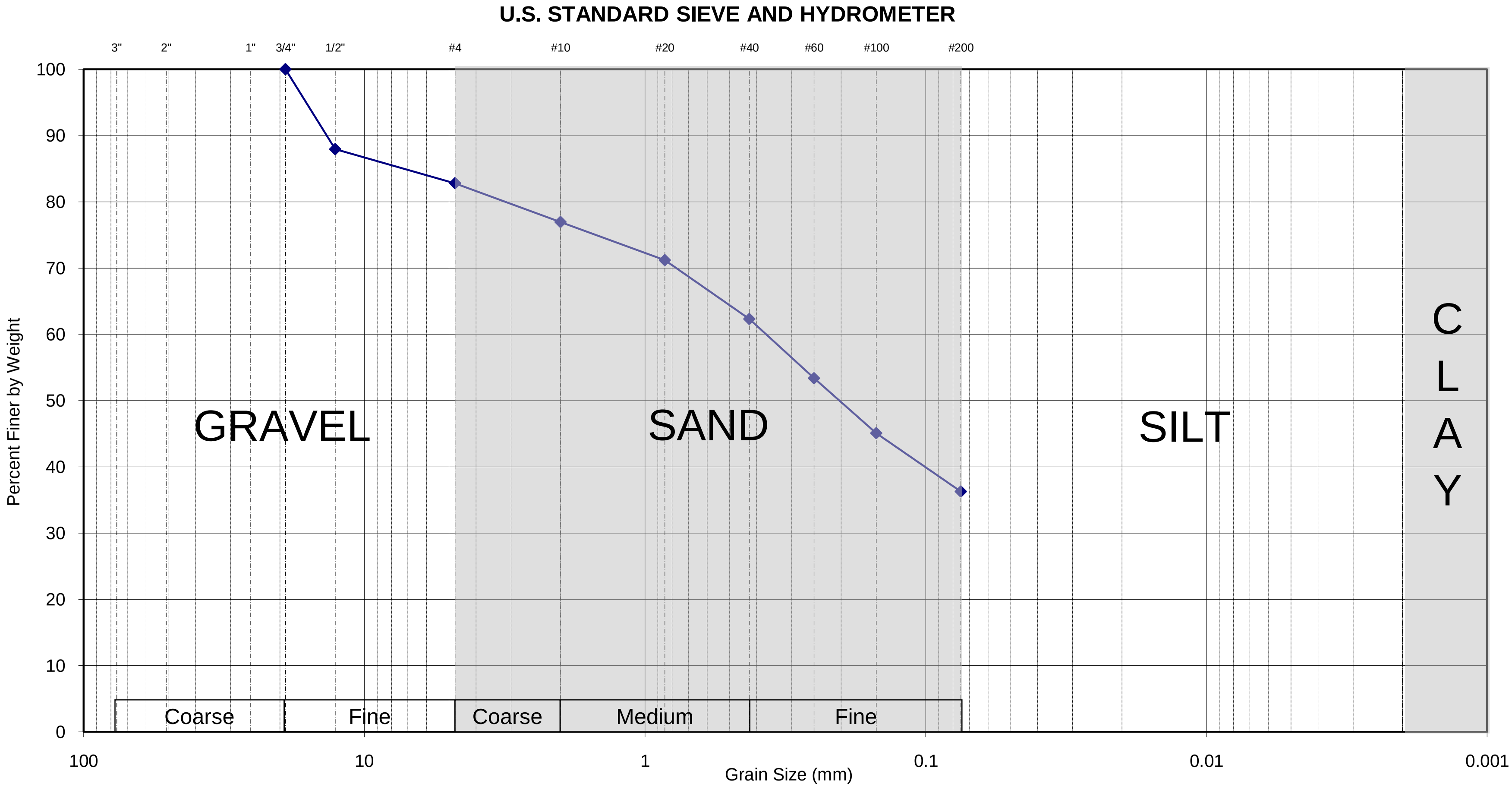
Sand 32.1%

Fines 66.3%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
8	BB-KMR-301	4D	7-9'	Brown Sandy Lean Clay (CL)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
Reviewed by:	MBP	Date:	6/22/10



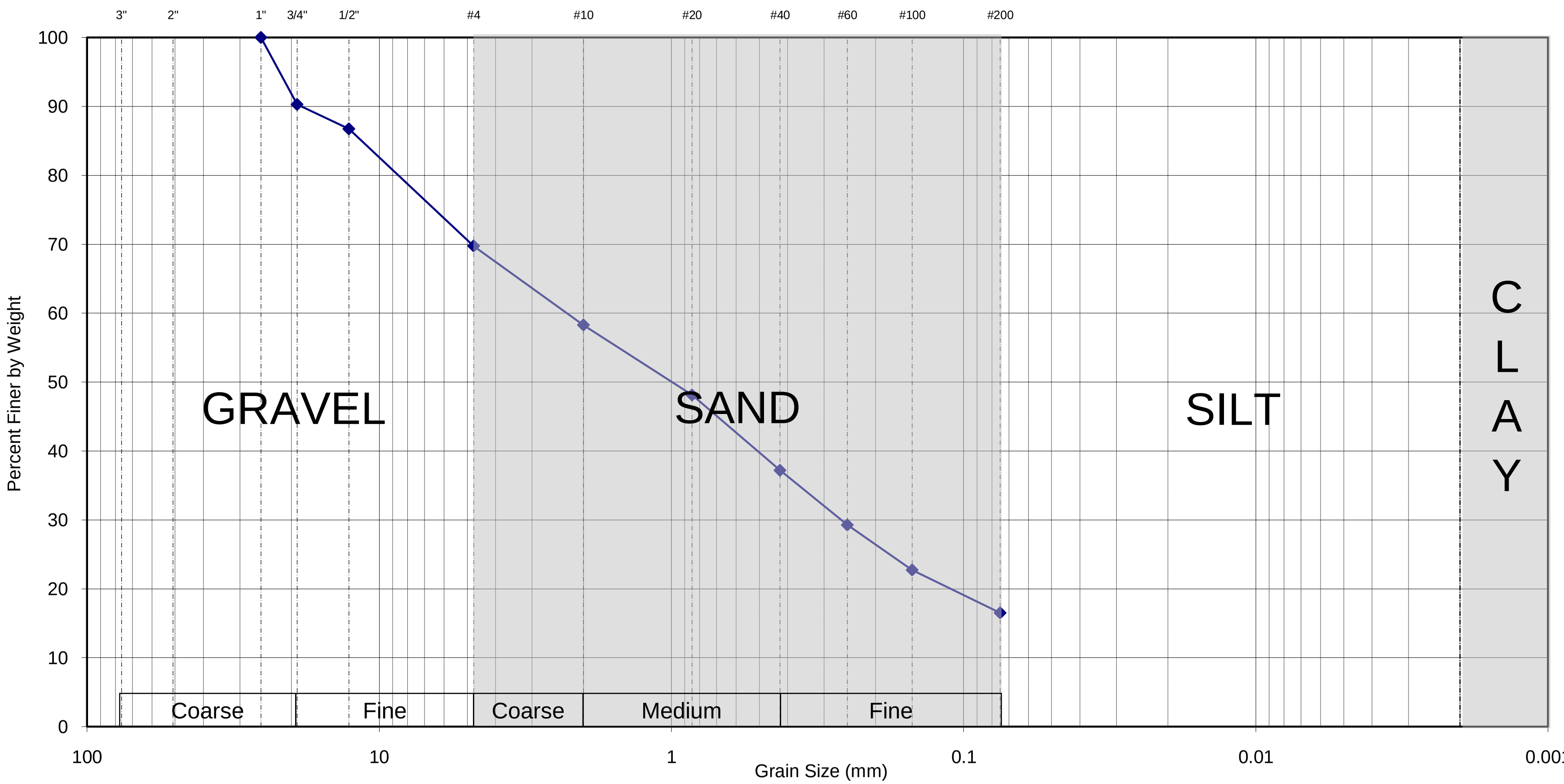
Gravel	Sand	Fines
17.2%	46.5%	36.3%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
9	BB-KMR-301	6D	11-13'	Brown Silty Sand with Gravel (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
Reviewed by:	MBP	Date:	6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



