MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

BARTLETT BRIDGE
OVER GILMAN STREAM
STATE ROUTE 16
NEW PORTLAND, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement of Bartlett Bridge on State Route 16 over Gilman Stream in New Portland, Maine. The proposed replacement bridge will consist of a 105 foot single span composite steel plate girder superstructure supported on short 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 existing abutments will be left in place as a scour counter measure. It is likely that the existing gravity abutments to remain will not be stable with the existing superstructure removed and earth pressure still applied. If this is the case, the abutments will be stabilized with rock anchors through the stem walls or tiebacks with concrete deadmen, if necessary. 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. Short piles supporting integral abutments should be designed in accordance with the design example found in Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" Chapter 5 and Appendix B and the MaineDOT Bridge Design Guide. The pile should be end bearing, driven to the required resistance on or within the bedrock. Using the assumption that 50 ksi steel will be used; the factored axial structural resistance of the piles exceeds the factored axial geotechnical capacity and therefore the geotechnical resistance governs. The Contractor is required to perform a wave equation analysis and dynamic pile analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.52. The factored pile load should be shown on the plans. The piles should be oriented for weak axis bending. Driven 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.

Stub Abutments and Wingwalls - Integral abutments and wingwalls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, superstructure loads, creep, and temperature and shrinkage deformations of the superstructure. They shall be designed for all relevant service and strength limit states. Integral abutment and wingwall sections shall be designed to resist Coulomb passive earth pressure, K_p, equal to 6.89. The designer may consider the Rankine passive earth pressure, K_p, of 3.26 when designing integral wingwall extensions. Wing wall sections that are independent of the stub abutment should be designed for the Rankine active earth pressure, K_a, of 0.307.

Bearing Resistance - It is anticipated that the project retaining walls and independent return wingwalls will be founded on the native soils at the site. Bearing resistance for any structure founded on the native soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf. A factored bearing resistance of 4 ksf may be used when analyzing the service limit state and for preliminary sizing of footings. In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

Scour and Riprap- If using integral abutment sat the site, pile lengths will be short and, therefore, scour protection will be critical, especially if the dam downstream were removed. For scour protection, the integral abutments should be moved away from the channel. Since the proposed bridge design will rely on the existing abutments and wingwalls to provide lateral support and scour protection for the integral abutment piles, it is critical that concrete spalling and deterioration of the existing substructures, especially below the water line, be repaired. The interface contact of the abutment toes with the bedrock bearing stratum should also be examined and improved, if necessary, with grouting. For scour protection, any footings for wingwalls or retaining walls, 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. Riprap conforming to item number 703.26 of the Standard Specification shall be placed at the toes of abutments, wingwalls and retaining walls. 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.

Settlement - The west bridge approach will be widened in order to accommodate the change in horizontal alignment of the proposed bridge which will shift the roadway slightly upstream (north). Additionally, the vertical alignment of the bridge will be raised approximately 2.5 feet at the west end and 4 feet at the east end. The maximum additional fill to be placed at the site is 4 feet and will result in approximately 1/2 to 1 inch of settlement. This settlement is anticipated to occur during construction and will have minimal effect of the finished structure.

Frost Protection - Any foundation placed on granular subgrade soils it should be founded a minimum of 6.5 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils and not those founded on bedrock. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT Bridge Design Guide.

Retaining Wall - A Precast Concrete Modular Gravity (PCMG) wall founded on bedrock was chosen to retain approach fills at the northwest corner of the bridge near the existing stacked granite retaining wall. This wall shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The wall will be approximately 19 feet high and 45 feet long. The PCMG Wall will be backfilled with flowable fill to improve performance and increase durability. Temporary sheeting will be required to maintain the adjacent driveway during construction. Bearing resistance for the PCMG wall founded on a leveling slab on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 30 ksf.

Seismic Design Considerations - Bartlett Bridge is located on State Route 16 and is not on the National Highway System (NHS). Therefore, the bridge is not considered to be functionally important. Since the bridge construction costs will not exceed $10 million, the bridge is not classified as a major structure. In conformance with the MaineDOT Bridge

Design Guide, these criteria eliminate the requirement to design the bridge substructures for seismic earth loads.

Construction Considerations - For construction of the PCMG wall construction of cofferdams and earth support systems may be required. It is recommended that the PCMG wall be constructed in dry conditions. Excavation of bedrock materials for placement of the leveling slab may require drilling and blasting techniques. The Contractor may need to conduct pre-and post-blast surveys in accordance with industry standards.

There is a potential for the remaining portion of the existing abutments to interfere with the installation of the integral abutment piles. If the existing abutments are encountered during pile installation the Contractor shall drill and clean a stable hole of the required diameter and length to provide minimum 6 foot long piles at the west abutment and 20 foot long piles at the east abutment. The drilling method selected by the Contractor should be able to drill a stable hole without detriment to the existing abutments. This condition should be noted on the plans and the work will be considered incidental to pile installation.

1.0 INTRODUCTION

A subsurface investigation and geotechnical design for the replacement of Bartlett Bridge on State Route 16 over Gilman Stream in New Portland, Somerset 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 1921 and consists of a concrete T-beam superstructure with two 35 foot spans. The bridge is supported on a mass concrete pier and mass concrete gravity abutments. The as-built plans show the abutments and pier to be founded on bedrock. Underwater inspection reports also indicate the substructures are founded on bedrock. The bridge lies within the headwater of a privately owned dam, located approximately 50 to 60 feet downstream. Maintenance records indicate that the bridge is in fair to good structural condition. Maintenance photographs show concrete spalling below the water line for all of the substructure units. It is understood that the existing bridge superstructure will be completely removed and replaced, the existing pier will be completely removed to below streambed, and the top portion of the existing abutments will be removed to an elevation 2 feet below the proposed low chord (2 feet of removal at the west abutment and a few inches at the east abutment).

The proposed bridge will consist of a 105 foot, single-span, composite steel plate girder superstructure supported on short H-pile supported integral abutments. This abutment type is considered experimental and is proposed based on the results to date of MaineDOT's short-pile integral abutment study. The existing abutments and wingwalls will be left in place as a scour countermeasure. It is likely that the existing gravity abutments to remain will not be stable with the existing superstructure removed and earth pressure still applied. If this is the case, the abutments will be stabilized with rock anchors through the stem walls or tiebacks with concrete deadmen, if necessary.

The horizontal alignment of the proposed bridge will be shifted slightly upstream (north) at the west end of the bridge and in the west approach. At all other locations, the proposed horizontal alignment matches the existing. The vertical alignment of the bridge will be raised approximately 2.5 feet at the west end and 4 feet at the east end. The bridge will have a 12° skew (head on the right). In order to retain approach fills and minimize slope impacts at the northwest corner of the bridge a Precast Concrete Modular Gravity (PCMG) Wall with moment distribution slab will be constructed at the northwest corner of the bridge near the existing stacked granite retaining wall.

The existing bridge will be closed to traffic during construction.

2.0 GEOLOGIC SETTING

Bartlett Bridge on Route 16 in New Portland crosses Gilman Stream approximately 0.2 miles east of Route 146 as shown on *Sheet 1 - Location Map* found at the end of this report. Gilman Stream flows in a southerly direction to the Carrabassett River which flows into the Kennebec River.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits. Soils in the site area are generally comprised of silt, clay, sand and minor amounts of gravel. Sand is dominant in some areas, but may be underlain by finer-grained sediments. The unit 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. Additional geologic units mapped nearby the site are ice-contact glaciofluvial deposits (sand, gravel and silt) and swamp, marsh and bog deposits (peat, muck, clay, silt and sand). The project is located in the area of the inland marine limit of the late-glacial marine submergence, as mapped by Thompson and others (1983).

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as Devonian muscovite-biotite granite. This igneous intrusion is identified as the Rome/Norridgewock pluton. The bedrock is anticipated to be hard and sound.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling two (2) test borings at the site. Test boring BB-NPGS-101 was drilled behind the location of Abutment No. 1 (west). Test boring BB-NPGS-102 was drilled behind the location of Abutment No. 2 (east). The exploration locations are shown on *Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile* found at the end of this report. The borings were drilled on October 16, 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 3 - Boring Logs* found end of this report.

The borings were drilled using driven cased wash boring and solid stem auger techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. The MaineDOT drill rig is newly equipped with a CME automatic hammer to drive the split spoon. The hammer was calibrated by MaineDOT in August of 2007 and was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. 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, identified field and laboratory testing requirements and 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 three (3) standard grain size analyses, three (3) grain size analysis with hydrometer and one (1) Atterberg Limits test. 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 3 - 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 clayey silt layer overlying bedrock. An interpretive subsurface profile depicting the site stratigraphy is show on *Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile* found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

Fill Materials. Beneath the pavement, a layer of fill materials was encountered in both borings. This layer was found to be brown, damp to wet, fine to coarse SAND, with little to trace silt, trace gravel and occasional cobbles with depth. The thickness of the fill layer ranged from approximately 12 feet in boring BB-NPGS-101 to approximately 20.3 feet in boring BB-NPGS-102. Corrected SPT N-values in the fill layer ranged from 4 to 21 blows per foot (bpf) indicating that the soil is loose to medium dense in consistency. Water contents from three (3) samples obtained within this layer range from approximately 5% to 18%. Three (3) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-2-4 by the AASHTO Classification System and a SP-SM by the Unified Soil Classification System.

Clayey Silt. Beneath the fill material a layer of clayey silt was encountered in both of the borings. This layer was found to be brown and olive changing to grey with depth, wet, clayey SILT, with trace sand and organics. The thickness of the clayey silt layer ranged from approximately 2.2 feet in boring BB-NPGS-101 to approximately 4.7 feet in boring BB-NPGS-102. Corrected SPT N-values obtained in the clayey silt layer ranged from 18 to >50 bpf indicating that the soil is very stiff to hard in consistency. Vane shear testing conducted within the clayey silt layer showed an undrained shear strength of approximately 1482 psf while the remolded shear strength was approximately 223 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the clayey silt was determined to have sensitivity of approximately 6.6 and is classified as sensitive. Water contents from three (3) samples obtained within this layer range from approximately 23% to 50%. Three (3) grain size analyses with hydrometer conducted on samples from this layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a SC-SM or CL-ML by the Unified Soil Classification System.

