MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

SPRUCE HEAD BRIDGE
OVER ATLANTIC OCEAN
SPRUCE HEAD ISLAND ROAD
SOUTH THOMASTON, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement of the Spruce Head Bridge on Spruce Head Island Road over the Atlantic Ocean in South Thomaston, Maine. The proposed replacement structure is a 100-foot single span concrete superstructure founded on semi-integral abutments supported on driven H-piles. The piles will be sleeved with a steel casing within the existing causeway granite blocks. The existing causeway will remain in place. Precast Concrete Modular Gravity walls will be constructed along the causeway on all four corners of the bridge parallel to the roadway to retain the widened roadway section. The roadway grade will be raised approximately 3 feet at the bridge location. The following design recommendations are discussed in detail in the attached report:

Semi-integral Abutment H-piles - The piles should be end bearing on or within the bedrock. The existing granite block causeway materials at the site will require rock coring and drilling for the installation of steel casings prior to driving the piles. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads. Piles should be installed with their weak axis perpendicular to the center line of the beams. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips and improve penetration. The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored strength limit state loads. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. One pile at each abutment should be dynamically tested to confirm resistance and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.52. The factored pile load should be shown on the plans.

Pile Protection and Scour - For pile protection and to facilitate pile installation, the H-piles supporting the semi-integral abutments should be sleeved with a steel casing within the existing granite block causeway. The sleeves should be sized to accommodate the design pile section with a minimum 6 inches of clearance around the piles. After the H-piles are driven and tested, the annular space between the sleeve and the H-pile should be filled with crushed stone, crushed gravel or concrete. The H-piles can be painted with a corrosion resistant coating applied according to the Manufacturer’s recommendations. Since the proposed bridge design will rely on the existing causeway granite blocks to provide scour protection for the semi-integral abutment and piles, causeway construction and block placement are of critical importance. The condition of the existing granite block causeway should be improved. The Project Plan Notes should include repairing and patching areas of the causeway where blocks have moved or cracked. The interface contact of the bottom course of granite blocks and ocean floor should be examined and improved, if necessary. Contract Documents should include a contingency item for injection grouting at the toe of the existing causeway if any portion is undermined or compromised. The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength, service and extreme limit states. These changes in foundation conditions shall be investigated at the abutments.

Semi-integral Stub Abutments - Semi-integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. The design of pile supported abutments at the strength limit state shall consider structural failure of the reinforced concrete abutment and backwall and the structural and geotechnical resistance of the pile group. Strength limit state design shall also consider foundation resistance after scour due to the design flood. Abutment design at the service limit state shall include: horizontal movement, overall stability and scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination. Extreme limit state design checks for abutments supported on piles shall include bearing resistance, pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the design flood can support the unfactored strength limit state loads. The unfactored strength limit state loads include any debris loads occurring during the flood event.

Semi-integral abutments that are rigid should typically be designed for active earth pressure over the abutment height and a uniform pressure distribution due to the height of soil behind the superstructure. The superstructure backwall should typically be designed for full passive pressure only. However, it is anticipated that these abutments will perform more like integral structures with forces and displacements into the approach fills. Therefore, the Designer should design the abutment stem wall and diaphragm to withstand a passive earth pressure state. Coulomb passive earth pressure state or log spiral approximation should be assumed. Additional lateral earth pressure due to construction surcharge or live load surcharge is required for the abutments. Use of an approach slab is required. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads on abutments is permitted. All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment. The approach slab should be underlain by 2 layers of 4 to 6 mil polyethylene sheets to minimize friction against horizontal movement of the superstructure backwall.

Compressible Inclusion Behind Semi-Integral Abutments - During design phase of the Spruce Head Bridge the use of a compressible inclusion behind the semi-integral abutments was discussed. An internet search found two products available from GeoTech Systems Corporation of Great Falls, Virginia: TerraFlex™ and Geoinclusion®. These products are both designed specifically for this type of application. When installed behind an integral bridge abutment, they provide a compressible layer which can accommodate abutment movement. Geoinclusion® also serves as a drainage medium eliminating hydrostatic pressure behind the abutments. The use of these products or any other similar product should be done in accordance with the Manufacturer's requirements and with the participation of the MaineDOT Research Section.

Settlement - The grades of the existing bridge approaches will be raised in order to accommodate the change in horizontal alignment of the proposed bridge. Additionally, the roadway will be widened to both sides at both abutments. The maximum fill to be placed at the site is approximately 8.0 feet and will result in less than 1 inch of settlement. This settlement is anticipated to occur over a period of years but should have minimal effect of the finished structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

Frost Protection – Any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection.

Seismic Design Considerations - Seismic analysis is not required for single-span bridges regardless of seismic zone. Spruce Head Bridge is not on the National Highway System. The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be satisfied.

Precast Concrete Modular Block Retaining Wall - Precast Concrete Modular Gravity retaining walls of varying lengths and heights will be constructed along the causeway on all four corners of the bridge to retain the widened roadway section and minimize impacts. These walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD and Special Provision 635. Bearing resistance for PCMG walls founded on a leveling slab on native silt or fill soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 10 ksf for wall system bases less than 8 feet wide and 11 ksf for bases from 8.5 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 4 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing. For footings on soil the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth ($1/4^{th}$) of the footing dimensions in either direction. The PCMG wall shall consist of Class "LP" concrete and epoxy coated rebar. The precast concrete units shall contain a minimum of 5.5 gallons per cubic yard of calcium nitrate solution or equivalent corrosion inhibitor. The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635.

Construction Considerations - The installation of any pile through the existing granite block causeway will require rock coring and drilling. Overburden drilling methods or other methods approved by the Resident will be required to install steel casings prior to installing the piles.

1.0 INTRODUCTION

A subsurface investigation for the replacement of Spruce Head Bridge on Spruce Head Island Road over the Atlantic Ocean in South Thomaston, Knox County, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

Spruce Head Bridge was built in 1955 and is a 127 foot long, 5-span, steel and concrete superstructure supported on concrete stub abutments and stub piers placed on a granite block causeway which extends under the bridge deck leaving only the main span clear. The two center piers were widened in 1989. At that time a 3-pile group was driven on each side of the existing pier cap and the cap was extended to encase the tops of the piles. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck is in “poor” condition (rating of 4), the bridge superstructure is in “poor” condition (rating of 4) and the substructure is in “satisfactory” condition (rating of 6). Year 2008 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 25.3.

The proposed replacement structure is a 100-foot single span concrete superstructure founded on semi-integral abutments supported on driven H-piles. The piles will be sleeved with a steel casing within the existing causeway granite blocks. The existing causeway will remain in place. Precast Concrete Modular Gravity walls will be constructed along the causeway on all four corners of the bridge parallel to the roadway to retain the widened roadway section. The roadway grade will be raised approximately 3 feet at the bridge location.

2.0 GEOLOGIC SETTING

Spruce Head Bridge carries Spruce Head Island Road over the Atlantic Ocean from the mainland in South Thomaston, Maine to Spruce Head Island as shown on Sheet 1 - Location Map found at the end of this report.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits. Soils in the site area are generally comprised of silt, clay, sand and minor amounts of gravel. Sand is dominant in some areas, but may be underlain by finer-grained sediments. The unit contains small areas of till that are not completely covered by marine sediments. The unit generally is deposited in areas where the topography is gently sloping except where dissected by modern streams and commonly has a branching network of steep-walled stream gullies. These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the site bedrock is identified as Devonian muscovite-biotite granite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling two (2) test borings at the site. Test boring BB-STFH-101 was drilled behind the location of Abutment No. 1 (north) and BB-STFH-102 was drilled behind the location of Abutment No. 2 (south). The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the site stratigraphy at the bridge location is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The borings were drilled between March 25 and April 2, 2009 using the Maine Department of Transportation (MaineDOT) drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on Sheet 4 - Boring Logs found at the end of this report.

The borings were drilled using rock coring and driven cased wash boring techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated February of 2009. The MaineDOT automatic hammer was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor to the raw field N-values. The hammer efficiency factor (0.84) and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in the borings using an NQ core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT geotechnical team member and/or a Certified Subsurface Inspector selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field testing requirements. A Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion if the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of four (4) grain size analyses with hydrometer and two (2) Atterberg Limits tests. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included in Appendix A – Boring Logs and on Sheet 4 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the test borings generally consisted of granite block fill, with occasional layers of sand underlain by clayey silt all underlain by bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 – Interpretive Subsurface Profile found at the end of this report. The following paragraphs are a brief summary description of the strata encountered during exploration activities:

Granite Block/Causeway Fill. The existing bridge superstructure spans extensions of the causeway which fill the area between the abutments and the piers on both sides of the bridge main span. These causeway extensions are constructed of stacked, cut, granite blocks. The borings at both abutments were drilled through the granite blocks using rock core and roller cone drilling techniques. The granite blocks were found to be approximately 26.5 feet thick in boring BB-STFH-101 and approximately 25.0 feet thick in boring BB-STFH-102. A layer of silty, fine to medium sand was encountered within the granite blocks in boring BB-STFH-101 at a depth of 17.0 feet below ground surface. The layer was approximately 2.0 feet thick. One SPT N-value in the sand was 10 blows per foot (bpf) indicating that the sand is loose in consistency.

Clayey Silt. The granite block/causeway fill is underlain by a layer of clayey silt in both of the borings. The clayey silt layer was found to be approximately 12.1 feet thick in boring BB-STFH-101 and approximately 12.5 feet thick in boring BB-STFH-102. The layer can be described as brown to dark grey, wet, clayey silt, with trace sand and trace gravel. SPT N-values in the clayey silt ranged from 14 to 78 bpf indicating that the clayey silt is stiff to very stiff in consistency. Water contents from four (4) samples obtained within this layer ranged from approximately 20% to 24%. Four (4) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-4 or A-6 by the AASHTO Classification System and a CL by the Unified Soil Classification System.

Table 1 summarizes the results of the Atterberg Limits test made from two (2) samples of the clayey silt:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-STFH-101 R5	Clayey Silt	22.8	34	18	16	0.30
BB-STFH-101 2D/B	Clayey Silt	23.1	27	17	10	0.61

Table 1 – Summary of Atterberg Limits Testing

Interpretation of these results indicates the clayey silt has a water content that falls between the liquid limit and plastic limit and liquidity indices of less than 1 indicating soils which are over consolidated.

Bedrock. Bedrock was encountered and cored in both of the borings. Table 2 below presents the bedrock findings:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB-STFH-101 Abutment No. 1	38.6 feet	-28.18 feet	53 – 74%
BB-STFH-102 Abutment No. 2	37.5 feet	-26.92 feet	78 – 87%

Table 2 – Summary of Bedrock Depths, Elevations and RQD

The bedrock is described as grey, white, black and pink, medium grained granite with iron staining in the upper 2 feet. The RQD of the bedrock ranges from 53 to 87 percent indicating a rock mass quality of fair to good.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered during the subsurface exploration program, the following foundation alternatives may be considered for the bridge replacement:

- Integral or semi-integral abutments supported on driven H-piles
- Abutments supported on small diameter drilled shafts
- Integral or semi-integral abutments supported on pipe piles
- Integral or semi-integral abutments supported on precast concrete piles

The installation of any pile through the existing granite block causeway will require rock coring and drilling to install steel casings to hold open the holes for pile installation. As this bridge is constructed within a granite block causeway in a tidal area, the abutment piles will be exposed to salt water and marine conditions at all times. There will be corrosion of steel piles over the life of the structure. Therefore, sleeveing the piles within the causeway fill material is required.

Discussions during the design phase of the project resulted in the decision to use semi-integral abutments supported on driven H-piles which will be sleeved within the granite block causeway to support the proposed replacement structure. Geotechnical recommendations addressed in this report will pertain only to the chosen foundation alternative.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The proposed replacement structure will be a 100-foot single span, concrete superstructure founded on sleeved H-pile supported semi-integral abutments. The existing granite block causeway will remain in place. The piles will be sleeved with a steel casing within the existing granite block causeway fill. Precast Concrete Modular Gravity (PCMG) walls will be constructed along the causeway on all four corners of the bridge to retain the widened roadway section. The roadway grade will be raised approximately 3 feet at the bridge location.

7.1 H-Pile Supported Semi-integral Abutments

The use of stub abutments founded on a single row of driven semi-integral H-piles is a viable foundation system for use at the site. The piles should be end bearing on or within the bedrock. The existing granite block causeway materials at the site will require rock coring and drilling for the installation of steel casings prior to driving the piles. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads. Piles should be installed with their weak axis perpendicular to the center line of the beams. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on the data in Table 3 below:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length
Abutment No. 1 BB-STFH-101	2.5 feet	38.6 feet	-28.18 feet	31 feet
Abutment No. 2 BB-STFH-102	2.5 feet	37.5 feet	-26.92 feet	30 feet

Table 3 - Estimated Pile Lengths for H-Piles

These pile lengths do not take into account the additional five (5) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 7.1.1 below.

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0. The design flood scour is defined in AASHTO LRFD Bridge Design Specifications 4th Edition (LRFD) Articles 2.6.4.4.2 and 3.7.5.

Since the abutment piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. An L-Pile® analysis is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, movements and pile head displacements. Achievement of an assumed pinned condition at the pile tip should also be confirmed with an L-Pile® analysis.

7.1.1 Strength Limit State

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances. Preliminary estimates of the factored structural axial compressive resistances of the five proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (good driving conditions) and a λ of 0.

The nominal geotechnical compressive resistances of the H-pile sections in the strength limit state were calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the five proposed H-pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45.