Gravel 30.3%

Sand 53.2%

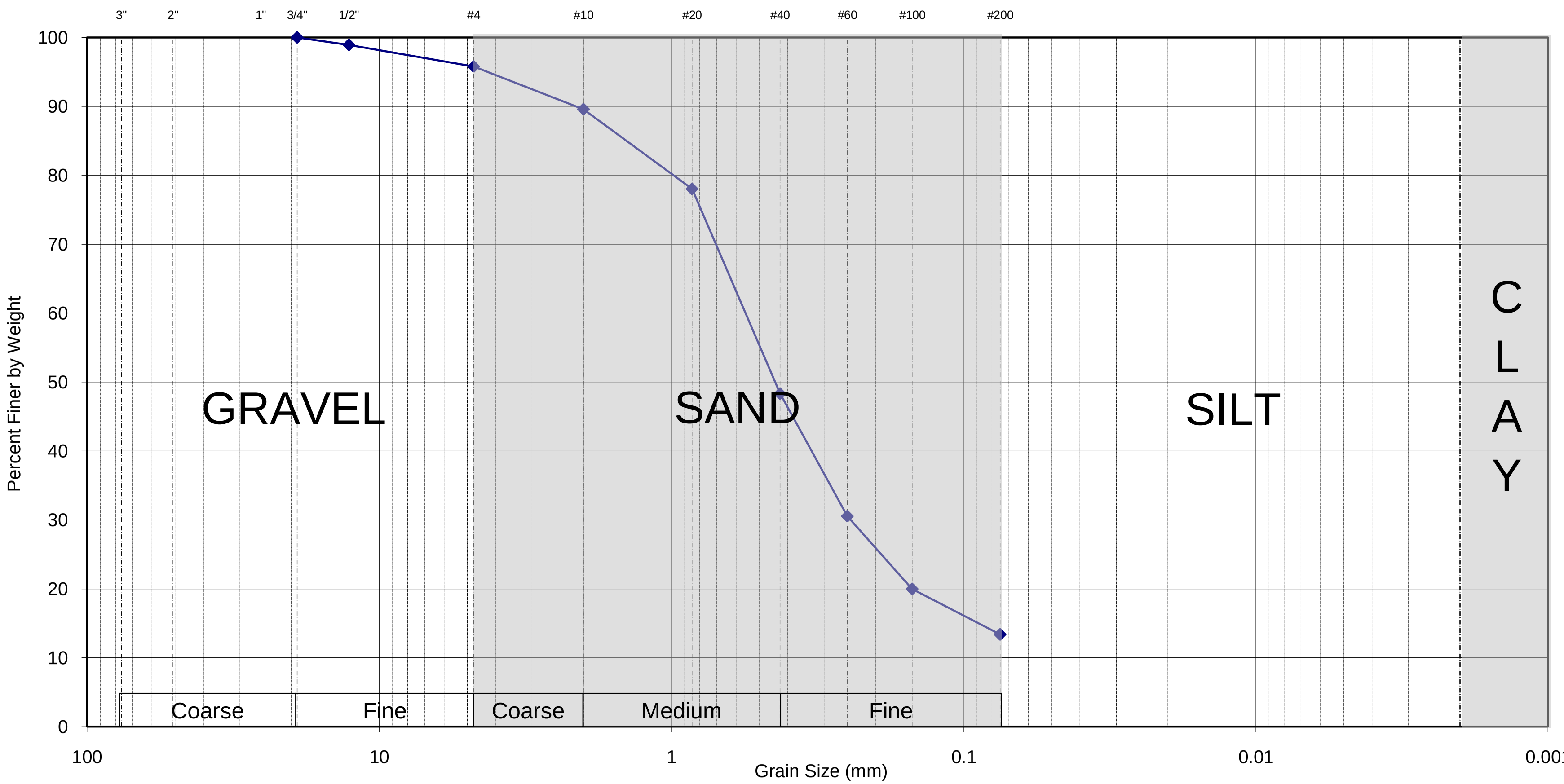
Fines 16.5%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
10	BB-KMR-301	8D	15-17'	Gray-brown Silty Sand with Gravel (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



