MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement/Rehabilitation of:

NEW MILLS BRIDGE
OVER COBBOSSEE STREAM
STATE ROUTE 126
GARDINER, MAINE

Prepared by:

Kathleen Maguire, P.E.
Geotechnical Engineer

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Kennebec County
PIN 12633.00

Soils Report No. 2008-05

Federal No. AC-BR-1263(300)X
April 2008

Table of Contents

Sheets

Appendices

GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement/rehabilitation of New Mills Bridge on State Route 126 over Cobbossee Stream in Gardiner, Maine. The proposed replacement bridge will consist of a 122 foot long, single span, welded plate girder founded on pile supported rehabilitated abutments directly in front of the existing abutments. The following design recommendations are discussed in detail in the attached report:

Foundation Alternatives - The following abutment foundations are viable: pile supported integral abutments behind the existing abutments; new abutments founded on spread footings; or rehabilitated abutments consisting of a new abutment face tied into and integral to the existing abutments. Any of these abutment alternatives must be engineered to satisfy all relevant structural and geotechnical strength, service and extreme limit states in accordance with AASHTO LRFD Bridge Design Specifications, 4^{th} Edition 2007 (herein referred to as LRFD).

Abutment H-piles - The structural designer has elected to leave the existing mass concrete abutments in place and drive H-piles in front of the existing abutment face to support the new supertructure. The existing abutments and new H-piles will be surrounded by stay-in-place sheet pile filled with concrete. It is recommended that a cross-tie and anchored deadman system be used to integrate the new abutment section with the old abutment section in the rehabilitation of the existing abutments. The bridge loads will be transferred to the new portion of the rehabilitated abutment. The H-piles should be end bearing, driven to the required resistance on or within the bedrock. <u>The design of the piles at the strength limit state shall consider the geotechnical and structural resistance of the piles and the loss of lateral support due to scour at the design flood event.</u> Piles should be fitted with driving points to protect the tips and improve penetration. Using the assumption that 50 ksi steel will be used; factored axial geotechnical resistance is less than the factored axial structural resistance. Therefore, the factored axial geotechnical resistance governs the design. The Contractor is required to perform a wave equation analysis and dynamic pile 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. The new abutment facing will provide lateral stability to the old abutment section and will support all bridge loads. Therefore, the new abutment facing shall be engineered to independently satisfy all relevant strength, service and extreme limit states. The new abutment facing shall be checked for stability and strength at the strength and service limit states considering the consequences from the design flood for scour.

Abutment Rehabilitation - It is recommended that a cross-tie and anchored deadman system be used to integrate the new abutment face with the old abutment section in the rehabilitation of the existing abutments. The bridge loads will be transferred to the new portion of the rehabilitated abutment. It is recommended that four anchored deadmen located equidistant between the five piles be used with a minimum of two cross ties which will fully penetrate the existing abutment. The deadman anchor system should be engineered to provide sufficient stability to the new abutment section when lateral pressures from the old abutment section and the new bridge superstructure loads are applied.

Scour and Riprap - The New Mills Bridge is located approximately 100 feet upstream of the New Mills Dam which maintains a water level of approximately 136 feet at the dam. An extensive hydrologic investigation was not conducted for the project due to the presence of the dam and the controlled water elevation. Stone riprap will be placed along the southwest approach in order to protect the stream bank.

In general, for new bridge substructures supported on piles, the seal should extend to a depth consistent with the design super flood scour elevation, and piles should achieve fixity below the design scour depth. The designer should check that there is enough pile length below the scour line to provide lateral stability and enough structural resistance to support the bridge loads. Since sheet pile is not considered a permanent scour countermeasure, maintaining pile fixity for the scour event will likely require socketing the piles in bedrock.

Settlement - The grades of the existing bridge approaches will not be raised in the construction of the proposed bridge; therefore, post-construction settlements are anticipated to be less than 0.5 inches and will occur during construction having negligible effect of the finished structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will also be negligible.

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. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively. The New Mills Bridge is located on State Route 126 and is not on the National Highway System (NHS). Therefore, the bridge is not considered to be functionally important. Since the bridge construction costs will not exceed $10 million, the bridge is not classified as a major structure. In conformance with the MaineDOT Bridge Design Guide, these criteria eliminate the requirement to design the bridge substructures for seismic earth loads.

Construction Considerations - Construction activities will include internally braced cofferdams at each abutment. Cofferdam structures will be needed for abutment rehabilitation as the proposed bottom of abutment elevations are below the stream level. It is possible that obstructions may be encountered during cofferdam construction. These obstructions may be cleared using conventional or other excavation methods. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that the abutments are constructed in the dry.

1.0 INTRODUCTION

A subsurface investigation for the replacement of New Mills Bridge on State Route 126 over Cobbossee Stream in Gardiner, Kennebec 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.

The bridge was constructed in 1908 as a railroad trolley bridge. It was acquired by the Maine Department of Transportation and in 1947 was widened and an open gird deck was added to accommodate vehicular traffic. The bridge is a single span steel through truss supported on mass concrete abutments on bedrock and timber piles. The bridge is approximately 100 feet upstream of the New Mills Dam. The bridge is generally in fair condition and is considered functionally obsolete. It is understood that the existing bridge superstructure will be completely removed and replaced.

The proposed bridge will consist of a single, 122-foot span welded plate girder with a concrete deck founded on rehabilitated pile supported abutments constructed directly in front of and tied into the existing abutments. The existing mass concrete abutments are to be left in place with H-piles driven in front of the existing abutment face to support the new structure. The existing abutments and new H-piles will be surrounded by stay-in-place sheet pile filled with concrete. The alignment of the proposed bridge will be unchanged from the existing alignment. The proposed bridge width is less than the State Standards in order to match existing corridor width. The existing bridge will be closed to traffic during construction.

2.0 GEOLOGIC SETTING

New Mills Bridge on Route 126 in Gardiner crosses Cobbossee Stream approximately 0.04 miles east of Harrison Avenue as shown on *Sheet 1 - Location Map* found at the end of this report. Cobbossee Stream flows in a northeasterly direction to the Kennebec River.

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 may contain 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. Additional geologic units mapped nearby the site are till deposits (sand, silt, clay and stones) and eskers (gravel and sands).

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as Ordovician-Precambrian age mafic (dark colored) to felsic (light colored) volcanic rocks.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling six (6) test borings at the site. Test borings BB-CS.GAR-101, BB-CS.GAR-101A, BB-CS.GAR-101B and BB-CS.GAR-101C were drilled behind the location of the west abutment. Test boring BB-CS.GAR-103 was drilled behind the location of the east abutment. Test boring BB-CS.GAR-201 was drilled at the location of a possible pier. Test boring BB-CS.GAR-102 was eliminated during drilling activities due to traffic control issues. The exploration locations are shown on *Sheet 2 - Boring Location Plan* and an interpretive subsurface profile depicting the site stratigraphy is show on *Sheet 3 - Interpretive Subsurface Profile* both found at the end of this report. Borings BB-CS.GAR-101, BB-CS.GAR-101A, BB-CS.GAR-101B, BB-CS.GAR-101C and BB-CS.GAR-103 were drilled between June 15 and 22, 2006. Boring BB-CS.GAR-201 was drilled on November 5, 2007. All of the borings were drilled by 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 *Sheets 4 and 5- Boring Logs* found end of this report.

The borings were drilled using driven cased and spun cased wash boring and solid stem auger 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 newly equipped with a CME automatic hammer to drive the split spoon. The hammer was calibrated by MaineDOT in August of 2007 and was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. This new hammer system was used when drilling boring BB-CS.GAR-201 in November of 2007. All N-values relating to this boring discussed in this report are corrected values computed by applying an average energy transfer factor of 0.77 to the raw field N-values. This hammer efficiency factor (0.77) and both the raw field N-value and the corrected N-value are shown on the boring logs. SPT sampling and testing for the remainder of the borings drilled for this project was performed with a standard rope and cathead system. N-values obtained using the rope and cathead system do not require correction as the field values are equivalent to corrected N-values.

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 selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field and laboratory testing requirements. The MaineDOT geotechnical team member and/or 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 two (2) standard grain size analyses and three (3) grain size analysis with hydrometer. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on *Sheets 4 and 5 - Boring Logs* found at the end of this report.

5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the abutments consisted of fill soils and concrete overlying sands, silts and clays all overlying bedrock. In the streambed gravel, silts and sands were encountered. An interpretive subsurface profile depicting the site stratigraphy is show on *Sheet 3 - Interpretive Subsurface Profile* found at the end of this report. Refer to the boring logs in Appendix A for detailed documentation of the conditions encountered in each boring. The following paragraphs discuss the subsurface conditions encountered at each proposed substructure in detail:

Abutment No. 1 (west): Beneath the pavement, concrete and fill soils were encountered. The concrete encountered is the existing abutment concrete and was observed to be in good to poor condition. Fill soils were encountered interbedded with the concrete in the upper approximately 5 feet. The fill soils were found to be brown and grey, damp, cobbles and gravel with fine to coarse sand and trace silt. The depth to the bottom of the concrete ranged from approximately 18.3 to 18.4 feet below ground surface (bgs) in borings BB-CS.GAR-101C and BB-CS.GAR-101, respectively.

The existing concrete abutment is underlain by interbedded native sand and clay. The upper sand deposit was found to be brown to grey, wet, silty fine to coarse sand with trace gravel. The thickness of the upper sand layer ranged from approximately 2.4 to 9.3 feet in borings BB-CS.GAR-101 and BB-CS.GAR-101C, respectively. One SPT N-value obtained in the upper sand was 2 blows per foot (bpf) indicating that the upper sand is very loose in consistency. A water content obtained from a sample of the upper sand was approximately 14%. A grain size analysis conducted on a sample of the upper sand indicates that the soil is classified as an A-4 by the AASHTO Classification System and a SM by the Unified Soil Classification System.

At the location of boring BB-CS.GAR-101 the upper sand layer was underlain by a layer of clay. This clay layer was not noted in boring BB-CS.GAR-101C. The clay layer was found to be grey, wet, silty clay with trace fine to coarse sand and gravel. The thickness of the clay layer was approximately 4.2 feet. One SPT N-value obtained in the clay was 16 bpf indicating that the clay is very stiff in consistency. A water content obtained from a sample of the clay was approximately 30%. A grain size analysis conducted on a sample of the clay indicates that the soil is classified as an A-6 by the AASHTO Classification System and a CL by the Unified Soil Classification System.

The upper sand and clay layers are underlain by a lower sand layer. The lower sand deposit was found to be grey, moist, silty fine to coarse sand with cobbles and boulders and little gravel. The thickness of the lower sand layer was not fully penetrated in boring BB-CS.GAR-101. The thickness of the lower sand layer was approximately 24.6 feet in boring BB-CS.GAR-101C. Two attempted SPT tests within this indicated that the layer is very dense with SPT N-values in excess of 50 bpf.

