MAINE DEPARTMENT OF TRANSPORTATION
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FINAL GEOTECHNICAL DESIGN REPORT

For the Replacement of:

NAPLES BAY BRIDGE
OVER CHUTES RIVER
US ROUTE 302
NAPLES, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to make geotechnical recommendations for the replacement of the Naples Bay Bridge over the Chutes River on US Route 302 in Naples, Maine. The existing movable swing span structure will be replaced with a combination single leaf hydraulic lift span and fixed span structure. The proposed bridge superstructure will be widened from the existing 28 ft to 53 ft to meet current design standards. There will be a 5-ft raise in finished grade for the vertical alignment. The horizontal alignment of the bridge may be shifted upstream as a part of the bridge widening. The following design recommendations are discussed in detail in this report:

Abutment No. 1 - This abutment will be founded on H-piles driven to bedrock. In addition to the typical axial and lateral loads and forces seen by an abutment, Abutment No. 1 will also be subjected to the lateral and uplift forces imparted on the substructure in opening and closing the lift span and while the span is in the open position. Abutment No. 1 should be designed to withstand uplift forces generated by the opening of the hydraulic lift span. The design uplift capacity of a single pile is specified as 1/3 the ultimate shaft resistance calculated in a static analysis method. It is recommended that the Structural Designer submit the loading requirements to the Geotechnical Designer during final design for geotechnical evaluation. Design axial, lateral and uplift loads should be shown on the plans.

Abutment No. 2 - This abutment will be founded on H-piles driven to bedrock. Analyses indicate that the geotechnical capacity of the piles exceeds the structural capacity and therefore the structural capacity governs. Design axial loads should be shown on the plans. The Rankine active earth pressure coefficient of $K_a = 0.307$ is recommended.

Pipe Pile Bent Pier - A pipe pile bent pier is one alternative proposed at the site. The steel pipe piles shall be end bearing on the bedrock. The use of Grade 3 steel (45 ksi yield strength) steel, a concrete core and a reinforcing cage providing 2% reinforcement is recommended. The piles can be driven open-ended with a stiffening ring/cutting shoe or close-ended with a conical point. Capacity calculations indicate that the structural capacity of the piles is less than the geotechnical capacity and therefore governs. The pier should be designed to account for any unbalanced forces due to the opening of the lift span. The pier bent should also be designed to account for additional unsupported pile length due to scour conditions present at the site. Design axial and lateral loads should be shown on the plans. It is recommended that the Structural Designer submit the loading to the Geotechnical Designer during final design for geotechnical evaluation.

Pile Supported Mass Concrete Pier - A mass concrete bridge pier supported on driven H-piles is one alternative proposed at the site. Analyses indicate that the geotechnical capacity of the H-piles exceeds the structural capacity and therefore the structural capacity governs. The pier should be designed to account for any unbalanced forces due to the opening of the lift span. The pier pile group should also be designed to account for additional unsupported pile length due to scour conditions present at the site. Design axial and lateral loads should be shown on the plans. It is recommended that the Structural Designer submit the loading to the Geotechnical Designer during final design for geotechnical evaluation.

Fender System - The use of a fender system to protect the substructures is planned. The fender system will consist of concrete filled pipe piles. The fender piles will be friction piles and should be designed with consideration of the unsupported length due to scour. The fender piles can be driven closed-ended with a conical point.

Bearing Capacity - The applied bearing pressure for any structure founded on the native sand layer should not exceed the calculated allowable bearing capacity of 3 tsf.

Settlement - Any settlement of the bridge foundations will be due to the elastic compression of the piling. Due to the presence of granular soils underlying the approaches, settlements in this area are anticipated to be less than 1 inch and will occur during construction having minimal effect on the finished structure.

Retaining walls - All retaining walls should be designed to achieve a factor of safety of 2.0 against overturning and a factor of safety of 1.5 against sliding. An active earth pressure coefficient, K_a, shall be calculated using Rankine Theory for cantilever wingwalls and Coulomb Theory for gravity shaped structures. Drainage behind structure shall be in accordance with Section 5.4.1.4 of the BDG.

Frost Protection - Any foundations placed on native subgrade soil should be founded a minimum of 5.5 ft below finished exterior grade for frost protection.

Scour - The following riverbed grain size parameters are to be used in scour analysis: AASHTO Soil Type A-2-4, D_{50} = 0.32 mm. A scour analysis will be performed as a part of the PDR process. The resulting scour depth should be considered in the design of the pier and fender system and will result in a significant unsupported length of the pile for design.

Seismic Design Considerations - The Naples Bay Bridge on US Route 302 is on the National Highway System (NHS) and is therefore considered to be functionally important. As a result, the bridge substructures should be designed for seismic earth loads. The bridge substructures should be designed for seismic earth loads assuming A=0.09.

Construction Considerations - The existing timber piles shall be cut off 1 ft below the bottom of the proposed abutment and/or 1 ft below the riverbed surface. There is a potential for the existing timber piles to interfere with the installation of the proposed piles. There is a potential for the proposed piles to encountered cobbles and boulders causing them to deflect during driving. Pre-drilling of the pile locations may be necessary in order to advance the piles through the cobbles and boulders.

Additional Work - A test boring will be conducted at the pier location in the Spring of 2006 in order to obtain information for final design.

1.0 INTRODUCTION

The purpose this Geotechnical Design Report is to present geotechnical recommendations for replacement of the Naples Bay Bridge over the Chutes River on US Route 302 in Naples, Cumberland County, Maine. This report presents the soils information obtained at the site during the subsurface investigation, geotechnical design recommendations for bridge replacement, and final foundation recommendations and information.

The existing Naples Bay Bridge was built in 1954 and is a single, swing span of 55 ft. The existing superstructure has a width of approximately 28 ft. The existing bridge substructures consist of two wall type cast-in-place concrete abutments supported on driven timber piles. The swing span is supported on a cast-in-place concrete center pier on driven timber piles.

The proposed bridge superstructure will be a two-span structure made up of a combination of a 45 ft movable span and a 42 ft fixed span with an intermediate pier. The movable span will be a hydraulic lift structure. The proposed bridge substructures will include two H-pile supported abutments and one pile supported pier. A fender system for the protection of the substructures adjacent to the channel will be used. The fender system will consist of two rows of concrete filled pipe piles. The bridge will be widened from the existing 28 ft to 53 ft to meet current design standards. There will be a 5-ft raise in finished grade in the vertical alignment. The horizontal alignment of the bridge may be shifted upstream as a part of the bridge widening. The existing bridge will be entirely removed.

2.0 GEOLOGIC SETTING

The Naples Bay Bridge on US Route 302 in Naples, Maine crosses the Chutes River approximately 0.3 miles north of Route 35 in Naples as shown on *Sheet 1 - Location Map* found at the end of this report. The Chutes River flows into the Bay of Naples (Brandy Pond) which in turn flows into the Songo River and ultimately into Sebago Lake.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of differing geologic units from east to west. To the east, the surficial soils consist of ice-contact glaciofluvial deposits made up of sand, gravel and silt. These soils are typically deposited by meltwater streams adjacent to stagnant glacial ice. To the west, the surficial soils consist of till deposits made up of a heterogeneous mixture of sand, silt, clay, and stones. These soils are generally deposited in a blanket deposit that conforms to the underlying bedrock topography. These soils are deposited directly by glacial ice.

According to the Bedrock Geologic Map of Maine (1985) the bedrock in the vicinity of the site consists of igneous carboniferous muscovite-biotite granite commonly known as the Sebago pluton. This bedrock is anticipated to be hard and sound.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored in March 2004. Borings BB-NBB-101 and BB-NBB-102 were drilled behind the location of the existing bridge abutments. Boring locations are shown on *Sheet 2 - Boring Location Plan* and *Sheet 3 - Interpretive Subsurface Profile* found at the end of this report. Boring BB-NBB-101 was drilled behind the western abutment to a depth of approximately 92.0 ft below the ground surface (bgs). Boring BB-NBB-102 was drilled in behind the eastern abutment to a depth of approximately 65.7 ft bgs. The borings were located in the field by use of a tape after completion of the drilling program.

The borings were drilled by the MaineDOT Materials Testing & Exploration team. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and graphically on *Sheet 4 - Boring Logs* found at the end of this report. Drilling in soil was performed using cased wash boring techniques. Soil samples were obtained at 5-ft intervals using Standard Penetration Test (SPT) methods. Drilling in bedrock was performed using diamond rock coring with a NQ-sized (1.88 inch) double tube core barrel with which rock core samples were obtained. Rock Quality Designations (RQDs) were calculated for the rock cores obtained. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field and laboratory testing requirements.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the two borings consisted of seven (7) Grain Size Analyses. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information is also shown on the Boring Logs in Appendix A and on *Sheet 4 - Boring Logs* found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the explorations generally consisted of **fill soils** overlying a layer of **sand** which is underlain by **bedrock**. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show on *Sheet 3 - Interpretive Subsurface Profile* found at the end of this report. A brief summary description of the strata encountered is as follows:

5.1 Fill

A layer of fill soils was encountered behind the abutments in both of the borings. The layer ranged in thickness from approximately 15 ft in boring BB-NBB-101 to approximately 14 ft in boring BB-NBB-102. The fill soils generally consisted of brown, damp, fine to coarse sand, with little to trace gravel and trace silt. SPT N-values in the fill layer ranged from 15 to 43 blows per foot (bpf) indicating that the layer is of a medium dense to dense consistency. A zone of cobbles and boulders was encountered at the bottom of this layer in boring BB-NBB-101. No laboratory testing was conducted on the soil samples collected from this layer.

5.2 Sand

A sand layer was encountered below the fill soils in both of the borings. The thickness of the layer ranged from approximately 71.9 ft in BB-NBB-101 to approximately 46.7 ft in BB-NBB-102. This layer is described as a light brown to golden brown in the upper portions changing to grey with depth. The layer was made up of wet, fine to medium sand with trace gravel and silt in the upper portions grading to wet, fine to coarse sand trace gravel with depth. SPT N-values in the layer ranged from 3 to 29 bpf indicating a soil of very loose to medium dense consistency. Blow counts were generally higher in the lower sands. The moisture content of seven (7) samples tested from this layer ranged from 14.6% to 23.7%. Grain size analyses conducted on the samples from the upper portion of the layer indicate that the soil is classified as an A-3 or an A-2-4 by the AASHTO Classification System and a SP-SM or SM by the Unified Soil Classification System. Grain size analyses conducted on one sample from the lower portion of the layer indicate that the soil is classified as an A-1-b by the AASHTO Classification System and a SW by the Unified Soil Classification System. Results of the moisture content testing can be found in Appendix B - Laboratory Data. This information is also shown on the boring logs in Appendix A and on *Sheet 4 - Boring Logs* found at the end of this report.

5.3 Bedrock

The bedrock at the site was cored in both of the borings. According to the Bedrock Geologic Map of Maine (1985) the bedrock in the vicinity of the site consists of igneous carboniferous muscovite-biotite granite commonly known as the Sebago pluton. The bedrock is pink in color and medium grained. Rock Quality Designation of the bedrock ranged from 43% to 100% indicating that the rock is of poor to excellent quality. The top of the bedrock surface slopes from an elevation of approximately 190.1 ft at the western abutment to an elevation of approximately 217.3 ft at the eastern abutment.

5.4 Groundwater

Groundwater was observed at a depths ranging from approximately 16 ft to 16.5 ft bgs at the abutments. These water levels are shown on the boring logs in Appendix A of this report. The water level readings were taken during drilling activities and may not represent the stabilized groundwater level. Groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

Both pile supported foundations and spread footings were considered for use at this site.

H-Pile Foundations. The use of driven H-piles foundations is a viable alternative for use at the site. The piles should be end bearing, driven to refusal on the bedrock.

Pipe Pile Foundations. The use of driven pipe piles foundations is a viable alternative for use at the site. The piles should be end bearing, driven to refusal on the bedrock.

Fender system. The use of a driven pipe pile fender system is viable for the protection of the structure.

Spread footings founded on native sands. The use of spread footings founded on native sands is not a viable foundation alternative for this site. Due to the presence of highly scour susceptible sands in the riverbed, the use of spread footings is not recommended.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following Subsections discuss foundation considerations and recommendations for pile supported foundations. In the event that spread footings founded on native sands are used for any structure, recommendations for their use are also presented.

7.1 Driven H-Pile Abutment Foundations

Both of the cast-in-place concrete abutments will be supported on driven H-piles. Three pile sizes were considered for use: HP 12 x 53, HP 14 x 73 or HP 14 x 89. It is assumed that 50 ksi steel will be used. The piles should be fitted with pile driving points to protect the tips and to improve penetration. The following paragraphs address the individual components of the H-pile foundations:

Pile Length. Pile length at the abutments can be estimated based on the following data:

Location	Ground Elevation	Depth to Rock	Approximate Top of Rock Elevation	Estimated Pile Length	Rock Quality Designation
Abutment #1 BB-NBB-101	277.0 ft	86.9 ft	194 ft	66 ft	100%
Abutment #2 BB-NBB-103	278.0 ft	60.7 ft	217 ft	43 ft	43%

Axial Capacity. For non-integral structures the MaineDOT Bridge Design Guide (BDG) recommends a factor of safety of 3.0 or 0.33Fy for the maximum axial design load. The geotechnical and structural capacities of the H-piles are summarized in the table below. Calculations can be found in Appendix C at the end of this report.

Pile Type	Allowable end bearing axial capacity, $Q_{t, allow}$ (Goodman's Bedrock Condition) FS = 2.25	Total Allowable $Q_{total, allow}$ Axial Structural Capacity: 50 ksi FS = 3
HP 12 x 53	266 kips	**256 kips**
HP 14 x 73	368 kips	**353 kips**
HP 14 x 89	449 kips	**431 kips**

Using the assumption that 50 ksi steel will be used; the geotechnical capacity of the piles exceeds the structural capacity and therefore **the structural capacity governs**. Design axial loads should be shown on the plans. No downdrag should be considered. The piles should be driven to refusal on or within the bedrock. The soils encountered at the site will provide sufficient lateral support to assume the H-piles are fully braced against Euler buckling. The Designer should check that pile axial stresses from the dead loads, live loads, and pile dead load forces do not exceed the allowable axial pile loads.

Lateral Capacity. In addition to the traditional axial and lateral forces seen by an abutment, Abutment No. 1 will also be subjected to the lateral forces imparted on the substructure during opening and closing the lift span and while the span is in the open position. Lateral loads may be resisted using battered piles. In lateral capacity analysis, fixity may be assumed at the pile tip/rock interface. The deflection of a pile group under a lateral load is typically 2 to 3 times larger than the deflection of a single pile loaded to the same intensity. This is due to the pile-soil-pile interaction that takes place in a pile group. Passive earth pressure on the pile cap, reduced by the design scour depth, may be considered as contributing to resistance of lateral loads. The magnitude of this lateral capacity will be determined during final design of the abutments. It is recommended that the Structural Designer submit the loading and horizontal movement criteria to the Geotechnical Designer during final design for geotechnical evaluation.

Uplift Capacity. In addition to the typical axial and lateral forces seen by an abutment, Abutment No. 1 will also be subjected to the uplift forces imparted on the substructure during opening and closing the lift span and while the span is in the open position. The design uplift capacity of a single pile is specified as 1/3 the ultimate shaft resistance calculated in a static analysis method.

The following table summarizes design uplift capacities for the piles proposed at the site:

Pile Size	Shaft Resistance (Ultimate) (kips)	Uplift Capacity (kips)
12 x 53	118	40
14 x 73	159	53
14 x 89	174	58

It is recommended that the Structural Designer submit the loading and design vertical movement criteria to the Geotechnical Designer during final design for geotechnical evaluation.

Wave Equation Analysis. The Contractor shall be required to perform a wave equation analysis for each substructure, pile type and proposed hammer for the approval of the Geotechnical Engineer. Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of the wave equation analysis. Contract documents should require that the contractor perform a wave equation analysis of the proposed pile driving system, and the piles be driven to 2.25 times the design (working) load. This factor of safety assumes field dynamic testing will be performed. A hammer should be selected which provides the required geotechnical capacity when the penetration resistance for the final 3 to 6 inches is 8 to 13 blows per 1 inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows. Allowable pile stresses during driving shall be less than 0.90Fy, per AASHTO 4.5.11.

Dynamic Pile Testing. The first pile driven at each substructure should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor. With this level of quality control, the piles shall be driven to an ultimate capacity of 2.25 times the design load.

7.2 Abutments

The Designer may assume Soil Type 4 (BDG Section 3.6.1) for abutment back fill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf and a soil-concrete friction coefficient of 0.45. If the abutment sections are designed as unrestrained, meaning that they are free to rotate at the top in an active state of earth pressure, the Rankine active earth pressure coefficient of $K_a = 0.307$ is recommended. If the abutment sections are designed to be fixed at the top in an at rest state of earth pressure, the earth pressure coefficient of $K_o = 0.47$ is recommended. If an approach slab is not specified, additional lateral earth pressure due to construction surcharge or traffic surcharge is required per Section 3.6.8 of the BDG. Use of an approach slab may be required per the BDG Sections 5.4.2.10 and 5.4.4.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the BDG.

Backfill within 10 ft of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

7.3 Driven Pipe Pile Bent Pier Foundation

The use of a driven pipe pile bent pier has been determined to be a viable foundation system. The steel pipe piles shall be end bearing on the bedrock and can be driven open-ended and cleaned out of soil or close-ended. It should be understood that a certain level of ambiguity regarding the driven factor of safety is introduced when piles driven open-ended are cleaned out after the dynamic testing is conducted. Piles driven open-ended typically develop a soil plug which should be left intact when the pile is cleaned out. Piles should be cleaned out the depth of pile fixity or to a depth equal to 20 times the pile diameter (PDCA Section 9.10.5, 1998) which ever correlates to a higher elevation. If the piles are driven open-ended, the clean out elevation will be determined during final design once the pile size has been finalized. The decision to use open-ended or closed-ended piles should be determined cooperatively by the Geotechnical Engineer and Structural Designer.

The use of Grade 3 steel (45 ksi yield strength) steel is recommended. The use of a reinforcing cage providing 2% reinforcement is also recommended. The reinforced pipe pile shall be filled with Class A concrete. Fusion bonded epoxy protective coating should be applied a minimum of 10 ft below the streambed or 2 ft below the total scour depth.

Seamless pipe piles do not need to be specified. Butt/seam weld spiral or longitudinal seam piles are acceptable. Spiral lap or longitudinal lap welded piles are not acceptable. Piles driven open-ended shall use a stiffening ring/cutting shoe. It is advised that the contract documents specify this cutting shoe to insure proper seating of the pipe pile on bedrock. Piles driven close-ended shall have a conical driving point. The piles may experience a "walking" problem due to the sloping of the bedrock.

The following steel pipe piles have been evaluated for use:

- 24 in diameter with 1/2 in wall thickness
- 24 in diameter with 5/8 in wall thickness
- 26 in diameter with 1/2 in wall thickness
- 26 in diameter with 5/8 in wall thickness
- 28 in diameter with 11/16 in wall thickness
- 30 in diameter with 1/2 in wall thickness
- 30 in diameter with 5/8 in wall thickness

Center-to-center pile spacing shall not be less than the greater of 30 in or of 2.5 pile diameters or widths (AASHTO LRFD 10.7.1.5). No borings were taken at the pier locations. For estimation purposes, a pile tip elevation of approximately 205 ft should be used as a minimum when ordering pile lengths. This pile tip elevation is based on a simplified subsurface profile *(Sheet 3 - Interpretive Subsurface Profile)* drawn of the site. The actual depth to bedrock may vary from that estimated here.

For pipe pile supported piers the MaineDOT Bridge Design Manual (BDG) recommends a Factor of Safety of 4.0 or 0.25Fy for the maximum design load. The geotechnical and structural capacities of several pipe piles are summarized in the following table. Calculations can be found in Appendix C at the end of this report. Using the assumption that Grade 3 steel (45 ksi) will be used, the structural capacity of the piles is less than the geotechnical capacity and therefore **the structural capacity governs**. Design axial loads should be shown on the plans. Preliminary axial pile capacities based on piles driven to and bearing on bedrock are summarized in the following table:

Pipe Pile Size	Allowable Geotechnical Pile Capacity (FS=2.25)	Allowable Structural Pile Capacity (Based on 0.25 Fy)
24 in dia, 1/2 in wall	573 kips	**310 kips**
24 in dia, 5/8 in wall	711 kips	**411 kips**
26 in dia, 1/2 in wall	619 kips	**336 kips**
26 in dia, 5/8 in wall	771 kips	**446 kips**
28 in dia, 11/16 in wall	916 kips	**540 kips**
30 in dia, 1/2 in wall	719 kips	**389 kips**
30 in dia, 5/8 in wall	896 kips	**517 kips**

The pipe piles should be embedded enough in the pile cap to provide a fixed condition at the top. It is also recommended that the pier bearing be fixed to provide additional lateral stability.