Gravel 4.2%

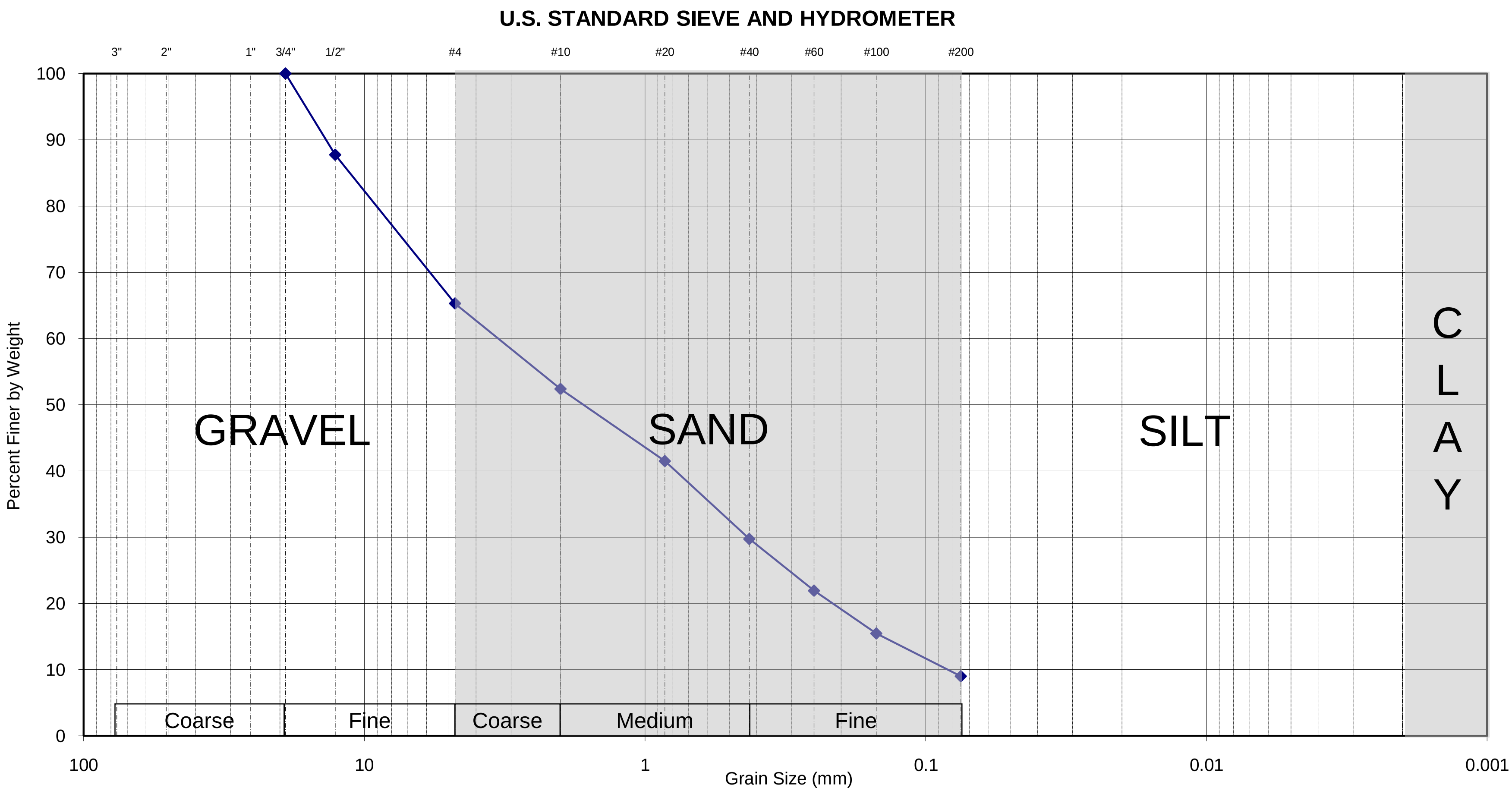
Sand 82.4%

Fines 13.4%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
11	BB-KMR-301	9D	17-19'	Brown Silty Sand (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10



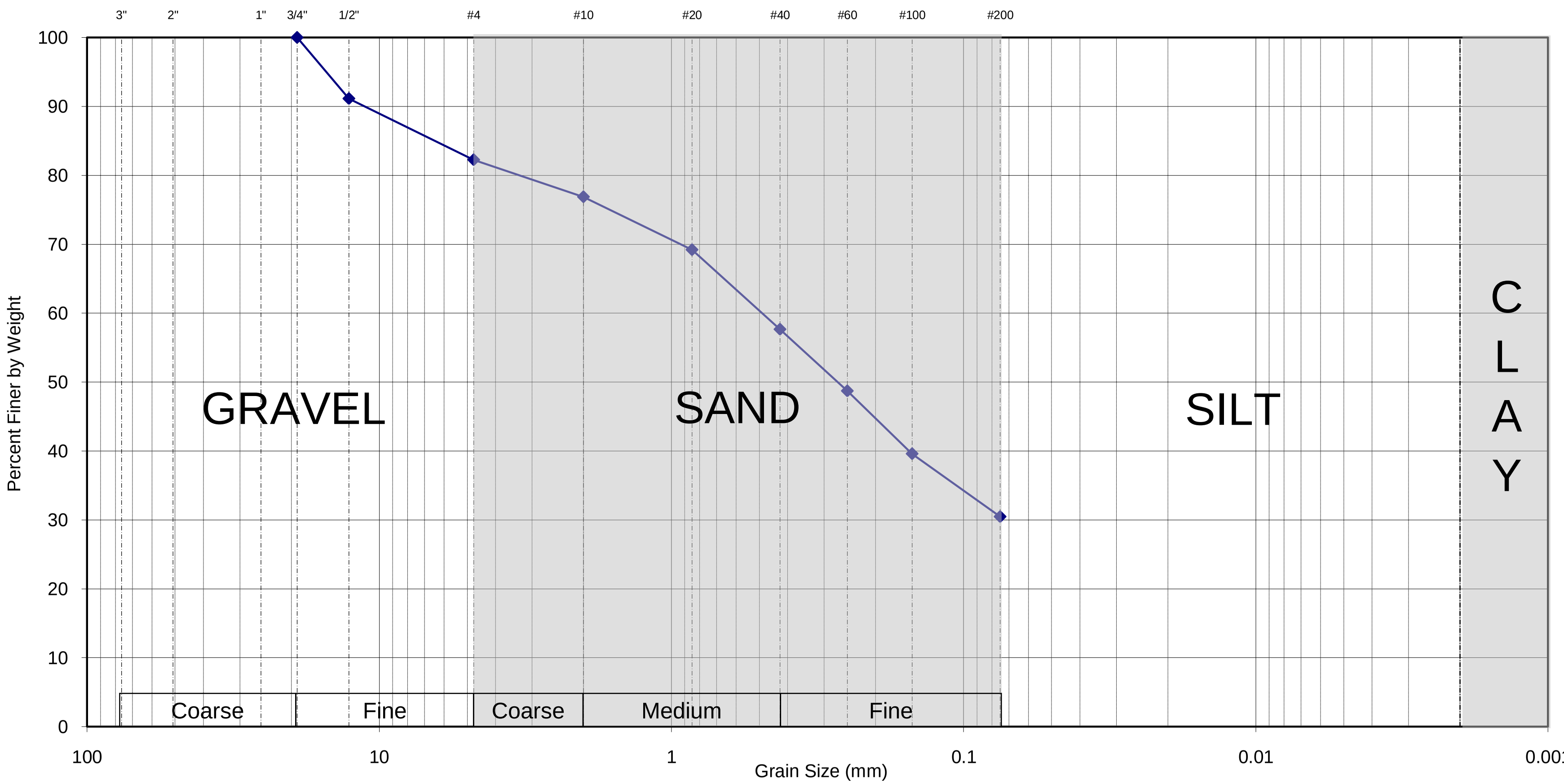
Gravel	Sand	Fines
34.7%	56.3%	9.0%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
12	BB-KMR-302	1D	0.5-2.0'	Brown Poorly-graded Sand with Silt and Gravel (SP-SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10