The following table summarizes the results of the Atterberg Limits test made from a sample of the clayey silt:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-NPGS-102 4D	Clayey Silt	23.5	28	22	6	0.25

Interpretation of these results indicates that the clayey silt is some-to-heavily overconsolidated as the natural water content is close to the plastic limit.

Bedrock. Bedrock was encountered and cored in both of the borings. The following table presents the bedrock findings:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB-NPGS-101/ Abutment No. 1	14.2 feet	368.3 feet	93 - 100%
BB-NPGS-102/ Abutment No. 2	28.0 feet	356.2 feet	65 - 95%

The bedrock at the site can be identified as grey, fine-grained, Devonian muscovite-biotite granite. The bedrock is a part of the Rome/Norridgewock pluton. The bedrock is generally massive with few sub-horizontal joints and minor oxidation. The RQD of the bedrock ranged from 65 to 100% indicating a rock of fair to excellent quality.

Groundwater. Groundwater was observed at a depths ranging from approximately 9.9 feet to 11.8 feet below the ground surface at the boring locations. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

The Preliminary Design Report (PDR) prepared for MaineDOT by T.Y. Lin International of Falmouth, Maine in June 2006 for the project considers several alternatives for the replacement of Bartlett Bridge. These alternatives included:

- One and two span structures
- Reuse of the existing substructures with widening
- Integral abutments on H-piles with or without rock sockets
- Semi-integral abutments on H-piles with or without rock sockets
- Full height gravity abutments founded on soil
- Full height gravity abutments founded on bedrock
- Leaving the existing abutments in place as a scour counter measure
- Removal of the existing abutments and placing new abutments in their location

After consideration of all of the alternatives, short pile integral abutments located behind the existing abutments (which will remain in place) was determined to be the most desirable foundation solution because they require minimal future maintenance. The presence of shallow bedrock at the site indicates that integral abutment piles would typically be socketed to achieve fixity. Preliminary results of a MaineDOT short-pile integral abutment study show that fixity may not be necessary.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for stub abutments founded on a single row of integral H-piles driven to bedrock which has been identified as the optimal substructure for the site. The existing abutments will be left in place as a scour counter measure. 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.

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 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with driving

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

points to protect the tips, improve penetration and improve friction a the pile tip to support a pinned pile tip assumption.

Pile lengths at the proposed abutments may be estimated based on the following data:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Rock Quality Designation	Estimated Pile Length
Abutment #1 BB-NPGS-101	374 feet	14.2 feet	368.3 feet	100%	6 feet
Abutment #2 BB-NPGS-102	375 feet	28.0 feet	356.2 feet	65 - 95%	20 feet

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed below. Short piles (less than 12 feet) should be designed in accordance with the design example found in Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" Chapter 5 and Appendix B and the MaineDOT Bridge Design Guide (BDG).

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. 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, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2. As the proposed piles for the project will be short and will not achieve fixity, the resistance for the pile will be determined for structural compliance with interaction equation.

7.1.1 Strength Limit State

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. The H-piles are fully embedded and λ shall be taken as 0. The factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60.

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

The drivability of the four proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conduced. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65. Table 10.5.5.2.3-3 requires that no less than three to four dynamic tests be conducted for sites with low to medium variability. As it is likely that only two dynamic tests will be conducted at the site, this resistance factor has been reduced by 20% resulting in a ϕ_{dyn}=0.52. The calculated drivability resistance values exceed the factored geotechnical resistance which will control the design.

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

Factored Axial Resistances for Abutment Piles at the Strength Limit State

Pile Section	Factored Resistance (kips)			
	Structural Resistance[a]	Geotechnical Resistance	Drivability	Design Resistance
Abutment 1				
12 x 53	465	296	569	296
14 x 73	642	395	760	395
14 x 89	783	404	778	404
14 x 117	1032	420	808	420
Abutment 2				
12 x 53	465	316	607	316
14 x 73	642	420	809	420
14 x 89	783	431	829	431
14 x 117	1032	449	864	449

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

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation.

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

7.1.2 Service/Extreme Limit States

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

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

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

Pile Section	Factored Resistance (kips)		
	Structural Resistance	Geotechnical Resistance	Design Resistance
Abutment 1			
12 x 53	775	657	657
14 x 73	1070	878	878
14 x 89	1305	898	898
14 x 117	1720	934	934
Abutment 2			
12 x 53	775	702	702
14 x 73	1070	934	934
14 x 89	1305	958	958
14 x 117	1720	998	998

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

7.1.3 Pile Resistance and Pile Quality Control

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

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

7.2 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 abutments and wingwalls at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

A resistance factor of ϕ= 1.0 shall be used to assess spread footing design at the service limit state including: settlement, horizontal movement, overall stability and scour at the design flood. Extreme limit state design checks for spread footings shall include bearing resistance, eccentricity, sliding and overall stability. A resistance factor of ϕ=1.0 shall be used for the extreme limit state.

Conventional wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using as active earth pressure coefficient, K_a, calculated using Rankine Theory for cantilever wingwalls and Coulomb Theory for gravity shaped structures. See *Sheet 4 - Rankine and Coulomb Active Earth Pressure Coefficients* at the end of this report for guidance in calculating these values. 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 wingwalls if an approach slab is not specified. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken form the table below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic = 0 feet	Distance from wall backface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
15	2.0	2.0

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface. A sliding resistance factor of ϕ_τ=0.8 shall be applied to the nominal sliding resistance of walls found on spread footings on sand.

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 for 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.

At the east end of the bridge, wingwalls will be independent of the stub abutment and will be supported on spread footings. The design of walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood. The wingwalls shall be designed as unrestrained, meaning that they are free to rotate at the top in an active state of earth pressure. The Rankine active earth pressure coefficient of K_a = 0.307 is recommended.

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.

The existing gravity abutments and wingwalls are to remain in place as scour counter measures. The existing structures will be cut 2 feet below the proposed super structure and a riprap bench area will be place in front of the proposed stub abutment. It is likely that the existing gravity abutments to remain will not be stable with the existing superstructure removed and earth pressure still applied. If this is the case, the abutments will be stabilized with rock anchors through the stem walls or tiebacks with concrete deadmen, if necessary.

7.3 Bearing Resistance

It is anticipated that the project independent return wingwalls will be founded on the native soils at the site while the retaining walls will be founded on bedrock. These elements will need to be designed to provide stability against bearing capacity failure. Applicable permanent and transient loads are specified in LFRD Articles 3.4.1 and 11.5.5. The soil distribution may be assumed to be uniformly distributed over the effective base as shown in LRFD Figure 11.6.3.2-1.

Bearing resistance for any structure founded on the native soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf. The bearing resistance factor, $\phi_{b,}$ for spread footings on soil is 0.45 based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 4 ksf may be used when analyzing the service limit state and for preliminary sizing of footings assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

Bearing resistance for any structure founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 160 ksf. The bearing resistance factor, $\phi_{b,}$ for spread footings on bedrock is 0.45. A factored bearing resistance of 30 ksf may be used when analyzing the service limit state and for preliminary sizing of footings assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

The bearing resistance for spread footings shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, footings shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0.

In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material. Any organic material encountered shall be removed to the full depth and replaced with compacted Granular Borrow, MaineDOT 703.19.

7.4 Scour and Riprap

If using integral abutment sat the site, pile lengths will be short and, therefore, scour protection will be critical, especially if the dam downstream were removed. For scour protection, the integral abutments should be moved away from the channel. Since the proposed bridge design will rely on the existing abutments and wingwalls to provide lateral support and scour protection for the integral abutment piles, it is critical that concrete spalling and deterioration of the existing substructures, especially below the water line, be repaired. The interface contact of the abutment toes with the bedrock bearing stratum should also be examined and improved, if necessary, with grouting.

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. These changes in foundation conditions shall be investigated at the abutments, wingwalls and retaining walls. For scour protection, any footings for wingwalls or retaining walls , 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 item number 703.26 of the Standard Specification shall be placed at the toes of abutments, wingwalls and retaining walls. Riprap shall be 3 feet thick. In front of the wingwalls and retaining wall, the bottom of the riprap section shall be constructed 6.5 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.

7.5 Settlement

The west bridge approach will be widened in order to accommodate the change in horizontal alignment of the proposed bridge which will shift the roadway slightly upstream (north). Additionally, the vertical alignment of the bridge will be raised approximately 2.5 feet at the west end and 4 feet at the east end. The maximum additional fill to be placed at the site is 4 feet and will result in approximately 1/2 to 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. Settlement of any spread footing founded directly on the native granular soils and sized for the service limit state is anticipated to be less than 1 inch.

7.6 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 1900 F-degree days. This correlates to a frost depth of 6.5 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 6.5 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils and not those founded on bedrock. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG. See Appendix D- Calculations at the end of this report for supporting documentation.

7.7 Retaining Wall

The PDR prepared for MaineDOT by T.Y. Lin International of Falmouth, Maine in June 2006 for the project considers several alternatives for retaining the approach fills at the northwest corner of the bridge. Those alternatives are:

- Conventional cast-in-place concrete cantilever wall
- Precast Concrete Modular Gravity (PCMG) Wall
- Metal bin wall
- Mechanically stabilized earth (MSE) wall
- Prefabricated concrete block gravity wall (i.e. Redi-Rock)
- Traditional riprap slope (eliminated due to associated environmental impacts)

After consideration of all the alternatives, the use of the PCMG Wall was chosen. The PCMG wall will retain approach fills, provide lateral support to the northwest corner of the pile group and minimize slope impacts at the northwest corner of the bridge near the existing stacked granite retaining wall. This wall shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The PCMG Wall shall be founded on bedrock. The wall will be approximately 19 feet high and 45 feet long. A moment distribution slab with concrete parapet and 2-bar steel bridge rail will be used on top of this wall. The PCMG Wall shall be designed considering a traffic surcharge equal to 2 feet of fill placed on the backfill surface. The PCMG Wall will be backfilled with flowable fill to improve performance and increase durability. Temporary sheeting will be required to maintain the adjacent driveway during construction of the PCMG Wall.