Bedrock was encountered in boring BB-CS.GAR-101C at a depth of approximately 52.2 feet bgs which corresponds to an approximate elevation of 93.9 feet. The bedrock is identified as augen gneiss transitioning to granite at approximate elevation 85 feet. The augen gneiss is white, grey, black and green coarse-grained, moderately hard, moderately weathered with steep to vertical foliation. The granite is described as light grey, pegmatite granite, massive, slightly weathered and very hard. The RQD of the bedrock ranged from 36 to 80% indicating a rock of poor to good quality. Overall, the bedrock was found to be sloping down to the west.

Groundwater was observed in boring BB-CS.GAR-101 at a depth of approximately 18.4 feet bgs (approximate elevation 127.7 feet). This groundwater level is presumed to be influenced by the presence of the New Mills Dam located approximately 100 feet down stream. The Gardiner Water District maintains the water level at the dam at approximate elevation 136 feet. The water level at the abutment is anticipated to fluctuate seasonally depending upon the local precipitation magnitudes.

Abutment No. 2 (east): Beneath the pavement, concrete and fill soils were encountered. The concrete encountered is the existing abutment concrete and was observed to be in good to poor condition. Fill soils were encountered interbedded with the concrete in the upper approximately 6 feet. The fill soils were found to be brown, damp, dense, cobbles and gravel with fine to coarse sand and trace silt. The depth to the bottom of the concrete was approximately 18.6 bgs in boring BB-CS.GAR-103.

The existing concrete abutment is underlain a sand layer. The sand deposit was found to be grey, moist to wet, fine to coarse sand with some silty and little gravel. The thickness of the sand layer was approximately 10.2 feet in borings BB-CS.GAR-103. SPT N-values obtained in the sand ranged from weight of rods (WOR) to 14 bpf indicating that the sand is very loose to medium dense in consistency. A water content obtained from a sample of the sand was approximately 41%. A grain size analysis conducted on a sample of the sand indicates that the soil is classified as an A-2-4 by the AASHTO Classification System and a SM by the Unified Soil Classification System.

Bedrock was encountered in boring BB-CS.GAR-103 at a depth of approximately 28.8 feet bgs which corresponds to an approximate elevation of 117.2 feet. The bedrock is identified as augen gneiss and is white, grey, black and green, coarse-grained, moderately hard, moderately weathered with steep to vertical foliation. The RQD of the bedrock ranged from 40 to 68% indicating a rock of poor to fair quality. Overall, the bedrock was found to be sloping down to the west.

Groundwater was observed in boring BB-CS.GAR-103 at a depth of approximately 18.6 feet bgs (approximate elevation 127.4 feet). This groundwater level is presumed to be influenced by the presence of the New Mills Dam located approximately 100 feet down stream. The

Gardiner Water District maintains the water level at the dam at approximate elevation 136 feet. The water level at the abutment is anticipated to fluctuate seasonally depending upon the local precipitation magnitudes.

Center Pier. Boring BB-CS.GAR-201 was drilled at the center of the existing bridge span in the event that a center pier was necessary in the replacement of the bridge. In the streambed gravel, silts and sands were encountered. The gravel encountered was found to be dark grey, wet, sandy gravel with trace silt and clay. The thickness of the gravel layer was approximately 6.0 feet. A SPT N-value obtained in the gravel was 23 bpf indicating that the sand is medium dense in consistency. A water content obtained from a sample of the gravel was approximately 20%. A grain size analysis conducted on a sample of the gravel indicates that the soil is classified as an A-1-a by the AASHTO Classification System and a GP-GC by the Unified Soil Classification System.

The gravel is underlain a silt layer which was found to be grey, wet, silt with some fine to coarse sand, little clay and trace gravel. The thickness of the silt layer was approximately 3.0 feet. A SPT N-value obtained in the silt was 67 bpf indicating that the silt is hard in consistency. A water content obtained from a sample of the silt was approximately 11%. A grain size analysis conducted on a sample of the sand indicates that the soil is classified as an A-4 by the AASHTO Classification System and a CL-ML by the Unified Soil Classification System.

The silt is underlain a thin gravel layer over the bedrock. The thickness of the gravel layer was approximately 1.1 feet. The gravel was not sampled but was only identified in the drilling wash water.

Bedrock was encountered at a depth of approximately 10.1 feet below the streambed which corresponds to an approximate elevation of 107.2 feet. The bedrock is identified as schist transitioning to augen gneiss at approximate elevation 105.8 feet. The upper bedrock is described as grey, black and white, coarse grained, metamorphic schist, very decomposed with iron staining, mica and pyrite. The augen gneiss is white, grey, black and green coarse-grained, moderately hard, moderately weathered with steep to vertical foliation. The RQD of the bedrock ranged from 73 to 82% indicating a rock of fair to good quality. Overall, the bedrock was found to be sloping down to the west.

The water level in Cobbossee Stream is controlled by the New Mills Dam located approximately 100 feet down stream. The Gardiner Water District maintains the water level at the dam at approximate elevation 136 feet.

6.0 FOUNDATION ALTERNATIVES

The subsurface conditions encountered at the site indicate that the bridge location is underlain by approximately 29 feet of soil at the east end and approximately 52 feet of soil at the west end. Due to the nature and depth of the soils, the following foundation alternatives are viable:

- Abutment alternatives:
  1. Pile supported integral abutment behind existing abutments
  2. Abutments supported on spread footings
  3. Rehabilitated abutments

- Pier alternatives:
  1. Rock socketed pipe pile pier bents
  2. Drilled shaft supported pier bent
  3. Mass concrete pier on bedrock

The Preliminary Design Report (PDR) prepared for the project considers both bridge rehabilitation and bridge replacement. Bridge replacement alternatives considered two possibilities: a two span structure founded on pile supported integral abutments behind the existing U-shaped, mass concrete abutments and a single span structure founded on rehabilitated abutments.

The recommended alternative chosen in the PDR is to replace the bridge with a single span structure founded on rehabilitated pile supported abutments constructed directly in front of the existing abutments. The existing mass concrete abutments are to be left in place with H-piles driven in front of the existing abutment face to support the new structure. The existing abutments and new H-piles will be surrounded by stay-in-place sheet pile filled with concrete. It is recommended that a cross-tie and anchored deadman system be used to tie the new abutment faces to the existing abutments, and that the composite 'new-plus-old' abutment section be engineered to satisfy all LRFD strength, service and extreme limit states.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for the rehabilitation of the existing U-shaped concrete abutments at the site. The replacement bridge superstructure will be founded on rehabilitated pile supported abutments constructed directly in front of the existing abutments. The existing mass concrete abutments are to be left in place with H-piles driven in front of the existing abutment face to support the new structure. The existing abutments and new H-piles will be surrounded by stay-in-place sheet pile filled with concrete. It is recommended that a cross-tie and anchored deadman system be used in the rehabilitation of the existing abutments. The bridge loads will be transferred to the new portion of the rehabilitated abutment.

7.1 Abutment H-piles

The rehabilitated pile supported abutments will consist of new abutment sections constructed directly in front of the existing abutments. The existing mass concrete abutments are to be left in place with H-piles driven in front of the existing abutment face to support the new substructure and superstructure loads. The existing abutments and new H-piles will be surrounded by stay-in-place sheet pile filled with concrete. It is recommended that a cross-tie and anchored deadman system be used in the rehabilitation of the existing abutments. The bridge loads will be transferred to the new portion of the rehabilitated abutment.

The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads, shear loads and bending moments. 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 following data:

Location	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Rock Quality Designation	Estimated Pile Length
Abutment #1 BB-CS.GAR-101C	52.2 feet	93.9 feet	36 - 80%	47 feet
Abutment #2 BB-CS.GAR-103	28.8 feet	117.2 feet	40 - 68%	24 feet

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

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Therefore, to maintain pile fixity during a scour event, socketing the H-pile into bedrock will likely be required and is recommended. (Sheet pile is not considered a permanent scour countermeasure.) 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.

Convention usually dictates that lateral loads be resisted by nattered pile. Since the abutment piles will be plumb and will be subjected to lateral loading, the piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2.

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. The H-piles are fully embedded and λ shall be taken as 0. The factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods and the FHWA computer program Driven. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45 for both end bearing and skin friction.

The drivability of the four proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. In accordance with LRFD Table 10.5.5.2.3-1 the resistance factor for a single pile in axial compression when a dynamic test is done 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 axial compressive factored structural and geotechnical resistances of the four proposed H-pile sections for both abutments are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Abutment Piles at the Strength Limit State

Pile Section	Factored Resistance (kips)		
	Structural Resistance	Geotechnical Resistance	Design Resistance
12 x 53	465	188	188
14 x 73	642	248	248
14 x 89	783	302	302
14 x 117	1032	397	397

The factored axial geotechnical resistance is less than the factored axial structural resistance and therefore, the factored axial geotechnical resistances govern the design.

The pile will be subjected to both axial and lateral loads, therefore, combined compression and flexure analysis is required for the portion of the pile above the point of fixity, as defined in LRFD Article C6.15.2. Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation.

For the strength limit state, the combined axial compression and flexure should be evaluated as shown in LRFD Article 6.9.2.2. The structural designer should evaluate the capacity of the pile in combined axial load and flexure when the loads and moments are calculated.

7.1.2 Service and Extreme Limit States

For the service and extreme limit states resistance factors of 1.0 are recommended for axial structural, geotechnical and drivability pile resistances.

The calculated axial factored structural and geotechnical resistances of the four proposed H-pile sections for both abutments are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States

Pile Section	Factored Resistance (kips)		
	Structural Resistance	Geotechnical Resistance	Design Resistance
12 x 53	775	418	418
14 x 73	1070	552	552
14 x 89	1305	672	672
14 x 117	1720	883	883

The factored axial geotechnical resistance is less than the factored axial structural resistance and therefore, the factored axial geotechnical resistance governs the design.

7.1.3 Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.52. The factored pile load should be shown on the plans. If three to four piles are dynamically tested, the resistance factor may be increased by 20 percent to 0.65. Calculations for the pile resistance required by a drivability wave equation analysis are included the Appendix C- Calculations.

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 Abutment Rehabilitation

The rehabilitated pile supported abutments will consist of new abutment sections constructed directly in front of the existing abutments. The existing mass concrete abutments are to be left in place with H-piles driven in front of the existing abutment face to support the new structure. The existing abutments and new H-piles will be surrounded by stay-in-place sheet pile filled with concrete. It is recommended that a cross-tie and anchored deadman system be used in the rehabilitation of the existing abutments. The bridge loads will be transferred to the new portion of the rehabilitated abutment.

It is recommended that four anchored deadmen located equidistant between the five piles be used with a minimum of two cross-ties which will fully penetrate the existing abutment. A preliminary cross-tie and anchored deadman configuration is shown in Figure 1, below. The deadman anchor system should be engineered to provide sufficient stability to the new abutment section when lateral pressure from the old abutment section and the new bridge superstructure loads are applied.