U.S. STANDARD SIEVE AND HYDROMETER

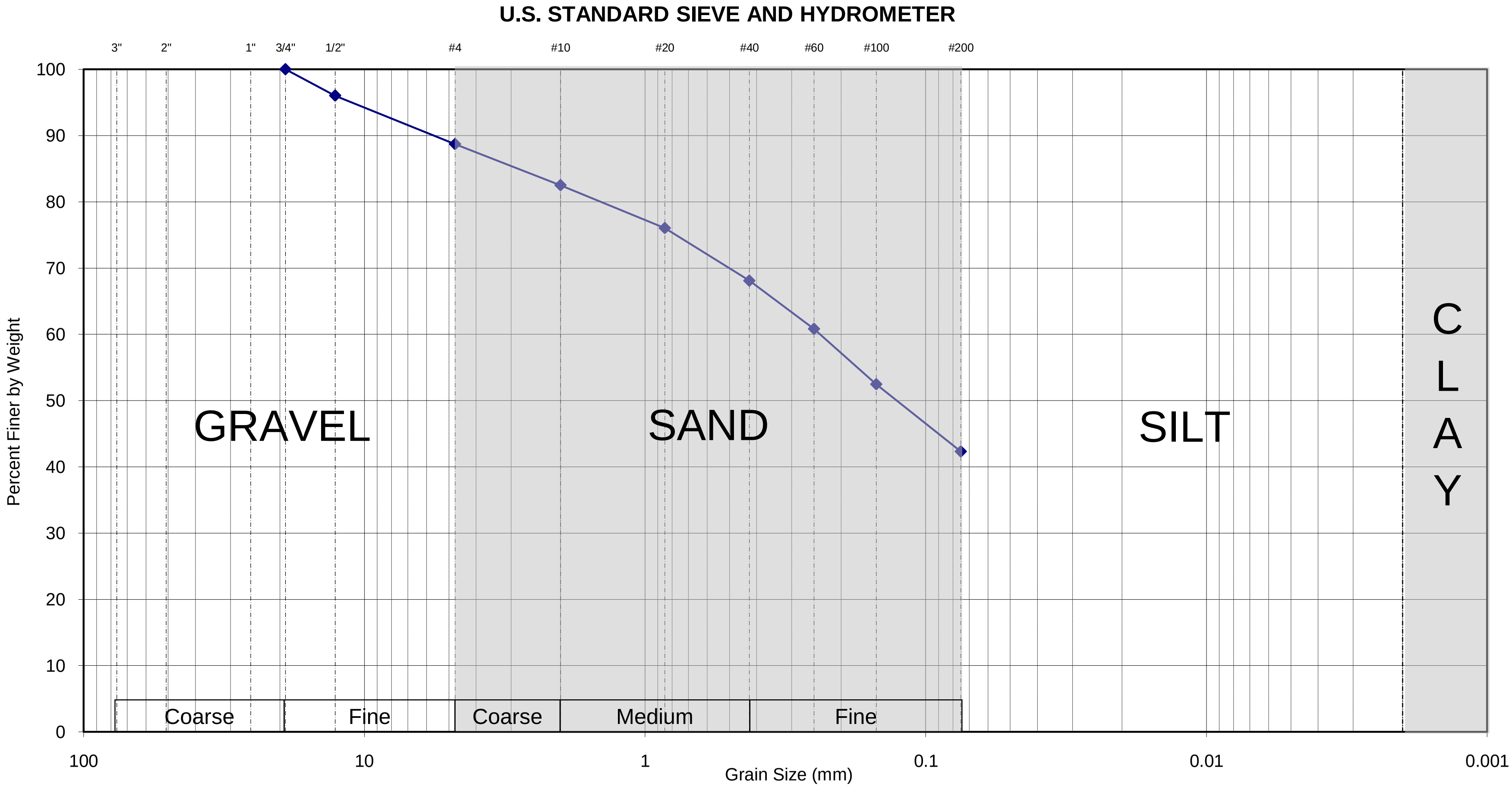


Gravel	Sand	Fines
17.7%	51.8%	30.5%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
13	BB-KMR-302	4D	6-8'	Brown Silty Sand with Gravel (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
Reviewed by:	MBP	Date:	6/22/10