The pier should be designed to account for any unbalanced forces due to the opening of the lift span. Transverse loads should be reacted primarily by using battered upstream and downstream piles. It is recommended that the Structural Designer submit longitudinal, transverse and axial loads for each ASSHTO load group for the pier during final design for geotechnical evaluation. This will allow for the evaluation of the Factors of Safety, deformations and induced stresses for the piles.

In lateral capacity analysis, fixity may be assumed at the pile tip/rock interface. Piles anticipated to be under uplift conditions, shall be analyzed for resistance to uplift. The design uplift capacity of a single pile is specified as 1/3 the ultimate shaft resistance calculated in a static analysis method.

Contract documents shall require that the contractor perform a wave equation analysis and two dynamic analyses at the pier (one test for plumb piles and one test for battered piles).

With this level of quality control, the piles shall be driven to an ultimate capacity of 2.25 times the design load. The wave equation analysis requirements shall be the same as those described in Section 7.1.

Due to scour conditions at the site, it is anticipated that the pier piles will need to be designed taking a considerable unsupported length into consideration. This length will be determined as a part of the required scour analysis. Refer to BDG Sections 2.3.11 Scour and 5.5.1.4 Geotechnical Design for information regarding scour depth. It is recommended that the design of the pile bent pier be a collaborative effort between the Structural Design Engineer and the Geotechnical Engineer during the final design phase.

7.4 Mass Concrete Pier on Driven H-Piles

The use of a mass concrete pier founded on driven H-piles is viable alternative for use at the pier location. Due to the presence of scour susceptible soils in the river bed, the H-piles will need to be designed considering an unsupported length equal to the predicted scour depth. The following piles were considered for use at the pier: HP 14 x 73 or HP 14 x 89. It is assumed that 50 ksi steel will be used.

Center-to-center pile spacing shall not be less than the greater of 30 in or of 2.5 pile diameters or widths (AASHTO LRFD 10.7.1.5). No borings were taken at the pier locations. For estimation purposes, a pile tip elevation of approximately 205 ft should be used as a minimum when ordering pile lengths. This pile tip elevation is based on a simplified subsurface profile *(Sheet 3 - Interpretive Subsurface Profile)* drawn of the site. The actual depth to bedrock may vary from that estimated here.

A factor of safety of 3.0 or 0.33Fy was use to calculate the allowable structural pile design load. The geotechnical and structural capacities of the H-piles are summarized in the following table. Calculations can be found in Appendix C at the end of this Report.

Pile Type	Allowable bearing capacity, $Q_{t, allow}$ (Goodman and Kulhaway & Goodman) FS = 2.25	Total Allowable $Q_{total, allow}$ Per Structural Capacity: 345 Mpa (50 ksi) FS = 3
HP 14 x 73	368 kips	**353 kips**
HP 14 x 89	449 kips	**431 kips**

Using the assumption that 50 ksi steel will be used; the geotechnical capacity of the piles exceeds the structural capacity and therefore **the structural capacity governs**. Design axial loads should be shown on the plans.

The piles should be embedded enough in the pier concrete to provide a fixed condition at the top. It is also recommended that the pier bearing be fixed to provide additional lateral stability.

The pier should be designed to account for any unbalanced forces due to the opening of the lift span. Transverse loads should be reacted primarily by using battered upstream and downstream and exterior piles. It is recommended that the Structural Designer submit longitudinal, transverse, lateral and axial loads for each ASSHTO load group for the pier during final design for geotechnical evaluation. This will allow for the evaluation of the Factors of Safety, deformations and induced stresses for the piles.

In lateral capacity analysis, fixity may be assumed at the pile tip/rock interface. Piles anticipated to be under uplift conditions, shall be analyzed for resistance to uplift. The design uplift capacity of a single pile is specified as 1/3 the ultimate shaft resistance calculated in a static analysis method.

The soils encountered at the site will provide sufficient lateral support to assume the H-piles are fully braced against Euler buckling. The Designer should check that pile axial stresses from the dead loads, live loads, pile dead load and stream and ice loads do not exceed the allowable axial pile loads shown in the table. Refer to BDG Section 5.5.1.4 Geotechnical Design for information regarding load cases for overturning for mass pier design.

Contract documents shall require that the contractor perform a wave equation analysis and two dynamic analyses at the pier (one test for plumb piles and one test for battered piles). With this level of quality control, the piles shall be driven to an ultimate capacity of 2.25 times the design load. The wave equation analysis requirements shall be the same as those described in Section 7.1.

Due to scour conditions at the site, it is anticipated that the pier piles will need to be designed taking a considerable unsupported length into consideration. This length will be determined as a part of the required scour analysis. Refer to BDG Sections 2.3.11 Scour and 5.5.1.4 Geotechnical Design for information regarding scour depth. It is recommended that the design of the mass pier pile group be a collaborative effort between the Structural Design Engineer and the Geotechnical Engineer during the final design phase.

7.5 Fender System

The use of a fender system to protect the substructures in the channel is planned. The fender system will consist of concrete filled pipe piles. The fender piles will be friction piles and should be designed with consideration of the unsupported length due to scour. The use 70 ksi yield strength steel is proposed. The pipe piles shall be filled with Class A concrete. The proposed fender system consists of two rows of 28 piles spaced at 6 ft on center using 12 in diameter pipe piles with a wall thickness of ½ inch and having an overall length of 45 ft.

An analysis of the fender system was conducted using LPile software by ENSOFT. This analysis used internally generated p-y curves and field data regarding the fender system to determine the amount of lateral movement the fender system would see on impact by a vessel. The vessel impact load was determined considering the Songo Queen river boat which uses the channel regularly. Using a total pile length of 45 ft (including 15 ft above the streambed), an impact force of 38,000 lbs, and a fixed condition at the pile head total movement of the piles was determined to be approximately 8.6 inches. The resulting bending moment was determined to be 523 ft-kips. An investigation into the allowable bending moment capacity and design criteria for horizontal movement of piles in a fender system should be conducted by the Structural Designer during final design.

Seamless pipe piles do not need to be specified. Butt/seam weld spiral or longitudinal seam piles are acceptable. Spiral lap or longitudinal lap welded piles are not acceptable. Piles driven closed-ended shall use a conical point. It is advised that the contract documents specify this conical point.

7.6 Bearing Capacity

The applied bearing pressure for any structure founded on the native sand layer should not exceed the calculated allowable bearing capacity of 3 tsf. See Appendix C- Calculations for supporting documentation. No footing shall be less than 2 ft wide regardless of the applied bearing pressure. Any organic material encountered shall be removed to the full depth and replaced with compacted granular fill.

7.7 Settlement

Any settlement of the bridge foundations will be due to the elastic compression of the piling. Due to the presence of granular soils underlying the approaches, settlements in this area are anticipated to be less than 1 inch and will occur during construction having minimal effect on the finished structure.

7.8 Retaining Walls

Project retaining walls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. All retaining walls should be designed to achieve a factor of safety of 2.0 against overturning and a factor of safety of 1.5 against sliding. An active earth pressure coefficient, K_a, shall be calculated using Rankine Theory for cantilever wingwalls and Coulomb Theory for gravity shaped structures. See *Sheet 5 - Rankine and Coulomb Active Earth Pressure Coefficients* at the end of this report for guidance in calculating these values. A live load surcharge should be applied when traffic loads are located with a horizontal distance equal to one-half of the wall height behind the back of the wall per Section 3.6.8 of the BDG.

The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface.

All retaining wall designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind structure shall be in accordance with Section 5.4.1.4 of the BDG.

7.9 Frost Protection

According to the MaineDOT design freezing index maps for the State of Maine, the site has a design-freezing index of approximately 1370 F-degree days. Grain size analyses conducted on soil samples in the upper layer of soils indicated that the soils are granular and have a water content of approximately 20%. These components correlate to a frost depth of 5.5 ft. Therefore, any foundations placed on native subgrade soil should be founded a minimum of 5.5 ft below finished exterior grade for frost protection.

7.10 Scour

A Grain Size Analysis was performed on a sample taken from boring BB-NBB-102 (Sample 3D, Reference No. 176582) in order to generate a grain size curve for determining parameters to be used in a scour analysis. This sample was assumed to be similar in nature to the soils likely to be exposed to scour conditions. The gradation curve is shown in Appendix B - Laboratory Testing. The following riverbed grain size parameters are to be used in scour analysis: AASHTO Soil Type A-2-4, $D_{50} = 0.32$ mm.

Where the calculated scour depth is significant, the foundation elements shall be designed for an unsupported length. The unsupported length should be the vertical distance from the bottom of the foundation element to the calculated scour depth. In designing deep foundation elements with unsupported length, an analysis of the foundation should be performed using actual loading and soil conditions.

7.11 Seismic Design Considerations

Per BDG 3.7.1.1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09 are designated a Seismic Performance Category (SPC) classification of A, and should be designed in conformance with Section 5 of Division 1-A of the AASHTO Standard Specifications for Highway Bridges. For SPC A, no detailed analysis is required other than connection design and bearing seat length, except if the bridge is functionally important or classified as a major structure.

The horizontal bedrock acceleration coefficient (A) for Naples is approximately 0.045g, based on Figure 3-4 of the BDG, Seismic Performance Categories for Maine, August 2003. Per Section 3.5 of Division 1-A of the AASHTO Standard Specifications for Highway Bridges Soil Profile Type I is applicable to the site and a site coefficient (S) of 1.0 would be used.

According to Figure 2-2 of the BDG, the Naples Bay Bridge on US Route 302 is on the National Highway System (NHS) and is therefore considered to be functionally important.

Per BDG Section 3.7.2 functionally important bridges with 2 or more spans shall be designed according to the requirements for SPC B with an acceleration coefficient of 0.09. As a result, the bridge substructures should be designed for seismic earth loads assuming A=0.09. The soils at the site are not considered to be liquefaction-susceptible (see Appendix C - Calculations).

7.12 Construction Considerations

The existing timber piles shall be cut off 1 ft below the bottom of the proposed abutment and the existing timber piles in the riverbed shall be cut off 1 ft below the riverbed surface. This work will be paid for under Item 202.19 - Removing Existing Bridge and should be noted in the General Construction Notes.

There is a potential for the remaining portion of the existing timber piles at the abutments to interfere with the installation of the proposed abutment and fender piles. If the existing timber piles are encountered during pile installation the timber piles shall be removed by the Contractor to the Resident's satisfaction. This condition should be noted on the plans and the work will be considered incidental to pile installation.

There is a potential for the piles to encountered cobbles and boulders causing them to deflect during driving. Pre-drilling of the pile locations may be necessary in order to advance the piles through the cobbles and boulders. Piles refusing on cobbles and/or boulders will be pulled, obstructions removed and pile re-driven to bedrock. This condition should be noted on the plans and the work will be considered incidental to pile installation.

8.0 ADDITIONAL WORK

In the Spring of 2006, an additional boring will be conducted at pier location to obtain information for final design. The information obtained during that investigation will be submitted to the Structural Designer in a memorandum as soon shortly after the work is completed.

It is the recommendation of the geotechnical team member that borings be conducted at the location of the temporary bridge in order to provide the Contractor with information to design the temporary bridge substructures. These borings can be conducted during the final design phase.

It is recommended that the Structural Designer submit the abutment loading and design vertical movement criteria to the Geotechnical Designer during final design for geotechnical evaluation. It is recommended that the design of the mass pier pile group or pile bent be a collaborative effort between the Structural Design Engineer and the Geotechnical Engineer during the final design phase.

9.0 CLOSURE

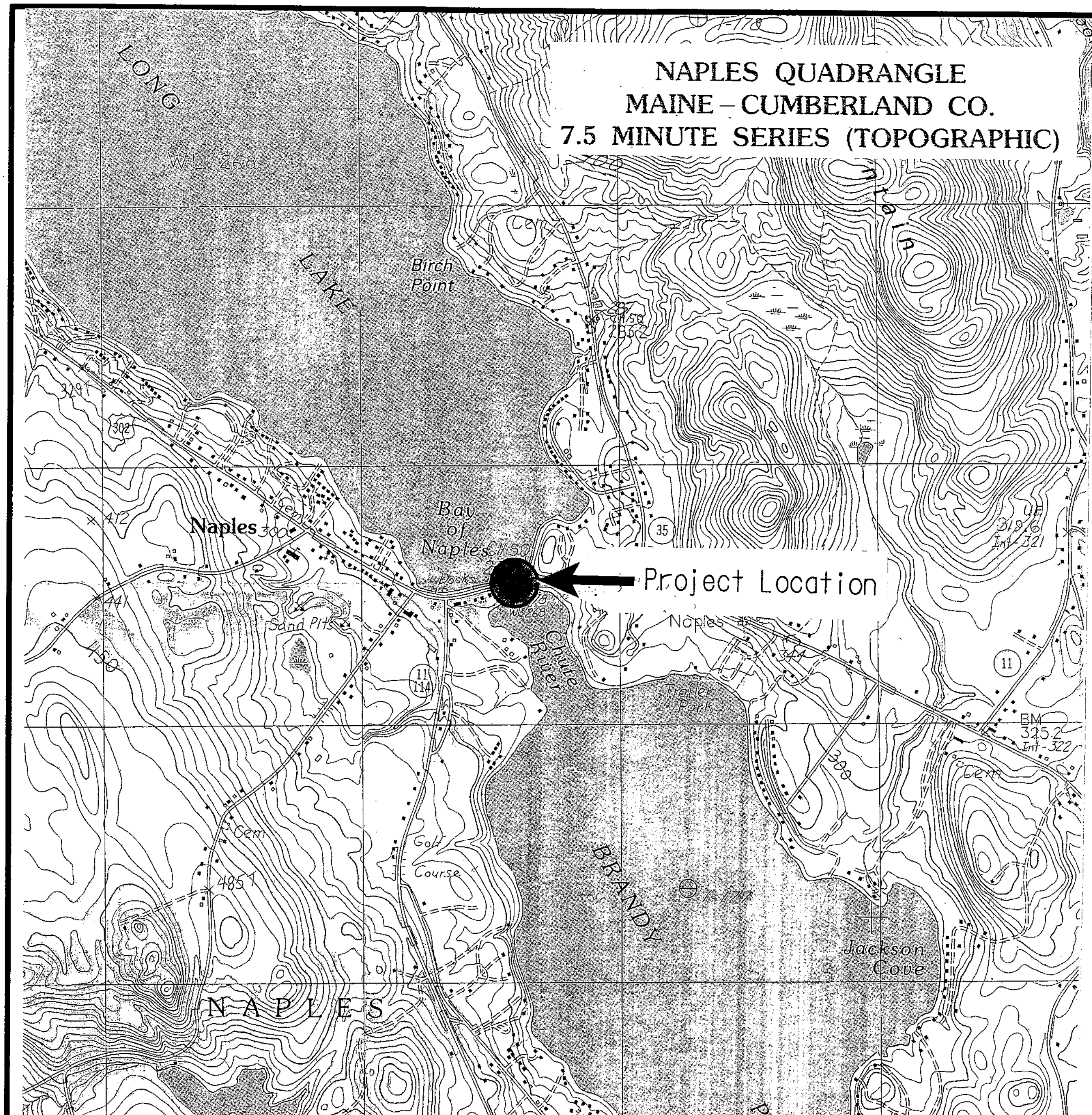
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Naples Bay Bridge in Naples, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Location Map

Sheet 1



Naples, Naples Bay Bridge over Chutes River, PIN. 11060.00

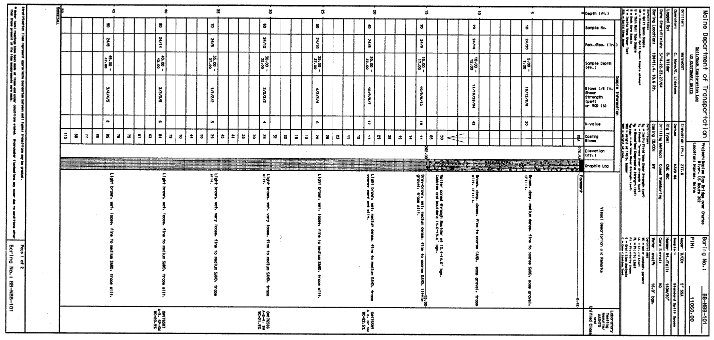
1 .5 0 1 MILE

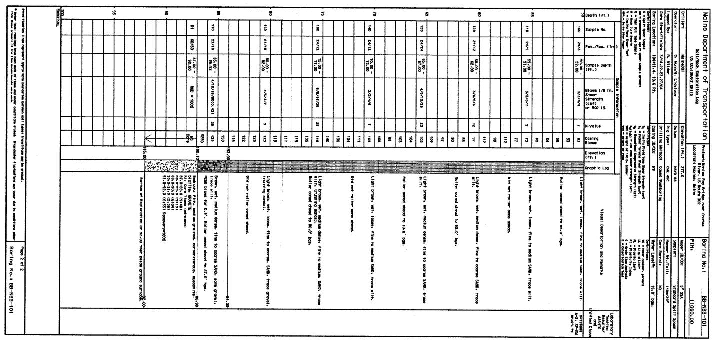
1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

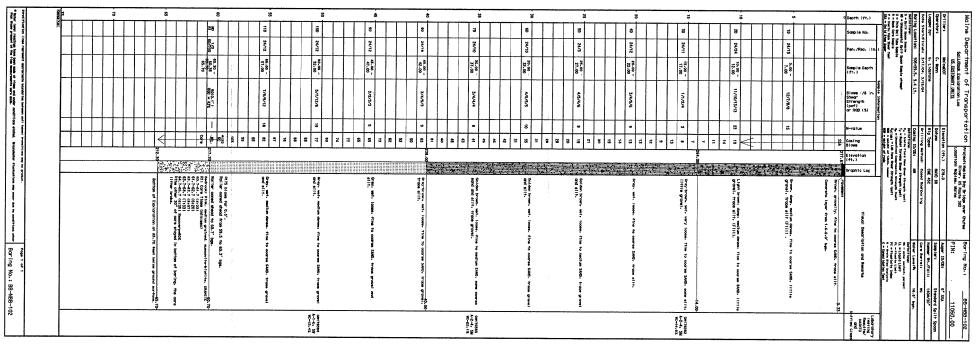
1 .5 0 1 KILOMETER

1:24000, 1"= 2000', 1 cm = 240 m

N







NAPLES BAY BRIDGE
CHUTES RIVER
NAPLES CUMBERLAND COUNTY

INTERPRETIVE SUBSURFACE PROFILE

PROJ. MANAGER	J. Wentworth	BY	DATE
DESIGN-DETAILED	K. Maguire	T. White	NOV 2005
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