Bearing resistance for the PCMG wall founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 30 ksf. The bearing resistance factor, ϕ_b, for spread footings on rock is 0.45. A factored bearing resistance of 160 ksf may be used when analyzing the service limit state assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

The bearing resistance for PCMG wall footings shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, PCMG wall footings shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0.

The PCMG wall shall consist of Class “LP” concrete and epoxy coated rebar. The precast concrete units shall contain a minimum of 5.5 gallons per cubic yard of calcium nitrate solution or equivalent corrosion inhibitor. Any irregularities in the existing bedrock surface or irregularities created during the excavation process will be backfilled with un-reinforced Class S fill concrete to the bearing elevation.

The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635 - Prefabricated Bin Type Retaining Wall (Prefabricated Concrete Modular Gravity Wall). The Special Provision reads: Hydrostatic forces - Unless specified otherwise, when a design high water surface is shown on the plans, the design stresses calculated from that elevation to the bottom of wall must include a 3 foot minimum differential head of flow able fill or saturated backfill. In addition, the buoyant weight of flowable fill mass or saturated soil shall be used in the calculation of pullout resistance.

7.7 Seismic Design Considerations

The horizontal bedrock acceleration coefficient (A) for New Portland is approximately 0.05g, based on Figure 3-4 of the BDG, Seismic Performance Categories for Maine, August 2003. Per LRFD Articles 3.10.4 and 3.10.5, the site is assigned to Seismic Zone 1 and Soil Profile Type I, and a site coefficient (S) of 1.0 should be used. 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 bridge seat dimensions shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

Per BDG Section 3.7.1.1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09g are designated to Seismic Performance Category (SPC) classification A. For SPC A, no detailed analysis is required other than connection design and bearing seat length, except if the bridge is functionally important or is classified as a major structure. According to Figure 2-2 of the BDG, Bartlett Bridge is not on the National Highway System (NHS) and is therefore not considered to be functionally important, and since the bridge construction costs should not exceed $10 million the bridge is not classified as a major structure.

7.8 Construction Considerations

If using integral abutment sat the site, pile lengths will be short and, therefore, scour protection will be critical, especially if the dam downstream were removed. For scour protection, the integral abutments should be moved away from the channel. Since the proposed bridge design will rely on the existing abutments and wingwalls to provide lateral support and scour protection for the integral abutment piles, it is critical that concrete spalling and deterioration of the existing substructures, especially below the water line, be repaired. The interface contact of the abutment toes with the bedrock bearing stratum should also be examined and improved, if necessary, with grouting.

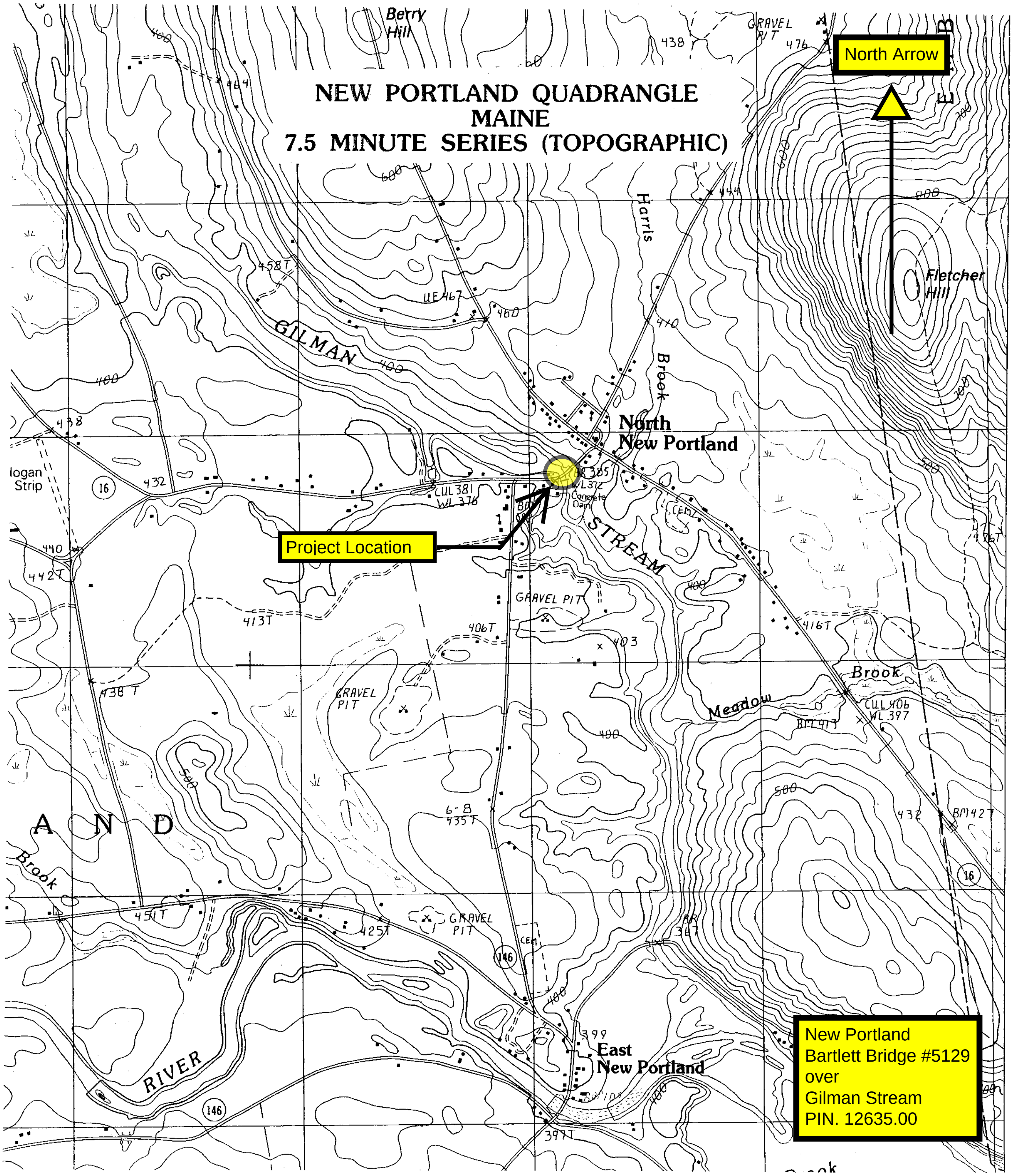
For construction of the PCMG wall construction of cofferdams and earth support systems may be required. It is recommended that the PCMG wall be constructed in dry conditions. Excavation of bedrock materials for placement of the leveling slab may require drilling and blasting techniques. Blasting should be done in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. The Contractor may need to conduct pre-and post-blast surveys in accordance with industry standards. All loose and fractured rock and soil debris should be removed from bearing surfaces before concrete is placed. It is likely that there will be seepage of water from fractures and joints exposed in the bedrock surface and cut slopes. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry. It is recommended that a person qualified by training and experience be present to inspect the condition of the bedrock bearing surfaces prior to pouring of the concrete.

There is a potential for the remaining portion of the existing abutments to interfere with the installation of the integral abutment piles. If the existing abutments are encountered during pile installation the Contractor shall drill and clean a stable hole of the required diameter and length to provide minimum 6 foot long piles at the west abutment and 20 foot long piles at the east abutment. The drilling method selected by the Contractor should be able to drill a stable hole without detriment to the existing abutments. This condition should be noted on the plans and the work will be considered incidental to pile installation.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Bartlett Bridge in New Portland, 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

Sheets



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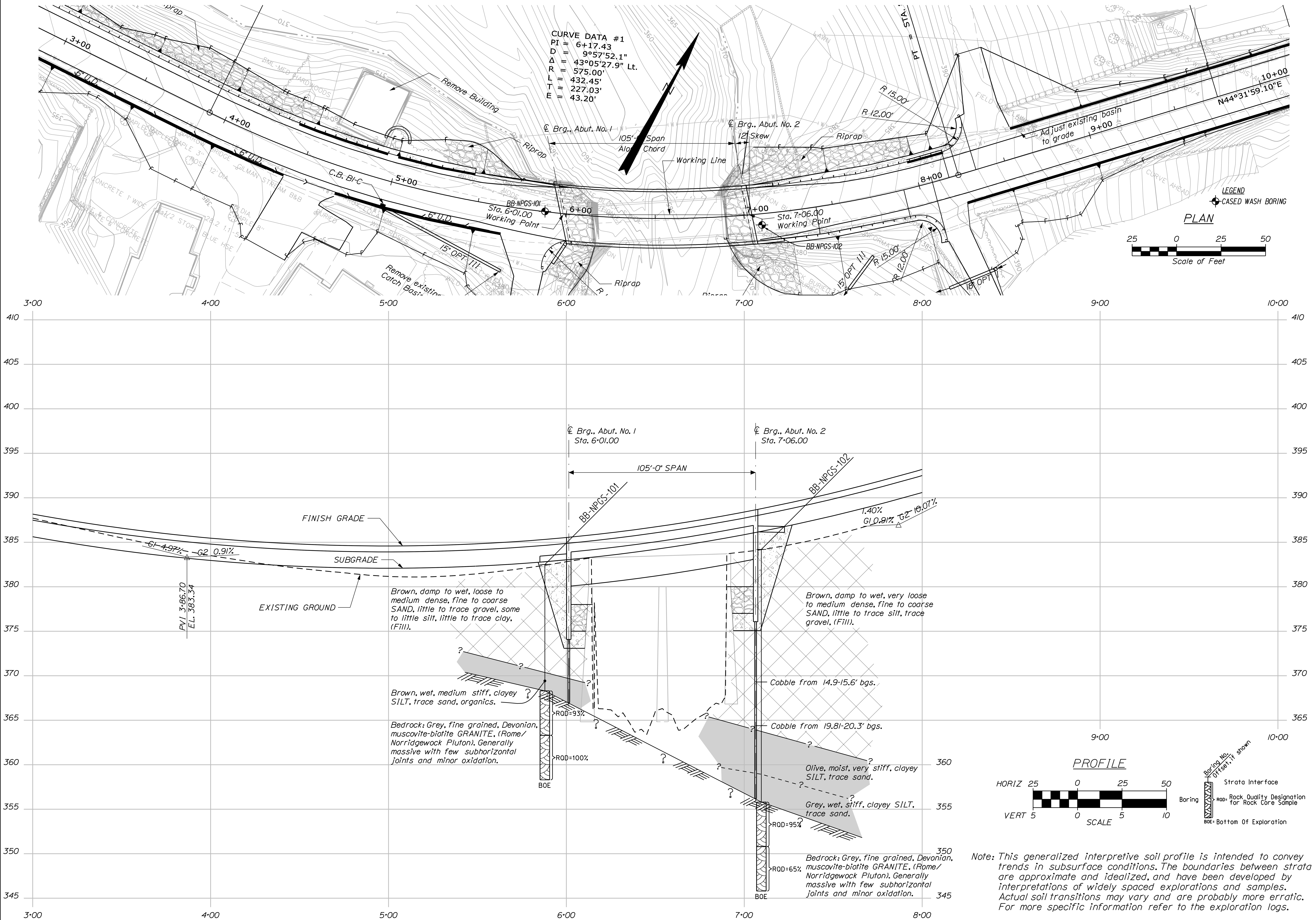
METERS 1000 0 1000 2000

