MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

GREAT WORKS RIVER BRIDGE
OVER THE GREAT WORKS RIVER
STATE ROUTE 236
SOUTH BERWICK, MAINE

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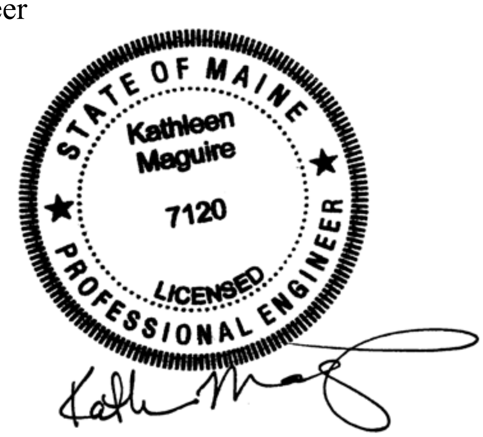

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York County | Soils Report No. 2009-04
PIN 15609.00 | Bridge No. 5610

February 5, 2009

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement of the Great Works River Bridge on State Route 236 over the Great Works River in South Berwick, Maine. The proposed replacement bridge will consist of welded steel plate girders on H-pile supported integral abutments. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-piles - The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be driven with their weak axis perpendicular to the center line of the beams. Piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption. 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. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and flexure. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Abutments and Wingwalls - Integral stub abutments and wingwalls shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported abutments and wingwalls at the strength limit state shall consider pile stability and structural resistance. Extreme limit state design shall also consider foundation resistance after scour due to the design flood. For abutments that are pile supported, design for resistance against sliding and overturning is not required. In designing integral abutments for passive earth pressure, the Rankine earth pressure coefficient (K_p) of 3.25 is recommended. All abutment designs shall include a drainage system to intercept any water. To avoid water intrusion behind the abutment, the approach slab should connect directly to the abutment.

Scour and Riprap- The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength, service and extreme limit states. These changes in foundation conditions shall be investigated at the abutments and wingwalls. For scour protection, any footings which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap.

Settlement - Post-construction settlements are anticipated to be less than 1 inch 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.

Frost Protection – Any foundations placed on granular soils should be founded a minimum of 4.5 feet below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection.

Seismic Design Considerations - The Great Works River Bridge is located on State Route 236 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. A detailed 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.

Construction Considerations - There is potential for boulders and cobbles to impact the pile driving/installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers or as approved by the Resident. Since the proposed bridge design will rely on the riprap slopes to provide scour protection for the integral abutment piles, slope construction and riprap placement are of critical importance. The existing riprap slopes shall be reconstructed in their entirety. Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Special Provisions 610 and 703 and the plans.

1.0 INTRODUCTION

A subsurface investigation for the replacement of the Great Works River Bridge on State Route 236 over the Great Works River in South Berwick, York 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 existing bridge was constructed in 1955 and consists of a 95 foot long, single span, riveted deck girder founded on pile supported concrete abutments. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the deck is in poor condition (rating of 4), the superstructure is in satisfactory condition (rating of 6) and the abutments are in good condition (rating of 7). Inspection notes state that the deck has several areas of large "pop outs" with exposed rebar, the bearings are heavily rusted, and the girder ends and diaphragms are delaminating with moderate section loss. The year 2008 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 71.6. It is understood that the existing bridge superstructure will be completely removed and replaced. The existing piles in the bridge substructure will be cut off below grade and left in place.

The proposed bridge will consist of a 100 foot long, single span, welded steel plate girder superstructure on driven H-pile supported integral abutments. The new structure will have a similar horizontal alignment to the existing bridge. The vertical alignment will have a 0.5 percent grade across the bridge section and will be raised less than 1 foot at both abutments. In order to minimize impacts due to slopes, 1H to 1.75V riprapped slopes will be utilized in front of the abutments.

2.0 GEOLOGIC SETTING

The Great Works River Bridge on Route 236 in South Berwick crosses the Great Works River approximately 0.28 miles northerly of York Woods Road as shown on Sheet 1 - Location Map found at the end of this report. The Great Works River flows in a westerly direction to the Salmon Falls River which flows south into the Piscataqua River which flows in a south easterly direction into Portsmouth Harbor and the Atlantic Ocean.

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

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as Silurian-Precambrian calcareous feldspathic sandstone of the Kittery Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling three (3) test borings at the site. Test boring BB-SBGWR-101 was drilled behind the location of Abutment No. 2 (north). Test borings BB-SBGWR-102 and BB-SBGWR-102A were drilled behind the location of Abutment No. 1 (south). The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The borings were drilled between November 19 and 29, 2007 using the Maine Department of Transportation (MaineDOT) drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on Sheet 4 - Boring Logs found end of this report.

The borings were drilled using driven cased wash boring, spun casing 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. All N-values 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.

In-situ vane shear tests were made where possible in soft soil deposits to measure the shear strength of the strata. 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. A 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 four (4) standard grain size analyses and four (4) grain size analyses 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 Sheet 4 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the abutments consisted of fill materials, overlying a thin gravel layer, overlying a silt layer, overlying sand and gravel layers all overlying bedrock. An interpretive subsurface profile depicting the site stratigraphy is show on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

Fill Materials with Cobbles and Boulders. Beneath the pavement, a layer of fill materials was encountered in all of the borings. This layer was found to be brown, damp to wet, fine to coarse sand, with some gravel, trace silt and frequent cobbles and boulders with depth. Drilling was very difficult through the cobbles and boulders present in the fill. The thickness of the fill layer ranged from approximately 21.7 feet in boring BB-SBGWR-101 to approximately 26.8 feet in boring BB-SBGWR-102A. Corrected SPT N-values in the fill layer ranged from 15 to 53 blows per foot (bpf) indicating that the soil is medium dense to very dense in consistency. Water contents from three (3) samples obtained within this layer range from approximately 3% to 5%. Three (3) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-1-b or A-1-a by the AASHTO Classification System and a SW-SM, SM or GW-GM by the Unified Soil Classification System.

Gravel. A thin layer of gravel was encountered beneath the fill in boring BB-SBGWR-102A. This layer was found to be grey, wet, gravel, with some fine to coarse sand, some silt and some clay. The thickness of the gravel layer was approximately 1.0 foot. One corrected SPT N-value in the gravel layer was 5 bpf indicating that the gravel is loose in consistency. One (1) water content from the gravel was approximately 26%. One (1) grain size analyses conducted on a sample from this layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a GC-GM by the Unified Soil Classification System.

Silt. A layer of silt was encountered in boring BB-SBGWR-102A beneath the gravel layer. This layer was found to be grey, wet, silt, with some to little clay, trace sand and trace gravel. The thickness of the silt layer was approximately 3.8 feet. Corrected SPT N-values obtained in the silt layer ranged from 5 to 9 bpf indicating that the soil is medium stiff to stiff in consistency. One vane shear test conducted within the silt layer showed an undrained shear strength of approximately 290 psf while the remolded shear strength was approximately 67 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the silt was determined to have sensitivity of approximately 4.3 and is classified as sensitive. Water contents from three (3) samples obtained within this layer range from approximately 22% to 36%. Three (3) grain size analyses with hydrometer conducted on samples from this layer indicate that the soil is classified as an A-7-5 or A-4 by the AASHTO Classification System and a CL or CL-ML by the Unified Soil Classification System.

Sand. A layer of sand was encountered beneath the silt in boring BB-SBGWR-102A. This layer was found to be grey to brown, wet, fine to coarse sand, with some gravel, trace silt and frequent cobbles with depth. The thickness of the sand layer was approximately 12.2 feet. Corrected SPT N-values in the layer ranged from 40 to 58 blows per foot (bpf) indicating that the soil is dense to very dense in consistency. A water content from one (1) sample obtained within this layer was approximately 11%. One (1) grain size analysis conducted on a sample from this layer indicated that the soil is classified as an A-2-4 by the AASHTO Classification System and a SM by the Unified Soil Classification System.

Gravel. A thin layer of gravel was encountered beneath the sand overlying the bedrock in boring BB-SBGWR-102A. This layer was found to be grey, wet, gravel, with some medium to coarse sand, and trace silt. The thickness of the gravel layer was approximately 1.4 feet.

Bedrock. Bedrock was encountered and cored in two of the borings. Table 1 below presents the bedrock findings:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB-SBGWR-102A Abutment No. 1	44.9 feet	47.4 feet	0%
BB-SBGWR-101 Abutment No. 2	21.7 feet	71.6 feet	0%

Table 1 – Summary of Bedrock Depths, Elevations and RQD

The bedrock at the site can be identified as grey, fine-grained, sedimentary, sandstone, which is highly fractured. The most notable feature of the rock is the presence of dissolution vugs within the rock matrix. Vugs are defined as small cavities inside rock made up of cracks and fissures which have been filled with secondary minerals which are later removed through the dissolution process leaving irregular voids. The inner surfaces of the voids are typically coated with some of the dissolved mineral matter. The vugs are oriented along healed fractures in the rock. The bedrock is a part of the Kittery Formation. The RQD of the bedrock was 0% indicating a rock mass quality of very poor quality.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered during the subsurface exploration program, the following foundation alternatives, with varying levels of risk and effectiveness, may be considered for the bridge replacement:

- Cast-in-place concrete or precast concrete integral abutments supported on driven steel H-piles
- Cast-in-place, full height abutments founded on spread footings bearing on native sand and/or bedrock

After consideration of the foundation alternatives the structural team chose to use the cast-in-place concrete integral abutments supported on driven steel H-piles. This report addresses only this foundation type.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for cast-in-place integral stub abutments and butterfly wingwalls founded on a single row of integral H-piles driven to bedrock which has been identified as the optimal substructure for the site. The use of short pile supported integral abutments is under consideration by the MaineDOT Bridge Program. Initial results indicate that although fixity is not achieved for piles less than 13 feet long, the structure can accommodate cyclic live and thermal loading without any major consequence. The current study[1] indicates that the use of short pile supported integral abutments for bridges with spans not exceeding 115 feet is applicable. However, in consideration of the consequences scour and pile exposure and the need to limit pile tip movement, a minimum pile length of 10 feet is recommended.

7.1 Integral Abutment H-piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads. Piles should be driven with their weak axis perpendicular to the center line of the beams. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption.

Pile lengths at the proposed abutments may be estimated based on the data in Table 2 below:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Rock Quality Designation	Estimated Pile Length
Abutment No. 1 BB-SBGWR-102A	81.0 feet	44.9 feet	47.4 feet	0%	35 feet
Abutment No. 2 BB-SBGWR-101	81.5 feet	21.7 feet	71.6 feet	0%	10 feet

Table 2 – Estimated Pile Lengths for Piles Installed to Bedrock Surface

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

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include

[1] MaineDOT Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1"

checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed below. Short piles supporting integral abutments should be designed in accordance with AASHTO LRFD criteria and checked for pile tip movement as described in the design example found in Appendix B of Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" and Chapter 5 of that report.

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

Since the abutment piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. An L-Pile® analysis is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, movements and pile head displacements. Achievement of an assumed pinned condition at the pile tip should also be confirmed with an L-Pile® analysis. As the proposed piles for Abutment No. 2 will be short and will not achieve fixity, the resistance for the pile will be determined for structural compliance with interaction equation.

The integrity of the bridge approach fills and riprap abutment slopes must be maintained as these provide the only lateral support to the short pile group. The stream velocity should be low and there should be low potential for scour action, wave action, storm surge, and ice damage.

7.1.1 Strength Limit State

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances. Preliminary estimates of the factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.50 (severe driving conditions) and a λ of 0.

The nominal geotechnical compressive resistances of the H-pile sections in the strength limit state were calculated using Goodman's Method and FHWA software Driven 1.0. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45.

The drivability of the four proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65.

The calculated factored axial compressive structural, geotechnical and drivability resistances of the four proposed H-pile sections for each abutment are summarized in Table 3 below. Supporting calculations are included in Appendix C- Calculations found at the end of this document.

Pile Section	Factored Resistance (kips)			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
Abutment No. 1				
12 x 53	388	347	298	298
14 x 73	535	479	335	335
14 x 89	653	499	440	440
14 x 117	860	529	647	647
Abutment No. 2				
12 x 53	388	84	223	223
14 x 73	535	116	304	304
14 x 89	653	141	390	390
14 x 117	860	186	547	547

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

Table 3 – Factored Axial Resistances for Abutment Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial drivability resistance is less than the factored axial structural resistance and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the factored drivability resistance shown in Table 3 above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.12.2.2.1-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.12.2.

7.1.2 Service and Extreme Limit States

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

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

Pile Section	Factored Resistance (kips)			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
Abutment No. 1				
12 x 53	775	771	459	459
14 x 73	1070	1065	516	516
14 x 89	1305	1108	677	677
14 x 117	1720	1177	996	996
Abutment No. 2				
12 x 53	775	186	343	343
14 x 73	1070	257	467	467
14 x 89	1305	313	600	600
14 x 117	1720	413	842	842

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

Table 4 - Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial drivability resistance is less than the factored axial structural resistance and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the maximum factored axial pile load used in design for the service and extreme limit states should not exceed the factored drivability resistance shown in Table 4 above.

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 0.65. The factored pile load should be shown on the plans.

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

7.2 Integral Stub Abutments and Wingwalls

Integral stub abutments and wingwalls shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported abutments and wingwalls at the strength limit state shall consider pile stability and structural resistance.

A resistance factor of ϕ= 1.0 shall be used to assess abutment design at the service limit state including: settlement, horizontal movement, overall stability and scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor,ϕ, of 0.65. Extreme limit state design checks for abutments supported on piles shall include pile structural resistance pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0. The unfactored strength limit state loads include any debris loads occurring during the flood event.