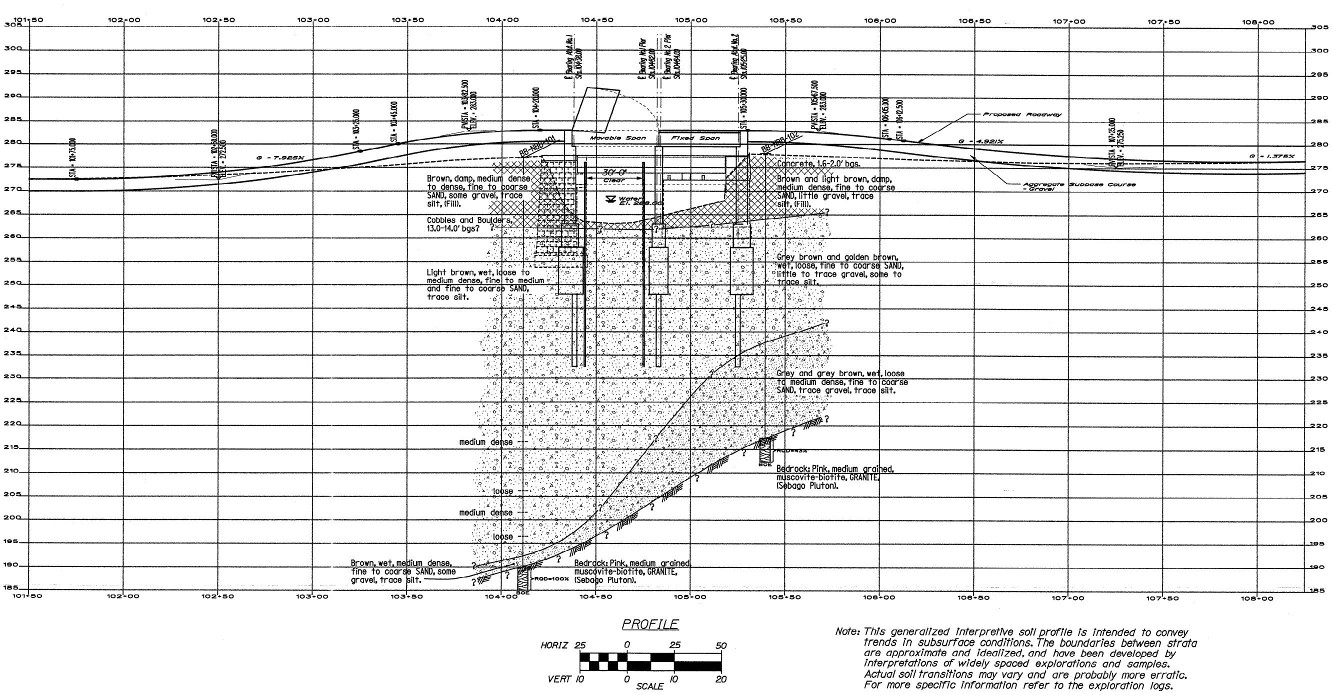
BH-A106(000)X

BRIDGE NO. 2047

PIN
11060.00

BRIDGE PLANS

SHEET NUMBER
4
OF 5



Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-A106(000)X

PIN 11066.00

NAPLES BAY BRIDGE
CHUTES RIVER
CUMBERLAND COUNTY

NAPLES

INTERPRETIVE SUBSURFACE PROFILE

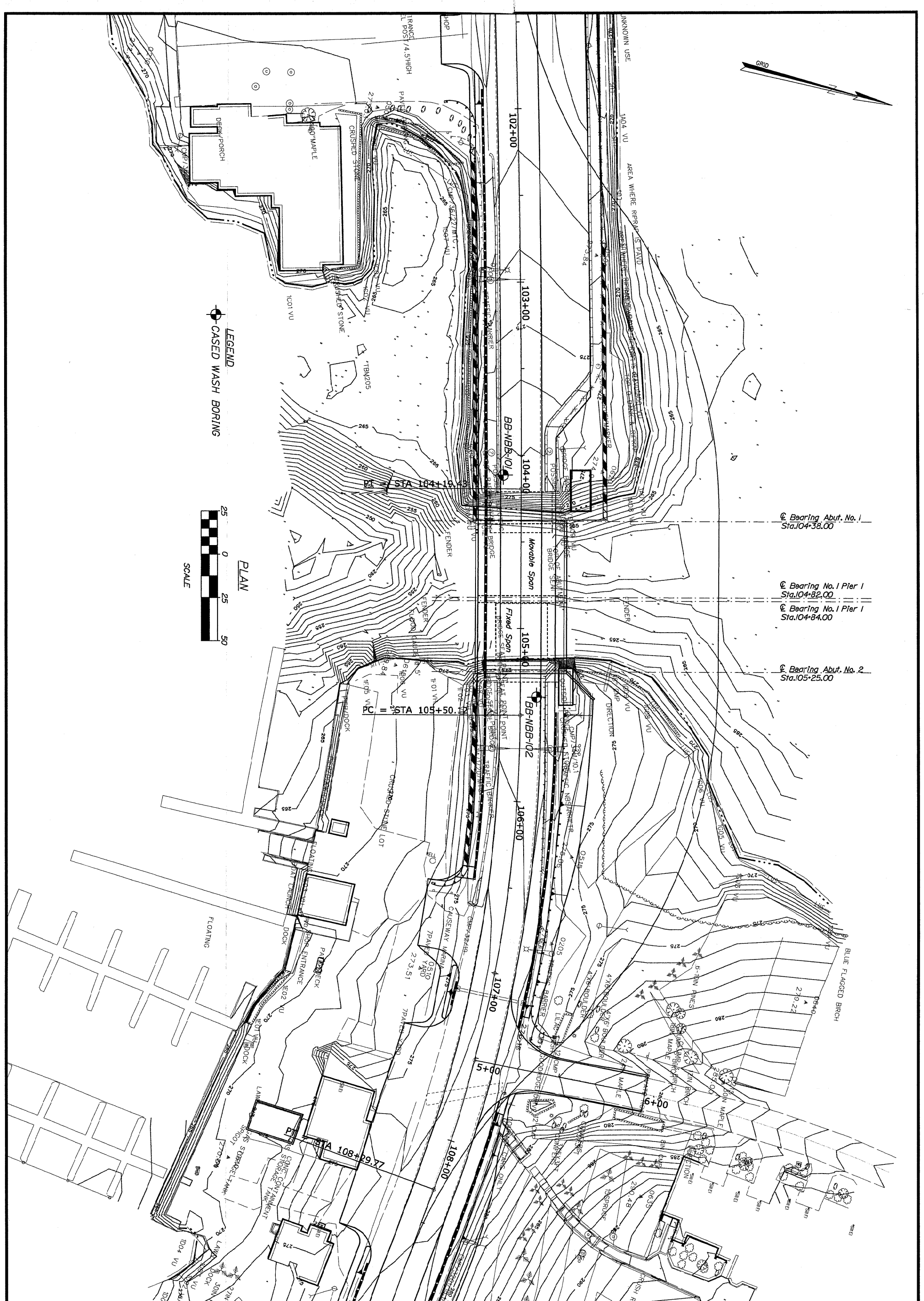
		BY	DATE
PROJ. MANAGER	J. Wentworth		
DESIGN-DETAILED	K. Maguire	T. White	NOV 2000

SIGNATURE

P.E. NUMBER

DATE

SHEET NUMBER 3 OF 5



NAPLES BAY BRIDGE
CHUTES RIVER
NAPLES CUMBERLAND COUNTY

BORING LOCATION PLAN

PROJ. MANAGER	J. Wentworth	BY	DATE
DESIGN-DETAILED	K. Maguire	T. White	NOV 2005
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

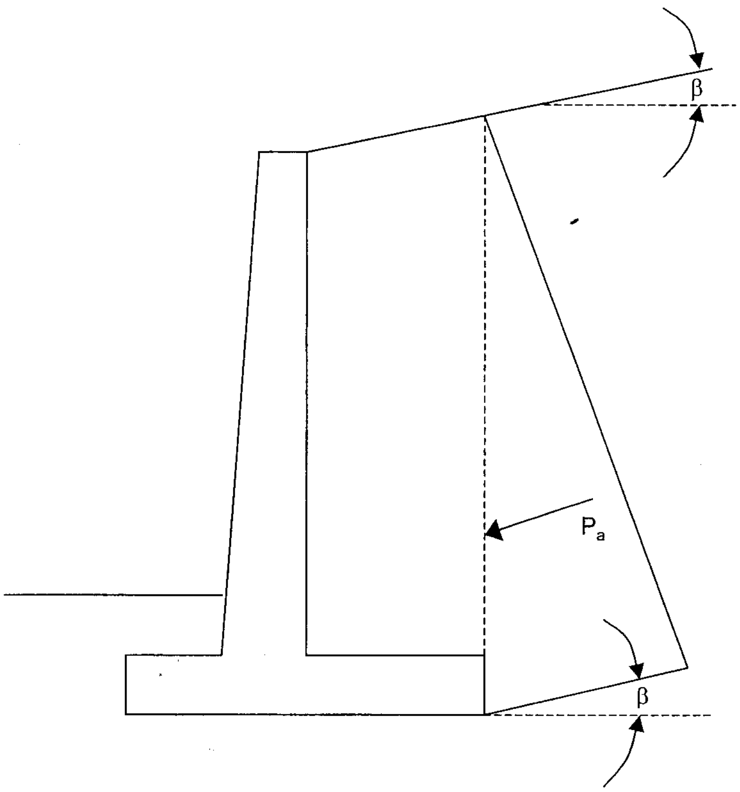
DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-A106(000)X

BRIDGE NO. 2047 PIN 11060.00 BRIDGE PLANS

SHEET NUMBER 2 OF 5



For cases where interface friction between the backfill and wall are 0 or not considered, use Rankine.

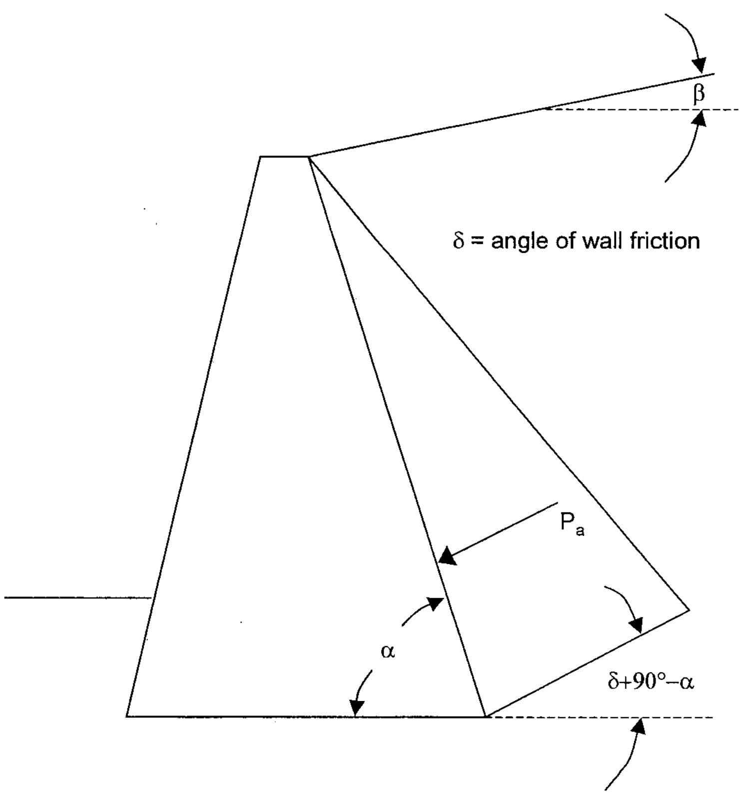
For a horizontal backfill surface, $\beta = 0°$:

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

For a sloped backfill surface, $\beta > 0°$:

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

P_a is oriented at β



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

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

P_a is oriented at $\delta + 90° - \alpha$

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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

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

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	0 - 2	0 - 250	Fist easily Penetrates
Soft	3 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

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

Rock Quality Description	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and Rock Mass Description (very poor, poor, fair, etc.)
Recovery

Sample Container Labeling Requirements:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Naples Bay Bridge over Chutes River, US Route 302 **Location:** Naples, Maine	**Boring No.:** BB-NBB-101 **PIN:** 11060.00

Driller:	MaineDOT	**Elevation (ft.)**	277.0	**Auger ID/OD:**	5" SSA
Operator:	C. Mann/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/14,22,23,27/04	**Drilling Method:**	Cased Washboring	**Core Barrel:**	NQ
Boring Location:	104+11.4, 10.6 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	16.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	276.58		Pavement ——0.4	
5	1D	24/20	5.0 - 7.0	15/12/8/8	20				Brown, damp, medium dense, fine to coarse SAND, some gravel, (Fill).	
10	2D	24/16	10.0 - 12.0	11/15/28/34	43				Brown, damp, dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
						50			Roller coned through Boulder at 13.4-14.0' bgs. Cobbles and boulders 14.0-15.0' bgs.	
						85	262.00		——15.0	
15	3D	24/9	15.0 - 17.0	10/9/9/12	18	14			Grey-brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	
						14				
						19				
						28				
						29				
20	4D	24/8	20.0 - 22.0	10/9/8/7	17	13			Light brown, wet, medium dense, fine to medium SAND, trace coarse sand and silt.	G#176585 A-3, SP-SM WC=22.5%
						12				
						11				
						23				
						28				
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: Naples Bay Bridge over Chutes River, US Route 302 Location: Naples, Maine	Boring No.: BB-NBB-101 PIN: 11060.00

Driller:	MaineDOT	Elevation (ft.)	277.0	Auger ID/OD:	5" SSA
Operator:	C. Mann/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/14,22,23,27/04	Drilling Method:	Cased Washboring	Core Barrel:	NQ
Boring Location:	104+11.4, 10.6 Rt.	Casing ID/OD:	NW	Water Level*:	16.0' bgs.

Definitions:
D = Split Spoon Sample
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U = Thin Wall Tube Sample
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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WC = water content, percent
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PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/10	25.0 - 27.0	4/3/3/4	6	20			Light brown, wet, loose, fine to medium SAND, trace silt.	
						15				
						18				
						22				
						31				
30	6D	24/12	30.0 - 32.0	2/2/2/2	4	34			Light brown, wet, very loose, fine to medium SAND, trace silt.	G#176586 A-2-4, SM WC=21.9%
						30				
						33				
						41				
						45				
35	7D	24/5	35.0 - 37.0	1/1/2/2	3	39			Light brown, wet, very loose, fine to medium SAND, trace silt.	
						36				
						44				
						70				
						79				
40	8D	24/14	40.0 - 42.0	3/3/3/5	6	84			Light brown, wet, loose, fine to medium SAND, trace silt.	G#176587 A-3, SP-SM WC=20.9%
						63				
						67				
						75				
						78				
45	9D	24/8	45.0 - 47.0	3/4/4/5	8	95			Light brown, wet, loose, fine to medium SAND, trace silt.	
						65				
						77				
						88				
						115				
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: Naples Bay Bridge over Chutes River, US Route 302 Location: Naples, Maine	Boring No.: BB-NBB-101 PIN: 11060.00

Driller:	MaineDOT	Elevation (ft.)	277.0	Auger ID/OD:	5" SSA
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Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/14,22,23,27/04	Drilling Method:	Cased Washboring	Core Barrel:	NQ
Boring Location:	104+11.4, 10.6 Rt.	Casing ID/OD:	NW	Water Level*:	16.0' bgs.

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SSA = Solid Stem Auger

Definitions:
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T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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Definitions:
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/3	50.0 - 52.0	3/3/4/4	7	83			Light brown, wet, loose, fine to medium SAND, trace silt.	G#176588 A-3, SP-SM WC=23.7%
						53				
						56			Roller coned ahead to 55.0' bgs.	
						64				
						82				
55	11D	24/1	55.0 - 57.0	3/3/6/7	9	73			Light brown, wet, loose, fine to medium SAND, trace silt.	
						77				
						112			Did not roller cone ahead.	
						90				
						112				
60	12D	24/12	60.0 - 62.0	3/6/6/6	12	97			Light brown, wet, medium dense, fine to coarse SAND, trace silt.	
						88				
						90			Roller coned ahead to 65.0' bgs.	
						97				
						109				
65	13D	24/24	65.0 - 67.0	4/8/15/29	23	103			Light brown, wet, medium dense, fine to coarse SAND, trace silt.	
						104				
						103			Roller coned ahead to 70.0' bgs.	
						88				
						108				
70	14D	24/12	70.0 - 72.0	3/3/4/6	7	109			Light brown, wet, loose, fine to medium SAND, trace silt.	
						111				
						134			Did not roller cone ahead.	
						136				
75						140				

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: Naples Bay Bridge over Chutes River, US Route 302 Location: Naples, Maine	Boring No.: BB-NBB-101 PIN: 11060.00

Driller:	MaineDOT	Elevation (ft.)	277.0	Auger ID/OD:	5" SSA
Operator:	C. Mann/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/14,22,23,27/04	Drilling Method:	Cased Washboring	Core Barrel:	NQ
Boring Location:	104+11.4, 10.6 Rt.	Casing ID/OD:	NW	Water Level*:	16.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	15D	24/12	75.0 - 77.0	8/9/16/28	25	118			Light brown, wet, medium dense, fine to medium, SAND, trace silt, (running sands). Roller coned ahead to 80.0' bgs.	
						135				
						119				
						117				
						118				
80	16D	24/18	80.0 - 82.0	4/5/4/7	9	125			Light brown, wet, loose, fine to coarse, SAND, trace gravel, (running sands).	
						125				
						122			Did not roller coned ahead.	
						118				
						150	193.00		84.0	
85	17D	23/10	85.0 - 86.9	4/10/19/60(0.42)	29	138			Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	
						a250			a250 blows for 0.9'. Roller coned ahead to 87.0' bgs.	
	R1	60/60	87.0 - 92.0	RQD = 100%		NQ Core	190.10		86.9 Bedrock: Pink, medium grained, carboniferous, muscovite-biotite, GRANITE R1: Core Times (min:sec) 87.0-88.0 (9:40) 88.0-89.0 (9:13) 89.0-90.0 (6:04) 90.0-91.0 (5:56) 91.0-92.0 (6:55) Recovery=100%	
90										
							185.00		92.0 **Bottom of Exploration at 92.00 feet below ground surface.**	
95										
100										

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Naples Bay Bridge over Chutes River, US Route 302 Location: Naples, Maine	Boring No.: BB-NBB-102 PIN: 11060.00

Driller:	MaineDOT	**Elevation (ft.)**	278.0	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/04, 3/15/04	**Drilling Method:**	Cased Washboring	**Core Barrel:**	NQ
Boring Location:	105+39.5, 9.4 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	16.5' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	277.67		Pavement —0.3 Brown, gravelly, fine to coarse SAND, trace silt. Concrete layer from 1.6-2.0' bgs.	
5	1D	24/15	5.0 - 7.0	12/7/8/8	15				Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt (Fill) .	
10	2D	24/24	10.0 - 12.0	11/10/13/13	23	19			Light brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
						14				
						11				
						7	264.00		—14.0	
						7				
15	3D	24/11	15.0 - 17.0	1/1/2/4	3	8			Grey-brown, wet, very loose, fine to coarse SAND, some silt, little gravel.	G#176582 A-2-4, SM WC=14.6%
						13				
						9				
						12				
						13				
20	4D	24/12	20.0 - 22.0	3/4/5/6	9	13			Grey, wet, loose, fine to coarse SAND, trace silt.	
						14				
						19				
						20				
						21				
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:** Naples Bay Bridge over Chutes River, US Route 302 **Location:** Naples, Maine	**Boring No.:** BB-NBB-102 **PIN:** 11060.00

Driller:	MaineDOT	**Elevation (ft.)**	278.0	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/04, 3/15/04	**Drilling Method:**	Cased Washboring	**Core Barrel:**	NQ
Boring Location:	105+39.5, 9.4 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	16.5' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/3	25.0 - 27.0	4/5/4/6	9	19			Golden brown, wet, loose, fine to medium SAND, trace gravel and silt.	
						20				
						28				
						29				
						31				
30	6D	24/10	30.0 - 32.0	4/5/5/5	10	35			Golden brown, wet, loose, fine to coarse SAND, trace gravel and silt.	
						31				
						42				
						46				
						46				
35	7D	24/10	35.0 - 37.0	3/4/4/4	8	41			Golden brown, wet, loose, fine to medium SAND, some coarse sand and silt, trace gravel.	G#176583 A-2-4, SM WC=22.1%
						43				
						48				
						60				
						61	238.00		40.0	
40	8D	24/16	40.0 - 42.0	3/4/5/3	9	63			Grey-brown, wet, loose, fine to coarse SAND, trace gravel, trace silt.	
						50				
						52				
						62				
						73				
45	9D	24/12	45.0 - 47.0	2/2/3/3	5	65			Grey, wet, loose, fine to coarse SAND, trace gravel and silt.	
						65				
						77				
						74				
50						80				

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:** Naples Bay Bridge over Chutes River, US Route 302 **Location:** Naples, Maine	**Boring No.:** BB-NBB-102 **PIN:** 11060.00

Driller:	MaineDOT	**Elevation (ft.)**	278.0	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/04, 3/15/04	**Drilling Method:**	Cased Washboring	**Core Barrel:**	NQ
Boring Location:	105+39.5, 9.4 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	16.5' bgs.

Definitions:	Definitions:	Definitions:
D = Split Spoon Sample	S_u = Insitu Field Vane Shear Strength (psf)	WC = water content, percent
MD = Unsuccessful Split Spoon Sample attempt	T_v = Pocket Torvane Shear Strength (psf)	LL = Liquid Limit
U = Thin Wall Tube Sample	q_p = Unconfined Compressive Strength (ksf)	PL = Plastic Limit
R = Rock Core Sample	$S_{u(lab)}$ = Lab Vane Shear Strength (psf)	PI = Plasticity Index
V = Insitu Vane Shear Test	WOH = weight of 140lb. hammer	G = Grain Size Analysis
SSA = Solid Stem Auger	WOR = weight of rods	C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample Information									
50	10D	24/12	50.0 - 52.0	5/7/12/6	19	77			Grey, wet, medium dense, fine to coarse SAND, trace gravel and silt.	G#176584 A-1-b, SW WC=15.4%
						69				
						88				
						76				
						87				
55	11D	24/12	55.0 - 57.0	7/9/9/12	18	82			Grey, wet, medium dense, fine to coarse SAND, trace gravel and silt.	
						65				
						59				
						103				
						a175 RC			a175 blows for 0.5'. Roller coned ahead from 59.5 to 60.3' bgs.	
60	MD	1/0	60.3 - 60.4	50(0.1')	---	NQ	217.30		Roller coned ahead to 60.7' bgs. ——— 60.7	
	R1	60/b30	60.7 - 65.7	RQD = 43%		Core			Bedrock: Pink, medium grained, muscovite-biotite, GRANITE R1: Core Times (min:sec) 60.7-61.7 (4:22) 61.7-62.7 (6:20) 62.7-63.7 (8:07) 63.7-64.7 (7:33) 64.7-65.7 (6:25) Recovery=50% bThe other 30" of core stayed in bottom of boring, the core lifter broke.	
65							212.30		——— 65.7 **Bottom of Exploration at 65.70 feet below ground surface.**	
70										
75										

Remarks:

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Naples — Project Number: 11060.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification Unified	Classification AASHTO	Classification Frost
BB-NBB-101, 4D	104+11.4	10.6 Rt.	20.0-22.0	176585	1	22.5			SP-SM	A-3	0
BB-NBB-101, 6D	104+11.4	10.6 Rt.	30.0-32.0	176586	1	21.9			SM	A-2-4	II
BB-NBB-101, 8D	104+11.4	10.6 Rt.	40.0-42.0	176587	1	20.9			SP-SM	A-3	0
BB-NBB-101, 10D	104+11.4	10.6 Rt.	50.0-52.0	176588	1	23.7			SP-SM	A-3	0
BB-NBB-102, 3D	105+39.5	9.4 Lt.	15.0-17.0	176582	2	14.6			SM	A-2-4	II
BB-NBB-102, 7D	105+39.5	9.4 Lt.	35.0-37.0	176583	2	22.1			SM	A-2-4	II
BB-NBB-102, 10D	105+39.5	9.4 Lt.	50.0-52.0	176584	2	15.4			SW	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.