MILES 1 .5 0 1

FEET 1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10 000

CONTOUR INTERVAL 10 FEET



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

12635.00

BRIDGE PLANS

PIN 12635.00

BRIDGE NO. 5129

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

SIGNATURE

P.E. NUMBER

DATE

BARTLETT BRIDGE
GILMAN STREAM
NEW PORTLAND SOMERSET COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 4

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date: 3/19/2008

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:3/19/2008

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Bartlett Bridge #5129 over Gilman Stream
Location: Route 16 in New Portland, Maine

Boring No.: BB-NPGS-101
PIN: 12635.00

Driller:	MaineDOT	Elevation (ft.)	382.5	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"
Date Start/Finish:	10/16/07; 13:45-16:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+87.9, 0.16' Lt.	Casing ID/OD:	NW	Water Level*:	See Remarks
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0	1D/AB	24/16	0.7 - 2.7	8/9/7/8	16	21	SSA
5	2D	24/18	5.0 - 7.0	2/2/3/2	5	6	5
							8
							7
							3
							3
10	3D/AB	24/7	10.0 - 12.0	5/3/2/2	5	6	15
							11
							11
							42
	R1	60/56	14.2 - 19.2	RQD = 93%			a25 NQ
15							
	R2	60/60	19.2 - 24.2	RQD = 100%			
20							
25							

Elevation (ft.)	Visual Description and Remarks	Depth	Laboratory Testing Results/ AASHTO and Unified Class
	Pavement	0.7	
381.80	(1D/A) 0.7-1.3' bgs. Brown, damp, medium dense, silty fine to coarse SAND, little gravel, (Fill). (1D/B) 1.3-2.7' bgs. Brown, damp, medium dense, fine to medium SAND, trace gravel, trace coarse sand, trace silt, (Fill).		
	Brown, damp, loose, fine to coarse SAND, little silt, trace gravel, (Fill).		G#209980 A-2-4, SP-SM WC=5.6%
	Brown, wet, loose, fine to coarse SAND, little gravel, (Fill).		
	(3D/A) 10.0-10.5' bgs. Brown, wet, loose, fine to coarse SAND, little gravel, (Fill). (3D/B) 10.5-12.0' bgs. Brown, wet, loose, SAND, some silt, little clay, trace gravel.	12.0	G#209981 A-4, SC-SM WC=49.9%
370.50	Brown, wet, medium stiff, clayey SILT, trace sand, organics.		
368.30	a25 blows for 0.1'. Casing to 14.1' bgs. Roller Coned ahead from 14.1-14.2' bgs.	14.2	
	Bedrock: Grey, fine-grained, Devonian muscovite-biotite GRANITE (Rome/Norridgewock pluton). Generally massive with few sub-horizontal joints and minor oxidation. R1:Core Times (min:sec) 14.2-15.2' (2:35) 15.2-16.2' (2:36) 16.2-17.2' (2:37) 17.2-18.2' (2:39) 18.2-19.2' (2:54) 93% Recovery R2:Core Times (min:sec) 19.2-20.2' (3:25) 20.2-21.2' (3:37) 21.2-22.2' (3:50) 22.2-23.2' (3:58) 23.2-24.2' (3:57) 100% Recovery Solid intact 5' core		
358.30	Bottom of Exploration at 24.20 feet below ground surface.	24.2	

Remarks:

9.9' bgs after R2, casing at 14.1' bgs, bottom of hole at 24.2' bgs.

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-NPGS-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Bartlett Bridge #5129 over Gilman Stream
Location: Route 16 in New Portland, Maine

Boring No.: BB-NPGS-102
PIN: 12635.00

Driller:	MaineDOT	Elevation (ft.)	384.2	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"
Date Start/Finish:	10/16/07; 08:00-13:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+09.6, 6.3' Rt.	Casing ID/OD:	NW	Water Level*:	See Remarks
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0	1D/AB	24/15	0.8 - 2.8	4/7/5/5	12	15	SSA
5	2D	24/14	5.0 - 7.0	1/1/2/2	3	4	6
							9
							8
							11
							12
10	3D	24/5	10.0 - 12.0	2/2/1/1	3	4	3
							4
							7
							21
							38
15							67
							32
							43
							107
							110
20	4D	24/8	20.6 - 22.6	8/7/7/9	14	18	46
							63
							87
							92
							84
25	5D V1	13.2/7	25.0 - 26.1 25.0 - 25.4	9/28/25(1.2") 1482/223 psf	---		68
							172
							58
	R1	60/59	28.4 - 33.4	RQD = 95%			37 NQ
30							
	R2	60/49	33.4 - 38.4	RQD = 65%			
35							
40							
45							
50							

Elevation (ft.)	Visual Description and Remarks	Depth	Laboratory Testing Results/ AASHTO and Unified Class
	Pavement	0.8	
383.40	(1D/A) 0.8-1.8' bgs. Brown, damp, medium dense, fine to medium SAND, little gravel, little silt, trace coarse sand, (Fill). (1D/B) 1.8-2.8' bgs. Brown, damp, medium dense, fine to medium SAND, little gravel, trace coarse sand, trace silt, (Fill).		
	Brown, damp, very loose, fine to coarse SAND, little silt, trace gravel, (Fill).		G#209982 A-2-4, SP-SM WC=5.4%
	Brown, wet, very loose, fine to coarse SAND, trace silt, trace gravel, (Fill).		G#209983 A-2-4, SP-SM WC=18.3%
	Cobble from 14.9-15.6' bgs. Roller Coned ahead from 15.0-15.8' bgs.		
	Cobble from 19.8-20.3' bgs. Roller Coned ahead from 20.0-20.6' bgs.	20.3	
363.90	Olive, moist, very stiff, clayey SILT, trace sand.	25.0	G#209984 A-4, CL-ML LL=28 PL=22 PI=6 WC=23.5%
359.20	Grey, wet, stiff, clayey SILT, trace sand. 55x110 mm vane raw torque readings: 33.2/5.0 ft-lbs V1 only penetrated 0.4' instead of required 1', but sheared it anyway. Roller Coned ahead from 27.0-28.2' bgs.	28.0	G#209985 A-4, CL-ML WC=28.3%
356.20	Weathered Bedrock.		
355.80	Roller Coned ahead from 28.2-28.4' bgs.	28.4	
	Bedrock: Grey, fine-grained, Devonian muscovite-biotite GRANITE (Rome/Norridgewock pluton). Generally massive with few sub-horizontal joints and minor oxidation. R1:Core Times (min:sec) 28.4-29.4' (2:48) 29.4-30.4' (2:31) 30.4-31.4' (2:25) 31.4-32.4' (2:17) 32.4-33.4' (2:32) 98% Recovery R2:Core Times (min:sec) 33.4-34.4' (1:38) 34.4-35.4' (2:10) 35.4-36.4' (1:43) 36.4-37.4' (2:32) 37.4-38.4' (2:38) 82% Recovery Seam from 34.0-34.3'		
345.80	Bottom of Exploration at 38.40 feet below ground surface.	38.4	

Remarks:

11.8' bgs after R2, casing at 28.2' bgs, bottom of hole at 38.4' bgs.

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-NPGS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