Integral abutments and wingwall sections that are integral with the abutment should be designed to withstand a passive earth pressure state. In designing for passive earth pressure associated with integral abutments, the Coulomb state is recommended. Experience in designing wingwalls and integral abutments has shown that the use of the Coulomb passive earth pressure K_p=6.89 may result in uneconomical wall sections. For this reason, consideration may be given to using a Rankine passive earth pressure, K_p=3.25 when designing integral abutments and integral wingwall extensions.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the return wings when traffic loads are located within a horizontal distance equal to one-half of the wall height behind the back of the wall. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads on abutments is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 5 below:

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

Table 5 – Equivalent Height of Soil for Estimating Live Load Surcharge

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.

All abutment designs shall include a drainage system behind the abutments to intercept any water. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. Geocomposite drainage board applied to the backsides of the abutments and wingwalls with weep holes will provide adequate drainage. To avoid water intrusion behind the abutment, the approach slab should connect directly to the abutment.

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

7.3 Estimated Depths to Pile Fixity

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

Preliminary depths to fixity for the four (4) proposed H-pile sections were calculated, assuming only axial loading and without consideration of lateral loads, using the methodology from the Mass Highway Bridge Manual (1999). Table 6 below summarizes the calculated depths to fixity for the four (4) proposed H-pile sections. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

H-pile Section	Preliminary Estimates of Depth to Fixity w/ no lateral loads applied
12 x 53	19 feet
14 x 73	20 feet
14 x 89	22 feet
14 x 117	23 feet

Table 6 - Preliminary Estimates of Depth to Fixity

In general it is recommended that piles be designed to achieve a fixed condition at the pile toe. Due to the depth of the overburden at the site, it is anticipated that the pile sections at Abutment No. 1 will all achieve a fixed condition while the pile sections at Abutment No. 2 will not achieve a fixed condition assuming a pile penetration to the top of bedrock. Short piles supporting integral abutments should be designed in accordance with AASHTO LRFD criteria and checked for pile tip movement as described in the design example found in Appendix B of Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" and Chapter 5 of that report.

When the lateral and axial pile load groups are known, this data should be provided to the geotechnical engineer. A more refined analysis of pile fixity can then be performed using LPile® software.

7.4 Buckling and Combined Axial and Flexure

Pile group design shall consider loading effects due to combined axial and flexural loading, as outlined in LRFD Article 6.15. For a pile group composed of only vertical piles which is subjected to lateral loads, the pile structural analysis shall include consideration of soil-structure interaction effects as specified in LRFD Article 6.9. The recommended design approach considers the non-linear response of soil with lateral displacement. Soil-structure interaction considering the non-linear response of soil can be modeled using L-Pile® computer software.

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

7.5 Scour and Riprap

If using integral abutments at the site, pile lengths will be short and, therefore, scour protection will be critical. For scour protection, the integral abutments should be located away from the channel. Since the proposed bridge design will rely on the riprap slopes to provide scour protection for the integral abutment piles, slope construction and riprap placement are of critical importance.

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength, service and extreme limit states. These changes in foundation conditions shall be investigated at the abutments and wingwalls. For scour protection, any footings for wingwalls, which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of abutments and wingwalls. Special Provisions 610 and 703 are provided in Appendix D – Special Provisions found at the end of this report. Riprap shall be 3 feet thick. In front of the wingwalls, the bottom of the riprap section shall be constructed 4.5 feet above the bottom of the structures for frost protection. The riprap shall extend 1.5 feet horizontally in front of the wall before sloping at a maximum 1.75H:1V slope to the existing ground surface. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "A" Erosion Control Geotextile per Standard Detail 610 (02-04).

7.6 Settlement

The grades of the existing bridge approaches will be raised in order to accommodate the change in horizontal alignment of the proposed bridge. Additionally, roadway will be widened to both sides at both abutments. The maximum fill to be placed at the site is approximately 5.5 feet and will result in less than 1 inch of settlement. This settlement is anticipated to occur during construction and will have minimal effect of the finished structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

7.7 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT frost depth maps for the State of Maine (MaineDOT BDG Figure 5-1) the site has a design-freezing index of approximately 1200 F-degree days. This correlates to a frost depth of 6.0 feet. The design frost depth was also calculated according to the US Army Corps Cold Regions Research and Engineering (USACE CRREL) Modberg computer program. According to the CRREL Modberg program, the site has a design freezing index of 1123 F-degree days. A water content of 5% from laboratory testing was used for the damp fill soils above the water table. These components correlate to a frost depth of 4.5 feet. It is believed that this frost depth is a more accurate assessment of the actual frost depth at the site.

Therefore, any foundations placed on granular soils should be founded a minimum of 4.5 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG. See Appendix C- Calculations at the end of this report for supporting documentation.

7.8 Seismic Design Considerations

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

- Peak Ground Acceleration coefficient (PGA) = 0.101g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.192g
- 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 0.109g (LRFD Eq. 3.10.4.2-6).

According to Figure 2-2 of the Maine DOT BDG, the Great Works River 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. 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.

7.9 Construction Considerations

Boulders and cobbles were encountered within the existing abutment backfill in both of the borings. There is potential for these obstructions to impact the pile driving and/or installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

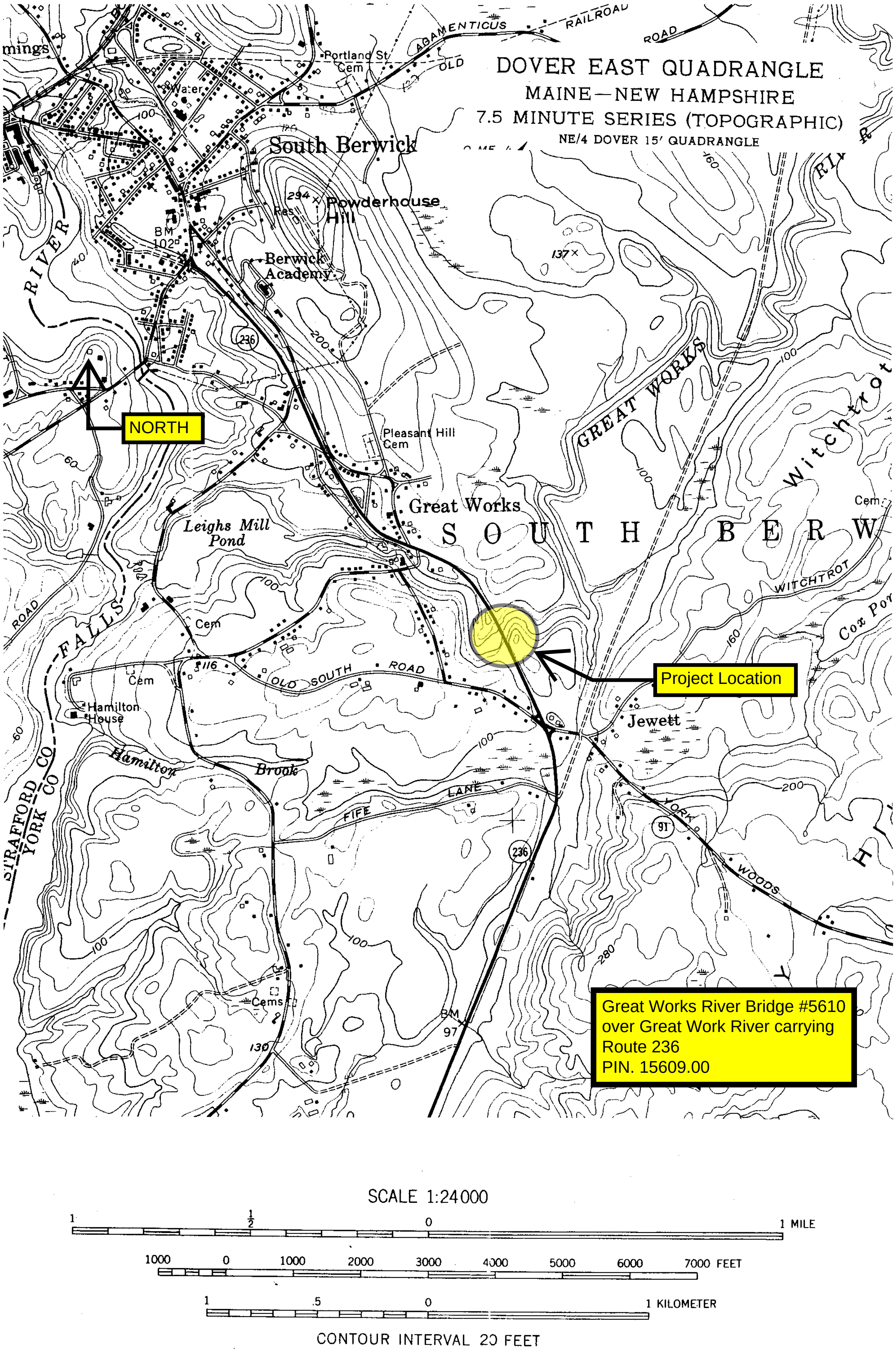
Since the proposed bridge design will rely on the riprap slopes to provide scour protection for the integral abutment piles, slope construction and riprap placement are of critical importance. The existing riprap slopes shall be reconstructed in their entirety. Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Special Provisions 610 and 703 and the plans.

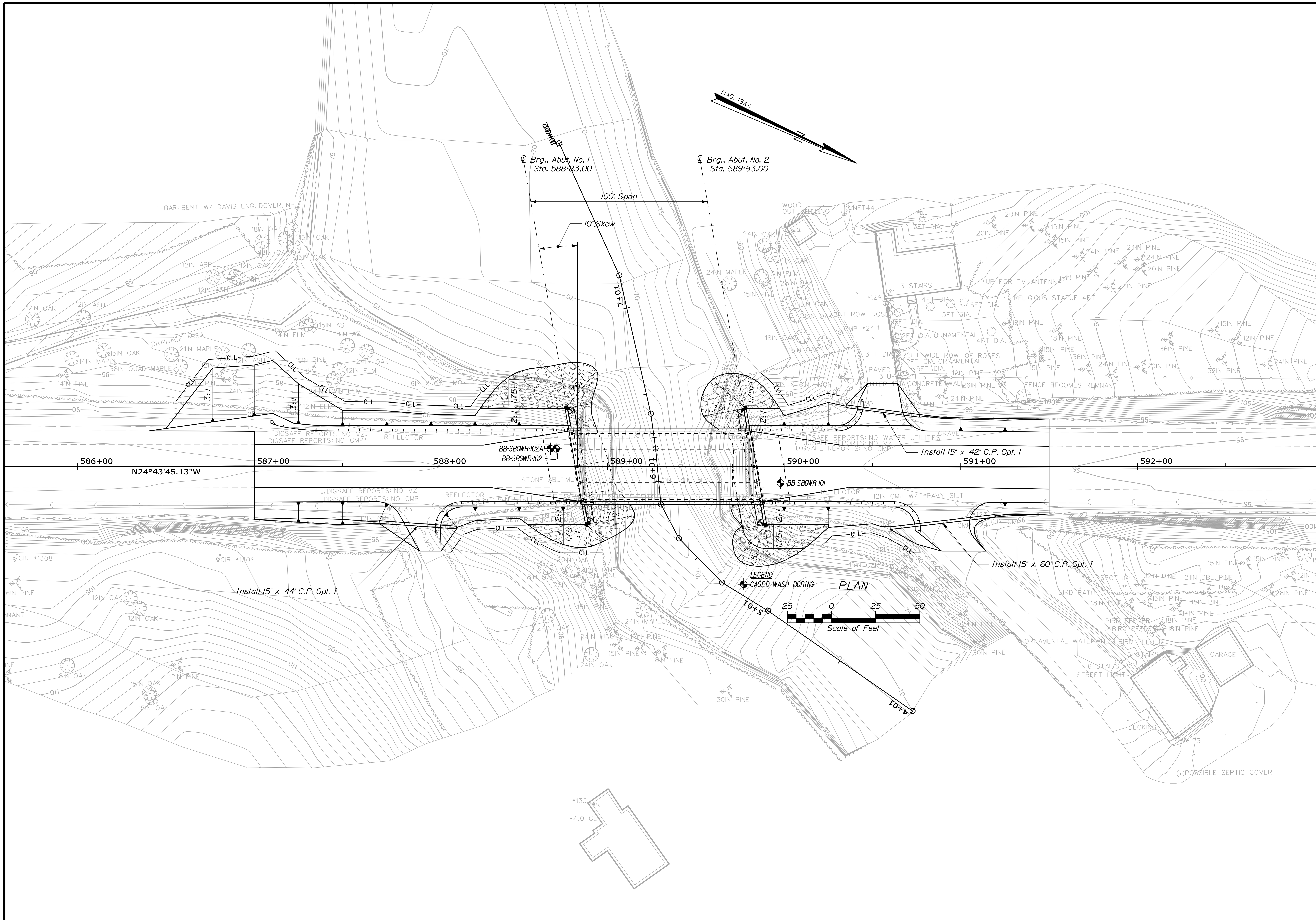
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Great Works River Bridge in South Berwick, 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


DOVER EAST QUADRANGLE
MAINE—NEW HAMPSHIRE
7.5 MINUTE SERIES (TOPOGRAPHIC)
NE/4 DOVER 15' QUADRANGLE
NORTH
Project Location
Great Works River Bridge #5610
over Great Work River carrying
Route 236
PIN. 15609.00
South Berwick
Powderhouse Hill
Berwick Academy
Pleasant Hill Cem
Great Works
Leighs Mill Pond
SOUTH BERW
Jewett
Hamilton House
Hamilton Brook
OLD SOUTH ROAD
FIFE LANE
GREAT WORKS
Witchtrot
STRAFFORD CO
YORK CO
SCALE 1:24000
1 MILE
7000 FEET
1 KILOMETER
CONTOUR INTERVAL 20 FEET