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

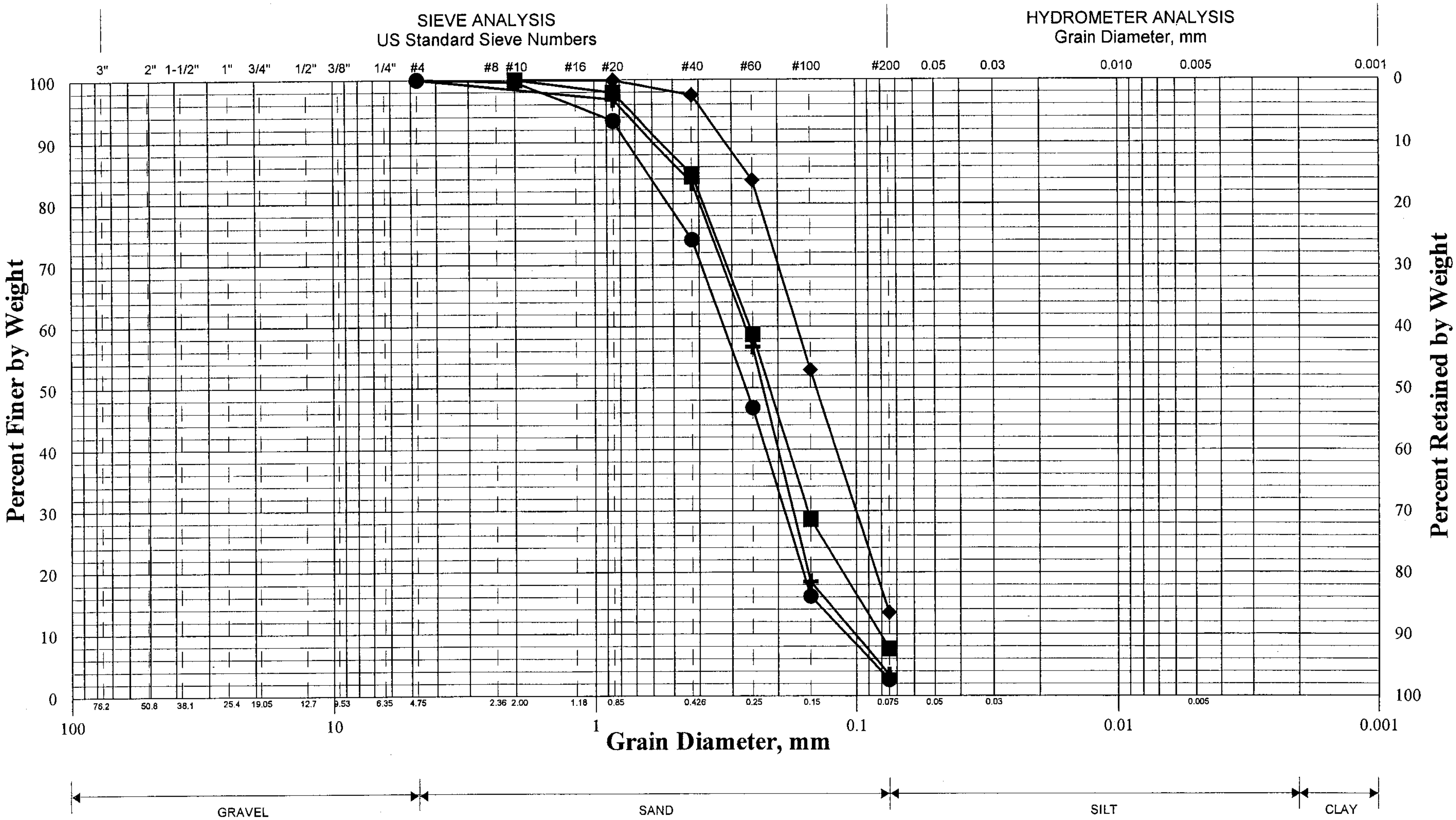
LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

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

SHEET NO. 1

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE

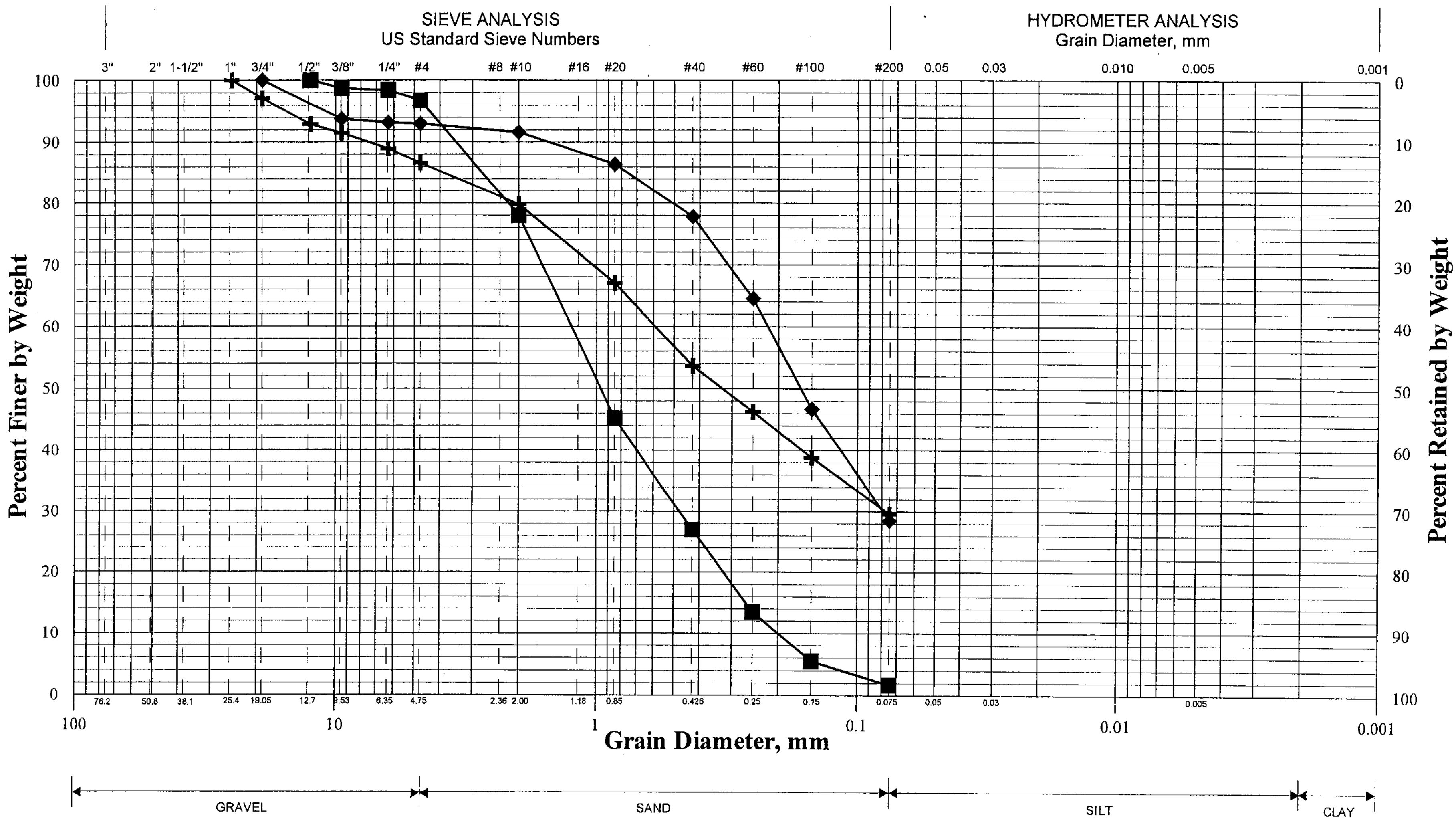


UNIFIED CLASSIFICATION

	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-NBB-101	4D	20.0-22.0	SAND, trace silt.	22.5			
◆	BB-NBB-101	6D	30.0-32.0	SAND, little silt.	21.9			
■	BB-NBB-101	8D	40.0-42.0	SAND, trace silt.	20.9			
●	BB-NBB-101	10D	50.0-52.0	SAND, trace silt.	23.7			
▲								
×								

PIN: 11060.00
Town: Naples
Reported by: T. White
Date: 3/30/04

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-NBB-102	3D	15.0-17.0	SAND, some silt, little gravel.	14.6			
◆	BB-NBB-102	7D	35.0-37.0	SAND, some silt, trace gravel.	22.1			
■	BB-NBB-102	10D	50.0-52.0	SAND, trace gravel, trace silt.	15.4			
●								
▲								
×								

PIN: 11060.00
Town: Naples
Reported by: T. White
Date: 3/30/04

Appendix C

Calculations

Definition of Units:

$$psf := \frac{lbf}{ft^2} \quad pcf := \frac{lbf}{ft^3} \quad tsf := g \cdot \left(\frac{ton}{ft^2}\right) \quad kip := 1000 \cdot lbf \quad ksf := \frac{kip}{ft^2} \quad ksi := \frac{kip}{in^2}$$

COMPUTE AXIAL STRUCTURAL CAPACITY OF H-PILES

Using 50 ksi steel and FS = 3 for piles per BDG (0.33Fy) Section 5.4.1.8

HP 12 x 53
HP 14 x 73 Note: All matrices set up in this order
HP 14 x 89

$$\sigma_a := 0.33 \cdot 50 \cdot ksi$$

$$\sigma_a = 16.5\,ksi \qquad Area_1 := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix} \cdot in^2$$

$$Q_{all} := \sigma_a \cdot Area_1 \qquad Q_{all} = \begin{pmatrix} 255.75 \\ 353.1 \\ 430.65 \end{pmatrix} kip$$

COMPUTE AXIAL GEOTECHNICAL CAPACITY OF H-PILES

H pile Capacity - end bearing on rock assume driven through granular soils and to bedrock

HP 12 x 53
HP 14 x 73 Note: All matrices set up in this order
HP 14 x 89

Method 1: Geotechnical Capacity
Based on Unconfined Compressive Strength of bedrock

From Fang Second Edition Table 3.8:
granite compressive strength =750 - 2,500 kg/cm^2
use q_{uc} = 1,600 kg/cm^2 = 23,000 psi

$$1600 \cdot \frac{kgf}{cm^2} = 22757.349\,psi$$

$$Area_1 := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix} \cdot in^2 \qquad Area_1 = \begin{pmatrix} 0.108 \\ 0.149 \\ 0.181 \end{pmatrix} ft^2$$

$$q_{uc} := 23000 \cdot psi$$

$$Q_{ult1} := q_{uc} \cdot Area_1 \qquad Q_{ult1} = \begin{pmatrix} 356.5 \\ 492.2 \\ 600.3 \end{pmatrix} kip$$

$$Q_{all\_tip} := \frac{Q_{ult1}}{2.25} \qquad Q_{all\_tip} = \begin{pmatrix} 158.444 \\ 218.756 \\ 266.8 \end{pmatrix} kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89

**Method 2: Geotechnical Capacity by Goodman's Method
Based on Unconfined Compressive Strength of Bedrock
Reference: Principles of Foundation Engineering, BM Das, Second Edition**

$\phi := 38\cdot\mathrm{deg}$ $\quad N_\phi := \tan\left(45\cdot\mathrm{deg} + \frac{\phi}{2}\right)^2$ $\quad N_\phi = 4.204$

$q_{pt\_ult} := \left(\frac{q_{uc}}{5}\right)\cdot\left(N_\phi + 1\right)$ $\quad q_{pt\_ult} = 23.937\,\mathrm{ksi}$

$q_{pt\_all} := \frac{q_{pt\_ult}}{2.25}$ $\quad q_{pt\_all} = 10.639\,\mathrm{ksi}$

$Q_{pt\_all} := q_{pt\_all}\cdot \mathrm{Area}_1$ $\quad Q_{pt\_all} = \begin{pmatrix} 164.901 \\ 227.67 \\ 277.672 \end{pmatrix}\mathrm{kip}$

**HP 12 x 53
HP 14 x 73
HP 14 x 89**

**Method 3: Geotechnical Capacity by Goodman's Method
Based on bedrock condition - in this case Granite RQD = 43 -100%
Reference: Pile Design and Construction Practice 4th Edition MJ Tomlinson**

Low friction: 20-27 for schists, shales
Medium Friction 27-34 for sandstone, siltstone, gneiss, slate
High Friction: 34-40 for granite

$\phi_2 := 38\cdot\mathrm{deg}$ $\quad N_\phi := \tan\left(45\cdot\mathrm{deg} + \frac{\phi_2}{2}\right)^2$ $\quad N_\phi = 4.204$

$q_b := \left(2\cdot N_\phi\right)\cdot\frac{q_{uc}}{5}$ $\quad q_b = 38.674\,\mathrm{ksi}$

$Q_{ult2} := q_b\cdot \mathrm{Area}_1$ $\quad Q_{ult2} = \begin{pmatrix} 599.454 \\ 827.633 \\ 1.009\times 10^3 \end{pmatrix}\mathrm{kip}$

$Q_{all\_tip2} := \frac{Q_{ult2}}{2.25}$ $\quad Q_{all\_tip2} = \begin{pmatrix} 266.424 \\ 367.837 \\ 448.624 \end{pmatrix}\mathrm{kip}$

**HP 12 x 53
HP 14 x 73
HP 14 x 89**

Method 4: Geotechnical Capacity
Allowable End Bearing Capacity $Q_{t,\ allow}$:
(Kulhway & Goodman, FS = 2.25)

Reference: Pile Design and Construction Practice, M.J. Tomlinson, Fourth Edition

Method ignores side resistance - use Driven to assess side friction

Corrections for wedge failure under a strip footing -
multiply the cN_c factor by 1.25 for a square pile
multiply γBN_γ factor by 0.8 for a square pile

Case I
For RQD of 0 - 70%:
$q_c = 0.33 \times Q_{uc}$
$c = 0.1 \times Q_{uc}$
$\phi = 30$ degrees

Case II
For RQD of 70 - 100%:
$q_c = 0.33$ to $0.88 \times Q_{uc}$
$c = 0.1 \times Q_{uc}$
$\phi = 30$ to 60 degrees

Assume Case I: as RQD = 43 to 100%

$q_{uc} = 2.3 \times 10^4 \text{psi}$ $\quad c := 0.1 \cdot q_{uc}$ $\quad c = 2.3 \times 10^3 \text{psi}$

$\gamma := 150 \cdot \text{pcf}$ $\quad B := \begin{pmatrix} 12.05 \\ 14.59 \\ 14.70 \end{pmatrix} \text{in}$ $\quad B = \begin{pmatrix} 1.004 \\ 1.216 \\ 1.225 \end{pmatrix} \text{ft}$

based on Pells & Turner from Tomlinson page 140 - $\phi = 38$ $\quad N_c := 21.34$ $\quad N_q := 17.67$ $\quad N_\gamma := 34.18$

$D := 2 \cdot \text{in}$ Depth of penetration into bedrock

$q_c := 0.33 \cdot q_{uc}$ $\quad q_c = 7.59 \times 10^3 \text{psi}$

$$q_{ub} := 1.25 \cdot c \cdot N_c + 0.8 \cdot 0.5 \cdot \gamma \cdot B \cdot N_\gamma + \gamma \cdot D \cdot N_q$$

$q_{ub} = \begin{pmatrix} 61.37 \\ 61.373 \\ 61.373 \end{pmatrix} \text{ksi}$ $\quad \text{Area}_1 := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \end{pmatrix} \cdot \text{in}^2$

$Q_{ult3} := \overrightarrow{\left(q_{ub} \cdot \text{Area}_1\right)}$ $\quad Q_{ult3} = \begin{pmatrix} 951.233 \\ 1.313 \times 10^3 \\ 1.602 \times 10^3 \end{pmatrix} \text{kip}$

$Q_{all\_tip4} := \dfrac{Q_{ult3}}{2.25}$ $\quad Q_{all\_tip4} = \begin{pmatrix} 422.77 \\ 583.724 \\ 711.927 \end{pmatrix} \text{kip}$

HP 12 x 53
HP 14 x 73
HP 14 x 89

Skip running Driven to assess side friction - Q_{all} already exceeds the structural capacity of the pile.

COMPUTE UPLIFT CAPACITY OF H-PILES

The design uplift capacity of a single pile is specified as 1/3 the ultimate shaft resistance calculated in a static analysis method. Use the Driven to obtain the shaft resistance for each pile type considered.

Abutment No. 1 will be subjected to uplift loading.

HP 12 x 53
HP 14 x 73
HP 14 x 89

These piles are proposed at the abutment

Driven summaries are on Sheets 5-7.