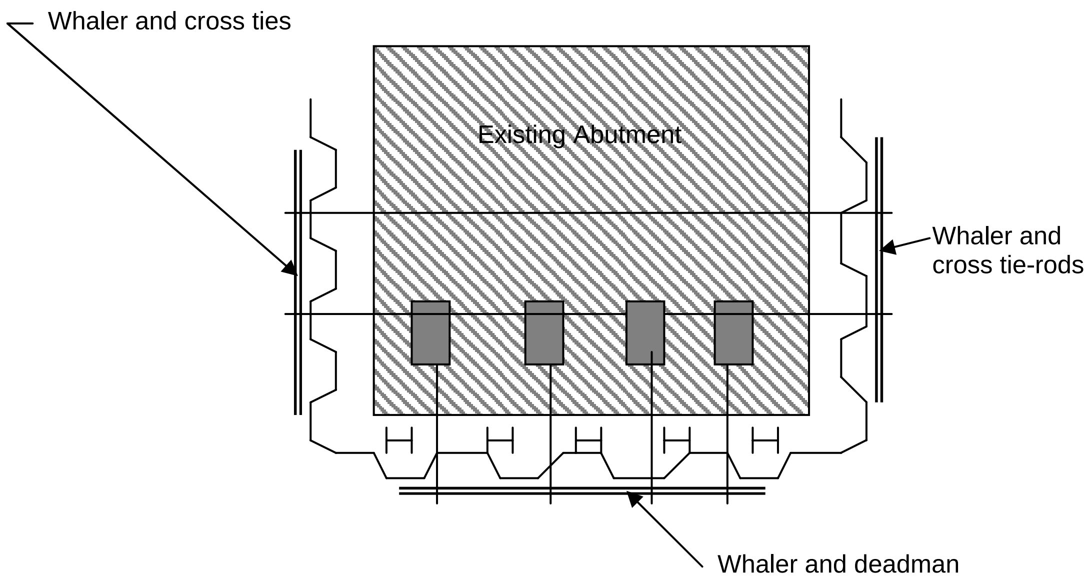


FIGURE 1 - Preliminary Cross-tie and Anchored Deadman Configuration

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT Bridge Design Guide (BDG) for the abutments and walls if an approach slab is not specified. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Abutment or Wall Height (feet)	Retaining Walls h_{eq} (feet): Distance from wall backface to edge of traffic = 0 feet	Retaining Walls h_{eq} (feet): Distance from wall backface to edge of traffic ≥ 1 foot	Abutments h_{eq} (feet)
5	5.0	2.0	4.0
10	3.5	2.0	3.0
15	2.0	2.0	2.0

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

The following subsections discuss the different models that could apply to the new abutment sections and the left-in-place sheet piling that will be used to contain the concrete for the proposed abutment facing.

7.2.1 Cantilever Abutment Walls

If the rehabilitated abutment sections have no connection to the existing abutment then the abutment wall should be modeled as a cantilever wall free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, calculated using Rankine Theory. See *Sheet 6 - Rankine and Coulomb Active Earth Pressure Coefficients* at the end of this report for guidance in calculating this value. The Rankine active earth pressure coefficient of $K_a = 0.307$ is recommended.

The new abutment sections should be designed to independently satisfy all applicable load combinations specified in LRFD Articles3.4.1 and 11.5.5.

7.2.2 Anchored or Tied Back Abutment Walls

With the use of the recommended cross-tie and anchored deadman system the abutment wall should be modeled as an independent fixed wall not allowed to move at all. Earth loads shall be calculated using an at-rest earth pressure coefficient, K_o. The at-rest earth pressure coefficient of K_o= 0.47 is recommended.

The new abutment sections should be designed to independently satisfy all applicable load combinations specified in LRFD Articles3.4.1 and 11.5.5.

7.2.3 Sheet Piling

Sheet piling shall be designed to withstand lateral earth pressures. Earth loads shall be calculated using an active earth pressure coefficient, K_a, calculated using Rankine Theory. See *Sheet 6 - Rankine and Coulomb Active Earth Pressure Coefficients* at the end of this report for guidance in calculating this value. The Rankine active earth pressure coefficient of K_a = 0.307 is recommended. Where passive earth pressure in front of the wall can be considered a passive earth pressure coefficient, $K_{p,}$ calculated using Rankine Theory may be used. The Rankine passive earth pressure coefficient of K_p = 3.255 is recommended.

The sheet pile design section should consider a sacrificial steel loss per the MaineDOT BDG. The designer should also consider a passive cathodic protection system consisting of aluminum anodes installed on the sheet piling 2 feet below the mean low water elevation.

7.3 Scour and Riprap

The New Mills Bridge is located approximately 100 feet upstream of the New Mills Dam. The New Mills Dam is operated by the Gardiner Water District which maintains a water level of approximately 136 feet at the dam. The highest water level recorded at the dam is approximately 138.5 feet during the severe flooding in 1987. The proposed structure will narrow the channel width between the abutments by approximately 6 feet. This reduction is not anticipated to affect flows over the dam.

The PDR states that an extensive hydrologic investigation was not conducted for the project due to the presence of the dam and the controlled water elevation. As the intent is to leave the existing mass concrete abutments in place and construct new pile supported abutment sections in front of the existing it is recommended that the designer confirm that the rehabilitated abutments will be resistant to damage from a rapid draw down event.

In general, for new bridge substructures supported on piles, the seal should extend to a depth consistent with the design super flood scour elevation, and piles should achieve fixity below the design scour depth. The designer should check that there is enough pile length below the scour line to provide lateral stability and enough structural resistance to support the bridge loads. Since sheet pile is not considered a permanent scour countermeasure, maintaining pile fixity for the scour event will likely require socketing the piles in bedrock.

Stone riprap will be placed along the southwest approach in order to protect the stream bank. Riprap shall conform to item number 703.26 of the Standard Specification. The toe of the riprap section shall be constructed 1 foot below the low water elevation. The riprap shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification. The riprap slope protection should be 3 feet thick with the toe 1 foot below the streambed elevation. The bedding material should be underlain by erosion control geotextile as shown in Standard Detail 610(03).

7.4 Settlement

The bridge will be widened from the existing 24 feet to 32 feet. The widening will be to the north (downstream). The grades of the existing bridge approaches will not be raised in the construction of the proposed bridge; therefore, post-construction settlements are anticipated to be less than 0.5 inches and will occur during construction having negligible effect of the finished structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will also be negligible.

7.5 Seismic Design Considerations

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

- Peak Ground Acceleration coefficient (PGA) = 0.078g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.161g
- Long-term (1.0-second period) spectral acceleration coefficient = 0.045g

Per LRFD Article 3.10.3.1 the site is assigned to Site Class D (stiff soil) based on the average N-value obtained at the site during drilling activities. Per LRFD Article 3.10.6 the site is assigned to Seismic Zone 1 based on a calculated S_{D1} of 0.108 (LRFD Eq. 3.10.4.2-6).

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

The horizontal bedrock acceleration coefficient (A) for Gardiner is approximately 0.05g, based on Figure 3-4 of the BDG, Seismic Performance Categories for Maine, August 2003. Per BDG Section 3.7.1.1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09g are designated to Seismic Performance Category (SPC) classification A. For SPC A, no detailed analysis is required other than connection design and a minimum support length requirement, except if the bridge is functionally important or is classified as a major structure. According to Figure 2-2 of the BDG, New Mills Bridge is not on the National Highway System (NHS) and is therefore not considered to be functionally important, and since the bridge construction costs should not exceed $10 million the bridge is not classified as a major structure.

7.6 Construction Considerations

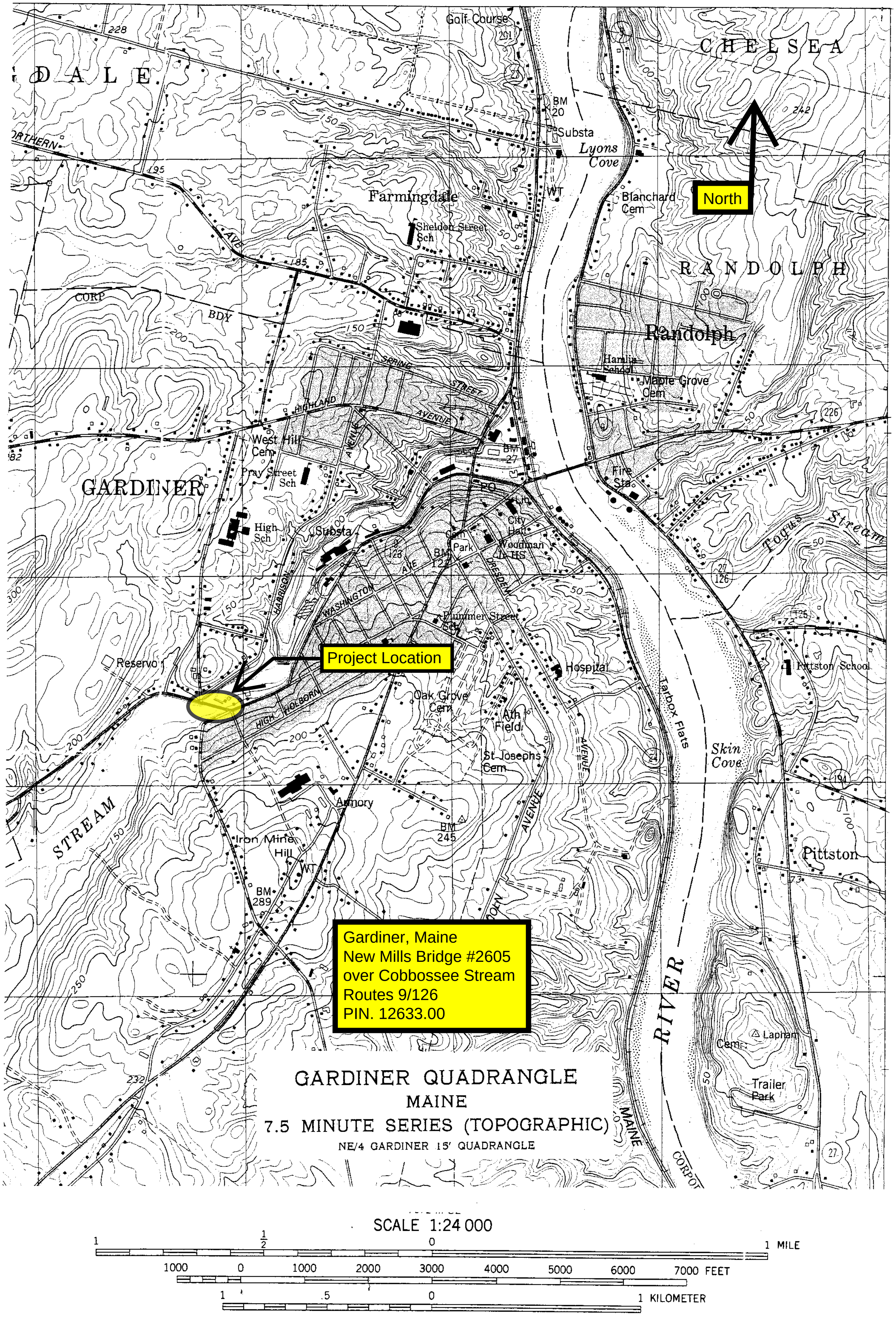
Construction activities will include internally braced cofferdams at each abutment. Cofferdam structures will be needed for abutment rehabilitation as the proposed bottom of abutment elevations are below the stream level and seals will be required. It is possible that obstructions may be encountered during cofferdam construction. These obstructions may be cleared using conventional excavation methods. Water should be controlled by pumping from sumps. The contractor should maintain the cofferdam and seal so that the abutments are constructed in the dry.