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

15609.00

BRIDGE NO. 5610

PIN
15609.00

BRIDGE PLANS

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

SIGNATURE

P.E. NUMBER

DATE

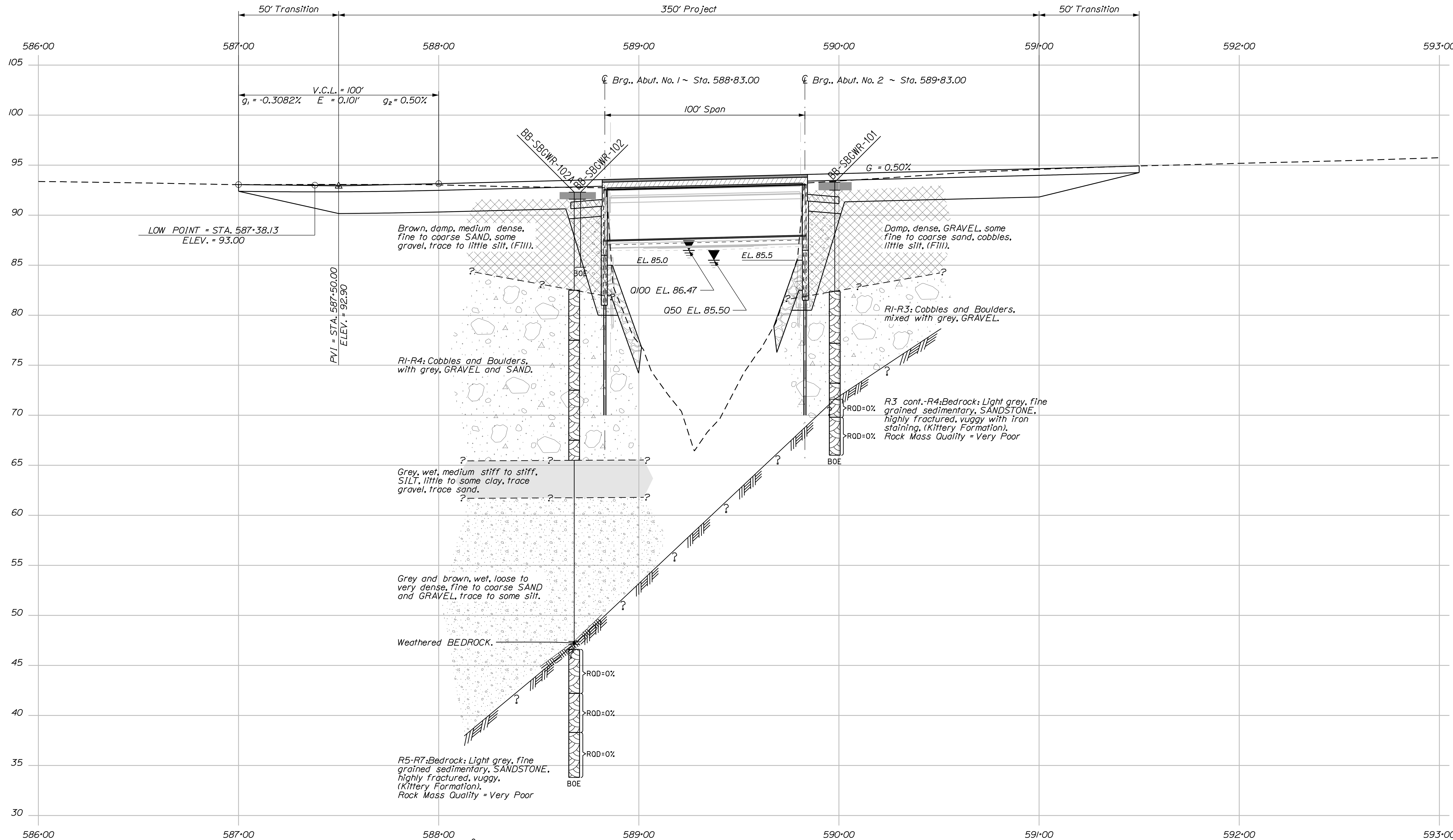
GREAT WORKS RIVER BRIDGE
GREAT WORKS RIVER
SOUTH BERWICK YORK COUNTY

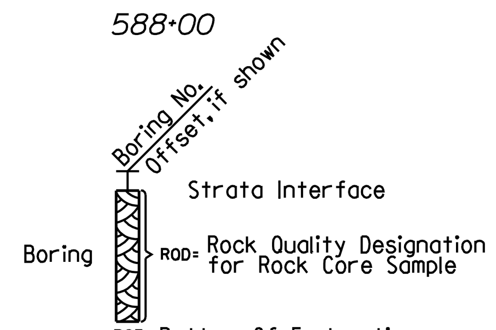
BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

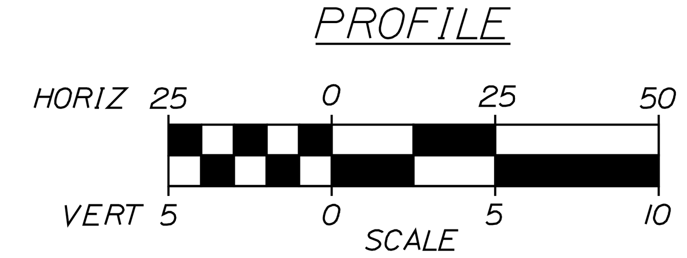
Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:1/30/2009

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:1/30/2009





PROFILE



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

15609.00

PIN 15609.00

BRIDGE PLANS

BRIDGE NO. 5610

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

SIGNATURE
P.E. NUMBER
DATE

GREAT WORKS RIVER BRIDGE
GREAT WORKS RIVER
SOUTH BERWICK   YORK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\msta\008_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:1/30/2009

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

Project: Great Works River Bridge #5610		Location: Route 236, South Berwick, Maine		Boring No.:	BB-SBGWR-102A
				PIN:	15609.00
Driller:	MaineDOT	Elevation (ft.)	92.3	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire/G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30" Auto
Date Start/Finish:	11/20,26-28/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	588+67.7, 10.0 Lt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
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, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		See Boring BB-SBGWR-102 for material description for 0.0-6.7' bgs.	
5							HW		Spun HW Casing from 5.0-10.5' bgs.	
6.70								85.60	Combination of BOULDERS, COBBLES and SOIL mixture from 6.7-25.0' bgs.	
	R1	60/47	9.80 - 14.80				NQ / NW		R1: Black and white, coarse grained Granite. Core Times (min:sec) 9.8-10.8' (2:08) 10.8-11.8' (2:58) 11.8-12.8' (2:40) 12.8-13.8' (2:35) 13.8-14.8' (3:05) 78% Recovery. Spun NW Casing from 10.5-45.2' bgs.	
	R2	60/51	14.80 - 19.80						R2: Black and white, coarse grained Granite. Core Times (min:sec) 14.8-15.8' (2:08) 15.8-16.8' (2:58) 16.8-17.8' (2:40) 17.8-18.8' (2:35) 18.8-19.8' (3:05) 85% Recovery	
	R3	60/47	19.80 - 24.80						R3: Black and white, coarse grained Granite and grey Sandstone. Core Times (min:sec) 19.8-20.8' (2:08) 20.8-21.8' (2:58) 21.8-22.8' (2:40) 22.8-23.8' (2:35) 23.8-24.8' (3:05) 78% Recovery. Pulled casing back, replaced spent spin shoe. Spun Casing to 29.5' bgs.	
	R4	24/4	24.80 - 26.80						R4: Sandstone. Core Times (min:sec) 24.8-25.8' (2:08) 25.8-26.8' (2:58) 17% Recovery	
26.80	1D/AB	24/14	26.80 - 28.80	3/2/2/2	4	5		65.50	(1D/A) 26.8-27.5' bgs. Grey, wet, loose, GRAVEL, some fine to coarse sand, some silt, some clay.	G#209926 A-4, GC-GM WC=26.2%
27.50								64.80	(1D/B) 27.5-28.8' bgs. Grey, wet, medium stiff, Clayey SILT, trace fine sand. Washed ahead of casing from 29.5-30.0' bgs.	G#209927 A-7-5, CL WC=36.4%
	2D/AC V1	24/20	30.00 - 32.00 / 30.17 - 30.54	WOH/2/5/7 Su=290/67 psf	7	9			(2D/A) 30.0-30.9' bgs. Grey, wet, stiff, SILT, some clay, trace sand. 55x110 mm vane raw torque readings: V1: 6.5/1.5 ft lbs. V1 stopped at 30.9' bgs, pulled back 0.1' and did shear test.	G#209928 A-4, CL-ML WC=27.2%
31.30								61.00	(2D/B) 30.9-31.3' bgs. Grey, wet, stiff, SILT, little clay, trace gravel and fine sand.	G#209929 A-4, CL-ML WC=22.1%
34.50								57.80	(2D/C) 31.3-32.0' bgs. Grey, wet, loose, silty fine SAND, trace gravel.	
	3D	24/14	35.00 - 37.00	17/25/20/21	45	58			Brown, moist, very dense, fine to coarse SAND, some gravel, some silt.	G#209930 A-2-4, SM WC=11.0%
38.00								54.30	Similar to above, but with cobbles.	
	MD	24/0	39.20 - 41.20	13/16/15/15	31	40			Failed sample attempt.	
43.50								48.80	Grey, wet, GRAVEL, some medium to coarse sand, trace silt.	
	4D	8.4/6	44.50 - 45.20	17/31(2.4")	---				Roller Coned ahead from 44.5-45.4' bgs, spun NW Casing from 44.5-45.5' bgs.	
44.90								47.40	Weathered ROCK.	
45.20	R5	52.8/47	45.70 - 50.10	RQD = 0%			NQ	47.10	Top of Bedrock at Elev. 47.4'. Roller Coned ahead from 45.5-45.7' bgs. NW Casing to 45.5' bgs. Bedrock: Light grey, fine grained, sedimentary, SANDSTONE, no obvious bedding, highly fractured, vuggy. (Kittery Formation). Rock Mass Quality = Verry Poor. R5:Core Times (min:sec) 45.7-46.7' (3:36) 46.7-47.7' (3:18) 47.7-48.7' (3:00) 48.7-49.7' (3:45) 49.7-50.1' (2:06) 90% Recovery. Core Blocked at 50.1' bgs.	
	R6	46.8/46.8	50.10 - 54.00	RQD = 0%					R6:Core Times (min:sec) 50.1-51.1' (4:01) 51.1-52.1' (3:25) 52.1-53.1' (2:58) 53.1-54.0' (3:50) 100% Recovery	
	R7	54/54	54.00 - 58.50	RQD = 0%					R7:Core Times (min:sec) 54.0-55.0' (2:40) 55.0-56.0' (3:12) 56.0-57.0' (2:40) 57.0-58.0' (2:47) 58.0-58.5' (1:54) 100% Recovery. Core Blocked at 58.5' bgs.	
58.50								33.80	Bottom of Exploration at 58.50 feet below ground surface.	

Remarks:

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

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

Page 1 of 1

Boring No.: BB-SBGWR-102A

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

Project: Great Works River Bridge #5610		Location: Route 236, South Berwick, Maine		Boring No.:	BB-SBGWR-102
				PIN:	15609.00
Driller:	MaineDOT	Elevation (ft.)	92.3	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire/G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30" Auto
Date Start/Finish:	11/19/07; 10:45-11:45	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	588+70.9, 10.0 Lt.	Casing ID/OD:	NW	Water Level*:	None Observed
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
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		Pavement	
0.70	1D	24/13	1.00 - 3.00	12/11/6/8	17	22		91.60	Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#209986 A-1-b, SW-SM WC=5.1%
3.80								88.50	Cobble from 3.8-4.5' bgs. (Fill).	
4.50	2D	24/12	5.00 - 7.00	13/6/6/9	12	15	30	87.80	Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#209987 A-1-b, SM WC=4.9%
							30			
							a50	84.80	a50 blows for 6". Broken rock at 7.0' bgs.	
7.50									Bottom of Exploration at 7.50 feet below ground surface. NO REFUSAL, "See Remarks"	

Remarks:

Abandoned hole at 7.5' bgs. Casing will not drive straight.