Uplift Capacity = Ultimate shaft Resistance/3

Summary table:

Pile Size	Shaft Resistance (Ultimate) (kips)	Uplift Capacity (1/3 shaft) (kips)
12 x 53	118	40
14 x 73	159	53
14 x 89	174	58

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename:
Project Name: Naples Bay Bridge
Project Date: 02/02/2006
Project Client: Naples
Computed By: km
Project Manager: jw

PILE INFORMATION

Pile Type: H Pile - HP12X53
Top of Pile: 0.00 ft
Perimeter Analysis: Pile
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	10.00 ft
	- Long Term Scour:	10.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	52.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesionless	3.00 ft	10.00%	125.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
18.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
19.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
20.00 ft	0.00 Kips	14.34 Kips	14.34 Kips
27.01 ft	11.62 Kips	24.40 Kips	36.02 Kips
36.01 ft	35.37 Kips	32.52 Kips	67.89 Kips
45.01 ft	69.07 Kips	32.52 Kips	101.59 Kips
51.99 ft	102.05 Kips	32.52 Kips	134.56 Kips
52.01 ft	102.15 Kips	32.52 Kips	134.67 Kips
54.99 ft	118.10 Kips	32.52 Kips	150.62 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename:
Project Name: Naples Bay Bridge
Project Date: 02/02/2006
Project Client: Naples
Computed By: km
Project Manager: jw

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 0.00 ft
Perimeter Analysis: Pile
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	10.00 ft
	- Long Term Scour:	10.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	52.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesionless	3.00 ft	10.00%	125.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
18.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
19.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
20.00 ft	0.00 Kips	20.07 Kips	20.07 Kips
27.01 ft	15.66 Kips	34.14 Kips	49.80 Kips
36.01 ft	47.70 Kips	45.49 Kips	93.19 Kips
45.01 ft	93.13 Kips	45.49 Kips	138.62 Kips
51.99 ft	137.59 Kips	45.49 Kips	183.08 Kips
52.01 ft	137.73 Kips	45.49 Kips	183.22 Kips
54.99 ft	159.25 Kips	45.49 Kips	204.74 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename:
Project Name: Naples Bay Bridge
Project Client: Naples
Computed By: km
Project Manager: jw

Project Date: 02/02/2006

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Pile
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	10.00 ft
	- Long Term Scour:	10.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	52.00 ft	10.00%	120.00 pcf	32.0/32.0	Nordlund
2	Cohesionless	3.00 ft	10.00%	125.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
18.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
19.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
20.00 ft	0.00 Kips	20.55 Kips	20.55 Kips
27.01 ft	17.09 Kips	34.95 Kips	52.04 Kips
36.01 ft	52.04 Kips	46.57 Kips	98.62 Kips
45.01 ft	101.62 Kips	46.57 Kips	148.19 Kips
51.99 ft	150.13 Kips	46.57 Kips	196.71 Kips
52.01 ft	150.29 Kips	46.57 Kips	196.86 Kips
54.99 ft	173.76 Kips	46.57 Kips	220.33 Kips

Coefficient of Earth Pressure:

Abutments designed to withstand a maximum lateral applied load equal to the Rankine active earth pressure, K_a.

$$\phi := 32 \cdot \text{deg} \qquad \beta := 0 \cdot \text{deg}$$

$$K_a := \tan\left(45 \cdot \text{deg} - \frac{\phi}{2}\right)^2 \qquad K_a = 0.307$$

At Rest Earth Pressure from Das Principles of Foundation Engineering Second Edition Eq. 5.3 pg 252

$$\phi := 32 \cdot \text{deg}$$

$$K_o := (1 - \sin(\phi))$$

$$K_o = 0.47$$

COMPUTE STRUCTURAL CAPACITY OF PIPE PILES

Pier - Pipe Pile Capacity - On bedrock, assume driven through granular soils to bedrock

Based on (0.25 * Fy); FS = 4

Pipe piles evaluated:

24 in diameter 1/2 in wall
24 in diameter 5/8 in wall
26 in diameter 1/2 in wall
26 in diameter 5/8 in wall
28 in diameter 11/16 in wall
30 in diameter 1/2 in wall
30 in diameter 5/8 in wall

Calculate the area of steel for piles assuming 1/8 in of sacrificial shell corrosion per BDG:

Look at piles with 1/2 in wall thickness:

$$dia := \begin{pmatrix} 24 \\ 26 \\ 30 \end{pmatrix} \cdot in \qquad wall_t := 0.5 \cdot in$$

1/8 in of shell corrosion: $c_{sc} := \frac{1}{8} \cdot in$

$$dia_{corr} := dia - 2 \cdot c_{sc} \qquad dia_{corr} = \begin{pmatrix} 23.75 \\ 25.75 \\ 29.75 \end{pmatrix} in \qquad wall_{corr} := wall_t - c_{sc}$$

$$wall_{corr} = 0.375\,in$$

$$A_{corr} := \pi \cdot \left(\frac{dia_{corr}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr}}{2}\right)^2$$

$$A_{corr} = \begin{pmatrix} 27.538 \\ 29.894 \\ 34.607 \end{pmatrix} in^2$$

FOR 1/2" PILES

Look at piles with 5/8 in wall thickness:

$$dia := \begin{pmatrix} 24 \\ 26 \\ 30 \end{pmatrix} \cdot in \qquad wall_t := 0.625 \cdot in$$

1/8 in of shell corrosion: $c_{sc} := \frac{1}{8} \cdot in$

$$dia_{corr} := dia - 2 \cdot c_{sc} \qquad dia_{corr} = \begin{pmatrix} 23.75 \\ 25.75 \\ 29.75 \end{pmatrix} in \qquad wall_{corr} := wall_t - c_{sc}$$

$$wall_{corr} = 0.5\,in$$

$$A_{corr} := \pi \cdot \left(\frac{dia_{corr}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr}}{2}\right)^2$$

$$A_{corr} = \begin{pmatrix} 36.521 \\ 39.663 \\ 45.946 \end{pmatrix} in^2$$

FOR 5/8" PILES

Look at pile with 11/16 in wall thickness:

$dia := 28 \cdot in$ $\quad wall_t := 0.6875 \cdot in$ $\quad$ 1/8 in of shell corrosion: $c_{sc} := \frac{1}{8} \cdot in$

$dia_{corr} := dia - 2 \cdot c_{sc}$ $\quad dia_{corr} = 27.75\,in$ $\quad wall_{corr} := wall_t - c_{sc}$

$wall_{corr} = 0.562\,in$

$$A_{corr} := \pi \cdot \left(\frac{dia_{corr}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr}}{2}\right)^2$$

$A_{corr} = 48.044\,in^2$ $\quad$ **FOR 1/16" PILES**

Using 45 ksi steel $\quad \sigma_a := 0.25 \cdot 45 \cdot ksi$

$\sigma_a = 11.25\,ksi$

Assuming 1/8 in of sacrificial shell corrosion per BDG

$$Area_1 := \begin{pmatrix} 27.538 \\ 36.521 \\ 29.894 \\ 39.663 \\ 48.044 \\ 34.307 \\ 45.946 \end{pmatrix} \cdot in^2$$

24 in diameter 1/2 in wall
24 in diameter 5/8 in wall
26 in diameter 1/2 in wall
26 in diameter 5/8 in wall
28 in diameter 11/16 in wall
30 in diameter 1/2 in wall
30 in diameter 5/8 in wall

$Q_{all} := \sigma_a \cdot Area_1$

$$Q_{all} = \begin{pmatrix} 310 \\ 411 \\ 336 \\ 446 \\ 540 \\ 386 \\ 517 \end{pmatrix} kip$$

24 in diameter 1/2 in wall
24 in diameter 5/8 in wall
26 in diameter 1/2 in wall
26 in diameter 5/8 in wall
28 in diameter 11/16 in wall
30 in diameter 1/2 in wall
30 in diameter 5/8 in wall

COMPUTE GEOTECHNICAL CAPACITY OF PIPE PILES

Pier - Pipe Pile Axial Capacity - On bedrock

Pipe piles evaluated:
24 in diameter 1/2 in wall
24 in diameter 5/8 in wall
26 in diameter 1/2 in wall
26 in diameter 5/8 in wall
28 in diameter 11/16 in wall
30 in diameter 1/2 in wall
30 in diameter 5/8 in wall

Bedrock at the site is identified as biotite-muscovite GRANITE.

$Q_{uc} := 21000 \cdot psi$ compressive strength of granite from Fang Table 3.8 pg 95
Granite 750 - 2500 kg/cm^2 Use 1500 kg/cm^2 = 21000 psi

$\phi_1 := 34 \cdot deg$ from Tomlinson pg 139 — Reference: Pile Design and Construction Practice, M.J. Tomlinson, Fourth Edition

Corrections for wedge failure under a strip footing -
multiply the cN_c factor by 1.2 for a circular pile
multiply γBN_γ factor by 0.7 for a circular pile

For RQD of 0 - 70%:	For RQD of 70 - 100%:
q_c = 0.33 x Q_{uc}	q_c = 0.33 to 0.88 x Q_{uc}
c = 0.1 x Q_{uc}	c = 0.1 x Q_{uc}
ϕ = 30 degrees	ϕ = 30 to 60 degrees

RQD = 43 to 100% Use: q_c = 0.33 x Q_{uc}
c = 0.1 x Q_{uc}
ϕ = 30 degrees

$Q_{uc} := 21000 \cdot psi$

$c := 0.1 \cdot Q_{uc}$ $\quad c = 2.1 \times 10^3\, psi$

$\gamma := 150 \cdot pcf$ for concrete

$B_{24} := 2 \cdot ft$ 24 in diameter pile

$B_{26} := 2.167 \cdot ft$ 26 in diameter pile

$B_{28} := 2.33 \cdot ft$ 28 in diameter pile

$B_{30} := 2.5 \cdot ft$ 30 in diameter pile

$D := 2 \cdot in$ Depth of possible embedment into bedrock surface during driving

$N_c := 13.86$ $\quad N_\gamma := 13.86$ $\quad N_q := 9.0$ based on Pells & Turner from Tomlinson page 140 - ϕ = 30

$q_c := 0.33 \cdot Q_{uc}$ $\quad q_c = 6.93 \times 10^3\, psi$

For Pipe Piles: **24 in diameter 1/2 in wall**
24 in diameter 5/8 in wall

$$q_{ub} := 1.2 \cdot c \cdot N_c + 0.7 \cdot 0.5 \cdot \gamma \cdot B_{24} \cdot N_\gamma + \gamma \cdot D \cdot N_q$$

$$q_{ub} = 34.939\,\text{ksi}$$

Area steel for 24 in dia, 1/2 in wall $A_{24x0.5} := 36.904 \cdot in^2$

$Q_{ult3} := q_{ub} \cdot A_{24x0.5}$ $Q_{ult3} = 1289.384\,\text{kip}$

$Q_{all\_tip4} := \dfrac{Q_{ult3}}{2.25}$ $Q_{all\_tip4} = 573.06\,\text{kip}$

Area steel for 24 in dia, 5/8 in wall $A_{24x0.625} := 45.77 \cdot in^2$

$Q_{ult3} := q_{ub} \cdot A_{24x0.625}$ $Q_{ult3} = 1.599 \times 10^3\,\text{kip}$

$Q_{all\_tip4} := \dfrac{Q_{ult3}}{2.25}$ $Q_{all\_tip4} = 710.734\,\text{kip}$

For Pipe Piles: **26 in diameter 1/2 in wall**
26 in diameter 5/8 in wall

$$q_{ub} := 1.2 \cdot c \cdot N_c + 0.7 \cdot 0.5 \cdot \gamma \cdot B_{26} \cdot N_\gamma + \gamma \cdot D \cdot N_q$$

$$q_{ub} = 34.94\,\text{ksi}$$

Area steel for 26 in dia, 1/2 in wall $A_{26x0.5} := 39.84 \cdot in^2$

$Q_{ult3} := q_{ub} \cdot A_{26x0.5}$ $Q_{ult3} = 1.392 \times 10^3\,\text{kip}$

$Q_{all\_tip4} := \dfrac{Q_{ult3}}{2.25}$ $Q_{all\_tip4} = 618.666\,\text{kip}$

Area steel for 26 in dia, 5/8 in wall $A_{26x0.625} := 49.64 \cdot in^2$

$Q_{ult3} := q_{ub} \cdot A_{26x0.625}$ $Q_{ult3} = 1.734 \times 10^3\,\text{kip}$

$Q_{all\_tip4} := \dfrac{Q_{ult3}}{2.25}$ $Q_{all\_tip4} = 770.848\,\text{kip}$

For Pipe Pile: 28 in diameter 11/16 in wall

$$q_{ub} := 1.2 \cdot c \cdot N_c + 0.7 \cdot 0.5 \cdot \gamma \cdot B_{24} \cdot N_\gamma + \gamma \cdot D \cdot N_q$$

$$q_{ub} = 34.939\,\text{ksi}$$

Area steel for 28 in dia, 11/16 in wall $\quad A_{28x0.688} := 59.006 \cdot \text{in}^2$

$$Q_{ult3} := q_{ub} \cdot A_{28x0.688} \qquad Q_{ult3} = 2061.6029\,\text{kip}$$

$$Q_{all\_tip4} := \frac{Q_{ult3}}{2.25} \qquad Q_{all\_tip4} = 916.268\,\text{kip}$$

For Pipe Piles: 30 in diameter 1/2 in wall
30 in diameter 5/8 in wall

$$q_{ub} := 1.2 \cdot c \cdot N_c + 0.7 \cdot 0.5 \cdot \gamma \cdot B_{26} \cdot N_\gamma + \gamma \cdot D \cdot N_q$$

$$q_{ub} = 34.94\,\text{ksi}$$

Area steel for 30 in dia, 1/2 in wall $\quad A_{30x0.5} := 46.305 \cdot \text{in}^2$

$$Q_{ult3} := q_{ub} \cdot A_{30x0.5} \qquad Q_{ult3} = 1.618 \times 10^3\,\text{kip}$$

$$Q_{all\_tip4} := \frac{Q_{ult3}}{2.25} \qquad Q_{all\_tip4} = 719.059\,\text{kip}$$

Area steel for 30 in dia, 5/8 in wall $\quad A_{30x0.625} := 57.704 \cdot \text{in}^2$

$$Q_{ult3} := q_{ub} \cdot A_{30x0.625} \qquad Q_{ult3} = 2.016 \times 10^3\,\text{kip}$$

$$Q_{all\_tip4} := \frac{Q_{ult3}}{2.25} \qquad Q_{all\_tip4} = 896.072\,\text{kip}$$

AXIAL STRUCTURAL CAPACITY GOVERNS

Evaluate Lateral Movement of Fender Pipe Piles

Fender - Pipe Pile Capacity - assume friction piles driven through granular soils

Use LPile to determine Lateral movement

For the Fender system R. Nouse proposes to use a:

12" diameter
1/2" wall thickness
70 ksi
Concrete filled steel pipe pile

Calculate the composite Moment of Inertia I_t

$E_{conc} := 4000 \cdot ksi \qquad E_{steel} := 30000 \cdot ksi$

$$n := \frac{E_{steel}}{E_{conc}} \qquad n = 7.5$$

$d_{conc} := 12 \cdot in - 2(0.5 \cdot in) \qquad d_{conc} = 11\, in$

$d_{steel} := 12 \cdot in$

$$I_{conc} := \frac{\pi \cdot d_{conc}^{\;4}}{64} \qquad I_{conc} = 718.688\, in^4$$

$$I_{steel} := \frac{\pi \cdot \left(d_{steel}^{\;4} - d_{conc}^{\;4}\right)}{64} \qquad I_{steel} = 299.188\, in^4$$

$$I_t := \frac{I_{conc}}{n} + I_{steel} \qquad I_t = 395.013\, in^4$$

Calculate composite Area, A_t

$$A_t := \frac{\frac{d_{conc}^{\;2}}{4} \cdot \pi}{n} + \left(\frac{d_{steel}^{\;2}}{4} \cdot \pi - \frac{d_{conc}^{\;2}}{4} \cdot \pi\right)$$

$A_t = 30.735\, in^2$

===

LPILE Plus for Windows, Version 4.0

LK 12/05

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

===

This program is licensed to:

Kate Maguire
Maine Department of Transportation

```
Path to file locations:      C:\Program Files\Ensoft\LpileP4\
Name of input data file:     napels1.lpd
Name of output file:         napels1.lpo
Name of plot output file:    napels1.lpp
Name of runtime file:        napels1.lpr
```

Time and Date of Analysis

Date: December 12, 2005   Time: 15: 6:22

Problem Title

Naples Bay Bridge Fender System Piles

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
```
- Number of pile increments              =        100
- Maximum number of iterations allowed =        100
- Deflection tolerance for convergence =   1.0000E-05 in
- Maximum allowable deflection          =   1.0000E+02 in
```

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

```
Pile Length                                =     540.00 in
Depth of ground surface below top of pile =     180.00 in
Slope angle of ground surface              =        .00 deg.
```

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	12.000	395.0130	30.7350	36500000.000
2	540.0000	12.000	395.0130	30.7350	36500000.000

Soil and Rock Layering Information

LK 12/05

The soil profile is modelled using 1 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 720.000 in
p-y subgrade modulus k for top of soil layer = 20.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 20.000 lbs/in**3

(Depth of lowest layer extends 180.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	180.00	.03300
2	720.00	.03300

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	180.000	.00000	32.00	------	------
2	720.000	.00000	32.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

---

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 38000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = .000 lbs

(Zero slope for this load indicates fixed-head condition)

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

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 38000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = .000 lbs

(Zero slope for this load indicates fixed-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	8.291	-6.281E+06	38000.0000	-9.869E-16	95411.8022	0.0000
5.400	8.285	-6.076E+06	38000.0000	-.002314	92294.9428	0.0000
10.800	8.266	-5.871E+06	38000.0000	-.004552	89178.0833	0.0000

LE 12/05

16.200	8.236	-5.666E+06	38000.0000	-.006712	86061.2239	0.0000
21.600	8.194	-5.461E+06	38000.0000	-.008796	82944.3644	0.0000
27.000	8.141	-5.255E+06	38000.0000	-.010802	79827.5050	0.0000
32.400	8.077	-5.050E+06	38000.0000	-.012732	76710.6455	0.0000
37.800	8.003	-4.845E+06	38000.0000	-.014585	73593.7861	0.0000
43.200	7.919	-4.640E+06	38000.0000	-.016362	70476.9267	0.0000
48.600	7.826	-4.435E+06	38000.0000	-.018061	67360.0672	0.0000
54.000	7.724	-4.229E+06	38000.0000	-.019683	64243.2078	0.0000
59.400	7.614	-4.024E+06	38000.0000	-.021229	61126.3483	0.0000
64.800	7.495	-3.819E+06	38000.0000	-.022698	58009.4889	0.0000
70.200	7.369	-3.614E+06	38000.0000	-.024090	54892.6294	0.0000
75.600	7.235	-3.409E+06	38000.0000	-.025405	51775.7700	0.0000
81.000	7.094	-3.203E+06	38000.0000	-.026643	48658.9105	0.0000
86.400	6.947	-2.998E+06	38000.0000	-.027805	45542.0511	0.0000
91.800	6.794	-2.793E+06	38000.0000	-.028889	42425.1916	0.0000
97.200	6.635	-2.588E+06	38000.0000	-.029897	39308.3322	0.0000
102.600	6.471	-2.383E+06	38000.0000	-.030828	36191.4728	0.0000
108.000	6.302	-2.177E+06	38000.0000	-.031682	33074.6133	0.0000
113.400	6.129	-1.972E+06	38000.0000	-.032459	29957.7539	0.0000
118.800	5.952	-1.767E+06	38000.0000	-.033159	26840.8944	0.0000
124.200	5.771	-1.562E+06	38000.0000	-.033782	23724.0350	0.0000
129.600	5.587	-1.357E+06	38000.0000	-.034329	20607.1755	0.0000
135.000	5.400	-1.151E+06	38000.0000	-.034799	17490.3161	0.0000
140.400	5.211	-946283.7047	38000.0000	-.035191	14373.4566	0.0000
145.800	5.020	-741083.7047	38000.0000	-.035507	11256.5972	0.0000
151.200	4.828	-535883.7047	38000.0000	-.035747	8139.7378	0.0000
156.600	4.634	-330683.7047	38000.0000	-.035909	5022.8783	0.0000
162.000	4.440	-125483.7047	38000.0000	-.035994	1906.0189	0.0000
167.400	4.245	79716.2953	38000.0000	-.036003	1210.8406	0.0000
172.800	4.051	284916.2953	38000.0000	-.035935	4327.7000	0.0000
178.200	3.857	490116.2953	38000.0000	-.035789	7444.5595	0.0000
183.600	3.664	695316.2953	37963.3751	-.035567	10561.4189	-13.5648
189.000	3.473	900120.7459	37823.8738	-.035269	13672.2702	-38.1023
194.400	3.283	1.104E+06	37548.5670	-.034893	16766.2451	-63.8631
199.800	3.096	1.306E+06	37138.0027	-.034442	19831.9337	-88.1977
205.200	2.911	1.505E+06	36601.7201	-.033916	22858.5575	-110.4255
210.600	2.730	1.701E+06	35955.7299	-.033316	25836.2714	-128.8302
216.000	2.552	1.893E+06	35208.3941	-.032642	28756.9236	-147.9609
221.400	2.377	2.081E+06	34335.7898	-.031898	31612.0407	-175.2259
226.800	2.207	2.264E+06	33319.7797	-.031084	34389.5463	-201.0741
232.200	2.042	2.441E+06	32135.3756	-.030203	37077.9917	-237.5941
237.600	1.881	2.611E+06	30737.2487	-.029257	39661.2015	-280.2307
243.000	1.726	2.773E+06	29095.9606	-.028249	42120.2909	-327.6538
248.400	1.576	2.925E+06	27184.7322	-.027182	44434.2552	-380.2086
253.800	1.432	3.067E+06	24979.2755	-.026060	46579.8166	-436.6272
259.200	1.294	3.195E+06	22458.7266	-.024887	48531.9863	-496.9095
264.600	1.163	3.309E+06	19602.2212	-.023669	50264.0637	-561.0555
270.000	1.039	3.407E+06	16388.8953	-.022411	51747.6372	-629.0652
275.400	.921221	3.486E+06	12797.8846	-.021121	52952.5839	-700.9387
280.800	.810695	3.545E+06	8808.3250	-.019804	53847.0692	-776.6760
286.200	.707339	3.581E+06	4399.3521	-.018469	54397.5476	-856.2769
291.600	.611226	3.593E+06	-449.8982	-.017126	54568.7619	-939.7417
297.000	.522379	3.576E+06	-5760.2903	-.015783	54323.7439	-1027.0702
302.400	.440765	3.530E+06	-11446.6572	-.014453	53623.8138	-1078.9916
307.800	.366291	3.453E+06	-16887.7790	-.013145	52445.9744	-936.2387
313.200	.298800	3.348E+06	-21564.8303	-.011871	50853.4541	-796.0026
318.600	.238080	3.220E+06	-25495.9242	-.010641	48908.3667	-659.9582
324.000	.183873	3.073E+06	-28707.6054	-.009463	46670.9693	-529.5534
329.400	.135880	2.910E+06	-31233.6214	-.008343	44199.0209	-406.0081
334.800	.093772	2.735E+06	-33113.6980	-.007286	41547.2425	-290.3166
340.200	.057196	2.552E+06	-34392.3398	-.006295	38766.8766	-183.2544
345.600	.025781	2.364E+06	-35117.6740	-.005375	35905.3431	-85.3879
351.000	-8.52E-04	2.173E+06	-35340.3542	-.004525	33005.9895	2.9137
356.400	-.023091	1.982E+06	-35112.5359	-.003747	30107.9264	81.4634
361.800	-.041320	1.794E+06	-34486.9360	-.003040	27245.9453	150.2403
367.200	-.055922	1.610E+06	-33515.9823	-.002403	24450.5090	209.3722
372.600	-.067268	1.432E+06	-32251.0599	-.001833	21747.8082	259.1176
378.000	-.075719	1.261E+06	-30741.8567	-.001329	19159.8763	299.8466
383.400	-.081618	1.100E+06	-29035.8092	-8.865E-04	16704.7531	332.0229
388.800	-.085293	947813.2964	-27177.6483	-5.031E-04	14396.6902	356.1849
394.200	-.087052	806247.1717	-25209.0416	-1.746E-04	12246.3894	372.9287
399.600	-.087179	675555.6471	-23168.3297	1.029E-04	10261.2671	382.8905
405.000	-.085940	556029.2108	-21090.3498	3.335E-04	8445.7354	386.7316
410.400	-.083577	447779.8692	-19006.3417	5.215E-04	6801.4957	385.1232
415.800	-.080308	350760.7209	-16943.9287	6.710E-04	5327.8356	378.7334
421.200	-.076330	264785.4392	-14927.1670	7.863E-04	4021.9249	368.2154
426.600	-.071816	189547.3175	-12976.6541	8.714E-04	2879.1050	354.1968
432.000	-.066919	124637.5744	-11109.6907	9.302E-04	1893.1667	337.2711
437.400	-.061770	69562.6578	-9340.4856	9.666E-04	1056.6132	317.9900
442.800	-.056480	23760.3294	-7680.3983	9.841E-04	360.9045	296.8572
448.200	-.051142	-13385.6443	-6138.2094	9.860E-04	203.3196	274.3239
453.600	-.045831	-42532.3324	-4720.4131	9.755E-04	646.0395	250.7858
459.000	-.040606	-64366.1062	-3431.5240	9.555E-04	977.6808	226.5805
464.400	-.035511	-79592.7913	-2274.3912	9.286E-04	1208.9646	201.9872
469.800	-.030577	-88929.5311	-1250.5149	8.970E-04	1350.7839	177.2263
475.200	-.025823	-93098.3517	-360.3574	8.629E-04	1414.1056	152.4616
480.600	-.021258	-92821.3908	396.3549	8.281E-04	1409.8988	127.8021
486.000	-.016880	-88817.7193	1020.3451	7.941E-04	1349.0855	103.3054
491.400	-.012682	-81801.6634	1512.5202	7.621E-04	1242.5160	78.9817
496.800	-.008649	-72482.5007	1873.7295	7.332E-04	1100.9638	54.7995
502.200	-.004763	-61565.3850	2104.5533	7.081E-04	935.1396	30.6908
507.600	-.001001	-49753.3251	2205.1266	6.873E-04	755.7218	6.5585
513.000	.002660	-37750.0180	2175.0012	6.709E-04	573.3991	-17.7161
518.400	.006245	-26263.3121	2013.0528	6.589E-04	398.9233	-42.2648