8.0 CLOSURE

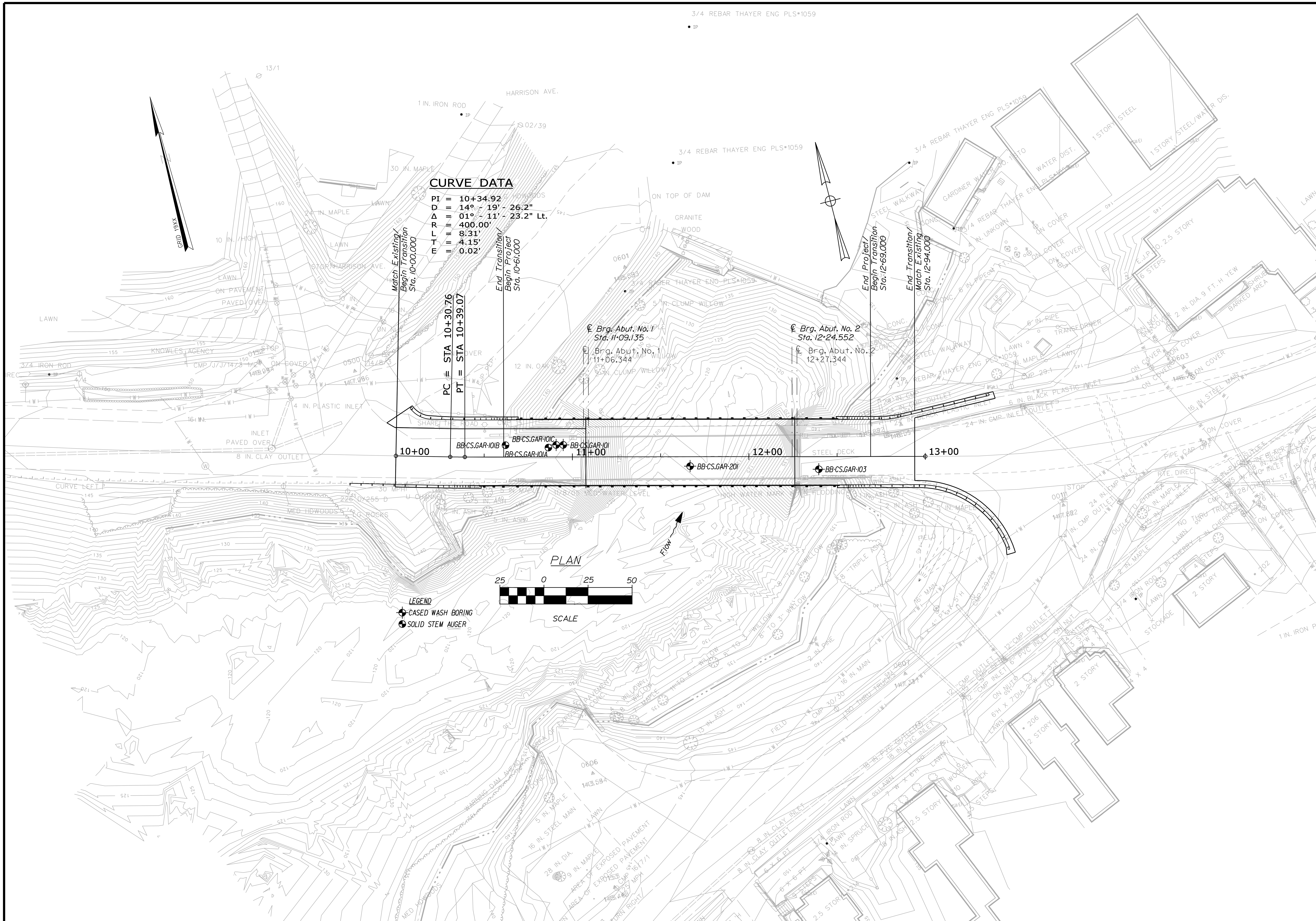
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the New Mills Bridge in Gardiner, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is implied. In the event 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


North
Project Location
Gardiner, Maine
New Mills Bridge #2605
over Cobbossee Stream
Routes 9/126
PIN. 12633.00
GARDINER QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)
NE/4 GARDINER 15' QUADRANGLE
SCALE 1:24 000
1 MILE
7000 FEET
1 KILOMETER
Farmingdale
Randolph
GARDINER
Pittston
CHELSEA
RANDOLPH
STREAM
RIVER
Golf Course
Lyons Cove
Blanchard Cem
Sheldon Street Sch
Hamlin School
Maple Grove Cem
West Hill Cem
Pray Street Sch
High Sch
Substa
Fire Sta
City Hall
Woodman HS
Park
Plummer Street Sch
Hospital
Oak Grove Cem
Ath Field
St Josephs Cem
Reservoir
Armory
Iron Mine Hill
Togus Stream
Tarbox Flats
Skin Cove
Pittston School
Lapham
Cem
Trailer Park

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:4/17/2008



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1263(300)X

PIN 12633.00

BRIDGE NO. 2605

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	T. WHITE	AUG 2006
CHECKED-REVIEWED	K. MAGUIRE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

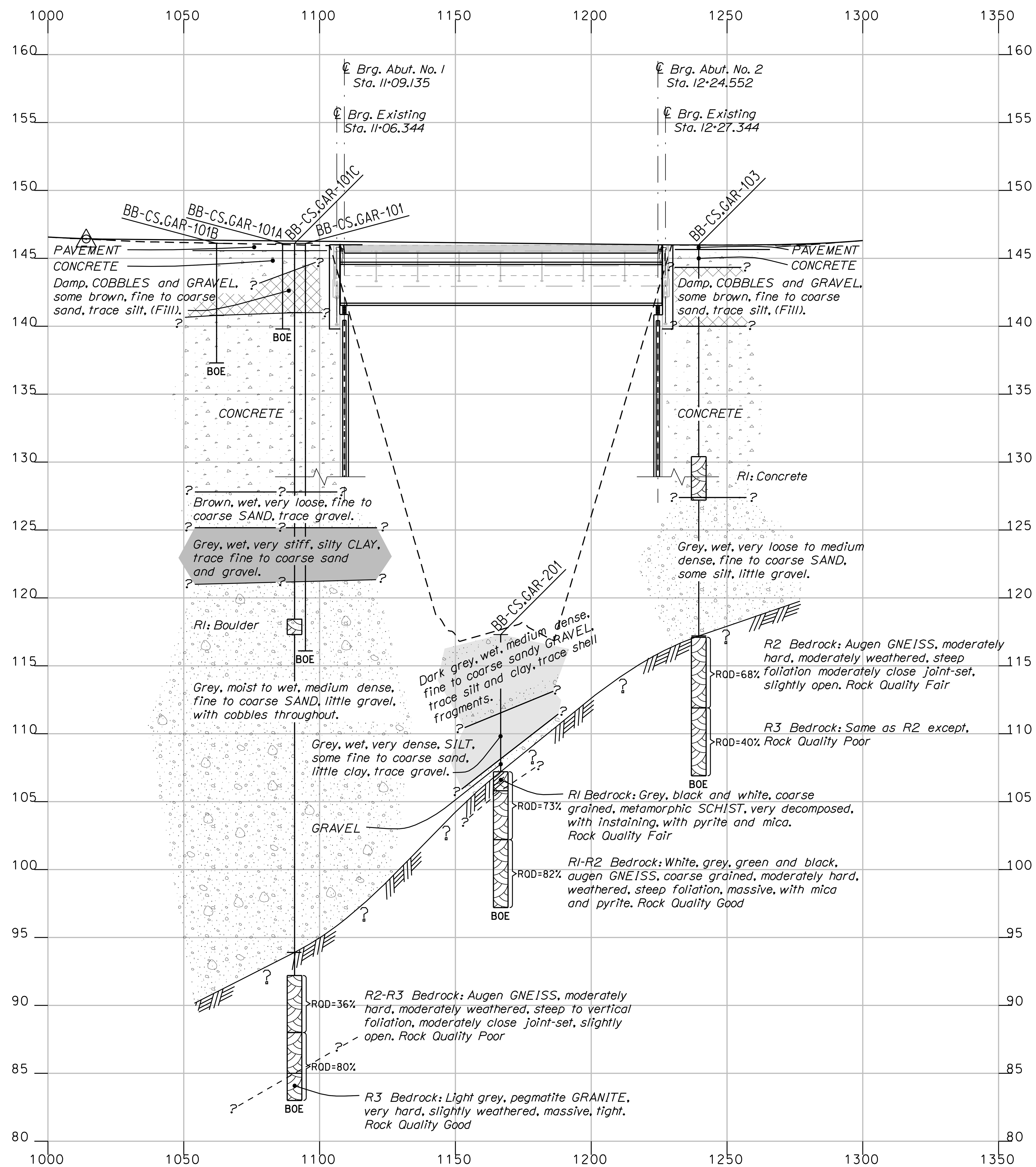
DATE

NEW MILLS BRIDGE
COBBOSSEE STREAM
GARDINER KENNEBEC COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 6

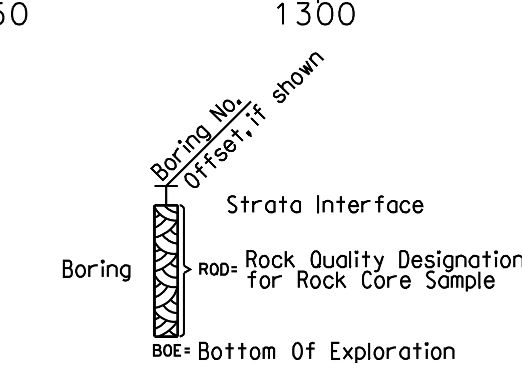
Filename: ...\00\geotech\msta\007_ISP1.dgn   Division: GEOTECH   Username: terry.white   Date:4/17/2008



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.