12635.00

PIN 12635.00

BRIDGE PLANS

BRIDGE NO. 5129

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

SIGNATURE

P.E. NUMBER

DATE

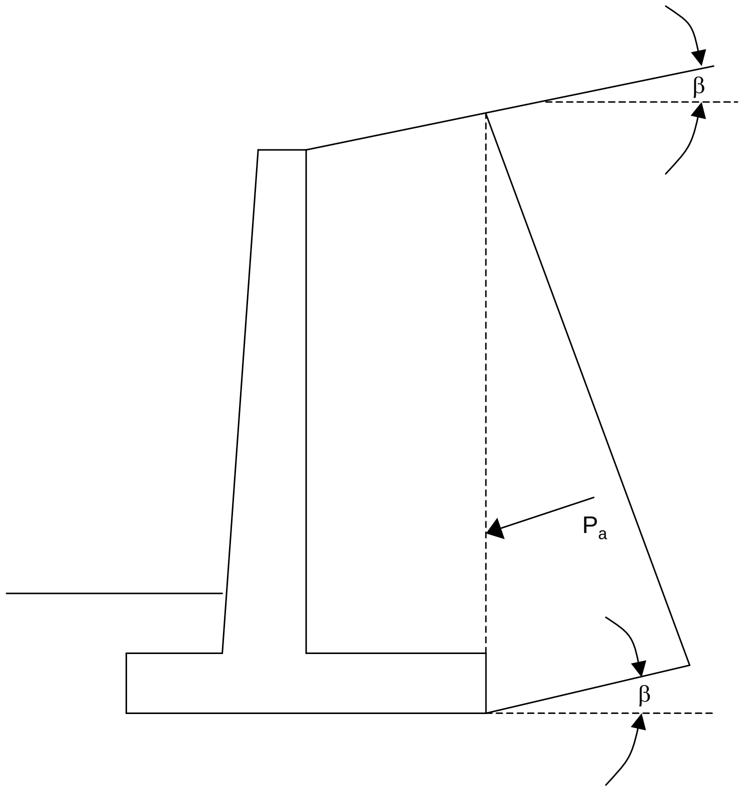
BARTLETT BRIDGE
GILMAN STREAM
NEW PORTLAND SOMERSET COUNTY

BORING LOGS

SHEET NUMBER

3

OF 4



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

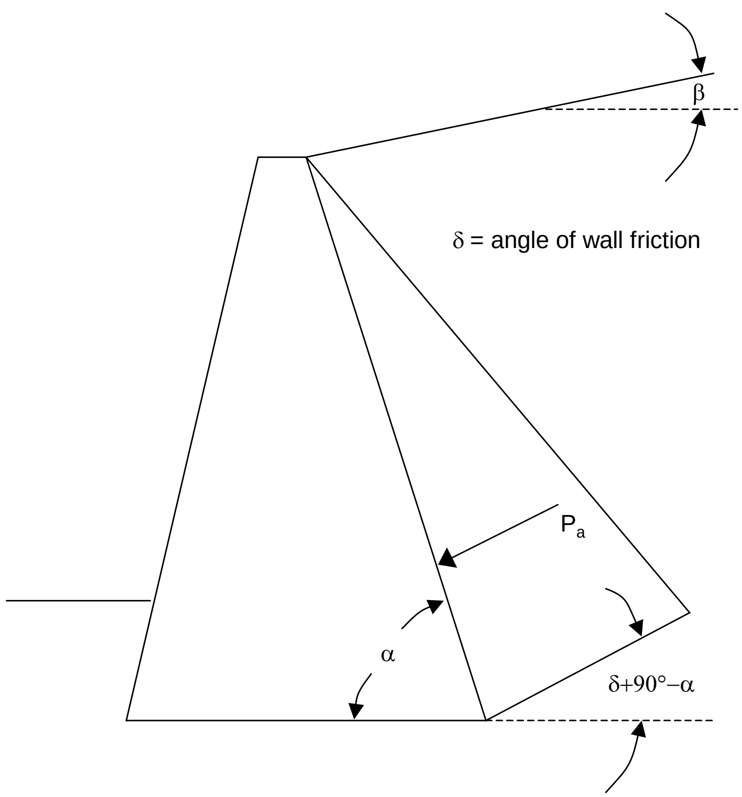
For a horizontal backfill surface, $\beta = 0°$:

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

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

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

P_a is oriented at β



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

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

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

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Bartlett Bridge #5129 over Gilman Stream	**Boring No.:**	BB-NPGS-101
Location:	Route 16 in New Portland, Maine	**PIN:**	12635.00

Driller:	MaineDOT	**Elevation (ft.)**	382.5	**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"
Date Start/Finish:	10/16/07; 13:45-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+87.9, 0.16' Lt.	**Casing ID/OD:**	NW	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D/AB	24/16	0.7 - 2.7	8/9/7/8	16	21	SSA	381.80		Pavement 0.7	
										(1D/A) 0.7-1.3' bgs. Brown, damp, medium dense, silty fine to coarse SAND, little gravel, (Fill).	
										(1D/B) 1.3-2.7' bgs. Brown, damp, medium dense, fine to medium SAND, trace gravel, trace coarse sand, trace silt, (Fill).	
5	2D	24/18	5.0 - 7.0	2/2/3/2	5	6	5			Brown, damp, loose, fine to coarse SAND, little silt, trace gravel, (Fill).	G#209980 A-2-4, SP-SM WC=5.6%
							8				
							7				
							3			Brown, wet, loose, fine to coarse SAND, little gravel, (Fill).	
							3				
10	3D/AB	24/7	10.0 - 12.0	5/3/2/2	5	6	15			(3D/A) 10.0-10.5' bgs. Brown, wet, loose, fine to coarse SAND, little gravel, (Fill).	G#209981 A-4, SC-SM WC=49.9%
							11			(3D/B) 10.5-12.0' bgs. Brown, wet, loose, SAND, some silt, little clay, trace gravel.	
							11	370.50		12.0 Brown, wet, medium stiff, clayey SILT, trace sand, organics.	
							42				
	R1	60/56	14.2 - 19.2	RQD = 93%			a25 NQ	368.30		a25 blows for 0.1'. Casing to 14.1' bgs. Roller Coned ahead from 14.1-14.2' bgs. 14.2	
15										Bedrock: Grey, fine-grained, Devonian muscovite-biotite GRANITE (Rome/Norridgewock pluton). Generally massive with few sub-horizontal joints and minor oxidation. R1:Core Times (min:sec) 14.2-15.2' (2:35) 15.2-16.2' (2:36) 16.2-17.2' (2:37) 17.2-18.2' (2:39) 18.2-19.2' (2:54) 93% Recovery	
	R2	60/60	19.2 - 24.2	RQD = 100%						R2:Core Times (min:sec) 19.2-20.2' (3:25) 20.2-21.2' (3:37) 21.2-22.2' (3:50) 22.2-23.2' (3:58) 23.2-24.2' (3:57) 100% Recovery Solid intact 5' core	
20											
								358.30		24.2 **Bottom of Exploration at 24.20 feet below ground surface.**	
25											

Remarks:

9.9' bgs after R2, casing at 14.1' bgs, bottom of hole at 24.2' bgs.

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: Bartlett Bridge #5129 over Gilman Stream
Location: Route 16 in New Portland, Maine

Boring No.: BB-NPGS-102
PIN: 12635.00

Driller:	MaineDOT	**Elevation (ft.)**	384.2	**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"
Date Start/Finish:	10/16/07; 08:00-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+09.6, 6.3' Rt.	**Casing ID/OD:**	NW	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D/AB	24/15	0.8 - 2.8	4/7/5/5	12	15	SSA	383.40		Pavement 0.8 (1D/A) 0.8-1.8' bgs. Brown, damp, medium dense, fine to medium SAND, little gravel, little silt, trace coarse sand, (Fill). (1D/B) 1.8-2.8' bgs. Brown, damp, medium dense, fine to medium SAND, little gravel, trace coarse sand, trace silt, (Fill).	
5	2D	24/14	5.0 - 7.0	1/1/2/2	3	4	6			Brown, damp, very loose, fine to coarse SAND, little silt, trace gravel, (Fill).	G#209982 A-2-4, SP-SM WC=5.4%
							9				
							8				
							11				
							12				
10	3D	24/5	10.0 - 12.0	2/2/1/1	3	4	3			Brown, wet, very loose, fine to coarse SAND, trace silt, trace gravel, (Fill).	G#209983 A-2-4, SP-SM WC=18.3%
							4				
							7				
							21				
							38				
15							67			Cobble from 14.9-15.6' bgs. Roller Coned ahead from 15.0-15.8' bgs.	
							32				
							43				
							107				
							110			Cobble from 19.8-20.3' bgs.	
20	4D	24/8	20.6 - 22.6	8/7/7/9	14	18	46	363.90		Roller Coned ahead from 20.0-20.6' bgs. 20.3 Olive, moist, very stiff, clayey SILT, trace sand.	G#209984 A-4, CL-ML LL=28 PL=22 PI=6 WC=23.5%
							63				
							87				
							92				
25							84				

Remarks:

11.8' bgs after R2, casing at 28.2' bgs, bottom of hole at 38.4' bgs.

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:	Bartlett Bridge #5129 over Gilman Stream	**Boring No.:**	BB-NPGS-102		
Location:	Route 16 in New Portland, Maine	**PIN:**	12635.00		
Driller:	MaineDOT	**Elevation (ft.)**	384.2	**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"
Date Start/Finish:	10/16/07; 08:00-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+09.6, 6.3' Rt.	**Casing ID/OD:**	NW	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D ~~V1~~	13.2/7	25.0 - 26.1 ~~25.0 - 25.4~~	9/28/25(1.2") ~~1482/223 psf~~	---		68	359.20		25.0 Grey, wet, stiff, clayey SILT, trace sand. 55x110 mm vane raw torque readings: 33.2/5.0 ft-lbs V1 only penetrated 0.4' instead of required 1', but sheared it anyway. Roller Coned ahead from 27.0-28.2' bgs.	G#209985 A-4, CL-ML WC=28.3%
							172				
							58				
	R1	60/59	28.4 - 33.4	RQD = 95%			37 NQ	356.20 355.80		28.0 Weathered Bedrock. Roller Coned ahead from 28.2-28.4' bgs. 28.4 Bedrock: Grey, fine-grained, Devonian muscovite-biotite GRANITE (Rome/Norridgewock pluton). Generally massive with few sub-horizontal joints and minor oxidation. R1:Core Times (min:sec) 28.4-29.4' (2:48) 29.4-30.4' (2:31) 30.4-31.4' (2:25) 31.4-32.4' (2:17) 32.4-33.4' (2:32) 98% Recovery	
30											
	R2	60/49	33.4 - 38.4	RQD = 65%						R2:Core Times (min:sec) 33.4-34.4' (1:38) 34.4-35.4' (2:10) 35.4-36.4' (1:43) 36.4-37.4' (2:32) 37.4-38.4' (2:38) 82% Recovery Seam from 34.0-34.3'	
35											
								345.80		38.4 **Bottom of Exploration at 38.40 feet below ground surface.**	
40											
45											
50											

Remarks:

11.8' bgs after R2, casing at 28.2' bgs, bottom of hole at 38.4' bgs.

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

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

<u>Appendix B</u>

Laboratory Data

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.