LK 12/05

523.800	.009776	-16009.0474	1717.4370	6.510E-04	243.1674	-67.2226
529.200	.013276	-7714.9925	1285.6001	6.466E-04	117.1859	-92.7170
534.600	.016759	-2124.5660	714.3512	6.447E-04	32.2708	-118.8567
540.000	.020239	0.0000	0.0000	6.443E-04	0.0000	-145.7178

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =     8.29108284 in
Computed slope at pile head       =   -9.86865E-16
Maximum bending moment            =   -6281483.705 lbs-in
Maximum shear force               =      38000.000 lbs
Depth of maximum bending moment   =          0.000 in
Depth of maximum shear force      =     5.40000000 in
Number of iterations              =             20
Number of zero deflection points  =              2
```

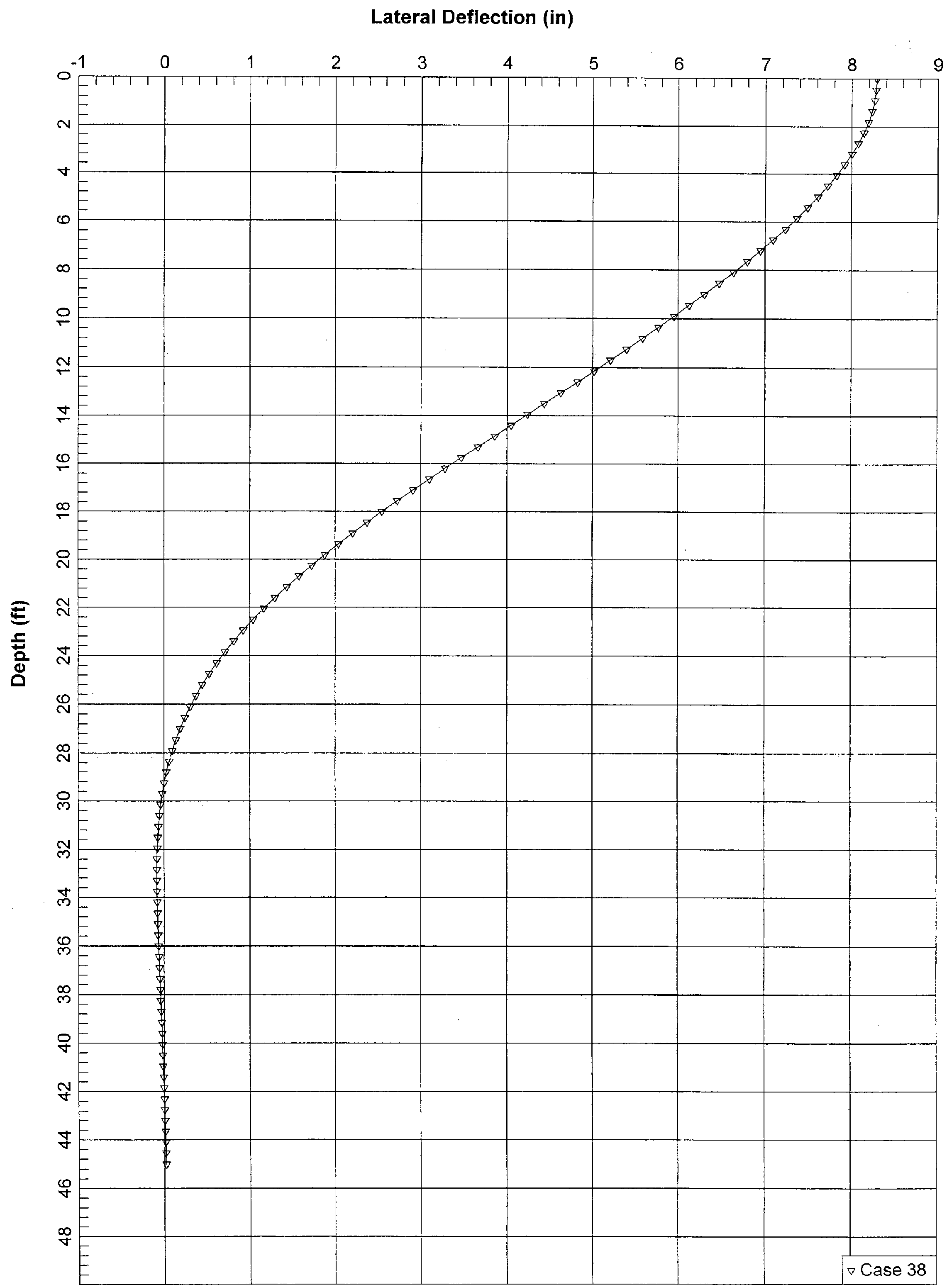
Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
2	V= 38000.000	S= 0.000	0.0000	8.2911	-6.281E+06	38000.0000

The analysis ended normally.



Lateral Deflection (in)
-1
0
1
2
3
4
5
6
7
8
9
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
Case 38

Bearing Capacity: Native Granular Soils

Any spread footing use at Naples will be founded on native granular soils.

Assumed parameters for the native granular sand layer:

$\gamma := 120 \cdot pcf$ $\quad \phi := 32 \cdot deg$ $\quad c := 0psf$ from Foundation Analysis and Design, Bowles 4th Edition Table 3-4 pg 141

$\gamma_1 := 120 \cdot pcf - 62.4 \cdot pcf$

$\gamma_1 = 57.6\,pcf$ unit weight of native granular soils = 120 pcf less 62.4 pcf unit weight of water for effective unit weight

Assume footing width of 10 feet

$B := 10 \cdot ft$

From Bowles 4th Edition Table 4-2 for $\phi = 32$

$N_q := 29.5 \quad N_c := 44.9 \quad N_\gamma := 27.9$

From Bowles 4th Edition Table 4-1
Assume strip footing:

$s_c := 1.0 \quad s_\gamma := 1.0$

Assume footing embedment, D_f of 5.5 feet for frost protection

$D_f := 5.5 \cdot ft \quad q_{bar} := \gamma_1 \cdot D_f \quad q_{bar} = 316.8\,psf$

$q_{ult} := c \cdot N_c \cdot s_c + q_{bar} \cdot N_q + 0.5 \cdot \gamma_1 \cdot B \cdot N_\gamma \cdot s_\gamma$

$q_{ult} = 8.322 \times 10^5\,Pa \quad q_{ult} = 1.738 \times 10^4\,psf$

$q_{all} := \frac{q_{ult}}{3} \quad q_{all} = 5.794 \times 10^3\,psf \quad q_{all} = 5.794\,ksf \quad q_{all} = 2.897\,tsf$

Use Q_{all} = 3 tsf

Settlement Analysis:

Schmertmann 1970/1978 Procedure
Reference: Fang - Foundation Engineering Handbook 1991
Section 5.5.3 pg 179

Any footing founded at a depth of 5.5 feet for frost protection on medium dense, fine to coarse sand.

Simplified soil profile:

Ground
Elevation 277.0 ft

Fill Sand
Assume: N = 25 bpf (medium dense)
γ = 125 pcf

Footing
Elevation 271.5 ft

Native fine to coarse sand
Elevation 262.0 ft

Groundwater
Elevation 261.0 ft

fine to coarse Sand
Assume: N=11 bpf (medium dense)
γ = 125 pcf

Top of Bedrock
Elevation 190.1ft

Schmertmann's 1978 procedure for strip footing:

Assume B = 2 ft - minimum allowable footing width

$B_1 := 2 \cdot ft \qquad L_{ft} := 10 \cdot ft$

$\frac{L_{ft}}{B_1} = 5$ Look at axisymmetric conditions L/B = 1

$q_{allw} := 3 \cdot tsf \qquad q_{allw} = 6000\,psf$ Based on bearing capacity calcs above

Δq is the change in vertical stress at the footing elevation

The thickness of the fill sand above is 15 ft
Assume γ = 125 pcf for the fill sands
Water table is at elevation 261.0 ft

$\Delta q := q_{allw} - (5.5 \cdot ft \cdot 125 \cdot pcf)$

$\Delta q = 5312.5\,psf$ net load intensity at foundation depth

$q_{vo} := (5.5 \cdot ft \cdot 125 \cdot pcf)$

$q_{vo} = 687.5\,psf$

$\sigma_{vp} := (5.5 \cdot ft \cdot 125 \cdot pcf) + (4 \cdot ft \cdot 125 \cdot pcf)$

$\sigma_{vp} = 1187.5\,psf$

$$I_{zp} := 0.5 + 0.1\left(\sqrt{\frac{\Delta q}{\sigma_{vp}}}\right) \qquad I_{zp} = 0.712$$

Determination of Es:

For medium dense silty sand N-value (Nv): $N_v := 25$ From Boring data

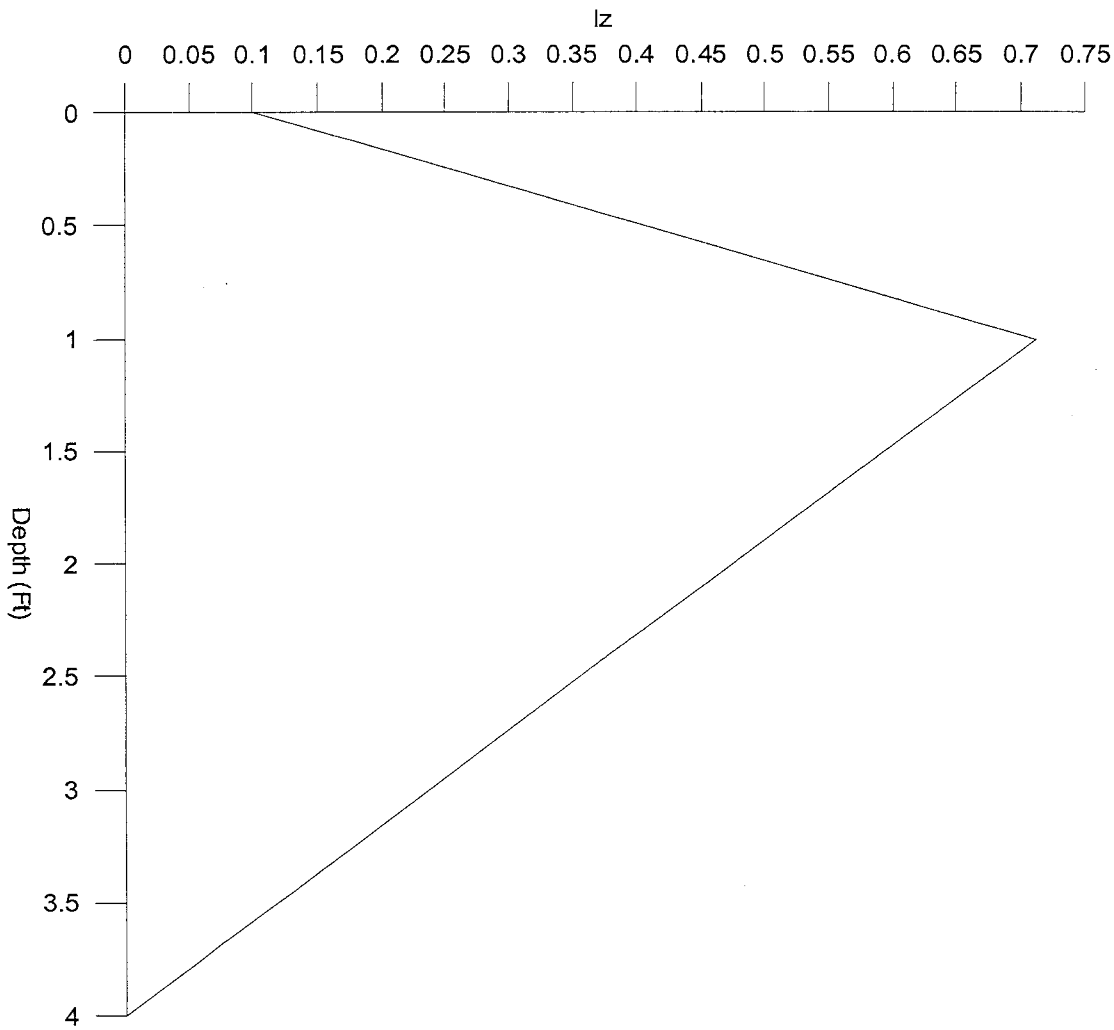
$q_c := N_v \cdot 3.5$ From table 5.6

$q_c = 87.5$

From Equation 5.11

$E_s := q_c \cdot 3.5 \qquad E_s = 306.25$

For axisymetric conditions - simplified strain influence factor distribution



Layer	z	Δz	I_z	q_c	E_s	$I_z \cdot \frac{\Delta z}{E_s}$
1	1 ft	0.5 ft	0.4	87.5	306.25	0.000653
2	3 ft	2.5 ft	0.36	87.5	306.25	0.002939
					Σ	0.009469

$\sigma_{vo} := q_{vo}$ $\qquad \sigma_{vo} = 687.5\,\text{psf}$

$$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_{vo}}{\Delta q}\right) \qquad C_1 = 0.935$$

$$C_2 := 1.0$$

Se = C1 x C2 x Δq x (Σ (Iz/Es) x Δz)

$$Se := 0.935 \cdot 1.0 \cdot \frac{5312.5}{2000} \cdot 0.009469$$

$Se = 0.02352$ Feet of settlement

$Se \cdot 12 = 0.282$ Inches of settlement

Frost Protection:

MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
Naples
DFI = 1370 degree-days

From the lab testing: soils are coarse grained with a water content = ~20%

$\text{Frost\_depth} := 64.8\text{in}$ (by interpolation)

$\text{Frost\_depth} = 5.4\,\text{ft}$

Use 5.5 feet

Note: The final depth of footing embedment may be controlled by the scour susceptibility of the foundation material and may, in fact, be deeper than the depth required for frost protection.

Determination of liquifaction potential of foundation soils

References:

NavFac MIL-HDBK-1007/3 (FKA DM-7.3)
Sections 2.4 Design Earthquake and 2.6 Liquefaction and Lateral Spreading

MaineDOT BDG Section 3.7

Seismicity of Site:

From MaineDOT BDG Figure 3-4: the peak rock acceleration at the site is 0.045g (possibly less). Per Section 3.7.1.1 bridge located in areas where the horizontal acceleration is less than or equal to 0.09 will be assigned to Seismic Performance Category A (SPC A).

The Naples Bay Bridge is on the National Highway System (NHS) and is considered a functionally important bridge. MaineDOT BDG Section 3.7.2 states that functionally important bridges with 2 or more spans in the SPC A category will be designed according to the requirements for SPC B with an acceleration coefficient of A = 0.09.

DESIGN FOR SPC B WITH A =0.09

Assumed values:

Total unit weight of soil: $\gamma_t := 120 \cdot pcf$

Saturated unit weight of soil: $\gamma_{sat} := 125 \cdot pcf$

Unit weight of water: $\gamma_w := 62.4 \cdot pcf$

Determineacceleration at ground surface due to seismic event and soil conditions.
NavFac MIL-HDBK-1007/3 Table 5 (page 76) goes to 0.10g.
Use this value with Soil Type C, and linear algebra to determine modified value of peak ground acceleration (a) at 0.09.

Peak Ground Acceleration Modified for Soil Conditions	Effective Peak Acceleration
X	0.09
0.16	0.10
0.28	0.20

Solve for X: X = 0.15g, therefore, a= 0.15

Determine total & effective stress within the sand strata. Use subsurface conditions at boring BB-NBB-101: with 87 feet of sand over bedrock. There is 19 ft of overburden considered for liquifaction and the water table at 16 feet below ground surface.

$d_{ob} := 19 \cdot ft \quad d_w := 16 \cdot ft$

$\sigma_{ob} := d_w \cdot \gamma_t + (d_{ob} - d_w) \cdot \gamma_{sat}$

$\sigma_{ob} = 2295\,psf$

$\sigma'_{ob} := d_w \cdot \gamma_t + (d_{ob} - d_w) \cdot (\gamma_{sat} - \gamma_w)$

$\sigma'_{ob} = 2107.8\,psf$

range variable→ $i := 2, 4 .. 68$

$z_i := d_{ob} + i \cdot ft$

total stress: $\sigma_i := \sigma_{ob} + \gamma_t \cdot i \cdot ft$

effective stress: $\sigma'_i := \sigma'_{ob} + (\gamma_{sat} - \gamma_w) \cdot i \cdot ft$

$z_i =$ (ft), $\sigma_i =$ (psf), $\sigma'_i =$ (psf)

	z_i (ft)	σ_i (psf)	σ'_i (psf)
0	21	2535	2233
1	23	2775	2358
2	25	3015	2483
3	27	3255	2609
4	29	3495	2734
5	31	3735	2859
6	33	3975	2984
7	35	4215	3109
8	37	4455	3235
9	39	4695	3360
10	41	4935	3485
11	43	5175	3610
12	45	5415	3735
13	47	5655	3861
14	49	5895	3986
15	51	6135	4111
16	53	6375	4236
17	55	6615	4361
18	57	6855	4487
19	59	7095	4612
20	61	7335	4737
21	63	7575	4862
22	65	7815	4987
23	67	8055	5113
24	69	8295	5238
25	71	8535	5363
26	73	8775	5488
27	75	9015	5613
28	77	9255	5739
29	79	9495	5864
30	81	9735	5989
31	83	9975	6114
32	85	10215	6239
33	87	10455	6365

Determined the stress reduction factor, r_d:

From NavFac MIL-HDBK-1007/3 page 82 the stress reduction factor varies from 1.0 at ground surface to a value of 0.9 at a depth of about 30 ft.

$$\frac{1 - 0.9}{0 \cdot ft - 30 \cdot ft} = -3.333 \times 10^{-3}\, ft^{-1}$$ ←Slope of r_d function, described above

$$r_{d_i} := \begin{array}{|l} a \leftarrow \left(-3.333 \cdot 10^{-3} \cdot ft^{-1}\right) \cdot \left(d_{ob} + i \cdot ft\right) + 1 \\ b \leftarrow 0.9 \\ x \leftarrow if\left[\left(d_{ob} + i \cdot ft\right) < 30 \cdot ft, a, b\right] \end{array}$$

$r_{d_i} =$

	0
0	0.93
1	0.923
2	0.917
3	0.91
4	0.903
5	0.9
6	0.9
7	0.9
8	0.9
9	0.9
10	0.9
11	0.9
12	0.9
13	0.9
14	0.9
15	0.9
16	0.9
17	0.9
18	0.9
19	0.9
20	0.9
21	0.9
22	0.9
23	0.9
24	0.9
25	0.9
26	0.9
27	0.9
28	0.9
29	0.9
30	0.9
31	0.9
32	0.9
33	0.9

Compute Cyclic Stress Ratio, CSR, as defined on page 82 of NavFac MIL-HDBK-1007/3

$a := 0.15 \cdot g$

$$CSR_i := 0.65 \cdot \left(\frac{a}{g}\right) \cdot \left(\frac{\sigma_i}{\sigma'_i}\right) \cdot r_{d_i}$$

$CSR_i =$

	0
0	0.103
1	0.106
2	0.109
3	0.111
4	0.113
5	0.115
6	0.117
7	0.119
8	0.121
9	0.123
10	0.124
11	0.126
12	0.127
13	0.129
14	0.13
15	0.131
16	0.132
17	0.133
18	0.134
19	0.135
20	0.136
21	0.137
22	0.137
23	0.138
24	0.139
25	0.14
26	0.14
27	0.141
28	0.142
29	0.142
30	0.143
31	0.143
32	0.144
33	0.144

Cyclic Resistance Ratio, CRR, from Figure 24, NavFac MIL-HDBK-1007/3, for N-value of 15 (approximate average of value of strata in question).

$CRR := 0.162$ Factor of safety for liquifaction during seismic event of 0.09g

$$FS_i := \frac{CRR}{CSR_i}$$

$FS_i =$

	0
0	1.57
1	1.53
2	1.49
3	1.46
4	1.44
5	1.41
6	1.39
7	1.36
8	1.34
9	1.32
10	1.3
11	1.29
12	1.27
13	1.26
14	1.25
15	1.24
16	1.23
17	1.22
18	1.21
19	1.2
20	1.19
21	1.18
22	1.18
23	1.17
24	1.17
25	1.16
26	1.15
27	1.15
28	1.14
29	1.14
30	1.14
31	1.13
32	1.13
33	1.12

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 2/16/06

Attention To: Jen Smith/Herb Macomber **Dept:** Reproduction Room

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: Naples, Naples Bay Bridge over Chutes River, PIN. 11060.00

Attached is one (1) copy of Soils Report 2006-06, entitled "FINAL GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: NAPLES BAY BRIDGE OVER CHUTES RIVER, US ROUTE 302, NAPLES, MAINE" dated: February 2006.

Please forward your copy to Kate Maguire after report has been scanned.

taw
att: 1 of 2006-06

COPY

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 2/27/06

To: Roger Naous **Dept:** Urban and Federal Bridge Program

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: Naples, Naples Bay Bridge over Chutes River, PIN. 11060.00

Attached is one (1) copy of Soils Report 2006-06, entitled "FINAL GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: NAPLES BAY BRIDGE OVER CHUTES RIVER, US ROUTE 302, NAPLES, MAINE" dated: February 2006.

[redacted]

taw
att: 1 of 2006-06

COPY

STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Interdepartmental Memorandum

Date 2/27/06

To: Matthew Steele **Dept:** Environment Office

From: Kate Maguire **Dept:** Urban and Federal Bridge Program
Geotechnical Section

Subject: Naples, Naples Bay Bridge over Chutes River, PIN. 11060.00

Attached is one (1) copy of Soils Report 2006-06, entitled "FINAL GEOTECHNICAL DESIGN REPORT for THE REPLACEMENT OF: NAPLES BAY BRIDGE OVER CHUTES RIVER, US ROUTE 302, NAPLES, MAINE" dated: February 2006.

[redacted]

taw
att: 1 of 2006-06

State of Maine

Department of Transportation
Bridge Program - Geotechnical
16 State House Station
Augusta, Maine 04333-0016

Kate Maguire, PE
Phone: 624-3415