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1263(300)X

PIN 12633.00

BRIDGE PLANS

BRIDGE NO. 2605

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	AUG 2006
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

NEW MILLS BRIDGE
COBBOSSEE STREAM
GARDINER KENNEBEC COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 6

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:4/17/2008

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

Project: New Mills Bridge #2605 over Cobbossee Stream	Location: Gardiner, Maine	Boring No.:	BB-CS.GAR-101B	PIN:	12633.00
Driller:	MaineDOT	Elevation (ft.)	146.1	Auger ID/OD:	5" Dia.
Operator:	E. Giguere/B. Hyland	Datum:	NAVD 88	Sampler:	N/A
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	6/20/06; 14:00-15:00	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	10+62.1, 6.7 Lt.	Casing ID/OD:	N/A	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	145.60		Pavement 0.50	
									No description given.	
5							140.80		CONCRETE 5.30	
							137.30		8.80 Bottom of Exploration at 8.80 feet below ground surface. STOPPED IN CONCRETE	
10										
15										
20										
25										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CS.GAR-101B

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

Project: New Mills Bridge #2605 over Cobbossee Stream	Location: Gardiner, Maine	Boring No.:	BB-CS.GAR-101A	PIN:	12633.00
Driller:	MaineDOT	Elevation (ft.)	146.1	Auger ID/OD:	5" Dia.
Operator:	E. Giguere/B. Hyland	Datum:	NAVD 88	Sampler:	N/A
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	6/20/06; 13:00-14:00	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	10+86.4, 5.3 Lt.	Casing ID/OD:	N/A	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	145.70		Pavement 0.40	
									CONCRETE	
							144.00		2.10 Damp, COBBLES and GRAVEL, some brown, fine to coarse sand, trace silt, (Fill).	
5							141.30		4.80 CONCRETE	
							140.60		5.50 GRAVEL, some brown, damp, fine to coarse sand, trace silt, (Fill).	
							140.30		5.80 CONCRETE	
							139.80		6.30 Bottom of Exploration at 6.30 feet below ground surface. STOPPED IN CONCRETE	
10										
15										
20										
25										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CS.GAR-101A

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

Project: New Mills Bridge #2605 over Cobbossee Stream	Location: Gardiner, Maine	Boring No.:	BB-CS.GAR-101C	PIN:	12633.00
Driller:	MaineDOT	Elevation (ft.)	146.1	Auger ID/OD:	N/A
Operator:	E. Giguere/B. Hyland	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30", 300#/16"
Date Start/Finish:	6/22/06; 08:00-17:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	10+90.8, 6.8 Lt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	C1	16.8/ 16.8	0.00 - 1.40	5" thin wall hand held core machine.			145.85		Pavement 0.25	
									CONCRETE	
	C2	4.8/4.8	1.40 - 1.80			SPUN HW	144.30		1.80 Damp, COBBLES and GRAVEL, some brown, fine to coarse sand, trace silt, (Fill).	
5							141.00		5.10 CONCRETE	
10						SPUN NW				
15										
							127.80		18.30 Brown, wet, silty fine to coarse SAND, trace gravel.	
20							125.10		21.00	
25	1D	24/7	25.00 - 27.00	8/7/12/8	19				Grey, wet, medium dense, silty fine to coarse SAND, little gravel.	G#180046 A-4, SM WC=14.4%
							118.50		27.60 R1: Boulder	
	R1	19.2/10	28.10 - 29.70	RQD = N/A%		NQ	117.20		28.90	
30										
35	MD	1/0	35.00 - 35.08	No blows given					Failed sample attempt, no recovery.	
									Roller Coned ahead of casing from 39.1- 40.0' bgs.	
40	2D	9.6/6	40.00 - 40.80	53/50(3.6")	---				Grey, moist, silty fine to coarse SAND, cobbles, little gravel.	
45										
50						aRC			aRoller Coned ahead from 49.6-53.9' bgs.	
							93.90		52.20 Top of Bedrock at Elev. 93.9'	
	R2	50.4/46	53.90 - 58.10	RQD = 36%		NQ			R2: Augen GNEISS, moderately hard, moderately weathered, steep to vertical foliation, moderately close joint-set, slightly open. Rock Quality = Poor R2:Core Times (min:sec) 53.9-54.9' (2:50) 54.9-55.9' (2:07) 55.9-56.9' (2:18) 56.9-57.9' (2:18) 57.9-58.1' (0:33) 92% Recovery	
55										
	R3	60/62	58.10 - 63.10	RQD = 80%					R3: 58.1 to 61.1': Same as R2 R3:Core Times (min:sec) 58.1-59.1' (2:13) 59.1-60.1' (1:42) 60.1-61.1' (1:47)	
60							85.00		61.10 R3: 61.1 to 63.1': Light grey, pegmatite GRANITE, very hard, slightly weathered, massive, tight. Rock Quality = Good Core Times: (min:sec) 61.1-62.1' (1:47) 62.1-63.1' (2:32) 103% Recovery	
							83.00		63.10 Bottom of Exploration at 63.10 feet below ground surface.	
65										
70										
75										

Remarks:

Saftey hammers, cathead and rope.

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-CS.GAR-101C

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

Project: New Mills Bridge #2605 over Cobbossee Stream	Location: Gardiner, Maine	Boring No.:	BB-CS.GAR-101	PIN:	12633.00
Driller:	MaineDOT	Elevation (ft.)	146.1	Auger ID/OD:	5" Solid Stem Auger
Operator:	E. Giguere/B. Hyland	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30", 300#/16"
Date Start/Finish:	6-15-06 / 6-20-06	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	10+94.8, 6.8 Lt.	Casing ID/OD:	NW	Water Level*:	18.4' bgs

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	145.80		Pavement 0.30	
									CONCRETE	
							144.80		1.30 Damp, COBBLES and GRAVEL, some brown, fine to coarse sand, trace silt, (Fill).	
5							141.00		5.10 CONCRETE	
	R1	60/60	8.80 - 13.80	RQD = N/A%		aS NQ			aSpun NW Casing from 8.3-8.7' bgs., then Roller Coned to 8.8' bgs., after R1 spun casing to 10.5'bgs. R1:Core Times (min:sec) 8.8-9.8' (3:40) 9.8-10.8' (2:55) 10.8-11.8' (2:45) 11.8-12.8' (3:20) 12.8-13.8' (3:15) 100% Recovery Concrete is in good condition.	
10										
	R2	60/55	13.80 - 18.80	RQD = N/A%					R2:Core Times (min:sec) 13.8-14.8' (3:15) 14.8-15.8' (3:00) 15.8-16.8' (3:00) Concrete is in good condition to 17.0' bgs. 16.8-17.8' (2:30) 17.8-18.8' (1:45) 92% Recovery Concrete is in poor condition from 17.0- 18.4' bgs.	
15										
	1D	24/1	18.80 - 20.80	WOR/1/1/1	2	2	127.70		18.40 Brown, wet, very loose, silty fine to coarse SAND, trace gravel.	
						3				
20	2D	24/7	20.80 - 22.80	6/9/7/6	16	9	125.30		20.80 Grey, wet, very stiff, silty CLAY, trace fine to coarse sand and gravel.	G#180045 A-6, CL WC=29.7%
						25				
						32				
						30				
						23				
25	3D/AB	24/6	25.00 - 27.00	18/15/12/8	27	18	121.10		25.00 (3D/A) 25.0-26.3' bgs. Grey-brown, moist, medium dense, silty fine to coarse SAND, little gravel.	
						35	119.80		26.30 (3D/B) 26.3-27.0' bgs. Grey, moist, medium dense, fine SAND, little silt.	
						45				
						45				
						45				
30							116.10		30.00 Bottom of Exploration at 30.00 feet below ground surface. Casing broke off driving to 30.0' bgs, retrieved casing, but couldn't get back down.	
35										
40										
45										
50										

Remarks:

Saftey hammers, cathead and rope.
6/15/06; 13:30-17:00, 6/16/06; 07:45-12:30, 6/20/06; 07:45-12:00
Drilled R2 then sampled 1D and 2D. NW spinning shoe wouldn't penetrate beyond 10.5' bgs, pulled NW Casing (shoe destroyed), then reamed hole with 4" Roller Cone, inserted NW Casing with drive shoe and continued boring.

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-CS.GAR-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1263(300)X

PIN 12633.00

BRIDGE PLANS

BRIDGE NO. 2605

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	
CHECKED-REVIEWED	T. WHITE	AUG 2006
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

NEW MILLS BRIDGE
COBBOSSEE STREAM
GARDINER KENNEBEC COUNTY

BORING LOGS

SHEET NUMBER

4

OF 6

Filename: ...\msta\009_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:4/17/2008

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: New Mills Bridge #2605 over Cobbossee Stream, Routes 9/126
Location: Gardiner, Maine

Boring No.: BB-CS-GAR-201

PIN: 12633.00

Driller:	MaineDOT	Elevation (ft.)	117.3	Auger ID/OD:	N/A
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	Auto Hammer
Date Start/Finish:	11/5/07; 09:30-14:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	11+66.7, 5.3 Rt.	Casing ID/OD:	NW	Water Level*:	11.8' below Bridge Deck
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0	1D	24/4	0.00 - 2.0	3/6/12/31	18	23	a31	
							38	
							12	
							14	
5							16	
							69	111.30
	2D	24/16	6.00 - 8.0	22/25/27/40	52	67	54	
							122	
							193	108.30
10							34 bWA	107.20
	R1	60/60	10.10 - 15.1	RQD = 73%			NQ CORE	105.80
15	R2	60/60	15.10 - 20.1	RQD = 82%				
20								97.20
25								

Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0 – 6.0	a31 blows for 0.8'. Dark grey, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt and clay, trace shell fragments.	G#209978 A-1-a GP-GC WC=19.9%
6.0 – 9.0	Grey, wet, hard, SILT, some fine to coarse sand, little clay, trace gravel.	G#209979 A-4, CL-ML WC=11.0%
9.0 – 10.1	Layer of gravelly material on top of rock approximately 1.1' thick. bWashed ahead to 10.1' bgs.	
10.1 – 11.5	Top of Bedrock at Elev. 107.2' R1: Grey, black and white, coarse grained, metamorphic, SCHIST very decomposed, with iron staining, pyrite mica. Rock Quality = Fair R1:Core Times (min:sec) at 650 psi	
11.5 – 20.1	10.1-11.1' (3:14) 11.1-12.1' (2:43) 12.1-13.1' (2:59) 13.1-14.1' (3:00) 14.1-15.1' (3:37) 100% Recovery R2: White, grey, green and black, coarse grained, moderately hard, weathered, augen GNEISS, steep foliation, massive, with mica and pyrite. Rock Quality = Good R2:Core Times (min:sec) 15.1-16.1' (3:53) 16.1-17.1' (3:14) 17.1-18.1' (2:46) 18.1-19.1' (2:21) 19.1-20.1' (2:23) 100% Recovery No water return from 15.6' bgs on R2.	
20.1	Bottom of Exploration at 20.10 feet below ground surface.	

Remarks:

Boring drill through Bridge Deck.
29.2' from Bridge Deck to top of River Bed.

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-CS-GAR-201

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: New Mills Bridge #2605 over Cobbossee Stream
Location: Gardiner, Maine

Boring No.: BB-CS.GAR-103

PIN: 12633.00

Driller:	MaineDOT	Elevation (ft.)	146.0	Auger ID/OD:	5" Solid Stem Auger
Operator:	E. Giguere/B. Hyland	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30", 300#/16"
Date Start/Finish:	6/21/06; 07:45-16:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+39.6, 7.2 Rt.	Casing ID/OD:	HW & NW	Water Level*:	18.6' bgs.