The drivability of the five proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, are limited to less than 45 ksi. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65. Table 10.5.5.2.3-3 requires that no less than three to four dynamic tests be conducted for sites with low to medium variability. As it is likely that only two dynamic tests will be conducted at the site, this resistance factor has been reduced by 20% resulting in a ϕ_{dyn}=0.52.

The calculated factored axial compressive structural, geotechnical and drivability resistances of the five proposed H-pile sections for each abutment are summarized in Table 4 below. Supporting calculations are included in Appendix C- Calculations found at the end of this document.

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
HP 12 x 53	465	354	255	354
HP 12 x 74	654	494	277	494
HP 14 x 73	642	446	276	446
HP 14 x 89	783	542	341	542
HP 14 x 117	1032	710	537	710

*based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

Table 4 – Factored Axial Resistances for Abutment Piles at the Strength Limit State

LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the factored geotechnical resistance shown in Table 4 above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.70 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2.1-1 or -2).

7.1.2 Service Limit State

For the service limit state resistance factors of 1.0 are recommended for structural and geotechnical pile resistances. For preliminary analysis, the H-piles were assumed fully embedded and λ was taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances.

The calculated factored axial structural and geotechnical resistances of the five proposed H-pile sections for each abutment are summarized in Table 5 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pile Section	Service Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
HP 12 x 53	775	786	491	775
HP 12 x 74	1090	1098	533	1090
HP 14 x 73	1070	991	531	991
HP 14 x 89	1305	1204	655	1204
HP 14 x 117	1720	1578	1033	1578

*based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

Table 5 - Factored Axial Resistances for Abutment Piles at the Service Limit State

LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. It is recommended that the governing resistance used in service limit state design be the resistances shown in the last column of Table 5 above. It should be noted that the factored structural resistance governs for the HP 12x53 and HP 12x74 pile sections while the remaining pile sections are governed by the factored geotechnical resistance.

7.1.3 Extreme Limit State

For the extreme limit state resistance factors of 1.0 are recommended for structural, geotechnical and drivability pile resistances. For preliminary analysis, the H-piles were assumed to have an unbraced length from the bottom of the pile cap to the bottom of the existing granite block causeway and λ was calculated for each pile section. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances.

The calculated factored axial structural and geotechnical resistances of the five proposed H-pile sections for each abutment are summarized in Table 6 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pile Section	Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
HP 12 x 53	599	786	491	599
HP 12 x 74	849	1098	533	849
HP 14 x 73	883	991	531	883
HP 14 x 89	1080	1204	655	1080
HP 14 x 117	1431	1578	1033	1431

Table 6 - Factored Axial Resistances for Abutment Piles at the Extreme Limit State

LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. As the factored axial structural resistance is less than the factored axial geotechnical resistance, it is recommended that the maximum factored axial pile load used in design for the extreme limit state should not exceed the factored structural resistance shown in Table 5 above.

7.1.4 Estimated Depth to H-Pile Fixity

Stability of the piles shall be evaluated in accordance with the provisions in LRFD Article 6.9 using an equivalent pile length that accounts for the laterally unsupported length of the pile plus the embedment depth to fixity. It is anticipated that the abutments will be protected by the existing causeway to remain in place. Historically, there have been no major scour issues at the site and the existing causeway has proven to be adequate. Therefore, no unsupported length of pile needs to be considered in the evaluation of pile fixity.

Preliminary depths to fixity for the five proposed H-pile sections were calculated, assuming only axial loading and without consideration of lateral loads (LRFD Article 10.7.3.13.4). Table 7 below summarizes the calculated depths to fixity for the five proposed H-pile sections. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

H-pile Section	Preliminary Estimates of Depth to Fixity w/ no lateral loads applied
HP 12 x 53	8 feet
HP 12 x 74	9 feet
HP 14 x 73	10 feet
HP 14 x 89	10 feet
HP 14 x 117	11 feet

Table 7 - Preliminary Estimates of Depth to Fixity

In general it is recommended that piles be designed to achieve a fixed condition at the pile toe. When the lateral and axial pile load groups are known, this data should be provided to the geotechnical engineer. A more refined analysis of pile fixity can then be performed using L-Pile® software.

7.1.5 Pile Resistance and Pile Quality Control

In order to verify the resistance of the installed H-pile, the pile shall be dynamically tested. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. One pile each at abutment should be dynamically tested to confirm resistance and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.52. The factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 8 to 13 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Buckling and Combined Axial and Flexure

Pile group design shall consider loading effects due to combined axial and flexural loading, as outlined in LRFD Article 6.15. For a pile group composed of only vertical piles which is subjected to lateral loads, the pile structural analysis shall include consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.9. The recommended design approach considers the non-linear response of soil with lateral displacement. Soil-structure interaction considering the non-linear response of soil can be modeled using L-Pile® computer software.

The factored structural resistances for pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural engineer.

7.3 Pile Protection and Scour

For pile protection and to facilitate pile installation, the H-piles supporting the semi-integral abutments should be sleeved with a steel casing within the existing granite block causeway. The sleeves should be sized to accommodate the design pile section with a minimum 6 inches of clearance around the piles. After the H-piles are driven and tested, the annular space between the sleeve and the H-pile should be filled with crushed stone, crushed gravel or concrete. The H-piles can be painted with a corrosion resistant coating applied according to the Manufacturer's recommendations.

Since the proposed bridge design will rely on the existing causeway granite blocks to provide scour protection for the semi-integral abutment and piles, causeway construction and block placement are of critical importance. The condition of the existing causeway should be improved. The Project Plan Notes should include repairing and patching areas of the causeway where blocks have moved or cracked. The interface contact of the bottom course of granite blocks and ocean floor should be examined and improved, if necessary.

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength, service and extreme limit states. These changes in foundation conditions shall be investigated at the abutments. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

7.4 Semi-integral Stub Abutments

Semi-integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported abutments at the strength limit state shall consider structural failure of the reinforced concrete abutment and backwall and the structural and geotechnical resistance of the pile group. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

A resistance factor of $\phi = 1.0$ shall be used to assess abutment design at the service limit state including: horizontal movement, overall stability and scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Semi-integral abutments that are rigid should typically be designed for active earth pressure over the abutment height and a uniform pressure distribution due to the height of soil behind the superstructure. The superstructure backwall should typically be designed for full passive pressure only. However, it is anticipated that these abutments will perform more like integral structures with forces and displacements into the approach fills. Therefore, the Designer should design the abutment stem wall and diaphragm to withstand a passive earth pressure state. Coulomb passive earth pressure state or log spiral approximation (LRFD Figure 3.11.5.4-1) should be assumed. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, of 0.307 is recommended. In designing for passive earth pressure, the Coulomb state is recommended. However, consideration may be given to using a reduced Coulomb passive earth pressure if the expected displacement of the abutment is significantly less than that required to mobilize full passive pressure (0.01 times the height of the abutment).

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the abutments. Use of an approach slab is required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. The approach slab should be underlain by 2 layers of 4 to 6 mil polyethylene sheets to minimize friction against horizontal movement of the superstructure backwall. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads on abutments is permitted per

LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 8 below:

Abutment Height	h_{eq}
5 feet	4.0 feet
10 feet	3.0 feet
≥20 feet	2.0 feet

Table 8 – Equivalent Height of Soil for Vehicular Loading on Abutments

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

All abutment 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 MaineDOT BDG. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

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

7.5 Compressible Inclusion Behind Semi-Integral Abutments

As discussed in "Integral-Abutment Bridges: Geotechnical Problems and Solutions Using Geosynthetics and Ground Improvement" by John S. Horvath, Ph.D., P.E. (2005), the use of integral and semi-integral abutment designs has its own inherent post-construction flaws. Integral bridge abutments experience stresses due to the cyclic expansion and contraction of the bridge deck, caused by daily and seasonal temperature variations, pushing the abutments into and out of the bridge embankment. The cyclic expansion can develop into large lateral earth pressures on the abutments as the expansion of the superstructure occurs. The lateral earth pressures increase over time as the soil behind each abutment becomes increasingly wedged in. This ratcheting up of the earth pressures over time represents a long term problem source. Secondly, a subsidence pattern develops behind the abutments which can develop into an irreversible soil-wedge slump.

The use of a compressible inclusion directly behind the abutment backwall can help to alleviate the build up of stresses. Horvath states that the presence of a compressible inclusion behind an integral or semi-integral abutment is totally ineffective for controlling subsidence and may even exacerbate the subsidence problem.

During design phase of the Spruce Head Bridge the use of a compressible inclusion behind the semi-integral abutments was discussed. An internet search found two products available from GeoTech Systems Corporation of Great Falls, Virginia: TerraFlex™ and

Geoinclusion®. These products are both designed specifically for this type of application. When installed behind an integral bridge abutment, they provide a compressible layer which can accommodate abutment movement. Geoinclusion® also serves as a drainage medium eliminating hydrostatic pressure behind the abutments.

The use of these products or any other similar product should be done in accordance with the Manufacturer's requirements and with the participation of the MaineDOT Research Section.

7.6 Settlement

The grades of the existing bridge approaches will be raised in order to accommodate the change in horizontal alignment of the proposed bridge. Additionally, the roadway will be widened to both sides at both abutments. The maximum fill to be placed at the site is approximately 8.0 feet and will result in less than 1 inch of settlement. This settlement is anticipated to occur over a period of years but should have minimal effect of the finished structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

7.7 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT frost depth maps for the State of Maine (MaineDOT BDG Figure 5-1) the site has a design-freezing index of approximately 1100 F-degree days. This correlates to a frost depth of 5.0 feet. The design frost depth was also calculated according to the US Army Corps Cold Regions Research and Engineering (USACE CRREL) Modberg computer program. According to the CRREL Modberg program, the site has a design freezing index of 1188 F-degree days. A water content of 20% from laboratory testing was used for the soils. These components correlate to a frost depth of 5.6 feet. Experience has shown that embedment for frost protection to a depth of 5.0 feet is adequate for protection of structures in the site area.

Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG. See Appendix C- Calculations at the end of this report for supporting documentation.

7.8 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the Maine DOT BDG, Spruce Head Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak Ground Acceleration coefficient (PGA) = 0.058g
- Short-term (0.2-second period) spectral acceleration coefficient (S_{DS}) = 0.320g
- Long-term (1.0-second period) spectral acceleration coefficient (S_{D1}) = 0.139g
- Site Class E (site soils with an average N-value less than 15 bpf or an undrained shear strength less than 1000 psf)
- Seismic Zone 1 (based on S_{D1} less than or equal to 0.15g)

See Appendix C- Calculations at the end of this report for supporting documentation.

7.9 Precast Concrete Modular Block Retaining Wall

Precast Concrete Modular Gravity (PCMG) walls of varying lengths and heights will be constructed along the causeway on all four corners of the bridge to retain the widened roadway section and minimize impacts. These walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD and Special Provision 635 which is included in Appendix D found at the end of this report.

The PCMG wall designs shall consider a live load surcharge estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 9 below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic = 0 feet	Distance from wall backface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥20	2.0	2.0

Table 9 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls

Bearing resistance for PCMG walls founded on a leveling slab on native silt shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 10 ksf for wall system bases less than 8 feet wide and 11 ksf for bases from 8.5 to 12 feet wide. The bearing resistance factor, ϕ_b, for spread footings on soil is 0.45. Based on presumptive bearing resistance values a factored bearing resistance of 4 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

The bearing resistance for PCMG bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. The PCMG units shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor ϕ, of 0.65.

The designer shall apply a sliding resistance factor ϕ_T of 0.85 to the nominal sliding resistance of precast concrete wall segments founded on spread footings on clay. For footings on soil the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth ($1/4^{th}$) of the footing dimensions in either direction (LRFD Article 10.6.3.3). Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.36x(tan 20º) at the foundation soil to soil interface. Recommended values of sliding frictional coefficients are based on LRFD Article 11.11.4.2, Table 10.5.5.2.2-1 and Table 3.11.5.3-1.

The PCMG wall shall consist of Class "LP" concrete and epoxy coated rebar. The precast concrete units shall contain a minimum of 5.5 gallons per cubic yard of calcium nitrate solution or equivalent corrosion inhibitor.

The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635.

7.10 Construction Considerations

The installation of any pile through the existing granite block causeway will require rock coring and drilling for the installation of steel casings prior to driving the piles. As this bridge is constructed within a granite block causeway in a tidal area, the abutment piles will be exposed to salt water and marine conditions at all times. There will be corrosion of steel piles over the life of the structure. Therefore, sleeveing the piles within the granite block causeway is rerquired. The sleeves should be sized to accommodate the design pile section with a minimum 6 inches of clearance around the piles. After the H-piles are driven and tested, the annular space between the sleeve and the H-pile should be filled with crushed stone or crushed gravel.