FAX: 624-3491
email: kate.maguire@maine.gov

Addendum #1

To: Roger Naous, PE
cc: Jim Wentworth, PE
From: Kate Maguire, PE
Date: June 8, 2006
Re: Addendum #1
To MaineDOT Soils Report No. 2006-06
Final Geotechnical Design Report
Naples Bay Bridge
Naples, Maine
PIN: 11060.00

This Addendum to the Final Geotechnical Design Report for the Naples Bay Bridge is to transmit information regarding an additional boring conducted in the vicinity of the proposed pier location.

Subsurface conditions at the pier location were explored between March 21 and 22, 2006. One boring, BB-NBB-201 was drilled at the location of the proposed pier. The boring locations for all three borings drilled at the site are shown on *Sheet 2 - Boring Location Plan* and *Sheet 3 - Interpretive Subsurface Profile* attached to this addendum. Boring BB-NBB-201 was drilled to a depth of approximately 62.3 ft below the river bed surface. The boring was located in the field by use of a tape after completion of the drilling program.

The boring was drilled by the MaineDOT Materials Testing & Exploration team. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring log attached to this addendum and graphically on *Sheet 5 - Boring Logs* also attached to this addendum (*Sheet 4 - Boring Logs* can be found in the original report and is unchanged by this addendum). Drilling in soil was performed using cased wash boring techniques. Soil samples were obtained at 10-ft intervals using Standard Penetration Test (SPT) methods. Drilling in bedrock was performed using diamond rock coring with a NQ-sized (1.88 inch) double tube core barrel with which rock core samples were obtained. The Rock Quality Designation (RQD) was calculated for the rock core obtained. The MaineDOT Geotechnical Team member selected the boring location and drilling methods, designated type and depth of sampling techniques, and identified field and laboratory testing requirements.

Laboratory testing for samples obtained in the boring consisted of six (6) Grain Size Analyses. The results of these laboratory tests are attached to this addendum. Moisture content information is also shown on the attached Boring Log and on *Sheet 5 - Boring Logs* attached to this addendum.

Subsurface conditions encountered in the boring were similar to those found in the abutment borings. The soil profile generally consisted of **fill soils** over-lying a layer of **sand** which is underlain by **bedrock**. An updated interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show on *Sheet 3 - Interpretive Subsurface Profile* attached to this addendum.

Pile Length. Pile length at the abutments and pier can be estimated based on the following data:

Location	Ground Elevation	Depth to Rock	Approximate Top of Rock Elevation	Estimated Pile Length	Rock Quality Designation
Abutment #1 BB-NBB-101	277.0 ft	86.9 ft	190.1 ft	60 ft	100%
Pier BB-NBB-201	263.9 ft	57.3 ft	206.6 ft	50 ft	94%
Abutment #2 BB-NBB-102	278.0 ft	60.7 ft	217.3 ft	43 ft	43%

All recommendations made in the original report entitled: Final Geotechnical Design Report for the Replacement of: Naples Bay Bridge over Chutes River US Route 302 Naples, Maine Soils Report No. 2006-06 remain unchanged.

If you have any questions or need any additional information, please let me know.

Attachments:

Sheet 2 - Boring Location Plan
Sheet 3 - Interpretive Subsurface Profile
Sheet 5 - Boring Logs
Boring Log BB-NBB-201
Laboratory Testing Summary Sheet
Grain Size Distribution Curves

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Naples Bay Bridge Over Chutes River	Boring No.:	BB-NBB-201		
Location:	US Route 302 Naples, Maine	PIN:	11060.00		
Driller:	MaineDOT	Elevation (ft.)	263.9	Auger ID/OD:	N/A
Operator:	Ervin Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/21/06-3/22/06	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	104+82.2, 2.5 Lt.	Casing ID/OD:	NW	Water Level*:	Boring in River

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/0	0.0 - 2.0	1/4/2/5	6	8			No sample recovery. River bottom very cobbley.	
						15				
						16	261.90		2.0	
						8				
						5				
5						6				
	1D	24/6	6.0 - 8.0	2/4/7/4	11	3			Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#175876 A-1-b, SP WC=17.7%
						16				
						73				
						117				
10						98				
						92				
						118				
						119				
						144				
15						143				
	2D	24/12	16.0 - 18.0	18/27/29/32	56	69			Brown, wet, very dense, fine to medium SAND, trace coarse sand and silt.	G#175877 A-3, SP-SM WC=14.3%
						96				
						122				
						130				
20						129				
						178				
						139				
						140				
						123				
25										

Remarks:

Casing and spoon driven with safty hammer.
13.7' from Bridge Deck to Ground.

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:** Naples Bay Bridge Over Chutes River **Location:** US Route 302 Naples, Maine	**Boring No.:** BB-NBB-201 **PIN:** 11060.00

Driller:	MaineDOT	**Elevation (ft.)**	263.9	**Auger ID/OD:**	N/A
Operator:	Ervin Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/21/06-3/22/06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	104+82.2, 2.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Boring in River

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25						124				
	3D	24/12	26.0 - 28.0	2/2/3/4	5	68			Golden brown, wet, loose, fine to medium SAND, trace silt.	G#175878 A-3, SP WC=23.0%
						78				
						110				
						110				
30						101				
						153				
						154				
						128				
						133				
35						127				
	4D	24/8	36.0 - 38.0	6/6/7/6	13	109			Light brown, wet, medium dense, fine to medium SAND, trace coarse sand and silt.	G#175879 A-3, SP WC=20.3%
						135				
						146				
						181				
40						161				
						144				
						149				
						145				
						168				
45						152				
	5D	24/12	46.0 - 48.0	3/3/4/4	7	152			Light brown, wet, loose, fine to medium SAND, trace coarse sand and silt.	G#175880 A-3, SP WC=18.5%
						135				
						152				
						124	214.90		49.0	
50										

Remarks:

Casing and spoon driven with safty hammer.
13.7' from Bridge Deck to Ground.

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: Naples Bay Bridge Over Chutes River Location: US Route 302 Naples, Maine	Boring No.: BB-NBB-201 PIN: 11060.00

Driller:	MaineDOT	**Elevation (ft.)**	263.9	**Auger ID/OD:**	N/A
Operator:	Ervin Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/21/06-3/22/06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	104+82.2, 2.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Boring in River

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50						126				
						146				
						162				
						143				
						157				
55						158				
	6D	6/2	56.0 - 56.5	16/50(0)	---	180			Grey brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, grey rock in nose of spoon.	G#175881 A-1-b, SW-SM WC=13.7%
	R1	60/60	57.3 - 62.3	RQD = 94%		a36 NQ	206.60		a36 blows for 0.3'. 57.3 Bedrock: White, grey, black, medium grained, hard, GRANITE. R1:Core Times (min:sec) 57.3-58.3' (3:43) 58.3-59.3' (3:13) 59.3-60.3' (2:20) VOID (?) at 60.0' bgs. 60.3-61.3' (2:50) 61.3-62.3' (2:06) 100% Recovery	
60										
							201.60		62.3 **Bottom of Exploration at 62.30 feet below ground surface.**	
65										
70										
75										

Remarks:

Casing and spoon driven with safty hammer.
13.7' from Bridge Deck to Ground.

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

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

Boring No.: BB-NBB-201

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Naples

Project Number: 11060.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-NBB-101, 4D	104+11.4	10.6 Rt.	20.0-22.0	176585	1	22.5			SP-SM	A-3	0
BB-NBB-101, 6D	104+11.4	10.6 Rt.	30.0-32.0	176586	1	21.9			SM	A-2-4	II
BB-NBB-101, 8D	104+11.4	10.6 Rt.	40.0-42.0	176587	1	20.9			SP-SM	A-3	0
BB-NBB-101, 10D	104+11.4	10.6 Rt.	50.0-52.0	176588	1	23.7			SP-SM	A-3	0
BB-NBB-201, 1D	104+82.2	2.5 Lt.	6.0-8.0	175876	3	17.7			SP	A-1-b	0
BB-NBB-201, 2D	104+82.2	2.5 Lt.	16.0-18.0	175877	3	14.3			SP-SM	A-3	0
BB-NBB-201, 3D	104+82.2	2.5 Lt.	26.0-28.0	175878	3	23.0			SP	A-3	0
BB-NBB-201, 4D	104+82.2	2.5 Lt.	36.0-38.0	175879	3	20.3			SP	A-3	0
BB-NBB-201, 5D	104+82.2	2.5 Lt.	46.0-48.0	175880	3	18.5			SP	A-3	0
BB-NBB-201, 6D	104+82.2	2.5 Lt.	56.0-56.5	175881	3	13.7			SW-SM	A-1-b	0
BB-NBB-102, 3D	105+39.5	9.4 Lt.	15.0-17.0	176582	2	14.6			SM	A-2-4	II
BB-NBB-102, 7D	105+39.5	9.4 Lt.	35.0-37.0	176583	2	22.1			SM	A-2-4	II
BB-NBB-102, 10D	105+39.5	9.4 Lt.	50.0-52.0	176584	2	15.4			SW	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.

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

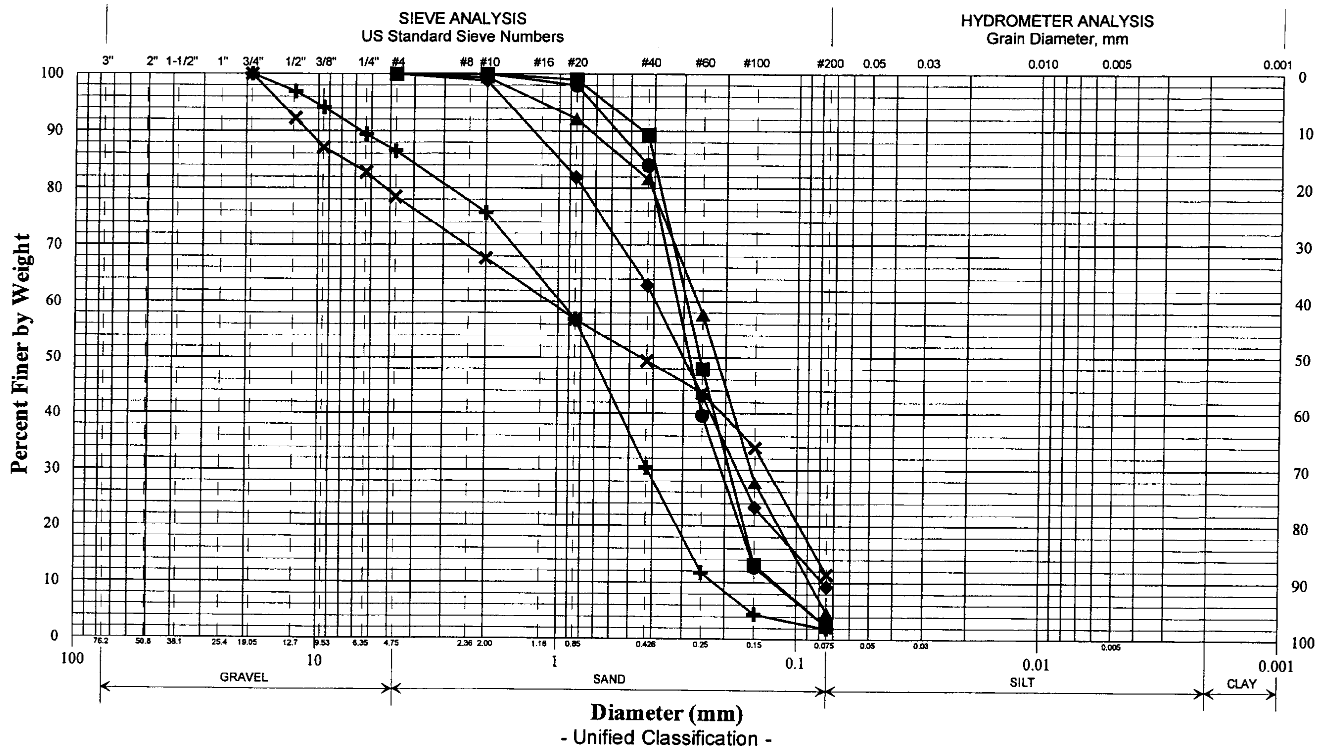
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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

State of Maine Department of Transportation

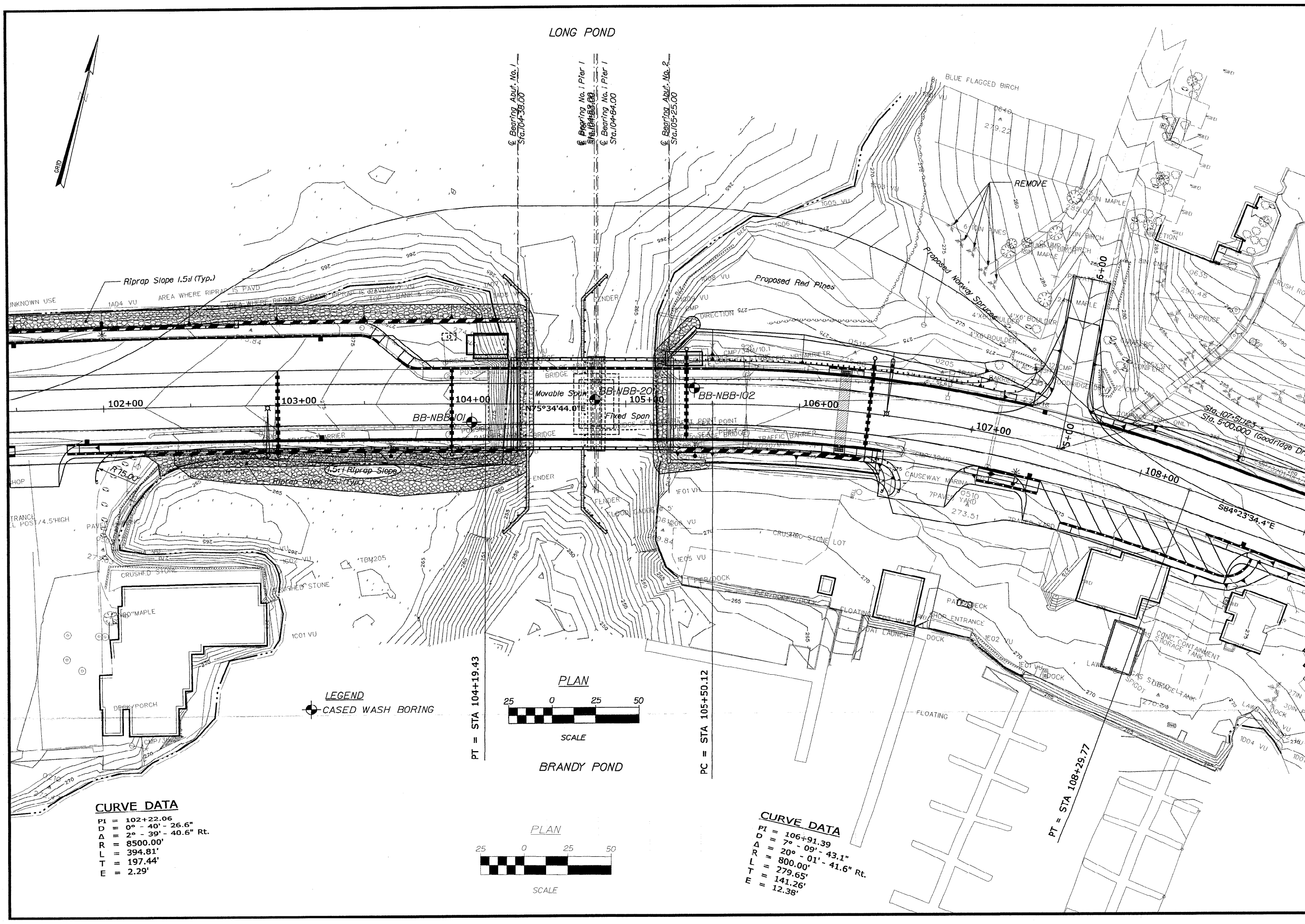
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NBB-201/1D	104+82.2	2.5 LT	6.0-8.0	SAND, little gravel, trace silt.	17.7			
◆	BB-NBB-201/2D	104+82.2	2.5 LT	16.0-18.0	SAND, trace silt.	14.3			
■	BB-NBB-201/3D	104+82.2	2.5 LT	26.0-28.0	SAND, trace silt.	23.0			
●	BB-NBB-201/4D	104+82.2	2.5 LT	36.0-38.0	SAND, trace silt.	20.3			
▲	BB-NBB-201/5D	104+82.2	2.5 LT	46.0-48.0	SAND, trace silt.	18.5			
✕	BB-NBB-201/6D	104+82.2	2.5 LT	56.0-56.5	SAND, some gravel, little silt.	13.7			

PIN:	011060.00
Town:	Naples
Reported by:	WHITE, TERRY A
Date:	4/14/2006

SHEET NO. 3



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-A106(000)X

PIN 11060.00

BRIDGE NO. 2047

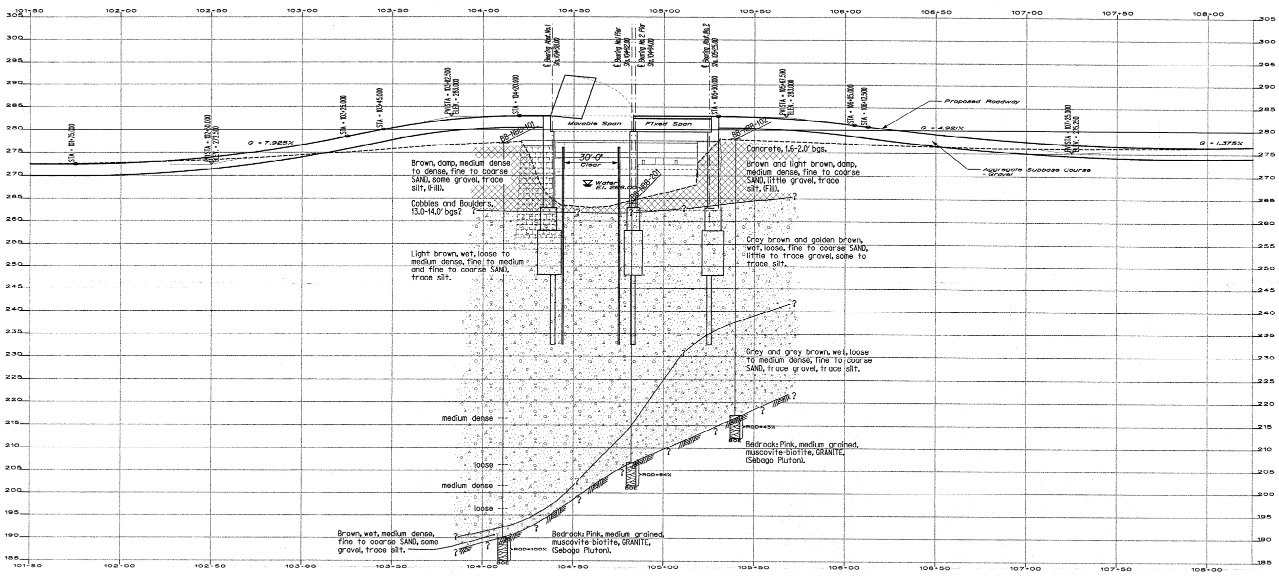
BRIDGE PLANS

NAPLES BAY BRIDGE
CHUTES RIVER

NAPLES CUMBERLAND COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF _



PROFILE

HORIZ 25 0 25 50
VERT 10 0 10 20
SCALE

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-A106(000)X

BRIDGE NO. 2047 | PIN 11060.00 | BRIDGE PLANS

PROJ. MANAGER	J. Wentworth	BY	DATE
DESIGN-DETAILED	K. Maguire	T. White	NOV 2005
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	-----	-----	-----
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	Changed 201 ledge elev.		JUN 2006
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE

P.E. NUMBER

DATE

NAPLES BAY BRIDGE
CHUTES RIVER
NAPLES CUMBERLAND COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF _

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Naples Bay Bridge over Chutes River Location: US Route 302 Naples, Maine	Boring No.: BB-NBB-201 PIN: 11060.00

Driller:	MaineDOT	Elevation (ft.)	263.9	Auger ID/OD:	N/A
Operator:	Ervin Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/21/06-3/22/06	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	104+82.2, 2.5 Lt.	Casing ID/OD:	NW	Water Level*:	Boring in River

Definitions:	Definitions:	Definitions:
D = Split Spoon Sample	S_u = Insitu Field Vane Shear Strength (psf)	WC = water content, percent
MD = Unsuccessful Split Spoon Sample attempt	T_v = Pocket Torvane Shear Strength (psf)	LL = Liquid Limit
U = Thin Wall Tube Sample	q_p = Unconfined Compressive Strength (ksf)	PL = Plastic Limit
R = Rock Core Sample	$S_{u(lab)}$ = Lab Vane Shear Strength (psf)	PI = Plasticity Index
V = Insitu Vane Shear Test	WOH = weight of 140lb. hammer	G = Grain Size Analysis
SSA = Solid Stem Auger	WOR = weight of rods	C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/0	0.00 - 2.00	1/4/2/5	6	8		No sample recovery. River bottom very cobbley.	
						15			
						16	261.90	2.00	
						8			
						5			
5						6			