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

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

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)
0						SSA	145.80
							144.50
5	1D	24/10	5.00 - 7.00	7/15/28/47	43	SPUN HW	140.00
10							
15	R1	38.4/37	15.60 - 18.80	RQD = N/A%		NQ	
	2D/AB	24/7	18.80 - 20.80	WOR/WOR/WOR/WOH	0		127.40
20						2	125.50
						2	
						8	
						13	
						7	
25	3D	24/11	25.00 - 27.00	5/6/8/15	14	21	121.00
						23	
						57	
	R2	62.4/62.4	28.90 - 34.10	RQD = 68%		a85 NQ	117.20
30							
35	R3	60/60	34.10 - 39.10	RQD = 40%			
							106.90
40							
45							
50							

Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0 – 0.20	Pavement	
0.20 – 1.50	CONCRETE	
1.50 – 6.00	Damp, COBBLES and GRAVEL, some brown fine to coarse sand, trace silt, (Fill). Bent, destroyed, split spoon on concrete.	
6.00 – 18.60	CONCRETE Roller Coned ahead from 10.0-11.0' bgs. Roller Coned ahead from 15.0-15.6' bgs. R1:CONCRETE R1:Core Times (min:sec) 15.6-16.6' (2:41) Concrete in good condition from 15.6-16.8' bgs. 16.6-17.6' (2:07) Concrete in fair to poor condition from 16.8-18.5' bgs. 17.6-18.6' (2:41) 18.6-18.8' (0:10) 97% Recovery	
18.60 – 20.50	(2D/A) 18.8-20.5' bgs. Grey, saturated, very loose, fine to coarse SAND, some silt, little gravel. Drilled R1, spun HW Casing to 20.0' bgs. , then drove NW Casing.	G#180047 A-2-4, SM WC=40.7%
20.50 – 25.00	(2D/B) 20.5-20.8' bgs. Grey, wet, very loose, silty fine to coarse SAND, trace organics.	
25.00 – 28.80	Grey, moist, medium dense, silty fine to coarse SAND, trace coarse sand, trace gravel. a85 blows for 10".	
28.80 – 39.10	Top of Bedrock at Elev. 117.2' R2: Augen GNEISS, moderately hard, moderately weathered, steep foliation, close joint-set, slightly open. Rock Quality = Fair R2:Core Times (min:sec) 28.9-29.9' (3:01) 29.9-30.9' (2:24) 30.9-31.9' (2:18) 31.9-32.9' (2:08) 32.9-33.9' (2:35) 33.9-34.1' (0:40) 100% Recovery R3: Same as R2. Rock Quality = Poor R3:Core Times (min:sec) 34.1-35.1' (2:45) 35.1-36.1' (3:20) 36.1-37.1' (1:53) 37.1-38.1' (2:06) 38.1-39.1' (2:51) 100% Recovery	
39.10	Bottom of Exploration at 39.10 feet below ground surface.	

Remarks:

Safty hammers, cathead and rope.

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-CS.GAR-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1263(300)X

BRIDGE PLANS

PIN 12633.00

BRIDGE NO. 2605

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	
CHECKED-REVIEWED	T. WHITE	AUG 2006
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

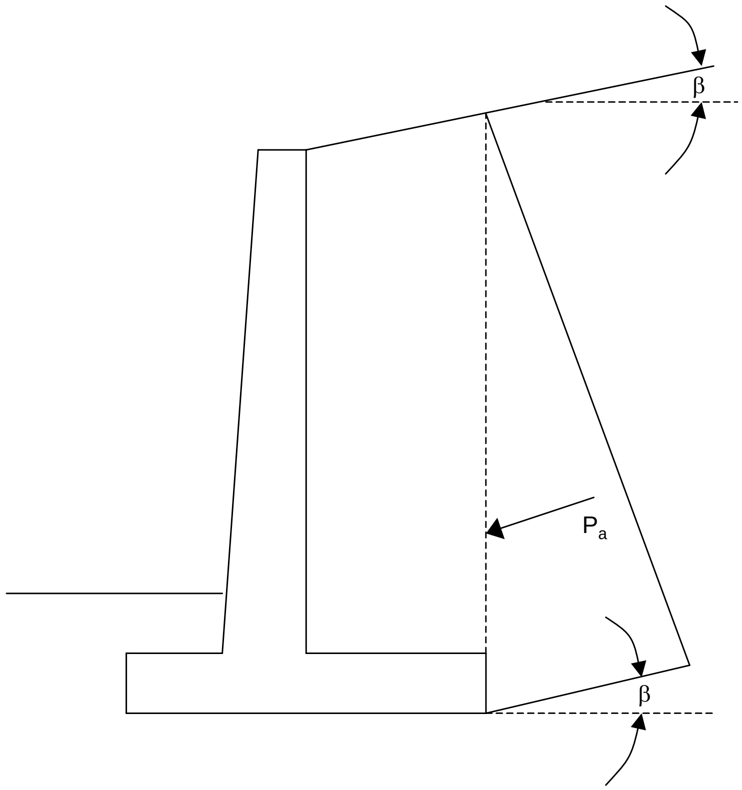
NEW MILLS BRIDGE
COBBOSSEE STREAM
GARDINER KENNEBEC COUNTY

BORING LOGS

SHEET NUMBER

5

OF 6



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

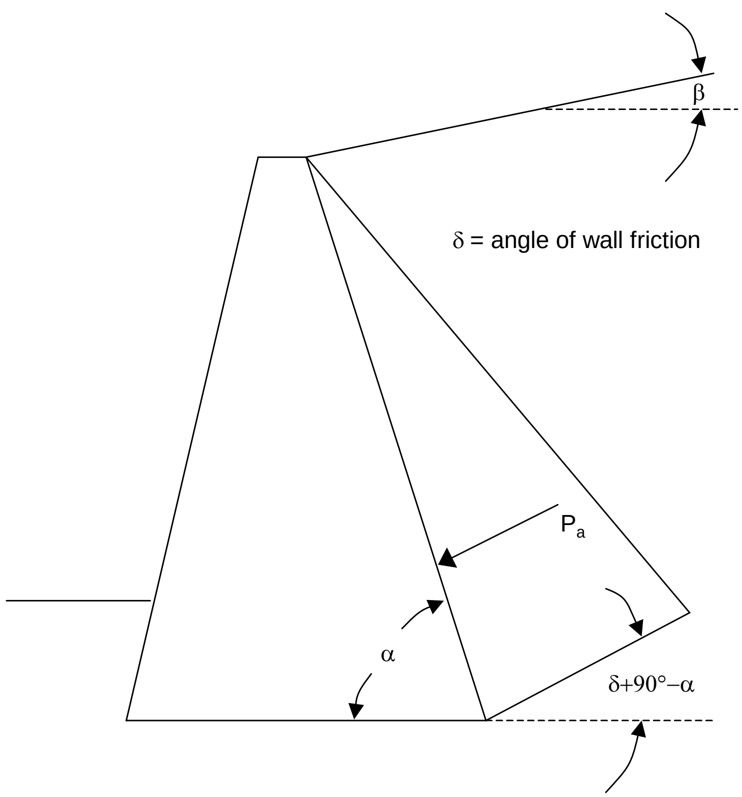
For a horizontal backfill surface, $\beta = 0°$:

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

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

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

P_a is oriented at β



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

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

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

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	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

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Mills Bridge #2605 over Cobbossee Stream
Location: Gardiner, Maine

Boring No.: BB-CS.GAR-101
PIN: 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.1	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30", 300#/16"
Date Start/Finish:	6-15-06 / 6-20-06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+94.8, 6.8 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	18.4' bgs

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	145.80		Pavement 0.3	
							144.80		CONCRETE 1.3	
									Damp, COBBLES and GRAVEL, some brown, fine to coarse sand, trace silt, (Fill).	
5							141.00		5.1 CONCRETE	
	R1	60/60	8.8 - 13.8	RQD = N/A%		aS NQ			aSpun NW Casing from 8.3-8.7' bgs., then Roller Coned to 8.8' bgs., after R1 spun casing to 10.5'bgs. R1:Core Times (min:sec) 8.8-9.8' (3:40) 9.8-10.8' (2:55) 10.8-11.8' (2:45) 11.8-12.8' (3:20) 12.8-13.8' (3:15) 100% Recovery Concrete is in good condition.	
10										
	R2	60/55	13.8 - 18.8	RQD = N/A%					R2:Core Times (min:sec) 13.8-14.8' (3:15) 14.8-15.8' (3:00) 15.8-16.8' (3:00) Concrete is in good condition to 17.0' bgs. 16.8-17.8' (2:30) 17.8-18.8' (1:45) 92% Recovery Concrete is in poor condition from 17.0- 18.4' bgs.	
15										
	1D	24/1	18.8 - 20.8	WOR/1/1/1	2	2	127.70		18.4 Brown, wet, very loose, silty fine to coarse SAND, trace gravel.	
						3				
20	2D	24/7	20.8 - 22.8	6/9/7/6	16	9	125.30		20.8 Grey, wet, very stiff, silty CLAY, trace fine to coarse sand and gravel.	G#180045 A-6, CL WC=29.7%
						25				
						32				
						30				
25						23				

Remarks:

Saftey hammers, cathead and rope.
6/15/06; 13:30-17:00, 6/16/06; 07:45-12:30, 6/20/06; 07:45-12:00
Drilled R2 then sampled 1D and 2D. NW spinning shoe wouldn't penetrate beyond 10.5' bgs, pulled NW Casing (shoe destroyed), then reamed hole with 4" Roller Cone, inserted NW Casing with drive shoe and continued boring.

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

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

Boring No.: BB-CS.GAR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	New Mills Bridge #2605 over Cobbossee Stream	**Location:**	Gardiner, Maine	**Boring No.:**	BB-CS.GAR-101
				PIN:	12633.00
Driller:	MaineDOT	**Elevation (ft.)**	146.1	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30", 300#/16"
Date Start/Finish:	6-15-06 / 6-20-06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+94.8, 6.8 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	18.4' bgs

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	3D/AB	24/6	25.0 - 27.0	18/15/12/8	27	18	121.10		25.0 (3D/A) 25.0-26.3' bgs. Grey-brown, moist, medium dense, silty fine to coarse SAND, little gravel.	
						35	119.80		26.3 (3D/B) 26.3-27.0' bgs. Grey, moist, medium dense, fine SAND, little silt.	
						45				
						45				
						45				
30							116.10		30.0 **Bottom of Exploration at 30.00 feet below ground surface.** Casing broke off driving to 30.0' bgs, retrieved casing, but couldn't get back down.	
35										
40										
45										
50										

Remarks:

Saftey hammers, cathead and rope.
6/15/06; 13:30-17:00, 6/16/06; 07:45-12:30, 6/20/06; 07:45-12:00
Drilled R2 then sampled 1D and 2D. NW spinning shoe wouldn't penetrate beyond 10.5' bgs, pulled NW Casing (shoe destroyed), then reamed hole with 4" Roller Cone, inserted NW Casing with drive shoe and continued boring.