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): New Portland Project Number: 12635.00

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-NPGS-101, 2D	5+87.9	0.16Lt.	5.0-7.0	209980	1	5.6			SP-SM	A-2-4	0
BB-NPGS-101, 3D/B	5+87.9	0.16 Lt.	10.5-12.0	209981	1	49.9			SC-SM	A-4	III
BB-NPGS-102, 2D	7+09.6	6.3 Rt.	5.0-7.0	209982	1	5.4			SP-SM	A-2-4	0
BB-NPGS-102, 3D	7+09.6	6.3 Rt.	10.0-12.0	209983	1	18.3			SP-SM	A-2-4	0
BB-NPGS-102, 4D	7+09.6	6.3 Rt.	20.6-22.6	209984	1	23.5	28	6	CL-ML	A-4	IV
BB-NPGS-102, 5D	7+09.6	6.3 Rt.	25.0-26.1	209985	1	28.3			CL-ML	A-4	IV

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

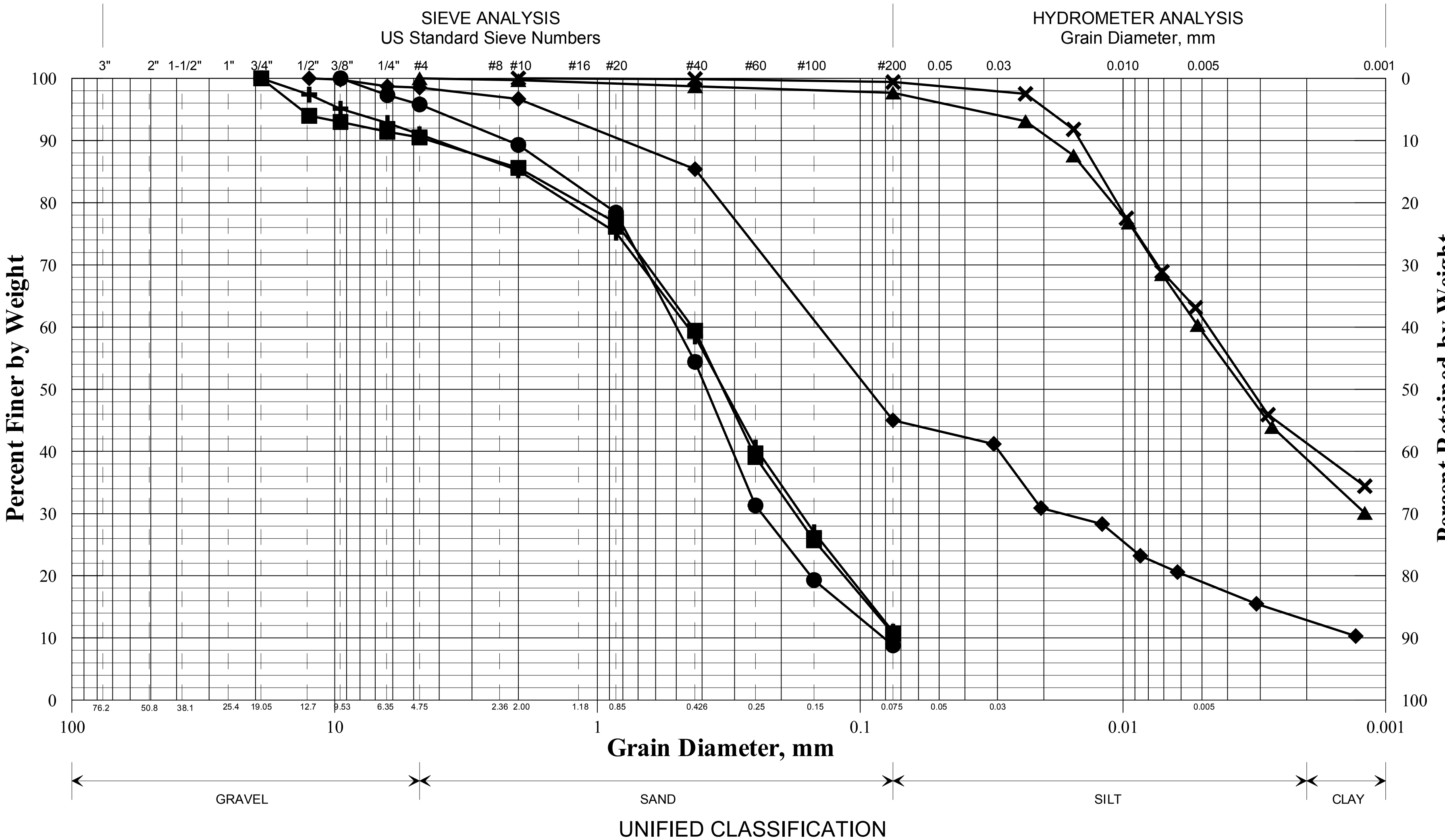
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

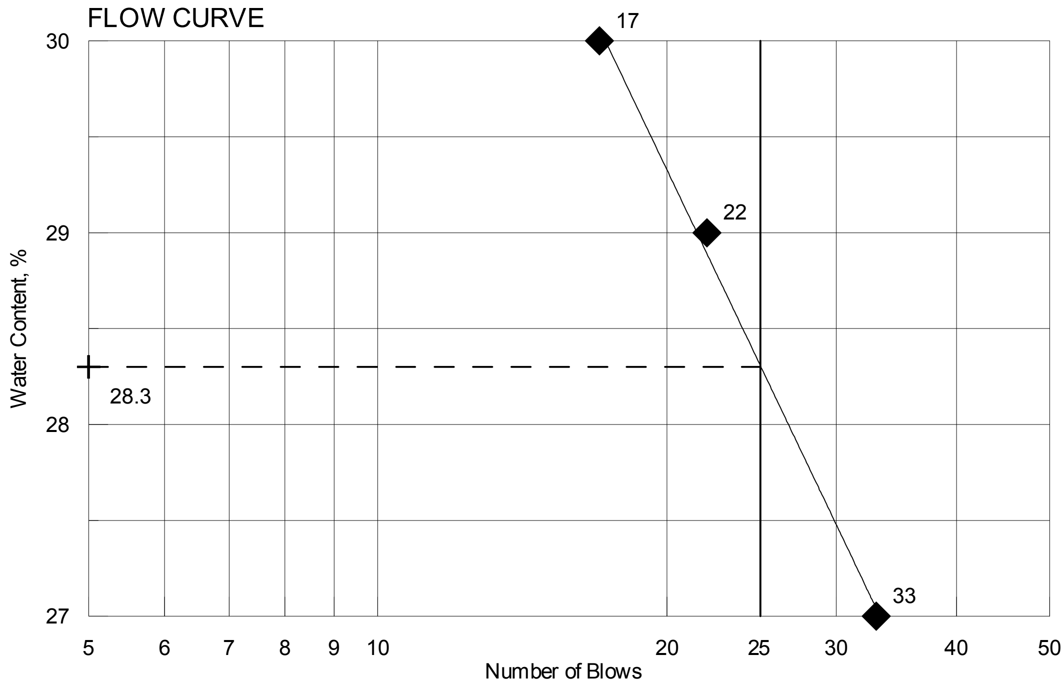


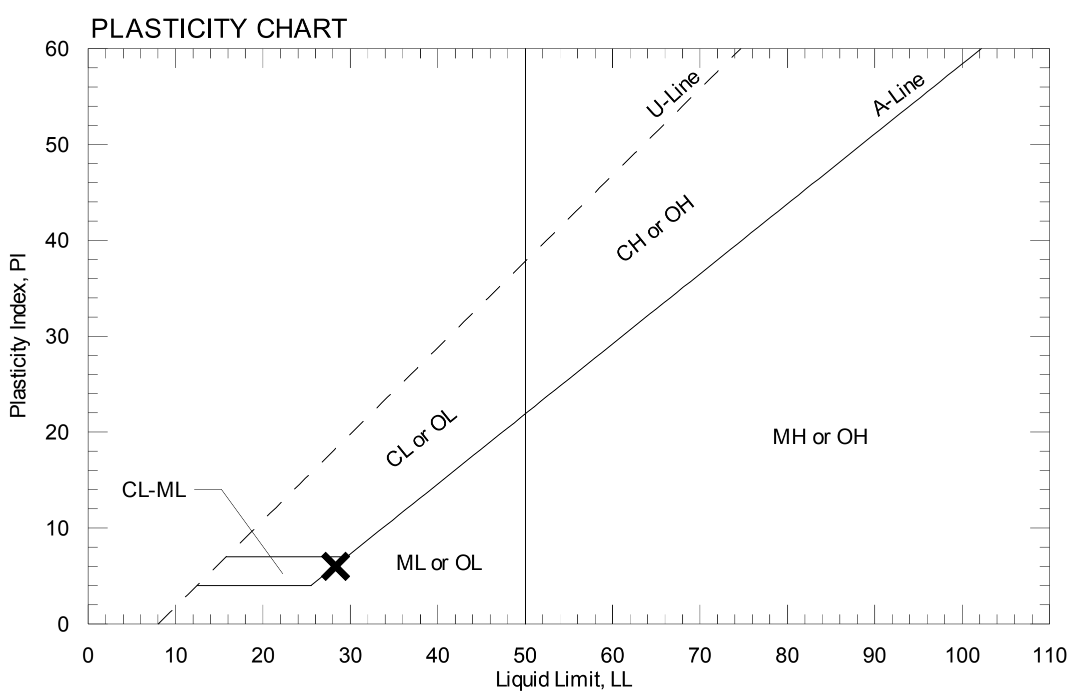
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NPGS-101/2D	5+87.9	0.16 LT	5.0-7.0	SAND, little silt, trace gravel.	5.6			
◆	BB-NPGS-101/3D/B	5+87.9	0.16 LT	10.5-12.0	SAND, some silt, little clay, trace gravel.	49.9			
■	BB-NPGS-102/2D	7+09.6	6.3 RT	5.0-7.0	SAND, little silt, trace gravel.	5.4			
●	BB-NPGS-102/3D	7+09.6	6.3 RT	10.0-12.0	SAND, trace silt, trace gravel.	18.3			
▲	BB-NPGS-102/4D	7+09.6	6.3 RT	20.6-22.6	Clayey SILT, trace sand.	23.5	28	22	6
✖	BB-NPGS-102/5D	7+09.6	6.3 RT	25.0-26.1	Clayey SILT, trace sand.	28.3			

PIN
012635.00
Town
New Portland
Reported by/Date
WHITE, TERRY A 12/19/2007

SHEET 1

TOWN	New Portland	Reference No.	209984
PIN	012635.00	Water Content, %	23.5
Sampled	10/16/2007	Plastic Limit	22
Boring No./Sample No.	BB-NPGS-102/4D	Liquid Limit	28
Station	7+09.6	Plasticity Index	6
Depth	20.6-22.6	Tested By	BBURR





A U T H O R I Z A T I O N   A N D   D I S T R I B U T I O N

Reported by: **FOGG, BRIAN** Date Reported: **12/18/2007**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

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:
New Portland, Maine
DFI = 1900 degree-days

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

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1900 frost penetration = 76.6 inches

$\text{Frost\_depth} := 76.6\text{in}$ $\qquad \text{Frost\_depth} = 6.3833\,\text{ft}$ **Use 6.5 feet**

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

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Farmington

--- ModBerg Results ---

Project Location: Farmington, Maine

Air Design Freezing Index = 2023 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1618 F-days
Mean Annual Temperature = 41.2 deg F
Design Length of Freezing Season = 145 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	93.5	15.0	125.0	31	40	2.9	1.8	2,700

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 = 7.79 ft = 93.5 in.