	1D	24/6	6.00 - 8.00	2/4/7/4	11	3		Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#175876 A-1-b, SP WC=17.7%
						16			
						73			
						117			
10						98			
						82			
						118			
						119			
						144			
15						143			
	2D	24/12	16.00 - 18.00	18/27/29/32	56	69		Brown, wet, very dense, fine to medium SAND, trace coarse sand and silt.	G#175877 A-3, SP-SM WC=14.5%
						96			
						122			
						130			
20						129			
						178			
						139			
						140			
						123			
25						124			
	3D	24/12	26.00 - 28.00	2/2/3/4	5	66		Golden brown, wet, loose, fine to medium SAND, trace silt.	G#175878 A-3, SP WC=23.0%
						78			
						110			
						110			
30						101			
						153			
						154			
						128			
						133			
35						127			
	4D	24/8	36.00 - 38.00	6/6/7/6	13	109		Light brown, wet, medium dense, fine to medium SAND, trace coarse sand and silt.	G#175879 A-3, SP WC=20.3%
						135			
						146			
						181			
40						161			
						144			
						149			
						145			
						168			
45						152			
	5D	24/12	46.00 - 48.00	3/3/4/4	7	152		Light brown, wet, loose, fine to medium SAND, trace coarse sand and silt.	G#175880 A-3, SP WC=18.5%
						135			
						152	214.90	49.00	
						124			
50						126			
						146			
						162			
						143			
						157			
55						158			
	6D	6/2	56.00 - 56.50	16/50(0)	---	180		Grey brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, grey rock in nose of spoon. Ø36 blows for 0.3'.	G#175881 A-1-b, SW-SM WC=13.7%
	R1	60/60	57.30 - 62.30	RQD = 94%		a/e NQ	206.60	57.30 Bedrock: White, grey, black, medium grained, hard, GRANITE. R1:Core Times (min:sec) 57.3-58.3' (3:43) 58.3-59.3' (3:13) 59.3-60.3' (2:20) VOID (?) at 60.0' bgs. 60.3-61.3' (2:50) 61.3-62.3' (2:06) 100% Recovery	
60									
							201.60	62.30 Bottom of Exploration at 62.30 feet below ground surface.	
65									
70									
75									

Remarks:

Casing and spoon driven with safty hammer.
13.7' from Bridge Deck to Ground.

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

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

Page 1 of 1

Boring No.: BB-NBB-201

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-A106(000)X

PIN 11060.00

BRIDGE NO. 2047

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	J. Rashward		
DESIGN-DETAILED	K. Maguire	T. White	JUN 2006
CHECKED-REVIEWED			
DESIGN-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

NAPLES BAY BRIDGE
CHUTES RIVER
NAPLES CUMBERLAND COUNTY

BORING LOGS

SHEET NUMBER

5

OF _

State of Maine

Department of Transportation
Bridge Program Geotechnical
16 State House Station
Augusta, Maine 04333-0016

Kate Maguire PE
Phone 624-3415
FAX 624-3491
email kate maguire@maine gov

Addendum #1

To	Roger Naous, PE
cc	Jim Wentworth, PE
From	Kate Maguire, PE
Date	June 8, 2006
Re	Addendum #1 To MaineDOT Soils Report No 2006-06 Final Geotechnical Design Report Naples Bay Bridge Naples, Maine
PIN	11060 00

This Addendum to the Final Geotechnical Design Report for the Naples Bay Bridge is to transmit information regarding an additional boring conducted in the vicinity of the proposed pier location

Subsurface conditions at the pier location were explored between March 21 and 22, 2006 One boring, BB-NBB-201 was drilled at the location of the proposed pier The boring locations for all three borings drilled at the site are shown on *Sheet 2 - Boring Location Plan* and *Sheet 3 - Interpretive Subsurface Profile* attached to this addendum Boring BB-NBB-201 was drilled to a depth of approximately 62 3 ft below the river bed surface The boring was located in the field by use of a tape after completion of the drilling program

The boring was drilled by the MaineDOT Materials Testing & Exploration team Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring log attached to this addendum and graphically on *Sheet 5 - Boring Logs* also attached to this addendum (*Sheet 4 - Boring Logs* can be found in the original report and is unchanged by this addendum) Drilling in soil was performed using cased wash boring techniques Soil samples were obtained at 10-ft intervals using Standard Penetration Test (SPT) methods Drilling in bedrock was performed using diamond rock coring with a NQ-sized (1 88 inch) double tube core barrel with which rock core samples were obtained The Rock Quality Designation (RQD) was calculated for the rock core obtained The MaineDOT Geotechnical Team member selected the boring location and drilling methods, designated type and depth of sampling techniques, and identified field and laboratory testing requirements

Laboratory testing for samples obtained in the boring consisted of six (6) Grain Size Analyses The results of these laboratory tests are attached to this addendum Moisture content information is also shown on the attached Boring Log and on *Sheet 5 - Boring Logs* attached to this addendum

Subsurface conditions encountered in the boring were similar to those found in the abutment borings The soil profile generally consisted of **fill soils** over-lying a layer of **sand** which is underlain by **bedrock**. An updated interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show on *Sheet 3 - Interpretive Subsurface Profile* attached to this addendum.

Pile Length. Pile length at the abutments and pier can be estimated based on the following data:

Location	Ground Elevation	Depth to Rock	Approximate Top of Rock Elevation	Estimated Pile Length	Rock Quality Designation
Abutment #1 BB-NBB-101	277.0 ft	86.9 ft	190.1 ft	60 ft	100%
Pier BB-NBB-201	263.9 ft	57.3 ft	206.6 ft	50 ft	94%
Abutment #2 BB-NBB-102	278.0 ft	60.7 ft	217.3 ft	43 ft	43%

All recommendations made in the original report entitled: Final Geotechnical Design Report for the Replacement of: Naples Bay Bridge over Chutes River US Route 302 Naples, Maine Soils Report No. 2006-06 remain unchanged.

If you have any questions or need any additional information, please let me know.

Attachments:

Sheet 2 - Boring Location Plan
Sheet 3 - Interpretive Subsurface Profile
Sheet 5 - Boring Logs
Boring Log BB-NBB-201
Laboratory Testing Summary Sheet
Grain Size Distribution Curves

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Naples Bay Bridge Over Chutes River
Location: US Route 302 Naples, Maine

Boring No.: BB-NBB-201
PIN: 11060.00

Driller:	MaineDOT	**Elevation (ft.)**	263.9	**Auger ID/OD:**	N/A
Operator:	Ervin Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/21/06-3/22/06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	104+82.2, 2.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Boring in River

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/0	0.0 - 2.0	1/4/2/5	6	8			No sample recovery. River bottom very cobbley.	
						15				
						16	261.90		2.0	
						8				
5						5				
						6				
	1D	24/6	6.0 - 8.0	2/4/7/4	11	3			Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#175876 A-1-b, SP WC=17.7%
						16				
						73				
10						117				
						98				
						92				
						118				
						119				
15						144				
						143				
	2D	24/12	16.0 - 18.0	18/27/29/32	56	69			Brown, wet, very dense, fine to medium SAND, trace coarse sand and silt.	G#175877 A-3, SP-SM WC=14.3%
						96				
						122				
20						130				
						129				
						178				
						139				
						140				
25						123				

Remarks:

Casing and spoon driven with safty hammer.
13.7' from Bridge Deck to Ground.

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: Naples Bay Bridge Over Chutes River Location: US Route 302, Naples, Maine	Boring No.: BB-NBB-201 PIN: 11060.00

Driller:	MaineDOT	Elevation (ft.)	263.9	Auger ID/OD:	N/A
Operator:	Ervin Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/21/06-3/22/06	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	104+82.2, 2.5 Lt.	Casing ID/OD:	NW	Water Level*:	Boring in River

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25						124				
	3D	24/12	26.0 - 28.0	2/2/3/4	5	68			Golden brown, wet, loose, fine to medium SAND, trace silt.	G#175878 A-3, SP WC=23.0%
						78				
						110				
						110				
30						101				
						153				
						154				
						128				
						133				
35						127				
	4D	24/8	36.0 - 38.0	6/6/7/6	13	109			Light brown, wet, medium dense, fine to medium SAND, trace coarse sand and silt.	G#175879 A-3, SP WC=20.3%
						135				
						146				
						181				
40						161				
						144				
						149				
						145				
						168				
45						152				
	5D	24/12	46.0 - 48.0	3/3/4/4	7	152			Light brown, wet, loose, fine to medium SAND, trace coarse sand and silt.	G#175880 A-3, SP WC=18.5%
						135				
						152				
50						124	214.90		49.0	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Naples Bay Bridge Over Chutes River	**Location:**	US Route 302 Naples, Maine	**Boring No.:**	BB-NBB-201
				PIN:	11060.00
Driller:	MaineDOT	**Elevation (ft.)**	263.9	**Auger ID/OD:**	N/A
Operator:	Ervin Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/21/06-3/22/06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
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50						126				
						146				
						162				
						143				
55						157				
						158				
	6D	6/2	56.0 - 56.5	16/50(0)	---	180			Grey brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, grey rock in nose of spoon.	G#175881 A-1-b, SW-SM WC=13.7%
	R1	60/60	57.3 - 62.3	RQD = 94%		a36 NQ	206.60		a36 blows for 0.3'. 57.3	
									Bedrock: White, grey, black, medium grained, hard, GRANITE. R1:Core Times (min:sec) 57.3-58.3' (3:43) 58.3-59.3' (3:13) 59.3-60.3' (2:20) VOID (?) at 60.0' bgs. 60.3-61.3' (2:50) 61.3-62.3' (2:06) 100% Recovery	
60										
							201.60		62.3	
									Bottom of Exploration at 62.30 feet below ground surface.	
65										
70										
75										

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State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Naples

Project Number: 11060.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-NBB-101, 4D	104+11.4	10.6 Rt.	20.0-22.0	176585	1	22.5			SP-SM	A-3	0
BB-NBB-101, 6D	104+11.4	10.6 Rt.	30.0-32.0	176586	1	21.9			SM	A-2-4	II
BB-NBB-101, 8D	104+11.4	10.6 Rt.	40.0-42.0	176587	1	20.9			SP-SM	A-3	0
BB-NBB-101, 10D	104+11.4	10.6 Rt.	50.0-52.0	176588	1	23.7			SP-SM	A-3	0
BB-NBB-201, 1D	104+82.2	2.5 Lt.	6.0-8.0	175876	3	17.7			SP	A-1-b	0
BB-NBB-201, 2D	104+82.2	2.5 Lt.	16.0-18.0	175877	3	14.3			SP-SM	A-3	0
BB-NBB-201, 3D	104+82.2	2.5 Lt.	26.0-28.0	175878	3	23.0			SP	A-3	0
BB-NBB-201, 4D	104+82.2	2.5 Lt.	36.0-38.0	175879	3	20.3			SP	A-3	0
BB-NBB-201, 5D	104+82.2	2.5 Lt.	46.0-48.0	175880	3	18.5			SP	A-3	0
BB-NBB-201, 6D	104+82.2	2.5 Lt.	56.0-56.5	175881	3	13.7			SW-SM	A-1-b	0
BB-NBB-102, 3D	105+39.5	9.4 Lt.	15.0-17.0	176582	2	14.6			SM	A-2-4	II
BB-NBB-102, 7D	105+39.5	9.4 Lt.	35.0-37.0	176583	2	22.1			SM	A-2-4	II
BB-NBB-102, 10D	105+39.5	9.4 Lt.	50.0-52.0	176584	2	15.4			SW	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.

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

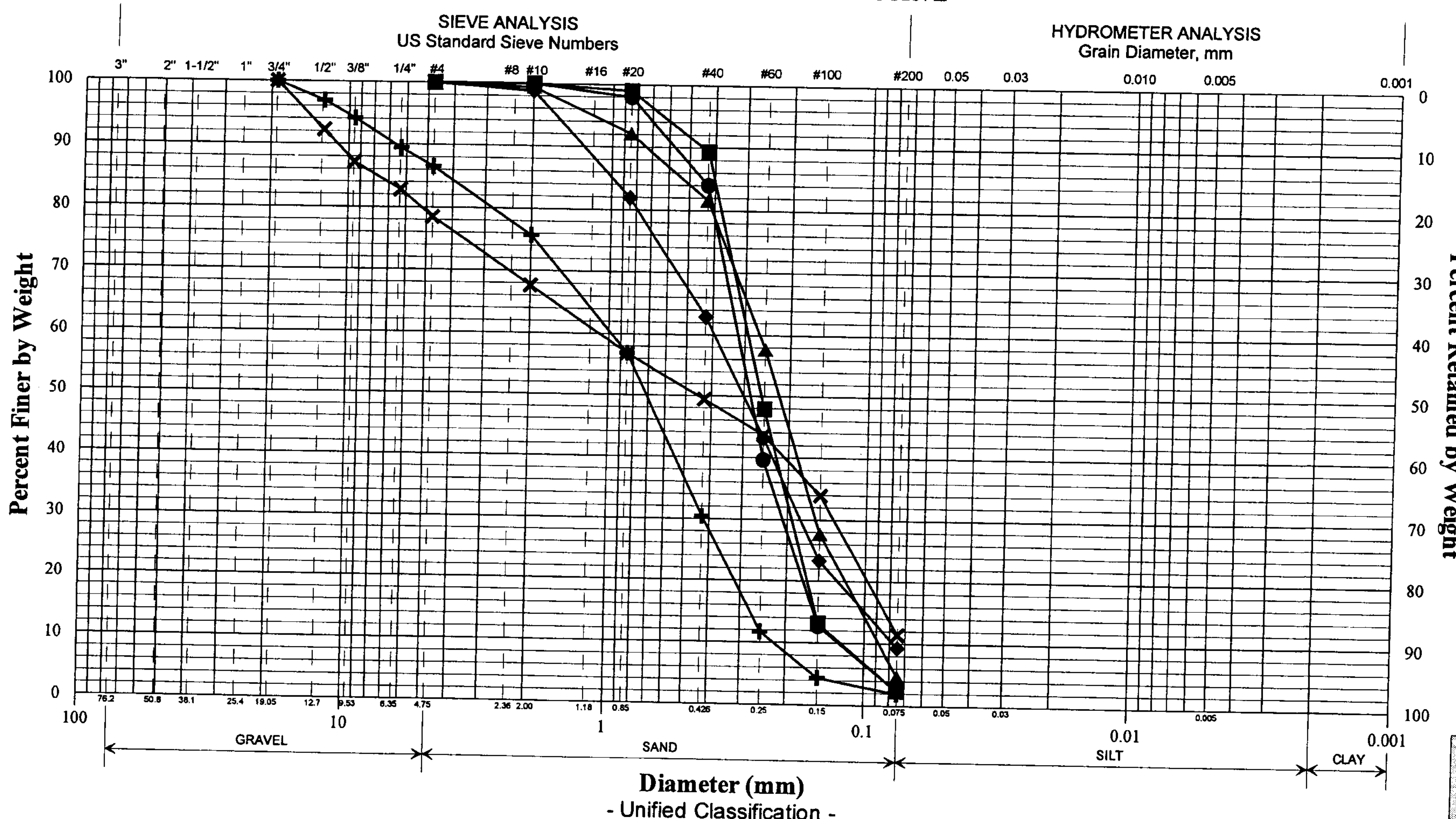
PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

SHEET NO. 3

PIN:	011060.00
Town:	Naples
Reported by:	WHITE, TERRY A
Date:	4/14/2006

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NBB-201/1D	104+82.2	2.5 LT	6.0-8.0	SAND, little gravel, trace silt.	17.7			
◆	BB-NBB-201/2D	104+82.2	2.5 LT	16.0-18.0	SAND, trace silt.	14.3			
■	BB-NBB-201/3D	104+82.2	2.5 LT	26.0-28.0	SAND, trace silt.	23.0			
●	BB-NBB-201/4D	104+82.2	2.5 LT	36.0-38.0	SAND, trace silt.	20.3			
▲	BB-NBB-201/5D	104+82.2	2.5 LT	46.0-48.0	SAND, trace silt.	18.5			
✖	BB-NBB-201/6D	104+82.2	2.5 LT	56.0-56.5	SAND, some gravel, little silt.	13.7			