Gravel 11.3%

Sand 46.4%

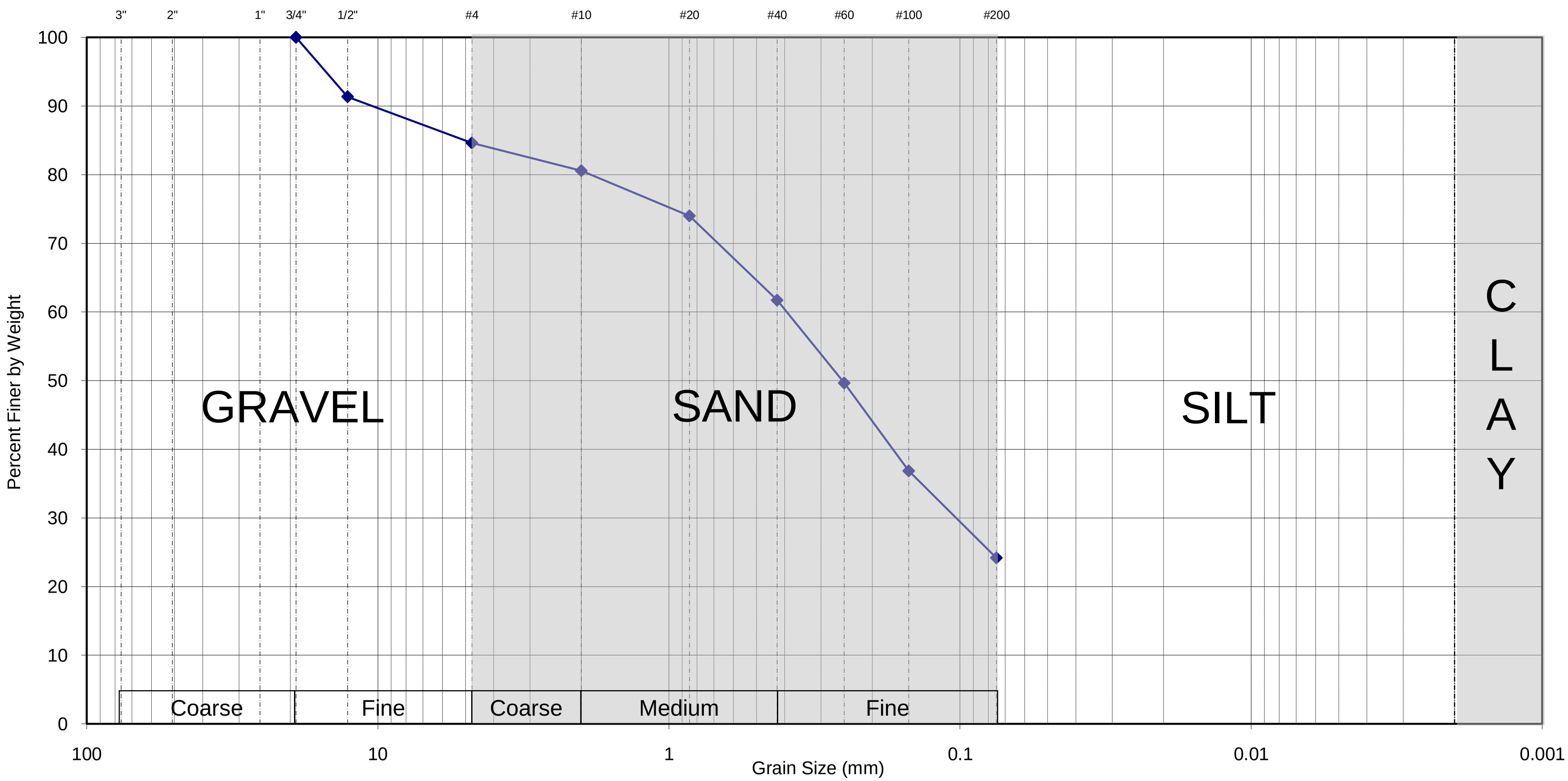
Fines 42.3%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
14	BB-KMR-302	6D	10-12'	Brown Silty Sand (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by: PEC/BB Date: 6/21/10
Reviewed by: MBP Date: 6/22/10