Use BDG Calc Frost Depth = 6.5 feet for design

Definition of Units:

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

LIQUIDITY INDEX (LI):

BB-NPGS-102 Sample 4D:

$wc := 23.5$

$PL := 22$

$LL := 28$

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

$LI := \frac{wc - PL}{LL - PL}$ $LI = 0.25$

Abutment Foundations: Integral driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

PDR Estimate based on HP 14 x 89 pile size

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

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} kip$ **HP 12 x 53** **HP 14 x 73** **HP 14 x 89** **HP 14 x 117**

STRENGTH LIMIT STATE:

Factored Resistance:

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance:

eq. 6.9.2.1-1 $P_f := \phi_c \cdot P_n$ $P_f = \begin{pmatrix} 465 \\ 642 \\ 783 \\ 1032 \end{pmatrix} kip$ **HP 12 x 53** **HP 14 x 73** **HP 14 x 89** **HP 14 x 117** Strength Limit State

SERVICE/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

λ=(Kl/$r_s\pi$)2*F_y/E eq. 6.9.4.1-3

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

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

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

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

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

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

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

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

Service/Extreme Limit States

Geotechnical Resistance

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

Bedrock Type: Devonian muscovite-biotite granite. Igneous intrusion identified as the Rome/Norridgewock pluton.
RQD ranges from 65 to 100%. Use RQD = 90% and ϕ = 34 to 40 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

PDR Estimate based on HP 14 x 89 pile size

Look at these piles:

HP 12 x 53
HP 14 x 73 Note: All matrices set up in this order
HP 14 x 89
HP 14 x 117

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

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

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

Calculate pile box area:

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

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

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

q_u for granite compressive strength
ranges for 2100 to 49000 psi

use $\sigma_c := 15000 \cdot psi$

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2
Spacing of discontinuities: $c := 12 \cdot in$ Assumed based on rock core

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

Footing width, b:

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

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

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

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

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 647 \\ 619 \\ 618 \\ 616 \end{pmatrix} ksf$$

Nominal Geotechnical Tip Resistance, R_p:

$$R_p := \overrightarrow{\left(q_a \cdot A_{box}\right)} \qquad R_p = \begin{pmatrix} 637 \\ 853 \\ 872 \\ 905 \end{pmatrix} kip$$

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

Factored Geotechnical Tip Resistance, R_f at Strength Limit State:

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

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

$$R_{tipf} := \phi_{stat} \cdot R_p \qquad R_{tipf} = \begin{pmatrix} 287 \\ 384 \\ 392 \\ 407 \end{pmatrix} kip$$

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

Strength Limit State

Factored Axial Geotechnical Skin Resistance of Single H-pile

The piles will be primarily end bearing. Skin friction in the overlying soils is computed using FHWA Program Driven 1.0

Driven software uses Nordlund/Thurman Method for side frictional resistance in cohesionless soil and User defined shear strength in cohesive soils.

Use a Resistance Factor of ϕ = 0.45 per LRFD Table 10.5.5.2.3-1

$$\phi_{stat} := 0.45$$

From Driven: Skin friction for Abutment 1

$$R_{skin.A1} := \begin{pmatrix} 20.12 \\ 25.32 \\ 26.72 \\ 28.86 \end{pmatrix} \cdot kip$$

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

Determine factored skin resistance, R_{sf}

$$R_{sf.A1} := R_{skin.A1} \cdot \phi_{stat} \qquad R_{sf.A1} = \begin{pmatrix} 9 \\ 11 \\ 12 \\ 13 \end{pmatrix} kip$$

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

Strength Limit State

From Driven: Skin friction for Abutment 2

$$R_{skin.A2} := \begin{pmatrix} 64.46 \\ 81.46 \\ 86.17 \\ 93.33 \end{pmatrix} \cdot kip$$

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

Determine factored skin resistance, R_{sf}

$$R_{sf.A2} := R_{skin.A2} \cdot \phi_{stat} \qquad R_{sf.A2} = \begin{pmatrix} 29 \\ 37 \\ 39 \\ 42 \end{pmatrix} kip$$

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

Strength Limit State

STRENGTH LIMIT STATE:

Total Factored Geotechnical Resistance, R_{gf}:

Abutment 1: $R_{gf.A1} := R_{tipf} + R_{sf.A1}$ $\quad R_{gf.A1} = \begin{pmatrix} 296 \\ 395 \\ 404 \\ 420 \end{pmatrix} kip$

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

Strength Limit State

Abutment 2: $R_{gf.A2} := R_{tipf} + R_{sf.A2}$ $\quad R_{gf.A2} = \begin{pmatrix} 316 \\ 420 \\ 431 \\ 449 \end{pmatrix} kip$

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

Strength Limit State

The factored axial geotechnical resistance is less than the structural resistance of the pile, therefore the geotechnical resistance governs.

SERVICE/EXTREME LIMIT STATES:

Nominal Geotechnical Tip Resistance, R_p:

$$R_p := \overrightarrow{(q_a \cdot A_{box})} \qquad R_p = \begin{pmatrix} 637 \\ 853 \\ 872 \\ 905 \end{pmatrix} kip$$

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

From Driven: Skin friction for Abutment 1

$$R_{skin.A1} := \begin{pmatrix} 20.12 \\ 25.32 \\ 26.72 \\ 28.86 \end{pmatrix} \cdot kip$$

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

From Driven: Skin friction for Abutment 2

$$R_{skin.A2} := \begin{pmatrix} 64.46 \\ 81.46 \\ 86.17 \\ 93.33 \end{pmatrix} \cdot kip$$

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 amd 10.5.8.3

$$\phi := 1.0$$

Total Factored Geotechnical Resistance, R_g:

Abutment 1: $R_{g.A1} := R_p + R_{skin.A1}$ $\quad R_{g.A1} = \begin{pmatrix} 657 \\ 878 \\ 898 \\ 934 \end{pmatrix} kip$

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

Service/Extreme
Limit States

Abutment 2: $R_{g.A2} := R_p + R_{skin.A2}$ $R_{g.A2} = \begin{pmatrix} 702 \\ 934 \\ 958 \\ 998 \end{pmatrix} \text{kip}$

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

Service/Extreme Limit States

Determine Geotechnical Resistance by past methods for comparison:

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

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

$\phi_2 := 36 \cdot \text{deg}$ $N_\phi := \tan\left(45 \cdot \text{deg} + \frac{\phi_2}{2}\right)^2$ $N_\phi = 3.8518$

q_{uc} for granite compressive strength ranges for 2100 to 49000 psi

use $q_{uc} := 15000 \cdot \text{psi}$

$q_b := \left(2 \cdot N_\phi\right) \cdot \frac{q_{uc}}{5}$ $q_b = 23.111 \text{ ksi}$

Steel area:

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

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

$Q_{nom} := q_b \cdot A_s$ $Q_{nom} = \begin{pmatrix} 358 \\ 495 \\ 603 \\ 795 \end{pmatrix} \text{kip}$

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

Factored Geotechnical Resistance:

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

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

$Q_{fac} = \begin{pmatrix} 161 \\ 223 \\ 271 \\ 358 \end{pmatrix} \text{kip}$

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

use CFM values above

DRIVABILITY ANALYSIS

Ref: LFRD 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)

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

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

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

Compute Resistance that must be achieved in a drivability analysis:

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

STRENGTH LIMIT STATE:

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

$\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium site variability. There will probably only be 8 to 10 piles total on the project. Only 1 or 2 piles will be tested - one per abutment will be requested. Therefore, reduce the ϕ by 20%

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

At Abutment 1:

$$Q_{drivability.A1} := \frac{R_{gf.A1}}{\phi_{dyn.reduced}} \qquad Q_{drivability.A1} = \begin{pmatrix} 569 \\ 760 \\ 778 \\ 808 \end{pmatrix} kip$$

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

Strength Limit State

At Abutment 2:

$$Q_{drivability.A2} := \frac{R_{gf.A2}}{\phi_{dyn.reduced}} \qquad Q_{drivability.A2} = \begin{pmatrix} 607 \\ 809 \\ 829 \\ 864 \end{pmatrix} kip$$

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

Strength Limit State

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: $\phi := 32 \cdot deg$

$$K_{p\_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}} \qquad K_{p\_rank} = 3.25$$

Bowles does not recommend the use of the Rankine Method for K_p when $\beta > 0$.

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

For a horizontal backfill surface:

$\phi := 32 \cdot deg$

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

Bearing Resistance - Fill Soils:

Nominal and factored Bearing Resistance - spread footing on fill soils

Presumptive Bearing Resistance for Service Limit State ONLY

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

Type of Bearing Material:	Consistency In Place:	Bearing Resistance Ordinary Range (ksf)	Recommended Value of Use (ksf)
Coarse to medium sand, with little gravel (SW, SP)	Very Dense	8 to 12	8
	Medium dense to dense	4 to 8	6
	Loose	2 to 6	3

Based on corrected N-values ranging from 4 to 21 - Soils are loose to medium dense

Recommended Value of Use: $4 \cdot ksf = 2\,tsf$

Therefore: $q_{nom} := 2 \cdot tsf$

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

$q_{factored\_bc} := 2 \cdot tsf$ or $q_{factored\_bc} = 4\,ksf$

Nominal and factored Bearing Resistance - spread footing on fill soils
At the Strength Limit State

Assumptions:

1. Footings will be embeded 6.5 feet for frost protection. $D_f := 6.5 \cdot ft$

2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)

   Saturated unit weight: $\gamma_s := 125 \cdot pcf$

   Dry unit weight: $\gamma_d := 120 \cdot pcf$

   Internal friction angle: $\phi_{ns} := 32 \cdot deg$

   Undrained shear strength: $c_{ns} := 0 \cdot psf$

3. Use Terzaghi strip equations as L>B

4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 10 \cdot ft$ Based on boring logs

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

Look at several footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$

Terzaghi Shape factors from Table 4-1

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

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

For ϕ=32 deg

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 20.8$

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

$q := D_w \cdot \gamma_d + (D_f - D_w) \cdot (\gamma_s - \gamma_w)$ $q = 0.4904\, tsf$

$$q_{nominal} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s - \gamma_w) B \cdot N_\gamma \cdot s_\gamma$$

$$q_{nominal} = \begin{pmatrix} 13 \\ 14 \\ 15 \\ 15 \\ 16 \end{pmatrix} tsf$$

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

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

$$q_{factored} = \begin{pmatrix} 6 \\ 6 \\ 7 \\ 7 \\ 7 \end{pmatrix} tsf$$

Based on these footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$

$$q_{factored} = \begin{pmatrix} 12 \\ 13 \\ 13 \\ 14 \\ 15 \end{pmatrix} ksf$$

Recommend factored bearing resistance of 7 tsf or 14 ksf for footings 10 feet wide or less on fill soils

Bearing Resistance - BEDROCK:

Nominal and factored Bearing Resistance - PCMG wall on bedrock

Presumptive Bearing Resistance for Service Limit State ONLY

Bedrock at the site is Granite.