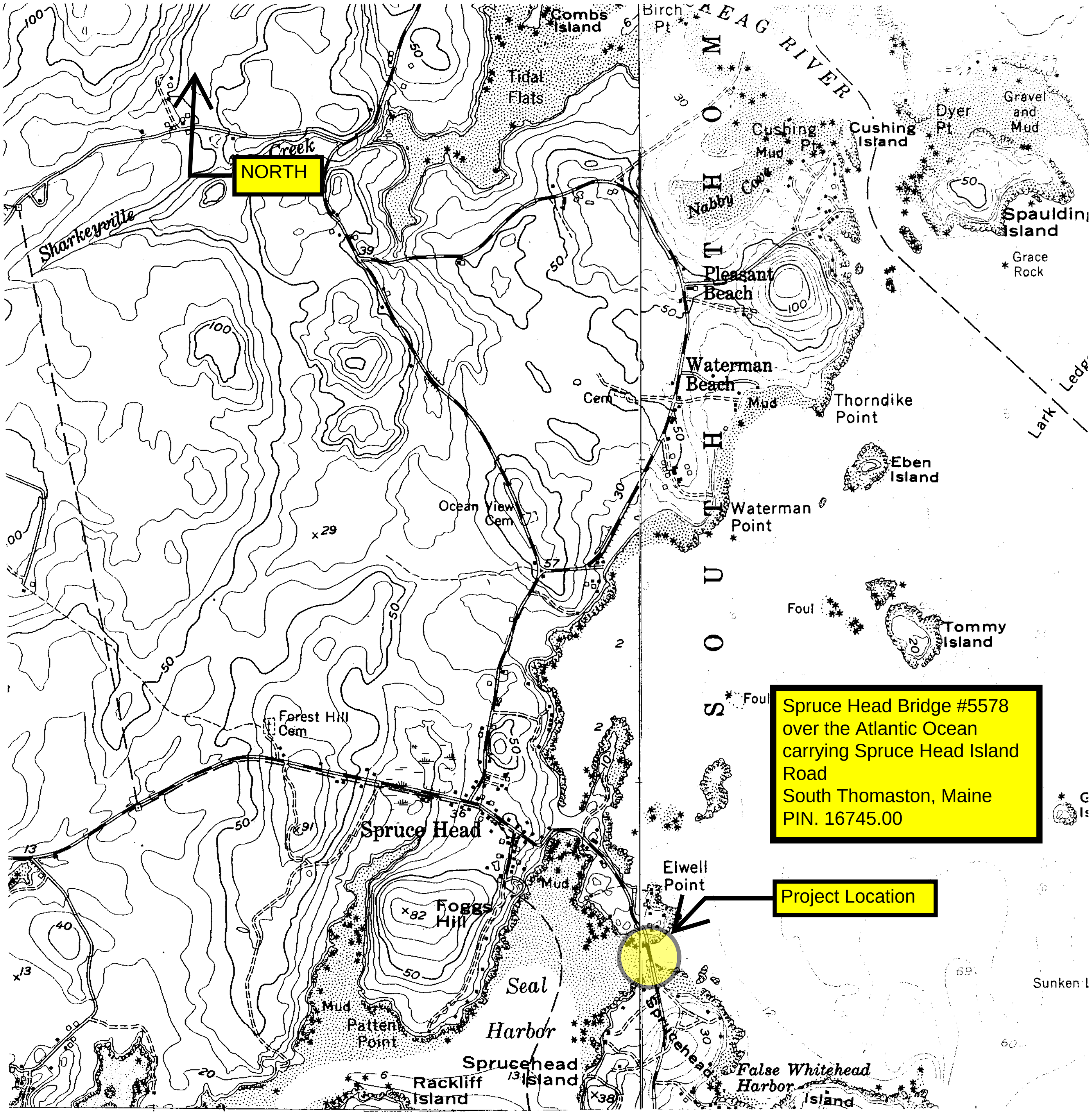
Since the proposed bridge design will rely on the existing causeway granite blocks to provide scour protection for the semi-integral abutment and piles, causeway construction and block placement are of critical importance. The condition of the existing causeway should be improved. The Project Plan Notes should include repairing and patching areas of the causeway where blocks have moved or cracked. The interface contact of the bottom course of granite blocks and ocean floor should be examined and improved, if necessary.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Spruce Head Bridge in South Thomaston, 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



THOMASTON QUADRANGLE
MAINE–KNOX CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
SW/4 ROCKLAND 15' QUADRANGLE

ROCKLAND QUADRANGLE
MAINE–KNOX CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24 000

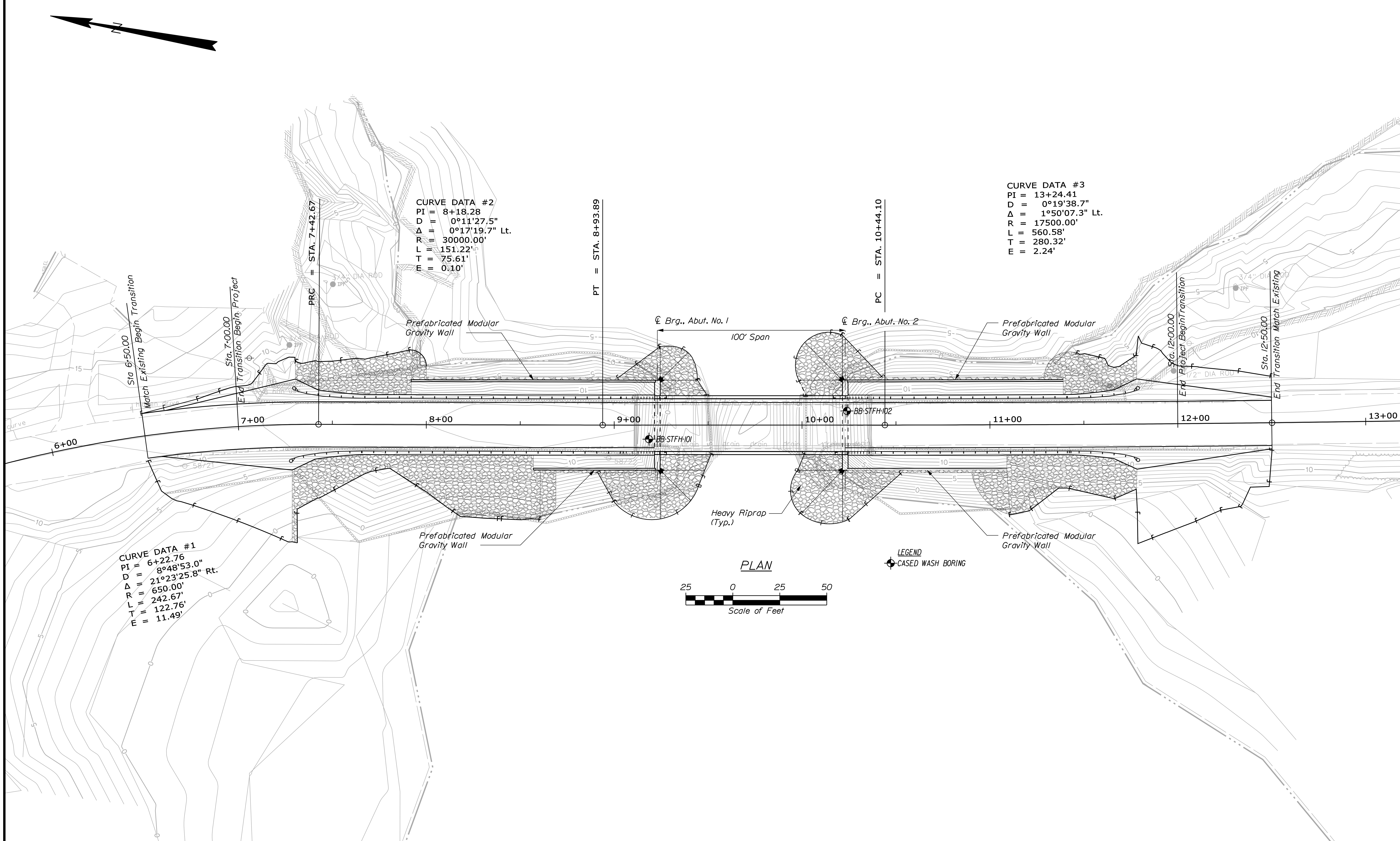
1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

CONTOUR INTERVAL 10 FEET

Filename: ...\00\GEOTECH\MSTA\002_BLP1.dgn   Division: GEOTECH   Username: terry.white   Date:5/26/2009



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1674(500)S

PIN 16745.00

BRIDGE NO. 5578

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	J.D.W.	
DESIGN-DETAILED	K.MAGUIRE	
CHECKED-REVIEWED	T. WHITE	APR 2009
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

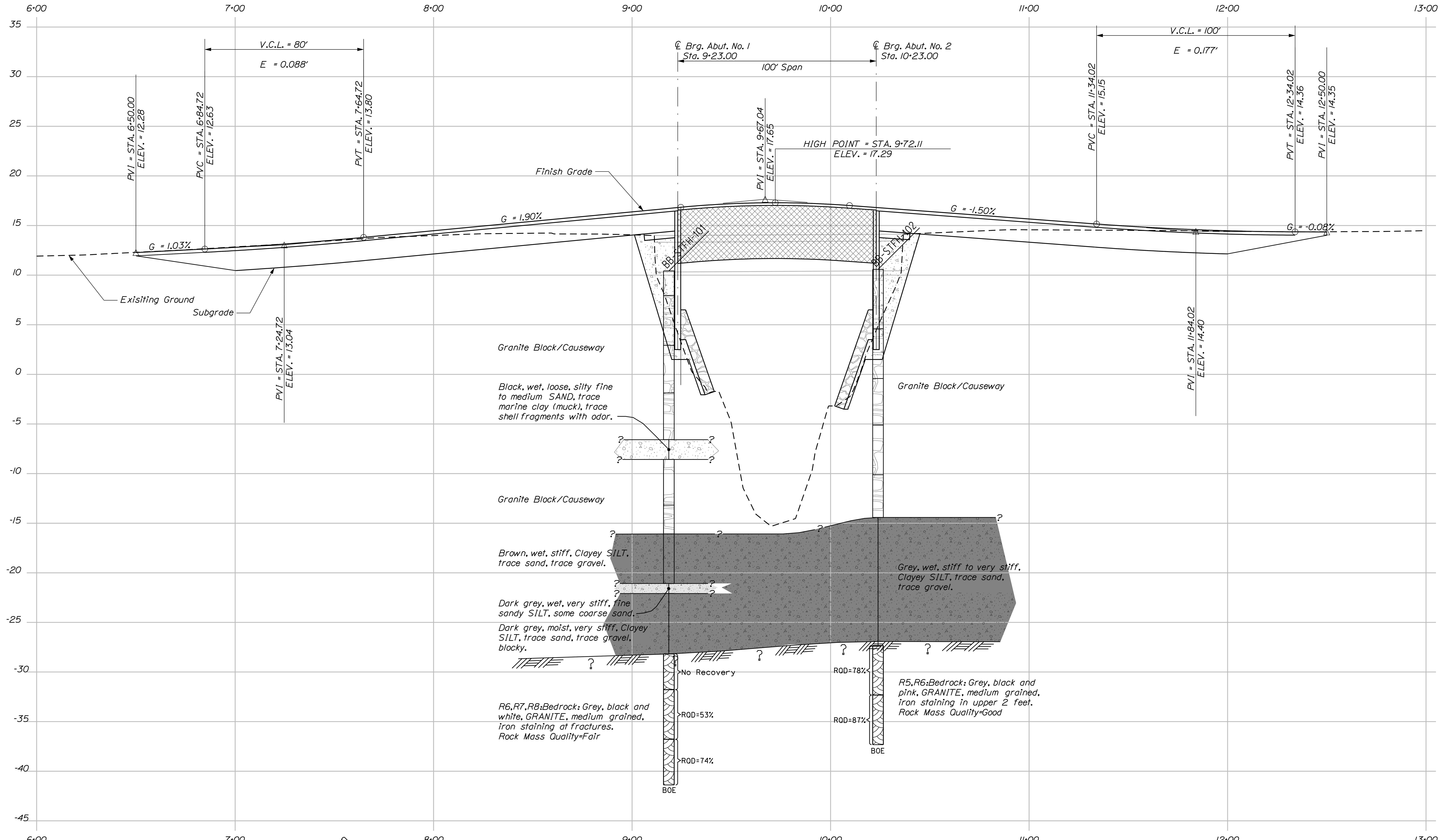
DATE

SPRUCE HEAD BRIDGE
ATLANTIC OCEAN
SOUTH THOMASTON KNOX COUNTY

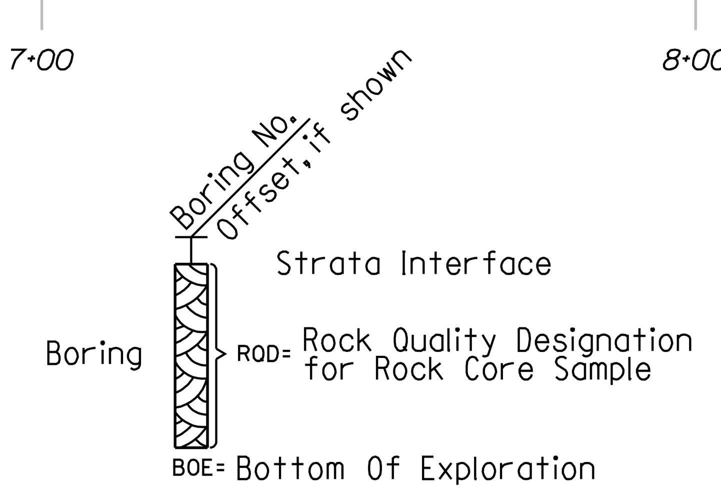
BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\00\geotech\msta\003_ISP1.dgn Division: GEOTECH Username: terry.white Date:5/26/2009



PROFILE



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

BR-1674(500)S

BRIDGE PLANS

PIN 16745.00

BRIDGE NO. 5578

	J.D.W.	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	K.MAGUIRE	T. WHITE	APR 2009
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

SPRUCE HEAD BRIDGE
ATLANTIC OCEAN
SOUTH THOMASTON KNOX COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\msta\004_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:5/26/2009

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Spruce Head Bridge #5578 over the Atlantic Ocean carrying Spruce
Location: South Thomaston, Maine