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-SBGWR-102

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

Project: Great Works River Bridge #5610		Location: Route 236, South Berwick, Maine		Boring No.:	BB-SBGWR-101
				PIN:	15609.00
Driller:	MaineDOT	Elevation (ft.)	93.3	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30" Auto
Date Start/Finish:	11/28/07-11/29/07	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	589+97.9, 9.4 Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		Pavement	
0.80	1D	24/13	1.30 - 3.30	10/13/16/12	29	37		92.50	Damp, dense, GRAVEL, some brown, fine to coarse sand, cobbles, little silt, (Fill).	G#210000 A-1-a, GW-GM WC=3.3%
									BOULDER from 4.4-5.4' bgs.	
	2D	24/5	5.50 - 7.50	7/11/30/24	41	53	SPUN HW		Damp, very dense, GRAVEL, cobbles, some brown, fine to coarse sand, little silt, (Fill). Spun HW Casing to 7.7' bgs.	
									COBBLE from 7.6-8.5' bgs. Roller Coned ahead from 7.7-8.6' bgs. Spun Casing from 7.7-10.5' bgs. Soil Layer from 8.5-9.8' bgs.	
									Granite BOULDER from 9.8-12.6' bgs. Roller Coned ahead from 10.5-10.9' bgs.	
11.00	R1	50.4/39	10.90 - 15.10				NW / NQ	82.30	R1: Granite and Sandstone. Core Times (min:sec) 10.9-11.9' (3:32) 11.9-12.9' (3:19) 12.9-13.9' (3:40) 13.9-14.9' (3:09) 14.9-15.1' (0:21). Spun NW Casing from 10.9-15.0' bgs. COBBLES and GRAVEL from 12.6-13.8' bgs. Granite BOULDER from 13.8-14.9' bgs. VOID from 14.9-16.1' bgs. Spun NW Casing from 15.0-20.0' bgs.	
	R2	48/22	16.10 - 20.10		---		NQ		Failed Sample attempt at 16.1' bgs 0" penatration/0" Recovery, 10 blows/0". Started R2. R2: Granite and Sandstone. Core Times (min:sec) 16.1-17.1' (2:29) 17.1-18.1' (1:38) 18.1-19.1' (2:42) 19.1-20.1' (1:37). Granite COBBLE from 16.1-16.9' bgs. Soil Layer from 16.9-17.2' bgs. Grey COBBLES from 17.2-17.8' bgs. Grey GRAVEL and COBBLES from 17.8-19.8' bgs. Soil Layer from 19.8-20.0' bgs. COBBLE from 20.0-20.6' bgs.	
	R3	40.8/26	20.10 - 23.50					71.60	R3: Sandstone. Core Times (min:sec) 20.1-21.1' (4:27) 21.1-22.1' (2:37) 22.1-23.1' (3:20) 23.1-23.5' (3:39). Soil Layer from 20.6-21.7' bgs.	
21.70	R4	45.6/45	23.50 - 27.30	RQD = 0%					Top of Bedrock at Elev. 71.6. Bedrock: Light grey, fine grained, sedimentary, SANDSTONE, no obvious bedding, highly fractured, vuggy, with iron staining, (Kittery Formation). Rock Mass Quality = Very Poor. R4:Core Times (min:sec) 23.5-24.5' (3:43) 24.5-25.5' (3:43) 25.5-26.5' (3:52) 26.5-27.3' (4:10) 98% Recovery	
27.30								66.00	Bottom of Exploration at 27.30 feet below ground surface.	

Remarks:

11/28/07; 12:00-14:30, 11/29/07; 9:15-14:30

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-SBGWR-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
15609.00

BRIDGE NO. 5610   PIN 15609.00   BRIDGE PLANS

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

GREAT WORKS RIVER BRIDGE
GREAT WORKS RIVER
SOUTH BERWICK   YORK COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

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

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: Great Works River Bridge #5610
Location: Route 236, South Berwick, Maine

Boring No.: BB-SBGWR-101
PIN: 15609.00

Driller:	MaineDOT	**Elevation (ft.)**	93.3	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30" Auto
Date Start/Finish:	11/28/07-11/29/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	589+97.9, 9.4 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		Pavement	
								92.5	0.8	
	1D	24/13	1.3 - 3.3	10/13/16/12	29	37			Damp, dense, GRAVEL, some brown, fine to coarse sand, cobbles, little silt, (Fill).	G#210000 A-1-a, GW-GM WC=3.3%
									BOULDER from 4.4-5.4' bgs.	
5	2D	24/5	5.5 - 7.5	7/11/30/24	41	53	SPUN HW		Damp, very dense, GRAVEL, cobbles, some brown, fine to coarse sand, little silt, (Fill). Spun HW Casing to 7.7' bgs. COBBLE from 7.6-8.5' bgs. Roller Coned ahead from 7.7-8.6' bgs. Spun Casing from 7.7-10.5' bgs. Soil Layer from 8.5-9.8' bgs. Granite BOULDER from 9.8-12.6' bgs.	
10	R1	50.4/39	10.9 - 15.1				NW NQ		Roller Coned ahead from 10.5-10.9' bgs.	
								82.3	11.0	
									R1: Granite and Sandstone Core Times (min:sec) 10.9-11.9' (3:32) 11.9-12.9' (3:19) 12.9-13.9' (3:40) 13.9-14.9' (3:09) 14.9-15.1' (0:21) Spun NW Casing from 10.9-15.0' bgs. COBBLES and GRAVEL from 12.6-13.8' bgs. Granite BOULDER from 13.8-14.9' bgs. VOID from 14.9-16.1' bgs.	
15	R2	48/22	16.1 - 20.1		---		NQ		Spun NW Casing from 15.0-20.0' bgs. Failed Sample attempt at 16.1' bgs 0" penatration/0" Recovery, 10 blows/ 0". Started R2. R2: Granite and Sandstone Core Times (min:sec) 16.1-17.1' (2:29) 17.1-18.1' (1:38) 18.1-19.1' (2:42) 19.1-20.1' (1:37) Granite COBBLE from 16.1-16.9' bgs. Soil Layer from 16.9-17.2' bgs. Grey COBBLES from 17.2-17.8' bgs.	
20	R3	40.8/26	20.1 - 23.5					71.6	Grey GRAVEL and COBBLES from 17.8-19.8' bgs. Soil Layer from 19.8-20.0' bgs. COBBLE from 20.0-20.6' bgs. R3: Sandstone Core Times (min:sec) 20.1-21.1' (4:27) 21.1-22.1' (2:37)	
	R4	45.6/45	23.5 - 27.3	RQD = 0%						
25										

Remarks:

11/28/07; 12:00-14:30, 11/29/07; 9:15-14:30

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:	Great Works River Bridge #5610	**Boring No.:**	BB-SBGWR-101
Location:	Route 236, South Berwick, Maine	**PIN:**	15609.00

Driller:	MaineDOT	**Elevation (ft.)**	93.3	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30" Auto
Date Start/Finish:	11/28/07-11/29/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	589+97.9, 9.4 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										22.1-23.1' (3:20) 23.1-23.5' (3:39) Soil Layer from 20.6-21.7' bgs. 21.7	
								66.0		Top of Bedrock at Elev. 71.6. Bedrock: Light grey, fine grained, sedimentary, SANDSTONE, no obvious bedding, highly fractured, vuggy, with iron staining, (Kittery Formation). Rock Mass Quality = Very Poor. R4:Core Times (min:sec) 23.5-24.5' (3:43) 24.5-25.5' (3:43) 25.5-26.5' (3:52) 26.5-27.3' (4:10) 98% Recovery 27.3	
30										**Bottom of Exploration at 27.30 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

11/28/07; 12:00-14:30, 11/29/07; 9:15-14:30

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: Great Works River Bridge #5610
Location: Route 236, South Berwick, Maine

Boring No.: BB-SBGWR-102
PIN: 15609.00

Driller:	MaineDOT	**Elevation (ft.)**	92.3	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30" Auto
Date Start/Finish:	11/19/07; 10:45-11:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	588+70.9, 10.0 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	91.6		Pavement —0.7	
	1D	24/13	1.0 - 3.0	12/11/6/8	17	22				Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#209986 A-1-b, SW-SM WC=5.1%
								88.5		—3.8	
								87.8		Cobble from 3.8-4.5' bgs, (Fill). —4.5	
5	2D	24/12	5.0 - 7.0	13/6/6/9	12	15	30			Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#209987 A-1-b, SM WC=4.9%
							30				
							a50	84.8		a50 blows for 6". Broken rock at 7.0' bgs. —7.5	
										Bottom of Exploration at 7.50 feet below ground surface. NO REFUSAL, "See Remarks"	
10											
15											
20											
25											

Remarks:

Abandoned hole at 7.5' bgs. Casing will not drive straight.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Works River Bridge #5610		**Boring No.:**	BB-SBGWR-102A
Location: Route 236, South Berwick, Maine		**PIN:**	15609.00

Driller:	MaineDOT	**Elevation (ft.)**	92.3	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30" Auto
Date Start/Finish:	11/20,26-28/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	588+67.7, 10.0 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
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WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			See Boring BB-SBGWR-102 for material description for 0.0-6.7' bgs.	
5							HW			Spun HW Casing from 5.0-10.5' bgs.	
								85.6		6.7 Combination of BOULDERS, COBBLES and SOIL mixture from 6.7-25.0' bgs.	
10	R1	60/47	9.8 - 14.8				NQ NW			R1: Black and white, coarse grained Granite. Core Times (min:sec) 9.8-10.8' (2:08) 10.8-11.8' (2:58) 11.8-12.8' (2:40) 12.8-13.8' (2:35) 13.8-14.8' (3:05) 78% Recovery Spun NW Casing from 10.5-45.2' bgs.	
15	R2	60/51	14.8 - 19.8							R2: Black and white, coarse grained Granite. Core Times (min:sec) 14.8-15.8' (2:08) 15.8-16.8' (2:58) 16.8-17.8' (2:40) 17.8-18.8' (2:35) 18.8-19.8' (3:05) 85% Recovery	
20	R3	60/47	19.8 - 24.8							R3: Black and white, coarse grained Granite and grey Sandstone. Core Times (min:sec) 19.8-20.8' (2:08) 20.8-21.8' (2:58) 21.8-22.8' (2:40) 22.8-23.8' (2:35) 23.8-24.8' (3:05) 78% Recovery Pulled casing back, replaced spent spin shoe. Spun Casing to 29.5' bgs.	
25	R4	24/4	24.8 - 26.8								

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Great Works River Bridge #5610 Location: Route 236, South Berwick, Maine	Boring No.: BB-SBGWR-102A PIN: 15609.00

Driller:	MaineDOT	**Elevation (ft.)**	92.3	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30" Auto
Date Start/Finish:	11/20,26-28/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	588+67.7, 10.0 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_V = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										R4: Sandstone. Core Times (min:sec) 24.8-25.8' (2:08) 25.8-26.8' (2:58) 17% Recovery	
	1D/AB	24/14	26.8 - 28.8	3/2/2/2	4	5		65.5		26.8	G#209926 A-4, GC-GM WC=26.2%
								64.8		(1D/A) 26.8-27.5' bgs. Grey, wet, loose, GRAVEL, some fine to coarse sand, some silt, some clay. 27.5	G#209927 A-7-5, CL WC=36.4%
										(1D/B) 27.5-28.8' bgs. Grey, wet, medium stiff, Clayey SILT, trace fine sand. Washed ahead of casing from 29.5-30.0' bgs.	
30	2D/AC V1	24/20	30.0 - 32.0 30.2 - 30.5	WOH/2/5/7 Su=290/67 psf	7	9				(2D/A) 30.0-30.9' bgs. Grey, wet, stiff, SILT, some clay, trace sand. 55x110 mm vane raw torque readings: V1: 6.5/1.5 ft lbs.	G#209928 A-4, CL-ML WC=27.2%
								61.0		V1 stopped at 30.9' bgs, pulled back 0.1' and did shear test. (2D/B) 30.9-31.3' bgs. Grey, wet, stiff, SILT, little clay, trace gravel and fine sand. 31.3	G#209929 A-4, CL-ML WC=22.1%
										(2D/C) 31.3-32.0' bgs. Grey, wet, loose, silty fine SAND, trace gravel.	
								57.8		34.5 Brown, moist, very dense, fine to coarse SAND, some gravel, some silt.	
35	3D	24/14	35.0 - 37.0	17/25/20/21	45	58					G#209930 A-2-4, SM WC=11.0%
								54.3		38.0 Similar to above, but with cobbles.	
	MD	24/0	39.2 - 41.2	13/16/15/15	31	40				Failed sample attempt.	
40											
								48.8		43.5 Grey, wet, GRAVEL, some medium to coarse sand, trace silt.	
	4D	8.4/6	44.5 - 45.2	17/31(2.4")	---					Roller Coned ahead from 44.5-45.4' bgs, spun NW Casing from 44.5-45.5' bgs.	
45	R5	52.8/47	45.7 - 50.1	RQD = 0%			NQ	47.4 47.1		44.9 Weathered ROCK. 45.2	
										Top of Bedrock at Elev. 47.4'. Roller Coned ahead from 45.5-45.7' bgs. NW Casing to 45.5' bgs. Bedrock: Light grey, fine grained, sedimentary, SANDSTONE, no obvious bedding, highly fractured, vuggy. (Kittery Formation). Rock Mass Quality = Verry Poor. R5:Core Times (min:sec)	
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Great Works River Bridge #5610	**Location:**	Route 236, South Berwick, Maine	**Boring No.:**	BB-SBGWR-102A
				PIN:	15609.00
Driller:	MaineDOT	**Elevation (ft.)**	92.3	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30" Auto
Date Start/Finish:	11/20,26-28/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	588+67.7, 10.0 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	R6	46.8/46.8	50.1 - 54.0	RQD = 0%						45.7-46.7' (3:36) 46.7-47.7' (3:18) 47.7-48.7' (3:00) 48.7-49.7' (3:45) 49.7-50.1' (2:06) 90% Recovery Core Blocked at 50.1' bgs. R6:Core Times (min:sec) 50.1-51.1' (4:01) 51.1-52.1' (3:25) 52.1-53.1' (2:58) 53.1-54.0' (3:50) 100% Recovery	
	R7	54/54	54.0 - 58.5	RQD = 0%						R7:Core Times (min:sec) 54.0-55.0' (2:40) 55.0-56.0' (3:12) 56.0-57.0' (2:40) 57.0-58.0' (2:47) 58.0-58.5' (1:54) 100% Recovery Core Blocked at 58.5' bgs.	
55											
								33.8		58.5 **Bottom of Exploration at 58.50 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): South Berwick Project Number: 15609.00