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
15.4%	60.4%	24.2%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
15	BB-KMR-303	4D	6-8'	Brown Silty Sand with Gravel (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
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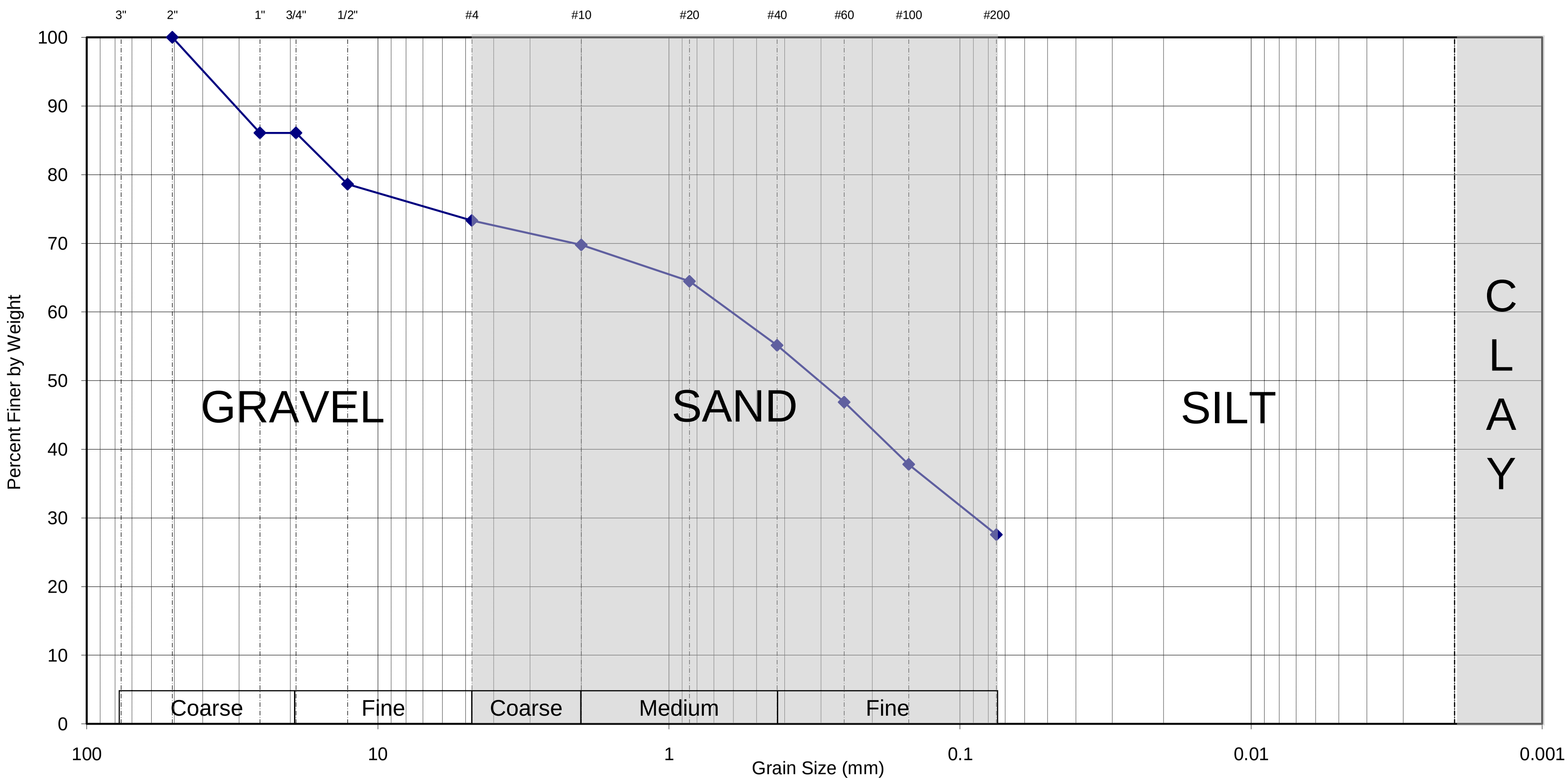
U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
26.7%	45.8%	27.6%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
16	BB-KMR-303	5D	8-10'	Brown Silty Sand with Gravel (SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
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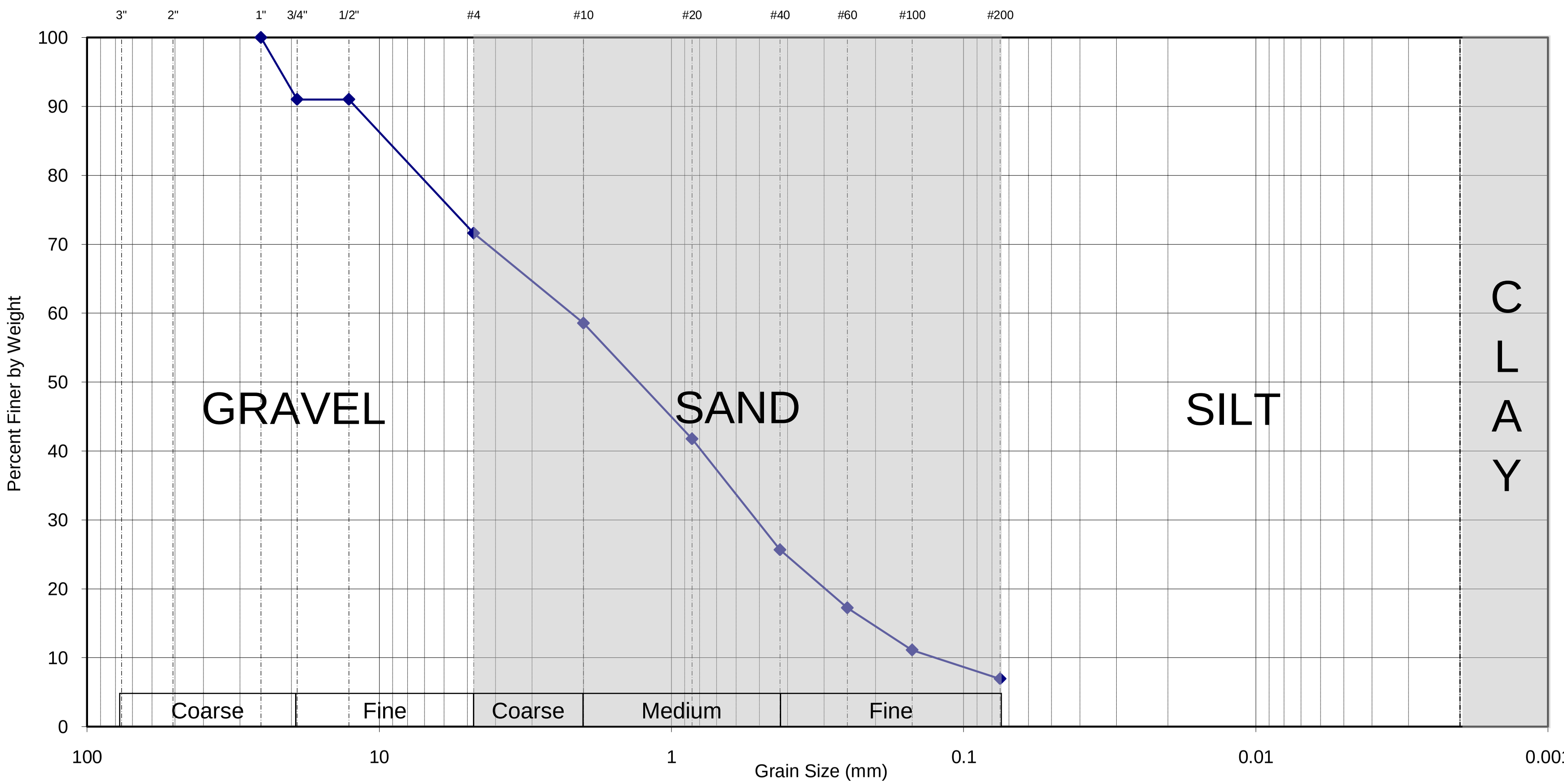

Gravel	Sand	Fines
28.4%	64.7%	6.9%

Lab #	Exploration	Sample	Depth (ft)	Description	WC	LL	PL	PI
17	PC-1	--	1-3'	Brown Poorly-graded Sand with Silt and Gravel (SP-SM)				

Kennebunk Mousam River Bridge
Portland, ME
GZA File # 09.0025597.10

Tested by:	PEC/BB	Date:	6/21/10
Reviewed by:	MBP	Date:	6/22/10

APPENDIX G

CALCULATIONS

GZA
GeoEnvironmental, Inc. *Engineers and Scientists*
4 Free Street
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
http://www.gza.com

JOB: 09.0025597.10
SUBJECT: Kennebunk Bridge, Bedrock Summary, Design-Phase
SHEET: 1
CALCULATED BY: A. Blaisdell, 6/28/10
CHECKED BY: C. Snow, 6/30/10

OBJECTIVE: Determine average RQD of bedrock including design-phase data to confirm applicability of preliminary foundation design evaluations and recommendations.