Boring No.: BB-STFH-101
PIN: 16745.00

Driller:	MaineDOT	Elevation (ft.)	10.42	Auger ID/OD:	N/A
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/25-27/09-3/30/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	9+18.5, 7.5 Rt.	Casing ID/OD:	NW	Water Level*:	Tidal
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	R1	30/14	0.00 - 2.50				NQ-2		Top of Granite Wall/Causeway. Grey and white, GRANITE pegmetite, coarse grained with mica. Spun NW Casing ahead to 23.6'. R1: Core Times average 5 minutes per foot in Granite blocks.	
2.5	R2	60/26	2.50 - 7.50						R2:Core Times average 6 minutes per foot in Granite Wall/Causeway. Grey, white and black, GRANITE, fine grained.	
5									Granite blocks.	
7.5							RC		Roller Coned ahead to 12.0' with large roller bit (3 7/8").	
10									Granite blocks.	
12	MD	4/0	12.00 - 12.33	30(3.6")	---		NQ-2		Failed sample attempt.	
12.3	R3	56.4/40	12.30 - 17.00						Grey and white, GRANITE pegmetite, coarse grained. R3:Core Times (min:sec) 12.3-13.3' (5:12) 13.3-14.3' (4:10) 14.3-15.3' (3:20) (6" drop) 15.3-16.3' (4:25) 16.3-17.0' (2:50) broke through	
17	1D	24/18	17.00 - 19.00	7/2/5/6	7	10		-6.58	17.00 Black, wet, loose, silty fine to medium SAND, trace marine clay (muck), trace shells fragments with odor.	
19	R4	54/28	19.00 - 23.50				NQ-2	-8.58	19.00 Grey and white, GRANITE pegmetite, coarse grained. Granite blocks and Cobbles. R4:Core Times (min:sec) 19.0-20.0' (4:37) 20.0-21.0' (3:25) 21.0-22.0' (2:25) 22.0-23.0' (1:16) 23.0-23.6' (0:10) Core Blocked	
23.6	R5	60/30	23.60 - 28.60				8 NQ-2		Grey and white, GRANITE pegmetite, coarse grained. Granite blocks and Cobbles. R5:Core Times (min:sec) 23.6-24.6' (1:20) 24.6-25.6' (1:40) 25.6-26.6' (1:15) 26.6-27.6' (1:20) 27.6-28.6' (1:25) Brown, Clayey SILT, trace sand, trace gravel in core barrel, roller coned ahead to 26.5' then drove casing.	G#210034 A-6, CL WC=22.8% LL=34 PL=18 PI=16
24.6							14			
25.6							22			
26.6							26			
27.6							41			
28.6							54			
29.6							137			
30.5							123			
31.5	2D/AB	24/24	31.50 - 33.50	8/11/4/7	15	21	113	-21.08	31.50 2D/A (31.5-32.5') Dark grey, wet, medium dense, fine sandy silt, some coarse sand.	
32.5							122	-22.08	32.50 2D/B (32.5-33.5') Dark grey, moist, very stiff, Clayey SILT, trace sand, trace gravel, blocky.	G#210035 A-4, CL WC=23.1% LL=27 PL=17 PI=10
33.5							127			
34.5							107			
35.5							116			
36.5	3D	24/2	36.50 - 38.50	26/29/27/23	56	78	106		Same as above, hard.	
37.5							43			
38.6	R6	42/0	38.60 - 42.10	No recovery			56	-28.18	38.60 R6:Core Times (min:sec) 38.6-39.6' (5:20) 39.6-40.6' (0:50) 40.6-41.6' (1:10) 41.6-42.2' (2:30) No Recovery.	
39.6							60			
40.6							42			
41.6							47			
42.2	R7	60/42	42.20 - 47.20	RQD = 53%			NQ-2		Grey, black and white, GRANITE, medium grained, iron staining at fractures. R7:Core Times (min:sec) 42.2-43.2' (3:44) 43.2-44.2' (7:10) 44.2-45.2' (4:08) 5" drop 45.2-46.2' (2:35) 4" drop 46.2-47.2' (5:25)	
47.2	R8	54/54	47.20 - 51.70	RQD = 74%					Grey, black and white, GRANITE, medium grained, iron staining at fractures. R8:Core Times (min:sec) 47.2-48.2' (3:45) 48.2-49.2' (5:03) 49.3-50.2' (5:07) 50.2-51.2' (5:24) 51.2-51.8' (10:00)	
51.8								-41.38	51.80 Bottom of Exploration at 51.80 feet below ground surface.	

Remarks:
12" thick concrete Bridge Deck at Elev. 13.92'.
Top of Granite Wall/Causeway at Elev. 10.42'.
3.5' from Bridge Deck to top of Granite Wall/Causeway.
Left 10 feet of casing in hole.

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

Page 1 of 1
Boring No.: BB-STFH-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Spruce Head Bridge #5578 over the Atlantic Ocean carrying Spruce
Location: South Thomaston, Maine

Boring No.: BB-STFH-102
PIN: 16745.00

Driller:	MaineDOT	Elevation (ft.)	10.58	Auger ID/OD:	N/A
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/31/09-4/02/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	10+23.9, 7.5 Lt.	Casing ID/OD:	NW	Water Level*:	Tidal
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	R1	72/60	0.00 - 6.00				NQ-2 NW		Top of Granite Wall/Causeway. Grey and white, GRANITE pegmatite, coarse grained. R1: Core Times average 6 minutes per foot in Granite Wall/Causeway.	
6	R2	60/46	6.00 - 11.00						Grey and white, GRANITE pegmatite, coarse grained. R2: Core Times average 7 minutes per foot in Granite Wall/Causeway.	
11	R3	42/20	11.00 - 14.50						Grey and white, GRANITE pegmatite, coarse grained. R3: Core Times average 7 minutes per foot in Granite Wall/Causeway. Core Blocked at 14.5' bgs.	
14.5									Roller Coned ahead with large roller from 14.5-15.7' bgs.	
15.7	R4	60/27	15.70 - 20.70						Grey and white, GRANITE pegmatite, coarse grained. R4(Core Times (min:sec) 15.7-16.7' (6:20) 16.7-17.7' (4:10) 4" drop 17.7-18.7' (2:10) 18.7-19.7' (4:20) 19.7-20.7' (0:58)	
19							14			
20							11		Broke casing, roller coned ahead to 25.0' with large roller bit. Dropped in NW Casing.	
21							12			
22							23			
23							24			
24							22			
25	1D	24/20	25.00 - 27.00	4/4/6/8	10	14	22	-14.42	25.00 Grey, wet, stiff, Clayey SILT, trace sand, trace gravel. Roller coned ahead to 30.0' with large roller bit.	G#209201 A-4, CL WC=23.5%
26							25			
27							30			
28							34			
29							36			
30	2D	24/20	30.00 - 32.00	5/7/8/9	15	21	37		Grey, wet, very stiff, Clayey SILT, trace sand, trace gravel.	G#209202 A-4, CL WC=20.3%
31							45			
32							53			
33							58			
34							54			
35	3D	24/2	35.00 - 37.00	3/4/7/9	11	15	56		Similar to above, stiff.	
36							57			
37	R5	60/57	37.90 - 42.90	RQD = 78%			65 NQ	-26.92	37.50 Roller Coned ahead to 37.9' bgs. Top of Bedrock at Elev. -26.92'. R5(Bedrock): Grey, black and pink, GRANITE, medium grained, iron staining in upper 2 feet. Core Times (min:sec) 37.9-38.9' (4:04) 38.9-39.9' (4:00) 39.9-40.9' (4:47) 40.9-41.9' (5:05) 41.9-42.9' (5:58) 95% Recovery	
38							65			
39							60 CORE			
42.9	R6	60/60	42.90 - 47.90	RQD = 87%					R6(Bedrock): Grey and black, GRANITE, medium grained. Core Times (min:sec) 42.9-43.9' (5:16) 43.9-44.9' (6:12) 44.9-45.9' (6:24) 45.9-46.9' (7:37) 46.9-47.9' (9:10) 100% Recovery	
47.9								-37.32	47.90 Bottom of Exploration at 47.90 feet below ground surface.	

Remarks:
13" thick concrete Bridge Deck at Elev. 14.08'.
Top of Granite Wall/Causeway at Elev. 10.58'.
3.5' from Bridge Deck to top of Granite Wall/Causeway.

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-STFH-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1674(500)S

BRIDGE PLANS

PIN 16745.00

BRIDGE NO. 5578

		BY	DATE
PROJ. MANAGER	J.D.W.		
DESIGN-DETAILED	K.MAGUIRE	T. WHITE	APR 2009
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	-----	-----	-----
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----		-----
REVISIONS 2	-----		-----
REVISIONS 3	-----		-----
REVISIONS 4	-----		-----
FIELD CHANGES	-----		-----

SIGNATURE
P.E. NUMBER
DATE

SPRUCE HEAD BRIDGE
ATLANTIC OCEAN
SOUTH THOMASTON KNOX COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

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.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	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 correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
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
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Spruce Head Bridge #5578 over the Atlantic Ocean carrying Spruce Head Island Road
Location: South Thomaston, Maine

Boring No.: BB-STFH-101
PIN: 16745.00

Driller:	MaineDOT	**Elevation (ft.)**	10.42	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/25-27/09-3/30/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+18.5, 7.5 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	R1	30/14	0.00 - 2.50				NQ-2			Top of Granite Wall/Causeway. Grey and white, GRANITE pegmetite, coarse grained with mica. Spun NW Casing ahead to 23.6'. R1: Core Times average 5 minutes per foot in Granite blocks.	
	R2	60/26	2.50 - 7.50							R2:Core Times average 6 minutes per foot in Granite Wall/Causeway. Grey, white and black, GRANITE, fine grained.	
5										Granite blocks.	
							RC			Roller Coned ahead to 12.0' with large roller bit (3 7/8").	
10										Granite blocks.	
	MD R3	4/0 56.4/40	12.00 - 12.33 12.30 - 17.00	30(3.6")	---		NQ-2			Failed sample attempt. Grey and white, GRANITE pegmetite, coarse grained. R3:Core Times (min:sec) 12.3-13.3' (5:12) 13.3-14.3' (4:10) 14.3-15.3' (3:20) (6" drop) 15.3-16.3' (4:25) 16.3-17.0' (2:50) broke through	
15											
	1D	24/18	17.00 - 19.00	7/2/5/6	7	10		-6.58		17.00 Black, wet, loose, silty fine to medium SAND, trace marine clay (muck), trace shells fragments with odor.	
	R4	54/28	19.00 - 23.50				NQ-2	-8.58		19.00 Grey and white, GRANITE pegmetite, coarse grained, Granite blocks and Cobbles. R4:Core Times (min:sec) 19.0-20.0' (4:37) 20.0-21.0' (3:25) 21.0-22.0' (2:25) 22.0-23.0' (1:16) 23.0-23.6' (0:10) Core Blocked	
20											
	R5	60/30	23.60 - 28.60				8 NQ-2 14			Grey and white, GRANITE pegmetite, coarse grained. Granite blocks and Cobbles. R5:Core Times (min:sec)	G#210034 A-6, CL WC=22.8%
25											

Remarks:

12" thick concrete Bridge Deck at Elev. 13.92'
Top of Granite Wall/Causeway at Elev. 10.42'.
3.5' from Bridge Deck to top of Granite Wall/Causeway.
Left 10 feet of casing in hole.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Spruce Head Bridge #5578 over the Atlantic Ocean carrying Spruce Head Island Road
Location: South Thomaston, Maine

Boring No.: BB-STFH-101
PIN: 16745.00

Driller:	MaineDOT	**Elevation (ft.)**	10.42	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/25-27/09-3/30/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+18.5, 7.5 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							22			23.6-24.6' (1:20) 24.6-25.6' (1:40) 25.6-26.6' (1:15) 26.6-27.6' (1:20) 27.6-28.6' (1:25) Brown, Clayey SILT, trace sand, trace gravel in core barrel, roller coned ahead to 26.5' then drove casing.	LL=34 PL=18 PI=16
							26				
							41				
							54				
							137				
30							123				
	2D/AB	24/24	31.50 - 33.50	8/11/4/7	15	21	113	-21.08		31.50 2D/A (31.5-32.5') Dark grey, wet, medium dense, fine sandy silt, some coarse sand.	
							122	-22.08		32.50 2D/B (32.5-33.5') Dark grey, moist, very stiff, Clayey SILT, trace sand, trace gravel, blocky.	G#210035 A-4, CL WC=23.1% LL=27 PL=17 PI=10
							127				
							107				
35							116				
	3D	24/2	36.50 - 38.50	26/29/27/23	56	78	106			Same as above, hard.	
							43				
	R6	42/0	38.60 - 42.10	No recovery			56	-28.18		38.60 R6:Core Times (min:sec) 38.6-39.6' (5:20) 39.6-40.6' (0:50) 40.6-41.6' (1:10) 41.6-42.2' (2:30) No Recovery.	
							60				
40							42				
							47				
	R7	60/42	42.20 - 47.20	RQD = 53%			NQ-2			Grey, black and white, GRANITE, medium grained, iron staining at fractures. R7:Core Times (min:sec) 42.2-43.2' (3:44) 43.2-44.2' (7:10) 44.2-45.2' (4:08) 5" drop 45.2-46.2' (2:35) 4" drop 46.2-47.2' (5:25)	
45											
	R8	54/54	47.20 - 51.70	RQD = 74%						Grey, black and white, GRANITE, medium grained, iron staining at fractures. R8:Core Times (min:sec) 47.2-48.2' (3:45) 48.2-49.2' (5:03) 49.3-50.2' (5:07)	
50											

Remarks:

12" thick concrete Bridge Deck at Elev. 13.92'
Top of Granite Wall/Causeway at Elev. 10.42'.
3.5' from Bridge Deck to top of Granite Wall/Causeway.
Left 10 feet of casing in hole.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Spruce Head Bridge #5578 over the Atlantic Ocean carrying Spruce Head Island Road	**Boring No.:**	BB-STFH-101		
Location:	South Thomaston, Maine	**PIN:**	16745.00		
Driller:	MaineDOT	**Elevation (ft.)**	10.42	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/25-27/09-3/30/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+18.5, 7.5 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										50.2-51.2' (5:24) 51.2-51.8' (10:00)	
								-41.38		51.80 **Bottom of Exploration at 51.80 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

12" thick concrete Bridge Deck at Elev. 13.92'
Top of Granite Wall/Causeway at Elev. 10.42'.
3.5' from Bridge Deck to top of Granite Wall/Causeway.
Left 10 feet of casing in hole.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Spruce Head Bridge #5578 over the Atlantic Ocean carrying Spruce Head Island Road
Location: South Thomaston, Maine

Boring No.: BB-STFH-102
PIN: 16745.00

Driller:	MaineDOT	**Elevation (ft.)**	10.58	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/31/09-4/02/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+23.9, 7.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	R1	72/60	0.00 - 6.00				NQ-2 NW			Top of Granite Wall/Causeway. Grey and white, GRANITE pegmetite, coarse grained. R1: Core Times average 6 minutes per foot in Granite Wall/Causeway.	
5											
	R2	60/46	6.00 - 11.00							Grey and white, GRANITE pegmetite, coarse grained. R2: Core Times average 7 minutes per foot in Granite Wall/Causeway.	
10											
	R3	42/20	11.00 - 14.50							Grey and white, GRANITE pegmetite, coarse grained. R3: Core Times average 7 minutes per foot in Granite Wall/Causeway. Core Blocked at 14.5' bgs.	
										Roller Coned ahead with large roller from 14.5-15.7' bgs.	
15	R4	60/27	15.70 - 20.70							Grey and white, GRANITE pegmetite, coarse grained. R4:Core Times (min:sec) 15.7-16.7' (6:20) 16.7-17.7' (4:10) 4" drop 17.7-18.7' (2:10) 18.7-19.7' (4:20) 19.7-20.7' (0:58)	
							14				
20							11			Broke casing, roller coned ahead to 25.0' with large roller bit, Dropped in NW Casing.	
							12				
							23				
							24				
							22				
25											

Remarks:

13" thick concrete Bridge Deck at Elev. 14.08'
Top of Granite Wall/Causeway at Elev. 10.58'.
3.5' from Bridge Deck to top of Granite Wall/Causeway.

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

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US CUSTOMARY UNITS

Project: Spruce Head Bridge #5578 over the Atlantic Ocean carrying Spruce Head Island Road
Location: South Thomaston, Maine

Boring No.: BB-STFH-102
PIN: 16745.00

Driller:	MaineDOT	**Elevation (ft.)**	10.58	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/31/09-4/02/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+23.9, 7.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	1D	24/20	25.00 - 27.00	4/4/6/8	10	14	22	-14.42		25.00 Grey, wet, stiff, Clayey SILT, trace sand, trace gravel. Roller coned ahead to 30.0' with large roller bit.	G#209201 A-4, CL WC=23.5%
							25				
							30				
							34				
							36				
30	2D	24/20	30.00 - 32.00	5/7/8/9	15	21	37			Grey, wet, very stiff, Clayey SILT, trace sand, trace gravel.	G#209202 A-4, CL WC=20.3%
							45				
							53				
							58				
							54				
35	3D	24/2	35.00 - 37.00	3/4/7/9	11	15	56			Similar to above, stiff.	
							57				
	R5	60/57	37.90 - 42.90	RQD = 78%			65 NQ	-26.92		Roller Coned ahead to 37.9' bgs. 37.50	
							65			Top of Bedrock at Elev. -26.92'. R5:Bedrock: Grey, black and pink, GRANITE, medium grained, iron staining in upper 2 feet. Core Times (min:sec) 37.9-38.9' (4:04) 38.9-39.9' (4:00) 39.9-40.9' (4:47) 40.9-41.9' (5:05) 41.9-42.9' (5:58) 95% Recovery	
							60 CORE				
40											
	R6	60/60	42.90 - 47.90	RQD = 87%							
										R6:Bedrock: Grey and black, GRANITE, medium grained. Core Times (min:sec) 42.9-43.9' (5:16) 43.9-44.9' (6:12) 44.9-45.9' (6:24) 45.9-46.9' (7:37) 46.9-47.9' (9:10) 100% Recovery	
45											
								-37.32		47.90 **Bottom of Exploration at 47.90 feet below ground surface.**	
50											

Remarks:

13" thick concrete Bridge Deck at Elev. 14.08'
Top of Granite Wall/Causeway at Elev. 10.58'.
3.5' from Bridge Deck to top of Granite Wall/Causeway.

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): South Thomaston Project Number: 16745.00

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-STFH-101, R5	9+18.5	7.5 Rt.	23.6-28.6	210034	1	22.8	34	16	CL	A-6	III
BB-STFH-101, 2D/B	9+18.5	7.5 Rt.	32.5-33.5	210035	1	23.1	27	10	CL	A-4	IV
BB-STFH-102, 1D	10+23.9	7.5 Lt.	25.0-27.0	209201	1	23.5			CL	A-4	IV
BB-STFH-102, 2D	10+23.9	7.5 Lt.	30.0-32.0	209202	1	20.3			CL	A-4	IV

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

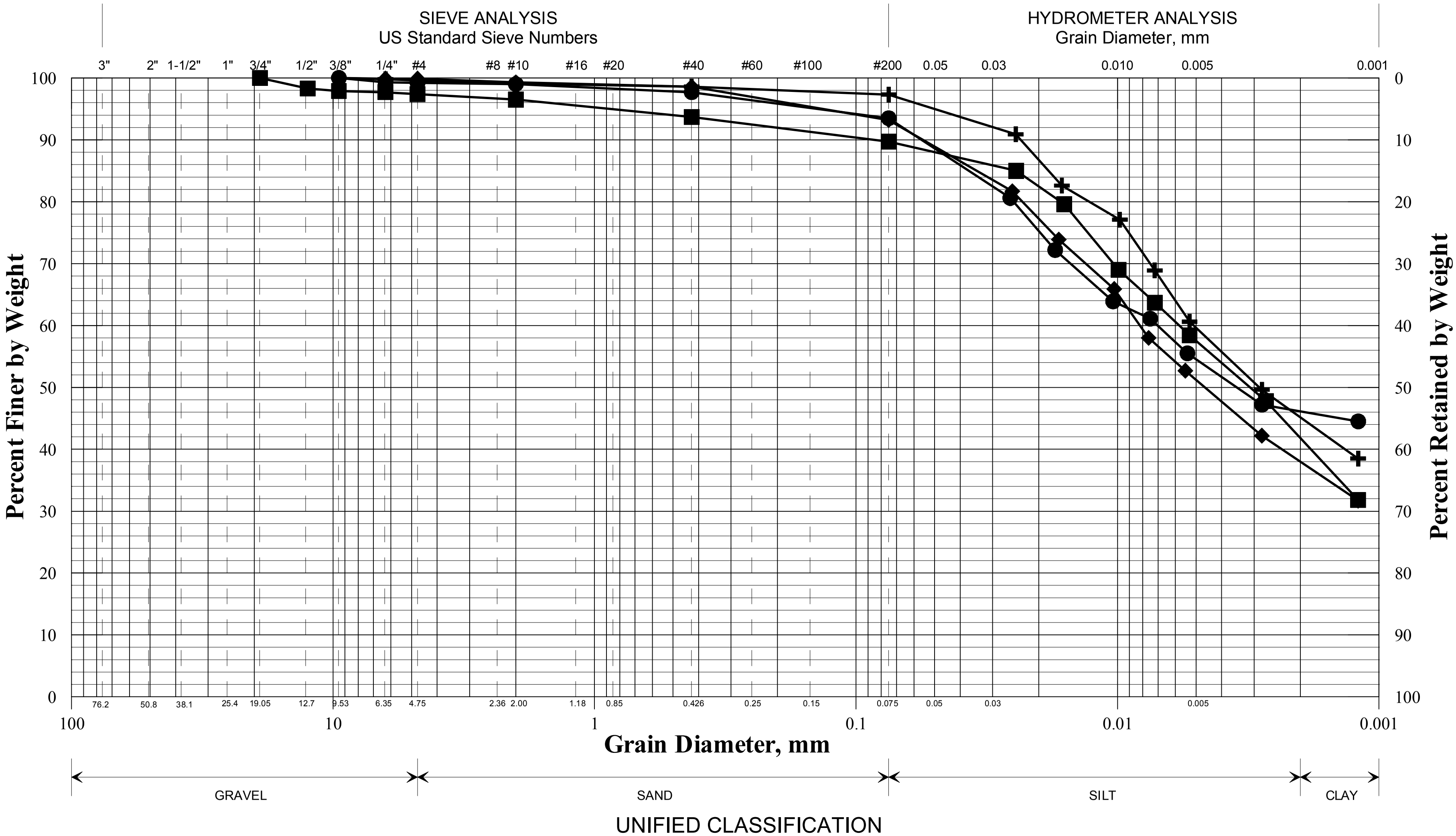
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

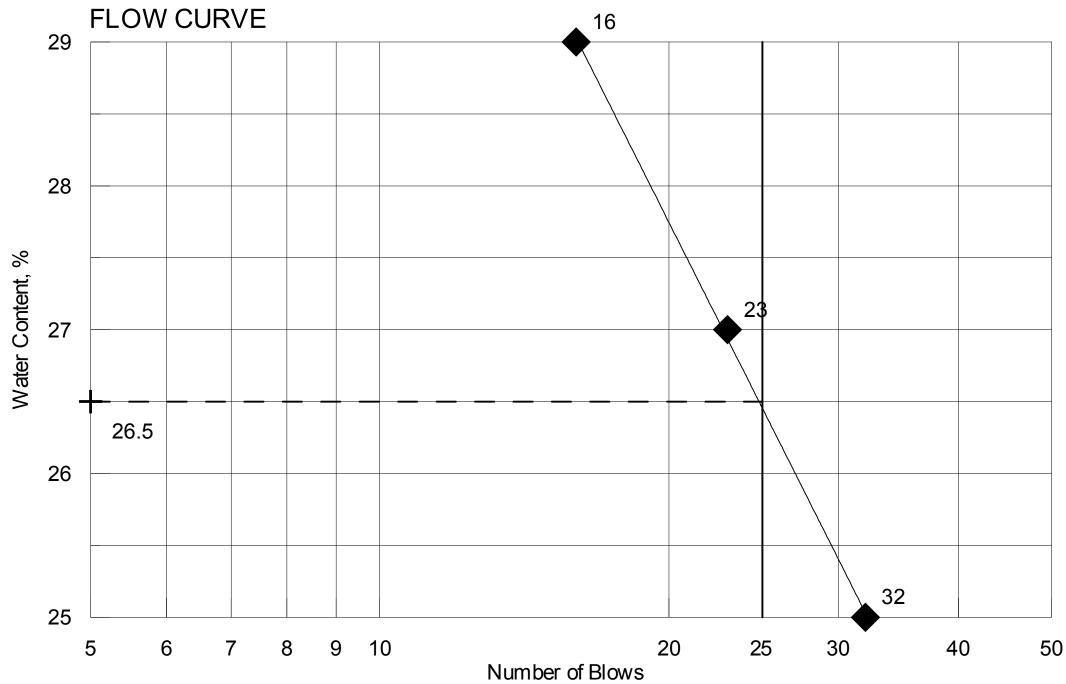


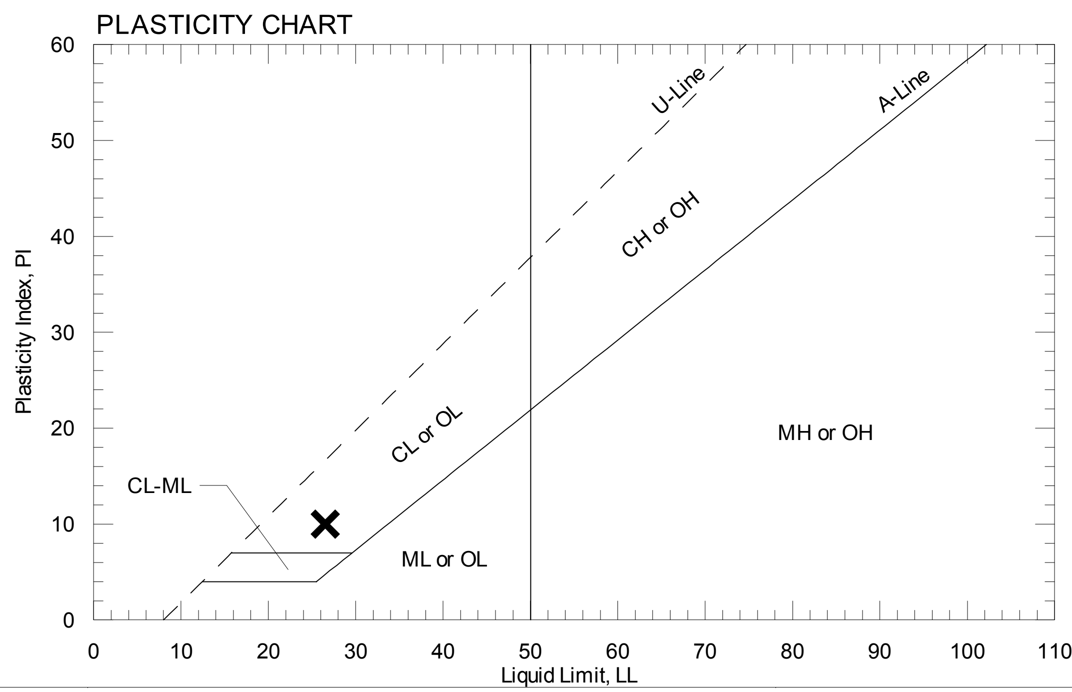
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-STFH-101/R5	9+18.5	7.5 RT	23.6-28.6	Clayey SILT, trace sand, trace gravel.	22.8	34	18	16
◆	BB-STFH-101/2D/B	9+18.5	7.5 RT	32.5-33.5	Clayey SILT, trace sand, trace gravel.	23.1	27	17	10
■	BB-STFH-102/1D	10+23.9	7.5 LT	25.0-27.0	Clayey SILT, trace sand, trace gravel.	23.5			
●	BB-STSH-102/2D	10+23.9	7.5 LT	30.0-32.0	Clayey SILT, trace sand, trace gravel.	20.3			
▲									
✖									