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-SBGWR-102A, 1D/A	588+67.7	10.0 Lt.	26.8-27.5	209926	2	26.2			GC-GM	A-4	III
BB-SBGWR-102A, 1D/B	588+67.7	10.0 Lt.	27.5-28.8	209927	2	36.4			CL	A-7-5	IV
BB-SBGWR-102A, 2D/A	588+67.7	10.0 Lt.	30.0-30.9	209928	2	27.2			CL-ML	A-4	IV
BB-SBGWR-102A, 2D/B	588+67.7	10.0 Lt.	30.9-31.3	209929	2	22.1			CL-ML	A-4	IV
BB-SBGWR-102A, 3D	588+67.7	10.0 Lt.	35.0-37.0	209930	2	11.0			SM	A-2-4	II
BB-SBGWR-102,1D	588+70.9	10.0 Lt.	1.0-3.0	209986	1	5.1			SW-SM	A-1-b	0
BB-SBGWR-102,2D	588+70.9	10.0 Lt.	5.0-7.0	209987	1	4.9			SM	A-1-b	II
BB-SBGWR-101,1D	589+97.9	9.4 Rt.	1.3-3.3	210000	3	3.3			GW-GM	A-1-a	0

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

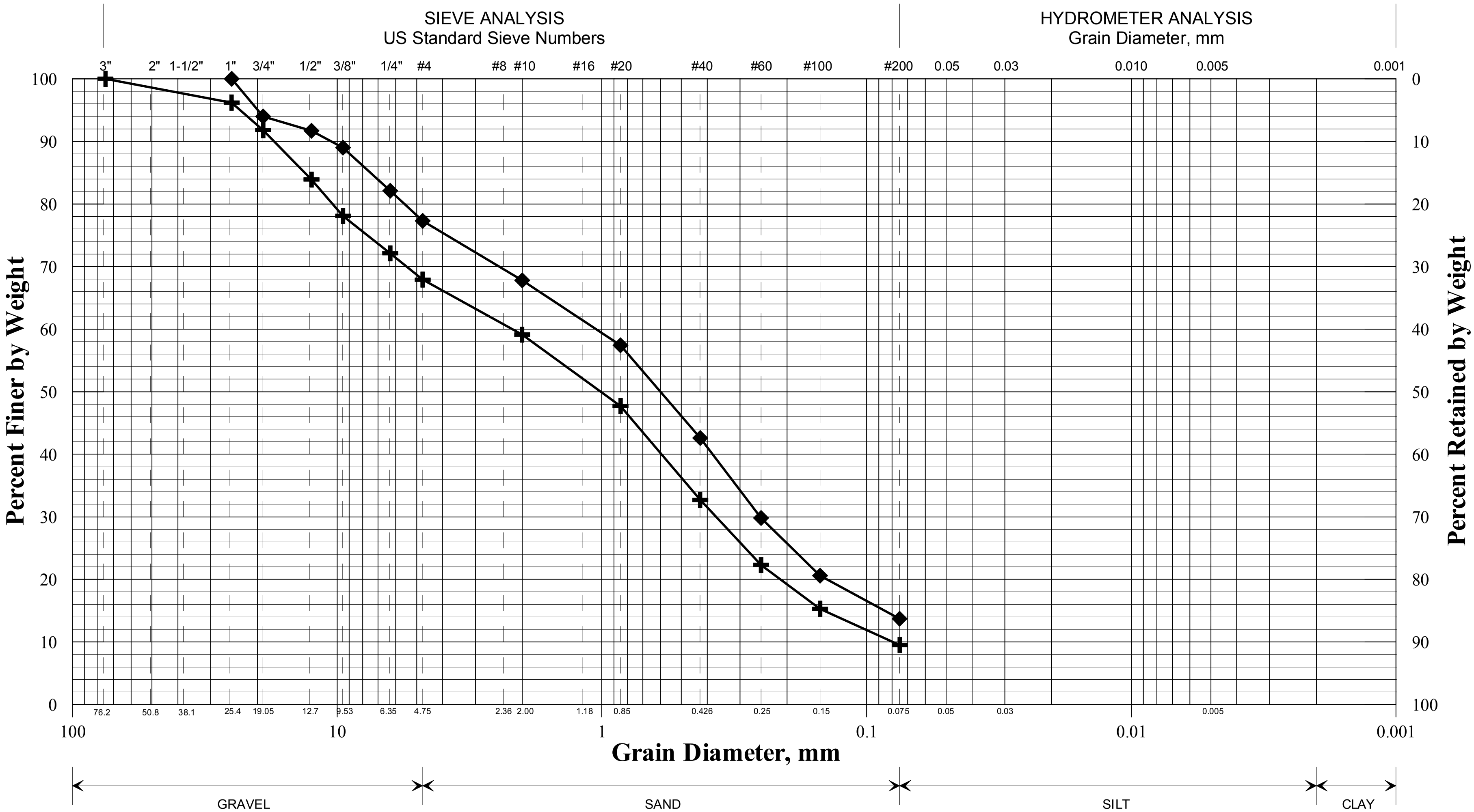
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

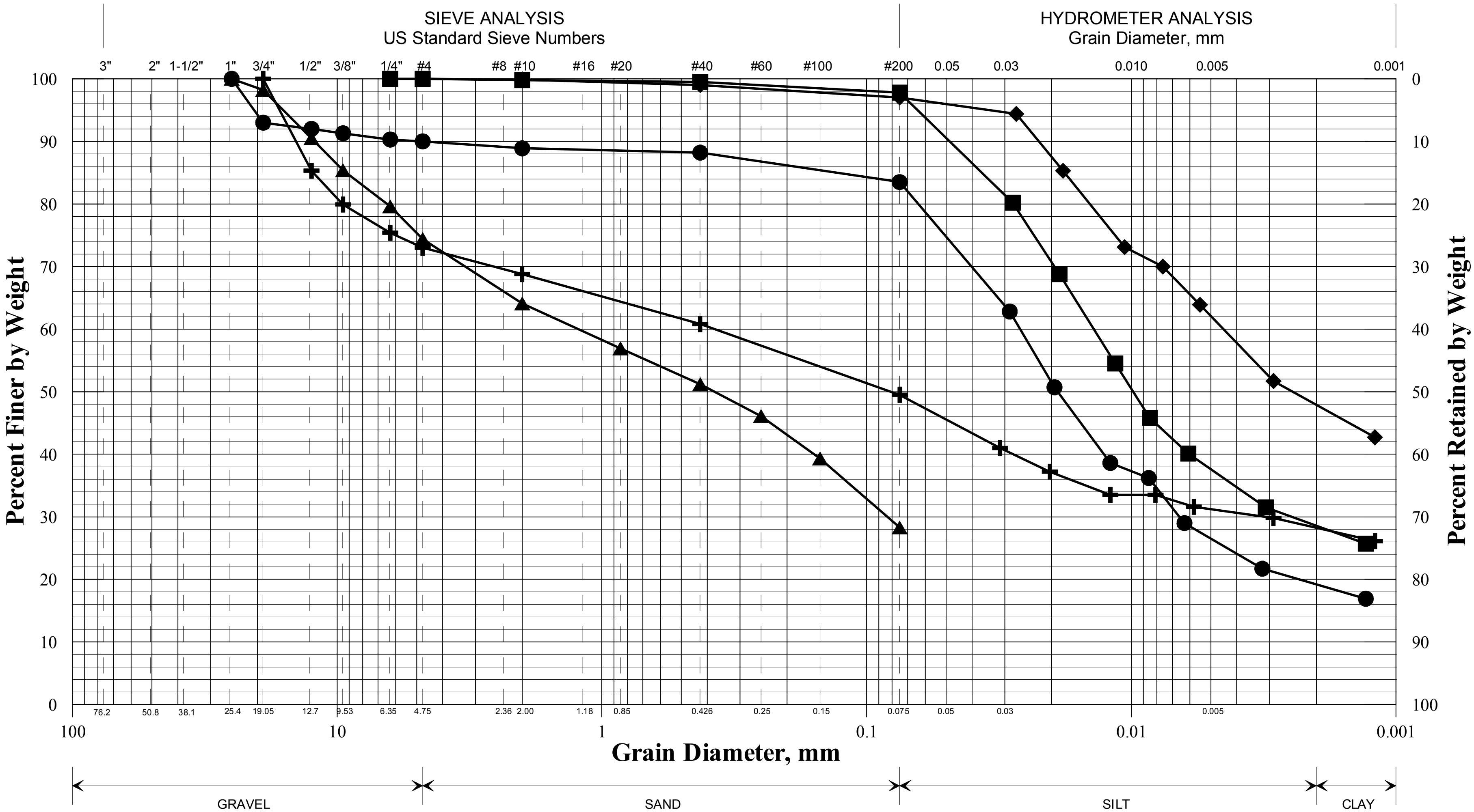


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGWR-102/1D	588+70.9	10.0 LT	1.0-3.0	SAND, some gravel, trace silt.	5.1			
◆	BB-SBGWR-102/2D	588+70.9	10.0 LT	5.0-7.0	SAND, some gravel, little silt.	4.9			
■									
●									
▲									
✖									

PIN	
015609.00	
Town	
South Berwick	
Reported by/Date	
WHITE, TERRY A	3/24/2008

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



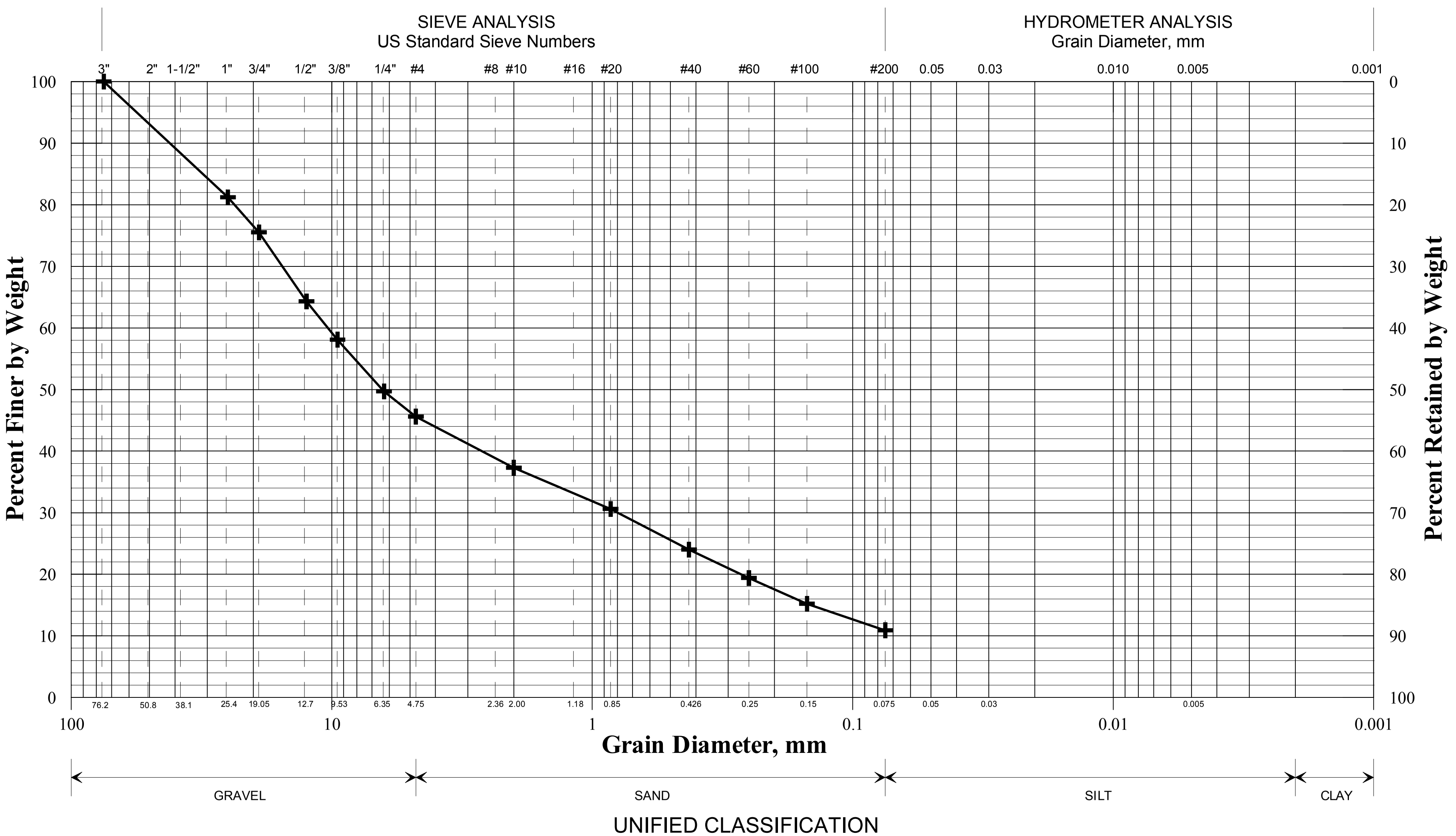
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGWR-102A/1D(A)	588+67.7	10.0 LT	26.8-27.5	GRAVEL, some sand, some silt, some clay	26.2			
◆	BB-SBGWR-102A/1D(B)	588+67.7	10.0 LT	27.5-28.8	Clayey SILT, trace sand.	36.4			
■	BB-SBGWR-102A/2D(A)	588+67.7	10.0 LT	30.0-30.9	SILT, some clay, trace sand.	27.2			
●	BB-SBGWR-102A/2D(B)	588+67.7	10.0 LT	30.9-31.3	SILT, little clay, trace gravel, trace sand.	22.1			
▲	BB-SBGWR-102A/3D	588+67.8	10.0 LT	35.0-37.0	SAND, some silt, some gravel.	11.0			
✖									

PIN
015609.00
Town
South Berwick
Reported by/Date
WHITE, TERRY A 4/29/2008

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGWR-101/1D	589+97.9	9.4 RT	1.3-3.3	GRAVEL, some sand, little silt.	3.3			
◆									
■									
●									
▲									
✖									

PIN
015609.00
Town
South Berwick
Reported by/Date
WHITE, TERRY A 4/29/2008

SHEET 3

Appendix C

Calculations

Abutment Foundations: Integral driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007 with 2008 Interims

Look at the following piles:

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

Note: All matrices set up in this order

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

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

Where λ=normalized column slenderness factor

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

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

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

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

STRENGTH LIMIT STATE:

Factored Resistance:

Strength Limit State Axial Resistance factor for H-piles in compression under severe driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance:

eq. 6.9.2.1-1 $P_f := \phi_c \cdot P_n$ $P_f = \begin{pmatrix} 388 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot kip$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

Where λ=normalized column slenderness factor

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

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

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

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

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

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1 $P_f := \phi\cdot P_n$

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

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

Service/Extreme Limit States

GEOTECHNICAL RESISTANCE OF H-PILES

Assume piles will be end bearing on bedrock driven through overlying fill, cobbles and boulders, silt and gravel.