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

Type of Bearing Material:	Consistency In Place:	Bearing Resistance Ordinary Range (ksf)	Recommended Value of Use (ksf)
Massive crystalline igneous and metamorphic rock: granite, diorite, gneiss, thoroughly cemented conglomerate (sound condiotion allows minor cracks)	Very hard, sound rock	120 - 200	160

Based on RQD values ranging from 65% to 100%

Recommended Value of Use: $160 \cdot ksf = 80\,tsf$

Therefore: $q_{nom} := 80 \cdot tsf$

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

$q_{factored\_bc} := 80 \cdot tsf$ or $q_{factored\_bc} = 160\,ksf$

At the Service Limit State

Nominal and factored Bearing Resistance - PCMG Wall spread footing on bedrock

Nominal Bearing Resistance for Strength Limit State

Bedrock at the site is GRANITE which is "fair to excellent" in quality.
RQD = 65 to 100%

Reference: AASHTO LRFD Bridge Design Specifications Third Edition Article 10.6.3.2:
For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating in Article 10.4.6.4.

Due to competency of bedrock (RQD 65 to 100%), RMR method is not required.

From Foundation Analysis and Design by JE Bowles Fouth Edition

Section 4-16 pg 233 Bearing Capacity of Rock

Assume: $\phi := 45 \cdot deg$ internal friction angle rock

$c_r := 0 \cdot psi$ cohesion (rock)

Bearing Capacity factors by Stagg and Zienkiewicz 1968

$$N_c := 5 \cdot \left(\tan\left(45 \cdot deg + \frac{\phi}{2}\right)^4 \right) \qquad N_c = 170$$

$$N_q := \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^6 \qquad N_q = 198$$

$$N_\gamma := N_q + 1 \qquad N_\gamma = 199$$

Terzaghi Shape factors from Table 4-1

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

Assume $\gamma_r := 165 \cdot pcf$ for the rock

$D_f := 0 \cdot ft$ footing placed on bedrock surface - no embedment $q := \gamma_r \cdot D_f$ $q = 0\,psf$

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$

Look at several footing widths

$$q_{ult} := c_r \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot \gamma_r \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{ult} = \begin{pmatrix} 82 \\ 131 \\ 164 \\ 197 \\ 246 \end{pmatrix} ksf$$

Reduce ultimate bearing based on RQD = 65%

$$q_{reduced} := q_{ult} \cdot 0.65^2$$

$$q_{reduced} = \begin{pmatrix} 35 \\ 55 \\ 69 \\ 83 \\ 104 \end{pmatrix} ksf$$

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

$$q_{factored} := q_{reduced} \cdot 0.45$$

$$q_{factored} = \begin{pmatrix} 16 \\ 25 \\ 31 \\ 37 \\ 47 \end{pmatrix} ksf$$

Say $q_{factored}$ = 30 ksf = 15 tsf

At the Strength Limit State

Settlement Analysis:

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

Any footing founded at a depth of 6.5 feet for frost protection on loose, fine to coarse sand.

Simplified soil profile:

Finished Grade
Elevation 388 ft

Fill Sand
Assume: 4 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Elevation 384 ft

Native fine to coarse sand
N = 6 bpf (loose)
γ = 125 pcf

Footing
Elevation 381.5 ft

Groundwater
Elevation 372.0 ft

Elevation 364 ft

Clayey silt
N=18 bpf (very stiff)
γ = 115 pcf

Top of Bedrock
Elevation 356 ft

Schmertmann's 1978 procedure for strip footing:

Assume B = 2 ft (minimum allowable footing width)

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

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

$q_{soil\_serviceLS\_factored} := 4 \cdot ksf$

Δq is the change in vertical stress at the footing elevation

The maximum thickness of the fill sand is 4 ft
Assume γ = 125 pcf for the fill sand
Water table is at elevation 372.0 ft
6.5 ft embedment for frost protection

$$\Delta q := q_{soil\_serviceLS\_factored} - (6.5 \cdot ft \cdot 125 \cdot pcf)$$

$\Delta q = 3187.5\,psf$ net load intensity at foundation depth

$$q_{vo} := (6.5 \cdot ft \cdot 125 \cdot pcf)$$

$$q_{vo} = 812.5\,psf$$

$$\sigma_{vp} := (6.5 \cdot ft \cdot 125 \cdot pcf) + (4 \cdot ft \cdot 125 \cdot pcf)$$

$$\sigma_{vp} = 1312.5\,psf$$

$$I_{zp} := 0.5 + 0.1\left(\sqrt{\frac{\Delta q}{\sigma_{vp}}}\right) \qquad I_{zp} = 0.6558$$

Determination of Es:

For loose fine to coarse sand N-value (Nv): $N_v := 6$ From Boring data

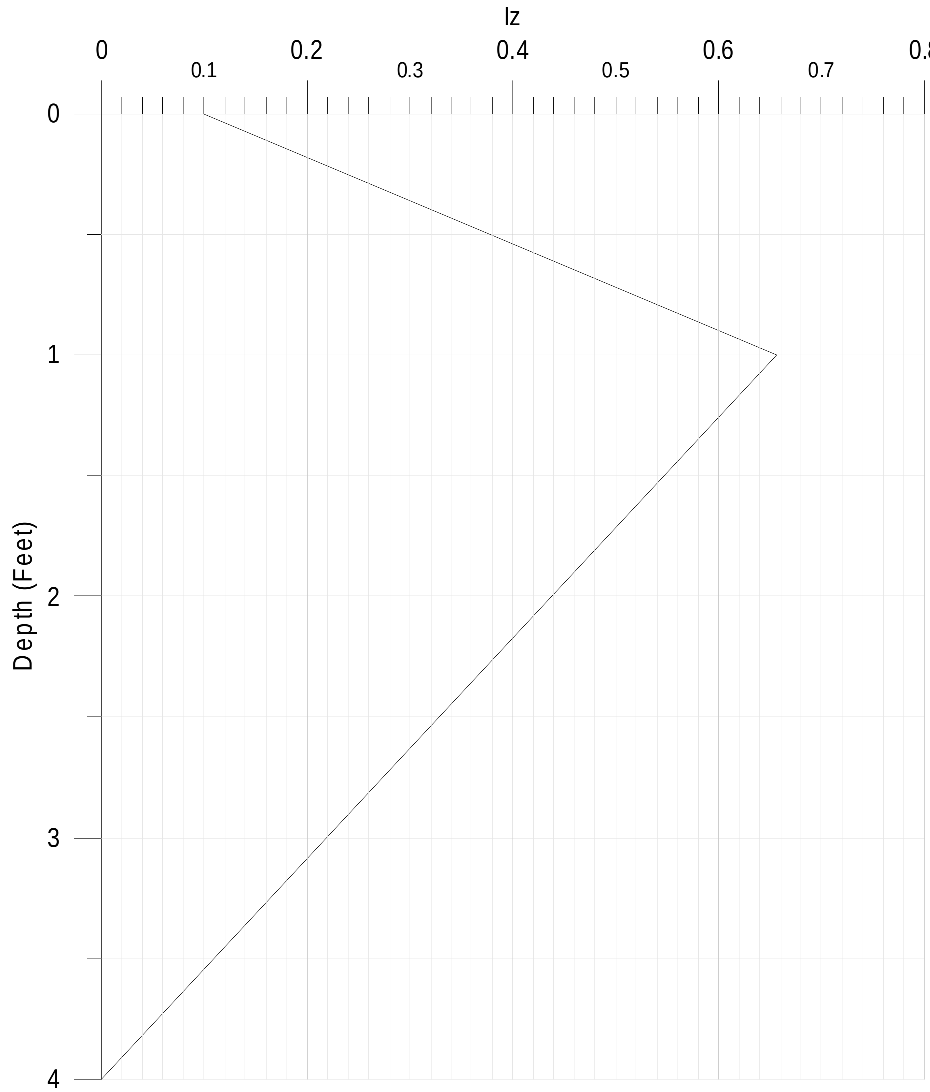
$q_c := N_v \cdot 4$ From table 5.6

$$q_c = 24$$

From Equation 5.11

$$E_s := q_c \cdot 2.5 \qquad E_s = 60$$

For axisymetric conditions - simplified strain influence factor distribution



Layer	z (ft)	delta z (ft)	Iz	qc	Es	Iz(delta z/Es)
1	1	0.5	0.33	24	60	0.0028
2	3	2.5	0.38	24	60	0.0158
					Sum	0.0186

$\sigma_{vo} := q_{vo}$ $\qquad \sigma_{vo} = 812.5\ \mathrm{psf}$

$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_{vo}}{\Delta q} \right)$ $\qquad C_1 = 0.8725$

$C_2 := 1.0$

Se = C1 x C2 x Δq x (Σ (Iz/Es) x Δz)

$Se := 0.935 \cdot 1.0 \cdot \frac{5312.5}{2000} \cdot 0.0186$

$Se = 0.04619$ Feet of settlement

$Se \cdot 12 = 0.5543$ Inches of settlement