DATA: Review bedrock RQDs from all test borings:

Boring	Depth of Core (feet) Top	Bottom	Thickness (feet)	RQD (%)	RQD * Thickness
BB-KMR-101	25.2	27.5	2.3	50	115
BB-KMR-102	2.5	7.0	4.5	37	167
	7.0	9.5	2.5	27	68
BB-KMR-103	0.5	5.5	5.0	27	135
	5.5	10.0	4.5	26	117
BB-KMR-104	0.0	5.0	5.0	55	275
BB-KMR-105	4.0	8.5	4.5	28	126
	8.5	13.0	4.5	0	0
BB-KMR-106A	25.5	30.5	5.0	63	315
BB-KMR-201	15.7	20.7	5.0	50	250
	20.7	21.9	1.2	0	0
	21.9	25.9	4.0	54	216
BB-KMR-202	20.2	21.6	1.4	47	66
	21.6	22.4	0.8	60	48
	22.4	27.5	5.1	66	337
	27.6	30.6	3.0	50	150
BB-KMR-203	13.2	18.2	5.0	85	425
	18.2	23.2	5.0	75	375

Total Thickness Cored (feet) **68.3** *(37.8' preliminary, 30.5' design-phase)*

Average RQD per core, Preliminary (%)	35
Average RQD per foot, Preliminary (%)	35
Average RQD per core, All (%)	44
Average RQD per foot, Preliminary (%)	47

CONCLUSION: Bedrock encountered in design-phase borings is of equal or higher quality to the rock cored in preliminary borings. Therefore, foundation evaluations presented in preliminary report are appropriate for design of proposed abutment, retaining wall and wing wall footings.

Preliminary recommendations will be used without modification.

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

Engineers and Scientists

JOB Kennebunk Bridge
SHEET NO. ______ OF ______
CALCULATED BY JRT DATE 6/15/09
CHECKED BY CCS DATE 6/24/09
SCALE ______

Bearing Resistance (For Service Limit State Design)
– Intact Bedrock

① NAVFAC 7.2-142
Allowable BP = 35 tsf
= 70 KSF (Foliated metamorphic rock)

② LRFD Section 10.6.2.6.1
Presumptive Bearing Resistance
Rec. = 70 ksf (Foliated metamorphic rock)

– Stone Masonry

① LRFD Section 10.6.2.6.1
Presumptive Bearing Resistance
Rec = ~~20 KSF~~ (Weathered or broken bedrock of any kind)
USE 14 KSF FOR STONE MASONRY (CONS.)

Strength Limit State:
$\phi = 0.45$
(LRFD Table 10.5.5.2.2-1)

$0.45(70) = 31.5$ ksf intact bedrock

$0.45(\cancel{20}\ 14) = \cancel{9}\ 6$ ksf stone masonry

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Ka calculation for recommended Soil properties

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$

MDOT Type 1 Soil $\phi = 29°$

Type 2 $\phi = 33°$

Type 4 $\phi = 32°$

(1) $K_a = \tan^2\left(45 - \frac{29}{2}\right) = 0.35$

(2) $K_a = \tan^2\left(45 - \frac{33}{2}\right) = 0.29$

(3) $K_a = \tan^2\left(45 - \frac{32}{2}\right) = 0.31$

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Rock Mass Elastic Modulus (E_m)

$$E_m \text{ (ksi)} = 145\left(10^{\frac{RMR-10}{40}}\right)$$

$$= 145\left(10^{\frac{57-10}{40}}\right) = 2170 \text{ ksi} = 2.17 \times 10^3 \text{ ksi}$$

Consistent w/in range for Siltstone in LRFD Table C10.4.6.5-1
(table mean = 2.34×10^3 ksi)

From LRFD Table C10.4.6.5-2
for Siltstone mean $\nu = 0.18$

LRFD Article
10.6.2.4.4
Settlements of Footings on Rock

Bearing on fair to v.good rock ... elastic settlements may generally be assumed to be less than 0.5 in.

When • Rel Rating for RQD + joint spacing ≤ 10 NO (=18)
• Rock joint condition ≤ 10 NO (= 20)
• Or fair to v.good rock not met NO (= Fair)

∴ Settlement analysis Not necessary

JRT 2/07

seismic sensor are synchronized in time based on the selected digital sampling rate of the seismograph. Each seismic event of the wavefield represents different travel paths, particle motions, and velocities of the energy spreading outward from the seismic source. Fig. 2 shows data acquired from a shot in the center of a line of seismic sensors

5.2 *Parameters Measured and Representative Values*—Tables 1 and 2 provide generalized material properties related to the seismic-reflection method.

5.2.1 The seismic-reflection method images changes in the acoustic (seismic) impedance of subsurface layers and features, which represent changes in subsurface material properties. While the seismic reflection technique depends on the existence of non-zero reflection coefficients, it is the interpreter who, based on knowledge of the local conditions and other data, must interpret the seismic-reflection data and arrive at a geologically feasible solution. Changes in reflected waveform can be indicative of changes in the subsurface such as lithology (rock or soil type), rock consistency (that is, fractured, weathered, competent), saturation (fluid or gas content), porosity, geologic structure (geometric distortion), or density (compaction).

5.2.2 *Reflection Coefficient or Reflectivity*—Reflectivity is a measure of energy expected to return from a boundary (interface) between materials with different acoustic impedance values. Materials with larger acoustic impedances overlying materials with smaller acoustic impedances will result in a negative reflectivity and an associated phase reversal of the reflected wavelet. Intuitively, wavelet polarity follows reflection coefficients that are negative when faster or denser layers overlie slower or less dense (for example, clay over dry sand) layers and positive when slower or less dense layers overlie faster or denser (for example, gravel over limestone) layers. A reflectivity of one means all energy will be reflected at the interface.