Bedrock Type: Sandstone - sedimentary Kittery Formation
RQD = 0%. Use RQD = 0% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

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

Note: All matrices set up in this order

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

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

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

Calculate pile box area:

$A_{box} := \overrightarrow{(d \cdot b)}$ $\quad A_{box} = \begin{pmatrix} 141.8901 \\ 198.5018 \\ 203.2318 \\ 211.5159 \end{pmatrix} \cdot in^2$

Calculate 33% of box area

$A_{33\%box} := A_{box} \cdot 0.33$ $\quad A_{33\%box} = \begin{pmatrix} 46.8237 \\ 65.5056 \\ 67.0665 \\ 69.8002 \end{pmatrix} \cdot in^2$

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

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

q_u for sandstone compressive strength
ranges for 9,700 to 25,000 psi

use $\sigma_c := 15000 \cdot psi$

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

Spacing of discontinuities: $c := 1 \cdot in$ Bedrock is vuggy

Aperture of discontinuities: $\delta := \frac{1}{8} \cdot in$ Vug openings are ~ 1/8 inch

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

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

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}} \qquad K_{sp} = \begin{pmatrix} 0.0497 \\ 0.0495 \\ 0.0494 \\ 0.0494 \end{pmatrix}$$

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

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \cdot 3 \qquad q_a = \begin{pmatrix} 322 \\ 320 \\ 320 \\ 320 \end{pmatrix} \cdot ksf$$

(multiply by 3 as K_{sp} includes a factor of safety of 3)

Nominal Geotechnical Tip Resistance, R_p:

$$R_p := \overrightarrow{\left(q_a \cdot A_{33\%box}\right)} \qquad R_p = \begin{pmatrix} 105 \\ 146 \\ 149 \\ 155 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Factored Geotechnical Tip Resistance, R_f at Strength Limit State:

Resistance factor, end bearing on rock (CGS method):

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat}

$$R_{tipf} := \phi_{stat} \cdot R_p \qquad R_{tipf} = \begin{pmatrix} 47 \\ 66 \\ 67 \\ 70 \end{pmatrix} \cdot kip$$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Nominal Geotechnical Tip Resistance, R_p:

$$R_p := \overrightarrow{(q_a \cdot A_{33\%box})}$$

$$R_p = \begin{pmatrix} 105 \\ 146 \\ 149 \\ 155 \end{pmatrix} \cdot kip$$

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

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$$\phi := 1.0$$

Total Factored Geotechnical Resistance, R_g:

$$R_{pfac} := R_p \cdot \phi$$

$$R_{pfac} = \begin{pmatrix} 105 \\ 146 \\ 149 \\ 155 \end{pmatrix} \cdot kip$$

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

Service/Extreme Limit States

Use of the Canadian Geotechnical Society method as recommended by AASHTO results in very low and unrealistic resistance values. Look at Goodman's Method for comparison.

Geotechnical Resistance by Goodman's Method

Based on Unconfined Compressive Strength of Bedrock

Reference: Principles of Foundation Engineering, BM Das, Fourth Edition
Section 9.14 Point Bearing Capacity of Piles on Rock

Bedrock Type: Sandstone - sedimentary Kittery Formation
RQD = 0%. Use RQD = 0% and ϕ = 27 to 45 deg (Das Table 9.4 pg 599)

σ_c for sandstone - compressive strength ranges from 10,000 to 20,000 psi (Das,Table 9.3)

use $\sigma_c := 15000 \cdot psi$

$$\phi := 30 \cdot deg \qquad N_\phi := \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^2 \qquad N_\phi = 3$$

$$q_{nom\_goodman} := \left(\frac{\sigma_c}{5}\right) \cdot (N_\phi + 1)$$

Divide by 5 to adjust for scale effect in rock (pg 599)

$$q_{nom\_goodman} = 12 \cdot ksi$$

Nominal Geotechnical Tip Resistance:

At Abutment No. 1 a soil plug should form - use 33% of box area

$$R_{nom\_goodman\_A1} := q_{nom\_goodman} \cdot A_{box} \cdot 0.33 \qquad R_{nom\_goodman\_A1} = \begin{pmatrix} 562 \\ 786 \\ 805 \\ 838 \end{pmatrix} \cdot kip$$

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

At Abutment No. 2 no soil plug will form - use area of steel

$$R_{nom\_goodman\_A2} := q_{nom\_goodman} \cdot A_s \qquad R_{nom\_goodman\_A2} = \begin{pmatrix} 186 \\ 257 \\ 313 \\ 413 \end{pmatrix} \cdot kip$$

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

Evaluate additional skin friction using FHWA Program Driven 1.0
Driven software uses Nordlund/Thurman Method for side friction resistance in cohesionless soils.

From Driven: Skin friction for Abutment No. 1:

$$R_{skin\_A1} := \begin{pmatrix} 209 \\ 279 \\ 303 \\ 339 \end{pmatrix} \cdot kip$$

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

See Driven runs next pages.

Skin friction will no develop at Abutment No. 2 due to short pile.

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\SB112X53.DVN
Project Name: Great Works River Bridge — Project Date: 11/24/2008
Project Client: South Berwick
Computed By: km
Project Manager: JWentworth

PILE INFORMATION

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

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	8.00 ft
	- Driving/Restrike	8.00 ft
	- Ultimate:	8.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	27.00 ft	10.00%	135.00 pcf	34.0/34.0	Nordlund
2	Cohesive	4.00 ft	10.00%	115.00 pcf	1000.00 psf	T-79 Steel
3	Cohesionless	14.00 ft	10.00%	125.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.01 Kips	0.01 Kips
7.99 ft	10.78 Kips	4.28 Kips	15.05 Kips
8.01 ft	10.83 Kips	4.29 Kips	15.12 Kips
17.01 ft	42.51 Kips	6.88 Kips	49.38 Kips
26.01 ft	88.89 Kips	7.91 Kips	96.80 Kips
26.99 ft	94.82 Kips	7.91 Kips	102.74 Kips
27.01 ft	94.94 Kips	0.97 Kips	95.91 Kips
30.99 ft	116.20 Kips	0.97 Kips	117.17 Kips
31.01 ft	116.31 Kips	3.55 Kips	119.86 Kips
40.01 ft	172.91 Kips	3.55 Kips	176.46 Kips
44.99 ft	208.88 Kips	3.55 Kips	212.43 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\SB114X73.DVN
Project Name: Great Works River Bridge — Project Date: 11/24/2008
Project Client: South Berwick
Computed By: km
Project Manager: JWentworth

PILE INFORMATION

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

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	8.00 ft
	- Driving/Restrike	8.00 ft
	- Ultimate:	8.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	27.00 ft	10.00%	135.00 pcf	34.0/34.0	Nordlund
2	Cohesive	4.00 ft	10.00%	115.00 pcf	1000.00 psf	T-79 Steel
3	Cohesionless	14.00 ft	10.00%	125.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.01 Kips	0.01 Kips
7.99 ft	14.69 Kips	5.91 Kips	20.59 Kips
8.01 ft	14.76 Kips	5.92 Kips	20.68 Kips
17.01 ft	57.93 Kips	9.50 Kips	67.43 Kips
26.01 ft	121.14 Kips	10.93 Kips	132.07 Kips
26.99 ft	129.23 Kips	10.93 Kips	140.16 Kips
27.01 ft	129.38 Kips	1.34 Kips	130.71 Kips
30.99 ft	153.98 Kips	1.34 Kips	155.32 Kips
31.01 ft	154.12 Kips	4.90 Kips	159.03 Kips
40.01 ft	230.45 Kips	4.90 Kips	235.35 Kips
44.99 ft	278.94 Kips	4.90 Kips	283.84 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\SB114X89.DVN
Project Name: Great Works River Bridge   Project Date: 11/24/2008
Project Client: South Berwick
Computed By: km
Project Manager: JWentworth

PILE INFORMATION

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

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	8.00 ft
	- Driving/Restrike	8.00 ft
	- Ultimate:	8.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	27.00 ft	10.00%	135.00 pcf	34.0/34.0	Nordlund
2	Cohesive	4.00 ft	10.00%	115.00 pcf	1000.00 psf	T-79 Steel
3	Cohesionless	14.00 ft	10.00%	125.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.01 Kips	0.01 Kips
7.99 ft	16.12 Kips	7.20 Kips	23.33 Kips
8.01 ft	16.20 Kips	7.22 Kips	23.42 Kips
17.01 ft	63.60 Kips	11.58 Kips	75.18 Kips
26.01 ft	132.99 Kips	13.33 Kips	146.32 Kips
26.99 ft	141.88 Kips	13.33 Kips	155.20 Kips
27.01 ft	142.03 Kips	1.63 Kips	143.66 Kips
30.99 ft	166.80 Kips	1.63 Kips	168.44 Kips
31.01 ft	166.95 Kips	5.98 Kips	172.93 Kips
40.01 ft	250.23 Kips	5.98 Kips	256.21 Kips
44.99 ft	303.14 Kips	5.98 Kips	309.12 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\SB114X17.DVN
Project Name: Great Works River Bridge   Project Date: 11/24/2008
Project Client: South Berwick
Computed By: km
Project Manager: JWentworth

PILE INFORMATION

Pile Type: H Pile - HP14X117
Top of Pile: 0.00 ft
Perimeter Analysis: Pile
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	8.00 ft
	- Driving/Restrike	8.00 ft
	- Ultimate:	8.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	27.00 ft	10.00%	135.00 pcf	34.0/34.0	Nordlund
2	Cohesive	4.00 ft	10.00%	115.00 pcf	1000.00 psf	T-79 Steel
3	Cohesionless	14.00 ft	10.00%	125.00 pcf	32.0/32.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.01 Kips	0.01 Kips
7.99 ft	18.25 Kips	9.50 Kips	27.74 Kips
8.01 ft	18.34 Kips	9.51 Kips	27.85 Kips
17.01 ft	71.97 Kips	15.27 Kips	87.24 Kips
26.01 ft	150.50 Kips	17.56 Kips	168.07 Kips
26.99 ft	160.56 Kips	17.56 Kips	178.12 Kips
27.01 ft	160.72 Kips	2.15 Kips	162.87 Kips
30.99 ft	185.78 Kips	2.15 Kips	187.93 Kips
31.01 ft	185.94 Kips	7.88 Kips	193.82 Kips
40.01 ft	279.48 Kips	7.88 Kips	287.37 Kips
44.99 ft	338.92 Kips	7.88 Kips	346.80 Kips

STRENGTH LIMIT STATE:

Factored Geotechnical Tip Resistance, $R_{f\_goodman}$ at Strength Limit State:

Resistance factor, end bearing on rock (use same factor as CGS method):

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Resistance factor, skin friction - sand (Norlund/Therman Method):

$\phi_{stat2} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored resistance of Single Pile in Axial Compression

For Abutment No. 1:

$$R_{f\_goodman\_A1} := \phi_{stat} \cdot R_{nom\_goodman\_A1} + \phi_{stat2} \cdot R_{skin\_A1}$$

$$R_{f\_goodman\_A1} = \begin{pmatrix} 347 \\ 479 \\ 499 \\ 529 \end{pmatrix} \cdot kip$$

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

Strength Limit State

For Abutment No. 2 (no sdkin friction) :

$$R_{f\_goodman\_A2} := \phi_{stat} \cdot \left(R_{nom\_goodman\_A2}\right)$$

$$R_{f\_goodman\_A2} = \begin{pmatrix} 84 \\ 116 \\ 141 \\ 186 \end{pmatrix} \cdot kip$$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Nominal Geotechnical Tip Resistance by Goodman Method:

At Abutment No. 1:

$$R_{nom\_goodman\_A1} = \begin{pmatrix} 562 \\ 786 \\ 805 \\ 838 \end{pmatrix} \cdot kip$$

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

At Abutment No. 2:

$$R_{nom\_goodman\_A2} = \begin{pmatrix} 186 \\ 257 \\ 313 \\ 413 \end{pmatrix} \cdot kip$$

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 Geotechnical Tip Resistance, $R_{f\_goodman\_se}$ at Service and Extreme Limit States:

For Abutment No. 1:

$$R_{f\_goodman\_se\_A1} := \left(R_{skin\_A1} + R_{nom\_goodman\_A1}\right)\cdot\phi$$

$$R_{f\_goodman\_se\_A1} = \begin{pmatrix} 771 \\ 1065 \\ 1108 \\ 1177 \end{pmatrix}\cdot kip$$

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

Service/Extreme Limit States

For Abutment No. 2: (no skin friction due to very short pile)

$$R_{f\_goodman\_se\_A2} := \left(R_{nom\_goodman\_A2}\right)\cdot\phi$$

$$R_{f\_goodman\_se\_A2} = \begin{pmatrix} 186 \\ 257 \\ 313 \\ 413 \end{pmatrix}\cdot kip$$

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

Service/Extreme Limit States

Goodman's Method results more realistic resistance values. Use these values for report.