PIN
016745.00
Town
South Thomaston
Reported by/Date
WHITE, TERRY A 5/13/2009

SHEET 1

TOWN	South Thomaston (Stimulus)	Reference No.	210035
PIN	016745.00	Water Content, %	23.1
Sampled	3/26/2009	Plastic Limit	17
Boring No./Sample No.	BB-STFH-101/2D/B	Liquid Limit	27
Station	9+18.5	Plasticity Index	10
Depth	32.5-33.5	Tested By	BBURR



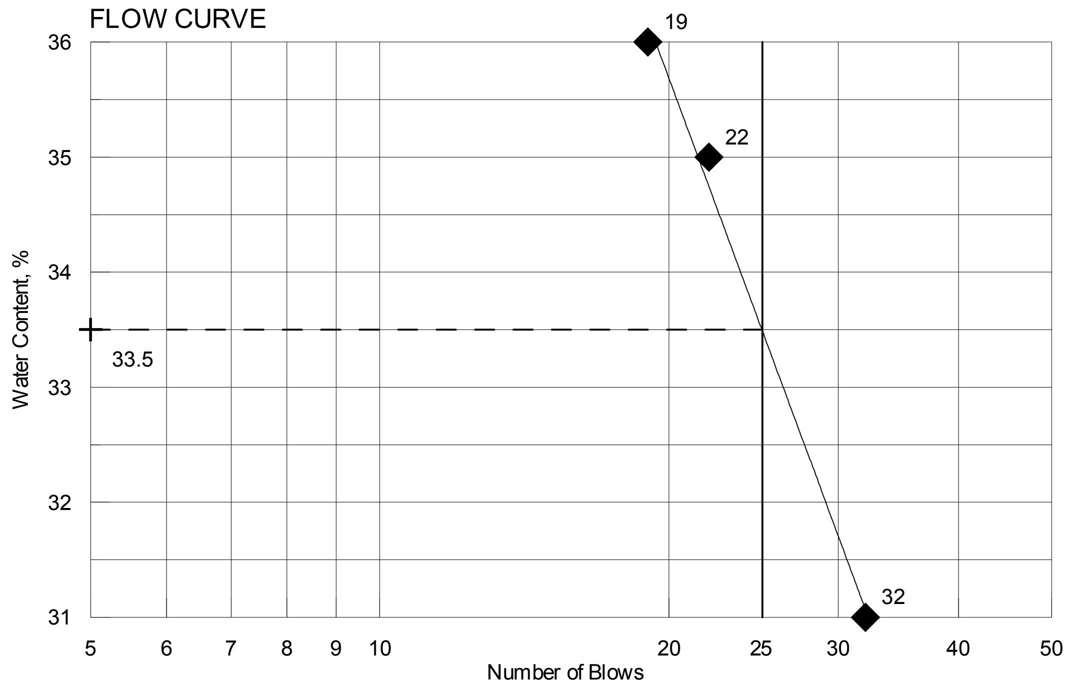


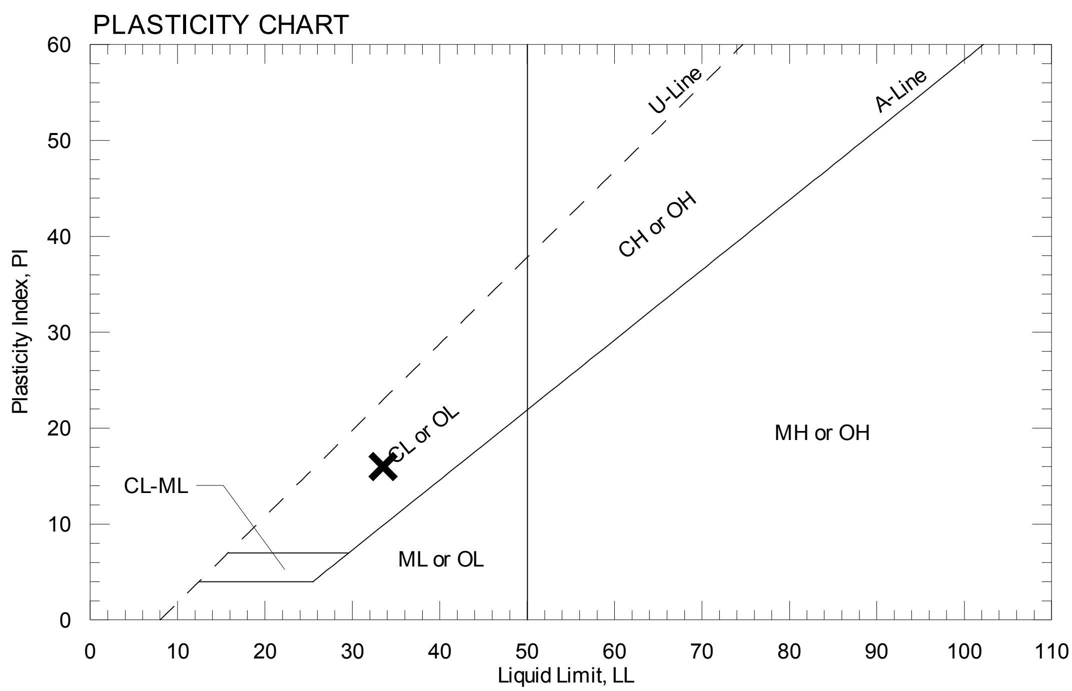
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **4/3/2009**

Paper Copy: Lab File; Project File; Geotech File

TOWN	South Thomaston (Stimulus)	Reference No.	210034
PIN	016745.00	Water Content, %	22.8
Sampled	3/26/2009	Plastic Limit	18
Boring No./Sample No.	BB-STFH-101/R5	Liquid Limit	34
Station	9+18.5	Plasticity Index	16
Depth	23.6-28.6	Tested By	BBURR





AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **4/3/2009**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

LIQUIDITY INDEX (LI):

Liquidity Index = (natural water content - Plastic Limit) / (Liquid Limit -Plastic Limit)

wc is close to LL	Soil is normally consolidated
wc is close to PL	Soil is some-to-heavily over consolidated
wc is intermediate	Soil is over consolidated
wc is greater than LL	Soil is on the verge of being a viscous liquid when remolded

Sample	Soil	WC	LL	PL	PI	LI	
BB-STFH-101 R5	Clayey Silt	22.8	34	18	16	0.30	overconsolidated
BB-STFH-101 2D/B	Clayey Silt	23.1	27	17	10	0.61	overconsolidated

ABUTMENT FOUNDATIONS: Integral H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at the following piles:

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

yield strength: $F_y := 50 \cdot ksi$

Nominal Compressive Resistance $P_n=0.66^{\lambda}*F_y*A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

$\lambda=(Kl/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3

$\lambda := 0$ as l unbraced length is 0

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$

$P_n = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Strength Limit State Axial Resistance factor for piles in compression under good driving conditions:

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance:

eq. 6.9.2.1-1 $P_{fstr} := \phi_c \cdot P_n$

$P_{fstr} = \begin{pmatrix} 465 \\ 654 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE LIMIT STATE:

Factored Compressive Resistance for Service Limit State:

Resistance Factors for Service Limit State $\phi := 1.0$ LRFD 10.5.5.1 and 10.5.8.3

eq. 6.9.2.1-1 $P_{fserv} := \phi \cdot P_n$ $P_{fserv} = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service Limit State

EXTREME LIMIT STATE:

Recalculate P_n with λ value due to scour. Assume all causeway material will scour ~25 feet of unbraced pile length.

Nominal Compressive Resistance $P_n = 0.66^{\lambda} * F_y * A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor $\lambda = (Kl/r_s\pi)^2 \times (F_y/E)$ eq. 6.9.4.1-3

K_{elf} = Effective length factor LRFD Article 4.6.2.5 $K_{elf} := 1.0$

l_{unbr} = unbraced length $l_{unbr} := 25 \cdot ft$

r_s = governing radius of gyration $r_s := \begin{pmatrix} 5.03 \\ 5.11 \\ 5.84 \\ 5.88 \\ 5.96 \end{pmatrix} \cdot in$

F_y = specified minimum yield strength $Fy := 50 \cdot ksi$

E_{steel} = modulus of elasticity $E_{steel} := 29000 \cdot ksi$

$$\lambda_{extreme} := \left(\frac{K_{elf} \cdot l_{unbr}}{r_s \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}} \qquad \lambda_{extreme} = \begin{pmatrix} 0.621 \\ 0.602 \\ 0.461 \\ 0.455 \\ 0.443 \end{pmatrix}$$

$$P_{nextreme} := \overrightarrow{\left[\left(0.66^{\lambda_{extreme}}\right) \cdot F_y \cdot A_s\right]} \qquad P_{nextreme} = \begin{pmatrix} 599 \\ 849 \\ 883 \\ 1080 \\ 1431 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Resistance Factors for Extreme Limit State $\phi := 1.0$ LRFD 10.5.5.1 and 10.5.8.3

Factored Compressive Resistance for Extreme Limit State:

eq. 6.9.2.1-1 $P_f := \phi \cdot P_{nextreme}$ $P_f = \begin{pmatrix} 599 \\ 849 \\ 883 \\ 1080 \\ 1431 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Extreme Limit State

Geotechnical Resistance

Assume piles will be end bearing on bedrock.

Bedrock Type:

Granite RQD ranges from 53 to 87%.

Use RQD = 60% and ϕ = 34 to 40 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

Steel area:

$$A_s = \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$$

Pile depth:

$$d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$$

Pile width:

$$b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

End bearing resistance of piles on bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

q_u for granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_{cG} := 30000 \cdot psi$

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b:

$$b = \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

HP 12 x 53
HP 12 x74
HP 14 x 73
HP 14 x 89
HP 14 x 117

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.563 \\ 0.559 \\ 0.514 \\ 0.513 \\ 0.51 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Design as end bearing on bedrock

Diameter of socket, B_s: $B_s := 1 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$q_a := \sigma_{cG} \cdot K_{sp} \cdot d_f$ $q_a = \begin{pmatrix} 2434 \\ 2417 \\ 2222 \\ 2215 \\ 2202 \end{pmatrix} \cdot ksf$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$R_p := \overrightarrow{(3q_a \cdot A_s)}$ $R_p = \begin{pmatrix} 786 \\ 1098 \\ 991 \\ 1204 \\ 1578 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$R_f := \phi_{stat} \cdot R_p$ $R_f = \begin{pmatrix} 354 \\ 494 \\ 446 \\ 542 \\ 710 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

$R_{fse} := \phi \cdot R_p$ $R_{fse} = \begin{pmatrix} 786 \\ 1098 \\ 991 \\ 1204 \\ 1578 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

Calculate Depth to Fixity for H-piles:

Consider Pile sizes:

HP 12x53
HP 12x74
HP 14x73
HP 14x 89
HP 14x117

Use LRFD Article 10.7.3.13.4:

$L_{fix} = 1.4\ (E_p{*}I_w/E_s)^{0.25}$ for clays

E_p = Modulus of elasticity of pile (ksi): $E_{steel} := 29000 \cdot ksi$

I_p = weak axis Moment of Inertia (ft^4): $I_p := \begin{pmatrix} 393 \\ 569 \\ 729 \\ 904 \\ 1220 \end{pmatrix} \cdot in^4$ $\quad I_p = \begin{pmatrix} 0.019 \\ 0.027 \\ 0.035 \\ 0.044 \\ 0.059 \end{pmatrix} \cdot ft^4$

E_s = Soil Modulus = $0.465{*}S_u$ for clays $\quad S_u := 1000 \cdot psf$

S_u = Undrained shear strength of clay (ksf): Assume S_u of clayey silt based on N-values: $S_u = 1 \cdot ksf$

$E_s := 0.465 \cdot S_u \qquad E_s = 3.229 \times 10^{-3} \cdot ksi$

$$L_{fix} := 1.4 \left(\frac{E_{steel} \cdot I_p}{E_s \cdot 144} \right)^{0.25} \qquad L_{fix} = \begin{pmatrix} 8 \\ 9 \\ 10 \\ 10 \\ 11 \end{pmatrix} \cdot ft$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
σ_{dr} = 0.9 x ϕ_{da} x f_y (eq. 10.7.8-1)

$f_y := 50 \cdot ksi$ yield strength of steel

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1
Pile Drivability Analysis, Steel piles

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ driving stresses in pile cannot exceed 45 ksi

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load
(must be less than the the factored geotechnical resistance from above as this governs)
divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium site variability. There will probably only be 4 to 5 piles total at each abutment. Only 1 or 2 piles will be tested - one per abutment will be requested. Therefore, reduce the ϕ by 20%