5.3 *Equipment*—Geophysical equipment used for surface seismic measurement can be divided into three general categories: source, seismic sensors, and seismograph. Sources generate seismic waves that propagate through the ground as either an impulsive or a coded wavetrain. Seismic sensors can measure changes in acceleration, velocity, displacement, or pressure. Seismographs measure, convert, and save the electric signal from the seismic sensors by conditioning the analog

TABLE 1 Approximate Material Properties 4,900 ft/s

Material	*P*-Wave[A] Velocity (m/s)	*S*-Wave[A] Velocity (m/s)	Density (kg/m^3)	Acoustic Impedance[B]
Dry sand/gravel	750[C]	200	1800	1.35×10^6
Clay	900	300	2000	1.80×10^6
Saturated sand	1500	350	2100	3.15×10^6
Saturated clay	1800	400	2200	3.96×10^6
11,500 ft/s Shale	3500	1500	2500	8.75×10^6
Sandstone	2850	1400	2100	5.99×10^6
Limestone	4000	2200	2600	10.4×10^6
7,350 ft/s Granite	6000	3500	2600	15.6×10^6

[A] Velocities are mean for a range appropriate for the material **(7)**.

[B] Acoustic impedance is velocity multiplied by density, specifically for compressional waves; the equivalent for shear waves is referred to as seismic impedance (units of kg/s·m^2).

[C] Subsonic velocities have been reported by researchers studying the ultra-shallow near surface .

4,600 ft/s

TABLE 2 Approximate Reflectivity of Interfaces Between Common Materials

Material Middle Layer[A]	Material Bottom Layer[B]	Approximate Reflectivity[C]
Dry Sand	Dry Sand	0.0
Dry Sand	Dry Clay / Saturated Clay	0.14 / 0.5
Dry Sand	Gravel	−0.08
Dry Sand	Saturated Sand	0.43
Dry Sand	Limestone	0.75
Dry Sand	Shale	0.72
Dry Sand	Sandstone	0.63
Dry Sand	Granite	0.84
Saturated Sand	Granite	0.66
Clay	Dry Sand	−0.14
Clay	Clay	0.0
Clay	Gravel	−0.17
Clay	Saturated Sand	−0.27
Clay	Limestone	0.71
Clay	Shale	0.66
Clay	Sandstone	0.54

[A] Layer 1 on Fig. 1.

[B] Layer 2 on Fig. 1.

[C] *R* in Eq 3. Absolute value *R* = 1 total reflectance.

signal and then converting the analog signal to a digital format (A/D). These digital data are stored in a predetermined standardized format. A wide variety of seismic surveying equipment is available and the choice of equipment for a seismic reflection survey should be made to meet the objectives of the survey.

5.3.1 *Sources*—Seismic sources come in two basic types: impulsive and coded. Impulsive sources transfer all their energy (potential, kinetic, chemical, or some combination) to the earth instantaneously (that is, usually in less than a few milliseconds). Impulsive source types include explosives, weight drops, and projectiles. Coded sources deliver their energy over a given time interval in a predetermined fashion (swept frequency or impulse modulated as a function of time). Source energy characteristics are highly dependent on near-surface conditions and source type (**8-11**). Consistent, broad bandwidth source energy performance is important in seismic reflection surveying. The primary measure of source effectiveness is the measure of signal-to-noise ratio and resolution potential as estimated from the recorded signal.

5.3.1.1 Selection of the seismic source should be based upon the objectives of the survey, site surface and geologic conditions and limitations, survey economics, source repeatability, previous source performance, total energy and bandwidth possible at survey site (based on previous studies or site specific experiments), and safety.

5.3.1.2 Coded seismic sources will generally not disturb the environment as much as impulsive sources for a given total amount of seismic energy. Variable amplitude background noise (such as passing cars, airplanes, pedestrian traffic, etc.) affects the quality of data collected with coded sources less than for impulsive sources. Coded sources require an extra processing step to compress the time-variable signal wavetrain down to a more readily interpretable pulse equivalent. This is generally done using correlation or shift and stack techniques.

5.3.1.3 In most settings, buried small explosive charges will result in higher frequency and broader bandwidth data, in comparison to surface sources. However, explosive sources generally come with use restrictions, regulations, and more

Jennifer Tooley

From: Christopher Snow [christopher.snow@gza.com]
Sent: Wednesday, January 07, 2009 10:01 AM
To: Rudy Rawcliffe
Cc: 'Jennifer Tooley'
Subject: FW: Seismic velocities
Attachments: Seismic velocities.pdf

Thanks Rudi. These are very helpful. Basically, the only rock that won't exceed 5,000 ft/sec is sedimentary or weathered. Most of our metamorphic rocks are going to exceed the shale number which is just under 5,000. Based on these data, I'm comfortable that hard meta siltstone and quartzite would exceed 5,000 ft/sec and be site class A.

Chris Snow

From: Rudy Rawcliffe [mailto:rudy.rawcliffe@gmail.com]
Sent: Wednesday, January 07, 2009 9:23 AM
To: Christopher Snow
Subject: Seismic velocities

Hi Chris: Attached is a page from the ASTM Guide for using Seismic Reflection method for shallow subsurface investigation (ASTM D 7128-05). Table 1 provides the approximate material properties including the P-wave and S-wave velocities. The velocities are in meters per second. If you want feet per second, multiply by 3.28.
let me know if you need any other information.
Rudy

--
Rudy Rawcliffe
Northeast Geophysical Services

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Note: Increased drill core RQD does not change relative rating. RMR is unchanged based on design phase explorations.
A. Blaisdell, 6/28/10

Rock Mass Strength (RMR system)
Table 10.4.6.4-1 AASHTO LRFD

① Compressive Strength : (17 ksi = 2448 ksf)
Relative Rating → 12

② Drill Core RQD : (On 106A, 103 & 105 = 30%) (All borings = 35%)
Relative Rating → 8

③ Spacing of Joints : typically 2in to 1 ft
Relative Rating → 10

④ Condition of Joints : Slightly rough, Hard joint wall rock
Relative Rating → 20

⑤ Groundwater : general conditions water under moderate pressure
Relative Rating → 4

RMR = 12 + 8 + 10 + 20 + 4 = 54

12/30
If assume low strength or max – still falls in the RMR = 41 – 60 range
Class No. III
Fair Rock

ϕ range – Med. friction – Sandstone, siltstone
$\phi = 27 - 34°$