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

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

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

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

Compute Resistance that can be achieved in a drivability analysis:

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

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

$\phi_{dyn} := 0.65$

There are 5 piles at each abutment. No reduction of Φ_{dyn} is necessary.

Look at Resistances for both abutments:
Abutment No. 1 pile length = 35 feet
Abutment No. 2 pile length = 10 feet

Abutment No. 1: Pile Size = 12 x 53

Assume Contractor will use a Delmag D19-42 hammer to install 12 x 53 piles

State of Maine Dept. Of Transportation — 24-Nov-2008
South Berwick Great Works River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
455.0	44.86	3.51	6.2	8.90	19.60
456.0	44.78	3.48	6.2	8.91	19.55
457.0	44.97	3.49	6.2	8.91	19.61
458.0	44.97	3.48	6.3	8.93	19.62
459.0	45.08	3.49	6.3	8.94	19.67
460.0	45.16	3.49	6.3	8.95	19.70
461.0	45.15	3.48	6.3	8.95	19.68
462.0	45.30	3.48	6.4	8.96	19.72
463.0	45.38	3.49	6.4	8.97	19.75
464.0	45.33	3.46	6.4	8.98	19.73

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_12x53\_A1\_factored} := 459 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_12x53\_A1\_factored} = 298 \cdot kip$$

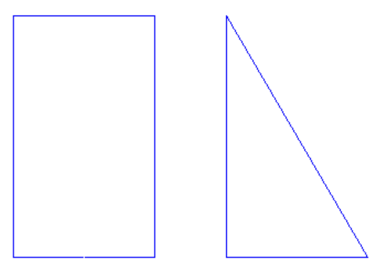
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_12x53\_A1\_servext} := 459 \cdot kip$$

DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	15.50	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

Abutment No. 1: Pile Size = 14 x 73

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 24-Nov-2008
South Berwick Great Works River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
510.0	44.59	0.94	2.9	7.39	33.96
511.0	44.64	0.95	2.9	7.39	33.97
512.0	44.69	0.95	2.9	7.40	33.96
513.0	44.75	0.95	2.9	7.40	33.96
514.0	44.81	0.95	3.0	7.41	33.96
515.0	44.97	0.96	3.0	7.42	34.11
516.0	45.03	0.96	3.0	7.42	34.11
517.0	45.12	0.96	3.0	7.43	34.13
518.0	45.12	0.96	3.0	7.42	34.02
519.0	45.18	0.97	3.0	7.43	34.03

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x73\_A1\_factored} := 516 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_14x73\_A1\_factored} = 335 \cdot kip$$

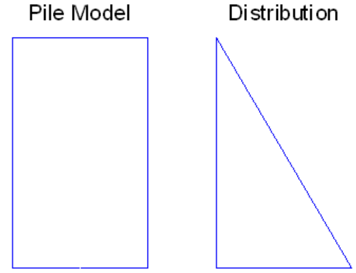
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_14x73\_A1\_servext} := 516 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

Abutment No. 1: Pile Size = 14 x 89

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 89 piles

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South Berwick Great Works River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
670.0	44.69	2.46	4.3	8.06	35.09
671.0	44.70	2.46	4.3	8.07	35.05
672.0	44.78	2.48	4.3	8.07	35.08
673.0	44.79	2.47	4.3	8.07	35.04
674.0	44.88	2.49	4.3	8.08	35.16
675.0	44.93	2.49	4.4	8.08	35.14
676.0	44.98	2.49	4.4	8.09	35.10
677.0	45.03	2.51	4.4	8.10	35.22
678.0	45.09	2.51	4.4	8.10	35.18
679.0	45.20	2.52	4.4	8.11	35.29

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x89\_A1\_factored} := 677 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_14x89\_A1\_factored} = 440 \cdot kip$$

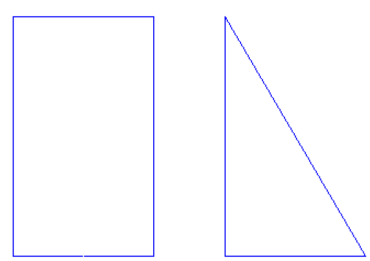
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_14x89\_A1\_servext} := 677 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	26.10	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

Abutment No. 1: Pile Size = 14 x 117

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 117 piles

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South Berwick Great Works River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
995.0	45.02	3.72	8.7	9.10	36.50
996.0	44.96	3.70	8.8	9.11	36.44
997.0	45.27	3.72	8.6	9.20	36.87
998.0	45.07	3.73	8.8	9.11	36.49
999.0	45.10	3.74	8.8	9.12	36.55
1000.0	45.03	3.71	8.9	9.12	36.40
1001.0	45.05	3.72	8.9	9.13	36.45
1002.0	45.15	3.73	8.9	9.13	36.53
1003.0	45.42	3.75	8.8	9.22	36.97
1004.0	45.19	3.75	9.0	9.14	36.57

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x117\_A1\_factored} := 996 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_14x117\_A1\_factored} = 647 \cdot kip$$

Service and Extreme Limit States:

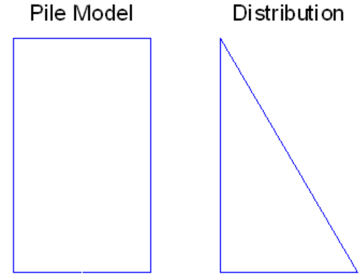
$$\phi := 1.0$$

$$R_{dr\_14x117\_A1\_servext} := 996 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

Abutment No. 2: Pile Size = 12 x 53

Assume Contractor will use a Delmag D19-42 hammer to install 12 x 53 piles

State of Maine Dept. Of Transportation
South Berwick Great Works River Bridge

24-Nov-2008
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
340.0	44.76	0.00	3.8	8.05	15.28
341.0	44.82	0.00	3.8	8.05	15.27
342.0	44.95	0.00	3.8	8.06	15.31
343.0	45.02	0.01	3.8	8.06	15.29
344.0	45.06	0.04	3.9	8.06	15.25
345.0	45.21	0.09	3.9	8.07	15.29
346.0	45.27	0.09	3.9	8.07	15.26
347.0	45.31	0.08	3.9	8.08	15.22
348.0	45.48	0.06	3.9	8.09	15.26
349.0	45.54	0.05	3.9	8.09	15.24

Limit to driving stress to 45 ksi

Strength Limit State:

$R_{dr\_12x53\_A2\_factored} := 343 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_12x53\_A2\_factored} = 223 \cdot kip$

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

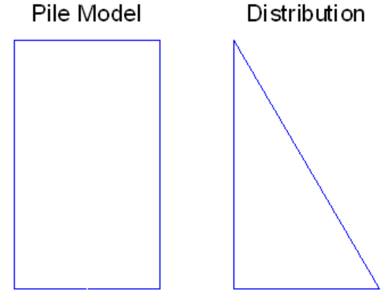
$R_{dr\_12x53\_A2\_servext} := 343 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	10.00	ft
Pile Penetration	10.00	ft
Pile Top Area	15.50	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

Abutment No. 2 Pile Size = 14 x 73

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 24-Nov-2008
South Berwick Great Works River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
465.0	44.85	0.00	2.8	7.27	28.02
466.0	44.93	0.00	2.8	7.27	28.02
467.0	45.01	0.00	2.8	7.27	28.03
468.0	45.10	0.00	2.9	7.28	28.04
469.0	45.20	0.00	2.9	7.28	28.04
470.0	45.28	0.00	2.9	7.29	28.04
471.0	45.37	0.00	2.9	7.29	28.04
472.0	45.31	0.00	2.9	7.29	27.90
473.0	45.40	0.00	2.9	7.29	27.91
474.0	45.47	0.00	2.9	7.30	27.91

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x73\_A2\_factored} := 467 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_14x73\_A2\_factored} = 304 \cdot kip$$

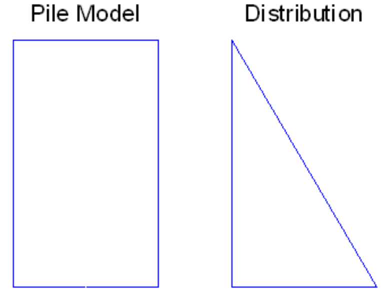
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_14x73\_A2\_servext} := 467 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	10.00	ft
Pile Penetration	10.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

Abutment No. 2 Pile Size = 14 x 89

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 89 piles

State of Maine Dept. Of Transportation — 24-Nov-2008
South Berwick Great Works River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
595.0	44.85	0.00	3.8	7.68	27.14
596.0	44.83	0.00	3.8	7.67	27.02
597.0	44.84	0.00	3.8	7.67	26.96
598.0	45.01	0.00	3.8	7.68	27.07
599.0	44.94	0.00	3.9	7.68	26.92
600.0	45.09	0.00	3.9	7.68	27.05
601.0	45.16	0.00	3.9	7.69	27.00
602.0	45.20	0.00	3.9	7.69	26.96
603.0	45.25	0.00	3.9	7.69	26.97
604.0	45.32	0.00	3.9	7.69	26.93

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x89\_A2\_factored} := 600 \cdot kip \cdot \phi_{dyn}$$

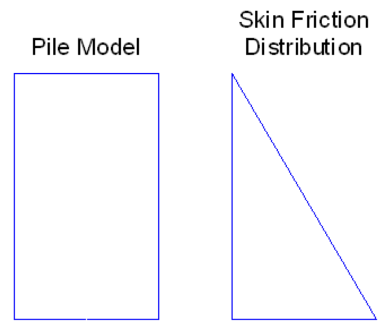
$$R_{dr\_14x89\_A2\_factored} = 390 \cdot kip$$

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

$$R_{dr\_14x89\_A2\_servext} := 600 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	10.00	ft
Pile Penetration	10.00	ft
Pile Top Area	26.10	in2



Res. Shaft = 25 %
(Proportional)

Abutment No. 2

Pile Size = 14 x 117

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 117 piles

State of Maine Dept. Of Transportation — 24-Nov-2008
South Berwick Great Works River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
840.0	44.92	0.00	5.8	8.29	27.09
841.0	44.91	0.00	5.8	8.30	27.17
842.0	45.01	0.00	5.8	8.31	27.10
843.0	45.04	0.00	5.8	8.30	27.07
844.0	45.07	0.00	5.8	8.31	27.16
845.0	45.12	0.00	5.9	8.31	27.09
846.0	45.17	0.00	5.9	8.31	27.06
847.0	45.13	0.00	5.9	8.32	27.14
848.0	45.24	0.01	5.9	8.32	27.07
849.0	45.29	0.01	5.9	8.32	27.06

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x117\_A2\_factored} := 842 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_14x117\_A2\_factored} = 547 \cdot kip$$

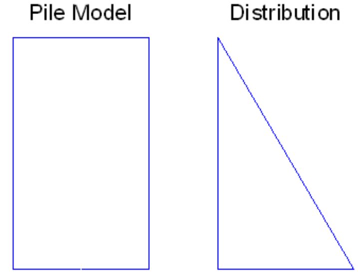
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_14x117\_A2\_servext} := 842 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	10.00	ft
Pile Penetration	10.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

H-piles Depth to Fixity

Abutment No. 1: Soil conditions at boring BB-SBGWR-102A
27 ft of fill (gravel, cobbles and boulders) over 4 ft of silt
over 13 ft of sand over bedrock.

Consider Pile sizes:
HP 12x53
HP 14x73
HP 14x 89
HP 14x117

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

LRFD Eq.10.7.3.13.4-2 for fixity in feet: $1.8*T_H = 1.8*(E_p I_w/n_h)^{0.2}$ (in sands)
E_p Young's modulus of pile in ksi
I_w moment of inertia of pile in ft^4
n_h= rate of increase of soil modulus with depth for sands
as specified in Table C10.4.6.3-2 in ksi/ft

E_p = Steel modulus: $E_{steel} := 29000 \cdot ksi$

Moment of Inertia: $I_w := \begin{pmatrix} 393 \\ 729 \\ 904 \\ 1220 \end{pmatrix} \cdot in^4$

use X - X axis
Y-Y axis will give even lower numbers

Rate of increase of soil modulus with depth:
for submerged loose sand

$n_h := 0.208 \cdot \frac{ksi}{ft}$

T_H parameter:

$$T_H := \left(\frac{E_{steel} \cdot I_w}{n_h}\right)^{0.2} \qquad T_H = \begin{pmatrix} 4.84 \\ 5.47 \\ 5.71 \\ 6.06 \end{pmatrix} \cdot ft$$

Depth of Fixity:

$$D_{fixH} := 1.8 \cdot T_H$$

$$D_{fixH} = \begin{pmatrix} 9 \\ 10 \\ 10 \\ 11 \end{pmatrix} \cdot ft$$

Depth to fixity for H-piles

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

seems low.....

Look at Fixity using MassHighway Bridge Manual

The length of pile from the base of the abutment to the point of fixity shall be the equvalent length, L_e, as defined as the theoretical equivalent length of free standing column with fixed/fixed support conditions translated though a distance δ_T.