$\phi_{dyn.reduced} := 0.65 \cdot 0.8$ $\phi_{dyn.reduced} = 0.52$

Pile Size = 12 x 53

Assume Contractor will use a Delmag D 19-42 hammer on 3rd fuel setting to install 12 x 53 piles

State of Maine Dept. Of Transportation — 27-Apr-2009
So Thomaston Spruce Head Bridge 16745.00 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
485.0	42.71	4.06	14.3	8.43	17.88
486.0	42.73	4.07	14.4	8.44	17.88
487.0	42.80	4.08	14.5	8.45	17.93
488.0	42.82	4.08	14.6	8.45	17.94
489.0	42.84	4.08	14.7	8.46	17.95
490.0	42.87	4.09	14.9	8.46	17.96
491.0	42.88	4.09	15.0	8.47	17.96
492.0	42.88	4.10	15.2	8.48	17.97
493.0	42.90	4.10	15.3	8.48	17.98
494.0	42.92	4.10	15.5	8.49	17.98

DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	15.50	in2

Limit blow count to 15 blows per inch

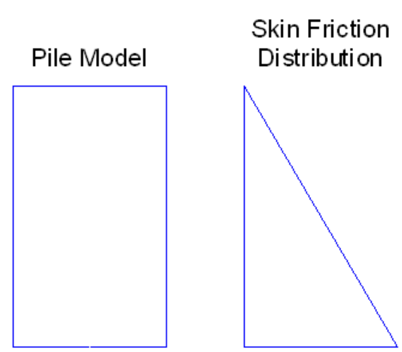
Strength Limit State:

$$R_{dr\_12x53\_factored} := 491 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_12x53\_factored} = 255 \cdot kip$$

Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_12x53\_servext} := 491 \cdot kip$$



Res. Shaft = 10 %
(Proportional)

Pile Size = 12 x 74

Assume Contractor will use a Delmag D 19-42 hammer on 3rd fuel setting to install 12 x 74 piles

State of Maine Dept. Of Transportation — 12-May-2009
So Thomaston Spruce Head Bridge 16745.00 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
530.0	34.70	3.91	14.8	8.35	16.60
531.0	34.73	3.93	14.8	8.35	16.63
532.0	34.74	3.95	14.9	8.36	16.65
533.0	34.78	3.96	15.0	8.37	16.67
534.0	34.80	3.98	15.1	8.37	16.69
535.0	34.81	3.99	15.3	8.38	16.67
536.0	34.85	4.01	15.3	8.38	16.69
537.0	34.88	4.02	15.4	8.39	16.71
538.0	34.87	4.04	15.5	8.39	16.73
539.0	34.91	4.05	15.7	8.40	16.71

Limit blow count to 15 blows per inch

Strength Limit State:

$$R_{dr\_12x74\_factored} := 533 \cdot kip \cdot \phi_{dyn.reduced}$$

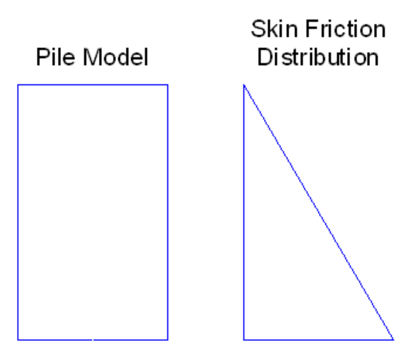
$$R_{dr\_12x74\_factored} = 277 \cdot kip$$

Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_12x74\_servext} := 533 \cdot kip$$

DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	21.80	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D 36-32 hammer on 2nd fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 27-Apr-2009
So Thomaston Spruce Head Bridge 16745.00 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
528.0	44.80	1.68	4.3	7.10	30.67
529.0	44.89	1.69	4.3	7.10	30.73
530.0	44.91	1.69	4.3	7.11	30.74
531.0	44.99	1.71	4.4	7.12	30.78
532.0	45.07	1.73	4.4	7.12	30.85
533.0	45.09	1.72	4.4	7.13	30.85
534.0	45.14	1.73	4.4	7.14	30.89
535.0	45.16	1.73	4.4	7.14	30.85
536.0	45.25	1.75	4.4	7.14	30.90
537.0	45.32	1.76	4.4	7.16	31.02

Limit driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x73\_factored} := 531 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_14x73\_factored} = 276 \cdot kip$$

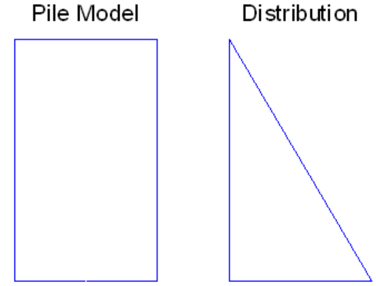
Service and Extreme Limit States:

$$R_{dr\_14x73\_servext} := 531 \cdot kip \qquad \phi := 1.0$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89

Assume Contractor will use a Delmag D 36-32 hammer on 3rd fuel setting to install 14 x 89 piles

State of Maine Dept. Of Transportation — 27-Apr-2009
So Thomaston Spruce Head Bridge 16745.00 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
650.0	44.80	1.89	4.9	8.27	36.34
651.0	44.88	1.92	4.9	8.27	36.48
652.0	44.91	1.93	4.9	8.28	36.49
653.0	44.95	1.94	4.9	8.28	36.51
654.0	44.98	1.95	5.0	8.29	36.52
655.0	45.01	1.95	5.0	8.29	36.52
656.0	45.03	1.96	5.0	8.30	36.53
657.0	45.12	1.97	5.0	8.31	36.62
658.0	45.10	1.97	5.0	8.31	36.54
659.0	45.15	1.98	5.0	8.31	36.55

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	26.10	in2

Limit driving stress to 45 ksi

Strength Limit State:

$R_{dr\_14x89\_factored} := 655 \cdot kip \cdot \phi_{dyn.reduced}$

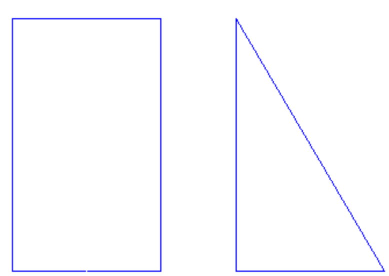
$R_{dr\_14x89\_factored} = 341 \cdot kip$

Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x89\_servext} := 655 \cdot kip$

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a Delmag D 36-32 hammer on 3rd fuel setting to install 14 x 117 piles

State of Maine Dept. Of Transportation — 27-Apr-2009
So Thomaston Spruce Head Bridge 16745.00 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1030.0	44.23	3.76	14.8	9.46	38.48
1031.0	44.22	3.74	15.0	9.46	38.35
1032.0	44.26	3.76	14.9	9.47	38.43
1033.0	44.27	3.76	15.0	9.48	38.41
1034.0	44.27	3.77	15.1	9.47	38.40
1035.0	44.27	3.79	15.1	9.48	38.49
1036.0	44.29	3.79	15.2	9.48	38.46
1037.0	44.29	3.79	15.3	9.48	38.45
1038.0	44.34	3.82	15.2	9.49	38.55
1039.0	44.34	3.82	15.3	9.49	38.53

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	34.40	in2

Limit blow count to 15 blows per inch

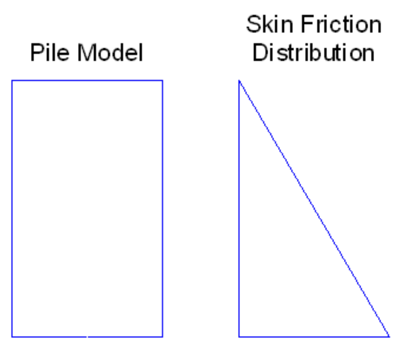
Strength Limit State:

$$R_{dr\_14x117\_factored} := 1033 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_14x117\_factored} = 537 \cdot kip$$

Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_14x117\_servext} := 1033 \cdot kip$$



Res. Shaft = 10 %
(Proportional)

Earth Pressures:

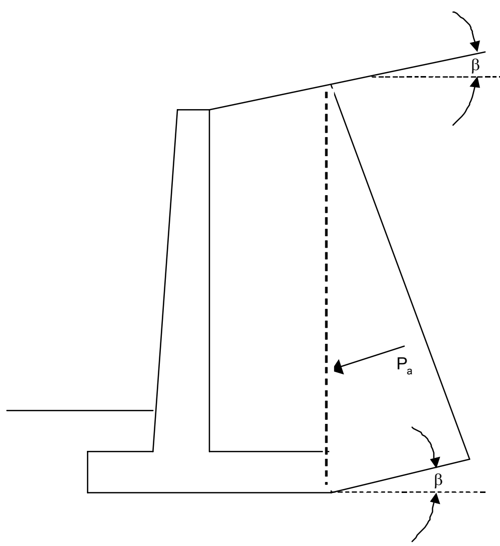
Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

unit weight: $\gamma_{type4} := 125 \cdot pcf$

Internal Friction Angle: $\phi_{type4} := 32 \cdot deg$

Cohesion: $c_{sand} := 0 \cdot psf$

Active Earth Pressure - Rankine Theory
from MaineDOT Bridge Design Guide Section 3.6.5.2 pg 3-7



Generally use Rankine for long heeled cantilever walls where the failure surface is un interrupted by the top of the wall system. The earth pressure is applied to a plane extending vertically up from the heel of the wall base and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or the backface of the wall.

For cantilever walls with horizontal backfill surface:

$$K_{a\_rankine} := \tan\left(45 \cdot deg - \frac{\phi_{type4}}{2}\right)^2 \qquad K_{a\_rankine} = 0.307$$

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

$\beta := 0 \cdot deg$

$$K_{a\_rankine\_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos\left(\phi_{type4}\right)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos\left(\phi_{type4}\right)^2}} \qquad K_{a\_rankine\_slope} = 0.307$$

Pa is oriented at an angle of β to the vertical plane.

Passive Earth Pressure - Coulomb Theory
from MaineDOT Bridge Design Guide Section 3.6.6 pg 3-8

Angle of back face of wall to the horizontal: $\alpha := 90 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 20 \cdot deg$

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

$$K_{p\_coulomb} := \frac{\sin(\alpha - \phi)^2}{\sin(\alpha)^2 \cdot \sin(\alpha + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \delta) \cdot \sin(\alpha + \beta)}}\right)^2}$$

$K_{p\_coulomb} = 6.89$

Passive Earth Pressure - Rankine Theory
from Bowles 5th Edition Section 11-5 pg 602

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

$$K_{p\_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$K_{p\_rank} = 3.25$

Bowles does not recommend the use of the Rankine Method for K_p when β>0.

Settlement Analysis:

Reference: FHWA Soils and Foundation Reference Manual - Volume I (FHWA NHI-06-088) 2006 pg 7-16

The roadway will be widened with a maximum fill height of 8 feet and an average fill height of 5 feet.
Look at Station 7+25 with 8.0 ft of fill. Simplified soil profile based on BB-STFH-101:

Finished Grade
Elevation 13.13 ft

Proposed Fill - Look at 8.0 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Groundwater Elevation 2.5 ft

$\gamma_w := 62.4 pcf$

Elevation 5.13 ft

Existing Clayey Silt

$H_1 := 12.5 \cdot ft$ $\quad \gamma_{silt} := 115 \cdot pcf$ $\quad N_{silt} := 18$

Elevation -7.37 ft

Bedrock

Silt

Determine corrected SPT value N':

N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{1o} := \left[\frac{H_1}{2} \cdot (\gamma_{silt} - \gamma_w)\right] \qquad \sigma_{1o} = 328.75 \cdot psf$$ at mid-point

SPT N-value (bpf) $\quad N_{silt} := 18$

AT P_o = 328 psf $\quad$ N'/N_{fill} = r1 = 2.0 $\quad r1 := 2.0$

Corrected Blow Count $\quad N' := r1 \cdot N_{silt} \qquad N' = 36$

From Figure 13 using the "Inorganic silt" curve

Bearing Capacity Index: $\quad C1 := 63$

Change in stress at the mid point of the layer under consideration

$$\Delta\sigma_{zsilt} := 360 \cdot psf$$

Upper Crust Silt:

$$\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{zsilt}}{\sigma_{1o}}\right) \qquad \Delta H_1 = 0.765 \cdot in$$

Frost Protection:

Method 1 - 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:
South Thomaston, Maine
DFI = 1100 degree-days

Soils are coarse grained. Assume a water content = ~20%

From MaineDOT BDG Table 5-1:
Depth of frost penetration = 57.8 inches

$\text{Frost\_depth} := 57.8\text{in} \qquad \text{Frost\_depth} = 4.817 \cdot \text{ft}$

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.

Method 2 - Check Frost Depth using ModBerg Software

Closest Station is Belfast

--- ModBerg Results ---

Project Location: Belfast, Maine

Air Design Freezing Index	= 1188 F-days
N-Factor	= 0.80
Surface Design Freezing Index	= 950 F-days
Mean Annual Temperature	= 45.5 deg F
Design Length of Freezing Season	= 118 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	67.5	20.0	125.0	34	46	3.8	1.9	3,600

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.

Total Depth of Frost Penetration = 5.63 ft = 67.5 in.