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

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

Boring No.: BB-CS.GAR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Mills Bridge #2605 over Cobbossee Stream
Location: Gardiner, Maine

Boring No.: BB-CS.GAR-101A
PIN: 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.1	**Auger ID/OD:**	5" Dia.
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	N/A
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	6/20/06; 13:00-14:00	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	10+86.4, 5.3 Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	145.70		Pavement 0.4	
									CONCRETE	
							144.00		2.1	
									Damp, COBBLES and GRAVEL, some brown, fine to coarse sand, trace silt, (Fill).	
5							141.30		4.8	
									CONCRETE	
							140.60		5.5	
							140.30		GRAVEL, some brown, damp, fine to coarse sand, trace silt, (Fill).	
							139.80		5.8	
									CONCRETE	
									6.3	
									Bottom of Exploration at 6.30 feet below ground surface. STOPPED IN CONCRETE	
10										
15										
20										
25										

Remarks:

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

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

Boring No.: BB-CS.GAR-101A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Mills Bridge #2605 over Cobbossee Stream
Location: Gardiner, Maine

Boring No.: BB-CS.GAR-101B
PIN: 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.1	**Auger ID/OD:**	5" Dia.
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	N/A
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	6/20/06; 14:00-15:00	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	10+62.1, 6.7 Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	145.60		Pavement	
									0.5 No description given.	
5							140.80		5.3 CONCRETE	
							137.30		8.8 **Bottom of Exploration at 8.80 feet below ground surface.** STOPPED IN CONCRETE	
10										
15										
20										
25										

Remarks:

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

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

Boring No.: BB-CS.GAR-101B

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** New Mills Bridge #2605 over Cobbossee Stream **Location:** Gardiner, Maine	**Boring No.:** BB-CS.GAR-101C **PIN:** 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.1	**Auger ID/OD:**	N/A
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30", 300#/16"
Date Start/Finish:	6/22/06; 08:00-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+90.8, 6.8 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	C1	16.8/16.8	0.0 - 1.4	5" thin wall hand held core machine.			145.85		Pavement 0.3	
									CONCRETE	
	C2	4.8/4.8	1.4 - 1.8			SPUN HW	144.30		1.8 Damp, COBBLES and GRAVEL, some brown, fine to coarse sand, trace silt, (Fill).	
5							141.00		5.1 CONCRETE	
10										
						SPUN NW				
15										
							127.80		18.3 Brown, wet, silty fine to coarse SAND, trace gravel.	
20										
							125.10		21.0	
25										

Remarks:

Safty hammers, cathead and rope.

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

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

Boring No.: BB-CS.GAR-101C

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: New Mills Bridge #2605 over Cobbossee Stream Location: Gardiner, Maine	Boring No.: BB-CS.GAR-101C PIN: 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.1	**Auger ID/OD:**	N/A
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30", 300#/16"
Date Start/Finish:	6/22/06; 08:00-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+90.8, 6.8 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	1D	24/7	25.0 - 27.0	8/7/12/8	19				Grey, wet, medium dense, silty fine to coarse SAND, little gravel.	G#180046 A-4, SM WC=14.4%
							118.50		27.6	
	R1	19.2/10	28.1 - 29.7	RQD = N/A%		NQ			R1: Boulder	
							117.20		28.9	
30										
35	MD	1/0	35.0 - 35.1	No blows given					Failed sample attempt, no recovery.	
									Roller Coned ahead of casing from 39.1- 40.0' bgs.	
40	2D	9.6/6	40.0 - 40.8	53/50(3.6")	---				Grey, moist, silty fine to coarse SAND, cobbles, little gravel.	
45										
50						aRC			aRoller Coned ahead from 49.6-53.9' bgs.	

Remarks:

Safty hammers, cathead and rope.

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** New Mills Bridge #2605 over Cobbossee Stream **Location:** Gardiner, Maine	**Boring No.:** BB-CS.GAR-101C **PIN:** 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.1	**Auger ID/OD:**	N/A
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30", 300#/16"
Date Start/Finish:	6/22/06; 08:00-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+90.8, 6.8 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										
							93.90		Top of Bedrock at Elev. 93.9' (52.2)	
	R2	50.4/46	53.9 - 58.1	RQD = 36%		NQ			R2: Augen GNEISS, moderately hard, moderately weathered, steep to vertical foliation, moderately close joint-set, slightly open. Rock Quality = Poor R2:Core Times (min:sec) 53.9-54.9' (2:50) 54.9-55.9' (2:07) 55.9-56.9' (2:18) 56.9-57.9' (2:18) 57.9-58.1' (0:33) 92% Recovery	
55										
	R3	60/62	58.1 - 63.1	RQD = 80%					R3: 58.1 to 61.1': Same as R2 R3:Core Times (min:sec) 58.1-59.1' (2:13) 59.1-60.1' (1:42) 60.1-61.1' (1:47)	
60										
							85.00		(61.1) R3: 61.1 to 63.1': Light grey, pegmatite GRANITE, very hard, slightly weathered, massive, tight. Rock Quality = Good Core Times: (min:sec) 61.1-62.1' (1:47) 62.1-63.1' (2:32) 103% Recovery	
							83.00		(63.1) **Bottom of Exploration at 63.10 feet below ground surface.**	
65										
70										
75										

Remarks:

Safty hammers, cathead and rope.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Mills Bridge #2605 over Cobbossee Stream
Location: Gardiner, Maine

Boring No.: BB-CS.GAR-103
PIN: 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.0	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30", 300#/16"
Date Start/Finish:	6/21/06; 07;45-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+39.6, 7.2 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	18.6' bgs.

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	145.80		Pavement —0.2	
									CONCRETE	
							144.50		—1.5	
									Damp, COBBLES and GRAVEL, some brown fine to coarse sand, trace silt, (Fill).	
5									Bent, destroyed, split spoon on concrete.	
	1D	24/10	5.0 - 7.0	7/15/28/47	43	SPUN HW	140.00		—6.0	
									CONCRETE	
10									Roller Coned ahead from 10.0-11.0' bgs.	
15									Roller Coned ahead from 15.0-15.6' bgs.	
	R1	38.4/37	15.6 - 18.8	RQD = N/A%		NQ			R1:CONCRETE R1:Core Times (min:sec) 15.6-16.6' (2:41) Concrete in good condition from 15.6-16.8' bgs. 16.6-17.6' (2:07) Concrete in fair to poor condition from 16.8-18.5' bgs. 17.6-18.6' (2:41) 18.6-18.8' (0:10) 97% Recovery	
	2D/AB	24/7	18.8 - 20.8	WOR/WOR/WOR/ WOH	0		127.40		—18.6	
									(2D/A) 18.8-20.5' bgs. Grey, saturated, very loose, fine to coarse SAND, some silt, little gravel. Drilled R1, spun HW Casing to 20.0' bgs. , then drove NW Casing.	G#180047 A-2-4, SM WC=40.7%
20						2	125.50		—20.5	
									(2D/B) 20.5-20.8' bgs. Grey, wet, very loose, silty fine to coarse SAND, trace organics.	
						2				
						8				
						13				
						7				
25										

Remarks:

Safty hammers, cathead and rope.

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: New Mills Bridge #2605 over Cobbossee Stream Location: Gardiner, Maine	Boring No.: BB-CS.GAR-103 PIN: 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	146.0	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	E. Giguere/B. Hyland	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30", 300#/16"
Date Start/Finish:	6/21/06; 07;45-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+39.6, 7.2 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	18.6' bgs.

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	3D	24/11	25.0 - 27.0	5/6/8/15	14	21	121.00		25.0 Grey, moist, medium dense, silty fine to coarse SAND, trace coarse sand, trace gravel.	
						23				
						57				
	R2	62.4/62.4	28.9 - 34.1	RQD = 68%		a85 NQ	117.20		a85 blows for 10". 28.8 Top of Bedrock at Elev. 117.2' R2: Augen GNEISS, moderately hard, moderately weathered, steep foliation, close joint-set, slightly open. Rock Quality = Fair R2:Core Times (min:sec) 28.9-29.9' (3:01) 29.9-30.9' (2:24) 30.9-31.9' (2:18) 31.9-32.9' (2:08) 32.9-33.9' (2:35) 33.9-34.1' (0:40) 100% Recovery	
30										
35	R3	60/60	34.1 - 39.1	RQD = 40%					R3: Same as R2. Rock Quality = Poor R3:Core Times (min:sec) 34.1-35.1' (2:45) 35.1-36.1' (3:20) 36.1-37.1' (1:53) 37.1-38.1' (2:06) 38.1-39.1' (2:51) 100% Recovery	
							106.90		39.1 **Bottom of Exploration at 39.10 feet below ground surface.**	
40										
45										
50										

Remarks:

Safty hammers, cathead and rope.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: New Mills Bridge #2605 over Cobbossee Stream, Routes 9/126
Location: Gardiner, Maine

Boring No.: BB-CS-GAR-201
PIN: 12633.00

Driller:	MaineDOT	**Elevation (ft.)**	117.3	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	Auto Hammer
Date Start/Finish:	11/5/07; 09:30-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	11+66.7, 5.3 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	11.8' below Bridge Deck
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.00 - 2.0	3/6/12/31	18	23	a31			a31 blows for 0.8'. Dark grey, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt and clay, trace shell fragments.	G#209978 A-1-a GP-GC WC=19.9%
							38				
							12				
							14				
							16				
5							69	111.30		6.0	
	2D	24/16	6.00 - 8.0	22/25/27/40	52	67	54			Grey, wet, hard, SILT, some fine to coarse sand, little clay, trace gravel.	G#209979 A-4, CL-ML WC=11.0%
							122				
							193	108.30		9.0	
							34 bWA	107.20		Layer of gravelly material on top of rock approximately 1.1' thick. bWashed ahead to 10.1' bgs. 10.1	
10	R1	60/60	10.10 - 15.1	RQD = 73%			NQ CORE	105.80		Top of Bedrock at Elev. 107.2' R1: Grey, black and white, coarse grained, metamorphic, SCHIST very decomposed, with iron staining, pyrite mica. Rock Quality = Fair R1:Core Times (min:sec) at 650 psi 11.5 10.1-11.1' (3:14) 11.1-12.1' (2:43) 12.1-13.1' (2:59) 13.1-14.1' (3:00) 14.1-15.1' (3:37) 100% Recovery	
15	R2	60/60	15.10 - 20.1	RQD = 82%						R2: White, grey, green and black, coarse grained, moderately hard, weathered, augen GNEISS, steep foliation, massive, with mica and pyrite. Rock Quality = Good R2:Core Times (min:sec) 15.1-16.1' (3:53) 16.1-17.1' (3:14) 17.1-18.1' (2:46) 18.1-19.1' (2:21) 19.1-20.1' (2:23) 100% Recovery No water return from 15.6' bgs on R2.	
20								97.20		20.1 **Bottom of Exploration at 20.10 feet below ground surface.**	
25											

Remarks:

Boring drill through Bridge Deck.
29.2' from Bridge Deck to top of River Bed.

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.