The equivalent length of pile Le is determined from the regression equation:

$L_e = A(EI/d)+B(\delta_T)+C$

where: A, B, & C are equation coefficients from Table 1 Mass Highway Bridge Manual Section 3.9.6.3
E = Modulus of elasticity of pile material
I = Moment of inertia
d = pile section depth
δ_T = pile head horizontal displacement

Look at four pile sizes:
HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices in this order

E = Steel modulus: $E := 29000 \cdot ksi$

Moment of Inertia: $I_w := \begin{pmatrix} 127 \\ 261 \\ 326 \\ 443 \end{pmatrix} \cdot in^4$ Use Y-Y axis for weak axis bending

Depth of pile $d_p := \begin{pmatrix} 299 \\ 446 \\ 351 \\ 361 \end{pmatrix} \cdot mm$ $d_p = \begin{pmatrix} 11.77 \\ 17.56 \\ 13.82 \\ 14.21 \end{pmatrix} \cdot in$

Assume pile head displacement: $\delta_T := 10 \cdot mm$ $\delta_T = 0.3937 \cdot in$

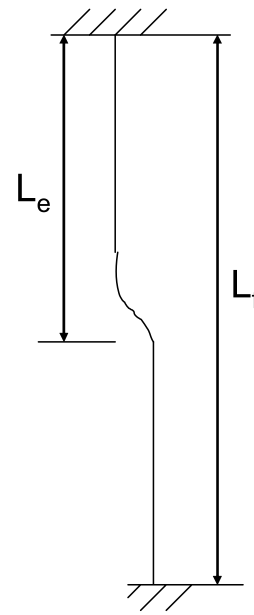
From Mass Highway Bridge Manual Section 3.9.6.3 Table 1
Assume soil condituions = Dry peastone over wet or dry sand

$$A := 7.4 \cdot 10^{-6} \cdot \frac{mm}{N \cdot mm \cdot 10^3}$$

$$B := 12 \cdot \frac{mm}{mm}$$

$$C := 2.3 \cdot mm \cdot 10^3$$

$$L_e := A \cdot \left(\overrightarrow{\frac{E \cdot I_w}{d_p}}\right) + B \cdot \delta_T + C \qquad L_e = \begin{pmatrix} 8.8 \\ 9.12 \\ 9.82 \\ 10.42 \end{pmatrix} \cdot ft$$



From Mass Highway Bridge Manual Section 3.9.6.3 Table 1
Fixity Ratio L_f/L_e = 2.2
Solve for L_f - length for fixity

$$L_f := L_e \cdot 2.2 \qquad L_f = \begin{pmatrix} 19 \\ 20 \\ 22 \\ 23 \end{pmatrix} \cdot ft$$

Piles at Abutment No. 2 will not achieve fixity.

Abutment and Wingwall Passive and Active Earth Pressure:

For cases where interface friction is considered (for gravity structures) use Coulomb Theory

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

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

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

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

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

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

$K_p = 6.89$

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

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

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

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

$K_{p\_rank} = 3.25$

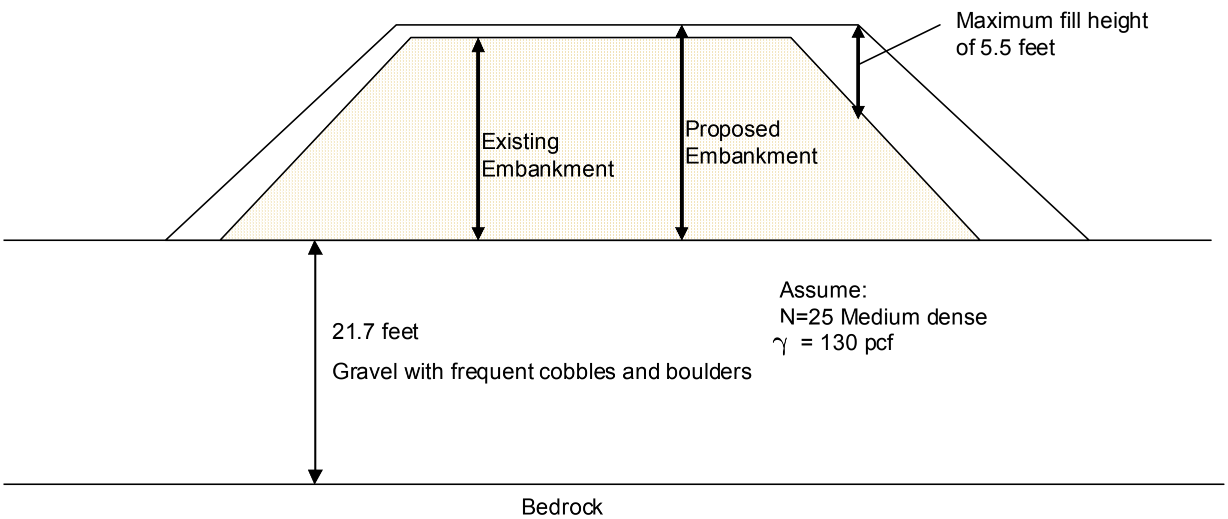
Bowles does not recommend the use of the Rankine Method for K_p when β>0.

Settlement Analysis:

Reference: FHWA Soils and Foundation Workshop Manual (FHWA HI-88-009) Bazaraa 1967 pg 168

Look at maximum fill location:

Widening of roadway directly behind Abutment No.2
Station 589+84.25
Maximum of ~5.5 feet of fill
Use BB-SBGWR-101 soil profile



Divide gravel layer up into 4 layers:

Layer 1:	$H_1 := 5 \cdot ft$	$N_1 := 20$
Layer 2:	$H_2 := 5 \cdot ft$	$N_2 := 25$
Layer 3:	$H_3 := 5 \cdot ft$	$N_3 := 20$
Layer 4:	$H_4 := 6.7 \cdot ft$	$N_4 := 25$

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Embank. slope a = 40.00(ft)
Embank. width b = 57.00(ft)
p load/unit area = 687.50(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 40.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	687.50
1.00	682.00
2.00	676.33
3.00	670.35
4.00	663.91
5.00	656.93
6.00	649.35
7.00	641.15
8.00	632.36
9.00	623.01
10.00	613.16
11.00	602.90
12.00	592.31
13.00	581.46
14.00	570.45
15.00	559.34
16.00	548.20
17.00	537.10
18.00	526.08
19.00	515.18
20.00	504.45
21.00	493.90
22.00	483.56

at 2.5 feet $\Delta\sigma_{z1} := 673.31 \cdot psf$

at 7.5 feet $\Delta\sigma_{z2} := 636.76 \cdot psf$

at 12.5 feet $\Delta\sigma_{z3} := 586.89 \cdot psf$

at 18.4 feet $\Delta\sigma_{z4} := 521.72 \cdot psf$

Layer 1: $H_1 := 5 \cdot ft$

Unit weight of sand and gravel: $\gamma_{gr} := 130 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{1o} := \frac{H_1}{2} \cdot \gamma_{gr}$ $\sigma_{1o} = 325 \cdot psf$ at mid-point

SPT N-value (bpf) $N_1 = 20$ At P_o = 325 psf N'/N = $r1 := 2.0$

Corrected Blow Count $N'_1 := r1 \cdot N_1$ $N'_1 = 40$

From Figure 13 using the "well graded fine to medium silty SAND" curve

Bearing Capacity Index: $C1 := 97$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z1} = 673.31 \cdot psf$

Layer 2: $H_2 := 5 \cdot ft$

Unit weight of sand and gravel: $\gamma_{gr} := 130 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{2o} := H_1 \cdot \gamma_{gr} + \frac{H_2}{2} \cdot \gamma_{gr}$ $\sigma_{2o} = 975 \cdot psf$ at mid-point

SPT N-value (bpf) $N_2 = 25$ At P_o = 975 psf N'/N = $r2 := 1.3$

Corrected Blow Count $N'_2 := r2 \cdot N_2$ $N'_2 = 33$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C2 := 110$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z2} = 636.76 \cdot psf$

Layer 3: $H_3 := 5 \cdot ft$

Unit weight of sand and gravel: $\gamma_{gr} := 130 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{3o} := (H_1 + H_2) \cdot \gamma_{gr} + \frac{H_3}{2} \cdot \gamma_{gr}$ $\sigma_{3o} = 1625 \cdot psf$ at mid-point

SPT N-value (bpf) $N_3 = 20$ At P_o = 1625 psf N'/N = $r3 := 0.98$

Corrected Blow Count $N'_3 := r3 \cdot N_3$ $N'_3 = 20$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C3 := 77$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z3} = 586.89 \cdot psf$

Layer 4: $H_4 = 6.7 \cdot ft$

Unit weight of sand and gravel: $\gamma_{gr} := 130 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{4o} := (H_1 + H_2 + H_3) \cdot \gamma_{gr} + \frac{H_4}{2} \cdot \gamma_{gr}$ $\sigma_{4o} = 2385.5 \cdot psf$ at mid-point

SPT N-value (bpf) $N_4 = 25$ At P_o = 2386 psf N'/N = $r4 := 0.88$

Corrected Blow Count $N'_4 := r4 \cdot N_4$ $N'_4 = 22$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C4 := 82$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z4} = 521.72 \cdot psf$

Settlement at each layer Interbedded sand and gravel:

$$\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{z1}}{\sigma_{1o}}\right) \qquad \Delta H_1 = 0.3 \cdot in$$

$$\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{2o} + \Delta\sigma_{z2}}{\sigma_{2o}}\right) \qquad \Delta H_2 = 0.12 \cdot in$$

$$\Delta H_3 := H_3 \cdot \frac{1}{C3} \cdot \log\left(\frac{\sigma_{3o} + \Delta\sigma_{z3}}{\sigma_{3o}}\right) \qquad \Delta H_3 = 0.1 \cdot in$$

$$\Delta H_4 := H_4 \cdot \frac{1}{C4} \cdot \log\left(\frac{\sigma_{4o} + \Delta\sigma_{z4}}{\sigma_{4o}}\right) \qquad \Delta H_4 = 0.08 \cdot in$$

Total settlement =

$$\Delta H_{A2} := \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 \qquad \Delta H_{A2} = 0.6091 \cdot in$$ At Abutment No. 2

Frost Protection:

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
South Berwick, Maine
DFI = 1200 degree-days

From the lab testing: soils are coarse grained assume a water content = ~5%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1900 frost penetration = 76.1 inches

$$\text{Frost\_depth} := 73.1\text{in} \qquad \text{Frost\_depth} = 6.0917 \cdot \text{ft}$$

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

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Sanford

--- ModBerg Results ---

Project Location: Sanford 2 NNW, Maine

Air Design Freezing Index = 1123 F-days
N-Factor = 0.80
Surface Design Freezing Index = 898 F-days
Mean Annual Temperature = 46.8 deg F
Design Length of Freezing Season = 116 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	55.3	5.0	125.0	24	28	1.2	1.3	900

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

Total Depth of Frost Penetration = 4.61 ft = 55.3 in.

Use Modberg Frost Depth = 4.5 feet for design

Seismic:

South Berwick Great Works Rive Bridge
Date and Time: 5/5/2008 1:33:46 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 03908
Zip Code Latitude = 43.233800
Zip Code Longitude = -070.791400
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.101	PGA - Site Class B
0.2	0.192	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 - 03908
Zip Code Latitude = 43.233800
Zip Code Longitude = -070.791400
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	0.161	As - Site Class D
0.2	0.308	SDs - Site Class D
1.0	0.109	SD1 - Site Class D

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 610
STONE FILL, RIPRAP, STONE BLANKET,
AND STONE DITCH PROTECTION

Add the following paragraph to Section 610.02:

Materials shall meet the requirements of the following Sections of Special Provision 703:

Stone Fill	703.25
Plain and Hand Laid Riprap	703.26
Stone Blanket	703.27
Heavy Riprap	703.28
Definitions	703.32

Add the following paragraph to Section 610.032.a.

Stone fill and stone blanket shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following paragraph to Section 610.032.b:

Riprap shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following to Section 610.032:

Section 610.032.d. The grading of riprap, stone fill, stone blanket and stone ditch protection shall be determined by the Resident by visual inspection of the load before it is dumped into place, or, if ordered by the Resident, by dumping individual loads on a flat surface and sorting and measuring the individual rocks contained in the load. A separate, reference pile of stone with the required gradation will be placed by the Contractor at a convenient location where the Resident can see and judge by eye the suitability of the rock being placed during the duration of the project. The Resident reserves the right to reject stone at the job site or stockpile, and in place. Stone rejected at the job site or in place shall be removed from the site at no additional cost to the Department.

SPECIAL PROVISION
SECTION 703
AGGREGATES

Replace subsections 703.25 through 703.28 with the following:

703.25 Stone Fill   Stones for stone fill shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for stone fill shall be angular and rough. Rounded, subrounded, or long thin stones will not be allowed. Stone for stone fill may be obtained from quarries or by screening oversized rock from earth borrow pits. The maximum allowable length to thickness ratio will be 3:1. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (500 lbs) shall have a maximum dimension of approximately 36 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension of 12 inches (200 lbs).

703.26 Plain and Hand Laid Riprap   Stone for riprap shall consist of hard, sound durable rock that will not disintegrate by exposure to water or weather. Stone for riprap shall be angular and rough. Rounded, subrounded or long thin stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (200 lbs) shall have an average dimension of approximately 12 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension greater than 9 inches (50 lbs).

703.27 Stone Blanket   Stones for stone blanket shall consist of sound durable rock that will not disintegrate by exposure to water or weather. Stone for stone blanket shall be angular and rough. Rounded or subrounded stones will not be allowed. Stones may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (300 lbs) shall have minimum dimension of 14 inches, and the maximum stone size (3000 lbs) shall have a maximum dimension of approximately 66 inches. Fifty percent of the stones by volume shall have average dimension greater than 24 inches (1000 lbs).

703.28 Heavy Riprap   Stone for heavy riprap shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for heavy riprap shall be angular and rough. Rounded, subrounded, or thin, flat stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for heavy riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (500 lbs) shall have minimum dimension of 15 inches, and at least fifty percent of the stones by volume shall have an average dimension greater than 24 inches (1000 lbs).

Add the following paragraph:

703.32 Definitions (ASTM D 2488, Table 1).

Angular: Particles have sharp edges and relatively plane sides with unpolished surfaces
Subrounded: Particles have nearly plane sides but have well-rounded corners and edges
Rounded: Particles have smoothly curved sides and no edges