Use BDG Calculated Frost Depth = 5.0 feet for design

Seismic:

South Thomaston Spruce Head Bridge PIN 16745.00
Date and Time: 5/11/2009 2:08:48 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04858
Zip Code Latitude = 44.034100
Zip Code Longitude = -069.143000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.058	PGA - Site Class B
0.2	0.128	Ss - Site Class B
1.0	0.040	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04858
Zip Code Latitude = 44.034100
Zip Code Longitude = -069.143000
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.144	As - Site Class E
0.2	0.320	SDs - Site Class E
1.0	0.139	SD1 - Site Class E

Bearing Resistance - Silt:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on silt

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications Third Edition
Table C10.6.2.6.1-1 "Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy *(1982)*"

Type of Bearing Material: Homogeneous inorganic clay, sandy or silty clay (CL, CH)

Based on corrected N-values ranging from 14 to 78 - Soils are stiff to hard

Consistency In Place: Medium Dense to Dense

Bearing Resistance: Ordinary Range (ksf) 2 - 6

Recommended Value of Use (ksf): 4 ksf

Recommended Value: $q_{nom} := 4 \cdot ksf$

Resistance factor at the **service limit state** Φ=1.0 (LRFD Article 10.5.5.1)

$q_{factored\_bc} := q_{nom} \cdot 1.0$ $q_{factored\_bc} = 4 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on silt

Reference: Foundation Analysis and Design by JE Bowles Fifth Edition

Section 4-2 Bearing Capacity

Assumptions:

1. Footings will be embedded 5.0 feet for frost protection. $D_f := 5.0 \cdot ft$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4, pg 163)
   - Saturated unit weight: $\gamma_s := 115 \cdot pcf$
   - Dry unit weight: $\gamma_d := 110 \cdot pcf$
   - Internal friction angle: $\phi_{ns} := 32 \cdot deg$
   - Undrained shear strength: $c_{ns} := 500 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 0 \cdot ft$ Tidal

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

Look at several footing widths

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$

Terzaghi Shape factors from Table 4-1 pg 220

For a strip footing: $s_c := 1.0$ $\qquad s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For ϕ=30 deg

$N_c := 30.13 \qquad N_q := 18.4 \qquad N_\gamma := 15.7$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$$q := D_w \cdot \gamma_d + (D_f - D_w) \cdot (\gamma_s - \gamma_w) \qquad q = 0.263 \cdot ksf$$

$$q_{ult} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s - \gamma_w) B \cdot N_\gamma \cdot s_\gamma \qquad q_{ult} = \begin{pmatrix} 22 \\ 23 \\ 24 \\ 25 \end{pmatrix} \cdot ksf$$

Assume this ultimate load is a nominal load. Apply 0.45 resistance factor to get factored resistance.

Resistance Factor: $\phi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$$q_{factored} := q_{ult} \cdot \phi_b$$

$$q_{factored} = \begin{pmatrix} 10.1 \\ 10.4 \\ 10.8 \\ 11.2 \end{pmatrix} \cdot ksf$$

Based on these footing widths:

$$B = \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 10 ksf for footings 8 feet or less.
Recommend a limiting factored bearing resistance of 11 ksf for footings 8.5 to 12 feet or less.

<u>Appendix D</u>

Special Provisions

SPECIAL PROVISION
SECTION 635
PREFABRICATED BIN TYPE RETAINING WALL
(Prefabricated Concrete Modular Gravity Wall)

The following replaces Section 635 in the Standard Specifications in its entirety:

635.01 Description. This work shall consist of the construction of a prefabricated modular reinforced concrete gravity wall in accordance with these specifications and in reasonably close conformance with the lines and grades shown on the plans, or established by the Resident.

Included in the scope of the Prefabricated Concrete Modular Gravity Wall construction are**:** all grading necessary for wall construction, excavation, compaction of the wall foundation, backfill, construction of leveling pads, placement of geotextile, segmental unit erection, and all incidentals necessity to complete the work.

The Prefabricated Concrete Modular Gravity Wall design shall follow the general dimensions of the wall envelope shown in the contract plans. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be at or below the elevation shown on the plans. The top of the face panels shall be at or above the top of the panel elevation shown on the plans.

The Contractor shall require the design-supplier to supply an on-site, qualified experienced technical representative to advise the Contractor concerning proper installation procedures. The technical representative shall be on-site during initial stages of installation and thereafter shall remain available for consultation as necessary for the Contractor or as required by the Resident. The work done by this representative is incidental.

635.02 Materials. Materials shall meet the requirements of the following subsections of Division 700 - Materials:

Gravel Borrow	703.20
Preformed Expansion Joint Material	705.01
Reinforcing Steel	709.01
Structural Precast Concrete Units	712.061
Drainage Geotextile	722.02

The Contractor is cautioned that all of the materials listed are not required for every Prefabricated Concrete Modular Gravity Wall. The Contractor shall furnish the Resident a Certificate of Compliance certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

Concrete Units:

Tolerances. In addition to meeting the requirements of 712.061, all prefabricated units shall be manufactured with the following tolerances. All units not meeting the listed tolerances will be rejected.

1. All dimensions shall be within (edge to edge of concrete) ±3/16 in.

2. Squareness. The length differences between the two diagonals shall not exceed 5/16 in.
3. Surface Tolerances. For steel formed surfaces, and other formed surface, any surface defects in excess of 0.08 in. in 4 ft will be rejected. For textured surfaces, any surface defects in excess of 5/16 in. in 5 ft shall be rejected.

Joint Filler. (where applicable) Joints shall be filled with material approved by the Resident and supplied by the approved Prefabricated Concrete Modular Gravity Wall supplier. 4 in. wide, by 0.5 in. preformed expansion joint filler shall be placed in all horizontal joints between facing units. In all vertical joints, a space of 0.25 in. shall be provided. All Preformed Expansion Joint Material shall meet the requirements of subsection 502.03.

Woven Drainage Geotextile. Woven drainage geotextile 12 in. wide shall be bonded with an approved adhesive compound to the back face, covering all joints between units, including joints abutting concrete structures. Geotextile seam laps shall be 6 in., minimum. The fabric shall be secured to the concrete with an adhesive satisfactory to the Resident. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Concrete Shear Keys. (where applicable) Shear keys shall have a thickness at least equal to the pre-cast concrete stem.

Concrete Leveling Pad. Cast-in-place concrete shall be Class A concrete conforming to the requirements of Section 502 Structural Concrete. The horizontal tolerance on the surface of the pad shall be 0.25 in. in 10 ft. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Backfill and Bedding Material. Bedding and backfill material placed behind and within the reinforced concrete modules shall be gravel borrow conforming to the requirements of Subsection 703.20. The backfill materials shall conform to the following additional requirements: backfill and bedding material shall only contain particles that will pass the 3-inch square mesh sieve and the plasticity index (PI) as determined by AASHTO T90 shall not exceed 6. Compliance with the gradation and plasticity requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

The backfilling of the interior of the wall units and behind the wall shall progress simultaneously. The material shall be placed in layers not over 8 in. in depth, loose measure, and thoroughly compacted by mechanical or vibratory compactors. Puddling for compaction will not be allowed.

Materials Certificate Letter. The Contractor, or the supplier as his agent, shall furnish the Resident a Materials Certificate Letter for the above materials, including the backfill material, in accordance with Section 700 of the Standard Specifications. A copy of all test results performed by the Contractor or his supplier necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based upon the materials Certificate Letter, accompanying test reports, and visual inspection by the Resident.

635.03 Design Requirements. The Prefabricated Concrete Modular Gravity Wall shall be designed and sealed by a Professional Engineer registered in accordance with the laws of the State of Maine. The design to be performed by the wall system supplier shall be in accordance with AASHTO LRFD Bridge Design Specifications, current edition, except as required herein. Design shall consider Strength, Service and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below:

A. Stability Analysis:
1. Overturning: For foundations on soil, the location of the resultant of the reaction forces shall be within the middle one-half of the base width.
2. Sliding: $R_R \geq \gamma_{p(max)} \cdot (EH+ES)$
Where: R_R = Factored Sliding Resistance
$\gamma_{p(max)}$ = Maximum Load Factor
EH = Horizontal Earth Pressure
ES = Earth Surcharge (as applicable)
3. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure
Where: q_R = Factored Bearing Resistance, as shown on the plans
Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.
4. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than, or equal to, 1.5.

Traffic surcharge loads transmitted to the wall through guardrail posts shall be calculated and applied in compliance with LRFD Article 3.11.6.4 and Section 11. Traffic impact loads transmitted to the wall through guardrail posts shall be calculated and applied in compliance with LRFD Article 11, where 11.10.10.2 is modified such that the upper 3.5 ft of concrete modular units shall be designed for an additional horizontal load of γP_{H1}, where γP_{H1}=300 lbs per linear ft of wall.

B. Backfill and Wall Unit Soil Parameters. For overturning and sliding stability calculations, earth pressure shall be assumed acting on a vertical plane rising from the back of the lowest wall stem. For eccentricity (overturning), the unit weight of the backfill within the wall units shall be limited to 96 pcf. For sliding analyses, the unit weight of the backfill within the wall units can be assumed to be 120 pcf. Both analyses may assume a friction angle of 34 degrees for backfill within the wall units.

These unit weights and friction angles are based on a wall unit backfill meeting the requirements for select backfill in this specification. Backfill behind the wall units shall be assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. The friction angle of the foundation soils shall be assumed to be 30 degrees unless otherwise noted on the plans.

C. Internal Stability. Internal stability of the wall shall be demonstrated using accepted methods, such as Elias' Method, 1991. Shear keys shall not contribute to pullout resistance. Soil-to-soil frictional component along stem shall not contribute to pullout resistance. The failure plane used to determine pullout resistance shall be found by the Rankine theory only for vertical walls with level backfills. When walls are battered or with backslopes > 0 degrees are considered, the angle of the failure plane shall be per Jumikus Method. For computation of pullout force, the width of the backface of each unit shall be no greater than 4.5 ft. A unit weight of the soil inside the units shall be assumed no greater than 120 pcf when computing pullout. Coulomb theory may be used.

D. Safety against Structural Failure. Prefabricated units shall be designed for all strength and reinforcement requirements in accordance with LRFD Section 5 and LRFD Article 11.11.5.

E. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic and seismic loads shall be accounted for in the design.

F. The maximum calculated factored bearing pressure under the Prefabricated Concrete Modular Gravity Wall shall be clearly indicated on the design drawings.

G. Stability During Construction. Stability during construction shall be considered during design, and shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, Extreme Limit State.

H. Hydrostatic forces. Unless specified otherwise, when a design high water surface is shown on the plans at the face of the wall, the design stresses calculated from that elevation to the bottom of wall must include a 3 ft minimum differential head of saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

I. Design Life. Design life shall be in accordance with AASHTO requirements, or 75 years; the more stringent requirements apply.

J. Not more than two vertically consecutive units shall have the same stem length, or the same unit depth. Walls with units with extended height curbs shall be designed for the added earth pressure. A separate computation for pullout of each unit with extended height curbs, or extended height coping, shall be prepared and submitted in the design package described above.

<u>635.04 Submittals</u>. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with Subsection 105.7 of the Standard Specifications and shall include, but not be limited to the following items:

A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the designation as to the type of prefabricated module, the distance along the face of the wall to where changes in length of the units occur, the location of the original and final ground line.

B. All details, including reinforcing bar bending details, shall be provided. Bar bending details shall be in accordance with Department standards.

C. All details for foundations and leveling pads, including details for steps in the leveling pads, as well as allowable and actual maximum bearing pressures shall be provided.

D. All prefabricated modules shall be detailed. The details shall show all dimensions necessary to construct the element, and all reinforcing steel in the element.

E. The wall plans shall be prepared and stamped by a Professional Engineer. Four sets of design drawings and detail design computations shall be submitted to the Resident.

F. Four weeks prior to the beginning of construction, the contractor shall supply the Resident with two copies of the design-supplier's Installation Manual. In addition, the Contractor shall have two copies of the Installation Manual on the project site.

635.05 Construction Requirements

Excavation. The excavation and use as fill disposal of all excavated material shall meet the requirements of Section 203 -- Excavation and Embankment, except as modified herein.

Foundation. The area upon which the modular gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the module. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density, determined using AASHTO T180, Method C or D. Frozen soils and soils unsuitable or incapable of sustaining the required compaction, shall be removed and replaced.

A concrete leveling pad shall be constructed as indicated on the plans. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Resident. Allowable elevation tolerances are +0.01 ft and -0.02 ft from the design elevations. Leveling pads which do not meet this requirements shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after 24 hours curing time of the concrete leveling pad.

Method and Equipment. Prior to erection of the Prefabricated Concrete Modular Gravity Wall, the Contractor shall furnish the Resident with detailed information concerning the proposed construction method and equipment to be used. The erection procedure shall be in accordance with the manufacturer's instructions. Any pre-cast units that are damaged due to handling will be replaced at the Contractor's expense.

Installation of Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

The maximum offset in any unit joint shall be 3/4 in. The overall vertical tolerance of the wall, plumb from top to bottom, shall not exceed 1/2 in per 10 ft of wall height. The prefabricated wall units shall be installed to a tolerance of plus or minus 3/4 inch in 10 ft in vertical alignment and horizontal alignment.

Select Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The maximum lift thickness shall be 8 in. (loose). Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the wall units. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T180, Method C or D. At the end of the day’s operations, the Contractor shall shape the last level of backfill so as to direct runoff of rain water away from the wall face.

635.06 Method of Measurement. Prefabricated Concrete Modular Gravity Wall will be measured by the square meter of front surface not to exceed the dimensions shown on the contract plans or authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the facing units. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the plans.

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square foot complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Prefabricated Concrete Modular Gravity Wall.

There will be no allowance for excavating and backfilling for the Prefabricated Concrete Modular Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation, as approved by the Resident. Payment for excavating unsuitable material shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work.

Payment will be made under:

Pay Item	Pay Unit
635.14 Prefabricated Concrete Modular Gravity Wall	Square Foot