<u>Appendix B</u>

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Gardiner Project Number: 12633.00

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-CS.GAR-101, 2D	10+94.8	6.8 Lt.	20.8-22.8	180045	1	29.7			CL	A-6	III
BB-CS.GAR-101C, 1D	10+90.8	6.8 Lt.	25.0-27.0	180046	1	14.4			SM	A-4	III
BB-CS.GAR-103, 2D/A	12+39.6	7.2 Rt.	18.8-20.5	180047	1	40.7			SM	A-2-4	II
BB-CS.GAR-201, 1D	11+66.7	5.3 Rt.	0.0-2.0	209978	1	19.9			GP-GC	A-1-a	0
BB-CS.GAR-201, 2D	11+66.7	5.3 Rt.	6.0-8.0	209979	1	11.0			CL-ML	A-4	IV

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

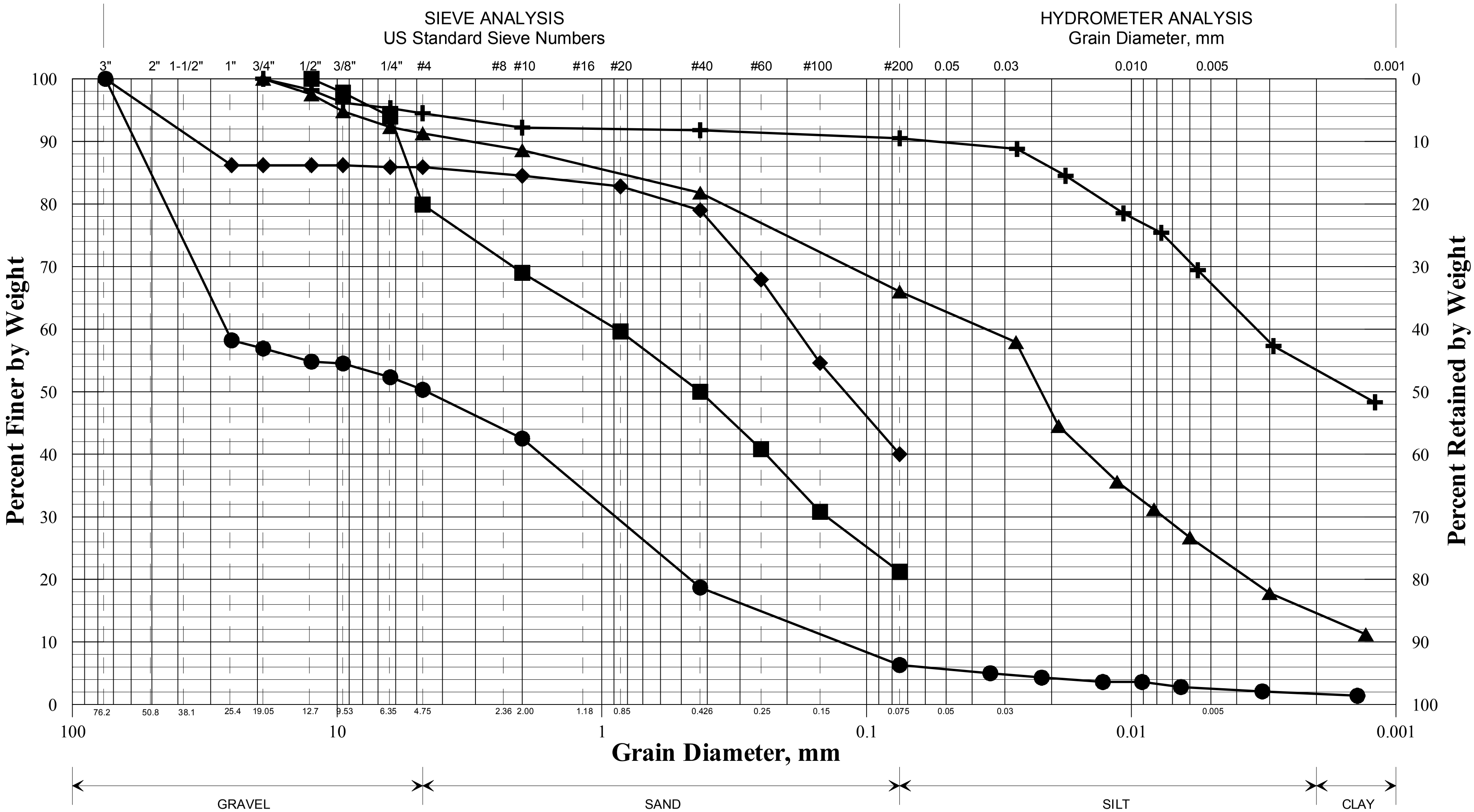
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-CS.GAR-101/2D	10+94.8	6.8 LT	20.8-22.8	Silty CLAY, trace sand, trace gravel.	29.7			
◆	BB-CS.GAR-101C/1D	10+90.8	6.8 LT	25.0-27.0	Silty SAND, little gravel.	14.4			
■	BB-CS.GAR-103/2D(A)	12+39.6	7.2 RT	18.8-20.5	SAND, some silt, little gravel.	40.7			
●	BB-CS.GAR-201/1D	11+66.7	5.3 RT	0.0-2.0	Sandy GRAVEL, trace silt, trace clay.	19.9			
▲	BB-CS.GAR-201/2D	11+66.7	5.3 RT	6.0-8.0	SILT, some sand, little clay, trace gravel.	11.0			
✖									

PIN
012633.00
Town
Gardiner
Reported by/Date
WHITE, TERRY A 3/19/2008

SHEET 1

Appendix C

Calculations

Definition of Units:

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

Abutment Foundations: Driven 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 14 x 73 Note: All matrices set up in this order
HP 14 x 89
HP 14 x 117

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$ yield strength: $F_y := 50 \cdot ksi$

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

Where λ=normalized column slenderness factor

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

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

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} kip$

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

STRENGTH LIMIT STATE:

Factored Resistance:

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance:

eq. 6.9.2.1-1 $P_f := \phi_c \cdot P_n$ $P_f = \begin{pmatrix} 465 \\ 642 \\ 783 \\ 1032 \end{pmatrix} kip$

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

Strength Limit State

SERVICE AND EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

During extreme or scour event there will likely be exposed pile length, so λ will have a value. Designer should be responsible to adjust factored and compressive resistances for that new λ.

$$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s \qquad P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \text{kip}$$

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

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

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$$P_f := \phi \cdot P_n \qquad P_f = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \text{kip}$$

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

Service/Extreme Limit States

Geotechnical Resistance

Assume piles will be end bearing on bedrock driven through overlying sand fill and silt and clay.

Bedrock Type: Augen Gneiss, moderately hard
RQD ranges from 36 to 80%. Use RQD = 60% and ϕ = 34 to 40 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Look at these piles:

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

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

Note: All matrices set up in this order

Steel area: $$A_s = \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} in^2$$

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

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

Calculate pile box area:

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.8901 \\ 198.5018 \\ 203.2319 \\ 211.5159 \end{pmatrix} in^2$$

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

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

q_u for gneiss compressive strength
ranges for 3500 to 45000 psi

use $\sigma_c := 30000 \cdot psi$

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

Spacing of discontinuities: $c := 12 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{32} \cdot in$ joints are tight

Footing width, b: $$b = \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} in$$

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

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}} \qquad K_{sp} = \begin{pmatrix} 0.2994 \\ 0.2864 \\ 0.286 \\ 0.2852 \end{pmatrix}$$

K_{sp} contains a factor of safety of 3 against lower bound bearing capacity of the foundation.

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

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

$q_a := \sigma_c \cdot K_{sp} \cdot d_f$ $q_a = \begin{pmatrix} 1294 \\ 1237 \\ 1235 \\ 1232 \end{pmatrix} ksf$

Nominal Geotechnical Tip Resistance, R_p:

Use the steel area of the pile as no plug will develop during driving
Take out the factor of safety included in K_{sp}.

$R_p := \overrightarrow{\left(3 q_a \cdot A_s\right)}$ $R_p = \begin{pmatrix} 418 \\ 552 \\ 672 \\ 883 \end{pmatrix} kip$

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

STRENGTH LIMIT STATE:

Factored Geotechnical Tip Resistance, R_f at Strength Limit State:

Resistance factor, end bearing on rock (CGS 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

$R_{tipfStr} := \phi_{stat} \cdot R_p$ $R_{tipfStr} = \begin{pmatrix} 188 \\ 248 \\ 302 \\ 397 \end{pmatrix} kip$

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

Strength Limit State

SERVICE AND EXTREME LIMIT STATES:

Factored Geotechnical Tip Resistance, R_f at Service/Extreme Limit State:

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

$\phi := 1.0$

$R_{tipfServEx} := \phi \cdot R_p$ $R_{tipfServEx} = \begin{pmatrix} 418 \\ 552 \\ 672 \\ 883 \end{pmatrix} kip$

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

Service/Extreme Limit States

Determine Geotechnical Resistance by past methods for comparison:

Geotechnical Nominal Resistance by Goodman's Method
Based on bedrock condition - in this case Granite RQD = 65-100%
Reference: Pile Design and Construction Practice 4th Edition MJ Tomlinson

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

$\phi_2 := 34 \cdot \text{deg}$ $\qquad N_\phi := \tan\left(45 \cdot \text{deg} + \frac{\phi_2}{2}\right)^2$ $\qquad N_\phi = 3.5371$

q_{uc} for gneiss compressive strength ranges for 3500 to 45000 psi $\qquad$ use $q_{uc} := 30000 \cdot \text{psi}$

$q_b := (2 \cdot N_\phi) \cdot \frac{q_{uc}}{5}$ $\qquad q_b = 42.4456\,\text{ksi}$

Reduce q_{uc} by 5 for scale effects in rocks per Das Principles of Foundation Engineering 2nd Edition Eq. 8.56

Steel area:

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

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

$Q_{nom} := q_b \cdot A_s$

$$Q_{nom} = \begin{pmatrix} 658 \\ 908 \\ 1108 \\ 1460 \end{pmatrix} \text{kip}$$

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

Factored Geotechnical Resistance:

From RFD Table 10.5.5.2.3-1 $\qquad \phi_{stat} := 0.45$

$Q_{fac} := \phi_{stat} \cdot Q_{nom}$

$$Q_{fac} = \begin{pmatrix} 296 \\ 409 \\ 499 \\ 657 \end{pmatrix} \text{kip}$$

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

use Canadian Foundation Manual values above

DRIVABILITY ANALYSIS

Ref: LFRD 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

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

Compute Resistance that must 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.

STRENGTH LIMIT STATE:

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 8 to 10 piles total on the project. Only 1 or 2 piles will be tested - one per abutment will be requested. Therefore, reduce the ϕ by 20%

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

$$Q_{drivability.A1} := \frac{R_{tipfStr}}{\phi_{dyn.reduced}} \qquad Q_{drivability.A1} = \begin{pmatrix} 361 \\ 477 \\ 581 \\ 764 \end{pmatrix} kip$$

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

Strength Limit State

Abutment Passive and At-rest Earth Pressure:

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

For a horizontal backfill surface:

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

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

At-Rest Earth Pressure from BM Das Principles of Foundation Engineering Second Edition Eq. 5.4

For normally consolidated granular soils:

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

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

$$K_o = 0.4701$$

Rankine Theory - Passive Earth Pressure from Das Principles of Foundation Engineering Second Edition Equation 5.23 pg 269

$$\phi := 32 \cdot deg$$

$$K_p := \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^2 \qquad K_p = 3.255$$

Seismic Design:

Gardiner New Mills Bridge
Date and Time: 3/25/2008 3:42:27 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04345
Zip Code Latitude = 44.231700
Zip Code Longitude = -069.789600
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.078	PGA - Site Class B
0.2	0.161	Ss - Site Class B
1.0	0.045	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04345
Zip Code Latitude = 44.231700
Zip Code Longitude = -069.789600
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.124	As - Site Class D
0.2	0.257	SDs - Site Class D
1.0	0.108	SD1 - Site Class D